SOIL STABILIZATION UTILISING WASTEPAPER SLUDGE ASH

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Collaborating Establishments:

Cementitious Slag Makers Association
Aylesford Newsprint Ltd.
Hanson Bricks Co. Ltd.
British Lime Association

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CERTIFICATE OF RESEARCH

This is to certify that, except where specific reference is made, the work described in this thesis is the result of the candidate. Neither this thesis, nor any part of it, has been presented, or is currently submitted, in candidature for any degree at any other university.

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DEDICATION

To my loving wife Norsalisma
and my adorable children
Ikmal, Imran, Eira and Nia

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ABSTRACT

Increase in environmental awareness over the past decades has resulted in increasing attention to industrial pollution and waste management control. The use of waste is becoming increasingly important in construction. Such materials including Wastepaper Sludge Ash (WSA), can be used to modify certain engineering properties of soils for specific uses to conserve non-renewable natural resources.

In lime-stabilization of sulfate-bearing clay soils, there has been increasing concern over the damage caused by the expansion which is produced when sulfate-bearing soils are encountered. The main objective of this research was to investigate the potential of utilizing WSA, an industrial by-product, as a soil stabilizer with or without blending it with quicklime (CaO), Portland Cement (PC) or with Ground Granulated Blastfurnace Slag (GGBS). The engineering behaviour (plasticity characteristics, compaction, unconfined compressive strength (UCS), linear expansion and California Bearing Ratio (CBR)) of the sulfate-bearing Lower Oxford Clay (LOC) soil and of a non sulfate-bearing soil-industrial Kaolinite (control)-were investigated.

Compacted cylinders of LOC and Kaolinite stabilized with quicklime ((CaO) at typical 2wt.%, 4wt.% and 6wt.%) and with various stabilizers incorporating WSA (WSA-Lime, WSA-PC and WSA-GGBS, at 10wt.%, 15wt.% and 20wt.%) were made under controlled laboratory conditions. The cylinders were made under either mellowed (compacted 3 days after mixing) or unmellowed (compacted immediately after mixing) conditions and then moist cured for 7, 28, 90, 180 and 365 days prior to UCS tests. CBR tests were also carried out, but only on selected stabilized LOC samples, at the lowest and highest stabilizer contents. Linear expansion of stabilized cylinders during moist curing and during subsequent soaking was monitored for at least 100 days.

The results obtained showed that the blended stabilizers incorporating WSA reduced the plasticity index (PI), reduced the maximum dry density (MDD) and increased the optimum moisture content (OMC) of both LOC and Kaolinite. The UCS values of stabilized systems incorporating WSA for both LOC and Kaolinite were higher than those systems stabilized with the traditional CaO. When WSA was blended with lime, PC or GGBS, the results indicated that in the LOC stabilized system, the strength development of unmellowed samples was generally better than for the mellowed samples. This is in contrast with the Kaolinite stabilized system where it is the mellowed samples that recorded higher strength than the unmellowed samples. The CBR values of the unmellowed samples were also higher than those for the mellowed samples. The linear expansion of unmellowed stabilized LOC system was significantly reduced. This is again quite the opposite in the Kaolinite system, where mellowed samples showed reduced expansion relative to the unmellowed ones. Sulfate and thermogravimetric analysis results suggest that the presence of sulfate in a soil plays a major role in the mellowing process. In conclusion, the findings in this research suggest that whether or not to mellow depends primarily on the stabilizer used. Other variables include the target material and site conditions, besides possibly other factors. There are technological, economic as well as environmental advantages of utilizing WSA and similar industrial by-products, in the stabilization of sulfate-bearing and other clay soils, as an alternative to the traditional stabilizers of lime and/or Portland Cement.

LIST OF ABBREVIATIONS AND SYMBOLS

Al_2O_3	Aluminium oxide (alumina)
$Al_2Si_2O_7$	Metakaolin
BaO	Barium oxide
C_2S	Dicalcium silicate
C_3A	Tricalcium aluminate
$C_3A.3C\bar{S}.H_{32}$	Ettringite
C_3S	Tricalcium silicate
C_4AF	Tetracalcium aluminoferrite
Ca	Calcium
$Ca(OH_2)$	Calcium hydroxide
$Ca_2Al_2SiO_7$	Gehlenite
$Ca_2SO_4.2H_2O$	Gypsum
$CaAl_2SiO_8$	Anorthite
$CaCO_3$	Calcium carbonate (calcite)
C-A-H	Calcium aluminate hydrate
CaO	Calcium oxide (quicklime)
C-A-S-H	Calcium alumino-silicate hydrate
C-A-S-$\bar{S}$-H	Calcium-sulpho-aluminate-silicate-hydrate
C-F-H	Calcium ferrite hydrate
CO_2	Carbon dioxide
C-S-H	Calcium silicate hydrate
CuO	Copper oxide
D_fT	Department for Transport
DTG	Derivative Thermo-Gravimetric
DTGA	Derivative Thermogravimetric Analysis
FA	Fly Ash
Fe_2O_3	Iron oxide (haematite)
g	gramme
GGBS	Ground Granulated Blastfurnace Slag
HCl	Hydrocloric Acid
ICL	Initial Consumption of Lime

K_2O	Potassium monoxide
kg	kilogram
Li_2O	Lithium oxide
LL	Liquid Limit
LOC	Lower Oxford Clay
LOI	Loss on Ignition
MDD	Maximum Dry Density
MgO	Magnesium oxide
MnO	Manganese oxide
N	Newton
Na_2O	Sodium monoxide
O	Oxygen
^{o}C	degree Celsius
OH^-	Hydroxyl
OMC	Optimum Moisture Content
P_2O_5	Phosphorus pentoxide
PC	Portland Cement
PFA	Pulverised Fuel Ash
PI	Plasticity Index
PL	Plastic Limit
SiO_2	Silicon oxide (Silica)
SO_3	Sulfate
SrO	Strontium oxide
TEM	Transmission Electron Microscopy
TGA	Thermogravimetric Analysis
TiO	Titanium oxide
UCS	Unconfined Compressive Strength
WSA	Wastepaper Sludge Ash
XRD	X-ray diffraction

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CHAPTER 1 - INTRODUCTION

This chapter gives a general introduction and an overview of problem identification on the global environmental issues faced by the construction industry. It also gives the objectives and outline of the current research and contribution to the body of knowledge in the public domain.

1.1 GENERAL

As environmental issues become of great concern and as valuable resources continue to be depleted further, it becomes increasingly critical that the development of the built environment infrastructure takes proper cognizance of its impact on the environment. The environment and economy of a given region can no longer be treated independently. Too much emphasis on the environment will limit the ability to deliver infrastructure improvements and hence improvements in living standards, particularly in the developing world, while too much emphasis on the economy will lead to the depletion of vital natural resources that cannot be readily replaced.

The balance between environment and conservation which aims to develop human infrastructure is known as sustainable development. The most popular definition of sustainable development is the one given in the World Commission on Environment and Development Report (1987): "*development that meets the needs of the present without compromising the ability of future generations to meet their own needs*". The definition is about the present generations' stewardship of resources. It means that for an economic activity to be sustainable it must neither degrade nor deplete the natural resources, nor have serious impacts on the global environment inherited by future generations. For example, if greenhouse gases continue to accumulate, if the ozone layer becomes further depleted, if soil quality continues to be degraded, if natural resources are severely depleted and water and air are further polluted, by human activity, this will jeopardize man's very existence. The present generation clearly prejudices the ability of future generations to support themselves.

Development is unequivocally associated with construction within the built environment. The construction industry is a major player in arriving at an effective balance between the environmental impact and economic success of construction projects. This is because most new projects involve some form of resource consumption combined with some modification in the environment. As a general rule, this diminishes environmental quality but increases capital wealth. For this reason projects which minimize impact on the environment while still providing the necessary economic and social advantages should be favoured, (Langston and Ding, 2001). From a practical point of view, it is important that resources be utilized at rates approximately equal to the natural rate of regeneration. In addition, consumption should be minimized, recycling or reuse maximized, and renewable resources optimized, subject to advances in technology.

1.2 PROBLEM IDENTIFICATION

In developing countries, enormous road systems need to be constructed in order to improve infrastructure and facilitate their targeted economic development. Good quality sub-grade soils are necessary for durable roads. Such soils are not always available. The Highway Engineer is likely to face weak or unsuitable sub-grade soils as a result of either their low load-bearing capacity, and/or their high swelling properties.

1.2.1 Low Load-Bearing Capacity and Expansive soils

The volume changes of some clayey soils, resulting from changes in their water content, represent one of the most serious problems in the field of foundation engineering. Volume changes may cause unpredictable movement of structures that are founded on such soils, resulting in heaving, shear failure, excessive settlement, cracking and breaking up. This is a particular problem with road pavements which are founded on such soils (Mowafy *et al.*, 1990). Greater thicknesses of base layers are required when using weak soils, compared to those built on suitable strong sub-grade materials. This will result in an increase in the initial and total expenditure of such projects.

The problem of expansive soils was not recognized by soil engineers until about 1940 (Chen, 1975). Prior to this, the damage caused by low load-bearing capacity and expansive soils to structures were attributed to poor construction and settlement of the foundation soils. In 1938, U.S. scientists first realized the role of expansive soil in damage to buildings (Chen, 1975). Since this period much research has been published on the problems of expansive clays and low load-bearing capacity soils in general. In addition, many problems associated with foundation on low load-bearing capacity and expansive soils have been reported from all over the world. These include the heaving, cracking, breaking up of pavements, building foundations, channel and reservoir 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 this form of structural damage in the U.S.A. alone is about $2.3 billion, more than twice the damage from earthquakes, hurricanes and floods (Bruer, 1973). The treatment of these low load-bearing capacity and expansive soils is not always easy or economical. To overcome this problem, various methods can be considered. For example, the in-situ materials can be improved by normal compaction methods and the design based on the modified soil properties. Also, suitable materials from the nearest convenient source may be used to replace the site materials. Further, the properties of the existing materials may be improved by incorporating some other materials. This process is known as "soil stabilization" (Ingles and Metcalf, 1972). The most appropriate treatment method of these soils will be determined principally by economic considerations. It may be cheaper to stabilize a soil using relatively expensive additives rather than excavate and dispose of unsuitable materials and import and place suitable fill. However, using additives is not viable in developing countries where construction costs are very critical.

1.2.2 Disposal of Industrial Waste

The disposal of industrial waste is an environmentally sensitive problem facing waste managers throughout the world. As environmental quality standards become more stringent and the volume of waste generated continues to increase, the traditional disposal methods are no longer acceptable and there is therefore great pressure to change. Ocean dumping is being phased out and landfilling is becoming an unattractive disposal option, due to the increasing levels of landfill tax and the decreasing availability of landfill sites. One possible long-term solution appears to be either recycling or utilising the waste for alternative beneficial purposes. This is already taking place in the paper industry, the target of the current research, where the environmental impact of paper manufacturing has been reduced by increasing the quantities of paper recycled (Frederick *et al.*, 1996).

Annual paper consumption in the UK (some of which is imported) is around 13 million tones, much of which is from recovered paper (Kilby, 2001). Over the past two decades the use of recovered paper by the UK paper industry has gradually increased to an annual amount of about 5 million tones (Péra and Ambroise, 1999). The amount of recovered paper used, as a proportion of the total output from paper mills for the UK is 72%, which is one of the highest in the world. The sludge is in some cases incinerated and the ash normally dumped to landfill, or the sludge is disposed of by land spreading. These processes are expensive and cause concern by their impact on the environment. According to Frederick *et al.*, (1996), an economic alternative to landfill disposal is either recovery of energy from the sludge or re-use of materials from the sludge, or both.

Traditional soil stabilization using lime and/or cement is well established. Although this type of stabilization is very popular and has been successful in the past, there is need to look for other alternative technologies which are more environmentally friendly and economical. Lime is an expensive material and is difficult to work with and similar to the manufacture of cement leads to environmental pollution. Mehta (1983) estimated that for every tonne of Portland Cement produced, one tonne of CO_2 is emitted into the atmosphere which is a major influence on climate change, due to the enhanced greenhouse effect.

1.3 RESEARCH OBJECTIVES AND CONTRIBUTION TO KNOWLEDGE

A chemical and mineralogical study of Wastepaper Sludge Ash (WSA), at the University of Glamorgan (Kinuthia *et al.*, 2001) has established that WSA from one of the largest paper recycling companies in the UK (Aylesford Newsprint Ltd. (ANL)) contains;

i) 30-50% CaO
ii) 20-40% soluble SiO_2
iii) (~ 5% free lime);

thus, this material possesses both hydraulic and latently hydraulic properties and its cementitious nature should be applicable for soil stabilization. Its use in this way would reduce environmental damage and enhance waste minimization through recycling, re-use and recovery of an industrial waste.

Although much work has been carried out on soil stabilization utilizing industrial waste/by-product materials such as Ground Granulated Blastfurnace Slag (GGBS), Pulverised Fuel Ash (PFA), bituminous materials or combinations of these materials, there is a paucity of published work directly referring to the engineering properties imparted by utilising wastepaper sludge ash, either alone or in combination with other materials for the stabilization of soils. The objective of the current investigation is therefore to identify the possible potential of utilising WSA as a soil stabilizer with or without blending it with lime, Portland Cement (PC), or with GGBS, for the enhancement of the engineering properties of soil. The outline aims and objectives of the research may be summarized as:-

1. To determine the effectiveness of WSA with or without blending with lime, PC or GGBS on the modification and/or stabilization of a naturally occurring sulfate-bearing clay soil, Lower Oxford Clay (LOC), and also of a non sulfate-bearing soil (industrial Kaolinite).

2. To compare and contrast the engineering properties of a natural sulfate-bearing clay soil with those of a non sulfate-bearing pure clay, both stabilized with various percentages of various stabilizers utilising WSA, and to establish the underlying reasons for any differences in the engineering properties.

3. To assess the impact of mellowing of stabilized soil prior to compaction.

The problem and objective trees are also schematically outlined in Figures 1.1 and 1.2.

1.4 STRUCTURE OF THESIS

This introduction is followed by the general description of the mineralogy of clays in Chapter 2, with a particular emphasis on its classification and composition. Chapter 3 opens with the discussion of classical soil stabilization with lime. Chapter 3 then deals with other methods of soil stabilization using Portland Cement (PC) and Ground Granulated Blastfurnace Slag (GGBS). In Chapter 4 the materials used in this research are discussed while Chapter 5 contains a discussion of the experimental procedures, equipment design and techniques used in the research. Chapter 6 reports the results obtained, Chapter 7 discusses them while Chapter 8 gives the conclusions drawn from the work. Chapter 8 ends with recommendations for future research, before listing the references quoted throughout the thesis. The thesis ends with the relevant appendices, including a paper published and presented by the author at the 6th International Conference on Pavement Unbound (UNBAR 6), University of Nottingham, 6th – 8th July 2004.

PROBLEM TREE

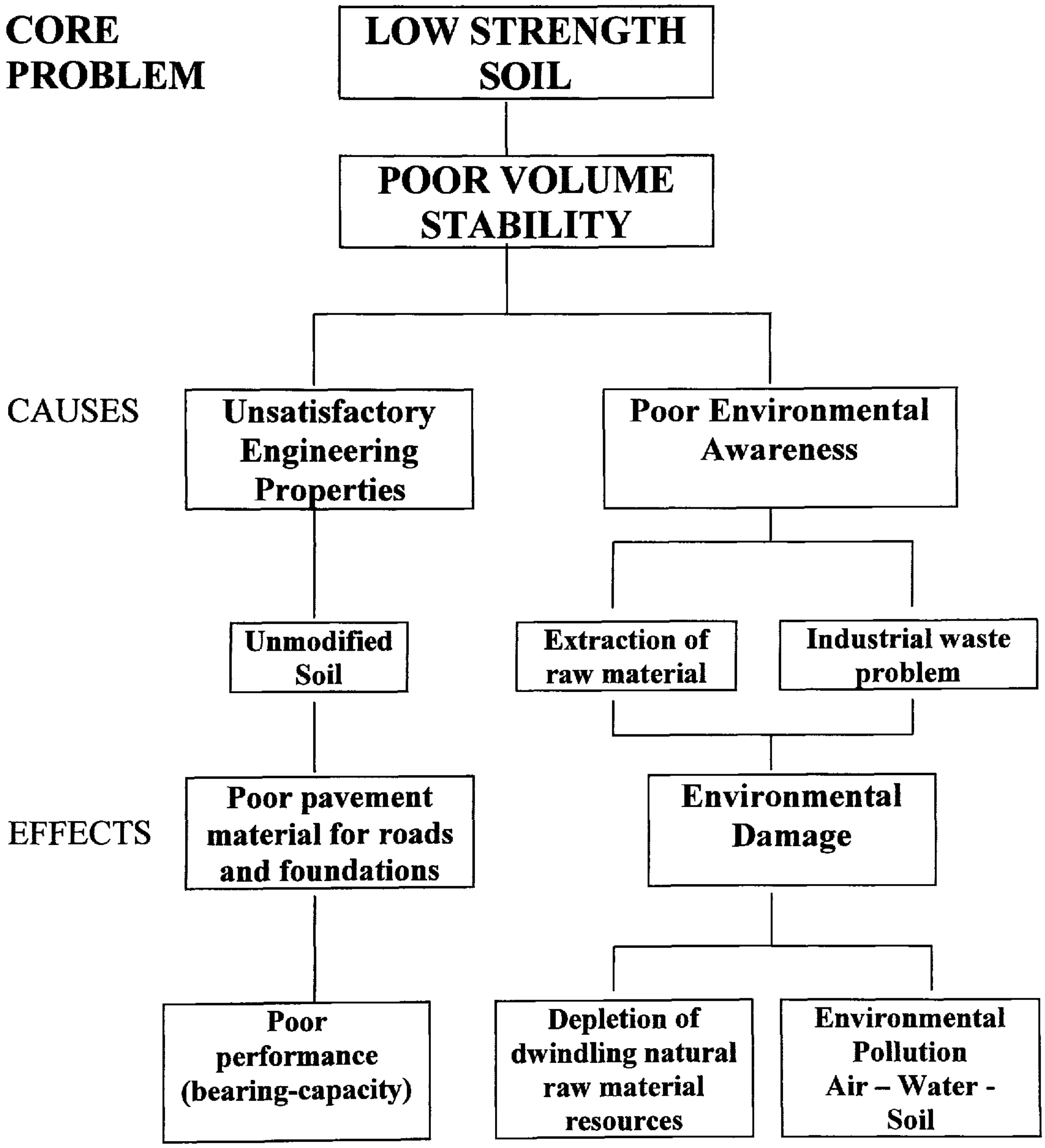


Figure 1.1: Schematic diagram of the Problem definition

OBJECTIVE TREE

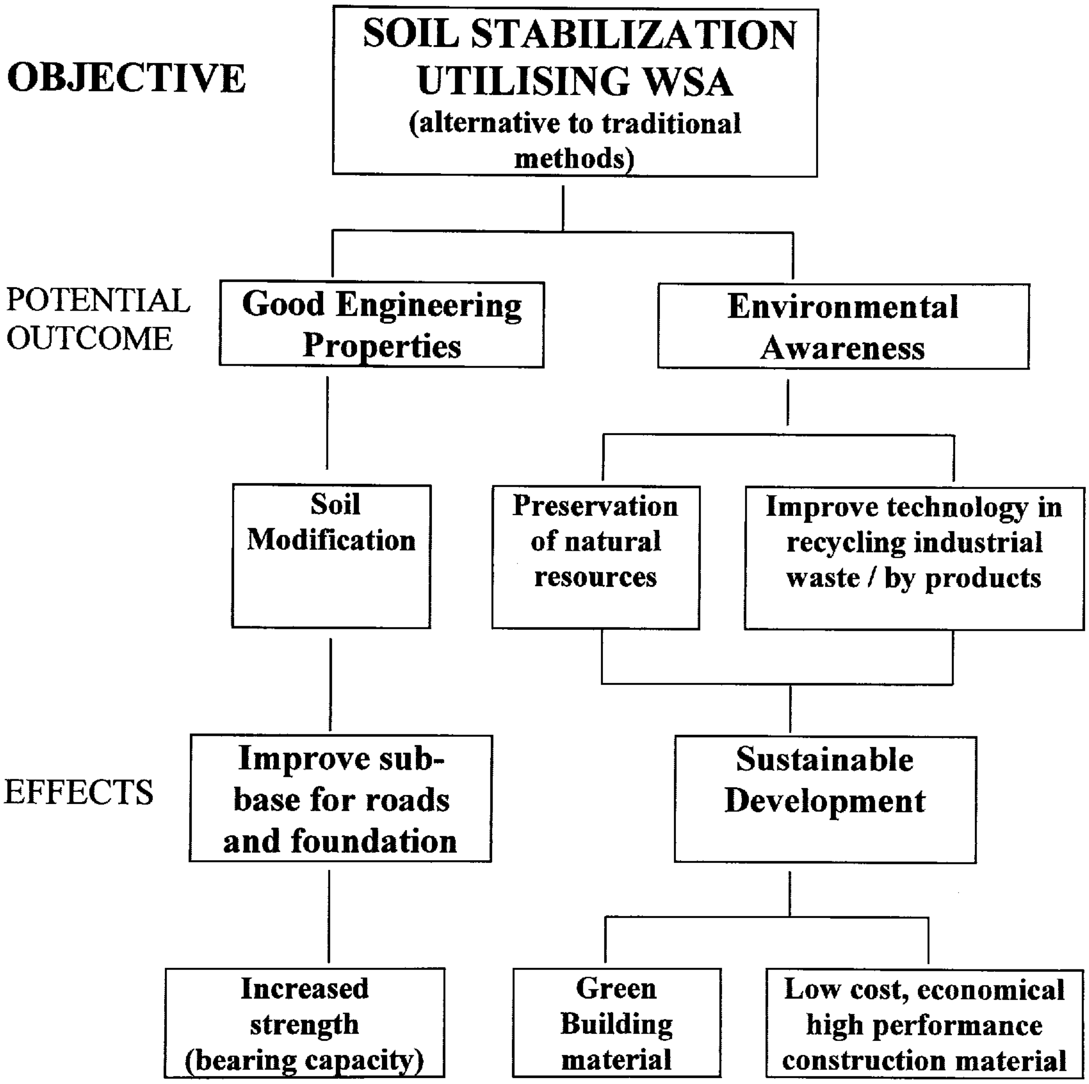


Figure 1.2: Schematic Diagram of Objective tree.

CHAPTER 2 – CLAY MINERALOGY

This chapter gives an overview of clay minerals classification, composition and types namely kaolinite, montmorillonite and illite, structure, characteristics and swelling potential, and suitability for stabilization.

2.1 INTRODUCTION

The term soil conveys varying meanings when it is used in different contexts. To a geologist it describes those layers of loose unconsolidated material extending from the surface to solid rock, which have been formed by weathering and disintegration of rocks themselves. An engineer, on the other hand, thinks of soil in terms of the work he may have to do on it, in it or with it without drilling or blasting. For engineering purposes, soil is best considered as a naturally occurring particulate material of variable composition having properties of compressibility, permeability and strength (Whitlow, 2001). Terzaghi and Peck (1967) defined soil as "natural aggregate of mineral grains that can be separated by such gentle means as agitation in water."

2.1.1 Mineral composition of soil

The large majority of soil consist of a mixture of inorganic material particles, together with some water and air, as illustrated in a soil model which has three phases: solid, liquid and gas (see Figure 2.1).

- *Organic matter.* Organic matter originates from plant or animal remains, the end product of which is known as humus, a complex mixture of organic compounds, which is a feature of topsoil or the upper layer. From an engineering point of view, organic matter has undesirable properties. It is highly compressible and will absorb large quantities of water, so that changes in load or moisture content will produce considerable changes in volume, posing serious settlement problems. Organic material also has very low shear strength and thus low load-bearing capacity; furthermore, its presence may affect the setting of cement and therefore provide difficulties in concreting and in the soil stabilization processes.

- *Water.* Water is a fundamental part of natural soil and has a greater effect on engineering properties than any other constituent. The movement of water through a soil mass needs to be studied with care in problems of seepage and permeability. Water can dissolve and carry in solution a wide range of salts and other compounds, some of which have undesirable effects.

- *Air.* Soil may be considered in a practical sense to be perfectly dry or fully saturated, or to be in a condition somewhere between these two extremes. In a dry soil, there will be water vapour present, while fully saturated soil may contain as much as 2 per cent air voids. Air is compressible and water vapour can freeze, both of which are significant in an engineering context.

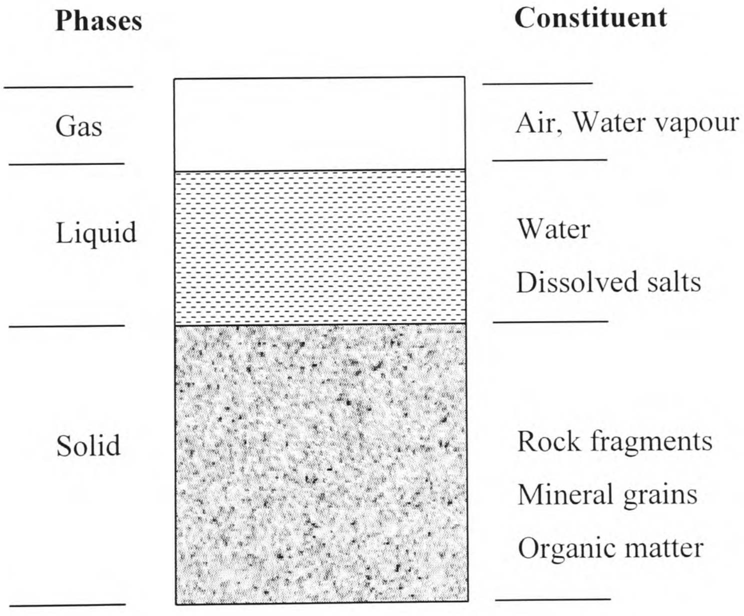


Figure 2.1: Three-phase soil model

2.2 CLAY MINERALS

Because of the importance of clay materials in ceramics, in the construction industry, in agriculture, in geology, and elsewhere, their investigation goes back far into ancient times. Many people have devoted much of their lives to the study of clay materials. Investigators learned that clays and soils had widely varying properties. Even soils and clays which had the same colour and general appearance and the same texture were found to differ widely in other characteristics. It was found that clays of the same ultimate chemical composition frequently had very different physical attributes, and that clays with substantially the same physical properties might have very different chemical composition (Grim, 1968).

The term ‘clay’ can have several meanings:

1. *Clay soil* – the soil behaves as a ‘clay’ because of its cohesiveness and plasticity even though the clay mineral content may be small.
2. *Clay size* – most classification systems describe particles less than 2 μm as ‘clay’ which is a reasonably convenient size. However some clay mineral particles may be greater than 2 μm and some soil particles less than 2 μm such as rock flour which may not contain many clay mineral particles at all.
3. *Clay minerals* – these are small crystalline substances with a distinctive sheet-like structure producing plate-shaped particles (Barnes, 2000)

In general terms, clay implies a natural, earthy, fine-grained material which develops plasticity when mixed with a limited amount of water. There are several factors affecting the properties of a clay soil or clay material including: (a) clay mineral composition, (b) non-clay mineral composition, (c) organic material, (d) exchangeable ions and soluble salts (Grim, 1968).

(a) **Clay mineral composition**. This refers to the identity and relative abundance of all clay mineral components. Certain clay minerals can have a tremendous influence on clay soil, even if present in only small amounts (e.g. the presence of smectite or montmorillonite (±5%) is likely to provide a material very different from another clay with the same composition in all ways except for the absence of smectite or montmorillonite). The crystallinity and variability in clay minerals affect such properties as plasticity (Grim, 1968).

(b) **Non-clay Minerals**. Minerals such as calcite, dolomite, mica, quartz, pyrite, feldspar, gibbsite and other minerals are abundant in clay soil, and the effect of these minerals on the properties of a clay soil can influence the potential uses of the clay soil. For example, a material with high pyrites content may, under certain conditions, prove problematic in the construction industry, as pyrites has been known to oxidize, and under the right conditions produce gypsum, the precipitation of which may cause significant heave (Thomas *et al.*, 1981; Cripps and Edwards, 1992).

(c) **Organic material**. 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, or as organic molecules adsorbed on the surface of the clay mineral particles or adsorbed between the silicate layers. Organic materials usually act as a pigment in clay soil giving the material a dark-grey or black colour. A very small amount of organic material present can have a major effect on the behaviour of soils (Kinuthia, 1997).

(d) **Exchangeable ions and soluble salts**. At the time of deposition, or as a result of weathering, water-soluble salts may be entrained in the clay. Common salts found in clay minerals are chlorides, sulphates, and carbonates of alkalis, alkaline earths, aluminium and iron. Clay minerals and some organic material that can be found in clay soils have significant ion-exchange capacities and can affect stabilization.

Construction failures have occurred because the strength properties of a soil that developed during construction could not be predicted adequately from empirical laboratory testing data. Without fundamental data on how and why clay minerals are held together, it is impossible always to predict safely from any empirical data how a clay material will act when load is applied, when the water table is altered, or when other conditions are changed (Grim, 1968).

2.3 CLAY MINERAL STRUCTURES

With the aid of X-ray diffraction, electron microscopy and differential thermal analysis, the study of clay minerals has been enhanced significantly. Hence, it is now known that clay minerals are essentially hydrous aluminium silicates containing in some cases iron and magnesium, and also with some containing forms of alkali metals as essential components. The clay minerals are generally derived from weathering of pre-existing materials.

The majority of clay minerals are insoluble in acids. They appear to have appreciable affinity for water, are plastic when wet, water retentive and coherent when dry. Most clay minerals are crystalline, with sheet-like or layered structures of two varieties: silica sheets and alumina sheets.

Clay mineral particles are quite small, less than 2 μm (1 micron, μm = 1 x 10^{-6}m), and electrochemically very active. For example, minute clay particles carry similar electrical charges, which induce mutual repulsion. Neutralization of these charges, say through electrolytes, can bring about coagulation and subsequent precipitation of the floccules of clay. Furthermore as the size of the particles decreases below 2μm, the electrical charges on the particles increases with the decrease in size. Hence, it would be useful to the civil engineer to be able to manipulate the cation-exchange feature towards a desired goal when confronted with a flocculation situation.

Clay minerals have complex mineral structures but can be visualized and classified by considering the basic 'building blocks'. The silica sheet is composed of tetrahedra, each tetrahedron being bounded by four triangular plane surfaces, with four equally-spaced oxygen atoms at the vertices and a silicon atom within the interior, equally spaced from the oxygen atoms. The basic unit arrangement is shown in Figure 2.2

The tetrahedra are combined into hexagonal units, in a repetitious manner, to form the lattice of the mineral. The alumina sheet has two-row units. One aluminium or magnesium or iron atom is at the center of an octahedron, with oxygen atoms or hydroxyl (OH) units at the vertices of alternate rows, respectively.

The oxygen and hydroxyl ions dominate the mineral structure because of their numbers and their size (they are about 2.3 times larger than an aluminium ion and about 3.4 times larger than a silicate ion). Even if their negative charges are satisfied, the O^{2-} and OH^- ions existing on the surface of the sheets will impart a slightly negative character.

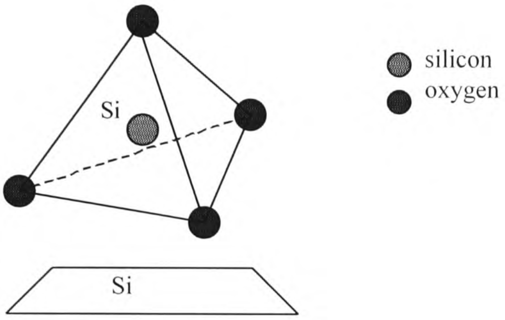
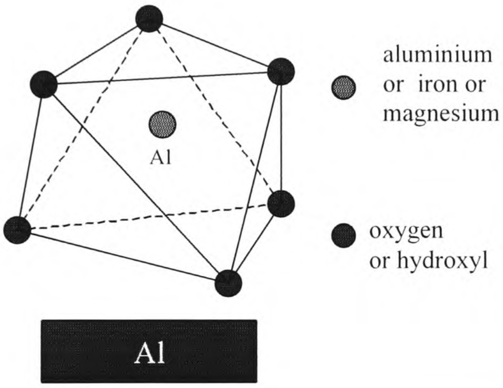
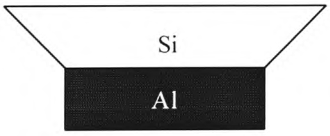
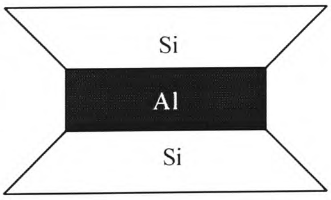
Tetrahedron Tetrahedral layer		4 oxygen ions enclosing one silicon ion (silicate)
Octahedron Octahedral layer		6 oxygen ions or hydroxyl ions enclosing one aluminium, magnesium, iron or other ion
Two- layer unit (1:1 mineral)		1 tetrahedral and 1 octahedral unit
Three- layer unit (2:1 mineral)		2 tetrahedral and 1 octahedral layer

Figure 2.2: Clay Minerals

2.4 CLAY MINERAL TYPES

The clay minerals are usually divided into three main groups, with the lattice structures of the minerals serving as the basis for their classification. The groups are the kaolinites, the montmorillonites and the illites. Most clays formed by sedimentation are a mixture of kaolinite and illite with a variable amount of montmorillonite whereas clays formed by chemical weathering of rocks may also contain chlorites and halloysites.

2.4.1 Kaolinite

The kaolinite minerals are formed of units consisting of a single tetrahedral silica and a single octahedral alumina sheet. These units may repeat themselves indefinitely to form the lattice of the mineral. Variation between members of the kaolinite subgroup consists of the way layers are stacked above each other and possibly in the position of aluminium ions within the available sizes in the octahedral sheet. Figure 2.3 gives a symbolic arrangement of the kaolinite minerals. Their general chemical composition is expressed by the formula;

$$(OH)_8\ Al_4Si_4O_{10}$$

Kaolinite is the most abundant constituent of residual clay deposits, derived mostly as a by-product of the weathering of rock or certain clay minerals, and is commonly intermixed with illites in sedimentary clay. Kaolinites are very stable, possess a tight cohesive structure that resists the penetration of water into the lattice and generally are not subject to expansion when saturated. Penetration of water molecules and ions between the layers is difficult because of the strong hydrogen bonding. Also the coefficient of internal friction is somewhat higher than that of most other clay minerals.

Halloysites are minerals that belong to the kaolinite family. They possess a round or flattened tube-like shape. Some other members of the kaolinite group are nacrite and dickite. The halloysites are distinguished by one additional water molecule to the basic kaolinite unit. This is given by the formula;

$$(OH)_8\,Al_4Si_4O_{10}.4H_2O$$

When wet, halloysite masses have a tendency to creep or flow horizontally. Thus, they may be viewed as potentially unstable, and less than desirable as materials for embankments. Both kaolinites and halloysites are common materials in the pottery industry (Cernica, 1995).

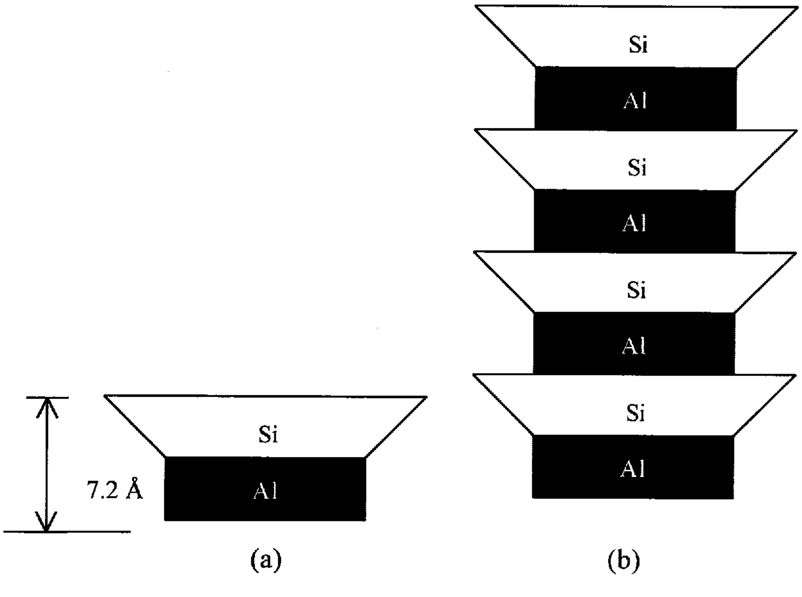


Figure 2.3: The kaolinite mineral. (a) Basic kaolinite unit. (b) Lattice of kaolinite mineral.

2.4.2 Montmorillonite

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. The chemical composition of this group is expressed by the formula;

$$(OH)_4Al_4Si_8O_{20}.\ nH_2O$$

The montmorillonites are made up of sheet-like units comprising an alumina octahedral sheet between two silica tetrahedral sheets, as shown in Figure 2.4. The bonding of these sheets is rather weak, resulting in a rather unstable mineral, especially when wet. In fact, montmorillonites display a significant affinity for water, with subsequent swelling and expansion. Conversely, upon drying a saturated montmorillonite, the result is appreciable shrinkage and cracking. In practical terms, such characteristics may be of significance to the engineer. For example, the expansion of clay may mean lifting of slabs, excessive lateral thrusts on retaining structures, and the endangering the stability of slopes (Cernica, 1995). Soils containing a large proportion 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).

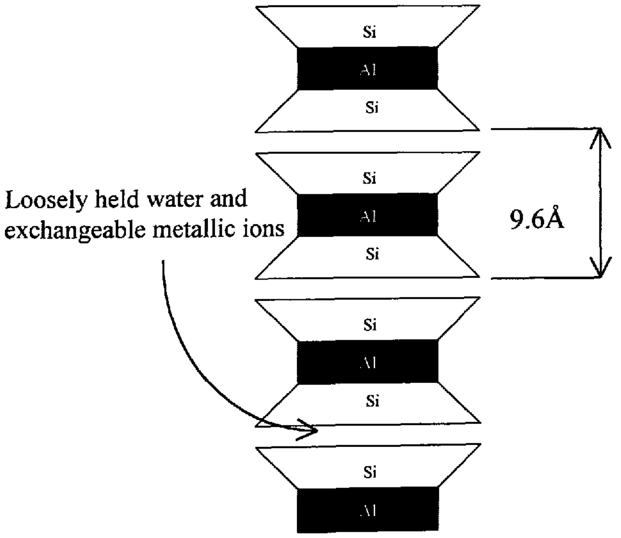


Figure 2.4: The montmorillonite minerals

2.4.3 Illites

The illites are somewhat similar to the montmorillonites in their structural units, but are different in their chemical composition. The chemical composition of illites is expressed by the formula;

$$(OH)_4K_y(Al_4Fe_4Mg_4)\ (Si_{8-y}Al_y)O_{20}$$

where y varies from 1 to 1.5. The symbolic structure of illites is shown in Figure 2.5 The basic structure of the illite unit consists of a gibbsite octahedral sheet between two silica tetrahedral sheets. Unlike montmorillonite particles, which are extremely small and have a great affinity for water, the illite particles will normally aggregate and due to the strongly bonding interlayer potassium atoms thereby develop less affinity for water than montmorillonites. Correspondingly, their expansion properties are less. The angle of internal friction is higher than that of montmorillonite. Illites usually occur as very small, flaky particles mixed with other clay and non-clay materials. Nature of clay mineral particles are described in Table 2.1.

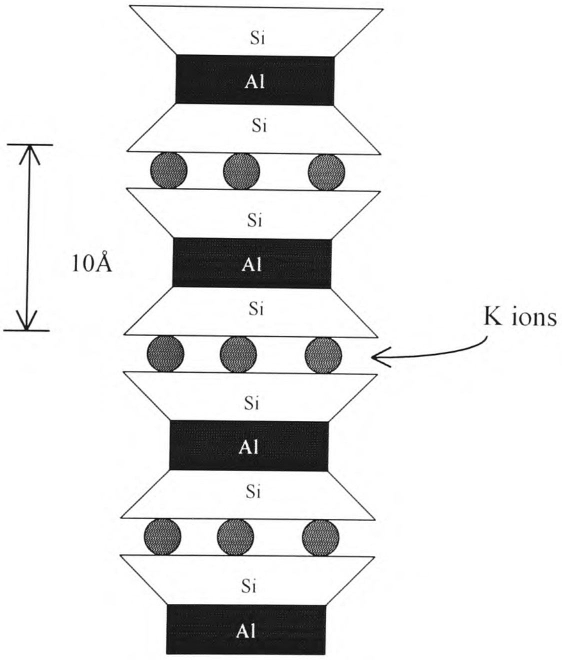


Figure 2.5: The illite clay mineral

Table 2.1: Nature of clay mineral particles (Barnes, 2000)

Mineral	**Diameter thickness ratio**	**Surface Area m^2/gram**	**Nature**
Kaolinite	10-20	10-70	Hydrogen bond prevents hydration and produces stacks of many layers (up to 100 per particle). Particle size up to 3 μm diameter, low shrinkage/swelling
Halloysite	-	40	Two water layers between stacks when fully hydrated ($4H_2O$) distort structure to a tubular shape. Low unit weight. Water (in crystal) irreversibly driven off at 60-75°C affecting moisture content, classification and compaction test results
Illite	20-50	80-100	Common mineral but varies in chemical composition. Particles flaky, small, diameter similar to montmorillonite but thicker. Moderate susceptibility to shrinkage/swelling
Montmorillonite (smectite)	200-400	800	High surface area due to small (<1 μm) and thin (<0.01μm) particles produced by water molecules and exchangeable ions entering between layered units and separating them. A good lubricant. Water readily attracted to mineral causing very high susceptibility to expansion, swelling and shrinkage.

2.5 CHARACTERISTICS OF CLAY MINERALS

2.5.1 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 minerals are used. Clay minerals have the property of sorbing certain anions and cations and retaining them in an exchangeable state; i.e. these ions are exchangeable for other anions or cations by treatment with such ions in a water solution (the exchange reactions may also take place in a non-aqueous environment). The exchangeable ions are held around the outside of the silica-alumina clay-mineral structural unit, and the exchange reaction generally does not affect the structure of the silica-alumina packet.

Grim (1962, 1968) stated that clay surfaces are usually negatively charged and this is the cause for cation attraction to the particle surface. There are three main causes of the negative charge:

1) ***Broken bonds*** around the edges of the silica-aluminate units leaving unbalanced charges which are balanced by adsorbed cations. The number of broken bonds per unit mass and hence the ion 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 particularly Mg^{2+}, for trivalent aluminium Al^{3+} in the octahedral sheet both result in unbalanced charges in the structural units of some clay minerals. This charge imbalance may be balanced either by substitution in other lattices (i.e. OH^-) for O^{2-} or adsorption of positive cations.
3) ***The hydrogen of exposed hydroxyls*** may be replaced by a cation which is exchangeable. Some hydroxyl groups are exposed around the broken edges of

all clay minerals, and cation exchange due to broken bonds can, in part at least, occur by replacement of the hydrogens of exposed hydroxyls.

Exchangeable cations are positively charged ions which are attracted 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).

Cations can be arranged in a series on the basis of their replacing power. Early studies stated that under a given set of condition, various cations were not equally replaceable and did not have the same replacing power. The general order of replacement of the adsorbed cations is : $Li^+ < Na^+ < H^+ < K^+ < NH^+ << Mg^{2+} < Ca^{2+} << Al^{3+}$. At equal cencentration, 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, because of increasing dissociation of weekly bonded $Si\text{-}OH^-$ groups on exposed clay crystal edges. Below pH 5 the cation exchange capacity is constant (Grim, 1962).

Mukherjee *et al.*, (1943) (from Grim, 1968) found that the exchange capacity might vary with the nature of the cation. It was considerably larger with divalent cations, such as Ca^{2+}, than with monovalent cations (e.g. Na^+). Other factors such as particle size, lattice distortion, clogging of exchange positions etc. may also affect the cation-exchange capacity (Grim, 1968).

Table 2.2 gives the cation exchange capacity of common clay minerals. 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 kaolinite, illite or chlorite.

Table 2.2: Cation exchange capacity of clay minerals in milliequivalents per 100 g at pH 7 (Grim, 1968)

Clay Minerals	**Exchange Capacity (meq/100g)**
Kaolinite	3-15
Illite	10-40
Chlorite	10-40
Montmorillonite	80-150

2.5.2 Water Adsorption and Swelling Properties of Clay Minerals

Clay soils may suffer from volume changes due to moisture content changes which result in swelling and shrinkage (Bell, 1983). The ability of clay to imbibe water leads to swelling and when it becomes dry, it shrinks. Clay particles in soils are almost always hydrated, i.e. surrounded by layers of water molecules adsorbed onto clay particles. This water layer affects all soil properties including plasticity, compaction, strength and water movement in soil (Yong and Warkentin, 1975; Gillot, 1987; Abdi, 1992).

Barshad (1955) stated that water adsorption in clays starts with the hydration of the external clay particle surface by a unimolecular layer of water. This layer builds up with time to multimolecular water layers at a rate dependent on the type of clay. These multilayers then find a way into the interior surfaces of the clay particles themselves through diffusion of water vapour through the clay particle and/or seepage at the clay particles edges. The interlayer cations are still in position at this stage attached as they are to the oxygen sheets. As more molecular water layers find their way into the interlayer spaces, and with subsequent expansion of the interlayer distances, the cations get detached creating an external surface. If the water is in vapour form (as during curing of stabilized soil), the expansion is minimal. If in liquid form, further expansion occurs. The next chapter now looks at several types of soil stabilization methods, and property changes in stabilized-soil.

CHAPTER 3 - SOIL STABILIZATION

This chapter describes the classic soil stabilization method with lime and examines the effects of lime addition to a clay soil. It also describes in detail the reactions and mechanisms that occur when modification and stabilization processes are carried out in the presence of sulfates and/or sulfides. This chapter also deals with other methods of soil stabilization where a soil modifier and/or activator is added to a clay soil in order to achieve long-term strength and volume stability.

3.1 INTRODUCTION

Soil stabilization may be defined as the alteration of the properties of an existing soil to meet the specified engineering requirements. The main properties that may require to be altered by stabilization are:

a) ***Strength*** – to increase the strength and thus stability and bearing capacity;
b) ***Volume stability*** – to control the swell-shrink characteristics caused by moisture changes;
c) ***Durability*** – to increase the resistance to erosion, by weathering or traffic usage, and
d) ***Permeability*** – to reduce permeability and hence the passage of water through the stabilized soil (Sherwood, 1993).

Soil stabilization is widely used in road construction to improve sub-bases and sub-grades, for railroad and airport construction, for embankments, as soil exchange in unstable slopes, as backfill for bridge abutments and retaining walls, as canal linings, for improvement of soil beneath foundation slabs and for lime piles.

3.2 LIME STABILIZATION

Lime stabilization is not an innovation. Various forms of lime stabilization have been used for thousands of years. Until the discovery of Portland Cement in the 19th Century, lime was widely used for building construction. McDowell (1966) mentioned that stabilized earth roads were used in ancient Mesopotamia and Egypt. Lime was extensively used during the Second World War for roads and pavement construction (Dumbleton, 1962). Today, stabilization of clay soil by the incorporation of lime is a technique widely used throughout the world to improve its use in construction.

The use of lime in soil stabilization 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 a short experimental length of dirt road in the American state of Missouri was treated with hydrated lime to reduce rutting during rain and snow (McDowell, 1966). With the expansion of roads to cater for growth of motor traffic in the 1930s, the use of stabilization of soils began to increase. In 1943, the U.S. Corps of Engineers used hydrated lime to reduce the plasticity of soil used in the construction of a Texas airfield. The sections treated with lime showed 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 stabilize soil, as well as the physical and chemical reactions which may occur. 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. Galloway and Buchanan (1951) 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 towards 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 stabilization was first used in the construction of the A38 in Worchester in 1951 (Brook-Bradley, 1952). In this project, part of the sub-base was stabilized with hydrated lime.

Lime stabilization of soils has been used when:-

- Materials are unacceptably wet or plastic.
- Workability and compaction properties are needed.
- Greater soil strength and stability is required.
- Off site disposal needs to be avoided.
- Materials up to sub-base quality are required from in-situ soils.
- There is a need to encapsulate difficult materials.
- Sites are easily affected by adverse weather conditions.
- It is a necessary to reduce swelling and shrinkage.

3.2.1 Clay-lime Reactions

When lime is added to a clay soil, it has an immediate effect on the properties of the soil as cation exchange begins to take place between the metallic ions associated with the surfaces of the clay particles and the calcium ions of the lime. Clay particles are surrounded by a diffuse hydrous double layer which is modified by the ion exchange of calcium. This alters the density of the electrical charge around the clay particles which leads to them being attracted closer to each other to form flocs, the process being termed flocculation. This process is responsible for the modification of the engineering properties of clay soils when they are treated with lime (Sherwood, 1993).

Lime used in soil stabilization may be in many forms such as quicklime (CaO), hydrated lime ($Ca(OH)_2$) and dolomitic lime. Quicklime is the direct product of the calcination of limestone and it is the more effective stabilizer than hydrated lime as it has a high ability to absorb water. Lime stabilization can be defined as the reaction between silica and alumina within the clay structure and lime and water to form calcium silicate hydrate and calcium aluminate hydrate gels (Rogers *et al.*, 1997). The reaction will be stronger in the case of high silicate content in the soil.

Several types of chemical reactions take place simultaneously when lime is added to a clay soil in the presence of water. They can be divided into two groups as:-

a) **Cation exchange**, flocculation and agglomeration happen during the soil modification process and occur rapidly after addition of lime to clay. This occurs typically within 24 hours, although sometimes it takes several hours depending on the clay minerals involved.

b) **Pozzolanic reactions** happen during the longer-term soil stabilization process and occur slowly, producing long-term strength gain by the progressive crystallization of gels that are created once lime has reacted with the clay minerals in the soil (Rogers *et al.*, 1997).

3.2.2 Lime Modification Process

Mixing quicklime with a wet soil immediately causes the lime to hydrate and an exothermic reaction occurs. The heat produced is sufficient to drive off some of the moisture within the soil as vapour, which reduces the moisture content of the soil. The second effect of ion substitution results in a reduction in plasticity as the clay particles flocculate. Modification with lime results in considerable changes in the engineering characteristics of a treated soil, and has been used as a mitigating measure on a number of waterlogged sites (Sherwood, 1992).

When quicklime is added to a clay soil the following dehydration reaction occurs :

$$CaO + H_2O \longrightarrow Ca(OH)_2 + Heat$$

Since the reaction is strongly exothermic (65kJ/mol), additional water is driven off through steam. The clay strength and workability are improved merely by the drying action of lime. In addition, calcium ions in the lime may exchange with metal ions associated with the clay lattice, by the process of cation exchange. This causes the following:

(a) substantial reduction in the thickness of the adsorbed water layer (electrical double layer), causing a reduced susceptibility of the clay to subsequent water addition,

(b) flocculation of the clay particles, caused by increased mutual attraction due to their closer proximity following electrical double layer thinning,

(c) increase in the internal angle of friction between the agglomerates and greater aggregate shear strength,

(d) a textural change from a plastic clay to a friable material that is granular in nature, and

(e) reduced plasticity.

3.2.2.1 Cation Exchange

In lime stabilization, 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.

The addition of extra Ca^{2+} cations, for example the addition of gypsum (Kinuthia, 1997) will lead to an increase in the overall number of cations attracted to the clay particle surfaces. The cation exchange will depend on the position of the added cation in the lyotropic series $Li^+ < Na^+ < K^+ < Mg^{2+} < Ca^{2+} < Ba^{2+} < Al^{3+} < H^+$ (Cobbe, 1988; Arabi & Wild, 1989) relative to the cations already present in the clay-lime system. The latter will predominantly be Ca^{2+} from the lime and any other inter-layer cations such as Na^+ and/or K^+. The ensuing cation exchange will determine whether flocculation-agglomeration or deflocculation-dispersion predominates. The divalent cations which result in enhanced neutralization of the repulsive negative charges on the clay particle surfaces and a reduced electrical double layer will lead to greater particle-particle attraction and an enlarged electrical double layer and hence an inclination towards deflocculation-dispersion.

3.2.2.2 Flocculation and Agglomeration

Flocculation is the phenomenon whereby clay particles clump together into larger sized aggregates. The forces acting between two particles close to each other are influenced by two sets of forces: (a) Inter-particle attraction due to Van der Walls or secondary bonding forces and (b) Repulsive forces due to the electrically negative nature of the particle surface and its adsorbed layer (Whitlow, 2001).

It has been suggested that cation exchange and the resulting modification of the electrical double layer alter the density of electrical charge around the clay particles causing them to become electrically attracted to each other (Hilt and Davidson, 1960; Eades and Grim, 1960). Flocculation produces an apparent change in texture as a result of larger-sized aggregates and the soil becomes more friable.

Researchers now believe that with the addition of a small amount of lime, the calcium present causes an initial ionic exchange, which results in flocculation (or edge to face reorientation of the clay plate-like particles). This flocculation has a dramatic effect on the soil, in terms of improved workability and shear strength. However, these strength gains may only be temporary, and can be easily reversed if the soil is exposed to moisture (Evans, 1997).

3.2.3 Stabilization Process

Clay minerals are natural pozzolans and have the ability to react with lime added to soil and to produce cementitious products. The hydroxyl ions released from the lime create a pH level that is sufficiently high that silica and alumina are dissolved from the clay minerals, from which new compounds are formed as a result of pozzolanic reactions. The silica and alumina within the clay structure react with the water and lime to form calcium-silicate-hydrates, calcium-aluminate-hydrates, and calcium-alumino-silicate-hydrates (C-S-H, C-A-H & C-A-S-H) gels, which subsequently crystallize to bind the structure together. As a result of these reactions the material becomes stronger and more brittle (Croft, 1964; Slone, 1965; Diamond and Kinter, 1966; Arabi and Wild, 1989; Rodgers *et al.*, 1997). Strength gain increases with the length of the curing periods, and is attributed to the progressive dissolution of SiO_2 and Al_2O_3 as the reaction continues (Croft, 1964).

The reactions between lime, water, silica and alumina in clay to form cementitious products are referred to as the soil-lime pozzolanic reactions. The reactions give rise to the formation of various types of hydrated cementing agents and are time dependent, with strength developing gradually over a long period of time (Bell and Coulthard, 1990). The actual reaction products formed vary depending on the type of clay, temperature, lime content and curing period, though it is generally recognized that the principal cementitious product is a C-S-H gel (Bell *et al.*, 1987; Cobbe, 1988; Arabi and Wild, 1989). Bell (1988) hypothesized that the long-term strength development of lime-clay material may be attributed to the gradual crystallization of C-S-H gel forming an interlocking structure, although this is disputed by Wild *et al.*, (1989) who argues that an impermeable pore filling gel is formed rather than a crystalline material. Small amounts of crystalline calcium aluminate hydrate phases (such as C_4AH_{13} and C_3AH_6) and calcium aluminate silicate hydrate phases (for example C_2ASH_8) form in some circumstances, particularly in lime-stabilized kaolinite which has a high alumina content (Arabi and Wild, 1989). Cation (base) exchange and chemical combination (silicate bonding or pozzolanic reaction) commence at the same time, the former being an immediate effect while the latter takes a considerable time to complete (Croft, 1964; Bell, 1988).

Pozzolanic reactions and reaction mechanisms of red tropical soil and lime were studied by Cabrera and Nwakanma (1979). They found that lime is consumed at a fast rate in the initial stages of reaction, up to 5 to 7 days, and the rate of lime consumption then decreases after that. The total lime consumption between 7 and 28 days represents about 8% of the lime consumed during the initial 7 days. They suggested that the increase in strength of the soil-lime system beyond the first 7 days cannot be explained in terms of the predicted pozzolanic reaction products as only a small amount of lime is consumed between 7 and 28 days. It is suggested that the increase in strength, with an increase in the curing period, shown by the soil 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 material depends on many factors, curing conditions and periods, clay type and content and moisture content at the time of mixing.

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

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

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

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

$Ca^{2+} + OH^- + SiO_2 + Al_2O_3$ (Clay Silica & Alumina) $\rightarrow$ C-A-S-H.

Many researchers have identified the products which may be formed in the soil-lime reaction products, 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 cementitious materials depend substantially upon the original clay mineral and upon the reaction conditions. Eades and Grim (1960) studied, using XRD and DTA, the reaction products of pure kaolinite clay and hydrated lime mixture cured at 140°F (60°C). They noticed that as the lime content and the curing period 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. Hilt *et al.*, (1960) and Glen *et al.*, (1963) studied the long term reaction products formed in a montmorillonite-lime mixture. They identified the formation of crystalline C-A-H and C-S-H at normal room temperature.

3.2.4 Property Changes in Lime-Stabilized Clay

3.2.4.1 Lime Stabilization and Consistency (Atterberg) Limits

The addition of lime affects many of the engineering properties of soils. Changes in material engineering properties include the liquid limit (LL), plastic limit (PL), plasticity index (PI), optimum moisture content (OMC), maximum dry density

(MDD), unconfined compressive strength (UCS), California bearing ratio (CBR) and linear expansion /swelling.

The liquid limit of clay soil is generally lowered by the addition of lime. Usually there is a general increase in the plastic limit on the addition of lime (Diamond and Kinter, 1964; Sabry and Parcher, 1979; Brandle; 1981; Akoto and Singh, 1981; Kinuthia, 1997; Thomas, 2001). The amount of lime needed to cause changes in the plastic limit varies from 1% – 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 effects 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 increase in plastic limit for illite was less than for montmorillonite, while kaolinite showed the smallest increase.

The effect of adding lime on the liquid limit of soil is not clear, and a general trend is not apparent. Some researchers have reported that the liquid limit increases (Dawson, 1956; Clare and Crutchley, 1957; Croft, 1964), while others reported that both increase and decrease can occur depending on the soil under test (Lund and Rumsey, 1959; Diamond and Kinter, 1964). 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; Abdi, 1992).

Dumbleton (1962) reported an increase in liquid limit and plastic limit at low lime content and a gradual decrease in liquid limit and plastic limit with further increase in lime content. He concluded that plasticity is affected by clay type, lime addition and time. Sherwood *et al.*, (1997) studied London Clay and found that the clay became easier to compact and workability was improved after adding lime. They concluded that the liquid limit was altered with low lime contents, whereas the plastic limit required greater lime addition to attain maximum change. They also studied the effect of curing period and found that different clays needed 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 and the plastic limit requires greater lime addition to attain a significant change. Clare and Crutchley (1957) found that the addition of 1% lime raised the liquid limit of 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 liquid limit values was observed after curing at higher lime content.

The effect of curing time on the plasticity properties has been studied by many researchers. Most of them observed further decrease in the plasticity index and further increases in plastic limit with curing time (Dumbleton, 1962; Akoto and Singh, 1981). 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 a long curing period as the new cementitious materials formed due to the pozzolanic reaction of lime with the clay minerals may produce misleading results as the soil is effectively a different material after curing.

The changes in consistency limits are attributed to flocculation and agglomeration as a result of the cation exchange upon lime addition. According to Croft (1964), the increase in liquid limit depends on the OH^- ion concentration in the pore liquid which modifies the affinity of clay particles to water. The addition of lime results in the introduction of more OH^- ions. Water molecules, which are polarized, are therefore attracted and bound to the clay layer surface by the formation of hydrogen bonding (Arabi and Wild, 1989).

In conclusion, most of the researchers reported an increase in liquid and plastic limit at low lime content and a gradual decrease in liquid and plastic limit with further increase in lime content. The overall result of these changes is that the plasticity index decreases with an increase in lime content.

3.2.4.2 Lime Stabilization and Compaction Characteristics

The compaction characteristics of lime stabilized soils have been extensively studied. The addition of lime to all clays in general increases the optimum moisture content and reduces the maximum dry density (Johnson, 1948; Lund and Ramsey, 1959 ; Andrews, 1966; Abdi and Wild, 1993; Kinuthia, 1997; Thomas, 2001). 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 replacement of clay with lime contributes to the reduction in maximum dry density because clay has a higher density than lime (Andrews and O'Flaherty, 1968; Abdi, 1992). The decrease in density is not only dependent on lime percentage, but also on the amount and type of clay minerals present. Thus the optimum moisture content increases with increasing clay fraction as the specific surface increases and requires more water for lubrication.

In lime stabilized kaolinite soils, higher densities are obtained than for the 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 cementitious products could cause the reduction in dry density (Herzog and Mitchell, 1963; Diamond and Kinter, 1964). Mitchell and Hooper (1961) confirmed the effect of the formation of cementitious 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 decrease in maximum dry density. These changes in the compaction characteristics due to time delay may be attributed to the flocculation of particles (Herin and Mitchell, 1961), carbonation and pozzolanic reactions. The cementitious particles before compaction may provide greater resistance to particle rearrangement and compaction, and this will cause resistance to densification and thus lower the density values.

3.2.4.3 Lime Stabilization and Strength Characteristics

Clay generally shows a significant increase in strength when stabilized with lime. The strength development of lime-stabilized soil is primarily dependent on pozzolans present and has been extensively studied by several researchers. It has been found that the strength gain is influenced by several factors such as clay type, amount and type of lime added, curing period, moisture content and time elapsed between mixing and compaction (Ingles and Metcalf, 1972; Bell and Coulthard, 1990). These factors are further described below;

i). Type of Clay

The type of clay mineral present in a soil has been found to have an important influence on the strength properties as each type of clay has different mineralogy which affects the reaction products. Expansive clays develop strength more rapidly in response to lime addition. Those having three layers (montmorillonite) are more effective than two layer clay minerals (kaolinite). The reaction of lime with montmorillonite is quicker than kaolinite clays although the final strength achieved is greater in kaolinite clays (Bell and Tyrer, 1987; Bell, 1988). Many researchers confirmed that montmorillonite and kaolinite react better with lime than illite and chlorite clay soils (Hilt and Davidson, 1960; Thompson and Harty, 1973; Bell, 1996). In many cases, only a small amount of clay is needed in a soil for reaction with lime to be effective, as the amount of silica or alumina required to sustain pozzolanic reaction in soils is relatively small (Bell and Coulthard, 1990).

ii). Type of Lime

Quicklime is the most frequently used lime product for lime stabilization in Europe whereas hydrated lime is used more often in the United States. Generally quicklime seems to be a more effective stabilizer than hydrated lime (Bell, 1988). Ingles and Metcalf (1972) found that montmorillonite clays produce lower strength when mixed with dolomitic limes rather than with high calcium limes. Kaolinite clays on the other hand have been observed to achieve the highest strengths when mixed with semihydraulic lime and the lowest strengths are obtained with high calcium limes (Bell and Coulthard, 1990).

iii). Amount of Lime

Strength does not increase linearly with lime content. Generally, lime addition to clayey soil increases its strength to a certain limit and excessive addition of lime reduces strength (Bell, 1988; Abdi, 1990). The optimum lime content tends to range between 4% to 8% with higher dosage values required in soil with higher clay fractions.

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 pozzolanic reactions. For economic reasons it is necessary to use the minimum amount of lime which achieves the required strength. The optimum lime content for the particular soil is the amount required to achieve a pH value of 12.4 (Allan *et al.*, 1977).

iv). Curing Period and Method

Curing conditions are of great importance in influencing strength increase and the final strength of soil mixtures. The strength increases rapidly at first, notably during the first seven days of curing (Rogers, 1988), then increases more slowly at a more or less constant rate for about 15 weeks. This supports the view that the primary cementitious products due to lime-clay reaction begin to form at an early stage even though reaction continues for a very long period. These products start forming during and after the flocculation process. Brandl (1981), in his work on lime stabilization of various types of clay (as reported by Bell, 1988 and by Bell and Coulthard, 1990), observed that the rate of increase in strength was very small between one to two years and that no further changes in strength took place, even in active clays after seven years.

Higher curing temperatures accelerate the reaction and result in higher early strength gain (Bell, 1988; Bell and Coulthard, 1990). This has been confirmed by many researchers including Marks and Halliburton (1972); Al-Rawi, (1981); Wild *et al.*, (1987); and Thomas (2001). Mateos (1964) reports that specimens cured at 35°C developed twice the strength of those cured at 25°C. Thompson (1970) reported that no pozzolanic reaction can take place below 4°C and strength development is therefore retarded.

v). Moisture Content

Lime-soil mixtures compacted at moisture contents slightly above optimum, and after brief periods of curing, attain higher strengths than those compacted with moisture content less than optimum (Bell, 1987). This is probably because the lime is more uniformly dispersed in a more homogenous environment when excess water is present. Also, sufficient water is available for cation exchange to take place (Cobbe, 1988). However, the strength of lime-stabilized clay soil decreases with increasing natural moisture content (Bell, 1988).

vi). Time elapsed between mixing and compaction

Mitchell and Hooper (1961), as reported by Bell and Coulthard (1990), reported that samples of lime-clay soil compacted within one hour of mixing attained higher strength than those which were compacted after 24 hours had elapsed. In contrast Arabi and Wild (1989) found that the effect of delaying compaction of a Devonion Red Marl treated with 6wt.% lime and cured at 25°C for 12 weeks in a moist environment led to no strength change.

Holt and Freer-Hewish (1998); and Thomas (2001) studied the effect of mellowing period on the compaction characteristics of respectively London and Oxford Clays mixed with lime. They observed a further increase in the optimum moisture content and further decrease in maximum dry density with increasing mellowing duration.

Thomas (2001) in his studies on Lower Oxford Clay-Lime mixtures, observed that specimens that were mellowed for 3 days before compaction, showed higher strengths after a one week curing period, compared with the unmellowed samples. However, after 4 weeks of curing, it was generally the unmellowed specimens that exhibited higher strength values.

3.2.5 Swelling in Lime-Stabilized Soils

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 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.

A reduction of swelling potential is obtained after lime-stabilization of soil. These modified characteristics are attributed to substitution of other cations by calcium. Two main factors involved in the reduction of swelling include the decreased affinity for water of the calcium saturated clay, and the formation of cementitious products 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 a California expansive clay. They found that swelling was reduced significantly after curing with lime. The addition of 4% hydrated lime to expansive soil reduced swelling from 17% to 3%. They also observed that the addition of lime to such clay soil increased the shrinkage limit.

Thomas (2001) in his studies on stabilization of the sulfide-rich LOC soil reported that increases in lime content generally resulted in increased linear expansion. He also observed that mellowing of LOC-lime samples (3 days at 20°C) prior to compaction substantially reduced expansion.

3.2.6 Effect of Sulfate on Lime Stabilization

3.2.6.1 Introduction

Sulfates are found in natural ground in solid form as crystals or nodules within the soil matrix or within open soil discontinuities such as joints, faults and fissures. They are also found as dissolved ions in groundwater occupying the pore spaces within the soils and filling joints and fissures below the groundwater table. Sulfate most commonly occurs as calcium sulfate in the form of selenite or gypsum ($CaSO_4.2H_2O$), but also as magnesium sulfate in the form of epsomite ($MgSO_4.7H_2O$), as potassium sulfate in the form of arcanite (K_2SO_4) and sodium sulfate in the form of thernadite ($Na_2SO_4.10H_2O$) or Glubers salt (Kinuthia, 1997; Thaumasite Expert Group, 1999).

Sulfates show a non-uniform distribution in many clay soils in the UK. Bassey and Lea (1953) suggested that sulfate concentration may vary widely due to the wide seasonal variations in rainfall. Calcium sulfate is commonly found in the UK, especially in Jurassic soil such as Lower Oxford Clay.

3.2.6.2 Sulfate attack in practice

Sulfate attack is a phenomenon which can result in the deterioration of concrete in various ways including cracking, spalling, loss of strength and expansion, and can finally lead to the destruction of the cement structure. In soil, sulfate attack is mostly accompanied by strength loss and large volume changes, resulting in substantial heave in stabilized earthworks (Veith, 2000).

Many researchers have reported examples of the detrimental effects of sulfates, either naturally present in the ground or artificially added when soils are modified or stabilized with lime and/or cement. Sherwood (1962) found that samples of cement stabilized clay exhibited strength loss of more than 50% compared with the control specimens when immersed in magnesium sulfate solution (0.2% SO_3).

Mitchell (1986) observed the substantial heave of lime-stabilized sub-base in Las Vegas, Nevada, USA. The affected areas were associated with lower density and high moisture content and contained a high percentage of sodium and calcium sulfates. Expansive reactions resulted in substantial heave of the sulfate-bearing clay soil forming the sub-base of the road in Las Vegas after 4.5% of quicklime had been added to condition the material (Hunter, 1988). Snedker and Temporal (1990) outlined the disastrous results in lime stabilization during the construction of the M40 motorway near Banbury, U.K. where the presence of 0.4% sulfates resulted in 60% heave of the stabilized capping layer. Parker (2004) reported that sulfate attack of the lime-stabilized capping layer of the new carriageways on the 7.5km A10 Wadesmill bypass U.K. resulted in heave that left up to 25% of the carriageways buckled, cracked and ridged.

3.2.7 Lime-Clay-Sulfate Reactions

At pH values of at least 12.4 (provided by a saturated lime solution), the clay fraction of the soil will react with sulfates (Snedker and Temporal, 1990). The more Ca^{2+} ions introduced to a clay soil, the more sulfate ions are adsorbed, especially in soil rich in iron and alumina (Bolan *et al.*,1993). The effects of sulfates on lime-treated soils differ from their effects on untreated clays. In lime-treated soil, the unconfined compressive strength (UCS) increases with low sulfate content (around 1%), especially when the water content of soil exceeds the optimum moisture content (OMC) or compaction moisture content (Bell, 1988). In contrast, the UCS of untreated clay (without stabilizer) is reduced by the presence of sulfates regardless of moisture content (Stevens and Littleton, 1989). Kinuthia *et al.*, (1999) found that added sulfates lowered the liquid limit of lime-stabilized kaolinite, and the magnitude of lowering depends on the nature of the sulfate cations. They also pointed out that the changes in Atterberg limits and in compaction characteristics result mainly from cation exchange processes that affect the viscosity of the clay-water mix. There is evidence that the early formation of ettringite and gypsum may also influence these material properties.

The presence of sulfates not only influences the engineering properties of lime-stabilized soil, but it can also influence both the reaction mechanisms and the reaction products. The pH increase due to lime addition allows the release of alumina and silica from the clay minerals. The calcium-sulpho-aluminate phase-ettringite ($C_3A.3C\overline{S}.H_{32}$) - is formed first at low sulfate concentrations. The metastable phase monosulfate ($C_3A.C\overline{S}.H_{12}$) may also be observed (Abdi and Wild, 1993; Waswa *et al.*, 1993; Kinuthia *et al.*, 1999). At low temperatures where carbonate is present, thaumasite ($Ca_2[Si(OH)_6]_2.(SO_4)_2(CO_3)_2 24H_2O$) can occur. It is well documented that ettringite and thaumasite can, under certain conditions cause severe damage to concrete structures and stabilized soil pavements (Hunter, 1988; Snedker and Temporal, 1990; Bickley *et al.*, 1995; Thaumasite Expert Group, 1999).

3.2.7.1 Ettringite Formation

The reactions of lime and clay provide further complication in the presence of sulfate, producing expansion principally as a result of ettringite formation (Abdi and Wild 1993). The term ettringite normally refers to "sulfate ettringite", which has a formula $C_6A\overline{S}_3H_{32}$. Ettringite can be presented either in cement chemistry notation as $C_3A_3C\overline{S}H_{32}$ or $C_6A\overline{S}_3H_{32}$ or in the structural chemistry notation as $Ca[Al(OH)_6]_2(SO_4)_3 26H_2O$). However, it should be noted that ettringite is in fact a general term used to denote a group of minerals, all with very similar crystal structures (Day, 1992). In the current work the term ettringite refers to both amorphous and crystalline ettringite.

After the dissolution of alumina and silica from the clay at high pH and of the sulfate minerals, Wild *et al.*, (1993) hypothesized that a colloidal product forms on the clay particle surfaces consisting of a complex calcium-aluminate-silicate-sulfate-hydrate (C-A-S-$\overline{S}$-H). The $\overline{S}$/S ratio of the colloidal product increases progressively with curing/soaking time due to its property of attracting sulfate ions and rejecting the silicate ones, due to the differing solubilities. Eventually within this colloidal product a compound with little or no silica in it is precipitated as ettringite.

Assuming that the chemical environment during the cement hydration process is similar to that in the lime-stabilized clay hydration process, a pore solution of high alkalinity would lead to the formation of fine ettringite crystals which form close to the clay particle surfaces where the necessary ion species are concentrated. If excess water is available, this ettringite formation would be accompanied by large expansion. However, at lower pH, $Al(OH)_4^-$ ions would migrate into the bulk pore solution and ettringite may precipitate in voids giving rise to little or no expansion (Glasser, 1996). Ettringite is not stable in low lime environments when the pH falls below approximately 10.5 (Gaze and Crammond, 2000; Santhanam *et al.*, 2001). In Portland Cement, sulfate, usually in the form of gypsum, is intentionally added during manufacture to the cement clinker to form ettringite and inhibit "flash setting" of the C_3A component of cement during hydration (Day, 1992; Neville, 1995). In cement related cases, ettringite withdraws water from the pore solution, increasing the concentration of OH^- ions and therefore increasing the pH of the pore solution. This helps in maintaining the pH for further ettringite formation until all the reactants are depleted.

3.2.7.2 Nature of Ettringite

The influence of ettringite on the expansive behaviour of stabilized clay soil is very complex because the nature and form of ettringite differs under differing chemical environments. The characteristics and stability of ettringite are strongly dependent on pH, sulfate activity and temperature.

- ***Influence of pH.*** According to Mehta (1983), above a pH ≈10, calcium-sulpho-aluminate hydrates precipitate from solution in ordinary Portland cement systems. Below this level of pH, only gypsum and aluminium sulfate are stable phases. Thus calcium sulpho-aluminate (ettringite - $C_3A.3C\bar{S}H_{32}$) appears at a pH ≈10.5 to 10.7 and on further increase in pH, monosulfate ($C_3AC\bar{S}H_{12}$) appears at pH ≈ 11.6. Ettringite that formed in the presence of lime (high pH) consists of small, needle-like crystals, while that formed in the absence of lime (lower pH) consists of much larger lath-like crystals, though the latter are almost colloidal in texture and are not formed of prismatic crystals (Mehta, 1983). Santhanam *et al.*, (2001) stated that ettringite is not stable at a pH as high as 11.5 and could decompose to form gypsum.

- ***Influence of Sulfate Ion Concentration.*** Almost all ettringite is eventually transformed to monosulfate at low sulfate concentrations (Neville, 1995). In the hydration of PC, when sulfate is low in the system, little or no ettringite is present after 24 hours (Mehta, 1983). The conversion of ettringite to monosulfate takes place when sulfate concentration falls below 2.35 mg/l. Also the conversion must be under CO_2 free conditions (< 0.5%) (Kuzel, 1996).

- ***Influence of Temperature.*** Mitchell (1986) reported that at temperatures below 15°C ettringite can (under the right circumstances) undergo carbonation to form thaumasite. Recent work using photo-micrographs concludes that in hardened cement pastes, thaumasite formation is the result of attack and transformation of C-S-H gel under moist conditions and low temperatures (Crammond and Halliwell, 1995). Ettringite remains stable until 50°C when its solubility increases and at 100°C it becomes unstable and disappears (Glasser, 1996).

3.2.7.3 Ettringite and Swelling

Expansion mechanisms resulting from ettringite formation have been explained by two theories : *Crystal Growth* theory and *Swelling* theory. The expansion in lime-stabilized clay in the presence of sulfates is believed to be partly caused by the growth of ettringite crystals formed on the clay particle surfaces (Mitchell, 1986). Other researchers have suggested that there is the osmotic pressure that induces water intake arising from concentration differences of dissolved ions between the solid particles and the surrounding liquid phase, separated by a colloidal gel from which ettringite crystallizes (Krahn and Fredlund, 1972).

Mehta (1983) found that ettringite formed in the presence of lime was needle-like, with a large surface area and unsatisfied negative charge. From the observation of swelling in a PC system, he concluded that the colloidal ettringite would imbibe large numbers of water molecules and generate swelling pressure leading to overall expansion of the system. Similarly, Wild *et al.*, (1993), testing kaolinite with various lime and gypsum contents, agreed that osmotic swelling would take place within the colloidal layer in regions of high sulfate concentration in close proximity to the developing ettringite rods at the clay particle surfaces.

3.3 OTHER METHODS OF SOIL STABILIZATION

3.3.1 Portland Cement (PC)

3.3.1.1 Introduction

Portland Cement is defined in BS 12: 1978 as "a product consisting mostly of calcium silicate, obtained by heating to partial fusion a pre-determined and homogeneous mixture of materials containing principally lime (CaO) and silica (SiO_2) with a small proportion of alumina (Al_2O_3) and iron oxide (Fe_2O_3)". Calcareous materials, typically chalk or limestone, provide the CaO and argillaceous materials, such as clay or shale, provide the SiO_2, Al_2O_3 and Fe_2O_3 (Sherwood, 1992). Cement has been used effectively in the strengthening of granular materials such as soils.

There are four compounds that are usually regarded as the major constituents of cement;

Tricalcium silicate (alite)	$3CaO.SiO_2$ (C_3S)
Dicalcium silicate (belite)	$2CaO.SiO_2$ (C_2S)
Tricalcium aluminate	$3CaO.Al_2O_3$ (C_3A)
Tetracalcium aluminoferrite	$4CaO.Al_2O_3.Fe_2O_3$ (C_4AF)

3.3.1.2 Hydration of Portland Cement

The hydration of standard Portland cement is a chemical reaction between the phases present in the cement powder and water. During production, about 4%-7% gypsum ($C\bar{S}H_2$) is added. As water is introduced to the cement, it reacts with the aforementioned phases. Hydration then commences resulting in the formation of a number of hydration products. These hydration products contribute to the properties of the hardened cementitious material. As the alite and belite phases come into contact with water, a calcium silicate gel (C-S-H), the primary binding component of the hardened PC paste is formed, as illustrated in the equation below;

$$2C_3S + 6H_2O \quad \rightarrow \quad C_3S_2H_3 + 3CH$$

$$2C_2S + 4H_2O \quad \rightarrow \quad C_3S_2H_3 + CH$$

Alite reacts at a faster rate with water and produces over three time as much portlandite (calcium hydroxide or hydrated lime – CH) as belite. Thus, portlandite or lime is a by-product of the Portland cement hydration process. It is partially soluble and has little or no cementitious value. As C_3A comes into contact with water, immediate stiffening of the cement paste, known as flash set occurs. To overcome this the added gypsum reacts with C_3A in the presence of water to form the calcium sulphoaluminate phase ettringite ($C_6A\bar{S}_3H_{32}$). This is then followed by further reaction between C_3A and the hydration products already formed to produce a monosulphoaluminate phase ($C_4A\bar{S}H_{12}$) and calcium aluminate hydrate (C-A-H) phases. These reactions are shown in the equation below;

$$C_3A + 3C\bar{S}H_2 + 26H_2O \quad \rightarrow \quad C_6A\bar{S}_3H_{32}$$

$$2C_3A + C_6A\bar{S}_3H_{32} + 4H_2O \quad \rightarrow \quad 3C_4A\bar{S}H_{12}$$

$$C_3A + CH + 12H_2O \quad \rightarrow \quad C_4AH_{13}$$

The reaction between C_3A and water is very rapid and the calcium sulphoaluminates are the first hydration products to form. It is these products that contribute to the early strength of cement. The C_4AF phase follows a similar hydration process to that of C_3A

producing the iron equivalent of ettringite ($C_6FS_3H_{32}$), monosulphoferrite (C_4FSH_{12}) and calcium ferrite hydrate (C-F-H)

3.3.1.3 Soil Stabilization with Portland Cement

Portland Cement is a primary stabilizing agent, which can be used alone to bring about a stabilizing action (Sherwood, 1993). The C-S-H gel and C-A-H phases that form during the hydration process, produce a strong, hard matrix in which a granular material, like clay soil is embedded. As the hydration reaction proceeds from the surface of the cement grains, the reaction is slow and the centre of the cement grains will normally remain unhydrated. There is a further reaction that takes place in most cement-stabilized soils. Lime that is liberated from Portland cement hydration, combines with clay minerals in the soils. Over time this reaction may form C-S-H, C-A-H and C-A-S-H strength promoting phases in a similar way to lime-stabilization. Herzog and Mitchell (1963) demonstrated this theory by testing cement-stabilized kaolinitic and montmorillonite clays. By using X-ray diffraction techniques, they found that the clay-cement specimens contained only a fraction of the free lime that is present if normal hydration of the cement took place without interaction between the cement and clay. It was concluded that the missing calcium hydroxide had participated in pozzolanic reactions with clay minerals. Further work by Herzog (1963) described cement stabilization as the development of a strong, semi-continuous skeleton consisting of a hardened cement core and clay particles bonded by primary and secondary cementitious material. He divided the cement-clay reaction into two parts:

- *The primary reaction*; where Portland cement hydrates and immediately produces cementitious gel subsequently producing a modification of the clay structure to a skeletal matrix-type system.
- *The secondary reaction*; where the lime originating from the cement reacts with clay minerals to form strength giving phases over time.

Cement can therefore in principle, be used to stabilize any type of soil. However, lime is usually preferred as a stabilizer for heavy clay soil because cement is difficult to mix with such soils. A bond forms between the hardened cement matrix and the particles of the stabilized material depending on the chemical composition of the material. Factors that affect the stabilization of clay soils by Portland cement are:-

(a) ***Soil mineralogy***. Croft (1967) in his studies on the mechanism of soil-cement stabilization and the influence of the common minerals on this process found that the weakly active clay minerals, such as kaolinite and illite had little effect on hydration and hardening of cement. He postulated that the increase in unconfined compressive strength with time can be attributed to the desiccation of gelatinous hydration products, the crystallization of new materials and to the reactions between lime and the clay minerals to produce secondary cementitious products. Montmorillonite is little affected by small additions of PC. The pH of the montmorillonite-cement aqueous phase drops from 12.2 to around 10.0 in less than one week in mixes containing up to 10% cement. In the absence of a strongly alkaline environment, hardening of montmorillonite-cement mixtures is inhibited and the strength development retarded.

(b) ***Sulfide and Sulfate Content***. The presence of sulfates may have a detrimental effect on cement stabilized clay soils. In the presence of water, sulfates may cause problems by reacting with hydrated cement to form products that occupy a greater volume than the combined volume of reactants. Calcium hydroxide, a product of cement hydration, may react with sulfates and alumina (released from the clay) to form ettringite. It is also possible that hydrated calcium aluminates (C-A-H phases) may react with calcium sulfate, again to form ettringite (Sherwood, 1992). The presence of sulfides in the cement-bound material may also prove problematic as it is known that sulfide oxidation can lead to increased sulfate levels, which in turn increases the potential for the formation of deleterious phases such as ettringite and/or thaumasite (Thomas *et al.*, 1981; Sherwood, 1992 and 1993; Thaumasite Expert Group, 1999; Hobbs and Taylor, 2000).

(c) ***Organic Material.*** The effect of organic matter is known to be closely associated with its ability to combine with calcium ions liberated by the hydrating cement. Certain organic compounds may retard or even inhibit the hardening of stabilized soils making them impossible to stabilize with cement (Sherwood, 1993). Similarly as with lime stabilization, it is the type of organic compound rather than the total amount that is important. The organic compounds are able to interfere with the hydration of cement by virtue of their ability to react with calcium ions at the high pH values that exist in cement-stabilized soils.

3.3.1.4 Soil Stabilization with Lime and Portland Cement

The main application of lime in relation to soil stabilization is the modification of the engineering properties of the potentially target material, and for use with clay soils that are difficult to stabilize with cement alone. Therefore it is effective to use a two-stage lime/cement stabilization process whereby lime is added primarily to modify the soil properties followed by the addition of cement to achieve a long-term increase in strength. Even where cement alone is effective, the use of both lime and PC may prove advantageous. Heavy clay soil of high pozzolanic activity, was shown to attain higher strengths when stabilized with lime and cement than it did with equivalent amounts of lime or cement used alone (Sherwood, 1993). Economic reasons dictate however, that this two-fold (modification and cementation stabilization) process is generally restricted to high plasticity soils that are difficult to mix with cement only.

Two-stage stabilization is becoming increasingly common in the U.K. as it is more economical to improve in-situ material rather than removal and replacement with costly imported materials. For example, in 1999 Tarmac Special Projects Ltd. stabilized a soft, sandy clay with 2% cement and 2.5% lime at the site of the new Acute and General Hospital, Dartford, UK (DoETR, Digest 058, 1999). Laboratory testing established that stabilization had resulted in improved California Bearing Ratio (CBR) values in excess of 30%. However, the stabilization of boulder clay along a length of the A421 Tingewick bypass, UK, using lime (1.5% CaO) and cement (8.5% PC), resulted in the heave of the stabilized material due to the oxidation of sulfides present in the clay to sulfates, causing sulfate attack. A further section of the same road stabilized using lime and ground granulated blastfurnace slag (GGBS) showed no signs of significant heave (Higgins and Kennedy, 1999).

3.3.2 Ground Granulated Blastfurnace Slag (GGBS)

3.3.2.1 Introduction

Blastfurnace slag is produced from iron blast furnaces as a by-product of the iron making industry. 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 blastfurnace slag, consists essentially of silicates and alumino-silicates of lime and other bases (Lee, 1974) with potential cementitious reactivity. Molten blastfurnace slag has a temperature of 1,300°C – 1,600°C and is chilled very rapidly to prevent crystallization. The granulated material produced is known as granulated blastfurnace slag. It is a latently hydraulic product that can be activated with lime, alkalis or Portland cement to give hydraulic properties (Gupta and Seehra, 1989). The latent hydraulic properties of blastfurnace slag were discovered in Germany in 1862 (Bijen, 1996). The latent hydraulicity means that once activated, the slag reacts with water to give cementitious products. In the United Kingdom, this potential of blastfurnace slag was first realized in Scotland in 1914 when granulated blastfurnace slag was ground with Portland cement. In slag manufacture, the granules or pellets are ground into a fine powder similar in fineness to PC (specific surface 320-380 m^2/kg) and with specific gravity in the range 2.85-2.94 (PC is ≈ 3.15).

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

Although GGBS is cementitious on its own, the process is very slow unless there is an activation. It is a hydraulic material and therefore if hydrated at elevated temperature and for a long time requires no additives other than water for hydration and hardening to take place (Song *et al.*, 2000; of North East Slag Cement Ltd. Data sheet 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 cement applications where it provides enhanced durability, high resistance to chloride penetration, resistance to sulfate attack and protection against alkali silica reaction (ASR). Its use in soil stabilization is however still a novel process in the UK although it has been used in South Africa. The well-established sulfate-resisting properties imparted to cements by blending them with GGBS suggests that by blending lime with GGBS, the latter may impart similar sulfate-resisting properties to lime-stabilized clay (Wild *et al*, 1996). In both hydrated PC and mixtures of sulfate-containing clay mixes and PC, the phases present are similar – ettringite, C-S-H, C-A-H and C-A-S-H gels. As PC hydration products are practically the same as those of slag-Portland cement blends, then it is likely that those of lime-slag-clay are also similar or at least comparable with those of cement-slag blends (Smolczyk, 1980, Regourd, 1980, Kinuthia, 1997).

The utilization of GGBS in soil stabilization is a relatively new phenomenon in the UK. The introduction of GGBS into a clay-lime hydration system modifies the clay-lime reaction products (Wild *et al.*, 1998). GGBS provides additional alumina, calcia, silica and magnesia to the system, depending on the type and amount of GGBS (Smolczyk, 1980). Since the principle reactants introduced by GGBS are also present in the clay-lime system as well as in PC-GGBS blends, the strength of hydrated clay-lime GGBS mixtures is likely to be governed by the same factors observed in hydrated PC-GGBS blends. These factors include water content and curing environment, the properties of the C-S-H gel, such as its amount, porosity and permeability, the fineness and composition of all reactants involved, the efficiency of mixing, the temperature and the curing period. GGBS also introduces extra and more freely available alumina and silica, due to its high reactivity in the presence of lime, thus enhancing strength by contributing silicates and aluminates. The lime in the clay-lime mix will provide the required alkaline environment for GGBS activation and hydration, whilst also modifying the clay (Wild *et al.*, 1998).

3.3.2.2 Chemical Composition and Properties of GGBS

3.3.2.2.1 GGBS Activation and its Hydration Products

GGBS can be activated in different ways, but the most common is chemical activation. In chemical activation, an activator is required and/or an alkaline medium. Several activators have been suggested to activate GGBS. Calcium hydroxide, calcium sulfate, standard PC, sodium hydroxide, sodium carbonate, sodium sulfate and sodium silicate are the most commonly used activators (Gjorv, 1989). Wu *et al.*, (1990) suggested that sodium hydroxide, sodium sulfate and potassium aluminium sulfate can be used as activators for GGBS and can help in breaking the Si-O and Al-O bondings. Wild and Tasong (1990) used lime as an activator in their study into the influence of GGBS in the sulfate-resistance of lime-stabilized kaolinite. They observed that the optimum lime/GGBS ratio is 1:5 to activate the GGBS, and to prevent attack caused by excess sulfate solution. They 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.

The most commonly used activator for GGBS is however PC. The reaction of PC with GGBS and water is complex. Water hydration of PC produces mainly calcium hydroxide ($Ca(OH)_2$) and C-S-H gel. In the hydration of blended PC, although minor amounts of alkalis are released, GGBS is mainly activated by the hydration product $Ca(OH)_2$ (Hakkinen, 1993; Bijen, 1996). Therefore lime in the form of $Ca(OH)_2$ may be added either as an additive or released from PC hydration. GGBS, due to its high alumina and silica content, produces slightly different hydrates from those formed when using standard PC. 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).

Douglas *et al.*, (1991) observed that 3% by weight of hydrated lime can retard the setting time of alkali-activated GGBS concrete. Maphee *et al.*, (1989) found that the GGBS hydraulic reactions are slower than the hydration of PC and have a "pore-blocking" effect which leads to a greater ultimate strength and lower permeability. This, together with the reduced $Ca(OH)_2$ and other improved binding and absorptive effects, enhances the resistance of GGBS concrete to sulfate attack.

Calcium sulfate is not only a successful activator but also plays an important role as a reactant (Daimon, 1980; Taylor, 1990). 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 (Ouf, 2001).

To understand how the addition of GGBS alters the soil properties, GGBS hydration should be studied in some detail. The effectiveness of GGBS hydration depends on many factors. These are the chemical composition of the GGBS, alkali concentration of the reacting system, glass content and fineness of GGBS, and temperature during the early stages of the hydration process (Kinuthia, 1997). The initial reaction during GGBS hydration produces coatings of alumino-silicate products on the surfaces of GGBS grains within a few minutes of exposure to water. These surface layers are impermeable to water, inhibiting further hydration reactions (Daimon, 1980). Therefore, GGBS used on its own shows only minimal hydration. Caijun and Day (1993) studied the hydration of a Canadian GGBS and found that when GGBS is in contact with water, an 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 C-S-H, C-A-H or C-A-S-H components. They also found that only a small amount of C-S-H was formed after 150 days of moist curing. For this reason therefore, utilization of GGBS is based on its activation.

When PC hydrates, the principal hydration products are calcium hydroxide (CH) and calcium-silicate-hydrate (C-S-H) gel, whereas GGBS hydration products using an alkali are mainly C-S-H and hydrotalcite type phase containing gypsum (Wild *et al.*, 1998; Tasong *et al.*, 1999). 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. Talling (1989) using XRD, studied lime-alkali-activated GGBS. He identified the presence of C_4AH_{13} as well as the formation of C-S-H. In addition to C-S-H and C-A-H, ettringite 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 the dissolution of Al_2O_3 and SiO_2. These are liberated from the GGBS and/or any other sources 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 ($C_3A.3CaSO_4.32H_2O$).

The microstructural features in the GGBS/activator mixture comprise a poorly crystallized hydrated layer on the grain surfaces (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 cement paste (Smolczyk, 1980; Taylor, 1990).

3.3.2.2.2 Clay-Lime-GGBS Reactions

The introduction of GGBS into the lime-clay hydration will undoubtedly modify the lime-clay reaction products. The clay-lime-GGBS reaction is different from the clay-lime reaction in that 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 known to consume lime. The second reaction is the typical 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-clay mix provides the required alkaline environment for GGBS activation and hydration. Thus, the observed increases in strength in GGBS-Portland cement blends are also likely to take place in lime-clay-GGBS mixes since the GGBS will introduce extra and more freely available alumina and silica thereby enhancing the formation of the strength contributing silicate and aluminate hydrates, compared to the clay-lime system.

Wild *et al.*, (1999) suggested that the total binder content (GGBS and lime) would be determined by the required engineering properties. The partial replacement of lime 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 lime replacement should not exceed a certain percentage, in order to keep a minimum lime content sufficient to fully activate the GGBS (Wild *et al.*, 1999).

3.3.2.3 Effects of GGBS on the Engineering Behaviour of Soil

3.3.2.3.1 Effects of GGBS on the Consistency (Atterberg) limit

Akinmurusu (1991) studied the effect of adding GGBS on the consistency, compaction characteristics and strength of lateritic soil. The slag content varied from 0% to 15% by dry soil weight. He observed a decrease in both liquid and plastic limit 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 the 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 a decrease 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. They 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.

3.3.2.3.2 Effects of GGBS on the Compaction Characteristics of Soils.

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.

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

3.3.2.3.3 Effects of GGBS on the Strength of Soils

The strength gain using GGBS activated by lime has been investigated by many researchers. Gupta and Seehra (1989) in their studies, found that lime-GGBS soil stabilized mixes with and without the 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. They concluded that partial replacement of GGBS with fly ash further increased the UCS. Akinmusuru (1991) studied the effect of adding GGBS on shear strength parameters. He stated that the CBR increased with an increase in GGBS percentage up to 10% GGBS content, and then started to decrease.

Wild *et al.*, (1998) studied the influence of GGBS and gypsum on kaolinite and Kimmeridge Clay when stabilized 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 were more significant in the presence of gypsum. After 28 days, the most significant strength enhancement was at high lime-GGBS ratio, and was 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 mixture with gypsum, due to the accelerating effect of gypsum on the lime-activated GGBS hydration. They also found that there was no effect on the 7 and 28 day strength of kaolinite clay when GGBS alone was added. Higgins *et al.*, (1998) observed that the optimum lime:GGBS ratio to achieve maximum UCS is 1:5 for kaolinite clay and about 2:3 for Kimmeridge Clay.

3.3.2.3.4 Effects of GGBS on the Swelling potential of Soils.

Higgins *et al.*, (1998) studied the effect of GGBS on the strength and swelling properties of lime-stabilized kaolinite in the presence of sulfate. They found that kaolinite clay containing gypsum and stabilized with lime produced large expansion when saturated with water. The addition of GGBS to the clay-lime-gypsum system results in great reduction in expansion. In their laboratory investigation and full scale trial, they demonstrated that lime-GGBS combinations can be successfully used for soil stabilization in the presence of sulfate to prevent swelling. They also found that the partial substitution of lime by GGBS could significantly reduce swelling and heave in the presence of sulfates. Wild *et al.*, (1999) stated that to eliminate problems of sulfate expansion, 60% to 80% of lime for stabilization is replaced by GGBS.

Higgins and Kennedy (1999) carried out a full site trial using GGBS and lime on a temporary diversion to carry the A421 Tingewick Bypass traffic. The soil on this site comprised a sulfate-containing boulder clay. GGBS activated by lime was used in particular sections and lime and cement in other sections. The temporary diversion performed well over a full year. No swelling problems were observed in the sections which were treated with GGBS activated by lime while expansion was detected in the sections which were treated with lime and cement without using GGBS.

Tasong *et al.*, (1999) observed in the kaolinite-lime-gypsum system, that expansion was found to be related to ettringite formation. Expansion increased with an increase in sulfate content and increased linearly with water absorption.

3.3.3 Wastepaper Sludge Ash (WSA)

3.3.3.1 Introduction

The residue effluent from recycling wastepaper consists of de-inking sludge and primary clarified sludge. The residue is dewatered in a screw press to about 50% solids before discharge (Péra and Ambroise, 1999). Dry wastepaper sludge comprises approximately equal amounts of organic and inorganic components, the latter consisting principally of limestone and kaolin. The latent energy of the organic component (mainly residual cellulose fibers) can be recovered by combustion of the sludge at temperatures in excess of 850°C, thereby reducing the volume of waste to be land-filled to around 40% or 50% of the original dry solids (Péra and Amrouz, 1998).

The composition of wastepaper sludge ash is a function of the type, grade and quality of the paper being recycled and also its thermal history. Péra and Ambroise (1998) have reported the kaolinite content in wastepaper sludge to be in the range 15%-75% and the calcite content ranges from 21%-70%. Kaolin undergoes dehydroxylation above 600°C to produce metakaolin ($Al_2Si_2O_7$), which is a semi-amorphous highly active pozzolan (Péra and Amrouz; Péra and Ambroise (1998). At 925°C it converts to crystalline spinel ($2Al_2O_3.3SiO_2$) and free silica (SiO_2), and the crystalline end products at 1,400°C are mullite ($3Al_2O_3.SiO_2$) and cristobalite (SiO_2). Calcite ($CaCO_3$) decomposes above 700°C to give quicklime (CaO) and carbon dioxide (CO_2). However heating calcite and kaolinite together will not necessarily result in the mixture of the above phases because quicklime and metakaolin react (depending on temperature and soaking time) to form a wide range of calcium alumino-silicate phases.

Péra and Ambroise (1998) reported that the two principal phases in wastepaper sludge heated at 630°C-750°C are gehlenite ($Ca_2Al_2SiO_7$) and anorthite ($CaAl_2Si_2O_8$), although for sludge with high calcite to kaolin ratios, calcium silicate (larnite Ca_2SiO_4) replaces anorthite. Larnite has latent hydraulic properties whereas gehlenite and anorthite are considered to be non-hydraulic (Taylor, 1990; Péra and Amrouz, 1998). Péra and Ambroise (1999) have also shown that wastepaper sludge calcined by soaking at 700°C-750°C (to give metakaolin and calcite) and used as a mineral admixture in high strength concrete, is equally as effective as a pozzolan as silica fume and metakaolin.

In the UK, one of the principal wastepaper recycling companies, Aylesford Newsprint Ltd., combusts wastepaper sludge in a fluidized bed and utilizes the resultant energy to run and operate the plant. The resultant ash (~700 tonnes/week) is currently dumped to landfill. Research work by Kinuthia *et al.* (2001) and by Bai *et al.* (2003) has also elucidated the composition of wastepaper sludge ash (WSA). Together, these researchers have carried out Transmission Electron Microscopy (TEM, X-ray diffraction (XRD) and Derivative Thermogravimetric Analysis (DTGA), with a view to identifying the mineral components in WSA, as well as the hydration products of WSA and WSA blended with GGBS. The principal crystalline components in WSA were found to be gehlenite, calcium oxide, bredigite and α'-C_2S, together with a small amount of anorthite and calcium carbonate and traces of calcium hydroxide and quartz. In addition the WSA also contains up to 20% amorphous glassy phase of composition within the range of commercial slags.

3.3.3.2 WSA Hydration

The hydration products identified in WSA paste of WSA from Aylesford Newsprint Ltd., Kent, UK were CH, C_4AH_{13}, $C_3A.0.5C\bar{C}.0.5CH.H_{11.5}$ and C-S-H gel plus possible evidence of small amounts of C_2ASH_8 and $C_3A.3C\bar{S}.H_{32}$. In addition the ash was found to be quite highly alkaline (pH 11-12) probably as a result of residual free lime which was found to be about 5wt%. This particular WSA has been used to establish the chemistry, hydration properties and possible utilization in concrete. (Bai *et al.*, 2003). The authors, found from XRD analyses of the hydrated WSA, that on hydration a rapid reduction in the principal CaO peaks occurs (i.e. peaks disappear after 1 day). Some of the CaO peaks were found to overlap with the α'-C_2S/bredigite peaks which also showed a rapid reduction in intensity (Bai *et al.*, 2003). They suggests that the α'-C_2S/bredigite also rapidly hydrates. Although the authors detected no metakaolin in the WSA, hydration products of WSA are similar to the ones observed by Pera and Amrouz (1998).

Investigations on the compressive strength of pastes carried out by Bai *et al.*, (2003), showed that the paste made from 100% WSA achieved very low strength as a result of a high level of coarse porosity and a significant degree of unsoundness. However when WSA is blended with GGBS, significant pore refinement occurs and the unsoundness is removed partially as a result of the increase in the effective water to WSA ratio which enables a greater degree of CaO hydration to occur prior to setting (Bai *et al.*, 2003). They suggested that the slow hydration of GGBS coupled with further hydration of one of the WSA components provides significant strength development. This hydraulicity can be attributed to the presence of lime in the WSA, which activates the GGBS hydration.

CHAPTER 4 - MATERIALS

This chapter describes the materials used in the current research and includes mineralogical, oxide and/or chemical composition of the subject material (Lower Oxford Clay and Kaolinite) and contains detailed analytical data on all stabilizers used (WSA, GGBS, Quicklime and PC)

4.1 LOWER OXFORD CLAY (LOC)

Lower Oxford Clay is the principal soil under investigation with regard to stabilization. The clay is grey in colour and is known to have high sulfate and sulfide contents, the former as gypsum, the latter as pyrites. It was supplied by Hanson Brick Ltd., Stewartby, Bedford. Mineralogy studies by Hanson Brick Ltd. found that the LOC that they use in their brick making process, contains illite (23%), kaolinite (10%), chlorite (7%), calcite (10%), quartz (29%), gypsum (2%), pyrite (4%), feldspar (8%) and organics (7%) (Thomas, 2001).

Investigation by Wild et al. (1996) established that Lower Oxford Clay consisted predominantly of chlorite, kaolinite, mica, feldspar (sanidine and anorthite), quartzite, and calcite, with minor quantities of gibbsite, geothite (a hydrated iron oxide) and anatase (titanium oxide).

Mineralogy and chemical analyses found that Lower Oxford Clay has many minor ingredients, such as pyrites and gypsum, which can be contributors to the formation of deleterious minerals such as ettringite and thaumasite during its stabilization with lime. It is therefore thought to be an excellent choice for investigative work. Table 4.1 shows the oxide composition and mineralogy of two LOC batches, from Hanson Brick Ltd. (Thomas, 2001) one of which was used by Wild et al., (1996). Table 4.2 shows some chemical composition data which include carbonate, sulfate, sulphur (as sulfide), chloride and soluble silica for the LOC used in the current study.

Approximately 900 kg of crushed clay was used in this study. The clay was air dried at room temperature, homogenized, and then sealed in polythene bags until it was required in the research work.

Table 4.1: Oxide and mineral analyses of the Lower Oxford Clay

Oxide	**Wild et al. (1996) (wt. %)**	**Hanson Brick Ltd. (Smith, 1999) (wt.%)**
SiO_2	55.42	46.73
TiO_2	0.86	1.13
Al_2O_3	19.88	18.51
Fe_2O_3	6.21	6.21
FeO	0.68	0.80
Mn_2O	0.07	0.07
CaO	8.56	6.15
MgO	1.83	1.13
K_2O	3.22	4.06
P_2O_5	-	0.17
Na_2O_3	-	0.52
LI	-	15.79
Mineralogy		
Chlorite	6.47	7
Illite	22.02	23
Gypsum	1.3	2
Kaolinite	7.62	10
Quartz	28.5	29
K-feldspar	2.37	8
Plagioclase	4.15	-
Calcite	10.44	10
Siderite	4.14	-
Anatase	0.82	-
Pyrite	3.25	4
Apatite	0.41	-
Organics	-	7

Table 4.2: Chemical composition of LOC (Bodycote Material Testing, July 2003)

Chemical Composition	**(%)**
CO_3	5.02
CaO	5.60
Cl	0.01
IR	91.6
S	0.018
SO_4	1.29
SiO_2	0.43
Total sulfur	1.50

4.2 KAOLINITE

Industrial kaolinite was used as a source of kaolinite. It was supplied by Imerys Pigments & Additives Ltd., Cornwall, UK, under the commercial trade name of "Standard Porcelain". It is a white powder consisting of 83% kaolinite, 13% mica, 1% feldspar and 2% of other minor mineral compositions. Table 4.3 shows the particle size distribution, oxide and mineralogical composition of the Standard Porcelain, as supplied by Imerys Pigments & Additives Ltd.

Table 4.3: Particle size distribution, oxide and mineralogical composition and engineering properties of "Standard Porcelain"

Sieve Size	%	Engineering properties	%
> 53 μm	0.03	Liquid Limit	61
> 10 μm	4	Plastic Limit	32
< 2 μm	70	Plasticity Index	29
Chemical Analysis		**Others**	
SiO_2	48	Specific gravity	2.57
Al_2O_3	37	Max. dry density (MDD) (Mg/m^3)	1.50
Fe_2O_3	0.65	Optimum Moisture Content (OMC) (%)	26
TiO_2	0.02	pH	5.9
CaO	0.07	Natural moisture content (%)	2.5
MgO	0.30		
K_2O	1.60		
Na_2O	0.10		
Loss on ignition (LOI)	12.5		
Mineralogical Comp.			
Kaolinite	83		
Micaceous material	13		
Feldspar	1		
Other materials	2		

4.3 STABILIZERS

4.3.1 Wastepaper Sludge Ash (WSA)

The wastepaper sludge ash (WSA) was supplied by Aylesford Newsprint Ltd. in 25kg airtight plastic bags, in the form of a dry fine grey powder with a small percentage (less than 10%) of coarse sand-size particles. Two samples were sent to Southern Water Ltd. for oxide composition analysis shown in Table 4.4.

Table 4.4: Oxide composition of Wastepaper Sludge Ash.

Oxide	**Sample 1**	**Sample 2**	**Average**
SiO_2	33.73	34.08	33.9
Al_2O_3	18.21	17.89	18.05
Fe_2O_3	0.95	0.97	0.96
CaO	37.32	36.33	36.82
MgO	5.12	5.76	5.44
Na_2O	0.29	0.26	0.27
K_2O	0.45	0.47	0.46
Li_2O	0.0037	0.0045	0.0041
P_2O_5	0.29	0.26	0.27
TiO	0.37	0.38	0.37
MnO	0.03	0.03	0.03
BaO	0.0195	0.0277	0.024
SrO	0.0497	0.0555	0.54
SO_3	0.26	0.22	0.24
CuO	0.0356	0.0376	0.0366

The WSA consignment that was used for this study is rather coarse (specific surface 409 m^2/kg) relative to previous consignments due to the presence of the sand-size particles from the fluidized bed used in the combustion of the wastepaper sludge. This has the effect of lowering the CaO content to 36%-37% (normally about 40%-45%), and increasing the SiO_2 content to about 34% (normally 22-26%). There is no significant change in the other parameters. A sample of the WSA was also sent to Bodycode Materials Testing Ltd. for Chemical analysis in accordance to BS 1881: Pt 124, whose results are shown in Table 4.5.

Table 4.5: Chemical analysis for WSA

Chemical composition	**(%)**
Insoluble Residue	38.6
Soluble Silica	18.6
Calcium Oxide	30.7
Sulfate as SO_3	0.33
Carbonate	1.85
Soluble Al_2O_3	12.1
Soluble MgO	1.89
Soluble Fe_2O_3	0.56

4.3.2 Ground Granulated Blastfurnace Slag (GGBS)

Ground Granulated Blastfurnace Slag (GGBS) was supplied by Civil and Marine Slag Cement Ltd, Llanwern, Newport, South Wales, UK. Table 4.6 shows its oxide composition and some physical properties.

Table 4.6: Oxide composition and some physical properties of GGBS

Oxide	**(%)**
CaO	42.0
SiO_2	35.5
Al_2O_3	12.0
MgO	8.0
Fe_2O_3	0.4
MnO	0.4
S_2	1.2
SO_3	0.2
Insoluble residue	0.3
Specific Gravity	2.9
Bulk Density, kg m^{-3}	1200
Colour	Off-white
Glass content	≈ 90

4.3.3 Lime

The main ingredient in lime commonly used in construction today is calcium oxide (CaO). This type of lime is called high calcium quicklime. Dolomitic lime, which contains significant amounts of magnesium oxide (MgO) is also available. For both compounds, the lime is formed by calcining crushed limestone (predominantly $CaCO_3$) at a temperature of about 980°C. The high temperature releases carbon dioxide, as illustrated by the following reaction:

$$CaCO_3 + \text{Heat} \rightarrow CaO + CO_2 \uparrow$$

Commercially produced lime is sold from the kilns as either quicklime or hydrated lime. Quicklime is the calcium oxide (CaO) produced from calcining limestone ($CaCO_3$) while hydrated lime is quicklime that has been slaked (mixed with a small amount of water). Hydrated lime is formed by the following reaction:

$$CaO + H_2O \rightarrow Ca(OH)_2 + \text{Heat} \uparrow$$

This reaction happens quickly and produces a significant amount of heat. After slaking, the lime becomes a very fine powder. The molecular weight of quicklime is 56.08, while the molecular weight of hydrated lime is 76.09 (see Table 4.7). From these weights, the ratio of hydrated to quicklime required to provide the same amount of calcium is 1.321 (Little, 1995).

Quicklime is less expensive to ship because for a given amount of Ca it weights less, but it can be more dangerous to handle because of the high energy released when it is mixed with water. Quicklime will react with atmospheric moisture and both forms of lime will react with carbon dioxide in the air to reform calcium carbonate. This carbonation is a reversal of the calcining reaction. It is a relatively slow reaction, but, once carbonated, lime is rendered ineffective for use in construction (Little, 1995) until it has been calcined again.

Table 4.7: Molecular Weights of various elements

Components	Molecular Weights
Hydrogen (H)	1.00794
Oxygen (O)	15.9994
Calcium (Ca)	40.078
Calcium Oxide (CaO)	56.077
Calcium Hydroxide ($Ca(OH)_2$)	74.093

Quicklime (CaO) in the current work was supplied by Buxton Lime Industries Ltd., in the form of a white powder of cement fineness. Table 4.8 shows its oxide composition and some of its physical properties.

Table 4.8: Oxide composition and physical properties of quicklime

Oxide Composition	(%)
CaO	95.9
SiO_2	0.9
Al_2O_3	0.15
MgO	0.46
Fe_2O_3	0.07
$CaCO_3$	2.2
Specific Gravity	2.3
Bulk Density, kg m^{-3}	480
Colour	White

The use of Quicklime has several advantages over hydrated lime (Ca $(OH)_2$):

- Quicklime has a higher available lime content per unit mass than hydrated lime. 3% CaO is normally equivalent to 4% hydrated lime,
- Quicklime is denser than hydrated lime, requiring less storage and transport space,

- Quicklime is considerably less dusty than hydrated lime,

- Quicklime produces a large reduction in moisture content due to consumption of water caused by its hydration reaction and evaporation due to the increase in temperature from the exothermic reaction. It is therefore particularly beneficial with wet soils.

- Quicklime generates heat that accelerates strength gain. This is of benefit in a temperate climate such as that found in the UK.

4.3.4 Portland Cement (PC)

The Portland cement was supplied by Rugby Cement, a division of Rugby group Plc., Crown House, UK. The oxide and compound compositions of the PC are shown in Table 4.9.

Table 4.9: Oxide and Compound compositions of Rugby PC (Rugby Group Plc.)

Oxide Composition	**(%)**
CaO	65.6
SiO_2	21.00
Al_2O_3	4.63
Fe_2O_3	2.26
MgO	1.18
SO_3	2.69
K_2O	0.78
Na_2O	0.16
Loss on Ignition	0.99
$C\bar{S}$	4.57
C_3A	8.20
C_3S	64.00
C_2S	11.00
C_4AF	6.70

4.4 OTHERS

4.4.1 Silica Gel

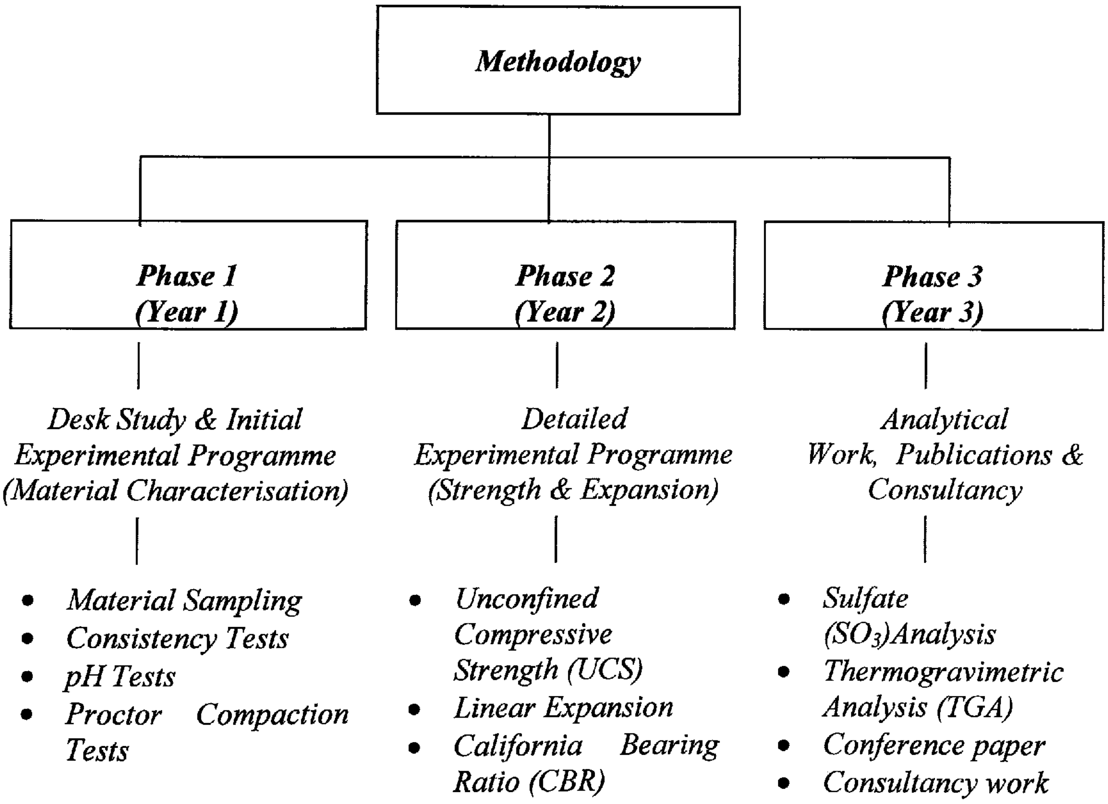
Silica Gel was used to dry soil samples in a desiccator cabinet for further analytical tests. It is a porous, granular form of silica, synthetically manufactured from sodium silicate. The internal structure of each small silica gel granule is composed of a vast network of inter-connecting microscopic pores, which attract and hold moisture by a phenomenon known as physical adsorption and capillary condensation. Silica gel can adsorb about 40 percent of its weight in moisture and can take the relative humidity in a closed container down to about 40 percent. Each hydrosorbent silica gel unit has a built-in indicator which turns from blue to grey/colourless signalling when the product is saturated with moisture and needs reactivation. Once saturated with moisture, silica gel can easily be reactivated in a conventional oven by heating it above 150°C for reuse.

CHAPTER 5 – EXPERIMENTAL PROCEDURES

This chapter describes the mix compositions, sample preparation and other experimental procedures and analytical techniques used in the current work, namely:

1. *Initial Consumption of Stabilizer*
2. *Consistency (Atterberg) Limits*
3. *Proctor Compaction Test*
4. *Unconfined Compressive Strength (UCS)*
5. *California Bearing Ratio (CBR)*
6. *Linear Expansion*
7. *Sulfate (SO_3) Analysis*
8. *Thermogravimetric Analysis (TGA)*

The methodology in this study was divided into 3 phases as illustrated below;



Methodology

Phase 1 (Year 1)

Desk Study & Initial Experimental Programme (Material Characterisation)

- *Material Sampling*
- *Consistency Tests*
- *pH Tests*
- *Proctor Compaction Tests*

Phase 2 (Year 2)

Detailed Experimental Programme (Strength & Expansion)

- *Unconfined Compressive Strength (UCS)*
- *Linear Expansion*
- *California Bearing Ratio (CBR)*

Phase 3 (Year 3)

Analytical Work, Publications & Consultancy

- *Sulfate (SO_3) Analysis*
- *Thermogravimetric Analysis (TGA)*
- *Conference paper*
- *Consultancy work*

5.1 INITIAL CONSUMPTION OF STABILIZER

Clay portions each weighing 20 g were placed in six conical flasks fitted with corks. Varying percentages of each of the different stabilizers shown in Table 5.1 were placed in each flask and 100 ml of de-ionised water was added into the flasks as illustrated in Figure 5.1. The flasks were shaken in order to mix the stabilizers with the target clay material. Both LOC and Kaolinite were used for this test. The temperature and pH of the solution were monitored and recorded after every 10 minutes for the first one hour and every 1 hour for the next 4 hours and at 24 hours.

BS 1924 recommends a temperature of 25°C for pH determination and reporting, and suggests a pH correction for temperature if tests are carried out above or below 25°C, viz:

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

Where pH_{25} = pH at 25°C

PH_T = pH at prevailing temperature

T = prevailing temperature (°C)

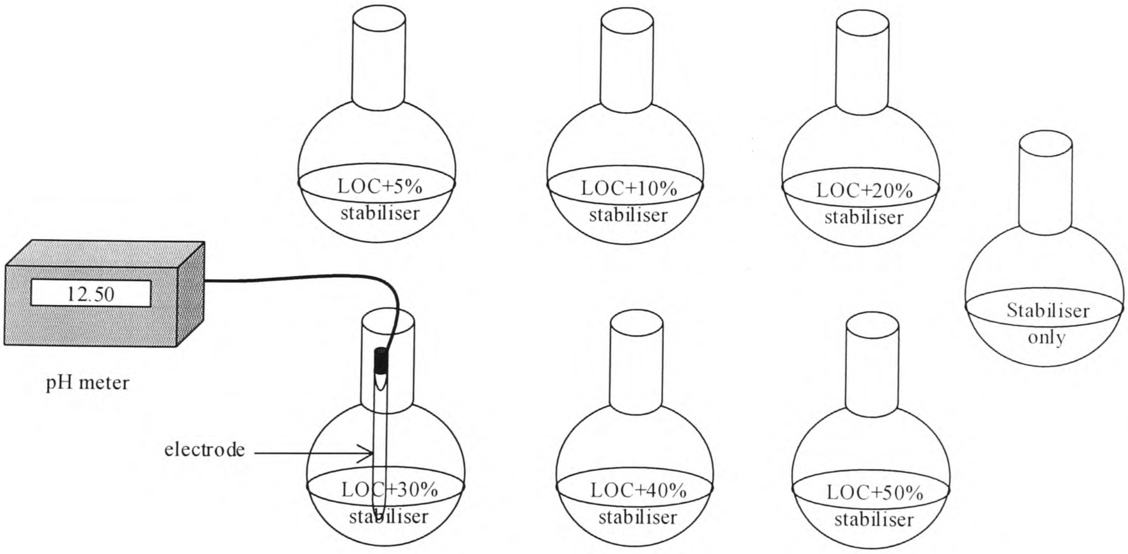


Figure 5.1: Illustration of pH test procedures.

5.1.1 Stabilizers Mix Composition

The stabilizers used to stabilize the LOC were WSA, with and without combined action with lime, PC or GGBS. The control mixes were LOC stabilized with 2%, 4% and 6% lime. The dosages of WSA, and WSA blends were 10%, 15% and 20%. These stabilizer levels had been established in previous basic and preliminary investigation, as the stabilizer levels likely to achieve the minimum CBR value of 15% that is normally stipulated by the Department for Transport (D_fT) for a lime-stabilized capping layer (Design Manual for Roads and Bridges, Part 6, 2000). For the blended binders, two blending ratios were investigated, as shown in Table 5.1, which shows a summary of all the stabilizer system investigated.

Table 5.1: Details of mix compositions for LOC + Stabilizers systems

Stabilizers	**Ratio**	**Wt.%**
Lime	100%	2, 4, 6
WSA	100%	10, 15, 20
WSA-lime	90:10	10, 15, 20
	80:20	10, 15, 20
WSA-PC	90:10	10, 15, 20
	80:20	10, 15, 20
WSA-GGBS	70:30	10, 15, 20
	50:50	10, 15, 20

In order to make a blended stabilizer, the stabilizers were weighed individually and blended thoroughly in a SE-401 Hobart 40 Qt mixer until an homogeneous mixture was obtained. The blended stabilizers were stored in well-sealed polythene bags and kept in different containers for further use.

5.2 CONSISTENCY (ATTERBERG) LIMITS

The Consistency Limit tests were carried out in accordance with BS 1377 (1990) Part 2 – Classification Test (British Standard Methods of Test for Soils for Civil Engineering Purposes). They were carried out on Lower Oxford Clay (LOC) that was dried, crushed and sieved passing 425 μm (in accordance with the BS). Additionally, tests were carried out on LOC and Kaolinite containing various percentages of the different stabilizers (shown in Table 5.1) in order to establish the effect of the stabilizers on Atterberg limits of stabilized LOC and Kaolinite.

5.2.1 Liquid Limit

A small amount of de-ionized water was first added to about 200g of LOC that had been homogeneously mixed with stabilizers. The wet materials were then thoroughly mixed with palette knives into a plastic paste. A small amount of this material was set aside for the later determination of the Plastic Limit. The remaining material was then used to fill a brass cup for cone penetration (56mm diameter and 41mm deep). A metal cone of angle 30 ± 1^0 and 35mm height (see Figure 5.2) was allowed to penetrate into the material's surface for 5 seconds (automatically timed by an attached electrical timing device). After 5 seconds, the penetration of the cone was automatically stopped by a locking mechanism incorporated in the penetration apparatus and the penetration of the cone determined by a scale and dial pointer attached to the cone. The penetration was recorded and a small amount of material taken within the zone of penetration, for moisture content determination. The moisture content was determined in accordance with BS 1377 (1990) using the oven drying method. The residual material in the cup was mixed with the rest of the sample and more water was added. Further penetration tests, together with their corresponding moisture content tests were performed so as to obtain at least 4 sets of points in the penetration range 5-30mm. The liquid limit was taken as the moisture content corresponding to a penetration of 20mm from a graph of penetration (in mm) against moisture content (%).

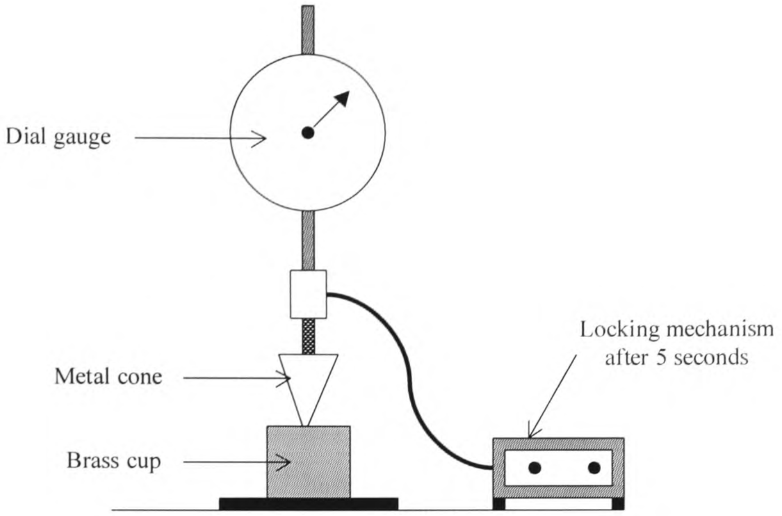


Figure 5.2: Cone Penetrometer apparatus.

5.2.2 Plastic Limit

Plastic Limit (PL) was determined by using the material that had been set aside after the initial mixing during the liquid limit testing. The material was gently rolled on a flat glass plate into rods, and rolling continued until the rods crumbled when they were approximately 3 mm in diameter. When the desired crumbling occurred, a few samples were taken for moisture content determination. This moisture content at which the clay rods crumble at 3 mm diameter is defined as the plastic limit of the material. From the liquid limit and the plastic limit, the plasticity index (PI) of the material was obtained as the difference between these two limits. The specification for the design and construction of lime stabilized capping (D$_f$T, 2000) states that clay soil must have a PI of 10% or more in order to be considered for lime stabilization.

5.3 BS PROCTOR COMPACTION TESTS

Compaction tests, in accordance with BS 1377 (1990) Part 2, were used to determine the maximum dry density (MDD) and the optimum moisture content (OMC) of LOC and Kaolinite with and without various percentages of the different stabilizers. The MDD and OMC were later used for the preparation of cylinder specimens for the determination of unconfined compressive strength (UCS), linear expansion and California bearing ratio (CBR) tests. Mixing of the dry materials was achieved using a Model SE-401 Hobart 40 Qt mixer. Compaction was achieved by dynamic compaction using an automatic Proctor/CBR compaction machine (see Figure 5.3). The material was compacted in 3 equal layers in a mould of dimensions 105 ± 0.5 mm in diameter and 115.5 ± 1 mm in height, each layer being subjected to 27 blows using a 2.5 kg rammer. After weighing the mould together with the compacted cylinder, a small amount of material was taken from the interior of the compacted material, for moisture content determination. The dry density-moisture content curves each with at least four data points were used to determine the MDD and OMC values.

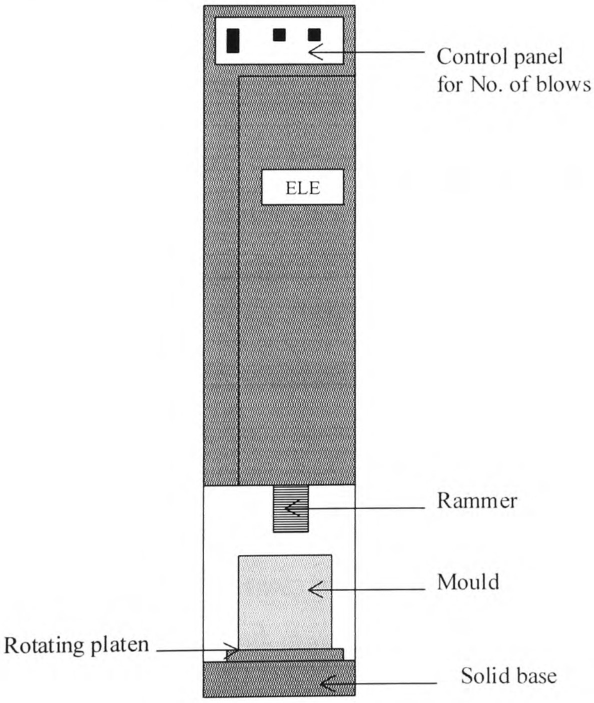


Figure 5.3: Automatic Soil Compactor

5.4 UNCONFINED COMPRESSIVE STRENGTH (UCS)

5.4.1 Specimen preparation

For sample preparation, it was necessary to establish a common dry density and moisture content. Therefore, results of the mean values of the dry density and optimum moisture content from BS Proctor compaction tests were adopted. In all the systems, LOC+Lime, LOC+WSA, LOC+WSA-Lime, LOC+WSA-PC and LOC+WSA-GGBS, the maximum dry density (MDD) ranged from 1.20 Mg/m^3 – 1.36 Mg/m^3 and the mean dry density value of 1.30 Mg/m^3 was adopted for all the mixes. For the optimum moisture content (OMC), the range was from 25% to 33%. Because the range is too wide, three mean moisture content values of 25%, 28% and 31% were adopted for the different specimen mixes. Within these mean OMC values, all test specimens were expected, within experimental error, to be of approximately comparable bulk-density, since the bulk volume was maintained as 50mm diameter and 100mm in height.

Using the mean dry density and the appropriate mean moisture content values, material meant for mellowing was mixed with the appropriate mean moisture content (25%, 28% or 31%) and stored in polythene bags to mellow for 3 days at 20 ± 1°C and 100% relative humidity. During specimen preparation, additional moisture equivalent to 20% of the mean OMC was applied to compensate for any moisture losses during the mellowing process and to facilitate compaction. For material that was not meant for mellowing, all the water necessary for the appropriate mean OMC was added and compaction carried out immediately. During this process, dry materials, enough to produce one compacted cylindrical test specimen were thoroughly mixed in a Kenwood Chef mixer for 2 min before slowly adding the pre-calculated amount of water. Intermittent hand mixing with palette knives was necessary to achieve a homogeneous mix. A steel mould fitted with a collar, so as to accommodate all the mixture, was used to compress the material into a cylinder of the prescribed dry density and moisture content. The pre-fabricated mould ensured that the material was not over compressed. Compaction was achieved using a hydraulic jack. After compaction, the test specimen cylinders were extruded using a steel plunger, trimmed, cleaned of releasing oil and wrapped in several runs of cling film, and cured for 7, 28, 90, 180 and 365 days in a temperature controlled chamber at 20 ± 1°C and 100% relative humidity. This humidity was ensured by storing the cling film wrapped specimens in a sealed polythene bag and then placed in a sealed plastic container.

5.4.2 Testing

Unconfined Compressive Strength (UCS) of stabilized clay samples was determined by using a Hounsfield testing machine capable of loading up to 10 KN at a compression rate of 1mm/min (see Figure 5.4). A self-leveling device was used to ensure uniaxial load application. Before testing, the end surfaces of samples were gently abraded to ensure a flat surface and good contact with the testing rig platens. Three cylinders per mix proportion were tested and the mean value taken. UCS tests were carried out on mellowed and unmellowed samples for all mix proportions on both LOC and Kaolinite. After testing, a small quantity of material was taken from the interior of the tested specimens and dried under silica gel in a low-temperature cabinet oven at 40°C. When the materials were totally dry, they were placed in plastic bottles, sealed and labeled and stored for further analytical tests later.

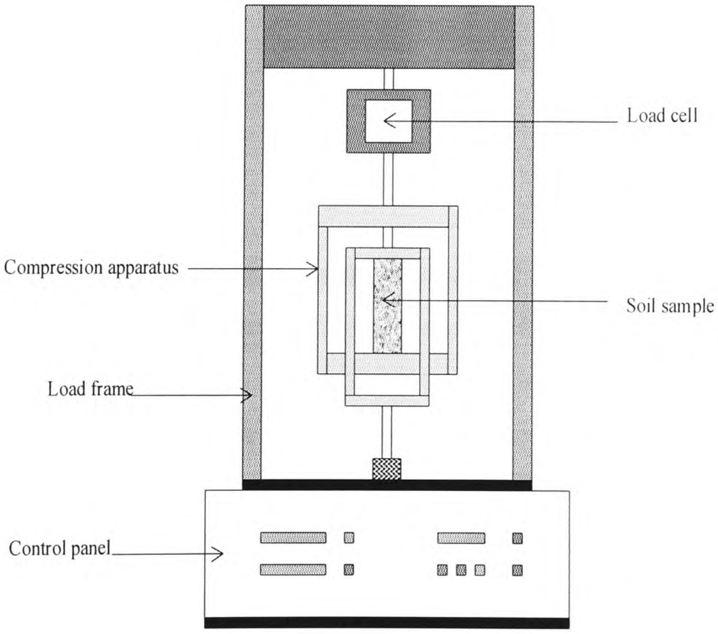


Figure 5.4: Hounsfield Test Equipment H10KM

5.5 CALIFORNIA BEARING RATIO (CBR)

The California Bearing Ratio Test was carried out in accordance with BS 1377- 4: 1990. The following test conditions were ensured before the test was started:

a) That the moisture content used was the same as that used for sample preparation for UCS tests.
b) That the CBR penetration test was carried out on both ends of the specimen.
c) That soaking of the specimen was carried out before testing.
d) That a two 2kg surcharge weight were placed on the specimen during soaking and during testing to represent traffic load.

About 6 kg (soil + stabilizer) of material was prepared for each sample to be tested. The sample was compacted in 3 equal layers in a CBR mould, having a nominal internal diameter of 152mm and fitted with a detachable baseplate and a removable extension collar. Each layer was subjected to 62 blows using a 2.5kg ram. After compaction, the extension collar was removed and the soil trimmed to level. The sample was then weighed and cured for 3 days in a temperature-controlled chamber at 20 ± 1°C and at least 65 ± 5°C relative humidity.

5.5.1 Soaking procedure

After 3 days of curing, the baseplate was removed from the mould and replaced with a perforated baseplate. The extension collar was fitted to the other end of the mould, and the screw threads covered with petroleum jelly to obtain a watertight joint. The mould was placed in an empty soaking tank and a filter paper placed on top of the sample, followed by a perforated swell plate. Two annular surcharge discs each 2kg in weight were placed on the perforated plate. A dial gauge support was mounted on top of the extension collar, a dial gauge secured in place and the stem adjusted on the perforated plate to give a convenient zero reading. The immersion tank was then filled with water to just below the top of the mould's extension collar. Readings of any swelling were recorded on a daily basis. After 3 days, the top of the sample was flooded and left to soak for another day, giving a total soaking period of 4 days. The soaking arrangement is illustrated in Figure 5.5.

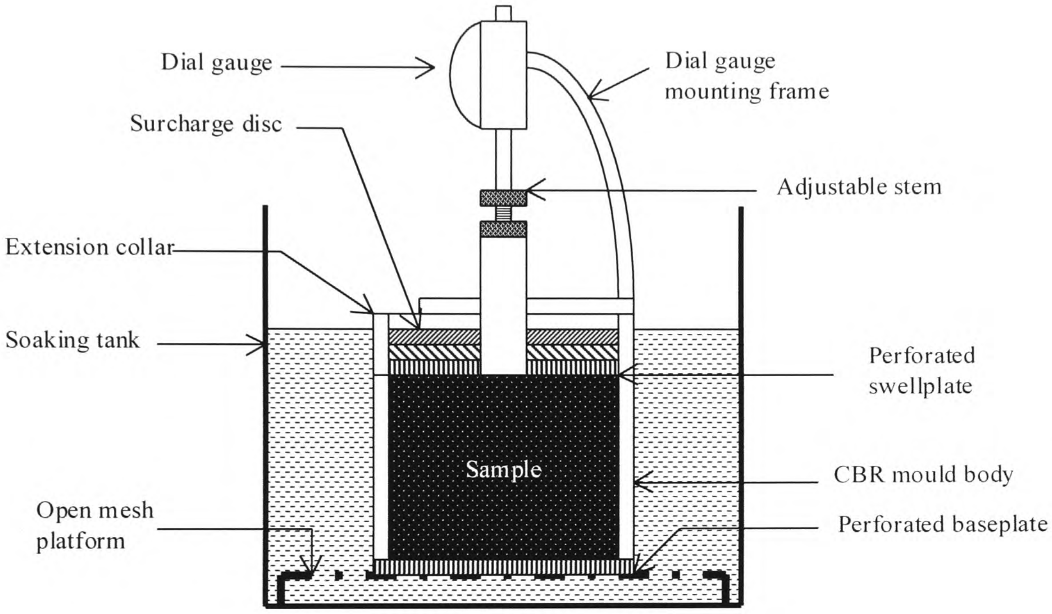


Figure 5.5: Arrangement for soaking condition

5.5.2 Penetration test procedure

The mould with baseplate containing the sample was placed on the lower platen of the testing machine with the top face of the sample exposed. Two 2kg annular surcharge discs were placed on top of the sample. The cylindrical plunger was positioned and a force-measuring device (proving ring) assembled with the face of the plunger resting on the surface of the sample. A seating force of 50N (for CBR value from 5% to 30%) was applied to the plunger. Both penetration dial gauge and force measuring device were set to zero. The general arrangement of the CBR test is as shown in Figure 5.6.

The penetration test was then started with the plunger penetrating the sample at a uniform rate of 1 ± 0.2 mm/min. Readings of the penetration force were taken from the force dial gauge at penetration intervals of 0.5mm, up to a total penetration of 7.5mm.

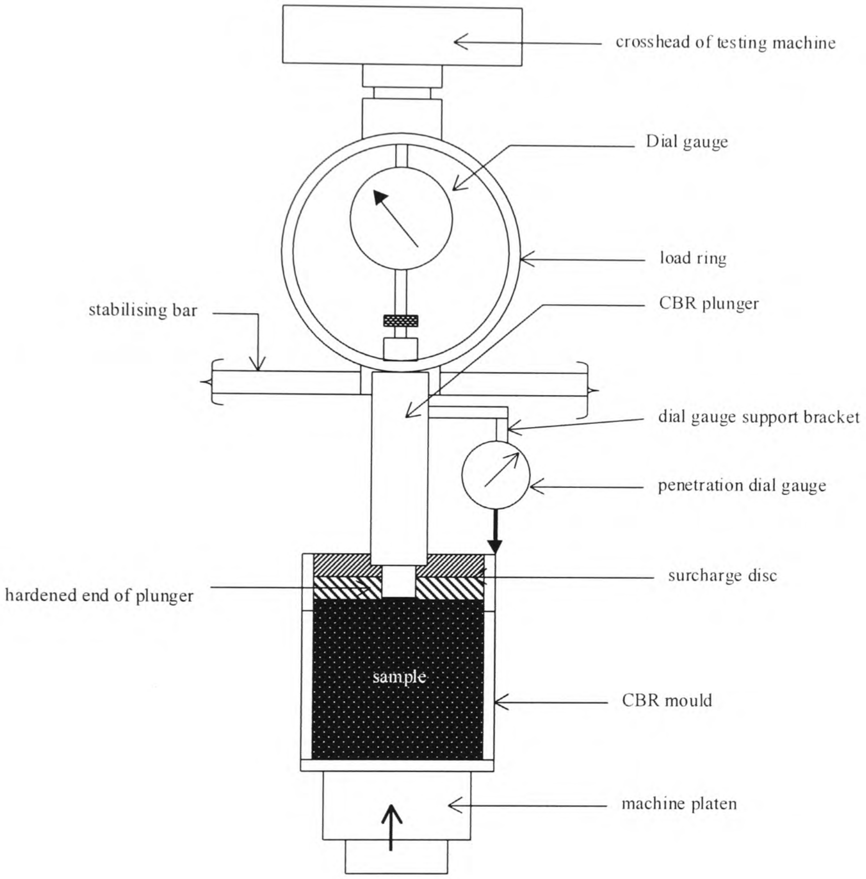


Figure 5.6: General arrangement for CBR test

5.6 LINEAR EXPANSION

Immediately after sample fabrication, approximately 10mm of the bottom of a test specimen meant for linear expansion test (one for each of the various mix compositions used) were exposed by cutting and removing the cling film and the specimen placed on a porous disc situated on a platform in a perspex container. Separate perspex containers were used for individual test specimens. The perspex containers were covered with lids fitted with dial gauges as shown in Figure 5.7.

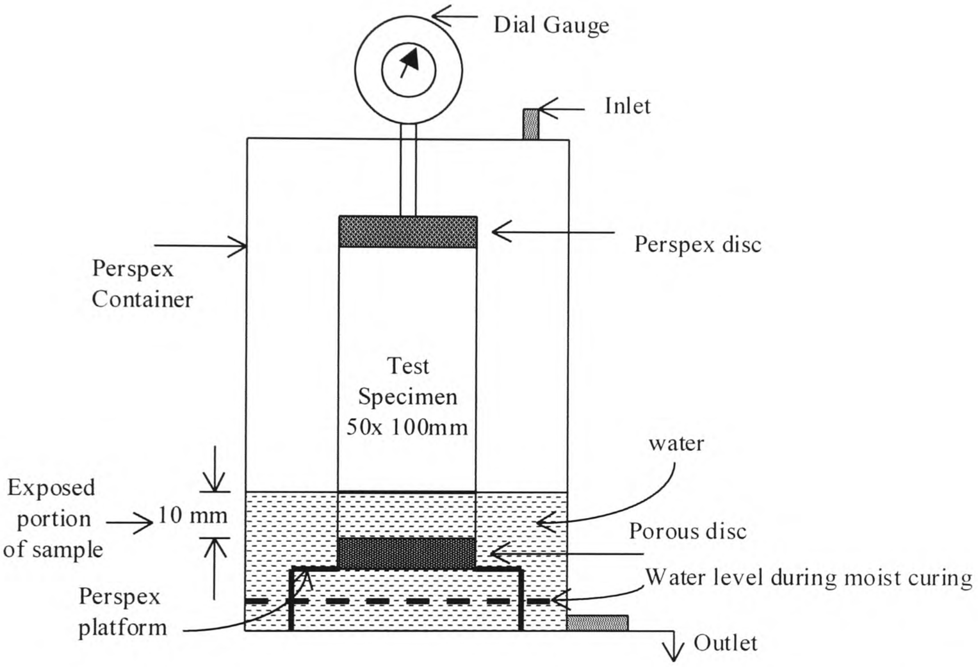


Figure 5.7: Schematic diagram of the test set-up for measuring linear expansion during moist curing and subsequent soaking.

A layer of water was always maintained below the platforms to provide high humidity and ensure that there was no excessive evaporation from the sample. This process which is termed moist curing was commenced immediately after sample fabrication. After moist curing for 7 days, the samples were partially immersed in water to a depth of 10mm above their bases by carefully increasing the water level in the perspex containers while ensuring that disturbance of the specimens was kept to a minimum. This process is termed soaking. During these processes, the containers were kept in a temperature controlled chamber (capable of maintaining temperatures to 20 ± 1°C and humidity to at least 65 ± 5% relative humidity). Thus the samples were cured in conditions maintained at about 20°C and 100% relative humidity. Linear expansion during moist curing and soaking was monitored on a daily basis for 100 days.

5.7 SULFATE (SO_3) ANALYSIS

In order to determine the amount of sulfate (SO_3) in the original Lower Oxford Clay and also to monitor subsequent changes in sulfate levels with time in the stabilized LOC, sulfate analyses were carried out according to BS 1047:1983 (sulfate analysis for air cooled blastfurnace slag aggregate for use in construction) – Appendix B3 (Method for the determination of acid soluble sulfate expressed as SO_3)

Approximately 1 g of soil was transferred into a beaker and 20ml 2-molar hydrochloric acid (HCl) added. The solution was brought to boil on a hot plate and 100ml boiling water added (see Figure 5.8). Three drops of methyl orange were added into the solution which was then made alkaline by means of adding ammonia. The solution was then filtered under gentle suction, leaving the residue in the funnel above the flask (see Figure 5.9). The filter paper that contained the residue was then transferred back to the beaker and re-dissolved with 5ml concentrated HCl and 70ml of boiling water and the same procedure repeated (added three drops of methyl orange, made alkaline by adding ammonia and filtered). The combined filtrates were then put into one beaker and made acid by adding drops of HCl, before adding 10ml of 10% barium chloride to the solution. The mixture was then kept for several hours before filtering into a pre-weighed porcelain crucible. The crucible containing filtrates was put in a furnace and ignited slowly to 800°C. It was then weighed and the total percentage of SO_3 calculated from the mass of $BaSO_4$ precipitated using the following equation;

$$\% \, SO_3 = \frac{\text{Mass of precipitate x 34.3}}{\text{Mass of sample}}$$

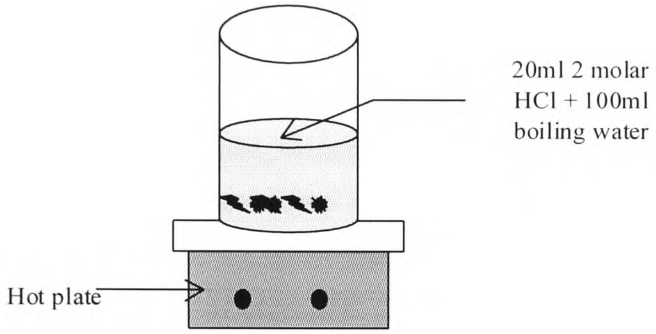


Figure 5.8: Boiling soil in HCl acid and water during sulfate analysis

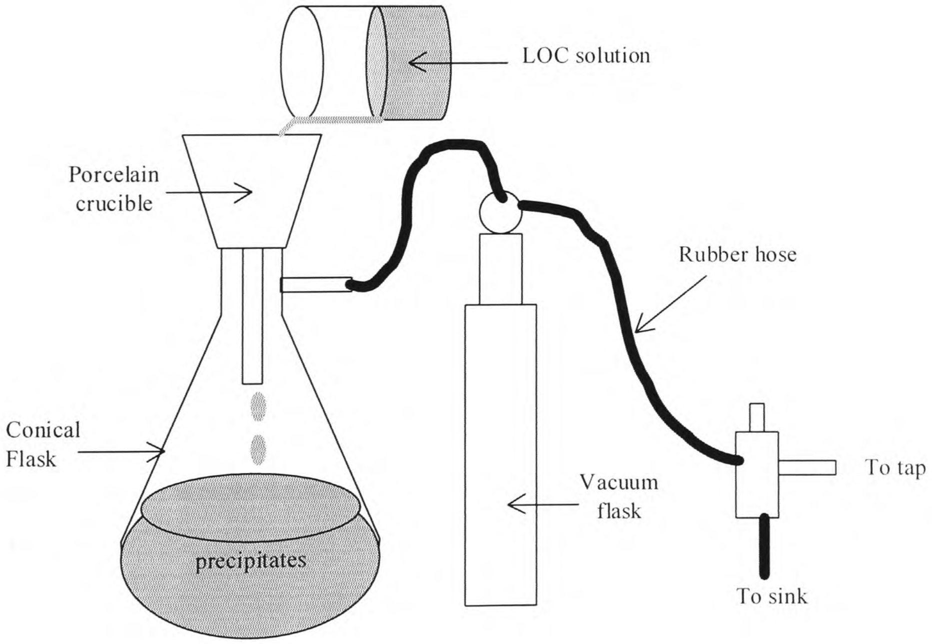


Figure 5.9: Filtration process during sulfate analysis

5.8 THERMOGRAVIMETRIC ANALYSIS (TGA)

Thermogravimetric analysis (TGA) is a thermal analysis technique for measuring the amount and rate of change in sample mass as a function of temperature and/or time. TGA can be used to characterize any material that exhibits weight change as a result of, for example, dehydration (loss of water of crystallization), dehydroxylation (loss of OH ions), oxidation (e.g. combustion of organic matter) or decarbonation (loss of CO_2).

Heating of clay results in a pattern of weight loss that is characterised by the attractive force with which water is retained within or on the clay structure at different temperatures. Similar weight losses occur on heating hydrated cement. During the hydration of cement a number of chemical changes result in the formation of various phases. These include formation of ettringite, C-S-H gel, $Ca(OH)_2$ and carbonation of $Ca(OH)_2$ to give $CaCO_3$.

On heating, each of the above phases show a weight loss over a specific temperature range due to either partial or total decomposition. The weight loss results from the loss of water (dehydration or dehydroxylation) or carbon dioxide (from decarbonation). From the thermogravimetric measurements it is therefore possible to estimate, and in some cases accurately determine, the amount of each phase which is present at different stages of the hydration process.

In thermal analysis tests the specimens are heated at a controlled rate and the weight changes can be recorded in two ways:-

(i). Percentage weight loss against temperature (TG) and/or

(ii). Rate of loss in weight against temperature (DTG), this being the derivative of the curve in (i).

A weight loss at a specific temperature produces a step in (i) and a peak (maximum) in (ii). A weight gain will also produce changes but in the opposite direction. The curves are useful in illustrating the manner in which the amount of any particular phase present changes with curing time, and also providing reliable quantitative data giving the actual amounts of particular phases present. However, this technique requires prior knowledge of the chemical composition of the individual phases and their characteristic TG and DTG curves or decomposition temperature. There are a number of factors that influence the values of the peak temperatures observed on TG plots, for example the amount of sample used in the test, the heating rate, the furnace atmosphere, the particle size of the sample and the degree of crystallinity of the compound being tested.

The major thermogravimetric effects that characterize clay soils can be observed at temperatures ranging between 60°C and 300°C (loss of adsorbed water and water of hydration) and between 450°C and 1000°C (dehydroxylation & decarboxylation). At temperatures beyond 900°C most clay minerals undergo an exothermic recrystallisation process and new crystals form from amorphous materials derived from old crystals destroyed at lower temperatures (Mitchell, 1993). However, it should be noted that the thermal behaviour of clay soils could differ considerably from that of pure clay minerals, due to the presence of various elements such as organic matter or carbonates that decompose on heating.

The TGA work was carried out using a TA Instrument 2950Hi-Res ^{TM} TGA thermogravimetric analyzer using a TA5000 Thermal Analyze Controller and software (see Figure 5.10). The heating rate was chosen as 10°C per minute within the temperature range from room temperature to 1000°C. The sample was taken from the interior of a compacted cylinder (used in the UCS tests) and dried in an oven at 40°C under silica gel and carbosorb (a carbon-dioxide-absorbing agent). After drying the sample was crushed to a powder in a Mixermill 2000. Between 8mg to 10mg of the crushed sample was ignited in a closed alumina pan. Heating was carried out in an inert environment of argon gas. Weight losses and temperature increases were plotted during the test and resulting graphs supplied the TGA weight loss curve (%) and the derivative weight loss curve (DTG). The former plots the total weight loss in percent over the temperature range to which the sample was subjected, whereas the latter shows the derivative of the weight loss with increase in temperature, resulting in a curve with pronounced peaks, thus allowing conclusions and calculations to be made as to the identity and quality of particular compounds or phases present.

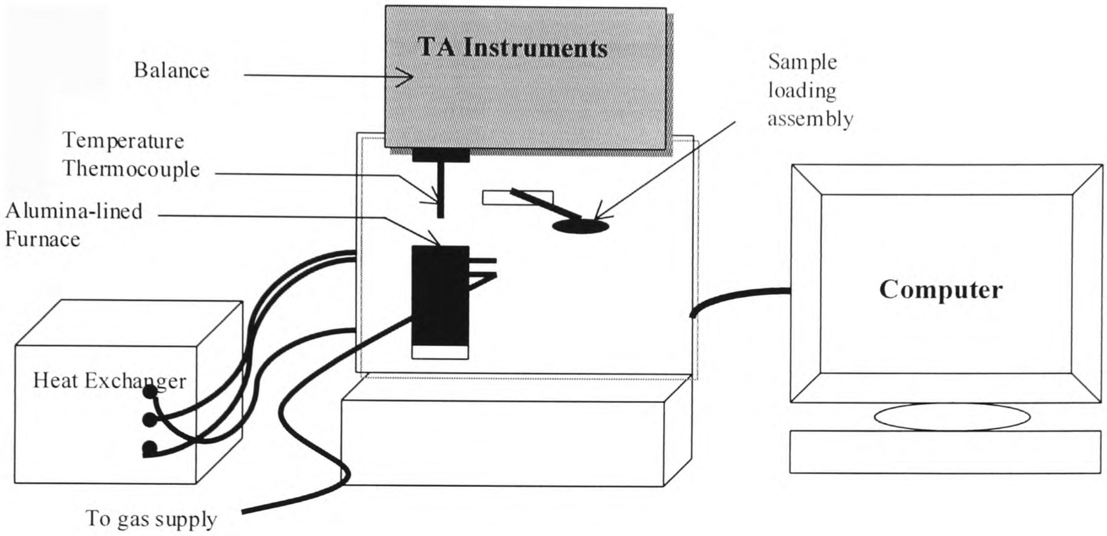


Figure 5.10: Diagram of a TGA 2950 thermogravimetric analyzer

CHAPTER 6 – EXPERIMENTAL RESULTS

In this chapter the results from experiments carried out on the Lower Oxford Clay and on Kaolinite, are presented. It deals with the interaction of these two soils with lime, and blended stabilizers incorporating WSA. It also reports on the effects of mellowing on the properties of both soils. The experimental results include initial consumption of stabilizer, consistency (Atterberg) limits tests, Proctor compaction tests, unconfined compressive strength (UCS), California bearing ratio (CBR) and linear expansion. The results of the analytical tests using sulfate (SO_3) analysis and thermogravimetric analysis (TGA) are also presented in this chapter.

6.1 INITIAL CONSUMPTION OF STABILIZER

The aim of measuring the pH is to determine the design (lime or other stabilizer) content that is necessary to achieve a pH of 12.4. This is the level of alkalinity required to release reactive cations and anions necessary for cementitious processes. It is referred to as the Initial Consumption of Stabilizer. It is usually the practice to add an additional amount of stabilizer to this initial amount, for the achievement of more long-term soil changes including strength enhancement, volume stability and other desirable long-term performance criteria. Figures 6.1-1(a) and (b) show the initial consumption of lime and WSA respectively.

In the LOC + lime mixtures (see Figure 6.1-1(a)) 2wt.% lime was needed to raise the pH to 11.8, whereas in the LOC + WSA mixtures (see Figure 6.1-1(b)) 10wt.% WSA and above was necessary to raise the pH to 11.4. Five percent WSA is clearly much less effective than even 1wt.% lime in the creation of an alkaline environment for the mixture. Infact between 10wt.%-30wt.% of WSA is necessary in order to significantly enhance the alkalinity of the LOC.

For the blended stabilizers i.e. WSA + GGBS, WSA + lime, WSA + PC, generally more than 10wt.% by dry weight of stabilizer is also required (see Figure 6.1-2) to attain the maximum achievable alkalinity necessary for the activation of the chemical reactions that are necessary to enhance the properties of the soil. For WSA-lime blends (see Figure 6.1-2(c)), high lime contents actually appear to lower the pH after one hour of mixing. This is thought to be due to high lime demand in short-term pozzolanic reactions involving WSA and lime, besides the cationic reactions with LOC.

In the case of kaolinite clay when this was mixed with lime or with WSA, about 1wt.% lime and more than 10wt.% WSA was required (see Figures 6.1-3(a) and (b)) to create a sufficiently enhanced alkaline environment. When kaolinite was mixed with the various blended stabilizers, generally more than 10wt.% stabilizer content was needed (see Figure 6.1-4) to raise the pH of the mixture to a sufficiently enhanced level. Therefore for this research it was decided that 10wt.%, 15wt.% and 20wt.% stabilizer contents would be used for stabilization and engineering property investigations.

6.1.1 Summary

1. In the LOC and kaolinite-lime mixtures, 2wt.% and 1wt.% lime respectively brought the solution to a sufficiently alkaline environment whereas in the LOC and kaolinite + blended stabilizer mixtures, (WSA+lime, WSA+PC, WSA+GGBS) more than 10wt.% stabilizer content is generally required to achieve maximum alkaline environment to enhance the property changes of the soils.

2. The pH of clay-lime mixtures falls only very slowly with age (up to 360 minutes) whereas the pH of clay-WSA mixtures falls quite significantly with age. This is clearly related to WSA hydration rather than WSA-clay reaction.

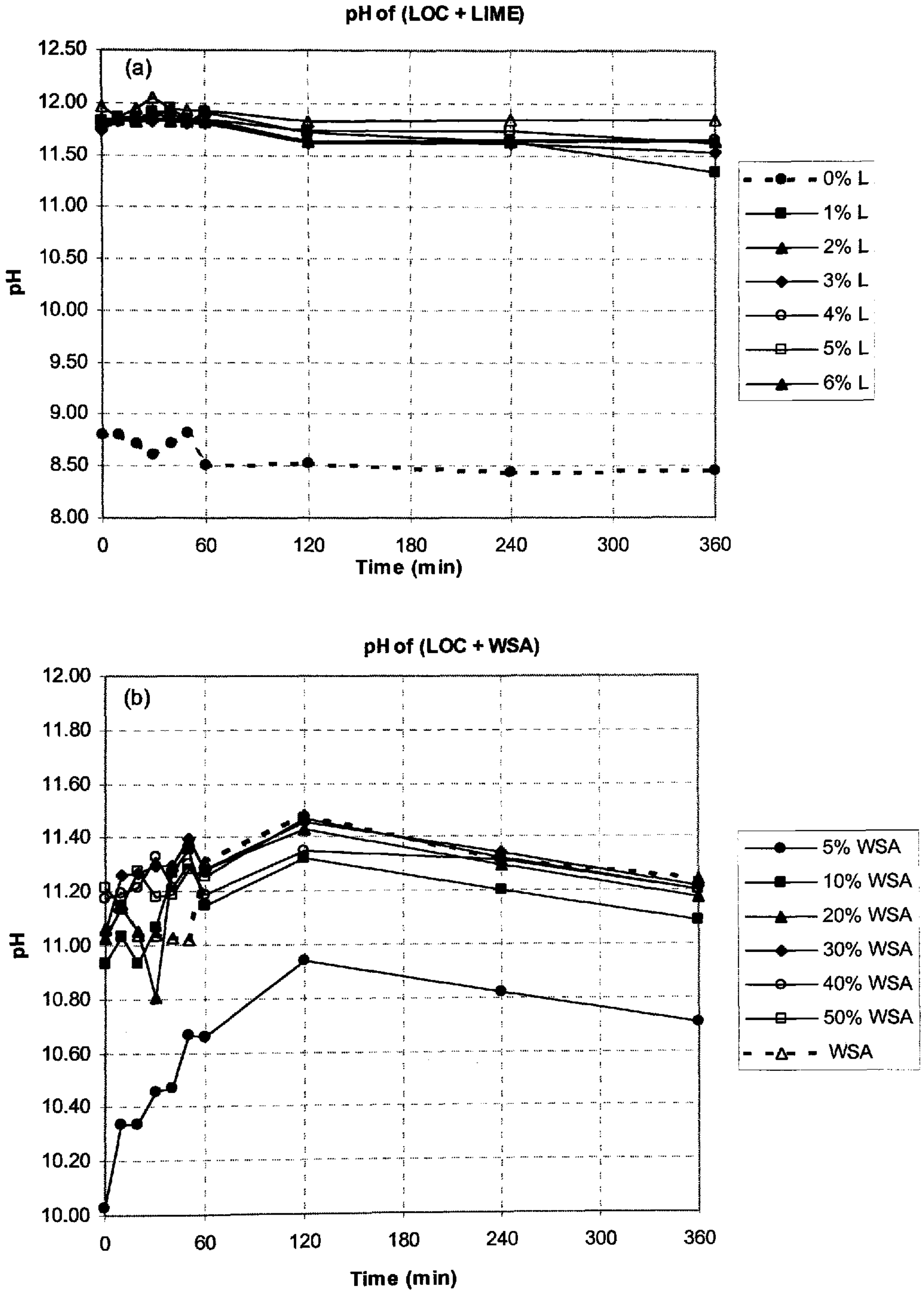


Figure 6.1-1: Initial consumption of (a) Lime and (b) WSA for LOC

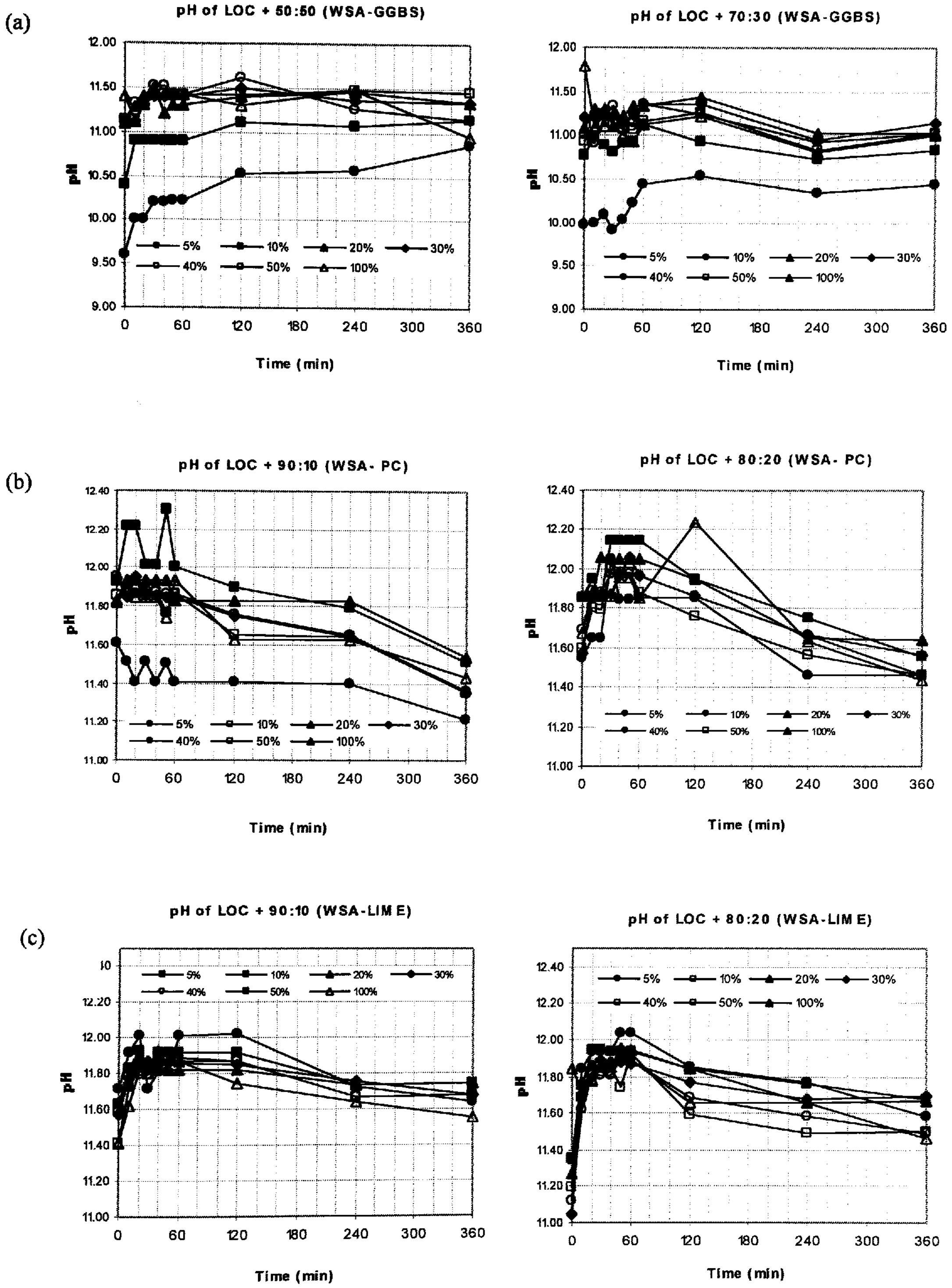


Figure 6.1-2: Initial consumption of blended stabilizers (a) WSA-GGBS (b) WSA-lime and (c) WSA-PC for LOC

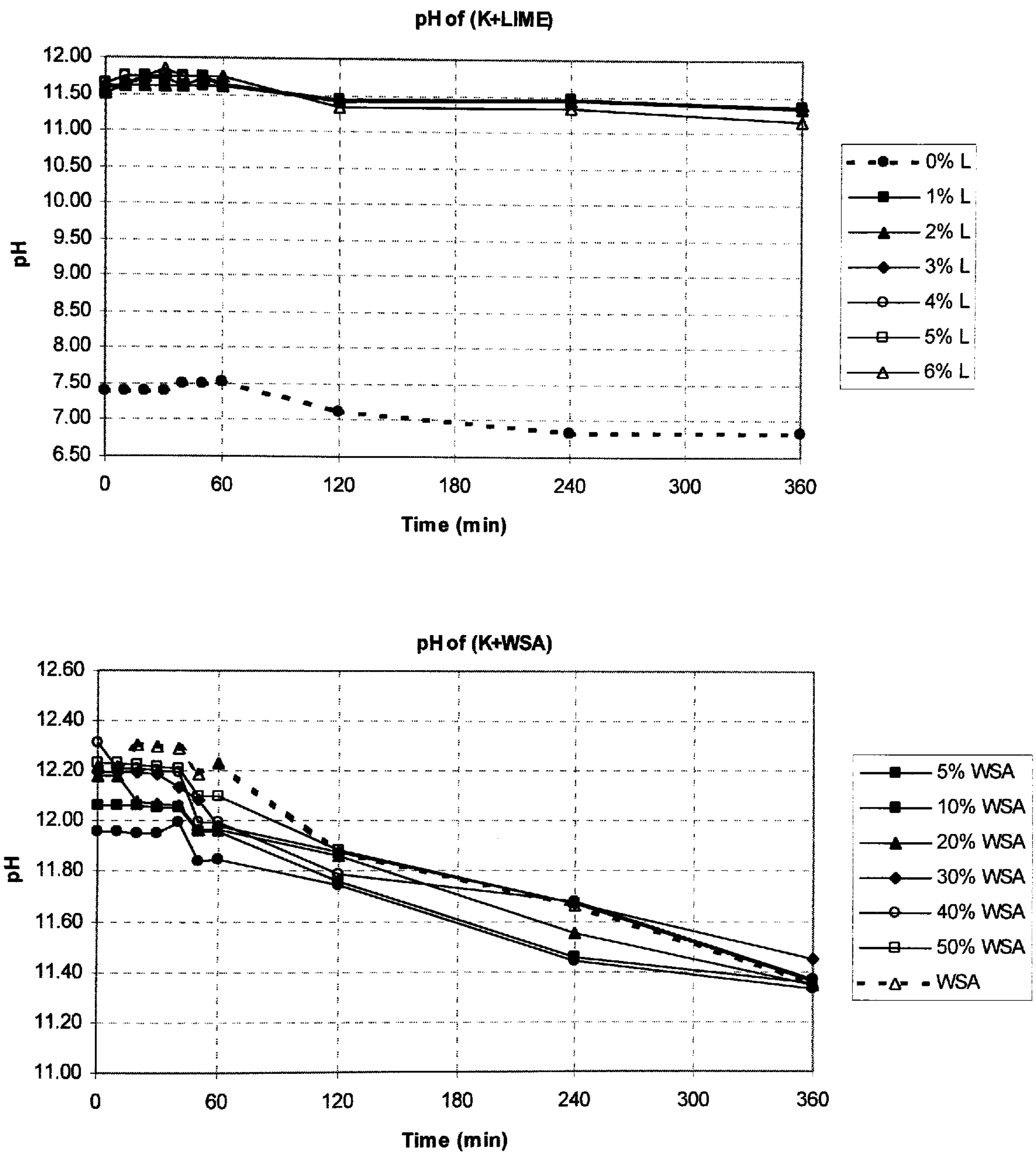


Figure 6.1-3: Initial consumption of (a) Lime and (b) WSA for kaolinite Clay

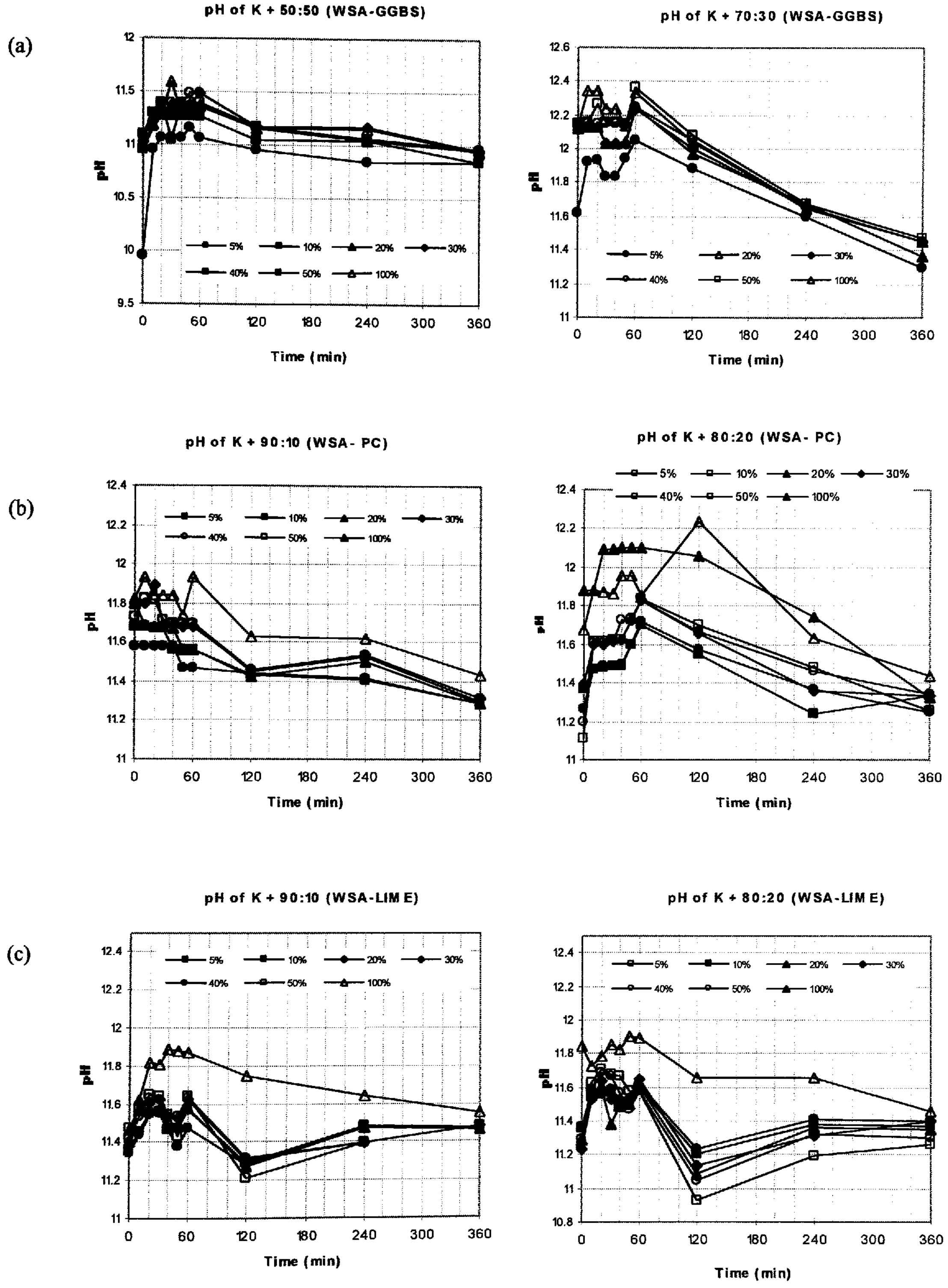


Figure 6.1-4: Initial consumption of blended stabilizers (a) WSA-GGBS (b) WSA-PC and (c) WSA-lime for kaolinite Clay.

6.2 CONSISTENCY (ATTERBERG) LIMITS

This Section presents soil mixture consistency data concerning the effects of lime, WSA and blended stabilizers incorporating WSA, on the consistency properties of LOC and kaolinite. The plasticity characteristics of soils are normally expressed in terms of their liquid limit (LL), plastic limit (PL) and plasticity index (PI) as proposed by Atterberg (1911) and as described in the British Standard (BS 1377:2:1990). Atterberg limits are expressed as the percentage of moisture by dry weight of soil commonly referred to as the moisture content of the soil. Atterberg Limits are used as index properties for the classification of soils. Certain ranges of Atterberg limits are associated with certain characteristic types of soil behaviour. However, only a general indication should be drawn from these limits, since consistency limits of clays are generally affected by their chemical environment and composition (Abdelkader, 1985).

Figures 6.2-1 to 6.2-4 illustrate the changes in Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI) of unmellowed LOC and kaolinite with increasing additions to the soil of lime (control), or of the different stabilizers all incorporating WSA.

6.2.1 Consistency (Atterberg) Limits of the LOC System

In the control system (LOC + lime) (see Figure 6.2-1(a)), the addition of a small amount of lime (t 2wt.% CaO) increases the LL of the LOC from 66% to 77%. The PL also increases steadily with the addition of lime. Further increase in lime (4wt.%, 6wt.%) results in no further increase in LL. In fact, there is a small drop in LL to 74% as the PL continues to increase up to 54% at 4wt.% lime. Overall, the PI decreases progressively with the increase in lime, resulting in an overall drop in PI from 31% to 20%. This trend on consistency of lime-stabilized LOC has also been observed by other numerous researchers, Kinuthia (1997), Veith (2000) and Thomas (2001). It has also been observed on other lime-clay mixtures, Sherwood (1993), Bell (1996) and Rogers (1997) among others. The same trend was also observed in the LOC + WSA system (see Figure 6.2-1(b)). In all the three blended stabilizer systems investigated, LOC + WSA-lime (see Figure 6.2-1(c) and (d)), WSA-PC (see Figure 6.2-2(a) and (b)) and WSA-GGBS (see Figure 6.2-2 (c) and (d)), the overall trends for LL and PL are comparable to the control system. However, in the blended stabilizer system, the PI showed a maximum within 5wt.%-10wt.% stabilizer content before decreasing with further increase in stabilizer. The lowest PI was recorded with the WSA-Lime 80:20 system (see Figure 6.2-1(d)) where the PI dropped from 31% to 12% with the addition of 30wt.% stabilizer. The least drop in PI was observed on the WSA-GGBS 50:50 system (Figure 6.2-2(d)) where the PI dropped by 3% only from 30% to 27% with the same amount of stabilizer (30wt.%). This is thought to be as a result of increased consumption of free lime by the GGBS. This is evidenced by the bigger reduction in PI by the 70:30 WSA-GGBS blend, which contains a relatively lower amount of GGBS.

The main objective of stabilizing a clay soil is to modify its properties, especially by reducing the PI. This phenomenon of reduced PI is due to a flocculation process as a result of cation exchange between lime (or other stabilizer) and the clay particles as discussed in Chapter 2. The introduction of lime modifies the consistency (Atterberg) limit of a clay soil as the calcium from the stabilizer causes flocculation, which enables the stabilized material to accommodate a greater volume of water. Further additions of stabilizer will further increase flocculation, hence increasing further the capacity to hold water. This continues up to a certain point at which the stabilizer will no longer continue to participate in flocculation due to cation saturation on the clay particle surfaces. This point, when the soil has satisfied the affinity for the stabilizer, is termed the stabilizer fixation point (Hilt and Davidson, 1960).

Previous studies have identified many trends and general characteristics of changes in soil properties due to the addition of lime. The universal effects of mixing lime with plastic soils is the reduction in plasticity. Little (1995) states that lime treatment causes a substantial reduction in the plasticity of a soil and the soil often becomes non-plastic. Laguros (1965) found that the PI of a soil was reduced from 47% to 15% with the addition of 6wt.% hydrated lime. Jan and Walker (1963) noted that the incremental reduction in plasticity decreases as the lime content increases. Other researchers have found that after approximately 2wt.% to 4wt.% hydrated lime addition, the additional effect on the plasticity of the soil is minimal (Sweeney *et al.*, 1988). Basma and Tuncer (1991) tested the plasticity of lime-treated soils at cure times of 1 hour to 28 days and found that cure time had little effect on the plasticity of lime treated soil.

6.2.2 Consistency (Atterberg) Limits of the Kaolinite System

The effects of lime and the various stabilizers incorporating WSA on the consistency of kaolinite was also studied and is illustrated in Figures 6.2-3 and 6.2-4. When kaolinite was treated with 2wt.% lime (see Figure 6.2-3(a)), its LL increased by 27%. Above 2wt.% addition of lime, the kaolinite suffered a significant reduction in LL. The reduction in the LL, along with the steady increase in the PL, produced a considerable reduction of the PI of the kaolinite when treated with lime (control). There was however an initial marked increase in PI, resulting from the initial marked increase in LL (up to 2wt.% lime).

The LL of kaolinite rose sharply with 5wt.% addition of WSA and of the blended stabilizers WSA-lime (see Figure 6.2-3(c) and (d)), WSA-PC and WSA-GGBS (see Figure 6.2-4). The LL and PL both increased steadily up to the addition of 30wt.% stabilizer content. Kaolinite stabilized by WSA-PC (see Figure 6.2-4(a) and (b)), behaved similarly to that stabilized with WSA-Lime, although the reduction in LL upon increased addition of stabilizer is less pronounced. However, when WSA-GGBS blends of stabilizer were used (see Figure 6.2-4(c) and (d)), the LL continued to increase with increase in stabilizer content up to 30wt.%. The PL also increased steadily with addition of stabilizer, with a great increase being observed initially with the addition of 5wt.% stabilizer. Taken together the changes in the LL and PL resulted in a general decrease in PI for the 70:30 WSA-GGBS blend. However, the 50:50 WSA-GGBS blend resulted in an overall increase in the PI, suggesting that WSA-GGBS blends of high GGBS content may present mixing problems in practice. The detailed data on the results of the consistency limits are presented in Table B.1-1 in Appendix B.1

6.2.3 Summary

1. The addition of lime to both LOC and kaolinite increased the liquid limit and plastic limit of the clay. The plasticity indices of both clays were generally reduced by the addition of lime.

2. The blended stabilizers WSA-lime, WSA-PC and WSA-GGBS and WSA produced overall increases in both the liquid limit and plastic limit (the former two blends more steeply then the latter) for both LOC and kaolinite, thus generally reducing their plasticity indices, except for the 50:50 WSA-GGBS blend.

3. Compared with the effects on LOC, kaolinite stabilized with all the various stabilizers recorded higher increases in both liquid limit and plastic limit values. The relative changes resulted in smaller reductions in the plasticity indices of the kaolinite systems compared to the LOC systems (see Table B.1-1 in Appendix B.1).

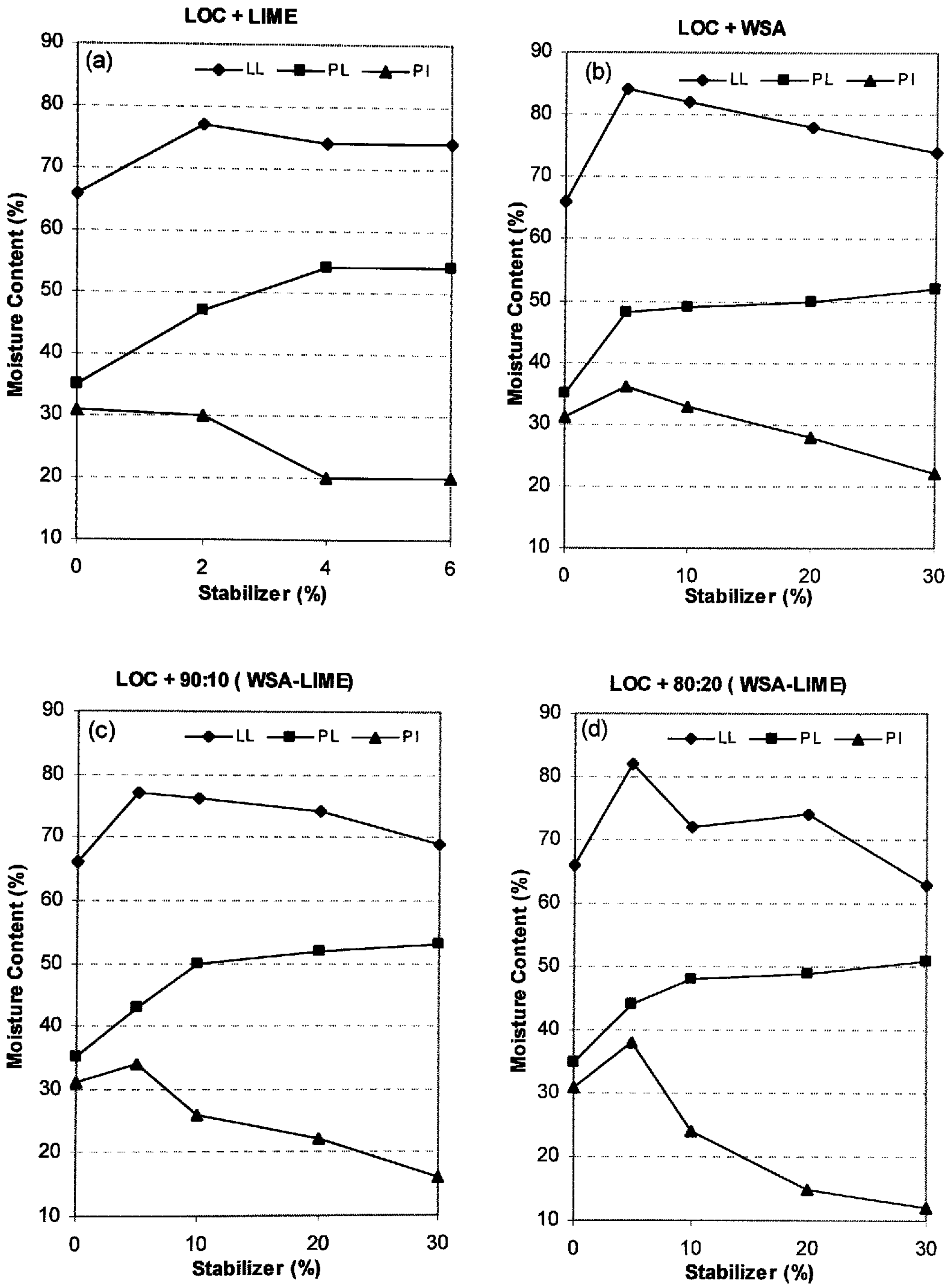
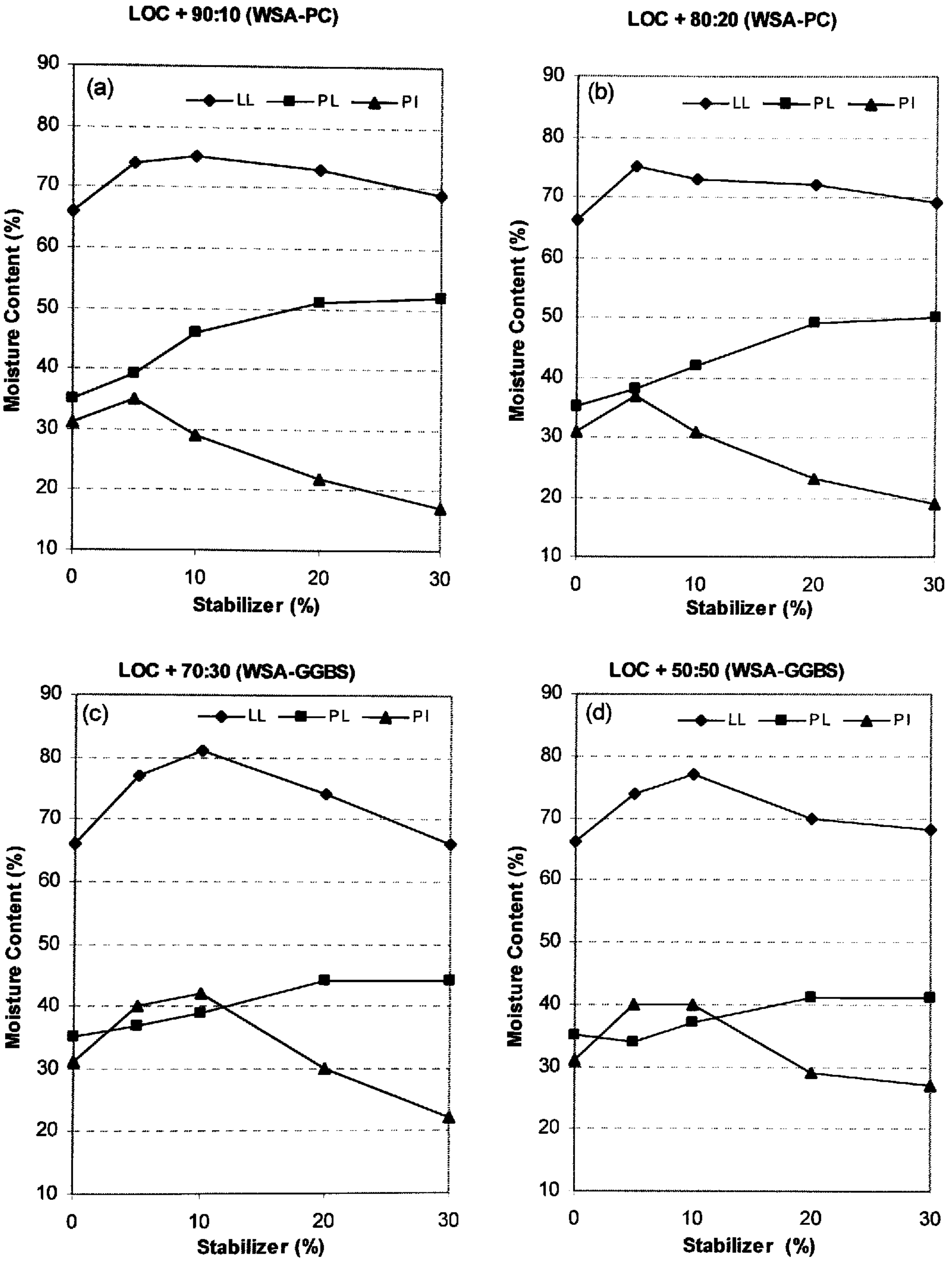


Figure 6.2-1(a-d): Consistency (Atterberg) Limits of LOC upon addition of various stabilizers


LOC + 90:10 (WSA-PC)

LOC + 80:20 (WSA-PC)

LOC + 70:30 (WSA-GGBS)

LOC + 50:50 (WSA-GGBS)

Figure 6.2-2(a-d): Consistency (Atterberg) Limits of LOC upon addition of various stabilizers.

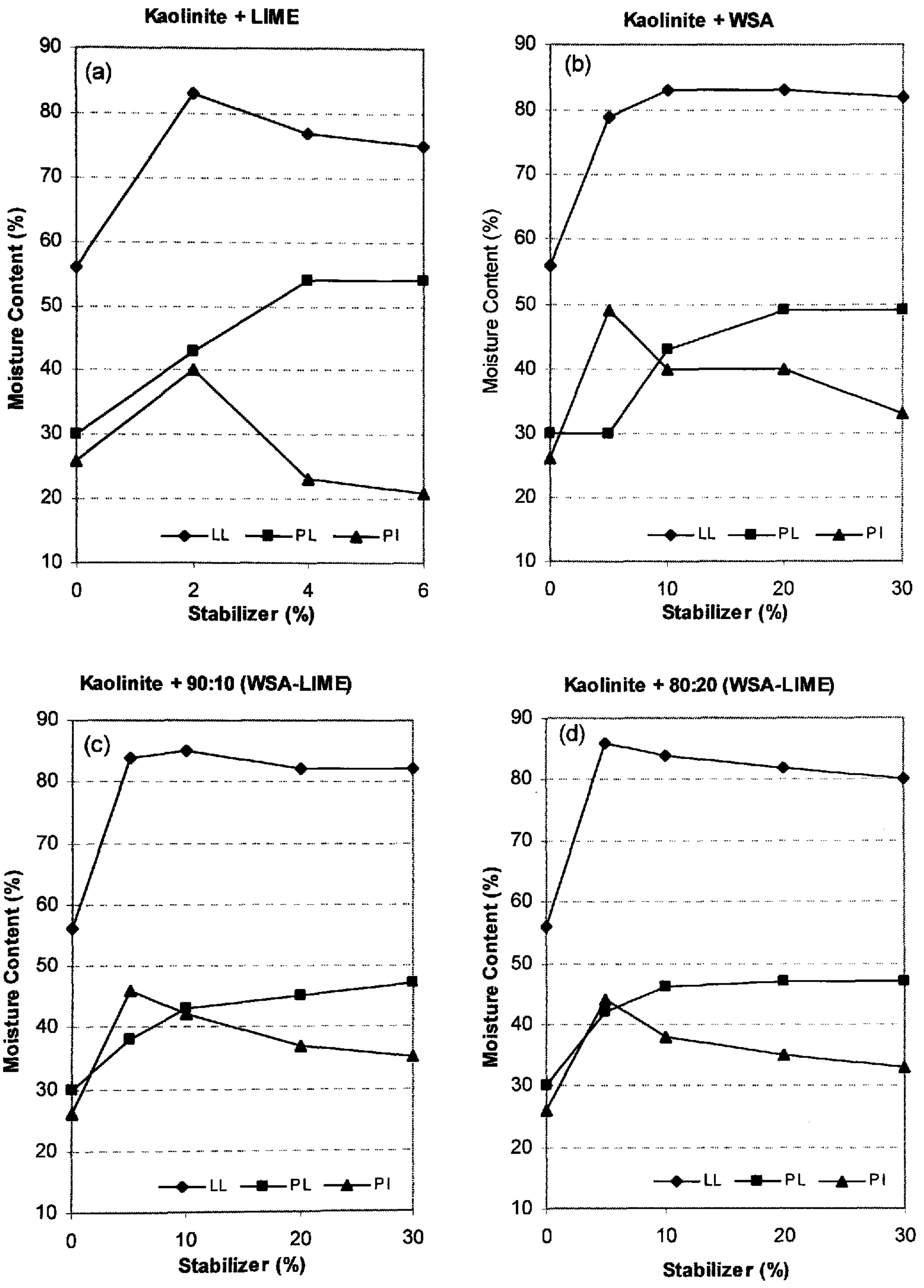


Figure 6.2-3(a-d): Consistency (Atterberg) Limits of kaolinite upon addition of various stabilizers

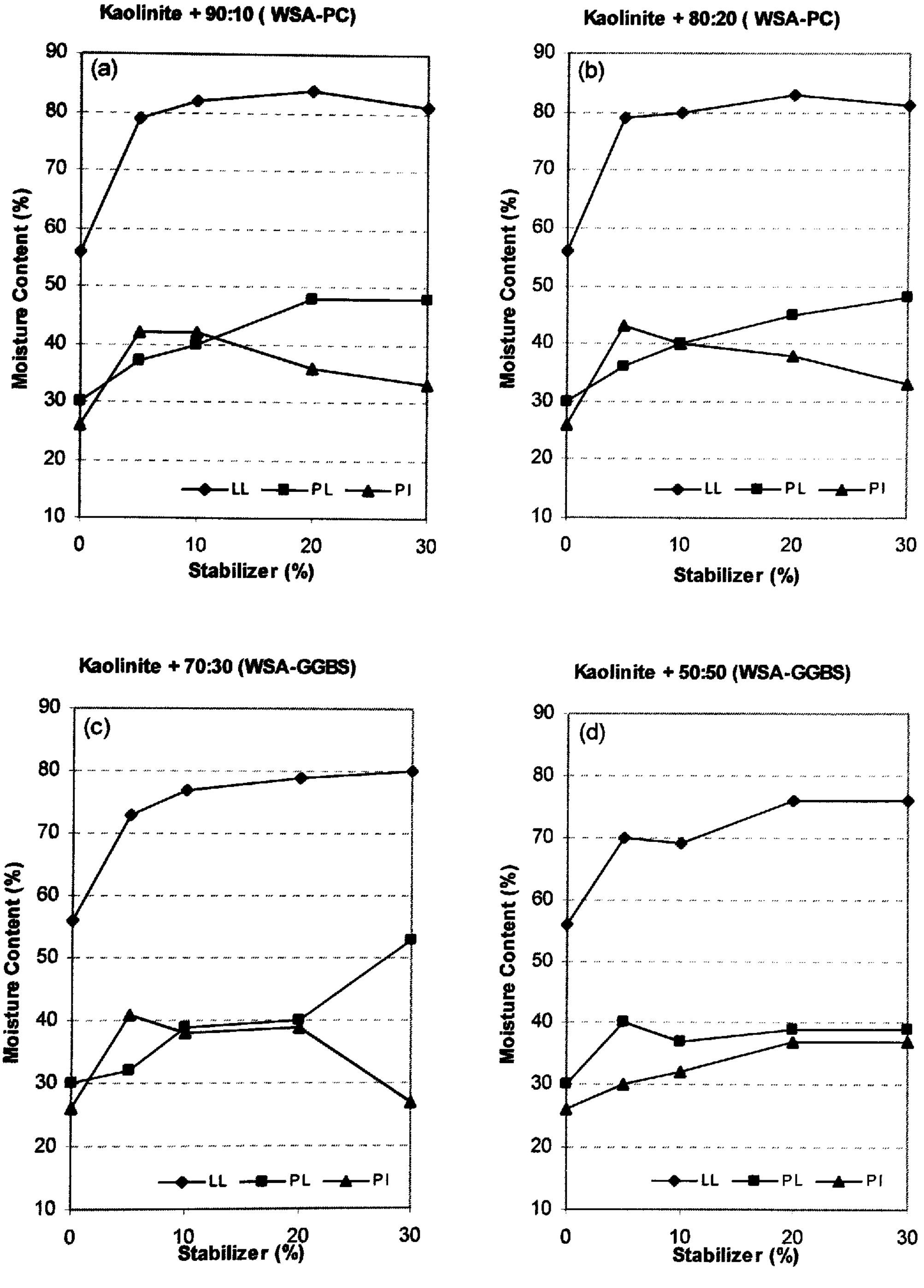


Figure 6.2-4(a-d): Consistency (Atterberg) Limits of kaolinite upon addition of various stabilizers

6.3 PROCTOR COMPACTION TESTS

Standard Proctor compaction tests were conducted on unmellowed samples to establish the approximate maximum dry density (MDD) and optimum moisture content (OMC) to be adopted during the preparation of specimens for the unconfined compressive strength and linear expansion tests. It was also to establish the variation in Proctor compaction properties upon the addition of various amounts of lime, WSA and various blended stabilizers incorporating WSA.

6.3.1 Compaction Characteristics of the Lower Oxford Clay (LOC) System

Figures 6.3-1(a) and (b) show the compaction characteristics (MDD and OMC respectively) of the unmellowed LOC after additions of 2wt.%, 4wt.% and 6wt.% lime (CaO) as a control. Addition of lime to LOC had the overall effect of lowering the MDD and raising the OMC. Addition of 2wt.% lime, resulted in the greatest decrease in MDD from 1.36 Mg/m^3 – 1.29 Mg/m^3, and of the greatest increase in OMC from 25% – 29%. Further increase in lime content (4wt.% and 6wt.%) steadily decreased the MDD and increased the OMC by a smaller degree. The causitive phenomena are the increased flocculation and agglomeration caused by cation exchange between the lime and the clay particles in the LOC. Elsekelly (1987) studied the compaction characteristics of an Egyptian soil treated with lime. He observed a decrease in MDD with an increase in the lime content. He suggested that this might be as a result of the replacement of soil particles by lime particles in a given volume as the lime particles partially filled the voids between the soil particles and prevented them from coming into a closer state of packing. However, this is not the principal mechanism in operation. The amounts of lime involved (2wt.%-6wt.%) are not sufficient to explain the large decrease in MDD. Thus, the principal mechanism suggested by numerous researchers is flocculation, which results in fewer particles per unit volume. The increase in OMC is due to the increase in void volume, and in the specific surface area that has to be lubricated, resulting in LOC-lime mixtures requiring more moisture to achieve the acquired MDD. Some researchers have also suggested that the formation of cementitious products immediately after mixing clay soil with lime (for example ettringite if sulfate is present in the system), may cause resistance to compaction and reduce the density (Wild *et al.*, 1993).

The WSA-lime stabilizers of 90:10 and 80:20 blending ratios produced very similar trends to those of the control (lime) system. Addition of the stabilizers to LOC at 10wt.%, 15wt.% and 20wt.% contents, reduced the MDD and increased the OMC (see Figure 6.3-2(a) and (b)). WSA alone generally lowered the MDD and increased the OMC of the stabilized LOC greater than using the blended stabilizer (WSA-lime), particularly at the lower stabilizer contents -10wt.%. This is strange, as the presence of lime in the blended stabilizers would have been expected to decrease rapidly the MDD and increase the OMC. However, it has been suggested previously (see section 6.1) that WSA-lime reactions may lower the sum total of the cationic exchange capacity of the different stabilizer (WSA & lime) used separately.

The effects of stabilizing LOC with blended WSA-PC are illustrated in Figures 6.3-3 (a) and (b). The trend is similar to LOC stabilized with lime (control) and with both WSA-lime blends. At all stabilizer contents the 90:10 WSA-PC ratio, produced the lowest MDD and highest OMC values, followed by the WSA stabilized LOC and lastly the 80:20 WSA-PC stabilized LOC. At 20wt.% stabilizer content, the MDD of the system dropped from 1.36 Mg/m^3 to about 1.20 Mg/m^3 and the OMC rose from 25% to 32%. The fact that the blend higher in PC (80:20) did not show expected behaviour (lower MDD and higher OMC than the WSA or the 90:10 WSA-PC blend) is again attributable to WSA-lime activity, this time the lime coming from the hydration of PC.

The effects of WSA-GGBS on the compaction characteristics of LOC has also been studied and illustrated in Figures 6.3-4(a) and (b). The blended WSA-GGBS stabilizer produced the same trends as the other stabilizers already discussed. The WSA-GGBS blend of 50:50 ratio produced the highest MDD and the lowest OMC at all stabilizer contents (10wt.%, 15wt.% and 20wt.%), followed by that at 70:30 ratio and lastly the WSA alone. The results appear to suggest that it is the WSA that is responsible for the reduction in MDD and increase in OMC, this ability being progressively eroded by the addition of GGBS. It is well established that GGBS consumes lime during its hydration (as no traces of free lime were identified in the GGBS used in this study).

Thus the free lime in WSA (~5wt.%), which is responsible for the changes in the Proctor properties, is gradually consumed by the increase in GGBS in the WSA-GGBS stabilizer blends, from 100:0 through 70:30 to 50:50 ratios. In all blended stabilizer systems that were investigated, the OMC plot is a mirror image of the MDD plot. The MDD and OMC curves for LOC stabilized with WSA-lime and WSA-PC systems are similar, but the curves in the WSA-GGBS system are much more separated from each other. At 20wt.% stabilizer content, WSA-GGBS stabilized LOC recorded the highest MDD values of 1.28 Mg/m^3 (at 50:50 blend) and 1.24 Mg/m^3 (at 70:30 blend) compared to WSA-PC system, that recorded 1.21 Mg/m^3 (at 80:20 blend) and 1.18 Mg/m^3 (at 90:10 blend) and compared to the WSA-lime system that marked 1.21 Mg/m^3 (at 80:20 blend) and 1.20 Mg/m^3 (at 90:10 blend). The WSA-GGBS system also recorded the lowest OMC values compared to both WSA-PC and WSA-lime blended stabilizers. These effects are best illustrated by Figure 6.3-5, which shows a combined plot of MDD and OMC for all the stabilizers investigated. It is hypothesised that the WSA-GGBS system will be better for strength development (but poor workability) as a result of lower volume of voids, whereas the other two systems will be better for soil modification resulting in reduced density and higher OMC (hence better workability).

6.3.2 Compaction Characteristics of the Kaolinite System.

Figures 6.3-6(a) and (b) illustrate the MDD and OMC values of the unmellowed kaolinite stabilized with 2wt.%, 4wt.% and 6wt.% lime (CaO). The trends are very similar to those of the LOC system (see Figure 6.3-1). Addition of 2wt.% lime lowered the MDD drastically from 1.43 Mg/m^3 to 1.33 Mg/m^3 and increased the OMC from 22% to 29%. Further addition of lime (4wt.% and 6wt.%) resulted in a less drastic decrease in the MDD and increase in the OMC.

Similar patterns were observed for the WSA stabilized kaolinite and with the kaolinite stabilized with the WSA-blended stabilizer - WSA-lime, WSA-PC and WSA-GGBS - as illustrated in Figures 6.3-7 to 6.3-9. The graphs illustrate that the OMC values of kaolinite are generally increased and the MDD values reduced with increasing stabilizer content in the various systems. This is due to increased flocculation and agglomeration caused by cation exchange between the lime and the clay particles of the kaolinite.

The combined effects of all the various stabilizers on the Compaction properties of kaolinite (MDD and OMC) are illustrated in Figures 6.3-10(a) and (b). As with the LOC system (see Figure 6.3-5) the WSA-GGBS stabilizers produced least reduction in MDD and the smallest increase in OMC compared with for example WSA or WSA-PC blended stabilizer. The reasons for this are similar to those advanced for the LOC system.

6.3.3 Summary

1. The addition of lime to both LOC and kaolinite systems lowers the MDD values and increases the OMC values.

2. The introduction of WSA or any of the blended stabilizers WSA-lime, WSA-PC and WSA-GGBS to LOC or kaolinite clearly increases the OMC and lowers the MDD values of the two clay soils.

3. Generally the OMC of kaolinite stabilized with all the blended stabilizers is lower than in the LOC system.

4. The MDD values of the LOC and kaolinite systems stabilized with blended WSA-GGBS stabilizer are higher and the OMC are lower compared to stabilization with the WSA-lime and WSA-PC blended stabilizers. This is perhaps an indication of a larger difference in available lime between 70:30 and 50:50 WSA-GGBS, compared to between 90:10 and 80:20 in both WSA-lime and WSA-PC blends.

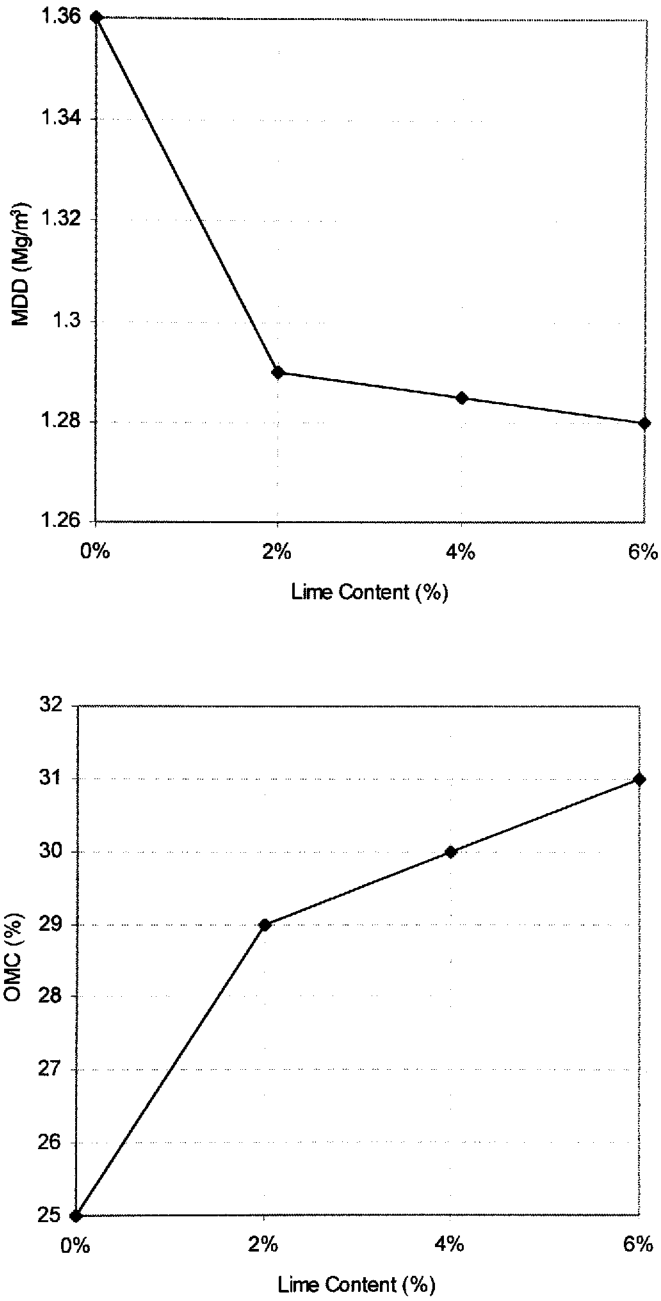


Figure 6.3-1: (a) Maximum Dry Density (MDD) vs. Stabilizer Content, and (b) Optimum Moisture Content (OMC) vs. Stabilizer Content of Lower Oxford Clay (LOC) stabilized with quicklime (CaO).

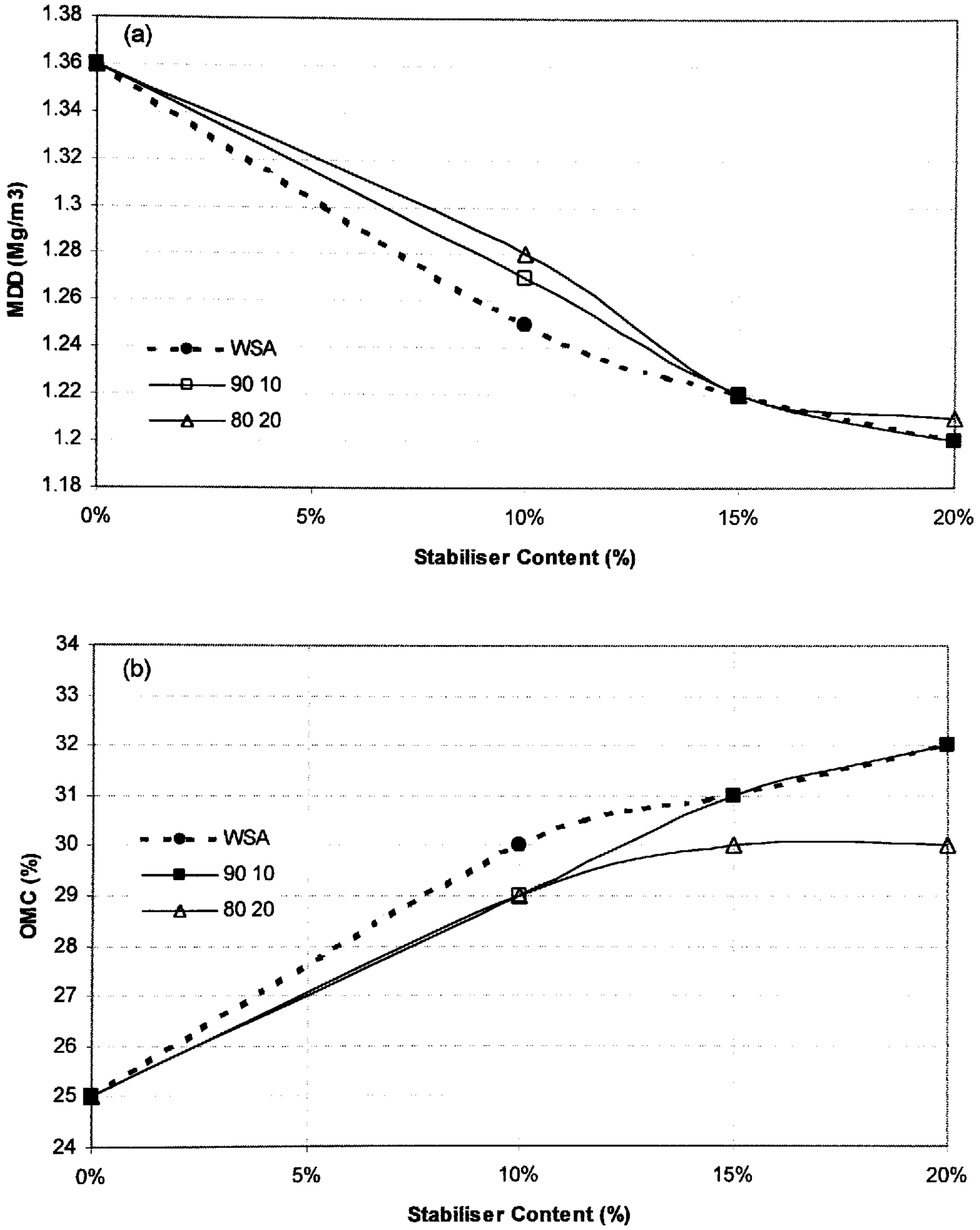


Figure 6.3-2: (a) Maximum Dry Density (MDD) vs. Stabilizer Content, and (b) Optimum Moisture Content (OMC) vs. Stabilizer Content of LOC stabilized with WSA and WSA-lime blends (at 90:10 and 80:20 blending ratios).

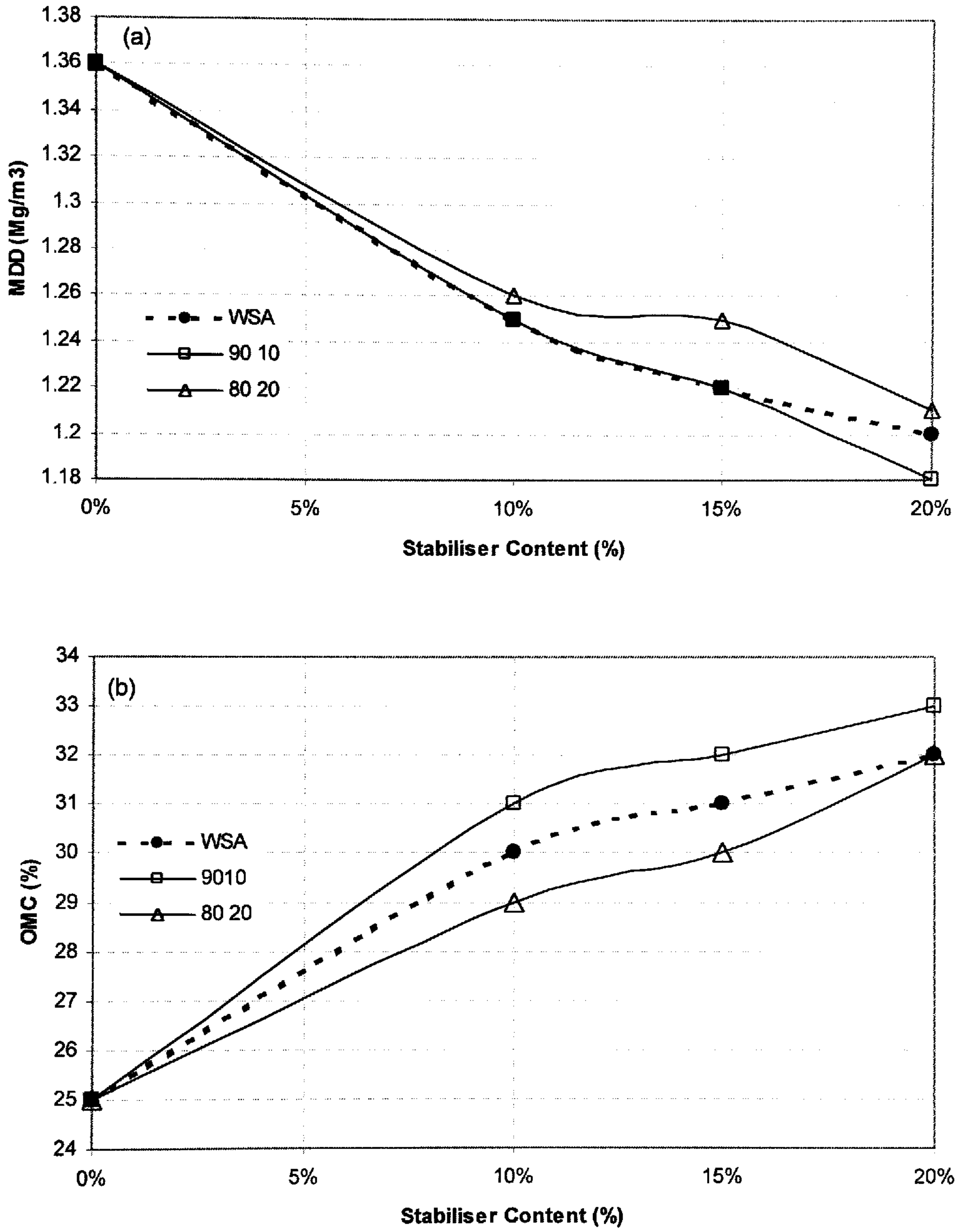


Figure 6.3-3: (a) Maximum Dry Density (MDD) vs. Stabilizer Content, and (b) Optimum Moisture Content (OMC) vs. Stabilizer Content of LOC stabilized with WSA-PC blends (at 90:10 and 80:20 blending ratios).

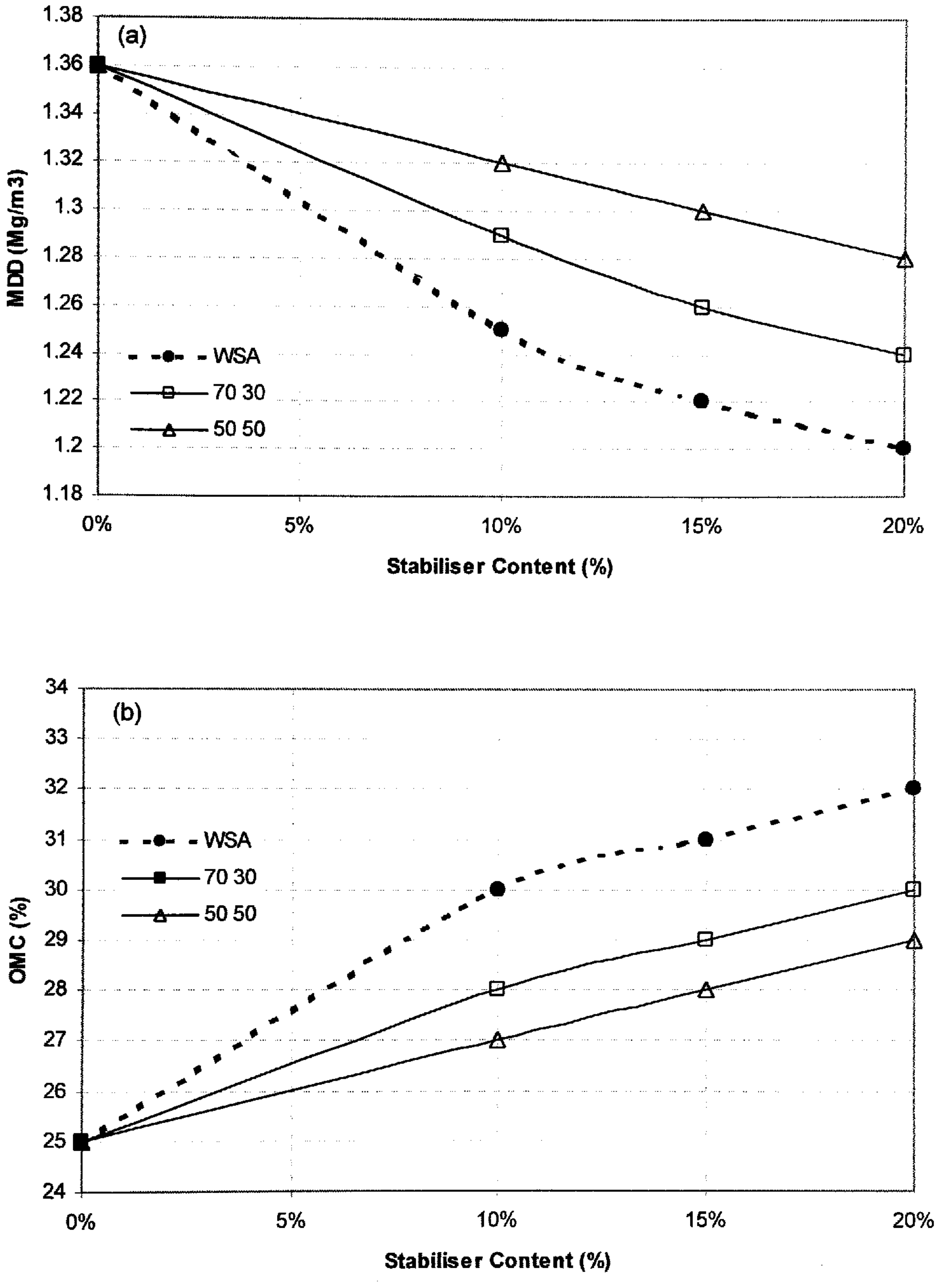


Figure 6.3-4: (a) Maximum Dry Density (MDD) vs. Stabilizer Content, and (b) Optimum Moisture Content (OMC) vs. Stabilizer Content of LOC stabilized with WSA-GGBS blends (at 70:30 and 50:50 blending ratios).

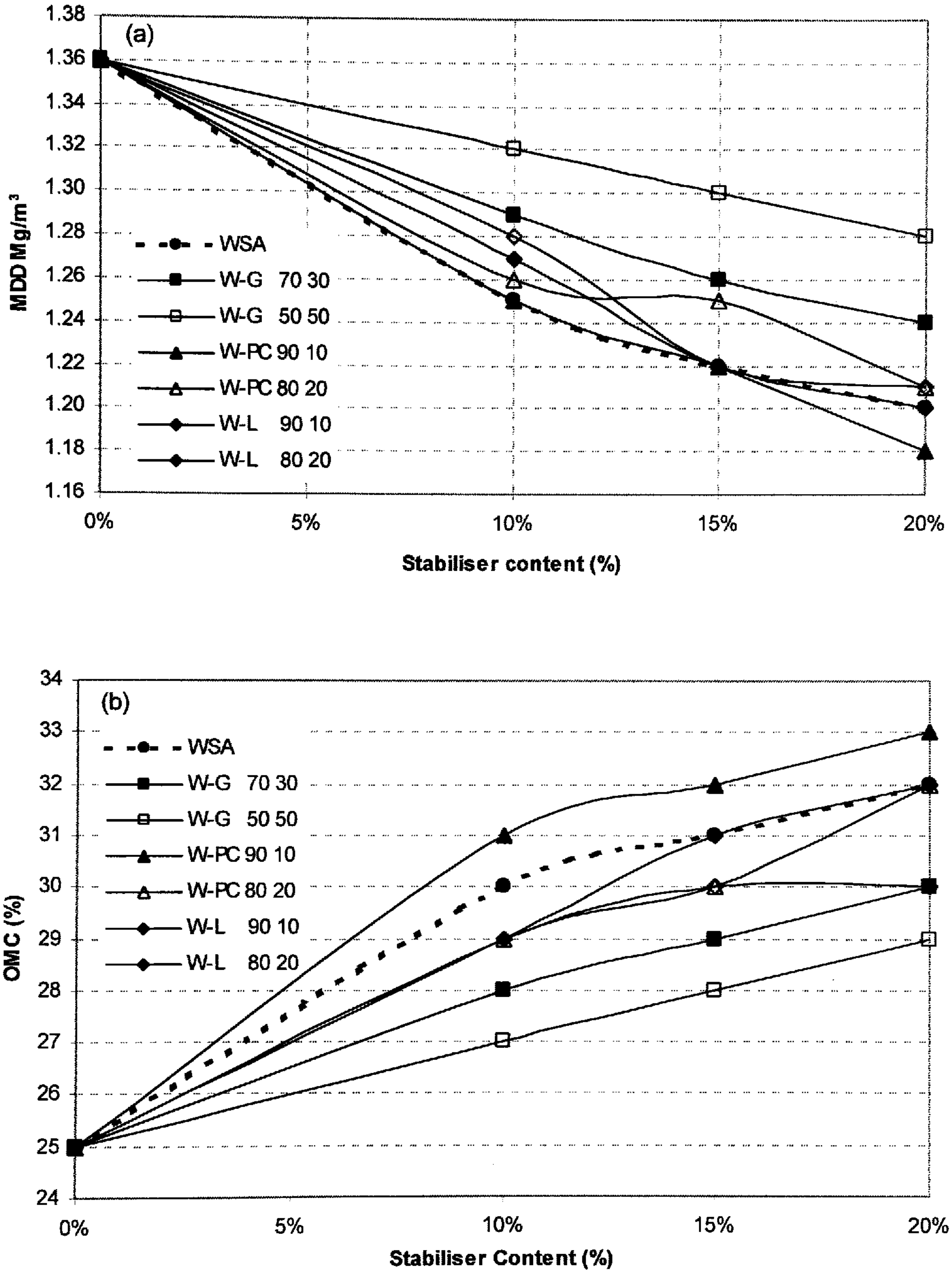


Figure 6.3-5: (a) Maximum Dry Density (MDD) vs. Stabilizer Content, and (b) Optimum Moisture Content (OMC) vs. Stabilizer content for Lower Oxford Clay (LOC).

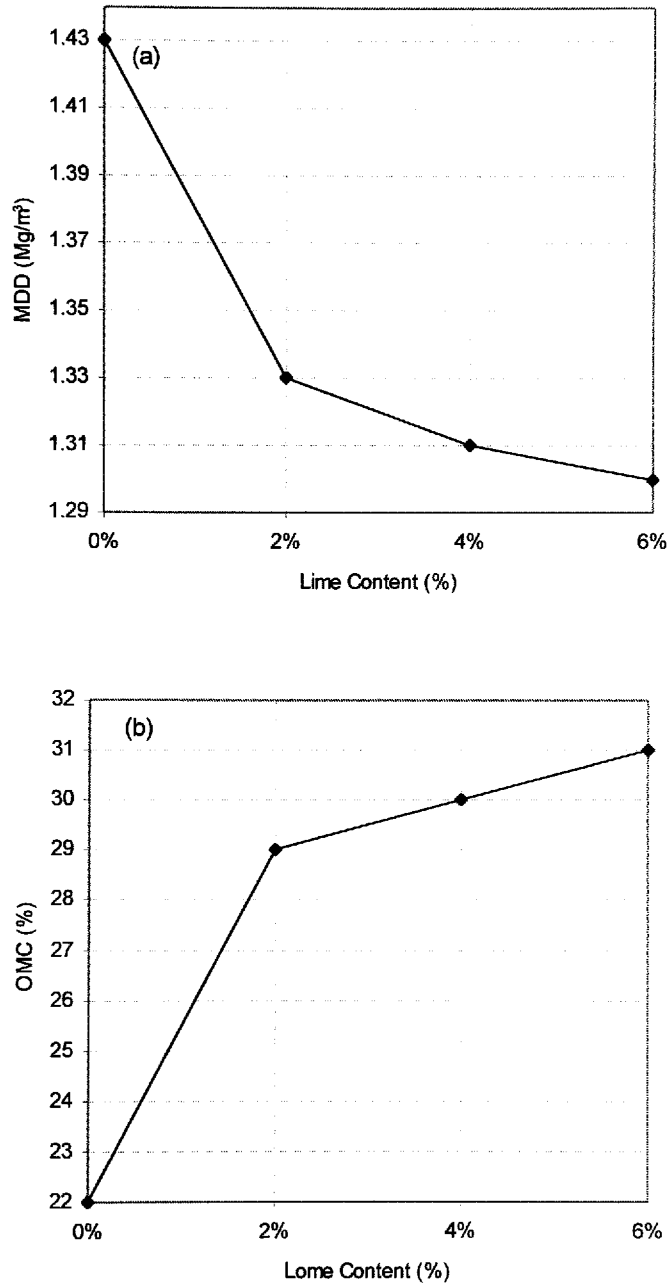


Figure 6.3-6: (a) Maximum Dry Density (MDD) vs. Stabilizer Content, and (b) Optimum Moisture Content (OMC) vs. Stabilizer Content of kaolinite stabilized with quicklime (CaO).

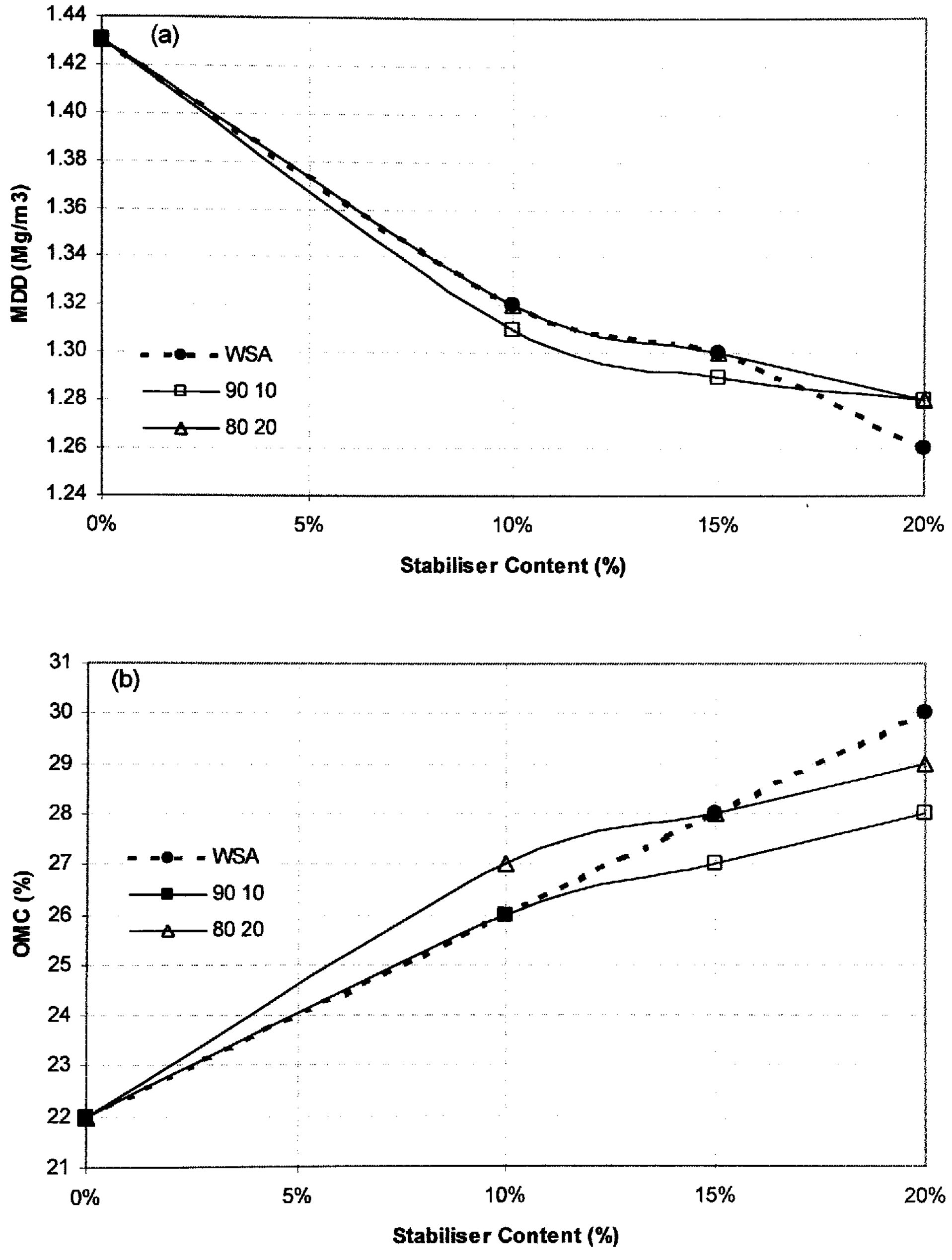


Figure 6.3-7: (a) Maximum Dry Density (MDD) vs. Stabilizer Content, and (b) Optimum Moisture Content (OMC) vs. Stabilizer Content of kaolinite stabilized with WSA-lime blends (at 90:10 and 80:20 blending ratios).

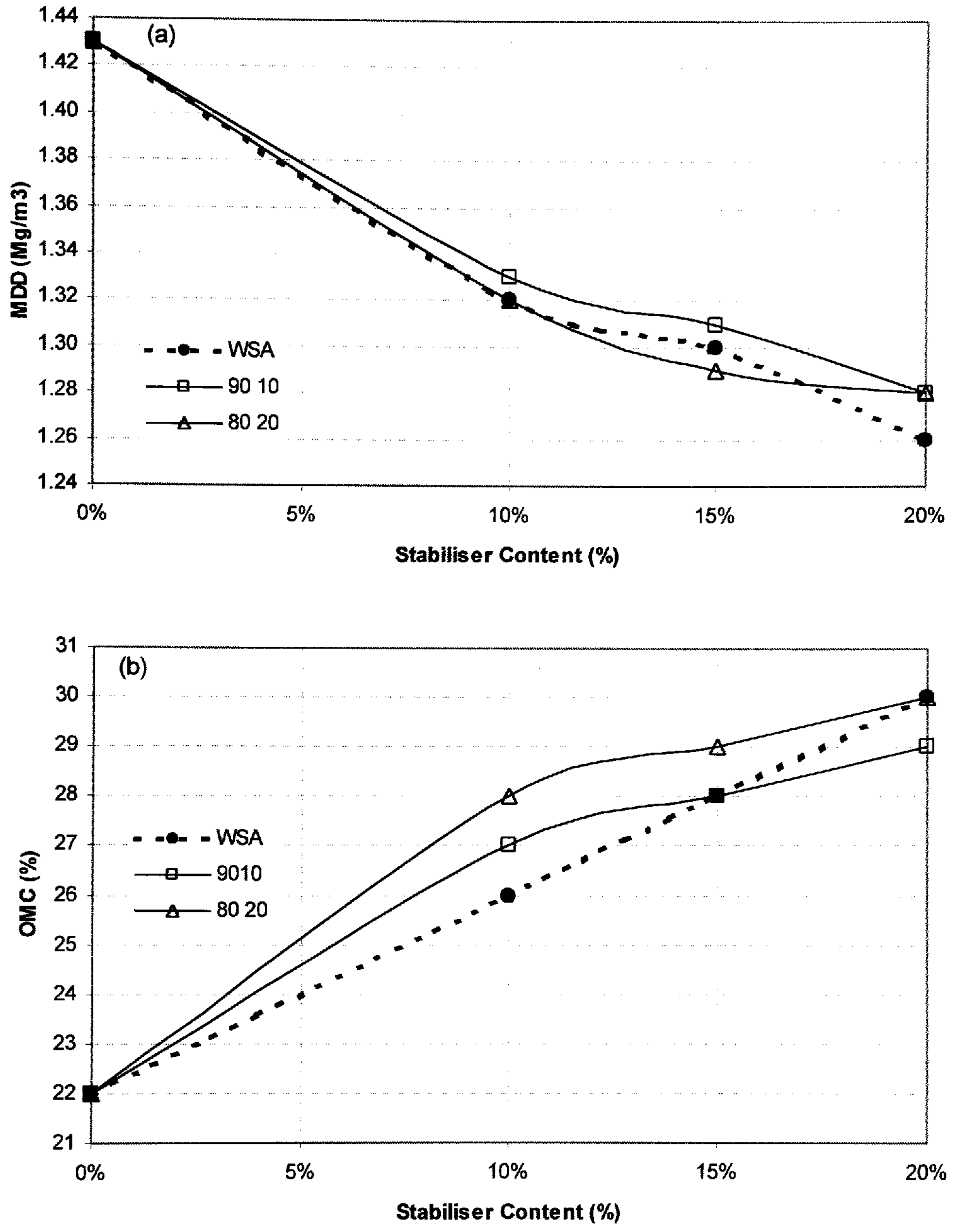
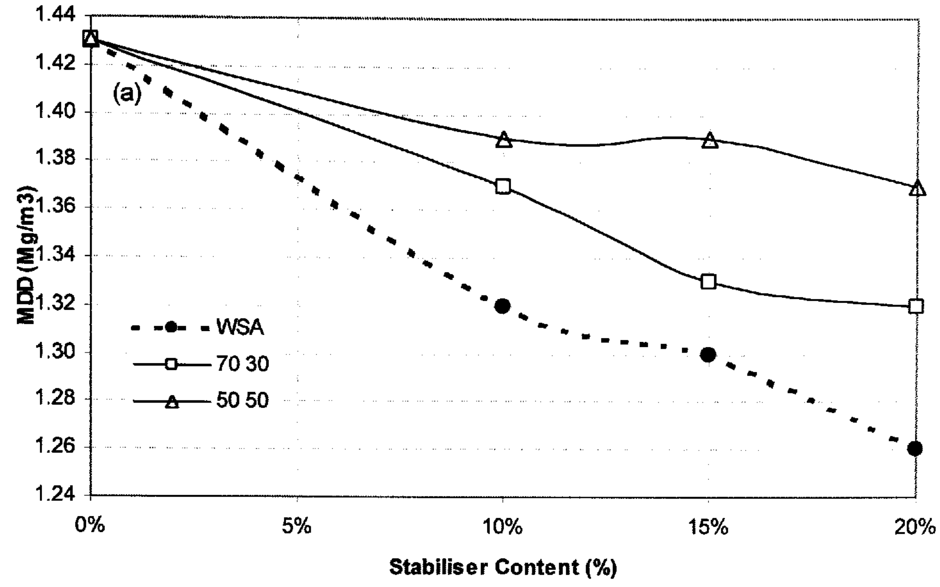


Figure 6.3-8: (a) Maximum Dry Density (MDD) vs. Stabilizer Content, and (b) Optimum Moisture Content (OMC) vs. Stabilizer Content of kaolinite stabilized with WSA-PC blends (at 90:10 and 80:20 blending ratios).



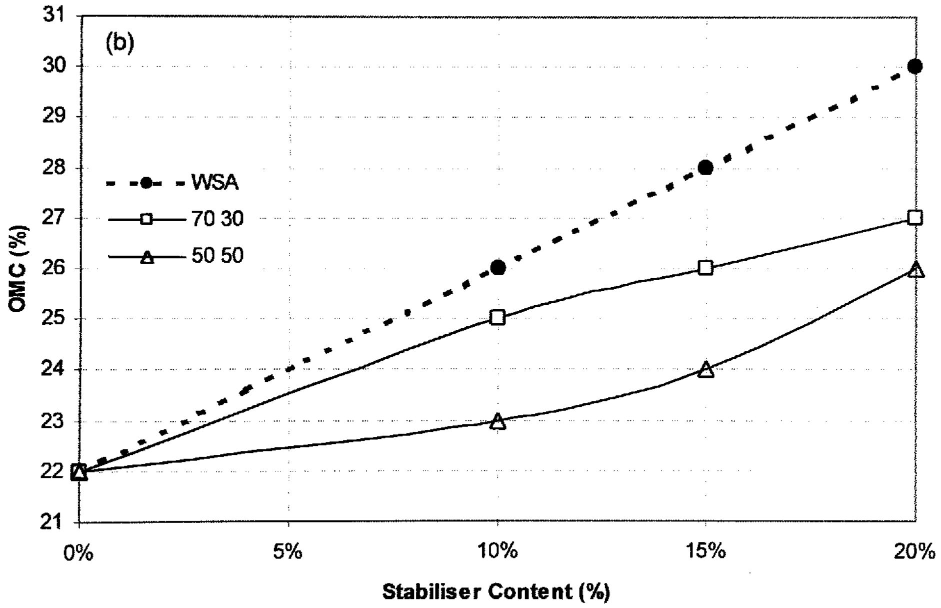


Figure 6.3-9: (a) Maximum Dry Density (MDD) vs. Stabilizer Content, and (b) Optimum Moisture Content (OMC) vs. Stabilizer Content of kaolinite stabilized with WSA-GGBS blends (at 70:30 and 50:50 blending ratios).

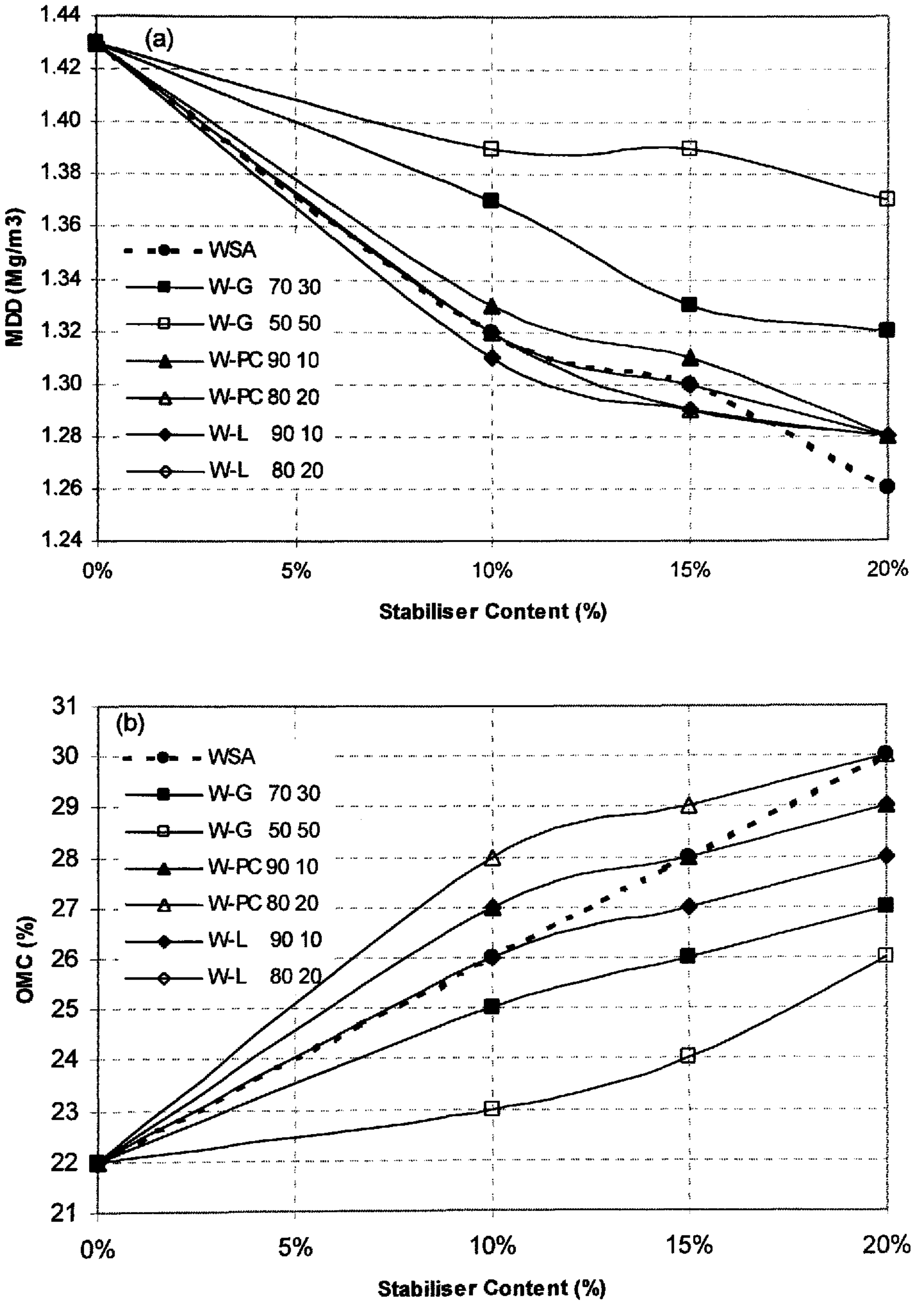


Figure 6.3-10: (a) Maximum Dry Density (MDD) vs. Stabilizer content, and (b) Optimum Moisture Content (OMC) vs. Stabilizer content for kaolinite.

6.4 UNCONFINED COMPRESSIVE STRENGTH (UCS) TESTS

The effects of various blended stabilizers (all utilising WSA) on the strength characteristics of LOC and kaolinite were studied. The UCS test, as described in the British Standard (BS 1377-7:1990), was used to assess the strength development of these soils incorporating varying ratios and contents of blended stabilizer and for various curing periods. Five curing periods were employed - 7, 28, 90, 180 and 365 days. Cylinders of dimensions 50mm in diameter and 100mm in height were made from both the unmellowed (i.e compacted immediately after mixing) and mellowed (compacted 3 days after mixing) soil-stabilizer mixtures. They were compacted at their approximate maximum dry density and optimum moisture content. As explained in section 5.4 it is not possible to compact each blend at its actual MDD and OMC, due to the wide ranges of these parameters. Three samples were tested for each of the blended soil mixtures and the mean value taken.

6.4.1 Unconfined Compressive Strength of the LOC System

Figure 6.4-1(a) shows the unconfined compressive strength (UCS) of the Lime-stabilized LOC for both mellowed and unmellowed conditions. It is evident that the overall performance of the unmellowed stabilized material is superior to that mellowed for 3 days, particularly beyond 90 days of curing. At the low lime level of 2wt.%, there is no significant improvement in the strength development with increase in curing time for both the mellowed and unmellowed conditions. Small lime addition allows only for modification, with little or no stabilization as there is insufficient lime to produce significant amounts of cementing reaction products. Lime availability is more critical in the mellowed material where the break-up of the material after 3 days of mellowing means that more lime is required to re-establish bonding. Thus, the unmellowed material shows marginally higher strength values, relative to the mellowed material, throughout the one year of moist curing. This is because in the unmellowed condition, both material modification and some cementation take place without interference. For the material stabilized with relatively higher lime levels of 4wt.% and 6wt.%, there is a marked improvement in strength upon prolonged moist curing, for both mellowed and unmellowed systems. However after moist curing beyond 180 days the unmellowed material shows superior strength development relative to the mellowed material. This indicates that in the unmellowed material sufficient unreacted lime is still present to provide continued reaction and cementation. This may not be the case in the mellowed material due to, among other reactions, some carbonation during mellowing. The unmellowed material with 6wt.% lime recorded the highest strength of 1,064 kN/m^2. It is possible to achieve higher strength with increased lime content but this is expensive and also has the effect of producing increased expansion on soaking, which is unacceptable.

Figure 6.4-1(b) shows the UCS of the WSA-stabilized LOC for both mellowed and unmellowed conditions. At all WSA levels (10wt.%, 15wt.%, and 20wt.%), there is no noticeable improvement in the strength development of the mellowed material with increasing curing time. This situation is similar to that with the Lime-stabilized LOC at 2wt.% lime, suggesting that the amount of lime available to the mellowed WSA-LOC systems (see Figure 6.4-1(b)) is rather low at all the three WSA contents. Interestingly, for the unmellowed material, there is significant and rapid increase in strength with WSA levels of above 10wt.% after 28 days of moist curing. The unmellowed material stabilized with 10wt.% WSA did not show a similar strength increase with curing time, although it recorded a higher UCS relative to the mellowed material. In both lime-LOC and WSA-LOC systems, it is apparent that the depletion of lime during the mellowing stage has a profound and long-term impact on both increase and rate of increase in strength. It is also apparent that in the unmellowed condition, WSA performs better than lime, at the content levels investigated.

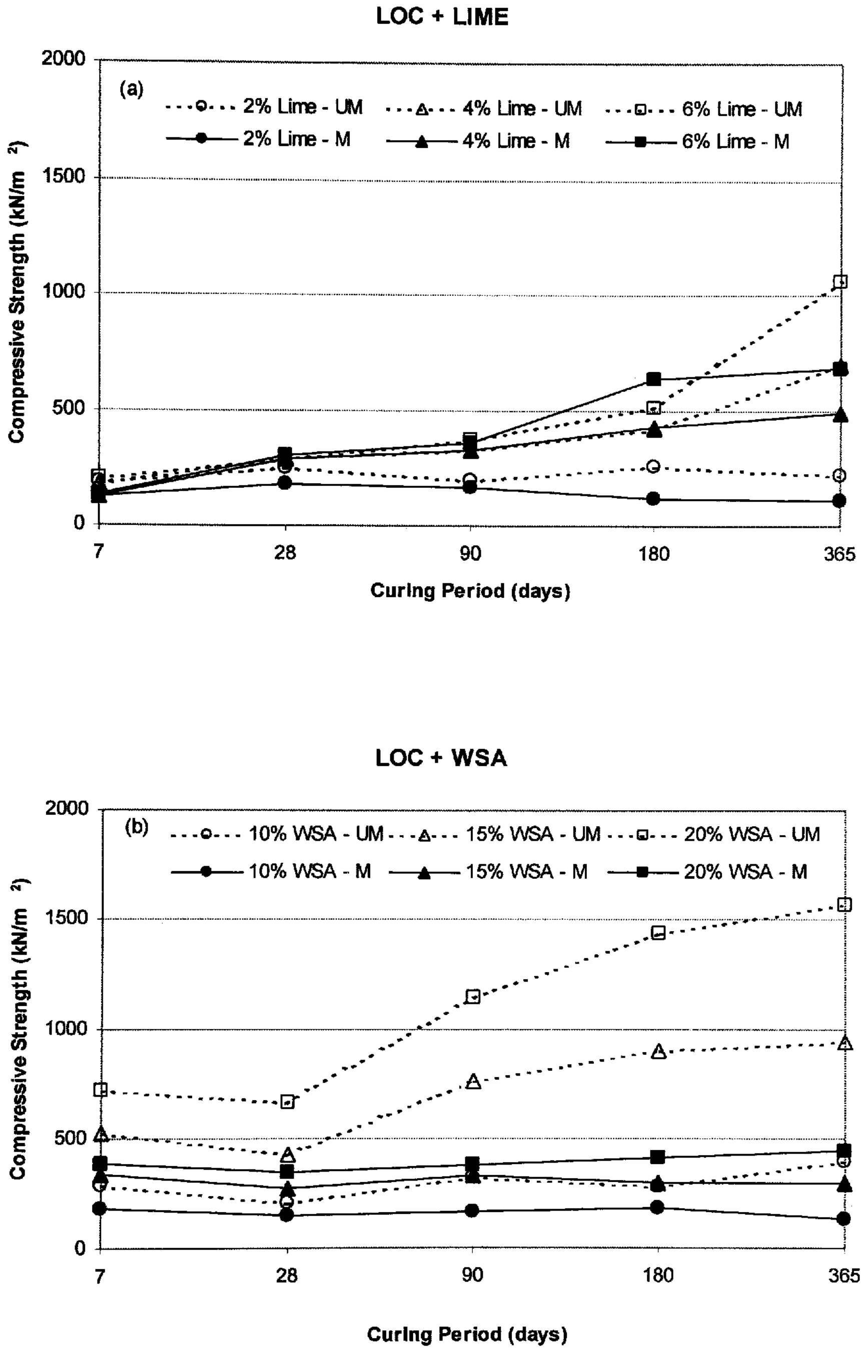


Figure 6.4-1: Compressive Strength vs. curing period of cylinders in both mellowed and unmellowed conditions for (a) Lime-stabilized LOC and (b) WSA-stabilized LOC.

Figures 6.4-2(a) and (b) illustrate the strength development when both WSA and lime are blended together to stabilize the LOC. Two different blends of WSA-lime were investigated, a 90:10 (WSA:lime) blend (see Figure 6.4-2(a)) and a blend relatively higher in lime content (80:20 (WSA:lime) Figure 6.4-2(b)). In conformity with results discussed on lime and WSA systems, the unmellowed stabilized material performs better than the mellowed one.

In contrast to the case when WSA is used on its own, by blending WSA with lime as seen in Figure 6.4-2(a) and (b), the mellowed material shows some slight strength increase with curing time, particularly at the higher dosage levels of the blended stabilizer (15wt.% and 20wt.%). The best strength improvement was recorded on the unmellowed material stabilized with the blend richer in WSA (90:10). The highest UCS value of 2,337 kN/m^2 was recorded in the unmellowed sample with 20wt.% stabilizer content. With respect to strength increase there is apparently no advantage of increasing the lime content from the 90:10 (WSA:lime) ratio to the 80:20 ratio.

The effects of blending WSA with PC, rather than with lime, are illustrated in Figures 6.4-3(a) and (b). The blending ratios were identical to those used in the WSA-lime blends (i.e. at 90:10 and 80:20 (WSA:PC)). The benefits on strength enhancement of blending WSA with PC are very similar to, but marginally greater than, those of blending WSA with lime. There is however one noticeable difference between the WSA-lime and WSA-PC systems, in that unlike in the WSA-lime blends, there is enhancement of strength development of the mellowed material when the level of PC is increased in the WSA-PC blends from 90:10 to 80:20. Increased amounts of stabilizer content again produce greater strength enhancement as exhibited by both the WSA-lime and WSA-PC systems especially when the samples were cured for 365 days. It is also noticeable that for the unmellowed samples of WSA-PC and WSA-lime stabilized LOC, the rapid strength enhancement was observed after 90 days of curing at all the stabilizer combinations used.

Figures 6.4-4(a) and (b) illustrate the effects of blending WSA with GGBS, a readily available material in the UK, as a by-product from steel manufacture. In a previous research on the use of WSA-GGBS blends as binder in concrete (Kinuthia *et al.*, 2001; Veerapan et al., 2003), a 50:50 (WSA:GGBS) blend was observed to show optimal strength and durability performance. This is the basis of the 50:50 (WSA:GGBS) ratio used in the current investigation. To improve on the economics of this blend for soil stabilization, a blend with a higher proportion of the WSA (i.e. a 70:30 (WSA:GGBS) ratio) was also investigated. Stabilizer content levels adopted were identical to those of the other WSA blends using lime or PC (i.e. 10wt.%, 15wt.% and 20wt.%). As with the WSA blends containing PC or lime, the WSA-GGBS blends also showed better strength development when the stabilized material was compacted without mellowing. The strength development was comparable with that achieved with WSA-lime and WSA-PC blended stabilizers, with the 50:50 blend showing better performance on the mellowed material particularly upon moist curing beyond 90 days (see Figure 6.4-4(b)). In the entire LOC stabilization system, the highest long-term (365 days) strength was shown by the unmellowed WSA-GGBS system (2,883 kN/m^2). Comparing the two WSA-GGBS blends, the only major advantage of the 50:50 blend is on its performance in the mellowed condition.

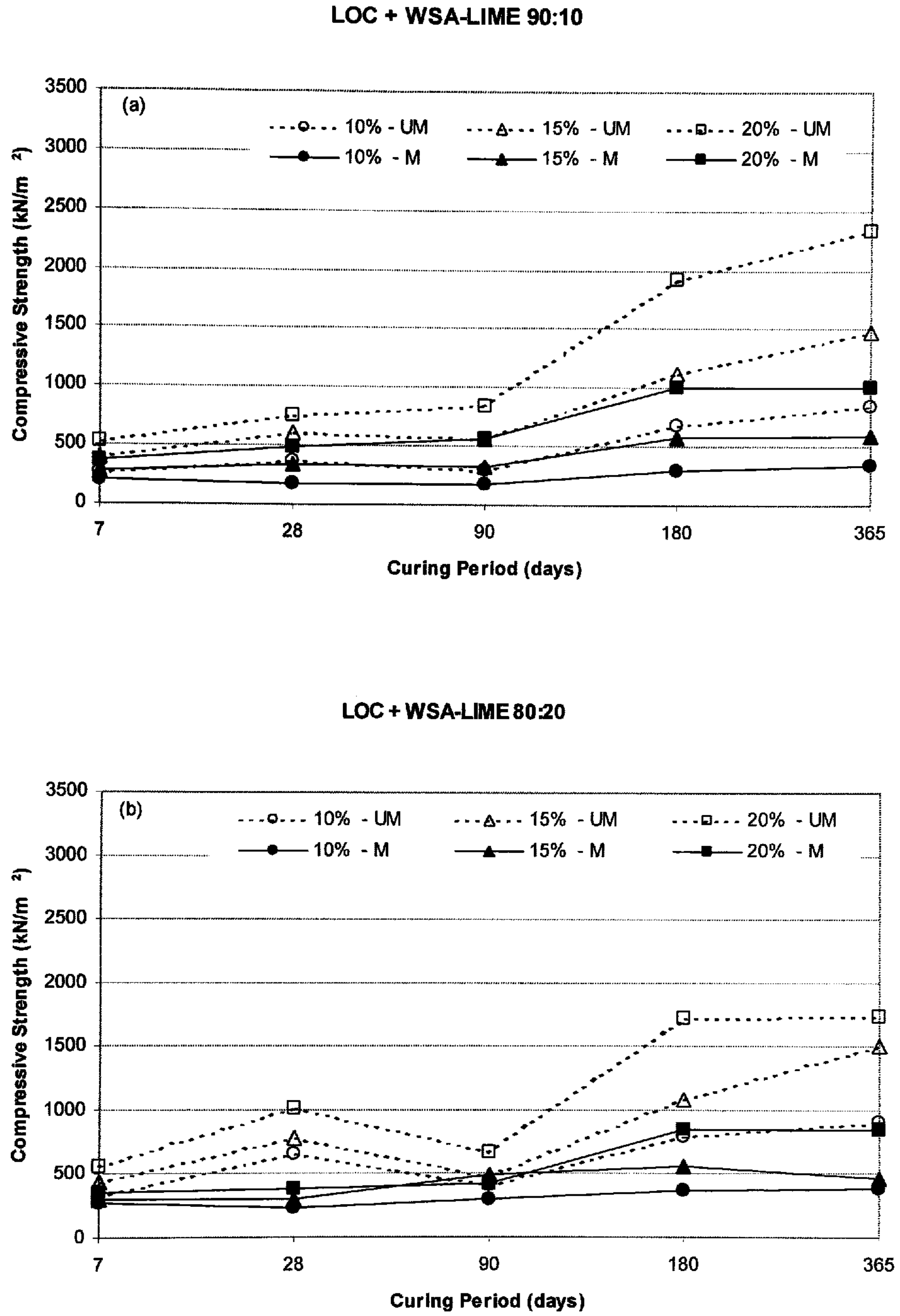
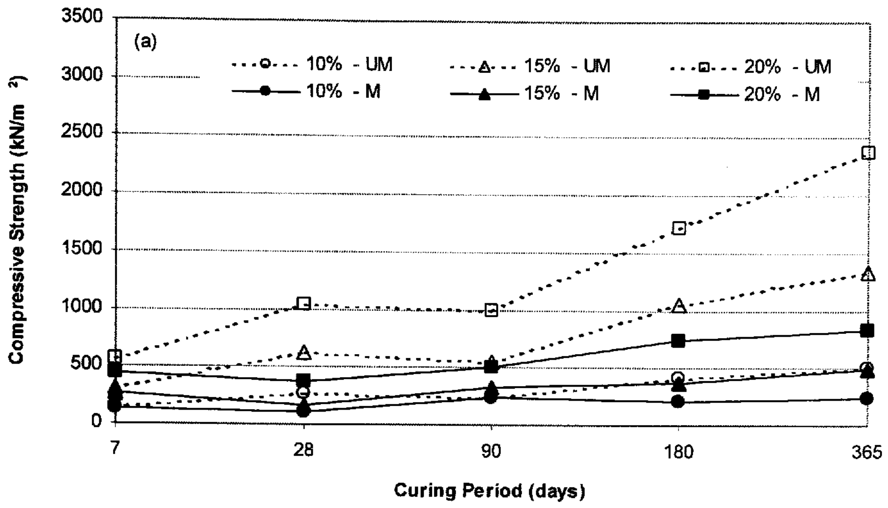


Figure 6.4-2: Compressive Strength of LOC stabilized with two WSA-lime blends (a) at 90:10 WSA:lime and (b) at 80:20 WSA:lime, for the mellowed and the unmellowed conditions.

LOC + WSA-PC 90:10



LOC + WSA-PC 80:20

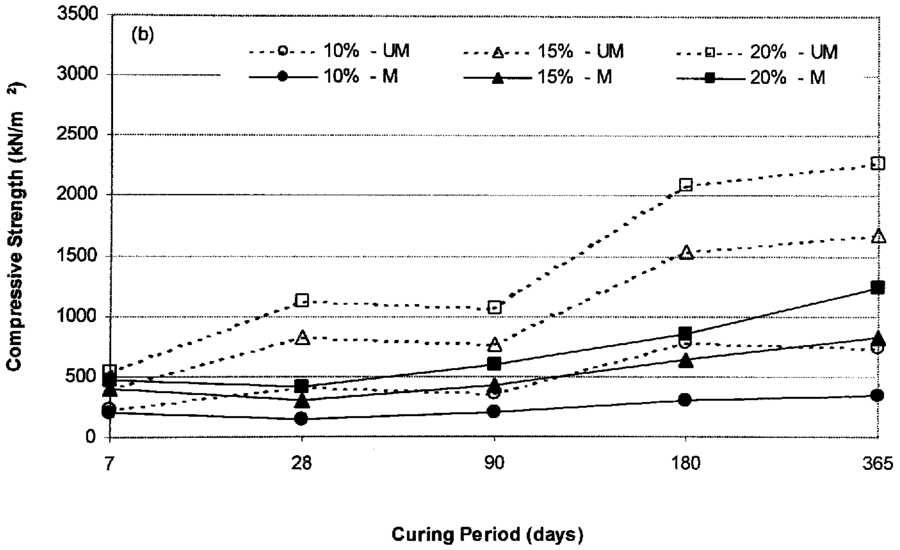


Figure 6.4-3: Compressive Strength of LOC stabilized with two WSA-PC blends (a) at 90:10 WSA:PC and (b) at 80:20 WSA:PC, for the mellowed and the unmellowed conditions.

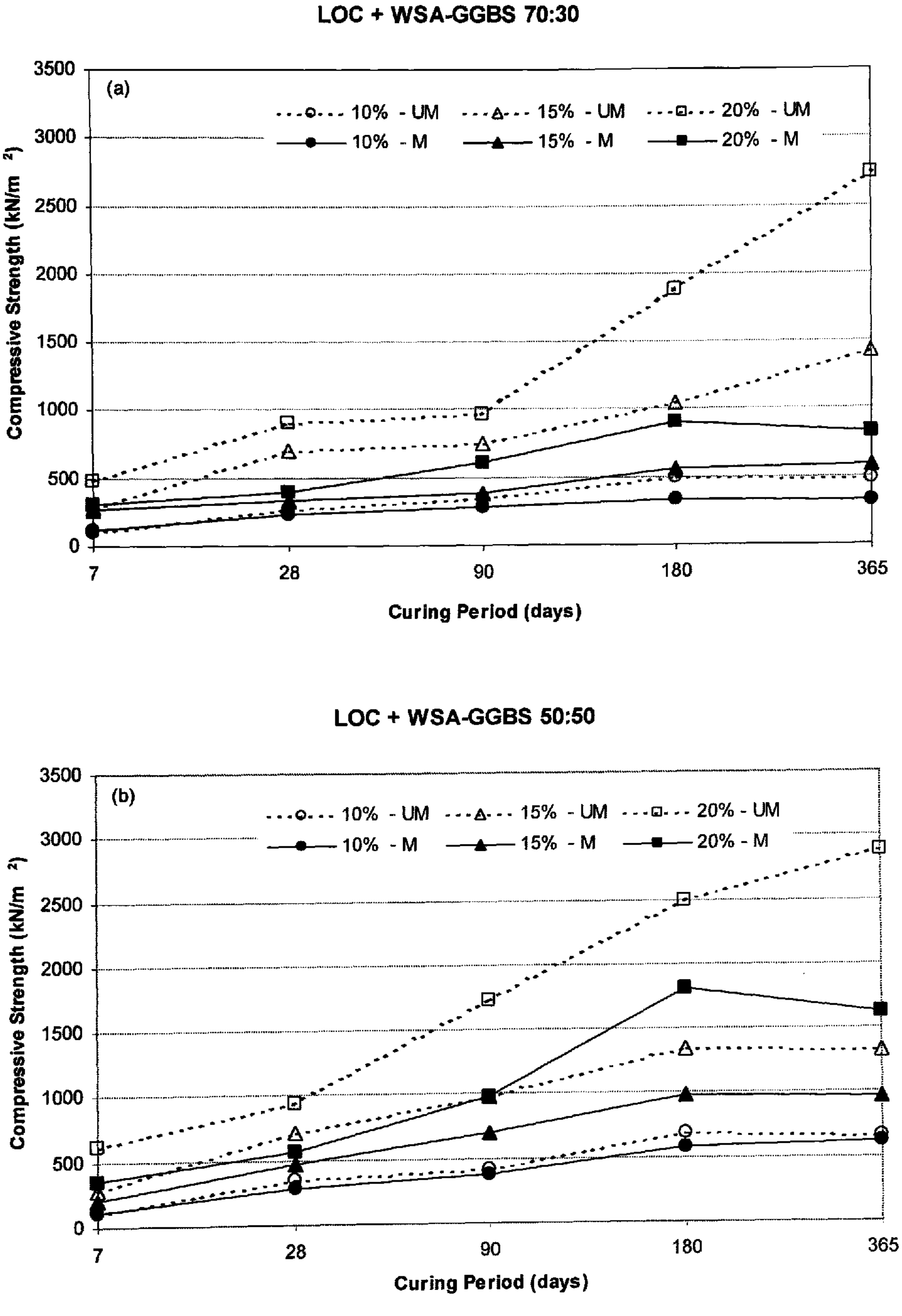


Figure 6.4-4: Compressive Strength of LOC stabilized with two WSA-GGBS blends (a) at 70:30 WSA:GGBS and (b) at 50:50 WSA:GGBS, for the mellowed and the unmellowed conditions.

6.4.2 Unconfined Compressive Strength of the Kaolinite System

Figures 6.4-5 to 6.4-8 illustrate the unconfined compressive strength of kaolinite, stabilized with lime, WSA, blended stabilizer - WSA-lime, WSA-PC (at 90:10 and 80:20) and WSA-GGBS (at 70:30 and 50:50) - for both mellowed and unmellowed conditions.

Figure 6.4-5(a) shows the UCS of Lime-stabilized kaolinite for both mellowed and unmellowed materials. As with Lime-stabilized LOC, the performance of the unmellowed stabilized material is in general greater than that of the mellowed material throughout the 365 days curing period. At all stabilizer contents, the specimens recorded comparable increasing trends in UCS values until 90 days of curing, after which the strength of both mellowed and unmellowed samples at 6wt.% lime content increased substantially from about 600-700 kN/m^2 to 2,085 kN/m^2 and 2,314 kN/m^2 respectively by 365 days of curing.

The UCS of WSA-stabilized kaolinite for both mellowed and unmellowed conditions is shown in Figure 6.4-5(b). At all WSA levels, the performance of the unmellowed stabilized material is superior to that of the mellowed material. The unmellowed Kaolinite stabilized with 20wt.% WSA recorded remarkable strength improvement throughout the one year curing period. In this system, the overall performance of the material stabilized with low WSA content (10%-15%) shows small improvement in strength development with increased curing period for both mellowed and unmellowed conditions. This is probably because the lower WSA content is insufficient to enhance the strength of the stabilized material. The unmellowed material however continued to record higher strength values, as in all other cases encountered so far.

Figures 6.4-6(a) and (b) illustrate the strength development of the Kaolinite stabilized with the WSA-lime blended stabilizer. As in the LOC system, two different blends of WSA-lime were investigated, at 90:10 and 80:20 (WSA:lime). In this Kaolinite system, the unmellowed specimens with 90:10 WSA-lime recorded higher UCS values than the mellowed ones up to 90 days of curing at all stabilizer contents. By 180 days of curing, the mellowed samples had reversed this trend and given greater UCS values. This increase in performance by the mellowed specimens was highest with the material stabilized with the highest amount of stabilizer (20wt.%), the other stabilizer contents showing small increase in UCS values over the mellowed material. By blending WSA with a relatively higher lime content, at 80:20 (WSA:lime), and unlike in the 90:10 blend, the results clearly show that the superior performance by the mellowed material was maintained throughout the one year curing period (see Figure 6.4-6(b)). The best strength improvement is again with the 20wt.% stabilizer which recorded 3,480 kN/m^2. This is also in contrast to the case where lime or WSA were used on their own where the unmellowed stabilized material generally recorded higher strength values compared with the mellowed material.

Figures 6.4-7(a) and (b) illustrates UCS performance of kaolinite stabilized with WSA-PC blended stabilizer, using the same blending ratios used as with WSA-lime blends (i.e 90:10 and 80:20 (WSA:PC)). In the unmellowed systems strength development was generally steady throughout the one year of curing. For the mellowed system, the strength was steady up to 180 days of curing, after which the rate of strength development either stopped or dropped slightly by one year of curing. Blending WSA with PC showed greater strength development than blending with lime especially at early curing periods (compare strength at 0-90 days in Figure 6.4-6 and 6.4-7). Also unlike in the WSA-lime system, there is a noticeable difference in the strength enhancement of the mellowed stabilized material when the ratio of PC to WSA is increased from 10:90 to 20:80.

Blending WSA with GGBS at 70:30 and 50:50 (WSA:GGBS) ratios was also studied as presented in Figure 6.4-8(a) and (b). With stabilizer blend relatively higher in WSA and lower GGBS content (at 70:30 WSA-GGBS), the UCS of the stabilized material increased progressively throughout the curing period (for both mellowed and unmellowed conditions) except for the material stabilized with 10wt.% stabilizer content. Above 90 days of curing, the mellowed stabilized material generally recorded higher strength than the unmellowed material.

In the system containing 50:50 WSA-GGBS, both mellowed and unmellowed stabilized material with low stabilizer content (10wt.%) showed little or no strength development over a period of one year of curing. With 15wt.%-20wt.% stabilizer content, a steady increase in UCS was observed after 28 days of curing, for both mellowed and unmellowed material. The mellowed material continued to demonstrate superior performance.

6.4.3 Summary

1. Increased amounts of stabilizer contents generally resulted in increased unconfined compressive strength with increasing curing period for stabilized LOC and kaolinite. Unmellowed stabilized LOC specimens consistently exhibited higher strength values than their mellowed counterparts throughout the one year curing period.

2. With the addition of 2wt.%-4wt.% lime and curing for up to 365 days, there was insignificant strength development of both mellowed and unmellowed Lime-stabilized LOC or kaolinite. This is because such low lime contents are only likely to achieve modification but little stabilization as there is insufficient lime to produce significant amounts of cementing reaction products.

3. For both mellowed and unmellowed conditions, UCS values for stabilized kaolinite were significantly higher compared with those for the LOC system, when stabilized with equal amounts of the various stabilizers. This is because of the inherent differences in the two clays.

4. The highest UCS values in both LOC and kaolinite were recorded in the system stabilized using WSA-GGBS. The strength development was very similar for WSA-PC and WSA-lime stabilized LOC and kaolinite.

5. Unlike the case for stabilized LOC, mellowed strengths for the stabilized kaolinite system are higher than those for the unmellowed system.

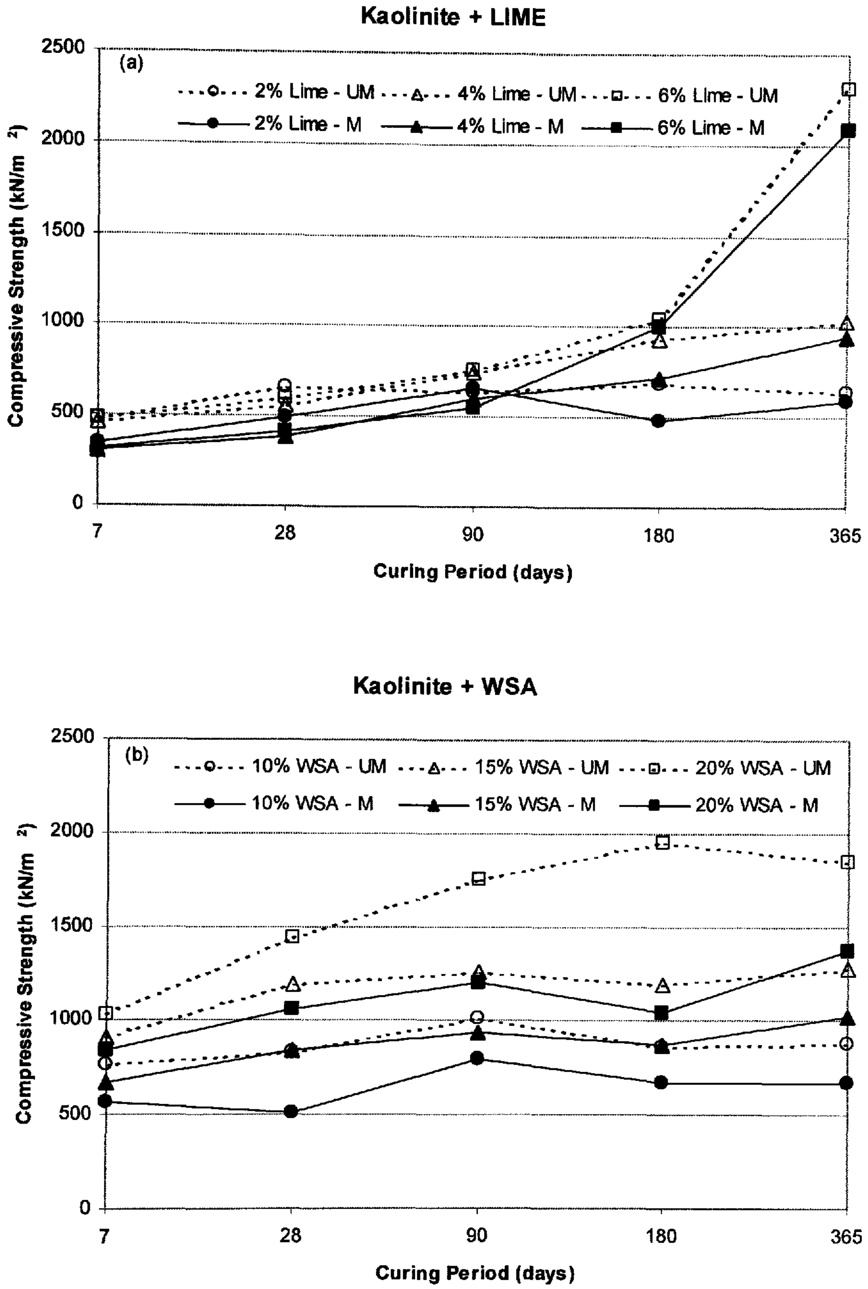
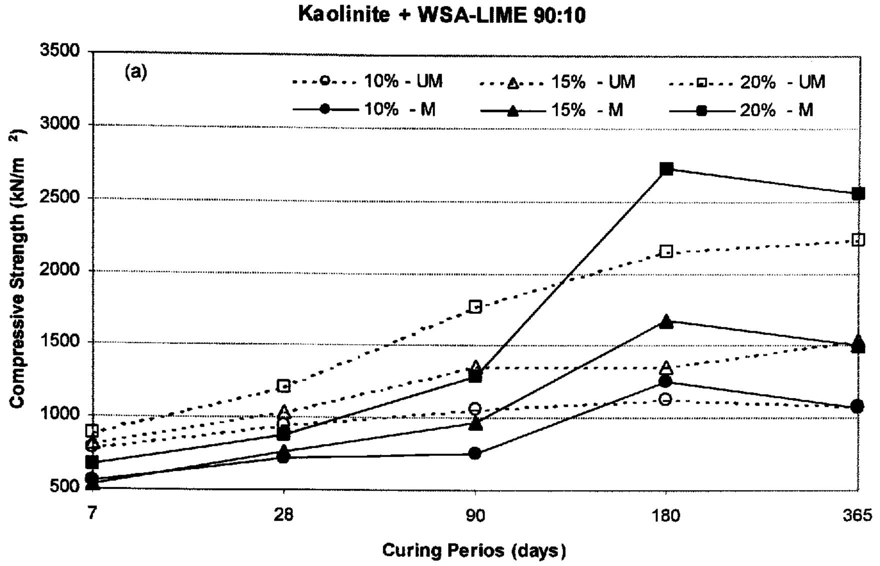


Figure 6.4-5: Compressive Strength vs. curing period of cylinder in both mellowed and unmellowed conditions for (a) Lime-stabilized kaolinite and (b) WSA-stabilized kaolinite.



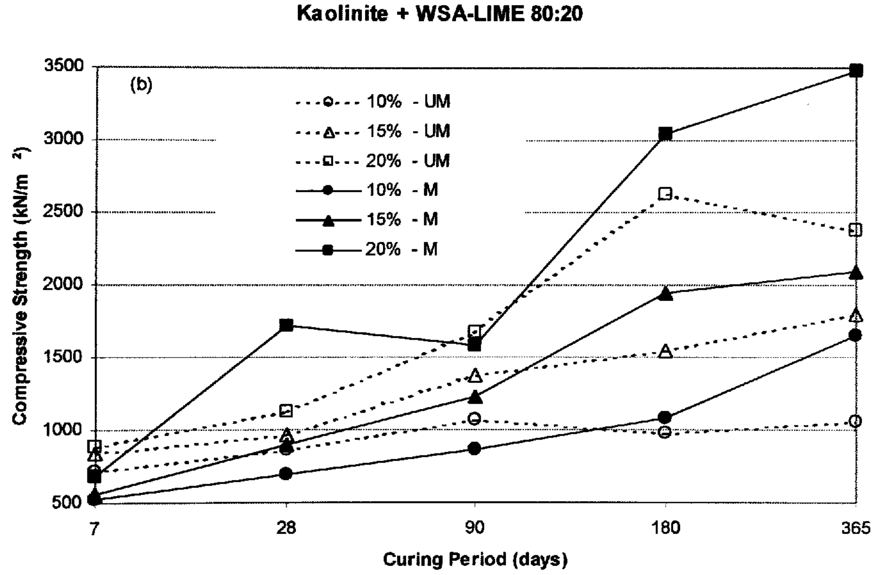


Figure 6.4-6: Compressive Strength of kaolinite stabilized with two WSA-lime blends (a) at 90:10 WSA:lime and (b) at 80:20 WSA:lime, for the mellowed and the unmellowed conditions.

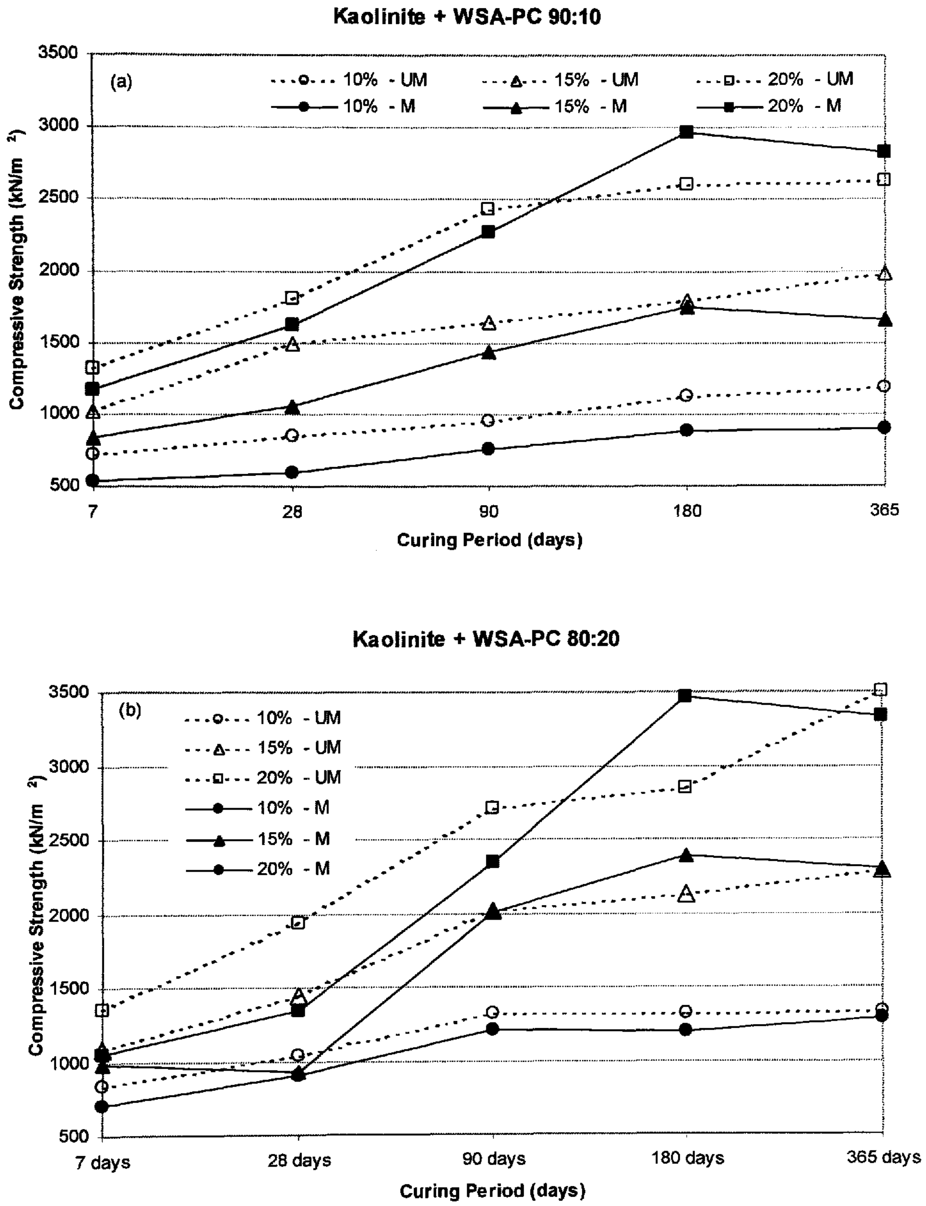
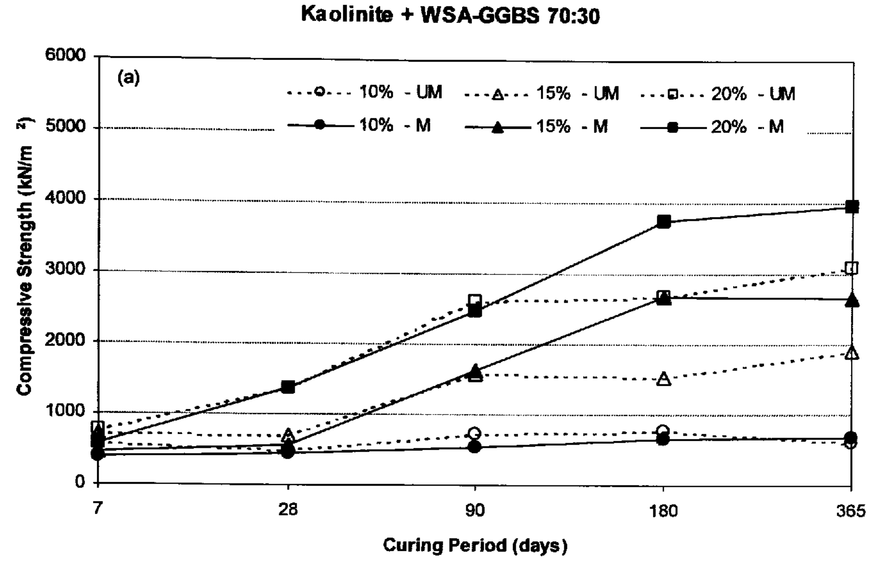


Figure 6.4-7: Compressive Strength of kaolinite stabilized with two WSA-PC blends (a) at 90:10 WSA:PC and (b) at 80:20 WSA:PC, for the mellowed and the unmellowed conditions.



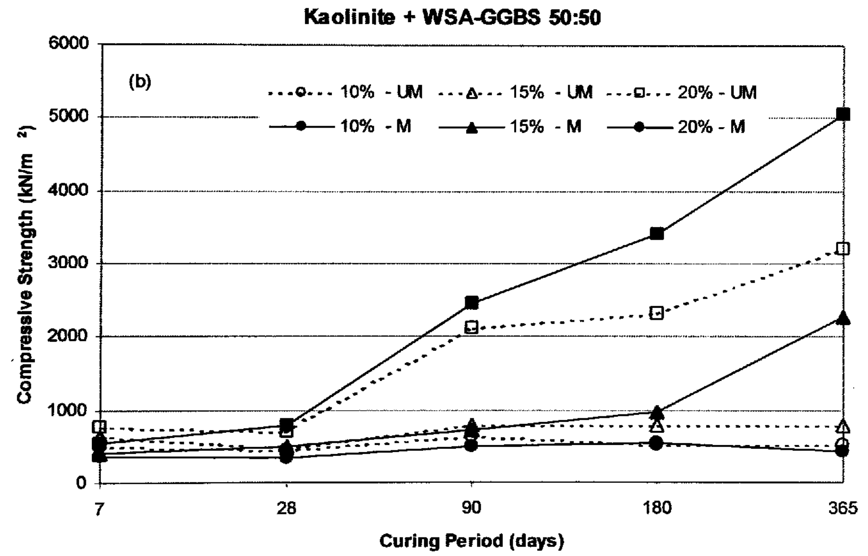


Figure 6.4-8: Compressive Strength of kaolinite stabilized with two WSA-GGBS blends (a) at 70:30 WSA:GGBS and (b) at 50:50 WSA:GGBS, for the mellowed and the unmellowed conditions.

6.5 CALIFORNIA BEARING RATIO (CBR) OF LOC SYSTEM

California Bearing Ratio tests were carried out on the LOC stabilized with various stabilizers at the lowest and highest stabilizer contents (2wt.% and 6wt.% lime, and 10wt.% and 20wt.% WSA-blended stabilizers). The test was carried out according to BS 1377-4:1999 as described in Section 5.5 in Chapter 5. The CBR was used as an additional strength test, in particular because it is the more commonly used strength test for highway pavement design by the Department for Transport. However, the test requires large amounts of soil and labour, and for laboratory investigation on strength, the UCS is usually the preferred test.

Figures 6.5-1 to 6.5-3 illustrate the CBR values for the LOC without stabilizer, and when stabilized with the blended stabilizers incorporating WSA. The test specimens were made either in the mellowed (3 days) or in the unmellowed condition, and then soaked for 4 days before testing for CBR penetration. The CBR values of both mellowed and unmellowed unstabilized LOC were lower than 5%, the minimum allowed for subgrades without a capping layer (Highway Agency, 2000). Stabilization of LOC would therefore be necessary during the construction of a foundation for highway pavement. The results show that there is no noticeable difference in the CBR value between the mellowed and unmellowed unstabilized LOC. This is expected, because changes during mellowing are due to the effects of stabilization with a cationic stabilizer. This is confirmed by the fact that the addition of 2wt.% and 6wt.% lime increased the unstabilized CBR value particularly for the unmellowed samples (see Figure 6.5-1(a)). These unmellowed specimens achieved higher CBR values than the minimum 15% CBR stipulated for lime/cement treated capping layers (Highway Agency, 2000). It is interesting to note that there is little difference in the CBR values of the unmellowed lime-stabilized LOC between 2wt.% and 6wt.% lime (see Figure 6.5-1(a). This is because the CBR test is more or less a short-period strength test (3 days of curing prior to 4 days soaking), a period insufficient to show major differences between two different stabilizer content.

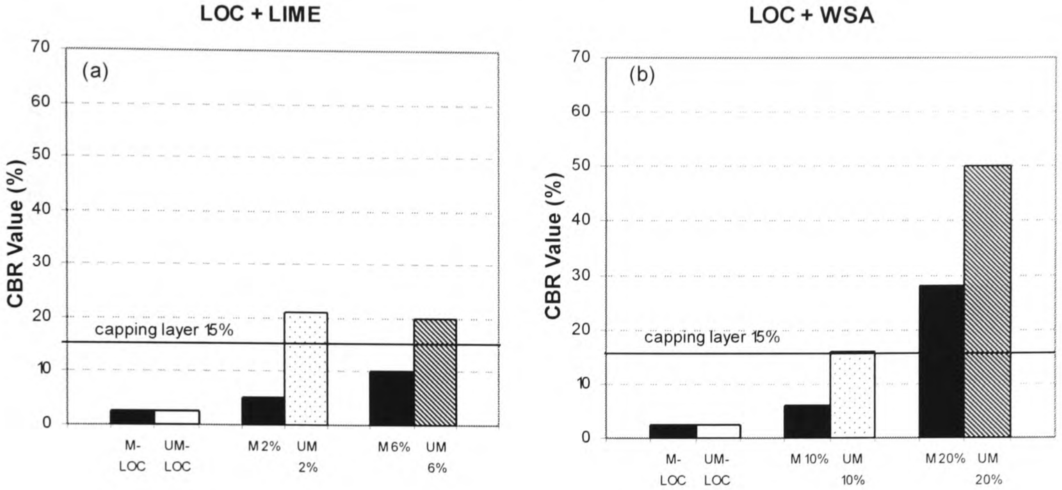


Figure 6.5-1: CBR value for (a) Lime stabilized LOC and (b) WSA-stabilized LOC

When WSA was used as a stabilizer, the CBR values were generally higher than when using lime, the addition of 20wt.% WSA resulting in significantly higher CBR values than 10wt.% WSA. Again, the unmellowed samples achieved better CBR values (see Figure 6.5-1(b)) compared with the mellowed ones. The higher CBR values are consistent with higher strength results observed in the UCS test as illustrated in Section 6.4.

Blending WSA with a controlled amount of lime was beneficial in increasing the CBR values of both lime-stabilized or WSA-stabilized LOC, especially with the higher stabilizer content of 20wt.% (see Figure 6.5-2(a) and (b)). There was no major advantage in incorporating higher amounts of lime into the system by changing from 90:10 to 80:20 (WSA-lime) on the CBR. On the contrary, the results show only a small improvement on the CBR values of the mellowed material, and a reduction in the CBR values for the unmellowed material. However, with the 80:20 WSA-lime blended stabilizer, all the specimens (mellowed and unmellowed) achieved at least 15% CBR while with the 90:10 blend, the mellowed specimens with 10wt.% stabilizer marginally failed the 15% CBR criterion.

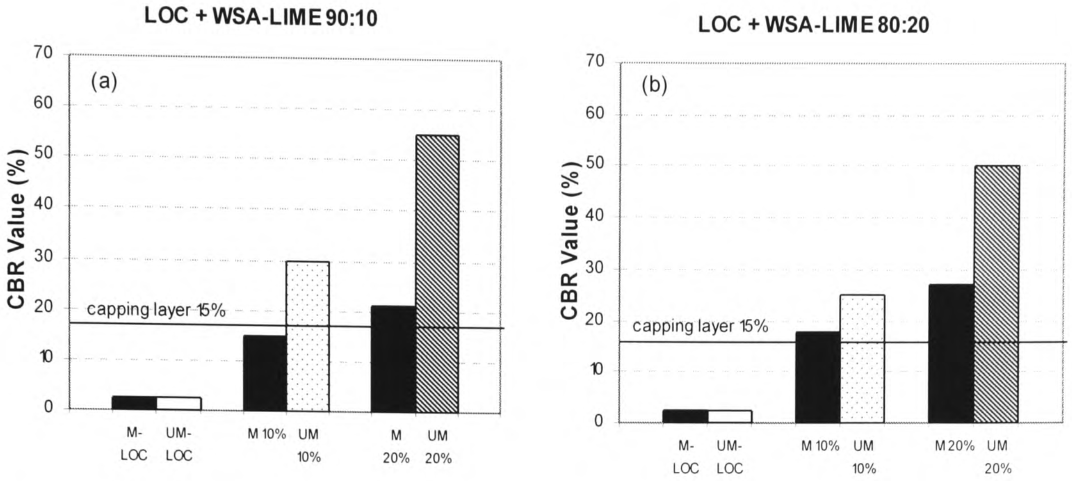


Figure 6.5-2: CBR value for WSA-lime stabilized LOC (a) at 90:10 and (b) at 80:20.

Blending WSA with PC at 90:10 and 80:20 (WSA-PC) blending ratios showed a similar effect on the CBR value as blending WSA with lime, as shown in Figure 6.5-3(a) and (b)). In both systems, the unmellowed condition with 20wt.% stabilizer content showed higher CBR values, with the 90:10 (WSA-PC) blend recording higher CBR values compared to the 80:20 (WSA-PC) blends. There was therefore no apparent significant advantage in increasing the PC content from 90:10 to 80:20 (WSA-PC). In both blending ratios, the mellowed specimens stabilized with 10% stabilizer content failed to achieve the 15% CBR threshold.

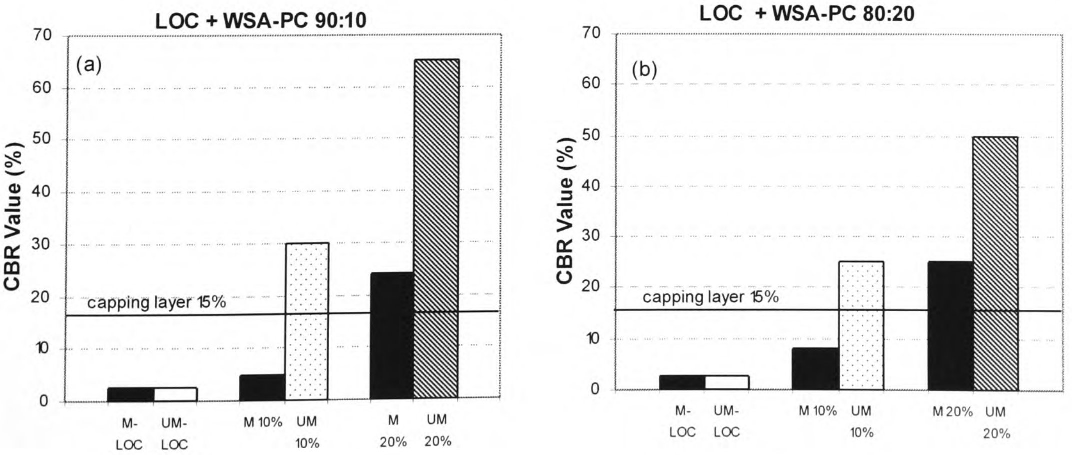


Figure 6.5-3: CBR value for WSA-PC stabilized LOC (a) at 90:10 and (b) at 80:20.

Figures 6.5-4(a) and (b) illustrate the CBR values for WSA-GGBS-stabilized LOC. Compared with the other blended stabilizers discussed so far, when GGBS was used to blend WSA as a stabilizer, there was a noticeable drop in CBR values, relative to WSA-lime and WSA-PC stabilizers, in both the 70:30 and 50:50 (WSA-GGBS) stabilizer ratios. This indicated that although the WSA-GGBS combination had recorded very good UCS development especially in the later curing periods (i.e. 28 days and later), the stabilizer does not produce good CBR values. The CBR test was carried out after only 3 days of curing (and 4 days of soaking) and thus, as earlier suggested, it is indicative of early strength development. The CBR results thus confirm the poor early strength development of the WSA-GGBS-stabilized LOC as also observed in the 7 day UCS results (see Figure 6.4-4(a) and (b) in section 6.4.1).

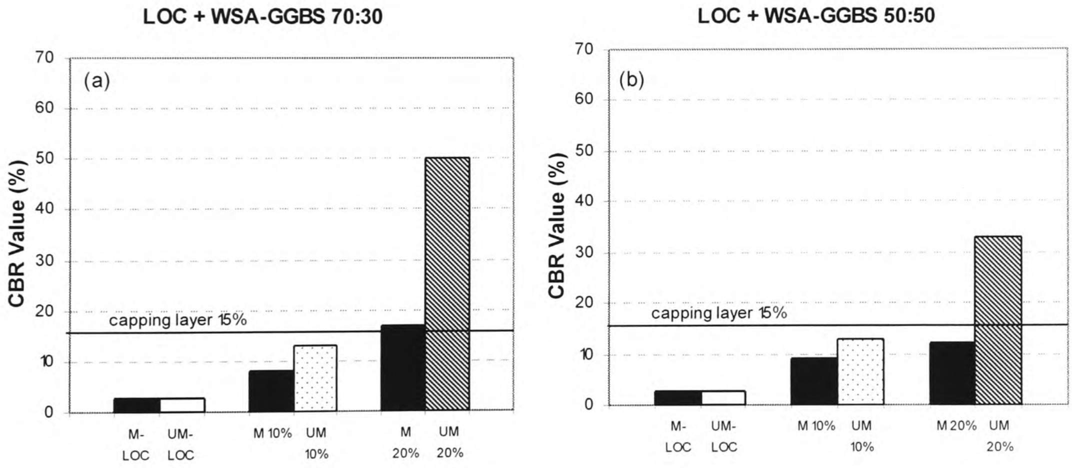


Figure 6.5-4: CBR value for WSA-GGBS stabilized LOC (a) at 70:30 and (b) at 50:50.

6.5.1 Summary

1. Addition of the blended stabilizers to Lower Oxford Clay increases the CBR value of the test soil.

2. With all the stabilizers used in the current study, the CBR values in the unmellowed system were higher compared to those in the mellowed condition. This supported the results on unconfined compressive strength.

3. The WSA-GGBS blend has significantly low early strength development especially at the higher GGBS content (50:50 blend), and only the unmellowed specimens with 20wt.% stabilizer in both blends (70:30 and 50:50) achieved CBR values significantly higher than the 15% requirement.

4. The lack of improved performance when the proportion of WSA is reduced in both WSA-lime and WSA-PC blended systems appears to suggest that of the three stabilizers WSA, lime and PC, WSA is the most reactive during the early stages.

6.6 LINEAR EXPANSION

This section presents data concerning the effects of lime, WSA and the blended stabilizers incorporating WSA - WSA-lime, WSA-PC and WSA-GGBS - on the linear expansion characteristics of the target soils, LOC and kaolinite. The test specimens were made using mellowed and unmellowed mixtures, compacted into cylinder form in the same way as the UCS test specimens. As stated earlier, compaction was carried out at the approximate MDD and OMC values. All the stabilized specimens were moist cured for 7 days prior to soaking. Monitoring of linear expansion was carried out during both the 7 day moist-curing period and during the subsequent soaking period for a total period of 100 days when no further significant expansion was observed.

6.6.1 Linear Expansion of the Lower Oxford Clay (LOC) System

Figures 6.6-1(a) and (b) illustrate the linear expansion of lime-stabilized LOC and WSA-stabilized LOC respectively, for both mellowed and unmellowed conditions. The effects of the blended stabilizer on the linear expansion are also illustrated in Figures 6.6-2 (WSA-lime), Figure 6.6-3 (WSA-PC), and Figure 6.6-4 (WSA-GGBS). In each of these Figures, a bar chart of the linear expansion of the stabilized specimens at 100 days is also presented.

Swelling and linear expansion of lime-stabilized, sulfate-bearing soil is common and is known to be associated with the formation of a colloidal product (a precursor to ettringite formation), which forms on the surface of the clay particles during curing (Wild *et al.*, 1993). When in a saturated condition, ettringite grows and develops from this colloidal product. It has a capability of imbibing large amounts of water and dramatically increases the swelling potential of the lime-stabilized soil. However, the introduction of a cementing agent such as WSA or PC with or without a combined action with GGBS, modifies the chemical interaction of the clay-lime system, thereby altering the types of reaction products and thus potentially altering any disruptions that the reaction products may cause.

Over the 100 days observation period, all the systems either attained terminal linear expansion or continued to expand at a negligible rate of increase. In most cases with the blended soil systems, about 90% of the ultimate expansion occurred within the first 30 days of soaking.

In the lime-stabilized LOC system (see Figure 6.6-1(a)), the mellowed specimens were observed to expand more than the unmellowed ones at all lime stabilizer contents. In this system, in both mellowed and unmellowed conditions, the highest expansion magnitudes were observed, compared with LOC stabilized with either WSA alone or with WSA blended with either lime, PC or GGBS at all stabilizer contents. The linear expansion was immediate when the specimens were soaked in water after the 7 day moist-curing period. This expansion was more stable after about 30 days of soaking. The WSA-stabilized LOC (see Figure 6.6-1(b)) recorded significantly lower expansion values compared with the lime-stabilized LOC, for both mellowed and unmellowed conditions (note the different y-axis scale). The mellowed material showed a small but consistent reduction in expansion with increasing amounts of stabilizer. At 100 days of soaking, the lime-stabilized LOC recorded above 5% linear expansion, compared to below 1% for LOC stabilized with WSA (see Figure 6.6-1(c)).

Blending WSA with a small quantity of lime (at 90:10 or 80:20 (WSA:lime) ratios) resulted in a further reduction in linear expansion of stabilized LOC (see Figures 6.6-2(a-c)) compared with either the lime-stabilized or the WSA-stabilized LOC. The mellowed LOC+(WSA-lime) continued to exhibit greater linear expansion compared with the unmellowed system, although the highest expansion magnitude was only about 0.7% recorded with the lowest stabilizer content (10wt.%) in the 80:20 (WSA-lime) system. In the mellowed condition, the 90:10 (WSA-lime) stabilizer that contains a relatively higher amount of WSA compared with the 80:20 (WSA-lime) blend recorded lower linear expansion. This is likely to be due to excess lime in the 80:20 blend which is richer in lime compared with the 90:10 blend, the system with less total stabilizer content (10wt.%) showing least stability.

By blending WSA with PC, the linear expansion trend was very similar to that observed with both WSA, lime and with the WSA-lime system, in that the highest expansion was generally exhibited by the mellowed system (see Figure 6.6-3). The WSA-PC system recorded the least expansion compared to all the other systems that were investigated. This is an indication of significant enhancement of the cementing capability of WSA when blended with PC.

Figure 6.6-4 shows the linear expansion of LOC specimens stabilized with WSA-GGBS blends (at 70:30 and 50:50 ratios). The mellowed WSA-GGBS system again showed a significantly greater linear expansion relative to the equivalent unmellowed system. The mellowed WSA-GGBS system recorded the highest expansion compared with WSA, WSA-PC or WSA-lime systems. The highest expansion (of about 1.45%) was observed for the mellowed condition with 15wt.% stabilizer content at 50:50 blend. Interestingly, there was very little expansion for the unmellowed condition.

When WSA is used either on its own, or when it is activated by either lime or PC, or when WSA is used to activate GGBS, the reduction in linear expansion is likely to be due to the formation of cementitious products. The cementitious gels cement the soil particles together and enable them to resist the considerable swelling pressures which can be generated when ettringite forms in the presence of water. The hydration of WSA, PC and/or GGBS is much more rapid compared with the pozzolanic reaction of lime with clay. This hydration reaction is known to consume lime and therefore the resistance to swelling in all systems incorporating WSA was enhanced by the reduction in residual lime. One possible explanation why the mellowed stabilized test specimens expanded more than the unmellowed ones is that the delayed compaction in the mellowed system allowed the hydration of WSA to commence and possibly consume all the water very rapidly during this period, and little or no ettringite was able to form. The formation of any ettringite would not have the capacity to influence the strength of the material in its mellowed and unprepared state (not compacted). Thus, when the samples were compacted, soaked and saturated in water, ettringite was then able to form and subsequently imbibe a large quantity of water which would lead to significant expansion. In the unmellowed system, compaction was carried out immediately upon water addition, and any ettringite and/or other hydration products were used to fill the voids thus enhancing both strength and volume stability upon subsequent soaking. The fact that higher strength magnitudes in the mellowed system were observed with the WSA-GGBS system (see Figure 6.4-4) and it is the same system that also expanded most suggest that volume stability is a sensitive balance/between void space and cementation. It is hypothesised that while the WSA-GGBS is well cemented, there are insufficient voids to cater for any significant additional of hydration/colloidal products. This is probably due to the poor modification of soil in the WSA-GGBS system due to depletion of lime by GGBS, compared with either the WSA-lime or WSA-PC system. It is therefore the WSA-PC that has both good modification and cementing properties which gives better volume stability.

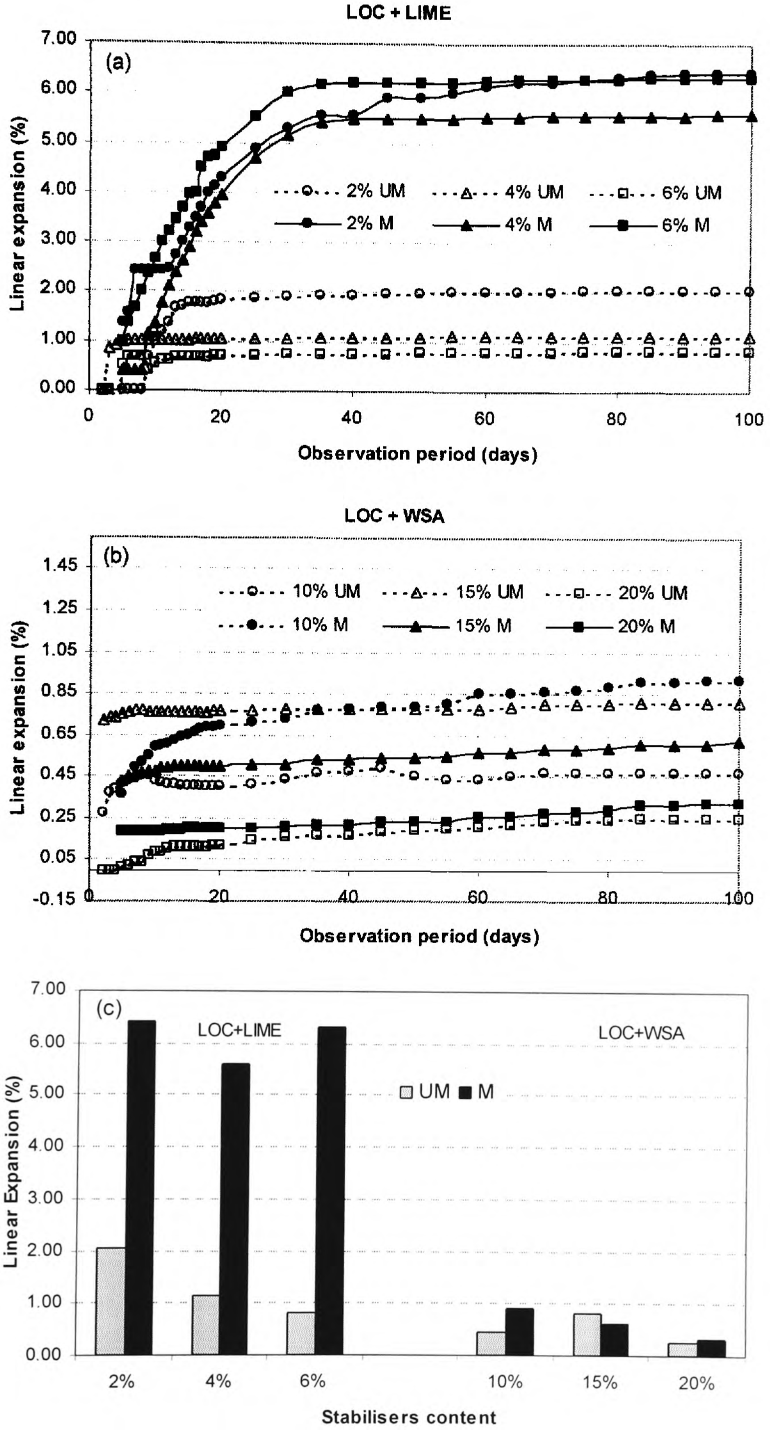


Figure 6.6-1: Linear Expansion of mellowed and unmellowed (a) lime-stabilized LOC (b) WSA-stabilized LOC and (c) Expansion at 100 days

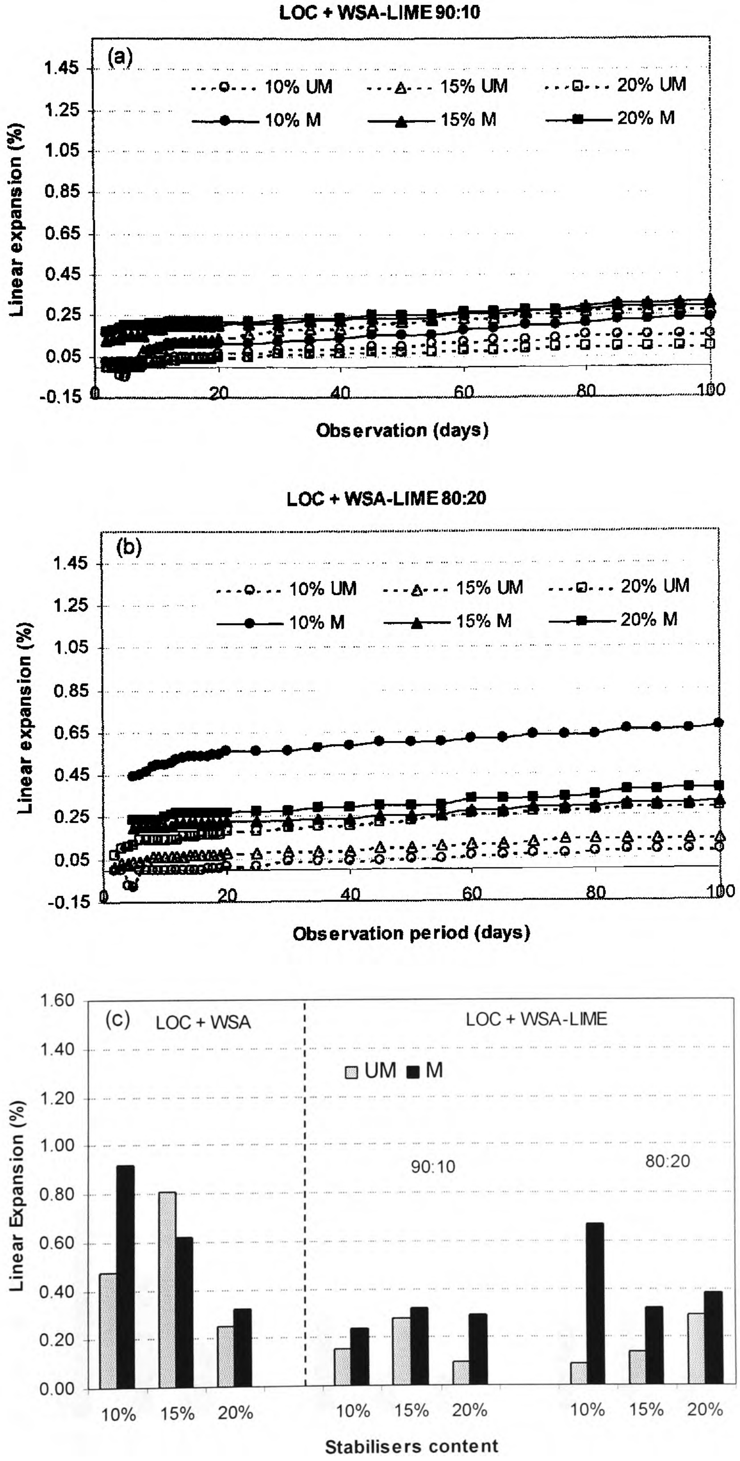


Figure 6.6-2: Linear Expansion of mellowed and unmellowed LOC stabilized with (a) at 90:10 and (b) at 80:20 (WSA-lime blends), and (c) Expansion after 100 days of soaking.

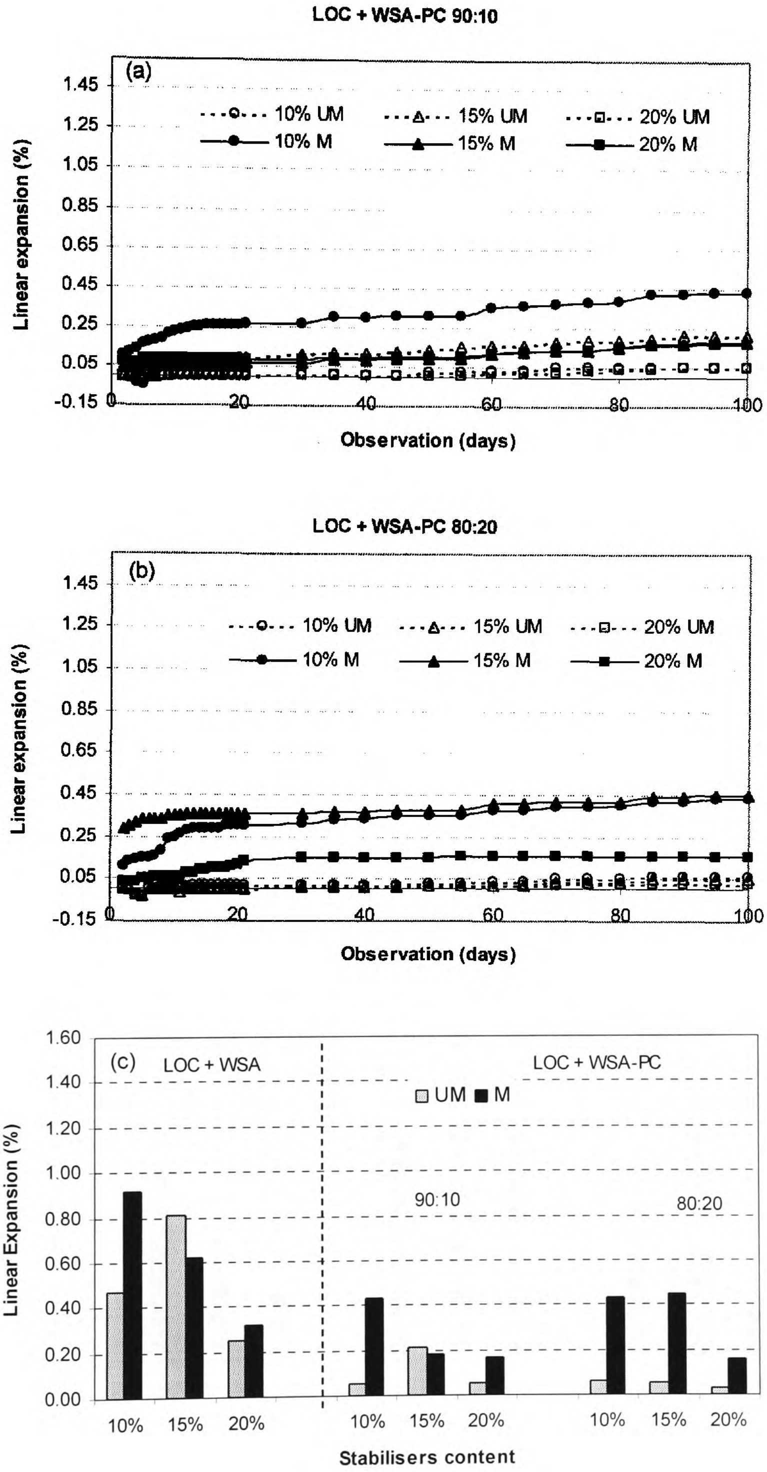


Figure 6.6-3: Linear Expansion of mellowed and unmellowed LOC stabilized with (a) at 90:10 and (b) at 80:20 (WSA-PC blends), and (c) Expansion after 100 days of soaking.

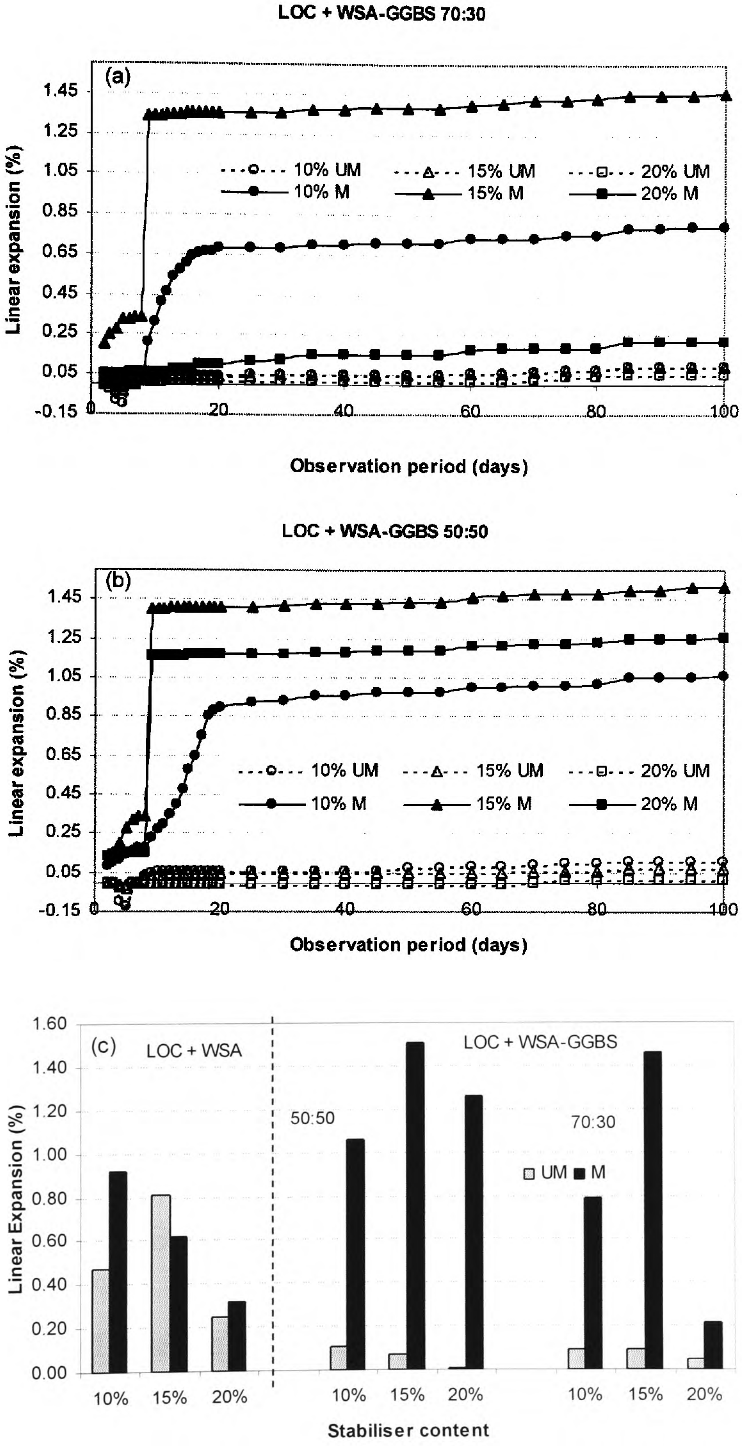


Figure 6.6-4: Linear Expansion of mellowed and unmellowed LOC stabilized with (a) at 70:30 and (b) at 50:50 (WSA-GGBS blends) and (c) Expansion after 100 days of soaking.

6.6.2 Linear Expansion of the Kaolinite System

Figures 6.6-5 to 6.6-8 illustrate the linear expansion of kaolinite stabilized with lime, WSA, and with the blended stabilizers - WSA-lime, WSA-PC and WSA-GGBS respectively.

In all the stabilizer systems, specimens of stabilized kaolinite which had undergone a mellowing period for 3 days before compaction into cylinders generally expanded less than the unmellowed specimens. Thus, unlike in the LOC system, mellowing reduced linear expansion in the kaolinite system. When kaolinite was stabilized using lime, the highest dosage used (6wt.%) resulted in the highest linear expansion, for both mellowed and unmellowed conditions (see Figure 6.6-5(a)). This is strange, as in the absence of sulfate, higher lime contents were expected to result in enhanced volume stability. One possible explanation is excessive flocculation, resulting in increased pore volume which imbibed large amounts of water in an osmotic suction process.

Linear expansion reduced with increasing amounts of stabilizer, for both mellowed and unmellowed conditions in the WSA-stabilized kaolinite system (see Figure 6.6-5(b)). Blending WSA with lime appeared to increase the linear expansion of kaolinite compared to when kaolinite was stabilized with WSA alone (see Figure 6.6-6). In this system, linear expansion increased with increasing amount of stabilizer for both WSA-lime blends used (at 90:10 and 80:20). The system with more WSA and less lime (at 90:10), in general expanded less than the system containing less WSA and more lime (at 80:20) (see Figure 6.6-6(a),(b) and (c)). The linear expansion was therefore proportional to the lime content and inversely proportional to WSA content.

Blending WSA with small amounts of PC significantly reduced the linear expansion compared with kaolinite stabilized with either lime, WSA or WSA-lime blends. Additional increase in the WSA-PC stabilizer content resulted in a further decline in expansion. For both mellowed and unmellowed specimens, this system exhibited the least expansion compared with the system stabilized with WSA or any of the other blended stabilizers (see Figure 6.6-7).

Figure 6.6-8 illustrates the effects on linear expansion of blending WSA with GGBS at 70:30 and 50:50 ratios. In this system the unmellowed stabilized specimens recorded the highest expansion at about 4.5% before collapsing after about 20 days of soaking, except for the sample with 20wt.% stabilizer level in the 50:50 blend which attained terminal linear expansion. In contrast, the mellowed specimens stabilized with both 70:30 and 50:50 (WSA:GGBS) blends collapsed immediately upon soaking after 7 days of moist curing indicating negligible cementation. This suggests that the WSA-GGBS system was very vulnerable to expansion and even possible collapse, in both the LOC and kaolinite systems. However, there was a distinct difference between the two systems. In the LOC system, it was the mellowed specimens that were vulnerable while in the kaolinite system, it was the unmellowed samples.

6.6.3 Summary

1. In both the LOC and kaolinite stabilized systems, linear expansion was immediate upon soaking after 7 days of moist curing. The rate of expansion varied depending on the type and amount of stabilizer added.

2. Over a 100-day period, all the LOC and kaolinite stabilized specimens either attained terminal linear expansion or continued to expand at a negligible rate of increase.

3. In the LOC system, unmellowed stabilized material exhibited less expansion compared to that of the mellowed system. On the other hand in the kaolinite system a period of mellowing (3 days) prior to compaction resulted in reduced linear expansion of specimens relative to the unmellowed specimens.

4. With all the stabilizers in LOC system, the mellowed specimens expanded more than the unmellowed ones, with exception of two specimens, one stabilized with 15wt.% WSA and one with 15wt.% of 90WSA:10PC. Whereas in the entire

stabilized kaolinite system, the mellowed samples recorded reduced linear expansion.

5. The WSA-GGBS system was vulnerable to expansion for both the LOC and Kaolinite systems. However, it was the mellowed specimens that showed excessive expansion in the LOC system. In the kaolinite system, it is the unmellowed specimens that showed excessive expansion. It is therefore very important that the advantages of mellowing or not mellowing be thoroughly investigated prior to adoption of either practice.

6. The WSA-PC stabilizer achieves both high strength and high volume stability. This exceptional performance of WSA-PC blends has application in both Highway Construction and in Building Construction (as confirmed by Veerapan *et al.*, 2003).

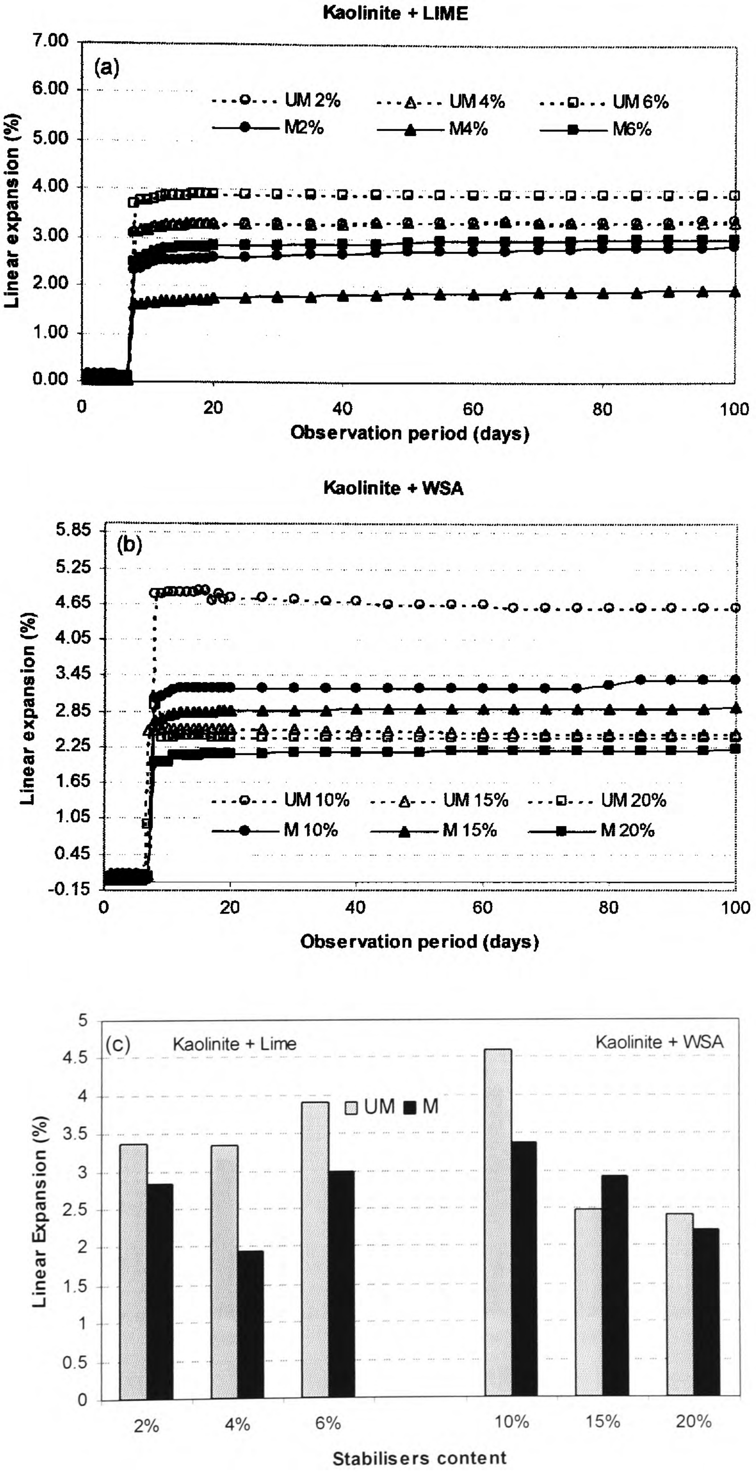


Figure 6.6-5: Linear Expansion of mellowed and unmellowed (a) lime-stabilized kaolinite, and (b) WSA-stabilized kaolinite, and (c) Expansion after 100 days of soaking.

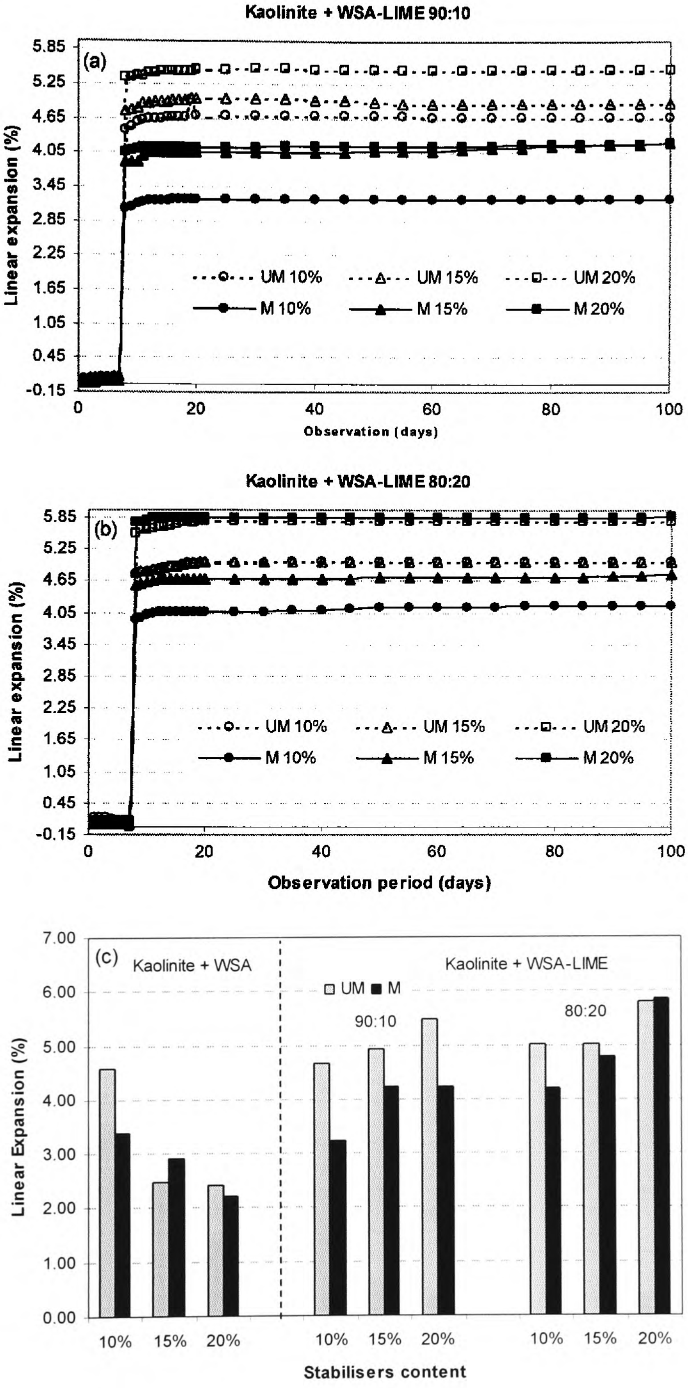


Figure 6.6-6: Linear Expansion of mellowed and unmellowed kaolinite stabilized with (a) at 90:10 and (b) at 80:20 (WSA-lime blends), and (c) Expansion after 100 days of soaking.

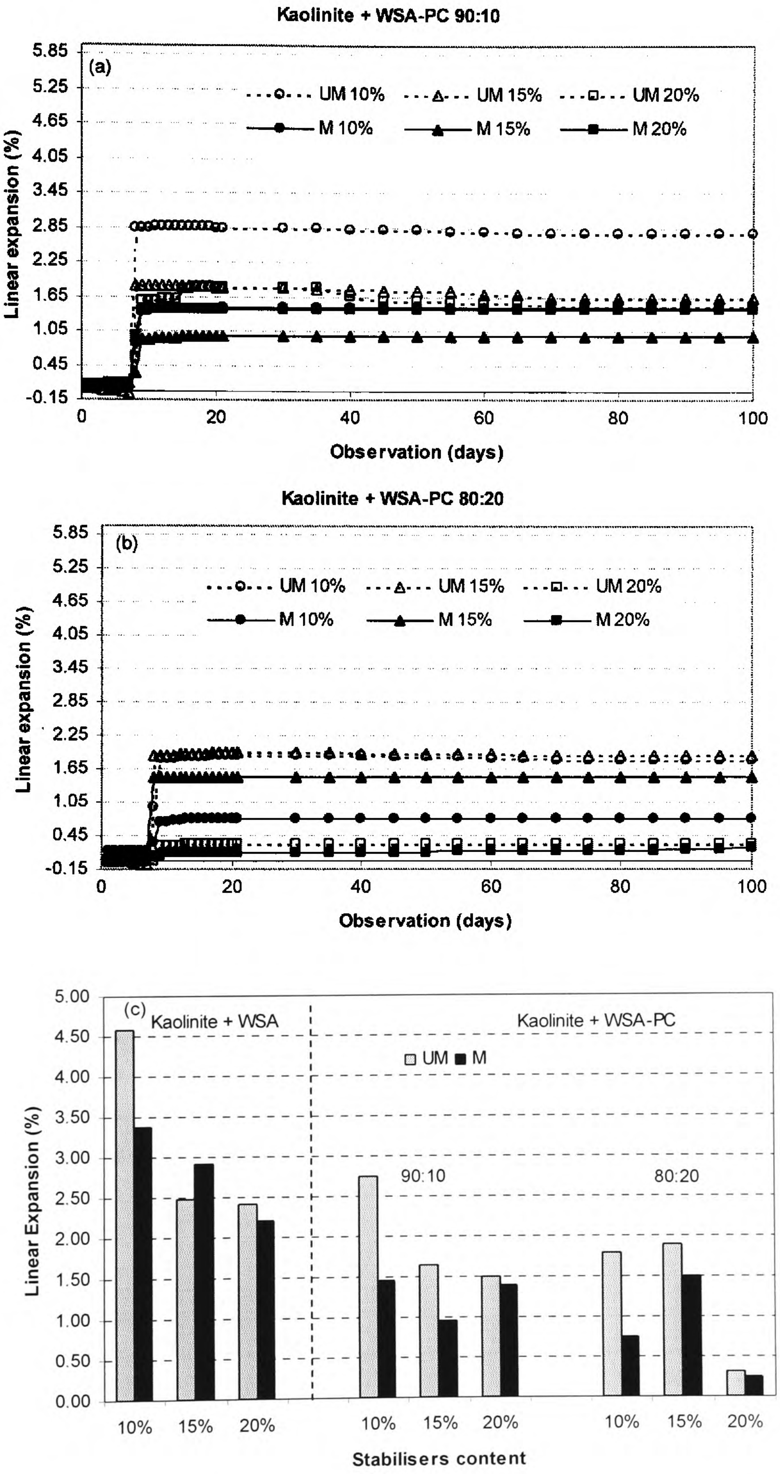


Figure 6.6-7: Linear Expansion of mellowed and unmellowed kaolinite stabilized with (a) at 90:10 and (b) at 80:20 (WSA-PC blends), and (c) Expansion after 100 days of soaking.

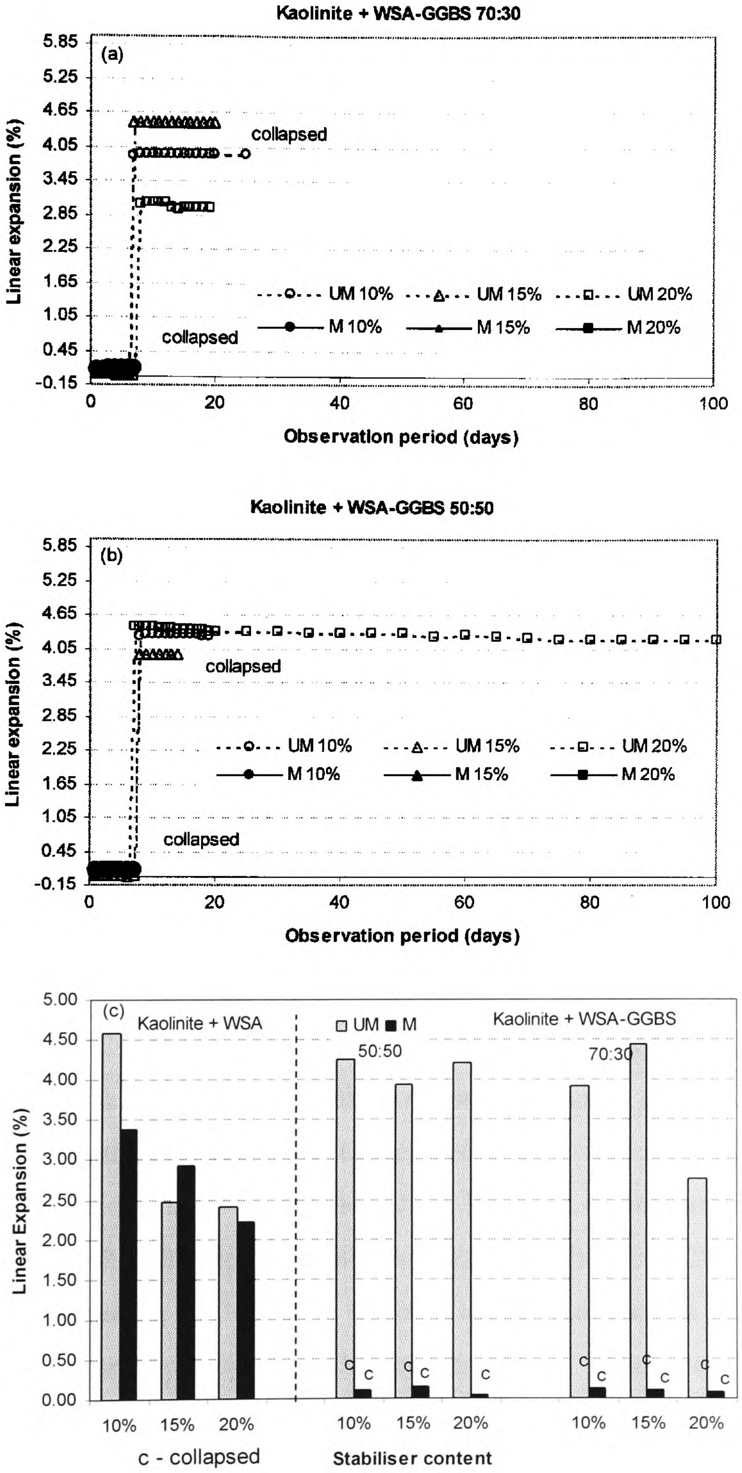


Figure 6.6-8: Linear Expansion of mellowed and unmellowed kaolinite stabilized with (a) at 70:30 and (b) at 50:50 (WSA-GGBS blends), and (c) Expansion after 100 days of soaking.

6.7 SULFATE (SO_3) ANALYSIS OF THE LOC SYSTEM

Sulfate (SO_3) analysis was carried out on all stabilized specimens that had undergone testing for UCS, but only for the selected curing periods of 0, 28, 180 and 365 days (omitting those cured for 7 and 90 days). Sulfate tests were also carried out on specimens after the linear expansion tests, after 100 days of soaking (experiments outlined in chapter 5, sections 5.7). Internal fragments were taken from each UCS or linear expansion test specimen, and then dried under silica gel in a desiccator (at 40°C) until constant weight was observed.

Figures 6.7-1 to 6.7-3 illustrate the SO_3 content of the LOC systems stabilized with various stabilizers, with and without incorporating WSA, and moist cured for various periods prior to testing for UCS. In all the systems, the sulfate contents are generally higher in the specimens prepared in the mellowed condition compared with the corresponding unmellowed ones. There is a general initial increase in SO_3 level during the first 28 days of curing/soaking before, in most cases, generally dropping or remaining unchanged between 28 and 180 days, except for the lime-stabilized LOC which showed continued increase in SO_3 level up to 180 days. There was a general decrease in SO_3 level after 180 days, especially for the mellowed systems.

In the lime-stabilized LOC system (see Figure 6.7-1(a)), sulfate levels increased significantly (especially in the mellowed condition) during the first 28 days of curing. In the mellowed condition, the SO_3 continued to increase at a slower rate between 28 and 180 days of curing, before dropping drastically by 365 days. In the unmellowed condition, the SO_3 level either dropped or remained unchanged between 28 and 180 days of curing, before increasing by 365 days. It would appear that there is a delayed increase in SO_3 in the unmellowed system. The reasons for the initial increase and also the subsequent drop in SO_3 are more fully discussed in the discussion - Chapter 7.

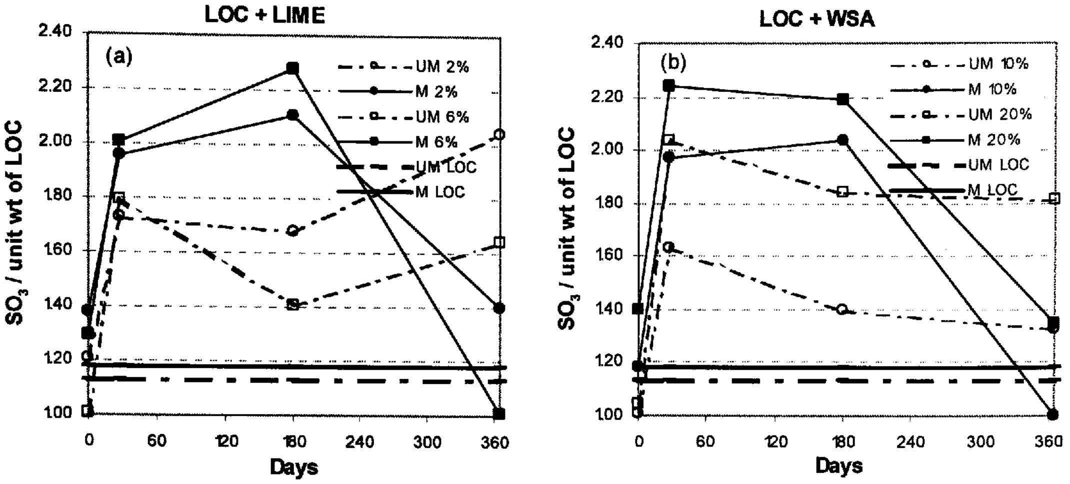


Figure 6.7-1: Sulfate Analysis for (a) lime-stabilized LOC and (b) WSA-stabilized LOC (from UCS test specimens).

Figure 6.7-1(b) illustrates the changes in the SO_3 levels in the WSA-stabilized LOC system. As in the lime system, the sulfate increased sharply during the first 28 days of curing for both mellowed and unmellowed conditions. Thereafter the sulfate levels in the mellowed condition did not level off until 180 days before dropping significantly by 360 days of curing. In the unmellowed condition, the sulfate levels generally decreased after 28 days, although there is a noticeable reduction in the rate of decrease after 180 days. As in the lime system, there would appear to be a delayed stabilizing effect in the decrease in the sulfate levels in the unmellowed system.

The effects of blending WSA with lime at 90:10 and 80:20 (WSA-lime) ratios are shown in Figure 6.7-2(a) and (b) respectively. As in the lime-stabilized LOC system, there was an initial increase in SO_3 levels for both blending ratios, in both mellowed and unmellowed conditions. The mellowed specimens continued to display higher SO_3 content, especially with the specimens higher in lime content (at 80:20 blending ratio). However, unlike for both the lime and WSA systems, there was either a significant reduction in the rate of decrease in SO_3 levels or, in some cases, some increase in SO_3 levels beyond 28 days of moist curing in both mellowed and unmellowed systems.

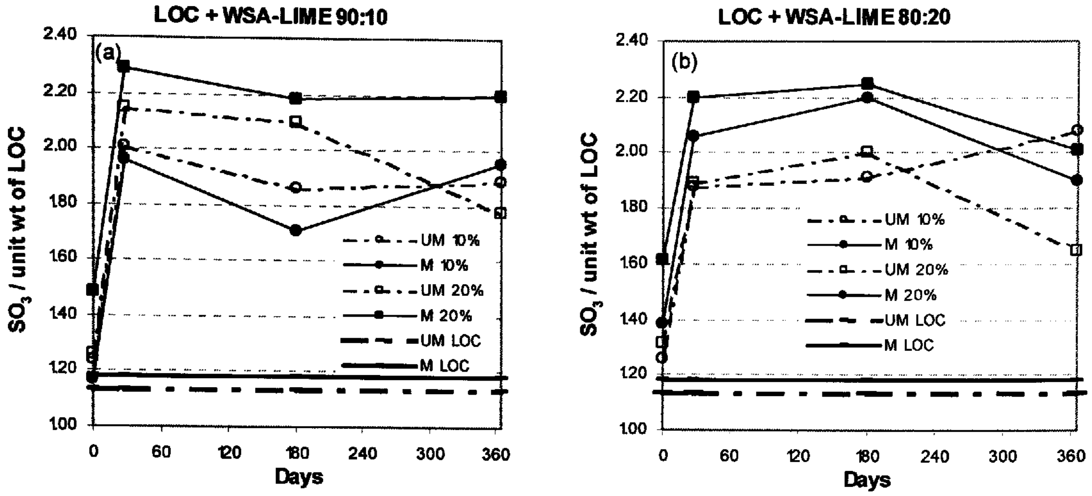


Figure 6.7-2: Sulfate Analysis for WSA-lime stabilized LOC (a) at 90:10 and (b) at 80:20 (form UCS test specimens)

In the WSA-PC-stabilized LOC system, the sulfate levels generally increased to a maximum value during the first 28 days of curing but started to drop slowly over the rest of the curing period. Once again, specimens in the mellowed condition recorded higher sulfate levels compared with their unmellowed counterparts (see Figure 6.7-3(a) and (b)). Sulfate levels in the unmellowed specimens stabilized with 10% WSA-PC blends increased significantly after 180 days of curing, in contrast with specimens with 20wt.% stabilizer content. There is no significant difference in sulfate levels between the two WSA-PC blending ratios.

Blending WSA with GGBS in stabilized LOC resulted in an increase in sulfate levels during the first 28 days but at a much reduced scale compared with all the other stabilizer systems discussed (see Figure 6.7-3(c) and (d)). The consumption of lime by GGBS appears to have had a significant effect on the production of sulfate in this system. The sulfate levels dropped after 180 days curing in almost all cases, and there is no case of increased sulfate levels (even with the unmellowed specimens) after 180 days of moist curing. In the 70:30 (WSA-GGBS) blend, the sulfate levels in most cases started to drop after 28 days of curing. In both blends the sulfate levels are higher in the mellowed condition compared with the unmellowed condition.

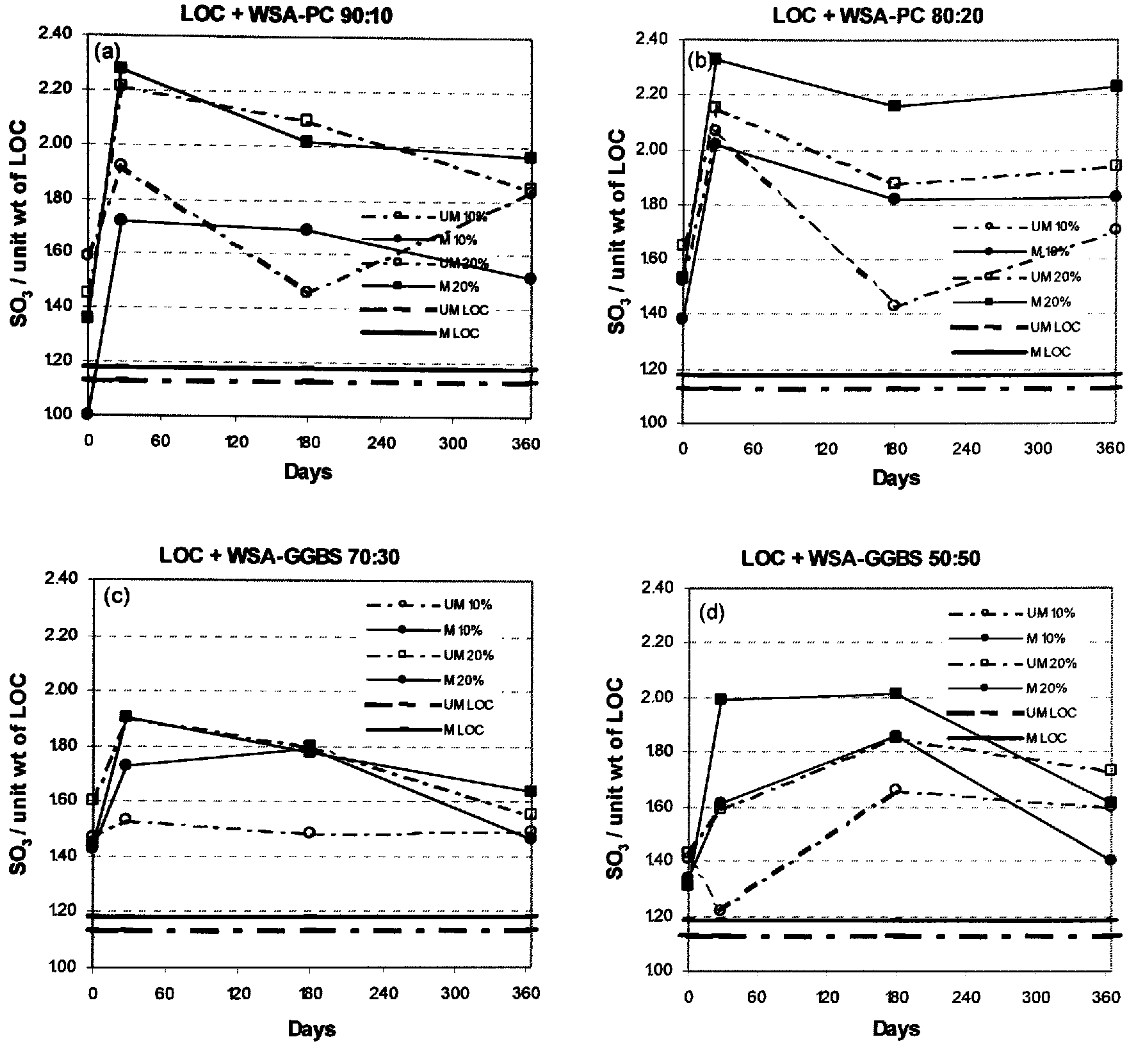


Figure 6.7-3: Sulfate Analysis for WSA-lime stabilized LOC (a) at 90:10 and (b) at 80:20 and WSA-GGBS stabilized LOC (c) at 70:30 and (d) at 50:50 (from UCS test specimens)

Figure 6.7-4 illustrates the sulfate analysis for samples subjected to linear expansion. It is generally concluded that sulfate levels are once again higher in the mellowed condition compared to the unmellowed condition, as observed with specimens that were subjected to moist curing during the unconfined compressive strength tests.

6.7.1 Summary

In summary, it is interesting to note that in the unmellowed system, the specimens stabilized with the lower stabilizer contents (2wt.% lime or 10wt.% WSA-blends) showed significant increase in SO_3 levels beyond 180 days of curing. This appears to suggest that at high lime levels, all the oxidation of LOC is likely occur early, leading to a reduction in subsequent oxidation. At very low lime levels, there is also little chance for subsequent oxidation after the initial oxidation occurring between 0-8 days. However, at intermediate lime levels, oxidation of LOC is likely to continue especially in the unmellowed systems thus recording further formation of SO_3. The following major aspects may be drawn:-

1. Sulfate levels are generally higher in the mellowed condition compared with those in the unmellowed conditions.

2. Sulfate levels generally drop after 28 or 180 days in most systems. The drop was more significant in the mellowed system stabilized with :-

   (i) Lime only
   (ii) WSA only
   (iii) 50:50 WSA-GGBS

3. There was delayed increase in sulfate levels, especially in the unmellowed systems that contained either lime or PC.

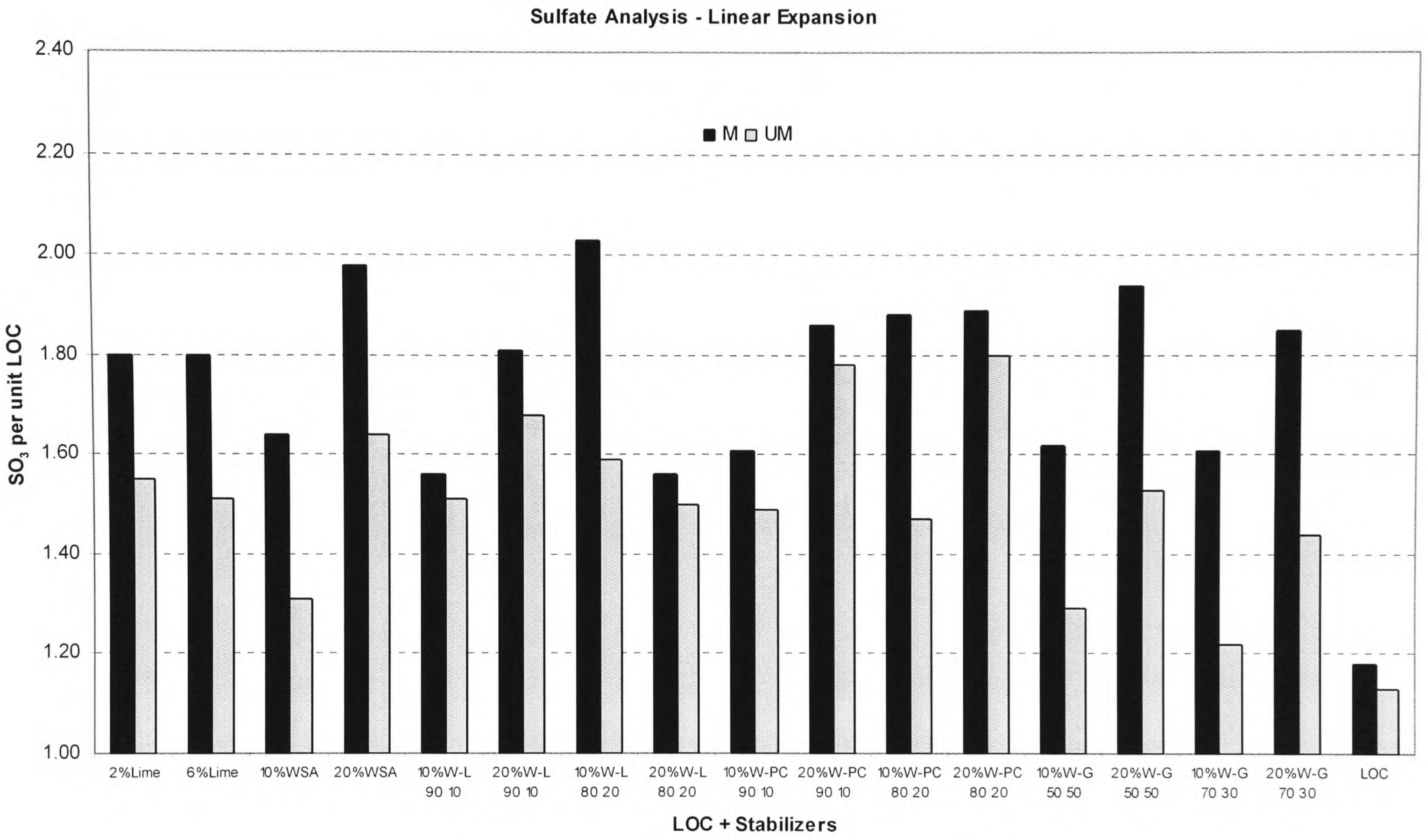


Figure 6.7-4: Sulfate analysis of LOC system after soaking for 100 days during tests for Linear Expansion.

6.8 THERMOGRAVIMETRIC ANALYSIS (TGA)

6.8.1 Introduction

Thermogravimetric and Derivative Thermogravimetric (TG/DTG) analysis was carried out on stabilized material at the highest stabilizer contents used - 6wt.% lime and 20wt.% for each of the blended stabilizers – for material that had undergone UCS testing, with and without mellowing. Only selected curing periods of 7, 28 and 90 days were used, omitting those cured for 180 and 365 days. The selection was based on preliminary trials which indicated that there were no major changes in TG/DTG traces after 90 days of curing, to warrant very many TG/DTG runs. In a separate TG/DTG regime, samples were mellowed for 1-3 days prior to drying and TG/DTG analysis at 1, 2 and 3 days of mellowing. In all the plots, the more informative DTG traces have been used, as opposed to both Thermogravimetric (TG) and Derivative Thermogravimetric (DTG) traces. There were various temperature zones where significant weight losses were observed, for both unstabilized and stabilized Lower Oxford Clay or Kaolinite.

1. **Zone 1** (< 100°C): This zone represents moisture loss, and therefore weight loss resulting from the expulsion of the chemically uncombined (i.e. adsorbed) water in the stabilized/unstabilized clay soil. This is the moisture in the material, that was not lost during the initial drying process using silica gel. The principal ettringite dehydration according to most reports (Giergiezny and Weryuska, 1989; Negro and Bachiorrini, 1989; De Silva and Glasser, 1990), occurs in the temperature region 70°C and 140°C. Thus the peaks between 70°C – 100°C may also be attributable to the ettringite, especially because the samples were thoroughly dried before TG analysis, removing most of the free water.

2. **Zone 2** (100°C-200°C): This zone represents weight loss due to water loss partly from ettringite as explained above and also from gypsum ($CaSO_4.2H_2O$). Gypsum loses combined water between 130°C and 170°C (one major peak at about 140°C and a smaller one at about 160°C). A similar

weight-loss peak (due to gypsum) over this temperature range was observed by Parson *et al.*, (1997) when analysing LOC using evolved gas analysis (EGA) techniques. Also occurring in this zone is the principal dehydration of the C-S-H phases (typically between 120°C-140°C) formed as hydration products of the pozzolanic reactions.

3. **Zone 3** (200°C-400°C): The report by Parson *et al.*, (1997) also suggested that organic matter is oxidised within this temperature region. It is thought that peaks due to the evolution of SO_2 are located between 300°C to 400°C, due to the oxidation of organic bound sulphur and also the oxidation of pyrites (Fe_2S).

4. **Zone 4** (400°C-650°C): This zone represents the loss of water due to the dehydroxylation of clay minerals. The dehydroxylation of clay minerals to weight losses occurring around 500°C. More specifically kaolinite loses its combined water at about 570°C (Liptay, 1974). Thus, as LOC contains about 10% kaolinite and 23% illite, significant weight losses are expected within this temperature region. El-Jazairi and Illston (1977), in their work on chemical shrinkage of hydrating cement paste, detected the dehydroxylation of lime between 400°C and 600°C. Thus, for stabilized materials where lime ($Ca(OH)_2$) is likely to be present, weight losses due to lime are also anticipated. Because of the possible overlap between clay and lime peaks within this temperature region, it is quite hard to quantify the amount of lime present in most hydrated lime-clay systems.

5. **Zone 5** (650°C-850°C) represents the loss of carbon dioxide (CO_2) evolved from calcium carbonate and other carbonates. The decomposition of sulphides, which is thought to occur as early as from 400°C to 950°C (Dunham *et al.*, 1992)), is also expected to contribute towards weight losses in this zone.

6.8.2 DTG of the Lower Oxford Clay (LOC) System

6.8.2.1 Effects of curing time

Figures 6.8-1 to 6.8-3 illustrate the DTG traces of the specimens that had been tested for UCS in both the mellowed and unmellowed conditions, after moist curing for 7, 28 and 90 days.

Plots of LOC stabilized with 2wt.% and 6wt.%wt of lime are presented in Figure 6.8-1(a) and (b). All sets of curves show broad low-temperature weight loss bands under 200°C, which may be attributed to the dehydration of calcium alumino-silicate hydrate (C-A-S-H) gels as well as ettringite and gypsum. The sharp peaks at about 100°C are most likely to be those of ettringite (too sharp to be any gel, and at too low temperature to be due to gypsum). Thus, the ettringite peaks increase with increasing curing time while the gypsum peaks disappear as curing time increases. Ettringite peaks increase sharply in the system with the higher lime level of 6wt.% (see Figure 6.8-1(b)). Weight losses thought to be due to calcium hydroxide were observed at 400°C to 450°C. The weight loss appears to be greater in the unmellowed condition. Increasing the curing period to 90 days caused a decrease in the weight loss band for lime. This is more evident at 6wt.% lime content. Weight losses due to the dehydroxylation of the clay were observed between 500°C and 650°C while carbonation peaks were observed between 650°C and 750°C.

DTG traces of WSA-stabilized LOC with 20%wt of stabilizer are shown in Figure 6.8-1(c). The ettringite peaks increase as curing time increases and there are little or no traces of gypsum. Lime is available in the system, but is slowly depleted.

Figures 6.8-2(a) and (b) show DTG analysis results for WSA-lime-stabilized LOC at 90:10 and 80:20 blending ratios at 20wt.% of stabilizer content. Both systems display similar trends, where ettringite peaks are clearly shown to increase with curing time. There are no definite gypsum peaks in the stabilized systems, except in the mellowed 80:20 system at 7 and 28 days of curing. These peaks fade at 90 days. There are definite lime traces in both systems with the most obvious peaks being observed in the 7 day cured, 80:20 unmellowed blends (see Figure 6.8-2(b)). In this system the carbonation peaks are marginally smaller compared with those in the lime-stabilized LOC system (see Figure 6.8-1(a) and (b)).

DTG traces of WSA-PC-stabilized LOC are shown in Figure 6.8-2(c) and (d). Similar trends were observed to those in the 90:10 WSA-lime system. However, the lime weight loss peaks are relatively less pronounced and it is hard to establish whether they decrease as curing time increases as would be expected. The ettringite peaks increase with increasing curing time, especially in the unmellowed system. Gypsum traces were not detected in this system, but carbonation peaks are still very pronounced.

Figures 6.8-3(a) and (b) illustrate the DTG traces of WSA-GGBS-stabilized LOC at 70:30 and 50:50 blending ratios at 20wt.% of stabilizer. Increasing the curing period to 90 days generally increased the ettringite peaks (at 100°C). The lime peaks are not very prominent, but appear to decrease with curing time. The decrease in lime suggests that part of the lime was used up by GGBS in the system and/or pozzolanic reactions. The carbonation peaks are still present and become marginally smaller as curing time increases.

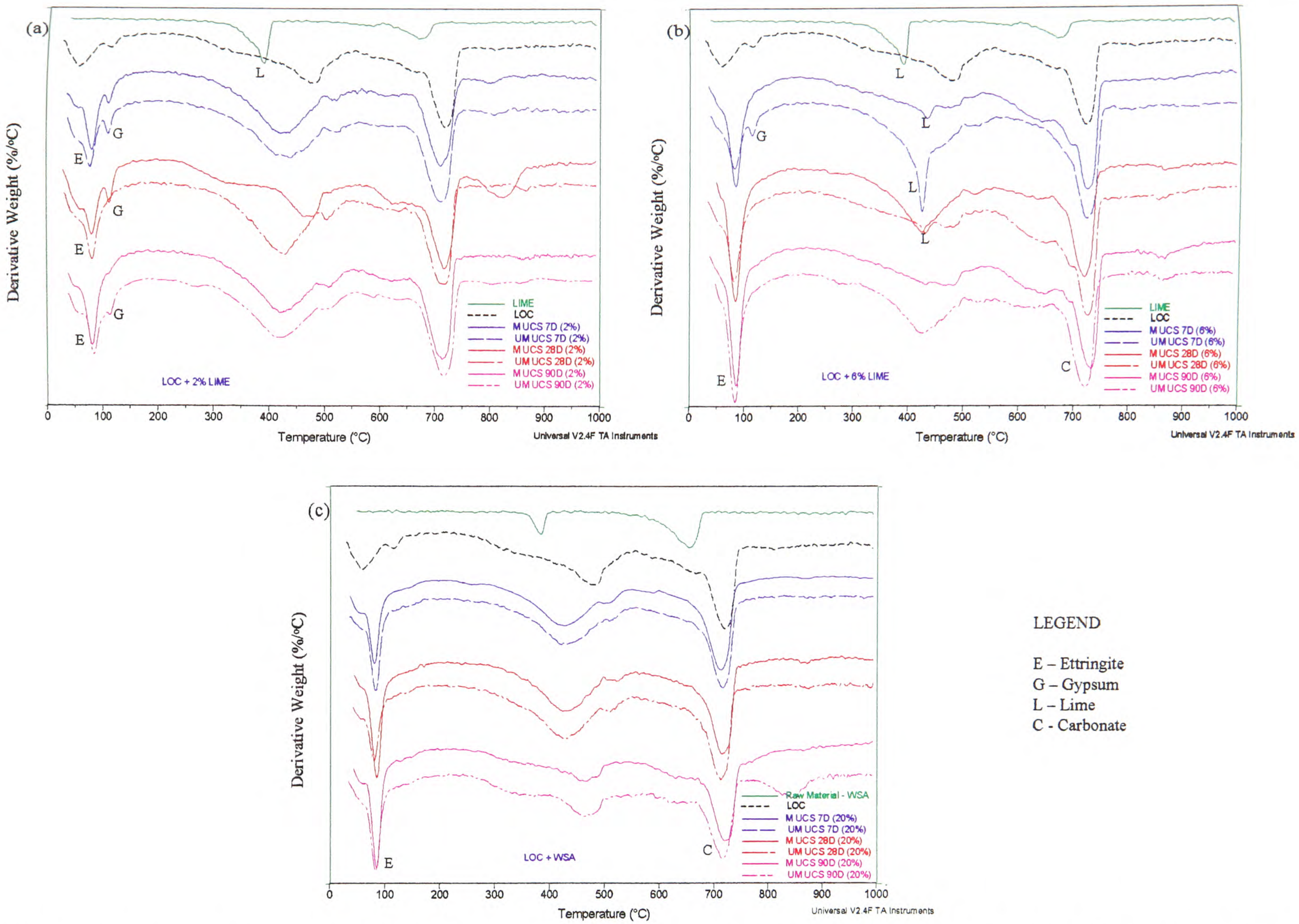


Figure 6.8-1: DTG traces of lime-stabilized LOC at (a) 2wt.% lime (b) 6wt.% lime and WSA-stabilized LOC at (c) 20wt.% WSA for both mellowed (M) and unmellowed (UM) condition for 7, 28 and 90 days curing period.

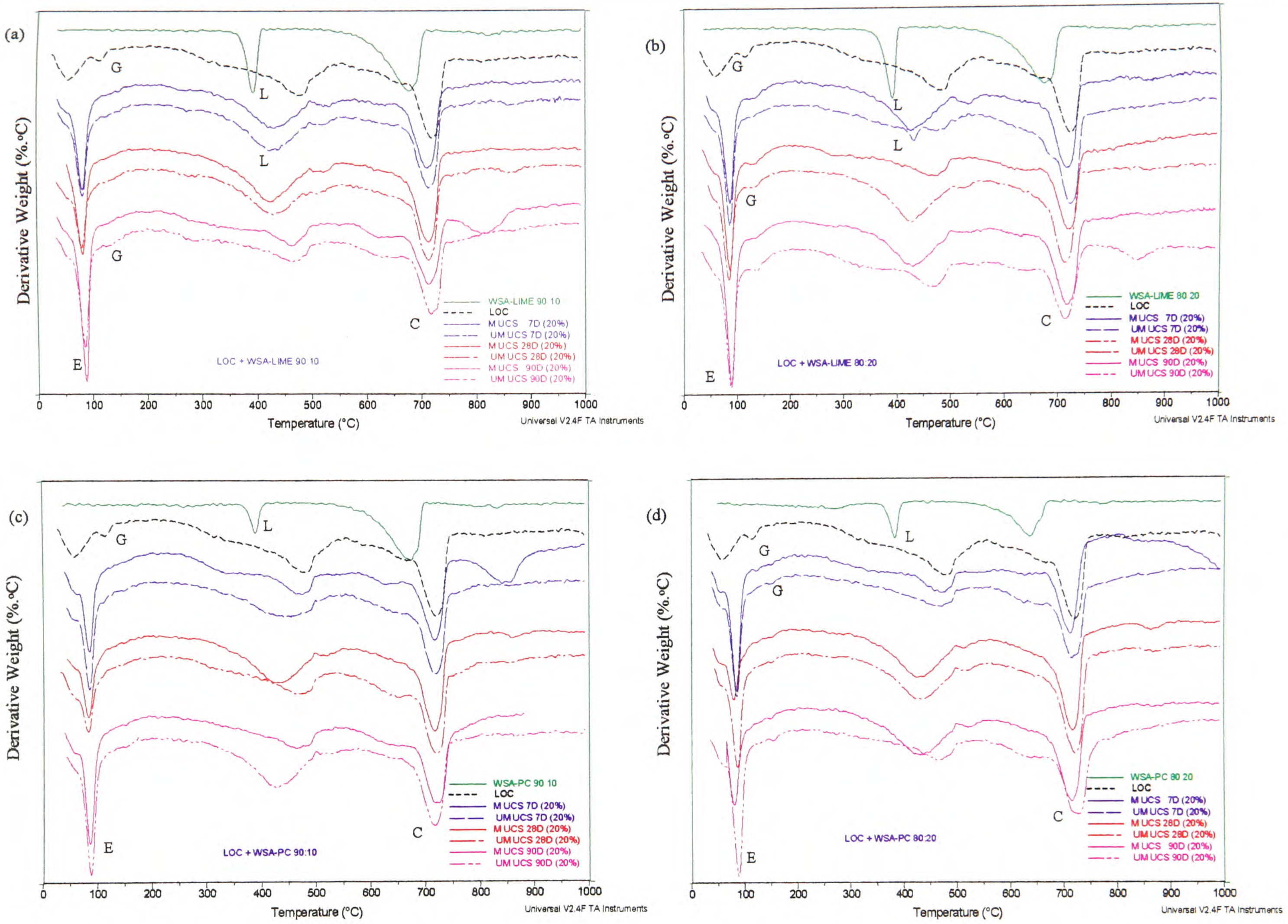


Figure 6.8-2: DTG traces of WSA-lime-stabilized LOC at (a) 90:10 and (b) 80:20 blending ratios and WSA-PC stabilized LOC at (c) 90:10 and (d) 80:20 blending ratios for both mellowed (M) and unmellowed (UM) condition for 7, 28 and 90 days curing period.

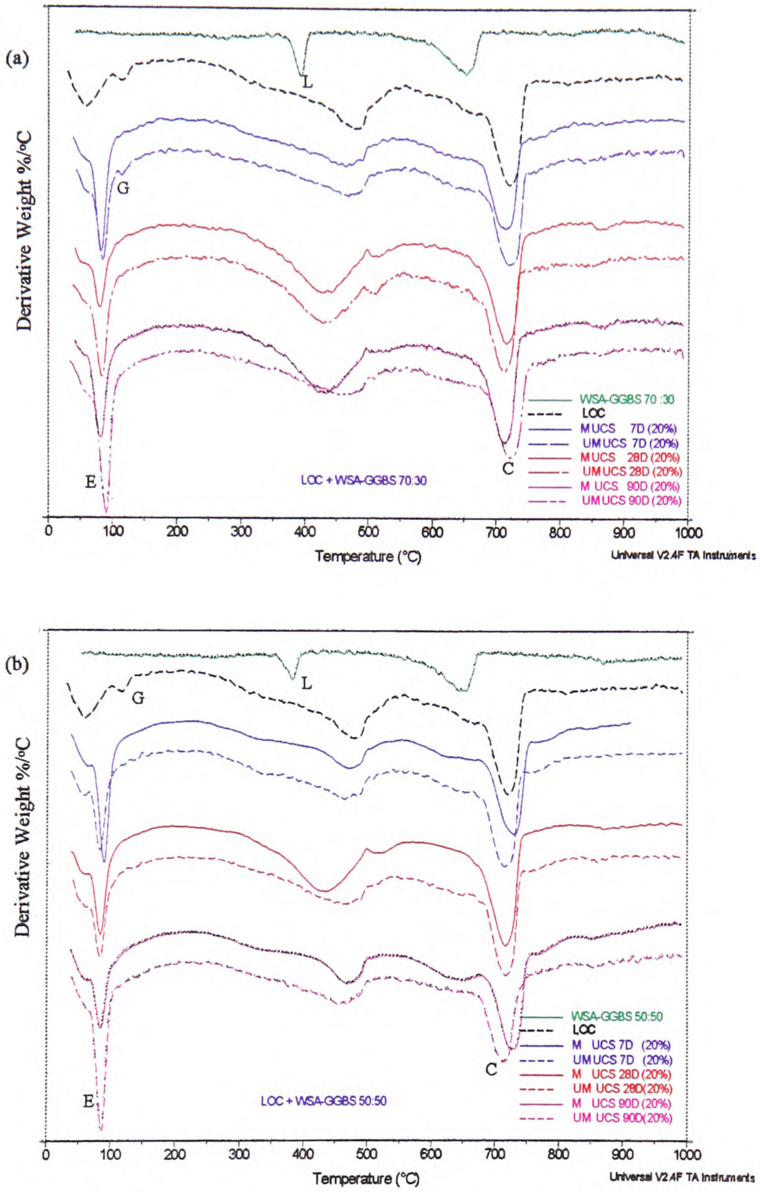


Figure 6.8-3: DTG traces of WSA-GGBS stabilized LOC at (a) 70:30 (b) 50:50 blending ratios for both mellowed (M) and unmellowed (UM) condition for 7, 28 and 90 days curing period.

6.8.2.2 Effects of mellowing on DTG traces of cured specimens

Figures 6.8-4 to 6.8-6 show DTG traces of LOC containing various stabilizers, after mellowing for 1, 2 and 3 days. The Figures also show DTG traces of the dry unstabilized LOC, and of the dry stabilizers (lime, WSA) and the blended stabilizers used in the study (WSA-lime, WSA-PC, at 90:10 and 80:20 blending ratios) and WSA-GGBS (at 70:30 and 50:50 blending ratios). The dry mixtures were tested to obtain the composition of the mixes prior to their hydration, and to provide reference standards to which the mellowed reacted material could be compared. To investigate the reactions taking place during the 3 days of mellowing, the target material (LOC) was thoroughly mixed with the blended stabilizers before mixing with the water at the appropriate OMC. The wet mixtures were then sealed in plastic bags and a little sample taken at 3 consecutive days and dried under silica gel in a desiccator at 40°C until constant weight was achieved before DTG analysis.

Figures 6.8-4(a) and (b) show the DTG peaks for LOC + lime. Similar to the specimens cured for 7, 28 and 90 days discussed in sub Section 6.8.2.1, the temperature peaks below 200°C are attributed to the dehydration of hydration products (Calcuim- Alumino-Silicate Hydrate (C-A-S-H)) gels. In addition, because of the presence of sulfate in LOC, the formation of Calcium-Sulfo-Aluminate Hydrates (C-A-$\bar{S}$-H e.g. ettringite) is also expected. The bands at 400°C to 550°C are respectively the result of dehydration of lime and/or of the dehydration of the clay in the stabilized mixture. The peaks attributed to ettringite and gypsum which are formed during the mellowing period are discussed in detail in the next Section (6.8.2.3). There is evidence of residual lime in the system stabilized with 6wt.% lime, with the amount of lime observed decreasing with increasing mellowing period. With 2wt.% lime, no lime is apparent even after the first day of mellowing, whereas with 6wt.% lime, lime appears to persist throughout the mellowing period.

Figures 6.8-4(c) and (d) illustrate the traces for LOC + WSA at 10wt.% and 20wt.% of stabilizer. The ettringite peaks appear to increase with increasing mellowing period, while those of gypsum appear to be decreasing. There are possible traces of lime on the 1st, 2nd and 3rd day of mellowing for the material stabilized with 20wt.% WSA (see Figure 6.8-4(d)). With 10wt.% WSA, there is little evidence of the presence of lime.

In the discussion Section in Chapter 7, an attempt is made to explain the appearance and disappearance of evidence of lime in the mellowed stabilized mixtures.

The DTG traces of LOC + WSA-lime at 90:10 and 80:20 blending ratios at 20wt.% stabilizer content are shown in Figures 6.8-5(a) and (b). In both systems, both ettringite peaks appear to increase with increasing mellowing period. Gypsum is also clearly present, with the 90:10 stabilizer blend clearly suggesting increase in gypsum with increasing mellowing period. This increase in gypsum is not certain in the 80:20 system. Infact the gypsum peaks appear to be decreasing with increasing mellowing period (see Figure 6.8-5(b)). Lime peaks overlap with other peaks from the clay, and only where lime is clearly evident is this reported. This is further made difficult by the shifting of the clay band as the clay is "attacked" by the lime during hydration. Lime is evident in both systems after 1 day of mellowing and absent (again in both systems) in day 2. Interestingly, lime is again clearly evident in day 3 of mellowing in the material containing the 80:20 WSA:lime blend (which is richer in lime (see Figure 6.8-5(b)). It is clear that there is secondary lime in the 80:20 system. In the discussion in Chapter 7, it is hypothesised that this secondary lime formation is as a result of hydration of WSA, and/or release of lime (Ca^{2+} ions) from the flocculated mellowed material. The carbonation peaks are also shown between 650°C and 850°C.

As with WSA-lime, DTG traces of LOC + WSA-PC (at 90:10 and 80:20) at 20wt.% stabilizer content also shows clear evidence of ettringite and gypsum (see Figure 6.8-5(c) and (d)). Minor traces of lime may be present in both systems as mellowing time increases. The carbonation peaks are comparable in both systems.

Figures 6.8-6(a) and (b) illustrate the DTG traces of LOC + WSA-GGBS (at 70:30 and 50:50 blending ratios) at 20wt.% of blended stabilizer. There is evidence of ettringite and gypsum in both blending ratios. There appears to be higher ettringite peaks (relative to those of gypsum) in the 70:30 blend compared to the 50:50 blend. The latter appears to have higher gypsum peaks (relative to the ettringite peaks) compared with the 70:30 blend. Therefore, the higher amount of WSA in the 70:30 blend appears to be associated with ettringite formation, and to consumption of gypsum. Traces of lime appear to be present after the 1st day of mellowing for the 70:30 blend. Neither of the blends show significant lime after mellowing for 2 days.

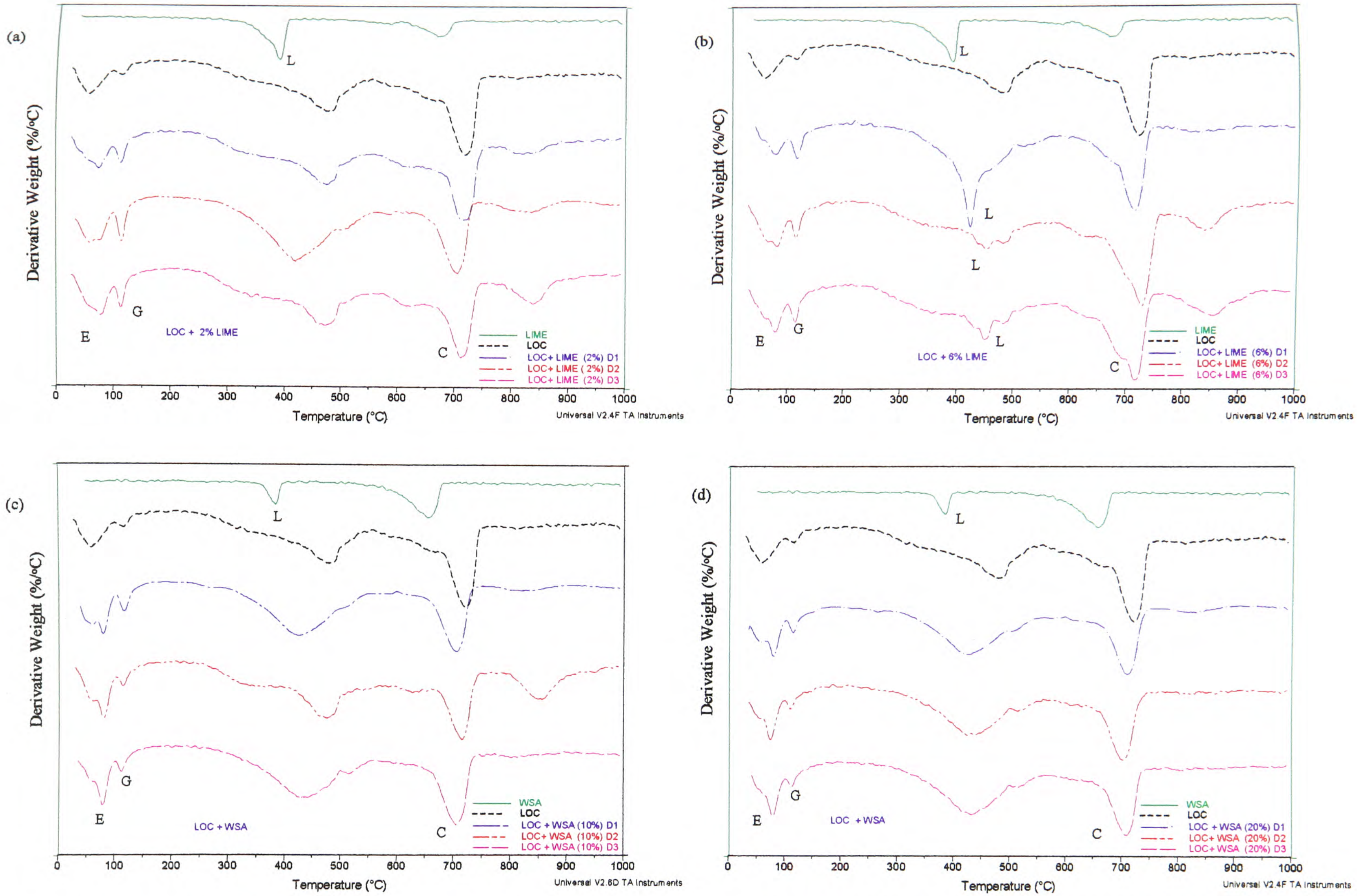


Figure 6.8-4: DTG traces at different mellowing period (1-3 days) for lime-LOC stabilized with (a) 2wt.% lime and (b) 6wt.% lime and WSA-LOC stabilized with (c) 10wt.% WSA and (d) 20wt.%WSA

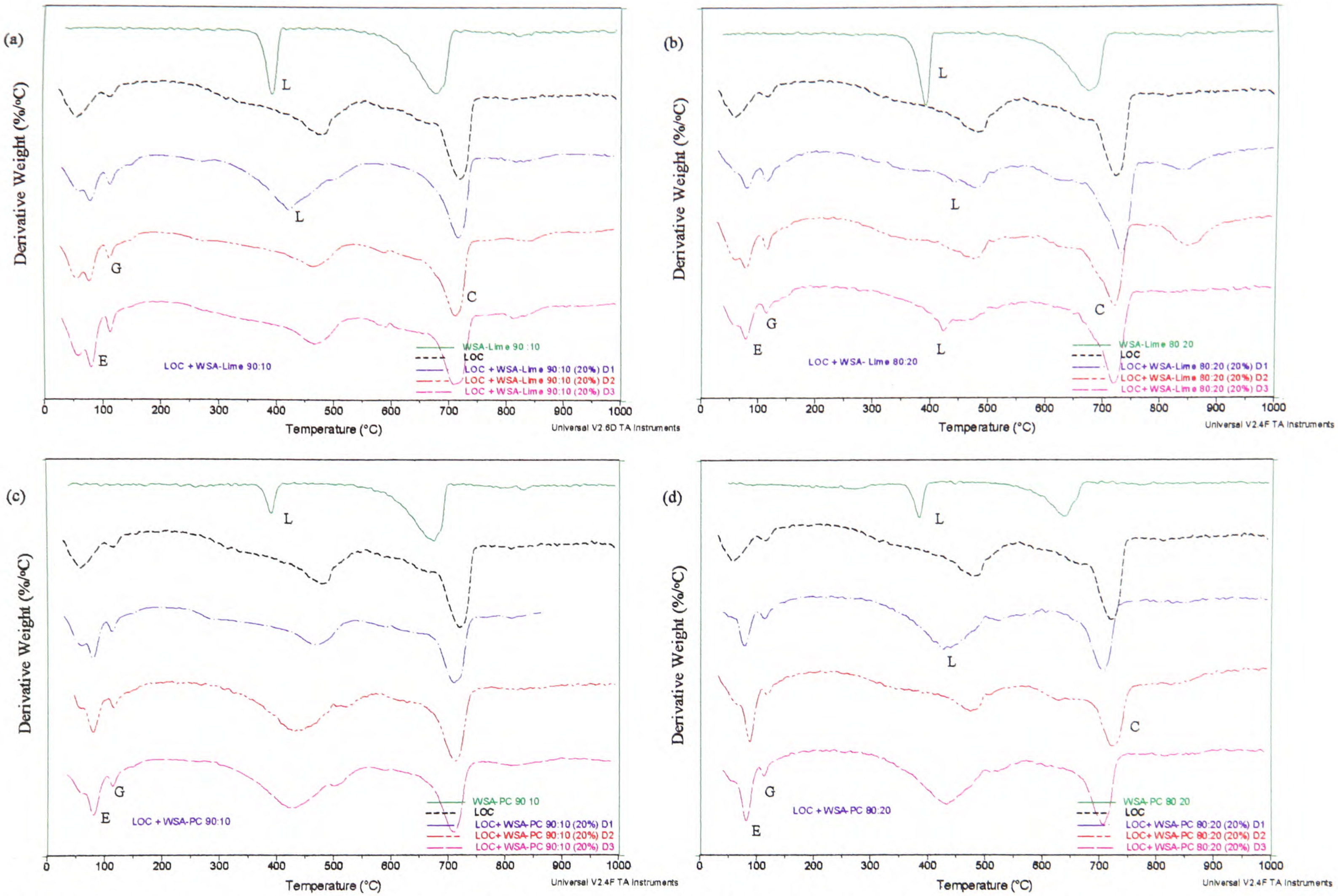


Figure 6.8-5: DTG traces at different mellowing periods (1-3 days) for LOC stabilized with (a) & (b) WSA-lime blends (at 90:10 and 80:20 ratios), and (c) &(d) with WSA-PC blends (at 90:10 and 80:20 ratios).

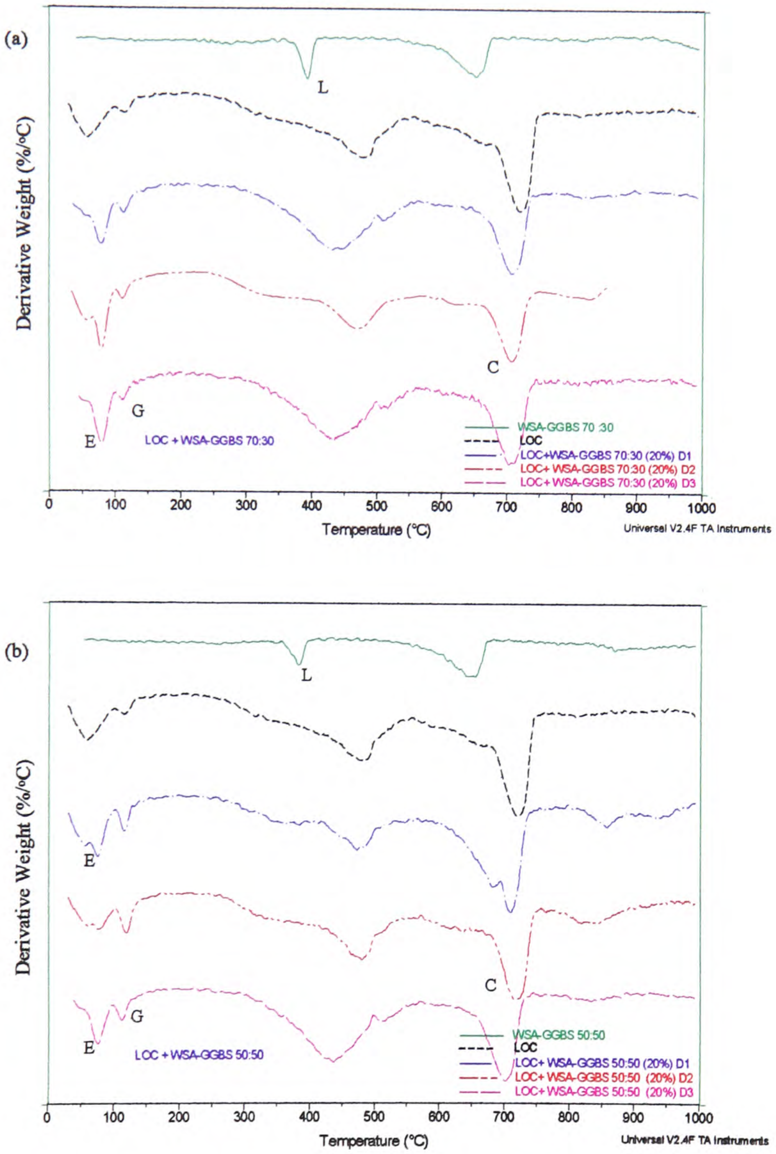


Figure 6.8-6: DTG traces at different mellowing periods (1-3 days) for LOC stabilized with WSA-GGBS blends at (a) 70:30 and (b) 50:50 blending ratios.

6.8.2.3 Quantities of Ettringite and Gypsum during mellowing and subsequent curing.

Figure 6.8-7(a) and (b) illustrate the estimated levels of Ettringite and Gypsum in the lime-stabilized LOC system after moist curing for 7, 28 and 90 days, with and without a 3 day mellowing period. After the first day of mellowing and throughout the 90 days of moist curing, the formation of ettringite appears to increase with the lime content. It is also apparent that during curing, the mellowed system generally produced more ettringite relative to the unmellowed system. This is more evident at the higher lime content of 6wt.%. During the initial 3-day mellowing period, some lime and already existing gypsum are used towards the formation of ettringite. The formation of ettringite is clear from Figure 6.8-7(c), but the consumption of gypsum (see Figure 6.8-7(d)) is masked by the production of gypsum from the on-going oxidation of LOC. In the unmellowed system, the 3-days headstart mellowing period when some ettringite is formed did not exist leading to a lower ettringite level during curing. The lack of significant increase in ettringite formation after the first day of mellowing is thought to be due to the progressive depletion of readily available gypsum in the system, prior to the slow (secondary) gypsum production from the oxidation of LOC. The secondary gypsum, formed due to oxidation of sulfide in the LOC and any residual lime, are used to sustain ettringite formation during curing. This explains why the system stabilized with more lime shows higher ettringite formation. A system higher in lime will not only provide more lime for ettringite formation, but will also enhance the oxidation of LOC to produce gypsum, further boosting ettringite formation.

At high lime levels (6wt.%) in the lime-LOC system, gypsum is depleted within the first 7 days in the mellowed system whereas in the unmellowed system, the consumption of gypsum is slower and gypsum lasts for at most 28 days of curing (see Figure 6.8-7(b)). On the other hand at the low lime content (2%), gypsum is consumed more slowly, such that in the mellowed system, where all the lime is consumed almost immediately after mixing and compaction (notice absence of lime peak in DTG traces in Figure 6.8-1(b)), gypsum lingers until and after 90 days of curing.

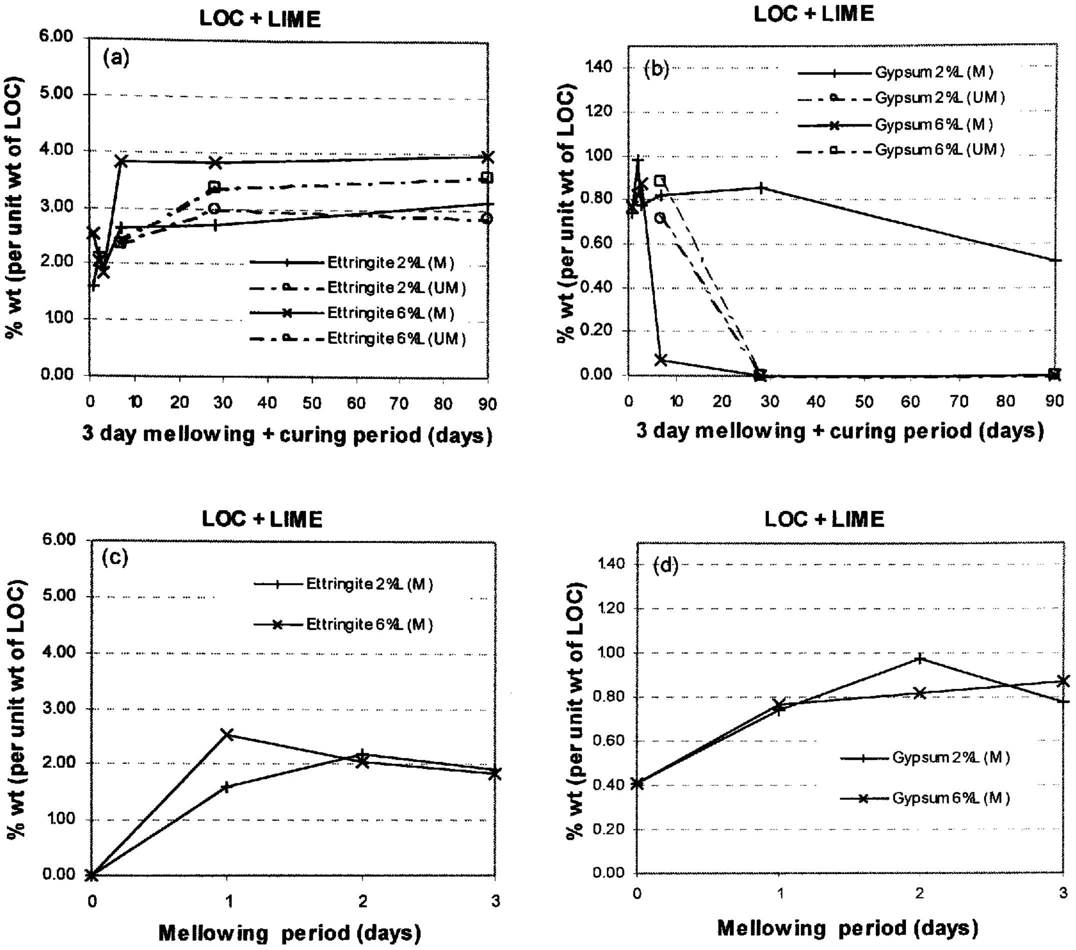


Figure 6.8-7: Estimated quantities of Ettringite and Gypsum from DTG analyses of lime-stabilized LOC, after moist curing for up to 90 days (a and b) and after mellowing for up to 3 days (c) and (d).

DTG work on cured specimens using WSA or WSA-blends was only carried out at the higher stabilizer content of 20wt.%, as the implications of low lime contents were obvious from the observation in the lime-LOC system. In the WSA-stabilized LOC (see Figure 6.8-8(a) to (d)), as in the lime-LOC system, ettringite formation is higher in the mellowed specimens compared to the unmellowed ones. Most of the ettringite is formed during the first 28 days of curing, when most free lime is drawn from the hydration of WSA. There is a drop in the formation of ettringite after the 1st day of mellowing (see Figure 6.8-8 (c)) due to the depletion of the initial quantities of free lime and also of gypsum reserves present in the LOC. Thus, most of the gypsum was consumed within the first 7 days. Unlike in the lime-LOC system, in the WSA-system, no increase in gypsum was noticed during mellowing (compare Figure 6.8-8(d) with 6.8-7(d)). This was due to little lime being available that would induce the oxidation of Lower Oxford Clay.

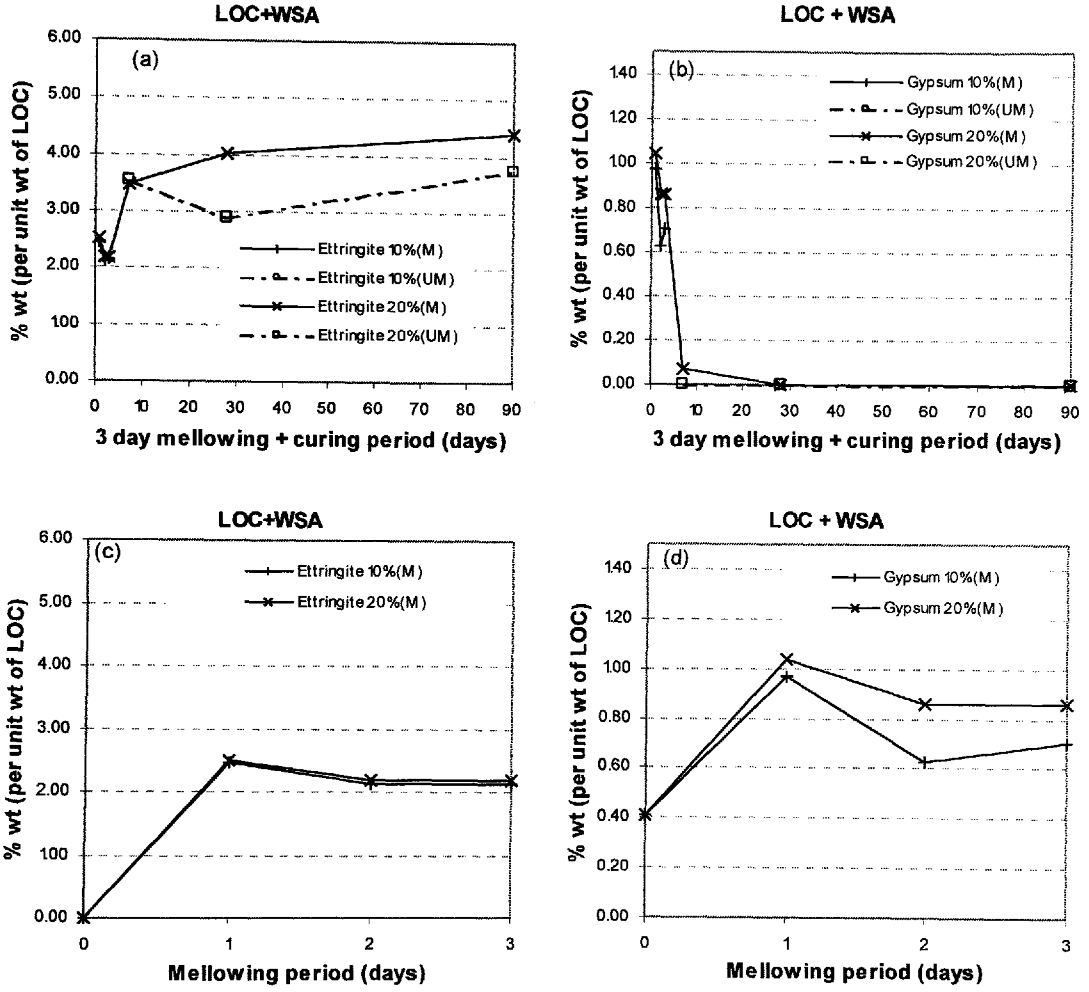


Figure 6.8-8: Estimated quantities of Ettringite and Gypsum from DTG analyses of WSA-stabilized LOC, after moist curing for up to 90 days (a and b) and after mellowing for up to 3 days (c) and (d).

Blending WSA with lime also shows trends of ettringite formation that are closely similar to those in the lime-stabilized LOC system (compare Figure 6.8-9(a) and (c) with Figure 6.8-7(a) and (c)). This is probably due to enhanced availability of lime in the WSA system, thus promoting the production of ettringite. With the 90:10 (WSA-lime) stabilizer ratio (see Figure 6.8-9(a)), there is less ettringite formation during the first 28 days of curing compared with the 80:20 (WSA-lime) system (see Figure 6.8-10(a)). There is also no evidence of secondary gypsum after the mellowing period (see Figure 6.8-9 (b)).

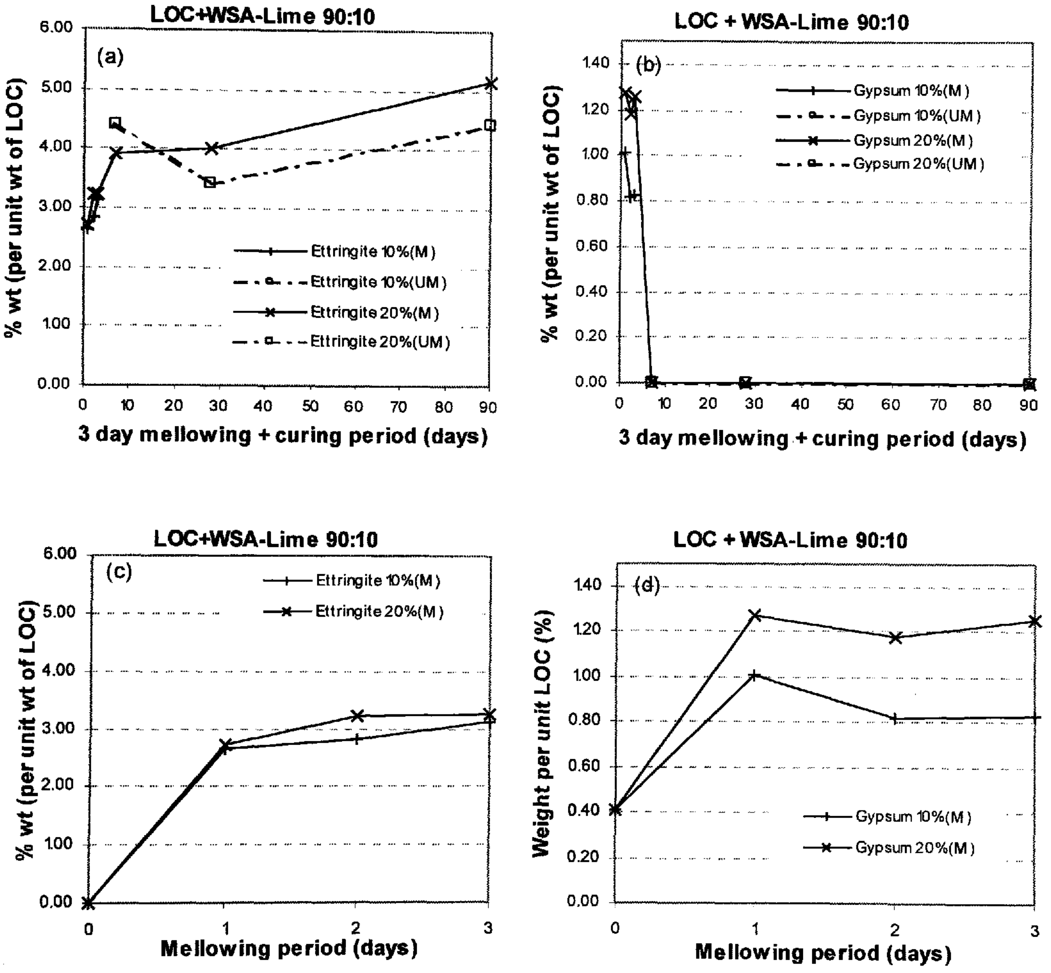


Figure 6.8-9: Estimated quantities of Ettringite and Gypsum from DTG analyses of WSA-lime-stabilized LOC (at 90:10 blending ratio), after moist curing for up to 90 days (a and b) and after mellowing for up to 3 days (c) and (d).

The lime available in the more lime-rich 80:20 system ensured a higher amount of initial ettringite formation as well as prolonged production of gypsum after its initial consumption during the first 7 days of mellowing and curing (see Figure 6.8-10(b) and (d)). However this secondary gypsum is also consumed in the formation of ettringite. Therefore, sustained presence of gypsum is dependent on the availability of lime in the curing system. Thus at 20wt.% stabilizer content, depletion of gypsum is quite rapid in the mellowed system where there is little lime to oxidise the LOC, rather than in the unmellowed system where compaction without mellowing is thought to allow a prolonged (albeit slower) oxidation of LOC to produce gypsum (see Figure 6.8-10(b)).

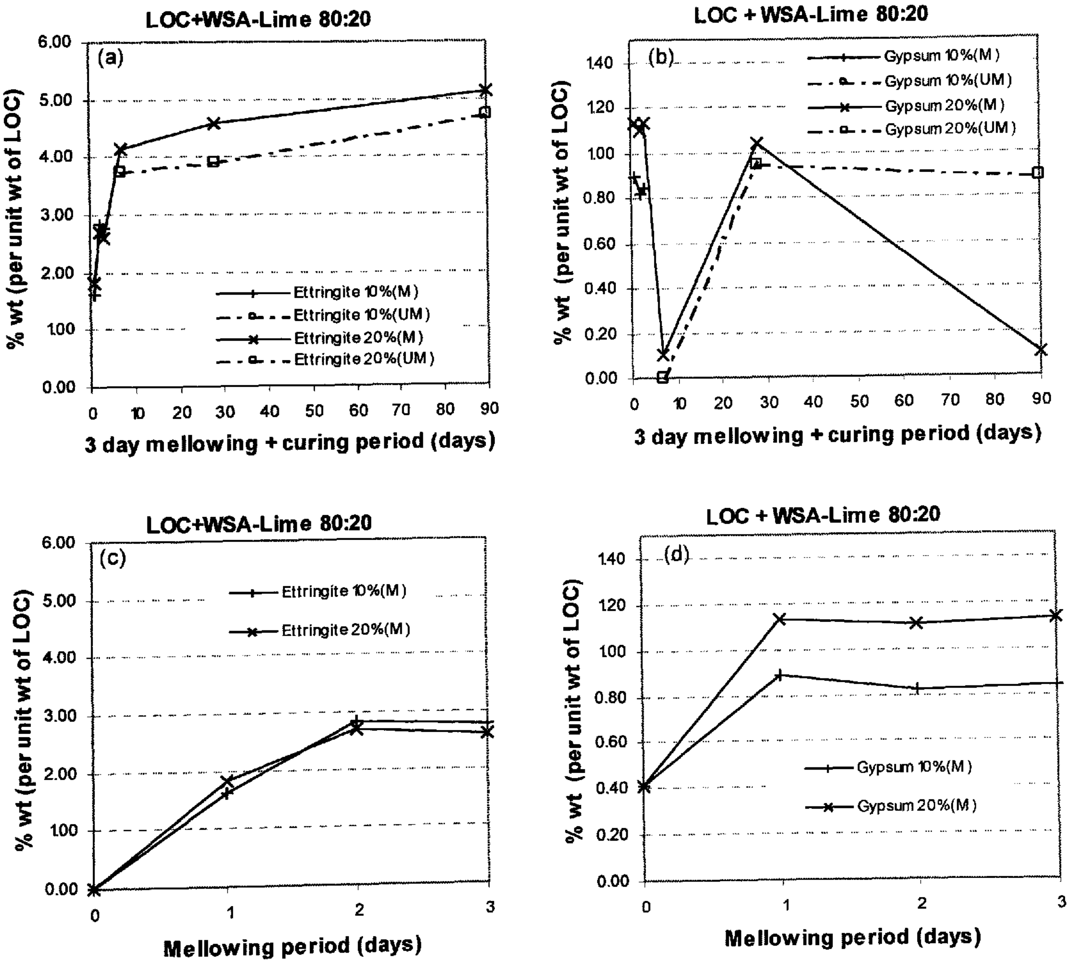


Figure 6.8-10: Estimated quantities of Ettringite and Gypsum from DTG analyses of WSA-lime-stabilized LOC (at 80:20 blending ratio), after moist curing for up to 90 days (a and b) and after mellowing for up to 3 days (c) and (d).

Blending WSA with PC at 90:10 and 80:20 (WSA-PC) stabilizer ratios (see Figure 6.8-11 and 6.8-12) resulted in a scenario closely similar to the WSA-lime stabilized LOC system, in that there was rapid formation of ettringite during the first 7 days. One significant difference with the WSA-lime system, however, is the significant drop in ettringite after 7 days in both mellowed and unmellowed WSA-PC systems. This is attributed to the fact that unlike in the WSA-lime system where lime is readily available, in the WSA-PC system, hydration of PC has first to take place to produce lime. This may cause a slump in the rate of both ettringite formation and of oxidation of LOC.

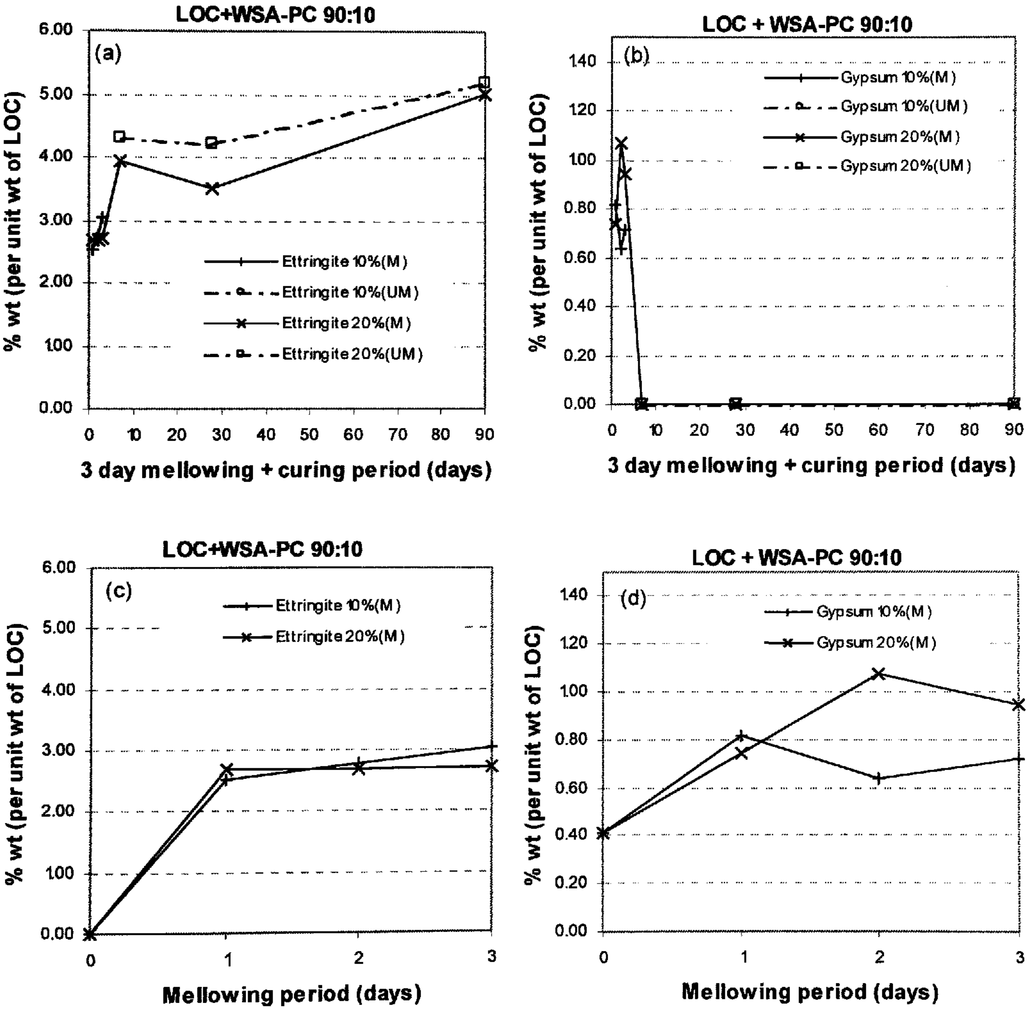


Figure 6.8-11: Estimated quantities of Ettringite and Gypsum from DTG analyses of WSA-PC-stabilized LOC (at 90:10 blending ratio), after moist curing for up to 90 days (a and b) and after mellowing for up to 3 days (c) and (d).

Another significant difference is the fact that there was more ettringite in the unmellowed system after the initial 7 days of curing. This is also possibly attributable to the fact that PC hydration during this time has a significant influence in the system, by producing lime and hence more ettringite in the unmellowed system. In the mellowed system, these reactions take place early. With the 90:10 blend, gypsum was depleted more rapidly (within 7 days) compared with the 80:20 blend, where secondary gypsum was produced using the more lime available in the system. Thus with the more lime-rich 80:20 system, gypsum lingers until 28 days of curing.

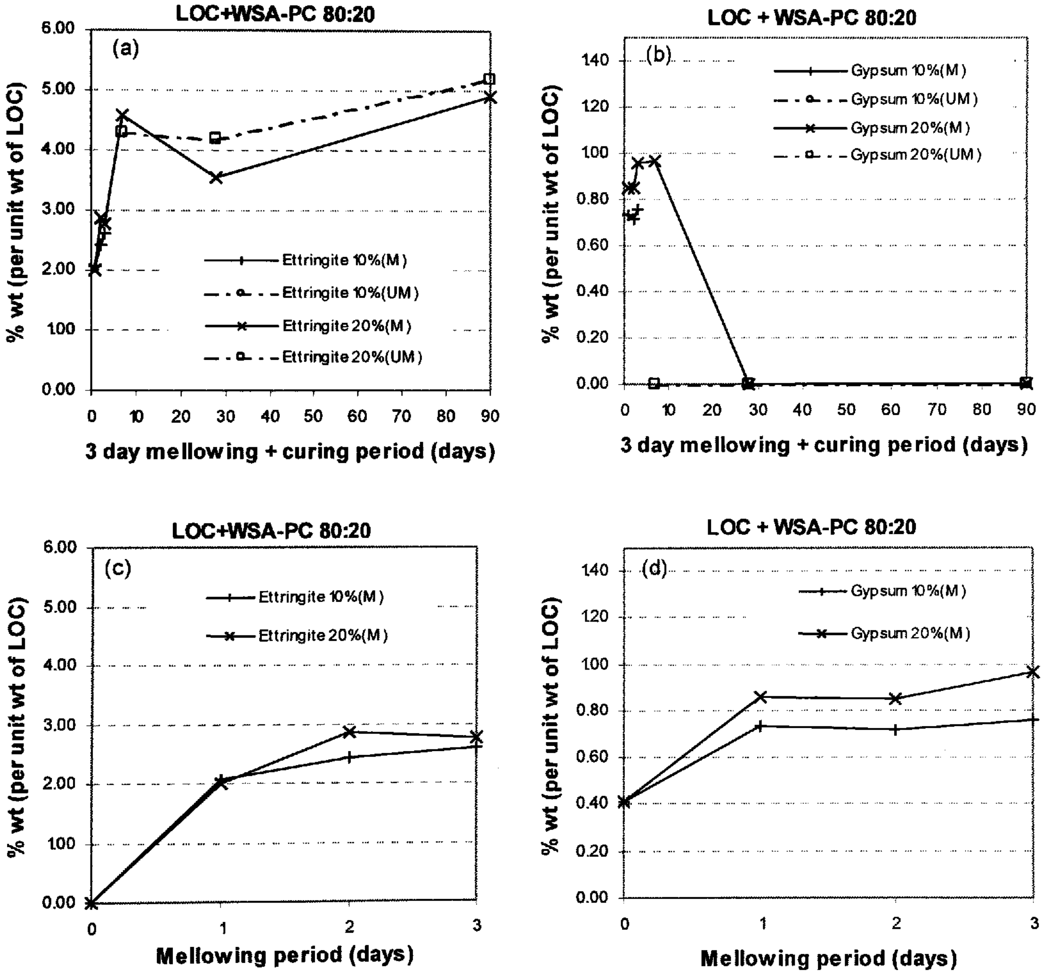


Figure 6.8-12: Estimated quantities of Ettringite and Gypsum from DTG analyses of WSA-PC-stabilized LOC (at 80:20 blending ratio), after moist curing for up to 90 days (a and b) and after mellowing for up to 3 days (c) and (d).

Figures 6.8-13 and 6.8-14 illustrate the effects of blending WSA with GGBS, in the stabilization of LOC. Using higher WSA and lower GGBS content (at 70:30 (WSA-GGBS) ratio), resulted in a slump in the ettringite level between 7 and 28 days of curing, followed by a gentle increase in ettringite formation. In the 50:50 (WSA-GGBS) blend (relatively lower in WSA (and hence lime) and higher in GGBS content), there was no significant increase in ettringite formation beyond 7 days of curing.

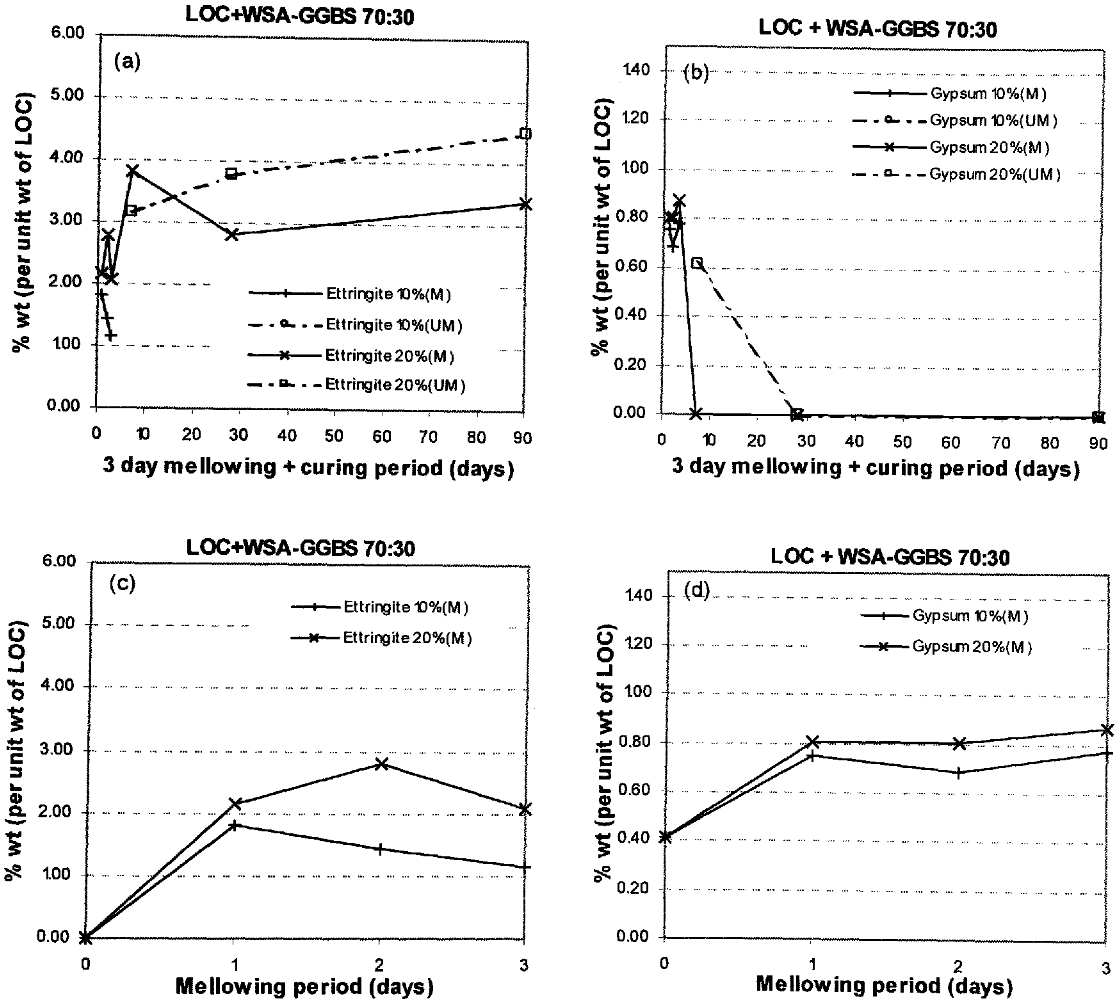


Figure 6.8-13: Estimated quantities of Ettringite and Gypsum from DTG analyses of WSA-GGBS-stabilized LOC (at 70:30 blending ratio), after moist curing for up to 90 days (a and b) and after mellowing for up to 3 days (c) and (d).

It is postulated that the GGBS consumed most of the available lime in the system during the 3 days of mellowing thus reducing the availability of lime for ettringite formation. As for systems already discussed, where lime is low, no secondary gypsum was produced. This was the case for both 70:30 and the 50:50 WSA-GGBS blends. As in the WSA-PC system, the unmellowed WSA-GGBS systems recorded higher ettringite formation beyond 28 days of moist curing, relative to the mellowed system. However, for the WSA-GGBS systems, this difference in ettringite formation between the mellowed and the unmellowed systems is likely to be due to the more significant suppression of ettringite formation in the mellowed system as a result of consumption of lime. In the WSA-PC system, the difference was due to the hydration of PC in the system.

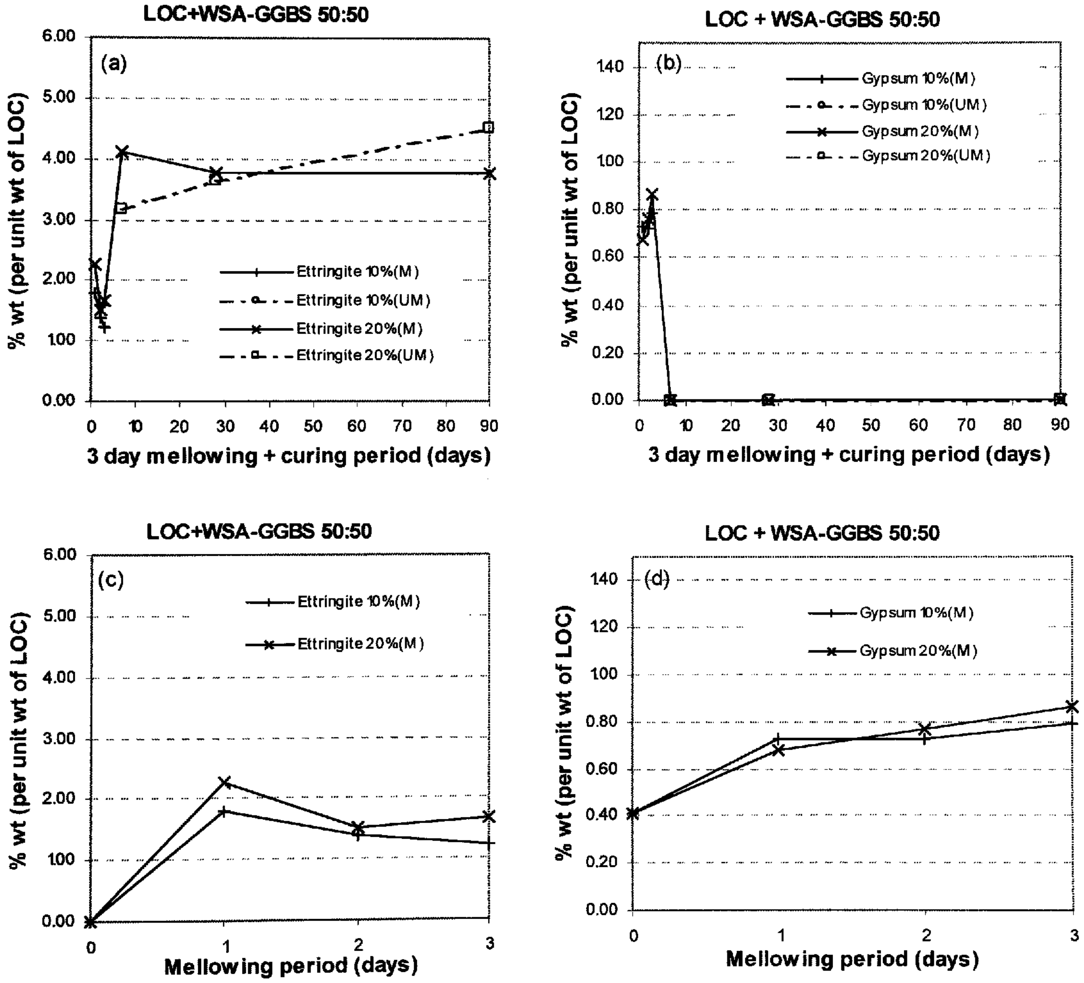


Figure 6.8-14: Estimated quantities of Ettringite and Gypsum from DTG analyses of WSA-GGBS-stabilized LOC (at 50:50 blending ratio), after moist curing for up to 90 days (a and b) and after mellowing for up to 3 days (c and d).

6.8.3 DTG of the Kaolinite System

Thermogravimetric and Derivative Thermogravimetric (TG/DTG) analyses were also carried out on stabilized kaolinite. Analysis was only carried out on samples where the highest stabilizer dosage was used, i.e. 6wt.% for lime and 20wt.% for the blended stabilizers. Test samples were selected from specimens that had undergone testing for UCS, after curing for 7, 28 and 90 days.

Being a processed and purified industrial clay, the mineralogical composition of kaolinite shows it to contain (84%) kaolinite clay content and little or no organic material or calcite (Table 4.3 in Chapter 4). This is clearly reflected in the DTG trace of unstabilized kaolinite which is presented in each of the graphs in Figure 6.8-15 to 6.8-17 as a baseline for the stabilized material. In contrast, LOC (a natural clay) contains 7% organic material, 4% pyrite 2% gypsum and only about 23% kaolinite clay (Table 4.1 in Chapter 4). Therefore there are significant differences in the TG/DTG traces of stabilized kaolinite compared with those of stabilized LOC.

In all the kaolinite-stabilizer systems, the DTG traces show less activity in the low temperature region (below 200°C) compared with traces for stabilized LOC. However, DTG traces of lime-stabilized kaolinite at 6wt.%, lime, WSA-stabilized kaolinite at 20% WSA, and WSA-lime stabilized kaolinite (at 90:10 and 80:20 ratios, Figure 6.8-15 and 6.8-16(a) and (b)) show some appreciable activity at temperatures below 200°C. This is due to the dehydration of hydration products, principally C-A-H and C-A-S-H phases. In the 6wt.% lime-stabilized kaolinite (see Figure 6.8-15(a)), there are traces of lime at 7 days of curing, more evident in the unmellowed condition. Their disappearance in both the mellowed and unmellowed condition at both 28 and 90 days curing is understandable, as a result of prolonged pozzolanic reactions.

In all the stabilized systems incorporating WSA, the only evidence of lime is in DTG traces of the dry stabilizers. These peaks disappear in the stabilized system as the curing progresses. This is due to the free lime from WSA being used for the hydration process.

Broad peaks were observed at temperatures between 450°C to 550°C in the entire stabilized system, which are attributed to dehydration of significant amounts of clay present in the system. The peaks are present at all curing periods, 7, 28 and 90 days, and in both mellowed and unmellowed conditions (see Figure 6.8-15 to 6.8-17).

6.8.4 Summary

1. In lime, WSA and WSA-lime-stabilized LOC the mellowed system generally produced more ettringite relative to the unmellowed system. It is suggested that this is due to most of the lime and the already existing gypsum plus that formed during mellowing being used towards the formation of ettringite during the initial 3-day mellowing period.

2. DTG traces of lime and gypsum become smaller and ultimately disappear with increasing mellowing and curing period. This is due to lime and gypsum being consumed over the curing period.

3. In the WSA-PC stabilized LOC system where lime is readily available during prolonged curing due to hydration of PC, ettringite levels are higher than in the unmellowed system.

4. In the kaolinite system, there is only little evidence of activity in the low temperature region below 200°C with some peaks due to C_2ASH_8. This is perhaps due to the inability of the DTG analysis to clearly detect non-crystalline gels which lose water over a broad temperature range, compared to the decomposition of the more crystalline ettringite and gypsum (in the LOC system), which lose water over a narrow temperature range. Broad clay peaks were however observed over the temperature zone 450°C to 550°C in all stabilized kaolinite systems.

5. There is evidence of lime after some period of curing in some systems. It is hypothesised that this lime results from the hydration of stabilizers that produces lime upon hydration (e.g. WSA and WSA blends with lime or PC).

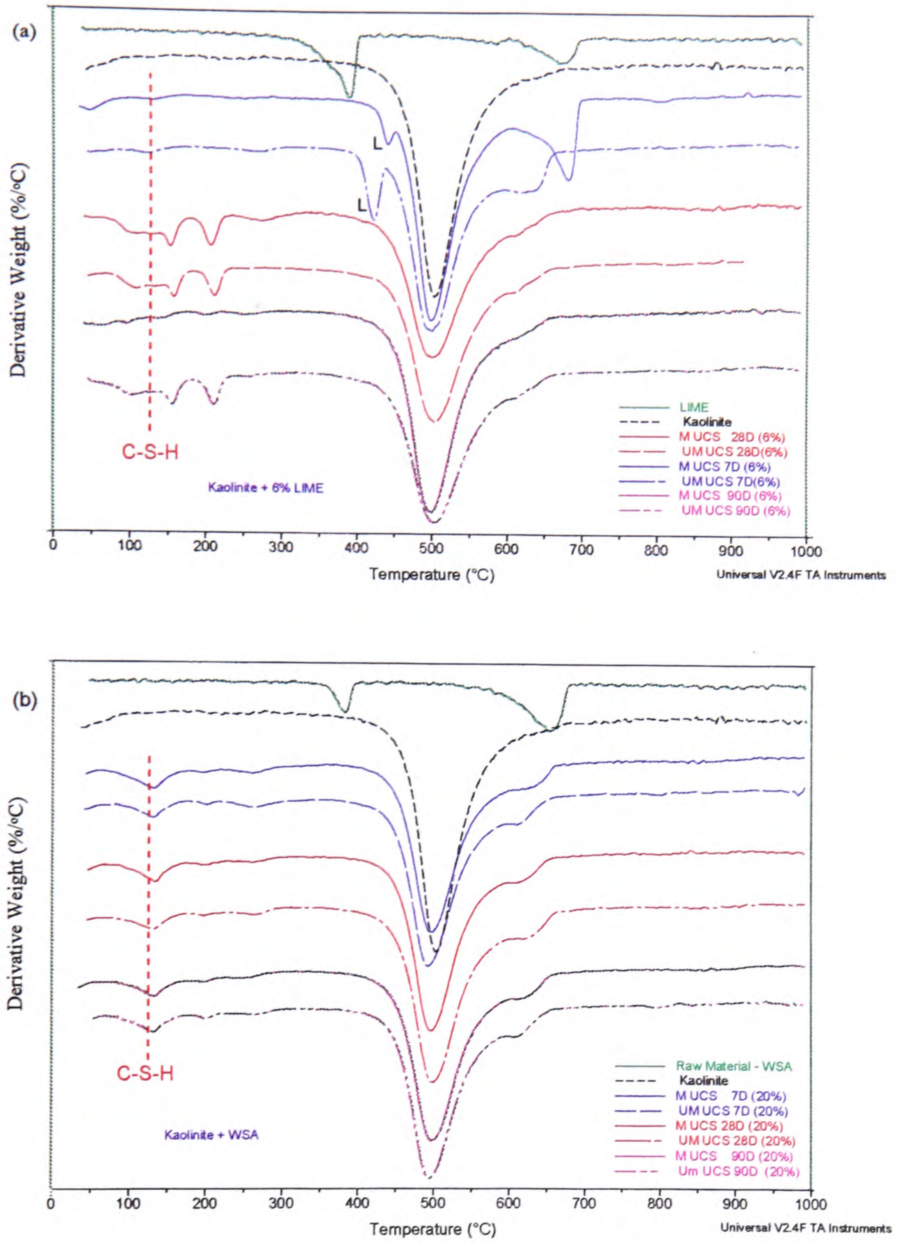


Figure 6.8-15: (a) DTG traces of lime-stabilized kaolinite at 6wt.% and (b) WSA-stabilized kaolinite at 20wt.% for both mellowed and unmellowed condition for 7, 28 and 90 days curing period.

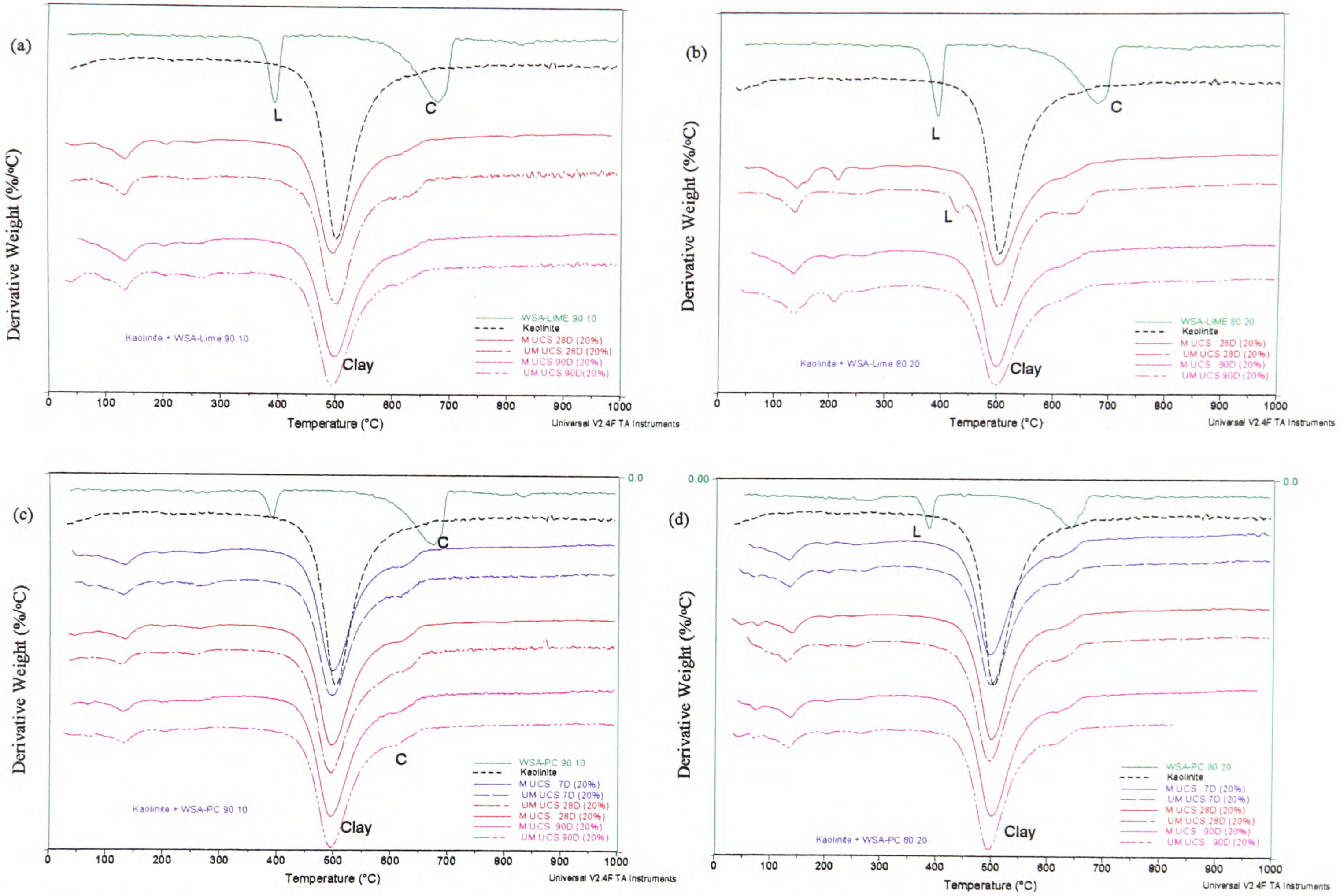


Figure 6.8-16: DTG traces of WSA-lime-stabilized kaolinite at (a) 90:10 (b) 80:20 and WSA-PC stabilized kaolinite at (c) 90:10 (d) 80:20 for both mellowed and unmellowed condition for 7, 28 and 90 days curing period.

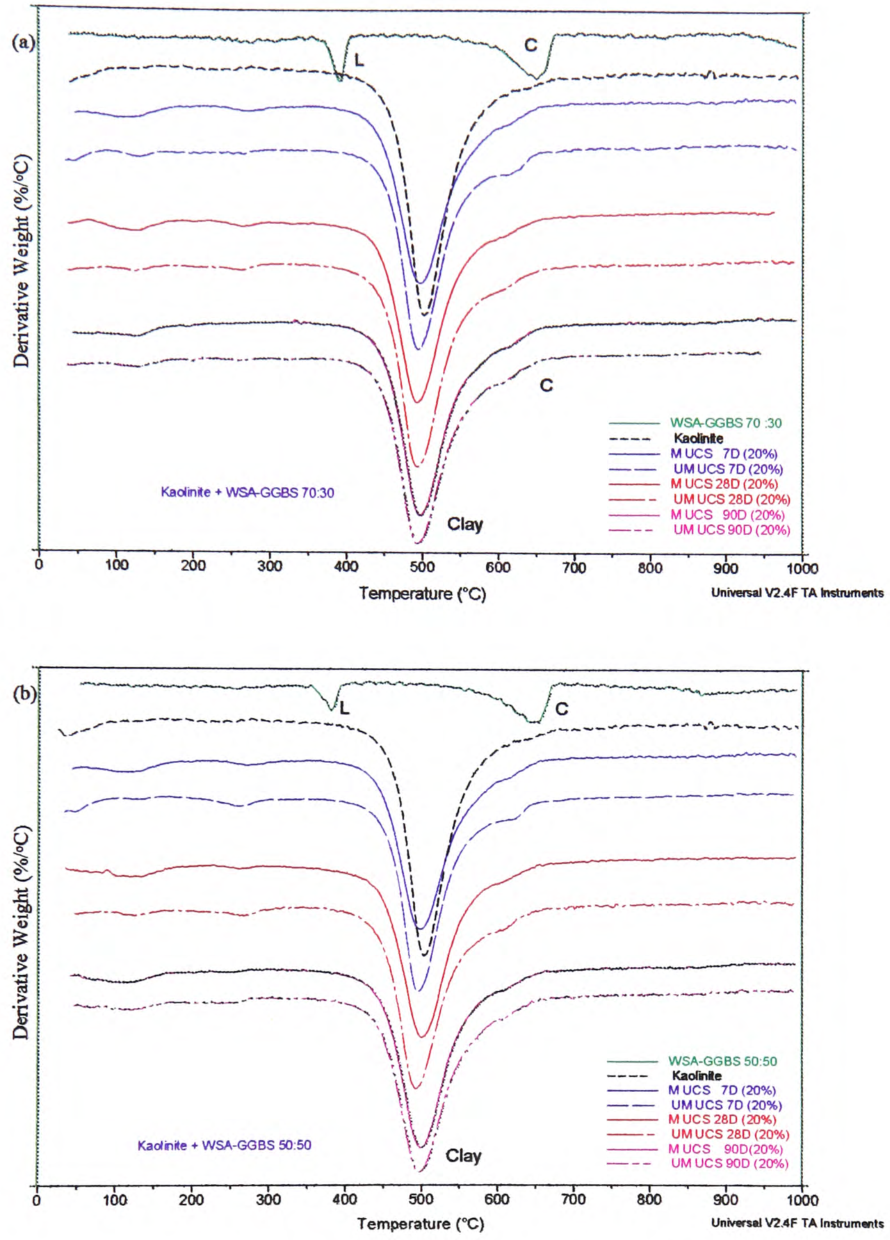


Figure 6.8-17: DTG traces of WSA-GGBS-stabilized kaolinite at (a) 70:30 (b) 50:50 for both mellowed and unmellowed condition for 7, 28 and 90 days curing period.

CHAPTER 7 – DISCUSSION

In this Chapter the experimental results that have been reported in Chapter 6 will be discussed in detail. Observations from experimental results are interpreted and suggestions for possible mechanisms, with respect to the research findings, made. Because of the critical significance of the presence of sulfide in Lower Oxford Clay with regard to the engineering properties, the discussion is divided into two main sections. The first section deals with the results on stabilized Lower Oxford Clay and the second analysis is on the observations made on stabilized and non-sulfide-bearing high purity industrial Kaolinite clay.

7.1 GENERAL

In the current research project, an effort was undertaken to investigate the overall performance of a sulfate-bearing clay soil (Lower Oxford Clay) and a non-sulfate-bearing clay of high purity – Kaolinite - stabilized with the traditional stabilizer-quicklime (CaO), and also stabilized with various novel blended stabilizers all incorporating Wastepaper Sludge Ash (WSA) - WSA-lime, WSA-PC and WSA-GGBS. The physical and chemical changes of the soil properties were analysed. A wide range of stabilizer combinations was employed, so as to permit a more complete assessment of the potential of the novel and sustainable (non-traditional) stabilizers for the modification and stabilization of clay soils.

The properties of stabilized clay soil may be divided into short-term properties that emanate from the immediate material changes that occur within a few hours during the stabilization process, and long-term properties that develop over a relatively longer time period, ranging from a few days to several years. The physical changes include the changes in consistency, compaction characteristics, unconfined compressive strength, California Bearing Ratio (CBR) and in linear expansion. The chemical effects include flocculation, pozzolanic reactions, hydration and changes in composition and mineralogy, that affect the strength and volume stability of the stabilized soil.

7.2 LOWER OXFORD CLAY (LOC) SYSTEM

7.2.1 Atterberg Limits

As soon as lime or blended stabilizers are added to the target test soil, cation exchange occurs (Grim, 1968; Cobbe, 1988). Clay particles have a net negative surface charge resulting from, among other factors, ionic substitutions in the clay particle lattice. At the clay particle crystal surface the negative charge is balanced by counter charged calcium ions of opposite (positive) charge from the stabilizer, producing a diffuse electrical double layer. The clay particles adsorb these calcium ions by a cation exchange process, as soon as lime or blended stabilizer is added to the clay soil. This modifies the properties of the electrical double layer to allow closer proximity of particles, which attract each other and stick together in floccs. Prior to this process, the negatively charged clay particles repel each other and each particle exists independently. The flocculation process is therefore responsible for the modification of the plasticity behaviour and enhances volume stability of clay soils when they are treated with lime (Sherwood, 1993). Two main properties that control the plasticity of clay soil are the plate-like structure of the clay particles and their association with the adsorbed water. The interaction between the adsorbed water of each clay platelet and the effect on particle interaction can bond platelets together and strengthen their bond, depending on the thickness of the oriented water layers (Grim, 1968).

Addition of small amounts of lime has an immediate effect on the clay fraction, resulting in changes in the Atterberg limits of the Lower Oxford Clay (Figure 6.2-1(a)). This, as explained above, is due to flocculation as a result of cation exchange between the lime and the clay minerals present in the LOC. Addition of 2wt.% lime leads to increases in the liquid limit and plastic limit. Further increase in lime content to 4wt.% of lime by dry soil weight, further increases the cation exchange and the formation of floccs. The flocculated nature of the lime-soil mixture weakens the bonds between the "floccs", as less water is required to lubricate them at the points of contact, to the stage where movement would occur and a shearing action can easily take place (Ouf, 2001). However, the soil is also capable of accommodating more water within the increased voids in the flocculated structure and, depending on the prevailing situation, the liquid limit of the stabilized soil generally increases. In the current research work, the liquid limit of the LOC increased to its maximum value of 77%, before dropping to a stable value of 74%. Addition of lime to the test soil increased the plastic limit to a maximum value of 54% at a lime content of 4wt.%. The increase in plastic limit is probably caused by water being held not only by the electrostatic forces on the clay minerals particles surfaces but also by capillary forces inside the flocculated clay structures (Daniels, 1971). Further increase in lime content had no effect on the plastic limit probably due to the saturation of the system with calcium ions. Due to the significant increase in the plastic limit relative to changes in the liquid limit, the plasticity index showed a general decline with increasing lime content. The same trends were also observed in the LOC + WSA system and with all the other blended stabilizers, LOC + WSA-lime, LOC + WSA-PC and LOC + WSA-GGBS. Although the WSA:GGBS blend high in GGBS content (50:50 blend) did not lower the plasticity by a very big margin, the overall magnitude of changes in liquid limit and plastic limit are comparable to those in the control (lime) system. This is encouraging, as it shows that the sustainable stabilizers incorporating WSA are capable of soil modification. The consequences of the subsequent behaviour of the stabilized soil mixture are now discussed below.

7.2.2 Proctor Compaction Tests

During the compaction of unstabilized clay soil, the dry density increases with increases in the initial water content (Cobbe, 1988; Bell and Coulthard, 1990). Researchers explain that this is due to the reduction of particle-particle frictional forces (George *et al.*, 1992). The water content however reaches an optimum point, beyond which its further addition results in fewer solid particles per unit volume as more water displaces the clay particles. In the case of a lime-stabilized soil system, flocculation and agglomeration, as a result of cation exchange processes, result in an increase in volume of voids and hence in material bulking. Thus as lime content is increased, lower MDD and higher OMC are normally observed (Cobbe, 1988; Locat *et al.*, 1990; Bell and Coulthard, 1990; Abdi and Wild, 1993; Kinuthia 1997). As mentioned earlier, the formation of flocs and agglomerates takes place within the first few hours after lime addition (Locat *et al.*, 1990; Abdi and Wild, 1993), and the material changes due to the cation exchange process, including consistency limits and compaction properties, are unlikely to be greatly affected by prolonged mellowing beyond 24 hours.

The effects of adding lime alone on the compaction characteristics of LOC are presented in section 6.3.1. The results show that the addition of lime to LOC had an overall effect of lowering the maximum dry density (MDD) and raising the optimum moisture content (OMC). The addition of 2wt.% lime resulted in the greatest decrease in MDD and greatest increase in OMC. Further addition of lime (4wt.% and 6wt.%) to the soil, resulted in a relatively smaller but steady decrease in MDD and increase in OMC. Thomas (2001), who studied the compaction characteristics of the same soil, also observed a similar trend. A study of lime-stabilized soil by Abdelkader and Hamdani (1985), indicated that the decrease in maximum dry density could also be due to the formation of cementitious products which reduce the compactibility of the treated soil. Similar to the causes of increases in liquid limit, the optimum moisture content increases also as a result of increased pore volume due to flocculation.

WSA-lime-stabilized LOC at 90:10 and 80:20 blending ratios resulted in trends of compaction characteristics similar to those in the lime (control) system. The same trend was observed when WSA-PC, and WSA-GGBS blends were used to stabilize LOC. When WSA was used on its own, the MDD was lowered and the OMC increased at marginally greater magnitudes than using the blended WSA-lime stabilizer. Of all the stabilizers used, the WSA-GGBS system at the blending ratio of 50:50 recorded the highest MDD and lowest OMC, followed by that at 70:30 ratio (see Figure 6.3-5). The fact that these stabilizer blends were the same ones that also showed least reduction in plasticity (especially the 50:50 blend) confirms the involvement of floccs in the material behaviour. The lime that is initially available in the WSA-GGBS system is also utilised in the activation of GGBS, resulting in reduced cationic exchange and flocculation. The results therefore show that irrespective of any benefits on strength or volume stability resulting from WSA-GGBS stabilization, the system is however poor in material modification, and mixing problems are likely to be encountered in practice.

In the WSA-PC and WSA-GGBS stabilized LOC systems, it is suggested that two reactions are involved in the stabilization process, the hydration of PC and of GGBS. The hydration of GGBS is activated by the free-lime from the WSA. These reactions normally start immediately after mixing the dry materials with water, and consume significant amounts of water. As a result, the water available for lubrication dramatically decreases and the air voids content increases. Thus, more water is needed to obtain the same level of lubrication at the same compactive effort. Strong exothermic reactions associated with the hydration of PC increase flocculation and further loss of moisture due to evaporation during mixing. Therefore these mixtures were found to be comparatively drier and possessed a more rigid material matrix. Due to the overall reduced flocculation in the WSA-GGBS system (due to consumption of lime in GGBS activation), lower OMC and higher MDD values were observed relative to WSA, WSA-lime or WSA-PC stabilizer. (see Figure 6.3-10).

7.2.3 Unconfined Compressive Strength

Unconfined Compressive Strength of stabilized LOC was investigated using compacted cylindrical specimens of 100mm in length and 50mm in diameter and made in both mellowed (compacted 3 days after initial mixing with water) and unmellowed (compacted immediately after mixing) conditions. Compaction was followed by subsequent curing periods of 7, 28, 90, 180 and 365 days. The UK specifications call for a mellowing period, during which time the lime/other stabilizer is allowed to diffuse through the moist soil which usually makes it easier to mix the stabilized mixture further (Sherwood, 1993).

The UCS of stabilized LOC significantly increased after curing, for both mellowed and unmellowed conditions. Generally, increasing the amount of stabilizer and curing time resulted in increased compressive strength. The unmellowed stabilized LOC specimens consistently exhibited higher strength values than their mellowed counterparts throughout the 365 days of curing. The mellowed specimens showed little strength development up to 90 days of curing before exhibiting significant strength increase from 90 days onwards.

In either lime-stabilized LOC or lime-stabilized Kaolinite, increasing addition of lime resulted in increased strength after extended curing periods. This is due to increased pozzolanic reaction between lime and the clay fractions. It is generally recognised that the principal cementitious product of pozzolanic reactions is calcium-alumino-silicate-hydrate (C-A-S-H) gel (Diamond *et al.*, 1964; Brandl, 1981; Lees *et al.*, 1982; Bell and Coulthard, 1990). However in the case of LOC the process is more complex due to the formation of significant quantities of ettringite which although contributing to strength also causes expansion in the presence of moisture or liquid. The strength development of clay-lime material may be attributed to either the gradual crystallisation of this gel (Bell, 1988) or to its continued formation, without necessarily developing a crystalline structure, but blocking pores and providing strength as it develops (Wild *et al.*, 1989). Addition of small amounts of lime (2wt.%) resulted in no significant improvement in strength development for both mellowed and unmellowed conditions. This is probably because of insufficient lime to produce both material modification and also cementing reaction products. Increasing the lime level to 4wt.% and 6wt.% produced a marked improvement in strength for both conditions, with the unmellowed specimens achieving higher strength values. Delay in compaction, with the mellowed specimens, allows the hydration process to commence. This hardens the uncompacted soil mixture, resulting in reduced long-term strength of the compacted soil, because the mix becomes more difficult to compact which results in lower final density (Ingles and Metcalf, 1972). Mellowing also consumes some of the stabilizer, such that subsequent hydration with the already reduced amount of stabilizer takes place in an open structure of reduced density.

In WSA-stabilized LOC, there was no noticeable improvement in strength development of the mellowed material with increasing curing time for all the stabilizer contents investigated (10wt.%, 15wt.% and 20wt.%). This is probably due to less free lime being available at all these stabilizer levels for sufficient pozzolanic reaction to take place in that any available lime is used up in the reactions taking place during the mellowing process. The unmellowed material showed significant and rapid increase in strength at 15wt.% and 20wt.% stabilizer contents, especially after 28 days of curing. Thus, strength development of unmellowed samples (which also exhibited less expansion as will be discussed later), is better than for the mellowed ones. It is hypothesised that compacting the material immediately after mixing results in a less permeable and more dense sample of better volume stability. For the mellowed WSA-stabilized LOC system, the open structure (due to enhanced flocculation) resulted in reduced density, which in most cases, determines the strength of granular material.

In the WSA-lime-stabilized LOC system, the mellowed material recorded small strength increases with curing time, particularly at the higher stabilizer contents (15wt.% and 20wt.%). However, the unmellowed material exhibited better strength improvement. This confirms with the results so far discussed on lime and WSA systems.

In the WSA-PC-stabilized LOC system, it was also observed that the unmellowed samples recorded higher strength values than the mellowed samples. This was confirmed by the results of the California Bearing Ratio test where the unmellowed material achieved higher CBR values than the mellowed material. The strength enhancements are very similar to, but marginally greater than those in the WSA-lime stabilized system. Strength developed significantly with increase in stabilizer content and also in curing period, especially after 90 days of curing. The enhancement also increased when the level of PC increased in the WSA-PC blends from 90:10 to 80:20 blending ratio. The WSA-PC system also exhibited higher early strength (at 7 days) compared with the other systems. PC (unlike GGBS) does not require an alkaline environment in order to hydrate and therefore is unaffected by any possible depletion of available lime levels and pH reduction after the period of mellowing. Also unlike lime ($Ca(OH)_2$), PC produces (C-S-H) gel and also (C-A-H) phases as cementing products during its hydration. In addition lime from PC hydration and also from the WSA can be involved in significant pozzolanic reactions with the clay minerals being particularly significant in WSA-PC stabilized soil, producing cementitious products over time.

In all the stabilizer systems investigated, the highest long-term strength was shown by the unmellowed WSA-GGBS system. The addition of 10wt.% (WSA-GGBS) stabilizer content to LOC had only a small effect on UCS particularly of the mellowed samples. However with 20wt.% stabilizer, a marked strength development was observed upon prolonged curing. In this (WSA-GGBS) stabilized system, both WSA and GGBS are involved in hydration. The GGBS hydrates upon activation by the prevailing alkaline environment provided by free-lime from WSA. These reactions release silica and alumina and eventually C-S-H, and C-A-S-H gels and other cementing products result in a much improved long-term strength, compared to lime-stabilized LOC. Although the UCS of the WSA-GGBS system recorded the highest values, showing remarkable strength development, the stabilizer did not show a good performance with the CBR, which suggested significantly poor early strength development. Only unmellowed specimens with 20wt.% stabilizer in both WSA-GGBS blends (at 70:30 and 50:50) achieved CBR values significantly higher than the 15% CBR value that is stipulated by the Highways Agency (HA, 2000).

7.2.4 Linear Expansion

When the non-sulfate bearing clays are subjected to moisture, they show intercrystalline swelling (Arabi and Wild, 1989). Water, is strongly adsorbed at the negatively charged particle surface. 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 or stabilizer modifies the electrical double layer, reducing the thickness of the adsorbed water layer and thus reducing the expansion capacity. Flocculation also occurs. In addition to the rapid ion exchange reaction, there is also a slow chemical reaction which produces cementitious C-A-S-H gels and in some cases crystalline calcium aluminate hydrate and calcium aluminate silicate hydrate phases. The cementing effect of these reaction products binds the clay particles together, thus further resisting expansion.

The linear expansion in a sulfate-bearing clay is caused by the sulfate modifying the cementitious products that are produced by the reaction between the soil and stabilizing agent. The "attack" forms expansive compounds such as ettringite. In Lower Oxford Clay soils, where gypsum is usually present, the formation of a colloidal product consisting of a complex calcium-sulpho-aluminate-silicate hydrate (C-A-S-$\bar{S}$-H) is reported to form on the surface of the clay plates (Wild *et al.*, 1993). From this colloidal surface product, a crystalline compound commonly known as ettringite ($C_3A.3C\bar{S}.H_{32}$) nucleates (Wild *et al.*, 1993). Ettringite is known to impart significant strength enhancement, due to its needle-like crystal crystalline morphology (Abdi and Wild, 1993; Wild et al., 1993, 1996, 1998a). On the other hand, the colloidal reaction products from which ettringite nucleates has the capability of imbibing large volumes of water (Mehta, 1973; Mitchell, 1986) and dramatically increasing the expansion potential of the stabilized soil. Wild *et al.*, (1998b) have established that this capability of imbibing water is only present during ettringite's formative stages and is absent with fully crystallized ettringite. The formation of expansive products that lead to linear expansion in the presence of moisture in the stabilized soil is further enhanced by the oxidation of pyrites (FeS_2) in the LOC. This oxidation is known to produce gypsum ($CaSO_4.2H_2O$) (Russell and Parker, 1979). Ettringite peaks in stabilized LOC are clearly evident in DTG traces of all LOC-stabilizer systems (see Figure 6.8-1 to 6.8-3)

In the lime-stabilized LOC system, mellowed samples exhibited immediate linear expansion upon soaking after 7 days of moist curing (see Figure 6.5-1(a)). The expansion was more stable after 30 days of soaking. This system showed the highest expansion relative to the other stabilized systems, as a result of readily available lime and gypsum. When WSA alone was used to stabilize LOC, a huge reduction in linear expansion was observed over a period of 100 days for both mellowed and unmellowed samples compared to lime-stabilized LOC (see Figure 6.5-1(c)). This is probably due to the two factors, firstly the reduction in free lime in the WSA-LOC system compared with the lime-LOC system as evidenced by the DTG traces (see Figure 6.8-1). Secondly, there is the formation of cementitious products from WSA hydration reactions. The combined factors result in enhanced strength in the WSA-LOC system, with subsequent reduction in linear expansion, compared with the lime-LOC system.

The WSA-lime stabilized LOC system recorded further reduction in linear expansion compared with either lime-stabilized or WSA-stabilized LOC (see Figures 6.5(a-c)). The 80:20 WSA-lime blend showed significantly higher expansion (especially for the mellowed samples) than the 90:10 WSA-lime blend. This is perhaps due to the more available lime in the 80:20 blend. This is confirmed by DTG traces which show clearer lime peaks with the 80:20 WSA-lime blend than with the 90:10 blend (see Figure 6.8-5(a and b)). Similar linear expansion trends were observed in the WSA-PC-stabilized LOC system, where the mellowed samples continued to show higher expansion than the unmellowed samples. This system recorded the least expansion compared to all the other stabilized systems, possibly because this system produces a wide range of reaction products resulting from hydration reactions of (i) WSA and PC themselves separately, (ii) WSA and PC with each other, and (iii) the separate and combined reactions of WSA and PC with the LOC. The results thus show a high potential for WSA when blended with PC.

In WSA-GGBS stabilized LOC, the mellowed samples recorded the highest expansion compared with LOC stabilized with WSA, WSA-lime or with WSA-PC. Interestingly, the expansion is almost negligible in the unmellowed specimens. This could be due to the GGBS being activated by the free-lime from the WSA resulting in cementation of the LOC. The principal reaction products formed are likely to be a wide range of C-A-S-H hydration products, and little or no ettringite due to the limited availability of lime. It is hypothesised that the C-A-S-H gels surround the surface of the clay plates, and then fill the capillary pores. One possible explanation why the mellowed samples expanded more than the unmellowed samples is that delayed compaction (3 days after mixing) allowed the WSA hydration to commence and rapidly consume all the water in the system. Any ettringite formed would not contribute to strength as the material was uncompacted. Thus when the weak samples were soaked and saturated in water, ettringite was able to form and imbibe large amounts of water leading to large expansions. In the unmellowed samples, ettringite formed in a compacted state, thus enhancing the strength of the cylinders such that subsequent soaking had a relatively subdued expansive effect.

In the entire stabilizer system under investigation, it was observed that the mellowed samples exhibited higher linear expansion compared to the unmellowed ones. The fact that this expansion is due to ettringite formation as has been discussed is further supported by DTG traces which show sharp and clear ettringite peaks that are higher in the mellowed samples than in the unmellowed samples in all the stabilizer systems (see Figure 6.8-1 to 6.8-3 and Figure 6.8-7 to 6.8-14). This is probably due to the high sulfide content (in the form of pyrite) in the LOC samples used in the current research (Table 4.1 and 4.2). Sulfides in themselves do not have a deleterious effect on stabilized soils but do have the potential to oxidise to form sulfate. Oxidation is likely to have occurred during the 3 days mellowing period. This is confirmed by the SO_3

analyses where the mellowed samples consistently recorded higher SO_3 levels than the unmellowed ones (see Figure 6.6-4). Clearly the sulfate levels increased due to oxidation of sulfide. However the sulfate levels of mellowed systems fell as curing time increased from 180 days to 365 days. It is hypothesised that sulfate must in the long term be combined in such a way that it is not released during the sulfate analysis test. Initially some sulfate will be taken up in the C-S-H gel which is intimately mixed with colloidal ettringite. It is thought that the ettringite, which generally increases with curing time, becomes less colloidal and more crystalline with age, hence less reactive and more difficult to break down and release its sulfate. Unmellowed samples on the other hand which were compacted immediately after mixing were less permeable as their unflocculated particles closely locked into each other, thus making it more difficult for the sulfide to oxidise. This resulted in less expansion upon soaking. The rapid expansion of the mellowed stabilized LOC specimens, in deionised water, suggested that oxidation of the sulfide to sulfate was occurring readily and rapidly, within a time-scale of days. Higgins (2002) reported that stabilization with cement or lime can produce both a physical disturbance and a chemical environment, that comprise conditions very conducive for pyrite to oxidise to sulfate.

The results of the present investigation have showed that for the sulfate-bearing Lower Oxford Clay, mellowing (a delay between mixing and compaction) is definitely detrimental in terms of compressive strength and linear expansion. Thus, it can be concluded that for sulfide-bearing clay soils mellowing can be a significant disadvantage.

7.3 KAOLINITE SYSTEM

In the current research, an investigation was also carried out on an industrial kaolinite clay stabilized with lime, WSA and the blended stabilizers (WSA-lime, WSA-PC and WSA-GGBS). This system was to be compared and contrasted with the engineering properties observed on the natural sulfate-bearing LOC, so as to make a logical attempt at establishing the underlying reaction mechanisms involved in both sulfate/sulfide-bearing and non sulfate/sulfide-bearing clay soils.

7.3.1 Atterberg Limits

The Liquid limits and the plastic limits of all stabilized kaolinite systems showed similar trends with those of the stabilized LOC systems. The addition of stabilizers (lime, WSA) and the blended stabilizers (WSA-lime, WSA-PC and WSA-GGBS) increased the liquid limit and plastic limit. However, the increase in the LL of stabilized kaolinite was much higher, being from 56% to as high as 86% (with 80:20 WSA:lime). With the LOC, the LL rose to a maximum of below 80% in most cases, with only a few WSA blended stabilizers raising the LL above 80%. With these large increases in LL with stabilized kaolinite, only lime managed to reduce the plasticity index of stabilized kaolinite. The mechanism behind these changes is similar to that discussed in sec. 7.2.1, but due to kaolinite being a pure clay containing about 83% clay content and with little or no organic material, the effects of flocculation were greater and led to higher increases in LL and PL. Other differences between the behaviour of stabilized kaolinite and stabilized LOC would result from the presence of sulfate in the LOC. The sulfate would consume lime in the stabilized LOC system, thus reducing further the effects of flocculation.

7.3.2 Proctor Compaction Tests

Proctor compaction tests of stabilized kaolinite showed that the addition of stabilizer (lime, WSA) and the blended stabilizers (WSA-lime, WSA-PC and WSA-GGBS) resulted in the lowering of the MDD values and in increases in the OMC. The OMC values of kaolinite stabilized with all the blended stabilizers were generally lower than in the LOC system. This difference is possibly due to the difference between the two clays. LOC being a natural clay contains sulfides, pyrites, calcite and organic materials whereas kaolinite is an industrial clay containing an untypically high clay content of 83% and free from sulfate or calcite. These can be seen in the DTG traces of dry kaolinite where a broad peak of clay was detected between 450°C and 550°C, and there is no peak of gypsum (between 100°C and 200°C) or of calcite (between 700°C and 800°C) (see Figure 6.8-15). Despite the differences in the magnitudes for

MDD and OMC, the changes in these parameters upon stabilization are the same in both clays, i.e. a maximum lowering of the MDD by 0.17 Mg/m^3 and a maximum increase in OMC by about 8% (see Figure 6.3-5 and 6.3-10).

7.3.3 Unconfined Compressive Strength

As with lime-stabilized LOC, lime-stabilized kaolinite exhibited higher strength magnitudes with the unmellowed stabilized material compared with the mellowed material, throughout the 356 days of curing (see Figure 6.4-5(a)). However, the difference in strength gained between the mellowed and the unmellowed material was relatively small in the case of kaolinite. This is likely to be due to the absence of sulfate in the kaolinite system, such that during mellowing, the only major effects are those due to flocculation as compared to both flocculation and sulfate-based reactions in the mellowed stabilized LOC. Lime will also carbonate during mellowing as there is an easy access of air. This can be seen in Figure 6.8-15(a) where the 7 days mellowed sample has less lime and more carbonate compared with the unmellowed sample. At more extended curing times, carbonate is consumed in the formation of carbo-aluminate phases. The mechanisms behind the strength gain in the lime-stabilized kaolinite samples is well researched and has been found to be strength gain resulting from the pozzolanic reactions which result in the formation of calcium-alumino-silicate-hydrate (C-S-H) gels as the main cementing agents (Wild *et al.*, 1987; Abdi, 1992). The addition of 2wt.% lime has no practical relevance with regard to improvement of the bearing capacity and results in strength increases which are of no significance in comparison to those obtained with higher lime additions (Arabi and Wild, 1989). In the stabilized kaolinite, there is no ettringite to further boost the strength, as clearly shown by the DTG traces, where there are no ettringite peaks in all stabilized kaolinite systems (see Figure 6.8-15 to 6.8-17). Thus the C-S-H and/or C-A-S-H gels are the main strength-enhancing agents.

The same UCS trends were observed in the WSA-stabilized kaolinite (as in the lime-stabilized kaolinite system) where the unmellowed material recorded higher strength enhancement particularly with the high stabilizer content (20wt.%). However as with lime-stabilized LOC and lime-stabilized kaolinite, the low stabilizer content (10wt.%)

showed no significant improvement in strength development due to insufficient stabilizer levels needed to both modify the material and then to enhance the strength of the stabilized material. However, the WSA-stabilized kaolinite showed higher early strength at 7 days of curing compared to the lime-stabilized kaolinite system (compare Figure 6.4-5(a) and (b)). This suggests that at the early stages the WSA is the principal contributor to strength. This is in agreement with the work by Veerapan (2003) on WSA-GGBS concrete.

In WSA-lime-stabilized kaolinite, the two different blends of WSA-lime, 90:10 and 80:20 generally showed the mellowed material to have a higher strength development in the longer curing period and with increasing stabilizer content, relative to the unmellowed material. The UCS values were higher in the 80:20 blend suggesting that there was more lime available in the system compared to that in the 90:10 blend (see Figure 6.4-6(a) and (b)). The WSA-PC-stabilized system also showed that the mellowed material had markedly higher UCS values compared to the unmellowed samples. The 80:20 WSA-PC blend recorded significantly higher UCS value particularly with 15wt.% and 20wt.% stabilizer content for both mellowed and unmellowed samples, compared with the 90:10 WSA-PC blend. This is probably due to the higher PC content in the 80:20 blend.

In the entire stabilized system, the mellowed and unmellowed WSA-GGBS-stabilized kaolinite with 20wt.% stabilizer content recorded the highest UCS values particularly at 365 days of curing. With the WSA-GGBS at 50:50 blend, there was little or no strength development over the 365 days curing period for both mellowed and unmellowed samples stabilized with 10wt.% stabilizer, while the highest UCS value (5,000 kN/mm^2) was recorded on the 20wt.% mellowed sample (see Figure 6.4-8(b)). Work by Bai *et al.* (2003), on WSA-GGBS blended paste suggested that 50:50 is the optimum blending ratio for WSA-GGBS. This is further supported by Veerapan (2003) in his work on WSA-GGBS concrete. In the WSA-GGBS 70:30 blended system, strength development also increased progressively throughout the curing period for both mellowed and unmellowed stabilized samples except those stabilized with 10wt.% stabilizer content. This suggests that the hydration process of WSA-GGBS occurs slowly but continuously and that the WSA is the principal contributor to strength development.

7.3.4 Linear Expansion

Linear expansion of stabilized kaolinite during the first 7 days of moist curing were relatively small (< 0.45%). In most cases about 90% of the ultimate expansion occurred within the first 7 days after soaking. Linear expansion in the entire stabilized kaolinite system (i.e. stabilization using lime, WSA, WSA-lime, WSA-PC or WSA-GGBS), generally showed the mellowed stabilized samples exhibiting less expansion compared to the unmellowed ones. One possible explanation for the reduction in expansion is that the delay in compaction allowed the cation exchange reactions to take place fully, with most of the lime being consumed during the initial 3 days of mellowing (see page 39). Consumption of lime reduced subsequent water adsorption upon soaking and also increased the strength of the stabilized material. For systems stabilized with WSA-GGBS at 70:30 and 50:50 blends, the unmellowed samples recorded the highest expansion magnitude before collapsing. The mellowed samples of both blends collapsed immediately upon soaking (after 7 days moist curing). This indicated less cementation in the systems and little or no interparticle bonding, bearing in mind that the WSA-GGBS system also recorded low early strength at 7 days curing (see Figure 6.4-8(a) and (b)). These samples also adsorbed large amounts of water through the capillary action as a result of inadequate flocculation. A flocculated structure is expected to have more voids and reduced capillary forces. In the entire stabilized kaolinite system mellowing was found to be beneficial except for the WSA-GGBS system where volume stability was very much reduced upon soaking.

CHAPTER 8 – CONCLUSIONS AND RECOMMENDATIONS

The main objective of this research work was to study the potential utilisation of waste, particularly WSA, in the stabilization of sulfate-bearing clay soils and to compare performance between a natural clay and the industrial clay, kaolinite. The study was also designed to assess the effect of mellowing of stabilized soil prior to compaction. Extensive laboratory work was carried out on the engineering properties and analytical tests were performed on the stabilized soil to establish the underlying reasons for any differences in engineering properties of mellowed and unmellowed material for both LOC and kaolinite.

This final Chapter summarises the conclusions which can be drawn based on the work carried out and suggests further research avenues which might result in a useful contribution in understanding the role of waste material especially WSA as a soil stabilizing agent.

8.1 CONCLUSIONS

The work presented in this thesis has critically assessed the effect of stabilizing the naturally occurring clay soil – Lower Oxford Clay (LOC) - and industrial kaolinite with the traditional stabilizer quicklime (CaO) and with WSA and also blended WSA (WSA-lime, WSA-PC and WSA-GGBS) on the engineering properties of the soil when it is modified and stabilized. The salient conclusions of the work are summarised below.

1. The addition of a small amount of lime (2wt.% CaO) modified the engineering properties of LOC and kaolinite by increasing the liquid limit (LL), plastic limit (PL) and reducing the plasticity index (PI). It also reduced the maximum dry density (MDD) and increased the optimum moisture content (OMC), due to flocculation as a result of cation exchange between the clay particles and the added lime. Further increase in lime (4wt.% and 6wt.%) resulted in a drop in LL, as PL continued to increase. These trends resulted with the PI of LOC

decreasing progressively with increase in lime content. However for Kaolinite the PI reduced considerably above 2wt.% addition of lime. An addition of WSA and WSA blended stabilizers, (WSA-lime, WSA-PC and WSA-GGBS) to LOC and kaolinite exhibited the same trends on Atterberg limits as the lime (control) system.

2. Clay Soil Stabilization with WSA alone or with any of the additives used (lime, PC or GGBS) increases the OMC and decreases the MDD values of both LOC and kaolinite.

3. Addition of 2wt.% to 4wt.% lime to LOC or kaolinite (mellowed or unmellowed) resulted in insignificant strength development up to 365 days of curing. This lime content only showed modification but little stabilization. Increasing the amount of lime (6wt.%) increased the strength of both soil types particularly after curing for 180 days. With WSA or WSA blends, there was increased strength development with increasing curing period when at least 15wt.% of stabilizer was used. Unmellowed stabilized LOC consistently exhibited higher strength values than the mellowed stabilized LOC throughout the 365 days curing period. In contrast, in the kaolinite system the mellowed samples recorded higher strength values. Strength development of WSA-lime and WSA-PC stabilized LOC and kaolinite was very similar. California Bearing Ratio test results also showed that the unmellowed system recorded higher CBR values. Both WSA and WSA blends showed potential for achieving a CBR value of 15% which is the requirement stipulated for lime/cement treated capping layers.

   Whether mellowing inhibits strength development depends both on the composition of the clay soil and the type of stabilizer employed. If the soil contains sulfate and lime is a component of the stabilizer, then mellowing inhibits strength development because lime which is rapidly consumed during mellowing to form sulfo-aluminate phases will subsequently make little contribution to strength after the soil has been re-mixed and compacted. In addition less lime will be available for pozzolanic reaction after compaction. Also if sulfide is present, mellowing allows oxidation to sulfate in the highly alkaline environment increasing further the sulfate level. If the stabilizer includes rapidly hydrating cementitious components (e.g. WSA) these will hydrate during the mellowing period and some of their contribution to cementation will be lost. In contrast if the soil is a clay (e.g. kaolinite) with no other reactive components then where lime is a component of the stabilizer, mellowing has little detrimental effect on the strength development. This is principally because the reaction of lime with clay to form cementitious products is very slow and mellowing can in fact be advantageaous in allowing a period for the lime to diffuse within the clay to extend cation exchange prior to compaction. Again however if the stabilizer contains a rapidly hydrating cementitious component (e.g. WSA) strength development can be reduced by mellowing because the contribution to cementation by this component is lost.

4. There was immediate linear expansion upon soaking after 7 days of moist curing in both LOC and kaolinite stabilized cylinders. However after 100 days soaking period, further expansion of the stabilized samples of LOC and kaolinite was negligible with most of the cylinders having attained terminal linear expansion. The mellowed stabilized LOC samples exhibited the greatest expansion of which the lime-stabilized LOC system exhibited the highest expansion values, in proportion to the amount of lime added (2wt.%-6wt.%). The WSA-GGBS stabilized samples were susceptible to expansion in both the LOC and kaolinite systems, while WSA-PC-stabilized cylinders exhibited the lowest expansion for both the mellowed and unmellowed conditions. In contrast, the unmellowed stabilized material only recorded very small expansion. Unlike for LOC, in the kaolinite system, it is the unmellowed stabilized material that shows slightly higher expansion to the mellowed samples. This contrasting behaviour, particularly when lime is the stabilizer, is related to the different processes that occur during mellowing within the two soils. In LOC, during mellowing sulfide oxidises to sulfate and the increased sulfate level results in increased ettringite formation at increased expansion. In kaolinite, mellowing allows cation exchange and calcium saturation of the clay to take place reducing its affinity for water and thus its swelling potential. Therefore whether mellowing reduces expansion is dependent on the composition of the clay soil and the type of stabilizer used. Upon stabilization

of sulfate-bearing clay soil with a stabilizer that contains rapidly hydrating cementitious components (e.g. WSA), mellowing allows hydration to take place and some of their contribution to cementation will be lost after compaction, resulting in expansion upon soaking. In contrast, the stabilization of a high purity clay soil (e.g. kaolinite) with rapidly hydrating cementitious (e.g. WSA) as component of stabilizer allows lime to be consumed during mellowing for cementing reaction to take place. This contributes to strength development and reduction in linear expansion.

5. Mellowing was not beneficial in the LOC system investigated. The disadvantages of mellowing were reduced long-term strength, without any compensating effects of improved linear expansion. As some benefits of mellowing were observed in the kaolinite system, it is therefore concluded that whether or not to mellow depends primarily on the particular combination of soil type and stabilizer used, other variables also include the site conditions. Sulfate and thermogravimetric analysis results suggest that the presence of both sulfide and sulfate plays a major role in the mellowing process, when ettringite is produced, and gypsum consumed. It was clearly shown that if mellowing is carried out sulfide oxidation is stimulated but the sulfate levels then tend to fall after 28 days of curing. Clearly sulfate cannot disappear, therefore it becomes bound in a form that is not soluble with respect to the sulfate test. It is suggested that this is as a result of ettringite becoming more crystalline with curing time and hence becoming more inert.

8.2 PRACTICAL IMPLICATIONS AND RECOMMENDATIONS FOR FURTHER STUDIES

Although this research work successfully covered a wide area on the effects of utilising WSA as a stabilizer on the engineering properties of sulfate- bearing LOC and non sulfate-bearing kaolinite, there are several questions that can be raised by the work presented in this thesis which could not be incorporated over the relatively short period of 3 years. The following are outlines of the practical implications of the current research work, and also other fields of research worthy of further study.

8.2.1 Practical Implications

- The current research has demonstrated that the treatment of the test soil particularly sulfide/sulfate-bearing clay must be carefully standardised, as this is a very important factor that influences the rate of oxidation of the soil and subsequent performance. There is no specification or clause in the Highway Agency Design Manual or in British Standards giving guidelines on how to treat the soil prior to stabilization and/or any testing. The Highways Agency should collate recent research studies with a view to offering more detailed and comprehensive guidelines on the stabilization of sulfide-bearing soil. For example the moisture content is a major parameter that influences the engineering properties of stabilized soil particularly when mellowing is employed. Although mellowing is outlined in the Department of Transport guidelines on soil stabilization (D_fT , 1997), there are no guidelines on how to deal with moisture content or addition of moisture prior to compaction. The current work has indicated that there is an increasing number of potential non-traditional sustainable soil stabilizing agent (including industrial waste and by-products), for possible future field trials by enthusiastic contractors and designers. In view of these shortcomings and also in view of the work carried out in the current research, the following recommendations are made.

8.2.2 Recommendations for further studies

- An extensive study on the effect of moisture content of mellowed stabilized material on subsequent performance after compaction would be beneficial, especially for sulfate-bearing clay soils.

- The apparatus used in the current study on linear expansion and California Bearing Ratio could be further improved to obtain more consistent results. For example a transducer system connected to a computer could be used to replace dial gauges so that the data collections is more continuous and convenient as well as minimising human and experimental errors.

- Other additives with possible potential for improving the engineering properties of stabilized soils (e.g. PFA) could be used for blending with WSA for further studies on stabilization of soils.

- Although the laboratory investigation confirmed that incorporating WSA with other stabilizer is successful in strength enhancement and reduces expansion, a field trial is needed to assess the use of these materials in practice.

- A wider range of analytical studies of the cured stabilized samples is advocated, especially on the microstructure of the mixture particularly during the 3 days mellowing period, such as X-Ray Diffraction (XRD) and Nuclear Magnetic Resonance (NMR), with a view to investigating the formation of cementitious materials and the nature and properties of the C-A-S-H phases formed.

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APPENDICES

Appendix A

Sample of calculation

A.1 Sample of material computation for sample fabrication

A.2 Calculation sample on sulfate (SO_3) content

Appendix B

Experimental Results

B.1 Atterberg Limits

B.2 Proctor Compaction tests

B.3 Unconfined Compressive Strength – Lower Oxford Clay

B.4 Unconfined Compressive Strength - Kaolinite

B.5 California Bearing Ratio tests

B.6 Linear Expansion

B.7 Sulfate (SO_3) Analysis

Appendix C

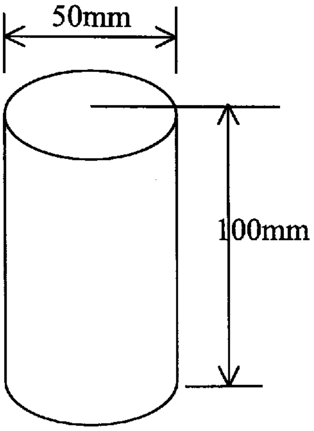
Publication

C.1 Paper presented in the 6th International Conference on Pavement Unbound (UNBAR), Nottingham, 6th – 8th July 2004, "Strength and swelling properties of Oxford Clay stabilized with wastepaper sludge ash.

APPENDIX A

A.1 Sample of material computation for specimen Fabrication

$$\rho d = \rho d\,(1 + w)$$
$$= 1.3\ \mathrm{Mg/m^3}\,(1 + 0.29)$$
$$= 1.677\ \mathrm{Mg/m^3}$$



Volume $= \dfrac{\pi D^2 t}{4}$

$= \dfrac{\pi (50)^2}{4}\ \mathrm{mm} \times 100\ \mathrm{mm}$

$= 196349.54\ \mathrm{mm^3}$

$= 0.000196\ \mathrm{m^3}$

Mass = **Density x Volume**

$= 1.677\ \mathrm{Mg/m^3} \times 0.000196\ \mathrm{m^3}$

$= 0.000329\ \mathrm{Mg}$

$= 329\ \mathrm{g}$

Allow 10% for wastage

$= 329\ \mathrm{g} + 32.9\ \mathrm{g}$

= <u>362 g</u> per sample

Soil (s), Stabiliser (ks), Water (0.25s), Air (0)

GROUP A

For 10%

s + ks + 0.25 s = 362.00 g
s + 0.1s + 0.25s = 362.00 g
1.35 s = 362.00 g
s = 268.15 g

stabiliser k = 268.14 (0.1)
k = 26.81 g

water = 268.15 (0.25) + 0.2 (67.03) = **13.41 g**
w = 67.04 g after 3 days mellowing

For 15 %

s + 0.15s + 0.25s = 362.00 g
1.4 s = 362.00 g
s = 258.57 g

stabiliser k = 258.57 (0.15)
k = 38.78 g

water = 258.57 (0.25)+ 0.2 (64.64) = **12.93 g**
w = 64.64 g after 3 days mellowing

For 20 %

s + 0.20s + 0.25s = 362.00 g
1.45 s = 362.00 g
s = 249.65 g

stabiliser k = 249.65 (0.20)
k = 49.93 g

water = 249.93 (0.25)+ 0.2 (62.48) = **12.48 g**
w = 62.41 g after 3 days mellowing

Table A.1-1: Material weight of samples for unconfined compressive strength and linear expansion tests.

	(%)	s (g) Soil	k (g) Stabiliser	w (g) Water	w after mellowing
Group A	10%	268.15	26.81	67.04	13.14
	15%	258.57	38.78	64.64	12.93
	20%	249.65	49.93	62.41	12.48
Group B	10%	262.32	26.23	73.45	14.69
	15%	253.15	37.97	70.88	14.17
	20%	244.59	48.92	68.48	13.70
Group C	10%	256.74	25.67	79.59	15.92
	15%	247.94	37.19	76.86	15.37
	20%	239.74	47.95	74.32	14.86
Lime	2% (B)	278.46	5.56	77.97	15.59
	4% (C)	268.15	10.72	83.12	16.62
	6% (C)	264.23	15.85	81.91	16.38

Table A.1-2: Material weight of samples for California bearing ratio

	(%)	s (g) Soil	k (g) Stabiliser	w (g) Water	w after mellowing
Group A	10%	3703.70	370.37	925.92	185.2
	20%	3448.27	689.65	862.07	172.48
Group B	10%	3623.18	362.31	1014.49	202.89
	20%	3378.37	675.67	945.94	189.18
Group C	10%	3546.09	354.61	1099.29	219.05
	20%	3311.25	662.25	1026.48	205.29
Lime	2%(B)	3846.15	76.92	1076.92	215.38
	6% (C)	3649.63	218.97	1131.38	226.27

A.2 Example of Sulfate (SO_3) content computation

Sample	UM-LOC/UCS-28 days LOC + WSA-GGBS 50:50				Date	04/01/2003
	1	2	3	4	5	6
	50:50 10%	50:50 10%	50:50 10%	50:50 20%	50:50 20%	50:50 20%
Mass of sample (g)	1.01	1.01	1.01	1.01	1.01	1.01
Mass of empty crucible (g)	24.95	24.69	24.59	24.83	24.45	24.51
Mass of crucible with precipitate (g)	24.99	24.72	24.62	24.86	24.49	24.55
Mass of precipitate (g)	0.03	0.03	0.03	0.04	0.04	0.04
%SO3 = $\frac{\text{Mass of precipitate x 34.3}}{\text{Mass of sample}}$	1.16	1.07	1.09	1.24	1.28	1.30
Average			1.10			1.27

3 tests were made for each samples and the average of the %SO_3 value was taken.

<u>See column no 1.</u>

Mass of sample = 1.0055 g

Mass of empty crucible = 24.9519 g

Mass of crucible + precipitate= 24.9858 g

Mass of precipitate = Mass of crucible + precipitate – Mass of empty crucible

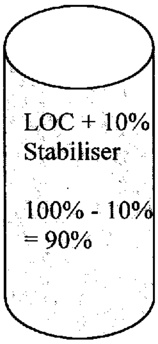
= 24.9858 – 24.9519 g

= 0.0339 g

% SO_3 = $\frac{\text{Mass of precipitate x 34.3}}{\text{Mass of sample}}$

= $\frac{0.0339 \text{ x } 34.3}{1.0055}$

= $1.156 \approx 1.16$



Average = $\frac{1.16 + 1.07 + 1.09}{3}$

= <u>1.10 %</u>

% SO_3 per unit weight of LOC + 10% stabiliser = 1.10 / 0.9

= <u>1.22 %</u>

APPENDIX B

B.1 Atterberg Limits

Table B.1-1: The Atterberg Limits value for LOC and kaolinite mixed with various stabilizers

STABILISER	RATIO	%	LL	PL	PI		RATIO	%	LL	PL	PI
LOC			66	35	31	**KAOLINITE**			56	30	26
LIME	100%	2	77	47	30		100%	2	83	43	40
		4	74	54	20			4	77	54	23
		6	74	54	20			6	75	54	21
WSA	100%	5	84	48	36		100%	5	79	30	49
		10	82	49	33			10	83	43	40
		20	78	50	28			20	83	49	40
		30	74	52	22			30	82	49	33
WSA-LIME	90:10	5	77	43	34		90:10	5	84	38	46
		10	76	50	26			10	85	43	42
		20	74	52	22			20	82	45	37
		30	69	53	16			30	82	47	35
	80:20	5	82	44	38		80:20	5	86	42	44
		10	72	48	24			10	84	46	38
		20	74	49	25			20	82	47	35
		30	63	51	12			30	80	47	33
WSA-PC	90:10	5	74	39	35		90:10	5	79	37	42
		10	75	46	29			10	82	40	42
		20	73	51	22			20	84	48	36
		30	69	52	17			30	81	48	33
	80:20	5	75	38	37		80:20	5	79	36	43
		10	73	42	31			10	80	40	40
		20	72	49	23			20	83	45	38
		30	69	50	19			30	81	48	33
WSA-GGBS	70:30	5	77	37	40		70:30	5	73	32	41
		10	81	39	42			10	77	39	38
		20	74	44	30			20	79	40	39
		30	60	44	22			30	80	53	27
	50:50	5	74	34	40		50:50	5	70	40	30
		10	77	37	40			10	69	37	32
		20	70	41	29			20	76	39	37
		30	68	41	27			30	76	39	37

B.2 Proctor Compaction Tests

Optimum Moisture Content (OMC) & Maximum Dry Density (MDD)

Table B.2-1: The MDD and OMC values for LOC and kaolinite mixed with various stabilizer.

STABILISER	RATIO	(%)	MDD Mg/m3	OMC (%)		RATIO	(%)	MDD Mg/m3	OMC (%)
LOC			1.36	25	**KAOLINITE**			1.43	22
LIME	100%	2	1.29	29		100%	2	1.33	29
		4	1.29	30			4	1.31	30
		6	1.28	31			6	1.30	31
WSA	100%	10	1.25	30		100%	10	1.32	26
		15	1.22	31			15	1.30	28
		20	1.20	32			20	1.26	30
WSA-LIME	90:10	10	1.27	29		90:10	10	1.31	26
		15	1.22	31			15	1.29	27
		20	1.20	32			20	1.28	28
	80:20	10	1.28	29		80:20	10	1.32	27
		15	1.22	30			15	1.30	28
		20	1.21	30			20	1.28	29
WSA-PC	90:10	10	1.25	31		90:10	10	1.33	27
		15	1.22	32			15	1.31	28
		20	1.18	33			20	1.28	29
	80:20	10	1.26	29		80:20	10	1.32	28
		15	1.25	30			15	1.29	29
		20	1.21	32			20	1.28	30
WSA-GGBS	70:30	10	1.29	28		70:30	10	1.37	25
		15	1.26	29			15	1.33	26
		20	1.24	30			20	1.32	27
	50:50	10	1.32	27		50:50	10	1.39	23
		15	1.30	28			15	1.39	24
		20	1.28	29			20	1.37	26

B3. UNCONFINED COMPRESSIVE STRENGTH TEST - LOWER OXFORD CLAY

Table B.3-1: Unconfined Compressive Strength values for mellowed and unmellowed stabilized LOC

Average of 3 samples

STABILISERS		%	7 days		28 days		90 days		180 days		365 days	
			M	UM	M	UM	M	UM	M	UM	M	UM
			kN/m^2	kN/m^2	kN/m^2	kN/m^2	kN/m^2	kN/m^2	kN/m^2	kN/m^2	kN/m^2	kN/m^2
LOC					21.39	28.86			9.51			
LIME		2%	*122.74*	183.35	*178.25*	249.22	*160.26*	190.48	*118.16*	252.95	*106.95*	215.60
		4%	*123.25*	181.31	*288.26*	292.34	*323.57*	323.91	*424.07*	415.59	*484.51*	699.77
		6%	*129.36*	201.17	*298.45*	288.26	*357.53*	371.45	*637.30*	514.73	*685.17*	1064.43
WSA	100%	10%	*181.31*	289.28	*145.66*	206.77	*162.30*	314.74	*175.88*	275.36	*130.38*	394.53
		15%	*338.17*	518.46	*265.85*	424.07	*326.29*	755.80	*295.39*	898.74	*292.68*	934.73
		20%	*379.93*	716.07	*344.79*	664.12	*372.80*	1145.58	*406.42*	1433.50	*444.79*	1570.33
WSA-LIME	90:10	10%	*211.87*	264.32	*182.33*	372.13	*180.80*	289.96	*292.34*	679.40	*341.91*	834.57
		15%	*283.17*	400.31	*343.77*	608.10	*327.48*	559.55	*568.71*	1115.36	*592.48*	1477.98
		20%	*362.11*	536.80	*483.83*	747.99	*558.70*	835.58	*1006.71*	191.93	*998.22*	2337.33
	80:20	10%	*275.53*	333.08	*229.18*	655.29	*305.07*	410.49	*374.50*	797.56	*383.33*	896.70
		15%	*300.48*	440.54	*297.43*	784.66	*494.02*	472.97	*558.53*	1085.82	*462.61*	1505.82
		20%	*361.60*	558.70	*385.54*	1007.39	*418.64*	669.21	*849.51*	1719.72	*852.66*	1733.81
WSA-PC	90:10	10%	*139.04*	144.64	*97.78*	259.74	*231.73*	239.03	*207.11*	411.85	*241.41*	509.30
		15%	*263.82*	309.65	*170.10*	618.96	*322.89*	541.89	*377.22*	1049.83	*493.68*	1340.13
		20%	*448.69*	557.17	*372.80*	1043.04	*500.13*	1000.26	*734.74*	1725.49	*825.40*	2379.09
	80:20	10%	*205.76*	222.05	*148.37*	407.78	*199.64*	360.24	*300.48*	782.62	*345.64*	734.07
		15%	*400.82*	401.33	*299.13*	818.27	*430.36*	769.04	*642.39*	1538.75	*828.79*	1675.24
		20%	*470.59*	545.46	*410.49*	1129.96	*599.95*	1070.54	*859.69*	2084.72	*1243.36*	2273.50
WSA-GGBS	50:50	10%	*106.95*	105.25	*286.56*	354.13	*374.16*	413.55	*572.11*	684.15	*616.59*	653.94
		15%	*195.57*	284.19	*465.16*	706.90	*699.09*	974.79	*985.32*	1339.45	*970.38*	1327.56
		20%	*347.34*	622.70	*567.02*	940.84	*981.24*	1727.19	*1804.94*	2488.76	*1619.22*	2883.63
	70:30	10%	*118.67*	98.29	*229.86*	264.83	*285.21*	348.02	*336.14*	491.64	*323.23*	486.21
		15%	*264.83*	274.00	*335.12*	693.66	*381.29*	752.74	*566.00*	1045.41	*591.12*	1428.78
		20%	*310.16*	481.28	*400.65*	907.57	*606.74*	963.59	*902.47*	1883.38	*833.21*	2732.88

B.4 UNCONFINED COMPRESSIVE STRENGTH TEST - KAOLINITE

Table B.3-2: Unconfined Compressive Strength values for mellowed and unmellowed stabilized kaolinite

Average of 3 samples			7 days		28 days		90 days		180 days		365 days	
STABILISERS		%	M	UM	M	UM	M	UM	M	UM	M	UM
			KN/m^2	KN/m^2	KN/m^2	KN/m^2	KN/m^2	KN/m^2	KN/m^2	KN/m^2	KN/m^2	KN/m^2
KAOLINITE												
LIME		2%	*347.85*	485.36	*489.94*	662.76	*656.99*	643.41	*483.49*	688.23	*586.37*	641.37
		4%	*300.99*	463.46	*387.40*	556.49	*604.70*	748.67	*720.31*	936.76	*942.88*	1029.46
		6%	*318.82*	482.81	*416.60*	595.54	*555.13*	762.59	*1008.75*	1047.45	*2085.57*	2314.24
WSA	100%	10%	*570.41*	769.55	*513.88*	830.49	*797.22*	1017.91	*670.23*	860.03	*670.91*	878.37
		15%	*674.82*	906.04	*843.90*	1189.04	*933.71*	1264.07	*873.27*	1193.45	*1026.40*	1278.67
		20%	*846.45*	1031.32	*1063.41*	1441.31	*1199.22*	1755.03	*1042.70*	1953.32	*1373.74*	1844.33
WSA-LIME	90:10	10%	*557.17*	785.33	*720.65*	943.90	*755.46*	1058.32	*1253.89*	1125.88	*1071.56*	1080.73
		15%	*532.72*	818.44	*759.36*	1041.00	*964.95*	1359.14	*1673.55*	1352.35	*1500.92*	1534.34
		20%	*676.34*	883.63	*872.42*	1209.41	*1286.82*	1770.31	*2729.49*	2156.36	*2558.36*	2236.15
	80:20	10%	*528.14*	714.54	*690.10*	860.71	*868.86*	1070.54	*1079.37*	981.58	*1642.99*	1056.62
		15%	*558.19*	838.30	*895.85*	962.91	*1223.33*	1372.04	*1944.15*	1549.28	*2094.90*	1793.74
		20%	*685.51*	890.76	*1710.73*	1129.96	*1580.52*	1672.87	*3048.65*	2627.97	*3480.87*	2375.70
WSA-PC	90:10	10%	*533.23*	721.67	*590.27*	850.86	*751.38*	954.42	*885.50*	1126.90	*898.74*	1184.96
		15%	*834.23*	1016.55	*1059.34*	1498.35	*1432.48*	1651.14	*1745.19*	1799.51	*1655.89*	1981.84
		20%	*1172.40*	1326.72	*1626.69*	1814.45	*2267.39*	2433.76	*2968.18*	2608.61	*2832.02*	2627.97
	80:20	10%	*702.83*	832.19	*910.11*	1036.25	*1202.96*	1316.02	*1200.92*	1321.45	*1291.23*	1327.90
		15%	*978.36*	1086.33	*929.47*	1448.44	*2004.59*	2020.21	*2386.22*	2125.46	*2295.23*	2285.38
		20%	*1045.08*	1352.18	*1340.47*	1942.79	*2343.78*	2712.85	*3463.55*	2852.96	*3330.80*	3497.84
WSA-GGBS	50:50	10%	*358.04*	495.04	*334.61*	457.69	*491.64*	628.13	*543.93*	510.99	*434.60*	527.97
		15%	*412.02*	638.32	*486.38*	476.36	*718.11*	800.27	*977.85*	797.90	*2282.00*	794.50
		20%	*552.59*	757.83	*794.50*	695.36	*2461.60*	2109.50	*3408.89*	2326.46	*5046.44*	3212.30
	70:30	10%	*386.05*	589.76	*434.43*	490.28	*531.37*	729.65	*661.75*	759.87	*674.99*	594.52
		15%	*461.93*	718.45	*565.32*	703.85	*1629.75*	1577.80	*2677.54*	1535.36	*2060.27*	1914.95
		20%	*573.98*	757.15	*1372.55*	1373.40	*2468.39*	2610.99	*3741.29*	2673.46	*3963.68*	3089.39

B.5 CALIFORNIA BEARING RATIO LOWER OXFORD CLAY

Table B.5: California Bearing Ratio value for stabilized LOC

			UNMELLOWED			MELLOWED		
			Mean CBR %		Overall	Mean CBR %		Overall
STABILISERS		%			CBR Mean (%)			CBR Mean (%)
			Top	Bottom		Top	Bottom	
LOC			2.30	2.80	3	2.80	2.80	3
LIME		2%	19.50	22.80	21	6.20	4.60	5
		6%	20.80	18.60	20	11.80	8.20	10
WSA	100%	10%	20.50	11.30	16	4.00	8.10	6
		20%	50.50	49.50	50	27.50	27.60	28
WSA-LIME	90:10	10%	32.70	28.30	30	17.10	11.60	15
		20%	61.90	46.70	55	23.60	17.60	21
	80:20	10%	25.90	28.90	25	18.70	15.70	17
		20%	54.40	48.50	50	33.30	19.30	26
WSA-PC	90:10	10%	29.50	34.20	30	7.20	3.00	5
		20%	64.60	65.30	65	24.40	23.20	24
	80:20	10%	32.50	18.40	25	7.00	8.20	8
		20%	35.20	59.00	50	30.20	18.60	25
WSA-GGBS	50:50	10%	14.00	11.60	13	11.70	5.40	9
		20%	28.00	39.00	33	9.30	14.70	12
	70:30	10%	10.30	16.30	13	7.70	8.90	8
		20%	54.40	45.20	50	19.10	14.80	17

Note: Results have been round up

1. To the nearest 1% for CBR values up to 30%
2. To the nearest 5% for CBR values between 30% to 100%
3. To the nearest 10% for CBR values greater than 100%

B.6-1 LINEAR EXPANSION

Table B.6-1: Linear expansion value for mellowed (M) and unmellowed (UM) samples of lime-stabilized LOC and WSA stabilized Kaolinite

LOC + LIME

	UM			M		
DAYS	2%	4%	6%	2%	4%	6%
1	0.00	0.00	0.00	0.00	0.00	0.00
2	-0.20	-0.37	-0.30	-	-	-
3	-0.21	0.87	-0.30	-	-	-
4	-0.21	0.92	-0.30	-	-	-
5	-0.21	0.98	0.50	1.35	0.40	1.00
6	-	-	-	-	-	-
7	-	-	-	-	-	-
8	-0.20	1.03	0.68	2.42	0.42	2.00
9	0.69	1.03	0.41	2.42	1.16	2.35
10	1.05	1.01	0.56	2.43	1.35	2.68
11	1.20	1.01	0.60	2.44	1.79	3.00
12	1.36	1.02	0.63	2.46	2.13	3.21
13	-	-	-	-	-	-
14	-	-	-	-	-	-
15	1.76	1.02	0.69	3.28	2.90	3.95
16	1.77	1.05	0.70	3.50	3.20	4.00
17	1.78	1.05	0.70	3.70	3.40	4.50
18	1.79	1.05	0.70	4.00	3.60	4.70
19	1.81	1.06	0.71	4.12	3.79	4.76
20	1.83	1.06	0.71	4.29	3.97	4.93
25	1.87	1.06	0.73	4.90	4.70	5.53
30	1.91	1.06	0.75	5.30	5.17	6.00
35	1.94	1.08	0.76	5.58	5.42	6.17
40	1.95	1.08	0.76	5.58	5.49	6.20
45	1.97	1.09	0.76	5.90	5.49	6.22
50	1.99	1.10	0.77	5.90	5.50	6.22
55	2.00	1.11	0.78	6.00	5.50	6.23
60	2.01	1.12	0.79	6.15	5.53	6.26
65	2.02	1.12	0.80	6.20	5.54	6.27
70	2.03	1.13	0.80	6.23	5.55	6.28
75	2.04	1.13	0.81	6.27	5.55	6.28
80	2.04	1.13	0.81	6.32	5.56	6.29
85	2.05	1.14	0.82	6.39	5.58	6.31
90	2.06	1.14	0.82	6.41	5.58	6.31
95	2.06	1.14	0.82	6.43	5.59	6.32
100	2.06	1.14	0.82	6.43	5.59	6.32

LOC + WSA

	UM			M		
DAYS	10%	15%	20%	10%	15%	20%
1	0.00	0.00	0.00	0.00	0.00	0.00
2	0.27	0.72	-0.01	-	-	-
3	0.37	0.74	0.00	-	-	-
4	0.39	0.74	0.00	-	-	-
5	0.41	0.75	0.01	0.36	0.43	0.18
6	-	-	-	-	-	-
7	-	-	-	-	-	-
8	0.45	0.77	0.04	0.52	0.47	0.18
9	0.45	0.76	0.07	0.55	0.48	0.18
10	0.43	0.76	0.09	0.59	0.48	0.18
11	0.42	0.76	0.09	0.60	0.49	0.18
12	0.41	0.76	0.10	0.61	0.49	0.19
13	-	-	-	-	-	-
14	-	-	-	-	-	-
15	0.40	0.76	0.11	0.65	0.50	0.20
16	0.40	0.76	0.11	0.66	0.50	0.20
17	0.40	0.76	0.11	0.68	0.50	0.20
18	0.40	0.76	0.11	0.69	0.50	0.20
19	0.40	0.77	0.12	0.69	0.50	0.20
20	0.40	0.77	0.12	0.70	0.50	0.20
25	0.41	0.77	0.14	0.71	0.51	0.20
30	0.44	0.78	0.16	0.73	0.51	0.21
35	0.47	0.78	0.17	0.77	0.53	0.22
40	0.48	0.78	0.17	0.78	0.53	0.22
45	0.49	0.78	0.18	0.79	0.54	0.23
50	0.45	0.78	0.19	0.79	0.54	0.23
55	0.44	0.78	0.20	0.80	0.55	0.23
60	0.44	0.78	0.21	0.85	0.57	0.26
65	0.45	0.79	0.22	0.85	0.57	0.26
70	0.47	0.80	0.23	0.86	0.58	0.27
75	0.47	0.80	0.24	0.87	0.58	0.28
80	0.47	0.80	0.24	0.88	0.59	0.29
85	0.47	0.81	0.25	0.91	0.61	0.31
90	0.47	0.81	0.25	0.91	0.61	0.31
95	0.47	0.81	0.25	0.92	0.61	0.32
100	0.47	0.81	0.25	0.92	0.62	0.32

B.6-2 LINEAR EXPANSION

Table B.6-2: Linear expansion value for mellowed (M) and unmellowed (UM) samples of WSA-lime-stabilized LOC.

LOC + WSA-LIME 90:10

	UM			M		
DAYS	**10%**	**15%**	**20%**	**10%**	**15%**	**20%**
1	0.00	0.00	0.00	0.00	0.00	0.00
2	-0.03	0.01	-0.01	0.02	0.13	0.17
3	-0.04	0.01	-0.01	0.02	0.14	0.18
4	-0.04	0.01	-0.01	0.02	0.14	0.19
5	-0.05	0.01	-0.01	0.02	0.15	0.20
6	-	-	-	-	-	-
7	-	-	-	-	-	-
8	0.01	0.08	0.02	0.02	0.15	0.20
9	0.03	0.09	0.02	0.08	0.19	0.21
10	0.03	0.10	0.02	0.09	0.19	0.21
11	0.03	0.10	0.02	0.10	0.19	0.21
12	0.04	0.11	0.03	0.11	0.20	0.22
13	-	-	-	-	-	-
14	-	-	-	-	-	-
15	0.05	0.12	0.04	0.11	0.20	0.22
16	0.05	0.13	0.04	0.11	0.20	0.22
17	0.05	0.13	0.04	0.11	0.20	0.22
18	0.05	0.13	0.04	0.11	0.20	0.22
19	0.05	0.13	0.04	0.11	0.20	0.22
20	0.06	0.14	0.05	0.11	0.20	0.22
25	0.07	0.16	0.05	0.11	0.21	0.22
30	0.08	0.18	0.06	0.12	0.21	0.23
35	0.09	0.19	0.06	0.13	0.23	0.24
40	0.09	0.19	0.06	0.14	0.23	0.24
45	0.10	0.20	0.07	0.15	0.24	0.25
50	0.10	0.22	0.07	0.15	0.24	0.25
55	0.11	0.23	0.07	0.15	0.24	0.25
60	0.12	0.24	0.08	0.18	0.26	0.27
65	0.13	0.24	0.08	0.19	0.26	0.27
70	0.13	0.26	0.09	0.20	0.27	0.28
75	0.14	0.26	0.10	0.20	0.28	0.28
80	0.15	0.27	0.10	0.21	0.29	0.28
85	0.15	0.28	0.10	0.23	0.31	0.29
90	0.15	0.28	0.10	0.23	0.31	0.29
95	0.15	0.28	0.10	0.24	0.31	0.29
100	0.15	0.28	0.10	0.24	0.32	0.29

LOC + WSA-LIME 80:20

	UM			M		
DAYS	**10%**	**15%**	**20%**	**10%**	**15%**	**20%**
1	0.00	0.00	0.00	0.00	0.00	0.00
2	-0.03	0.02	0.07	-	-	-
3	-0.05	0.03	0.10	-	-	-
4	-0.07	0.03	0.11	-	-	-
5	-0.08	0.04	0.12	0.44	0.20	0.24
6	-	-	-	-	-	-
7	-	-	-	-	-	-
8	-0.05	0.06	0.14	0.49	0.21	0.24
9	-0.03	0.06	0.14	0.50	0.21	0.24
10	0.00	0.06	0.14	0.50	0.21	0.25
11	0.00	0.06	0.14	0.51	0.21	0.26
12	0.00	0.06	0.14	0.52	0.22	0.27
13	-	-	-	-	-	-
14	-	-	-	-	-	-
15	0.00	0.07	0.16	0.54	0.22	0.27
16	0.00	0.07	0.17	0.54	0.22	0.27
17	0.01	0.07	0.17	0.54	0.22	0.27
18	0.01	0.07	0.17	0.55	0.22	0.27
19	0.01	0.07	0.17	0.55	0.22	0.27
20	0.02	0.08	0.18	0.56	0.23	0.27
25	0.02	0.08	0.18	0.56	0.23	0.28
30	0.04	0.09	0.20	0.56	0.23	0.28
35	0.04	0.09	0.21	0.58	0.24	0.29
40	0.04	0.09	0.21	0.59	0.24	0.29
45	0.04	0.10	0.22	0.60	0.25	0.30
50	0.05	0.10	0.23	0.60	0.25	0.30
55	0.05	0.11	0.25	0.60	0.25	0.30
60	0.06	0.12	0.26	0.62	0.28	0.33
65	0.06	0.12	0.26	0.62	0.28	0.33
70	0.07	0.13	0.27	0.63	0.29	0.33
75	0.07	0.14	0.28	0.63	0.29	0.34
80	0.08	0.14	0.28	0.63	0.29	0.35
85	0.09	0.14	0.29	0.66	0.31	0.37
90	0.09	0.14	0.29	0.66	0.31	0.37
95	0.09	0.14	0.29	0.66	0.31	0.38
100	0.09	0.14	0.29	0.67	0.32	0.38

B.6-3 LINEAR EXPANSION

Table B.6-3: Linear expansion value for mellowed (M) and unmellowed (UM) samples of WSA-PC-stabilized LOC.

LOC + WSA-PC 90:10

	UM			M		
DAYS	**10%**	**15%**	**20%**	**10%**	**15%**	**20%**
1	0.00	0.00	0.00	0.00	0.00	0.00
2	-0.01	0.00	-0.01	0.10	0.04	0.07
3	-0.04	0.00	-0.02	0.12	0.04	0.07
4	-0.04	0.00	-0.02	0.14	0.04	0.07
5	-0.05	0.00	-0.03	0.16	0.05	0.08
6	-	-	-	-	-	-
7	-	-	-	-	-	-
8	-0.01	0.05	0.00	0.19	0.06	0.08
9	0.00	0.06	0.00	0.21	0.06	0.08
10	0.00	0.06	0.01	0.22	0.06	0.08
11	0.00	0.06	0.01	0.23	0.06	0.08
12	0.00	0.07	0.01	0.24	0.06	0.08
13	-	-	-	-	-	-
14	-	-	-	-	-	-
15	0.00	0.08	0.01	0.26	0.07	0.08
16	0.00	0.08	0.01	0.26	0.07	0.08
17	0.00	0.08	0.01	0.26	0.07	0.08
18	0.00	0.08	0.01	0.26	0.07	0.08
19	0.00	0.09	0.01	0.26	0.07	0.08
20	0.00	0.09	0.01	0.26	0.07	0.08
21	0.00	0.09	0.01	0.27	0.07	0.08
30	0.01	0.11	0.01	0.27	0.07	0.08
35	0.01	0.12	0.01	0.30	0.08	0.09
40	0.01	0.12	0.01	0.30	0.08	0.09
45	0.01	0.13	0.01	0.31	0.09	0.10
50	0.02	0.14	0.01	0.31	0.09	0.10
55	0.02	0.15	0.01	0.31	0.09	0.10
60	0.03	0.16	0.02	0.35	0.12	0.12
65	0.03	0.16	0.02	0.36	0.13	0.13
70	0.05	0.18	0.02	0.37	0.14	0.14
75	0.05	0.19	0.03	0.38	0.14	0.14
80	0.05	0.19	0.04	0.39	0.15	0.15
85	0.05	0.20	0.04	0.42	0.17	0.16
90	0.05	0.21	0.05	0.42	0.17	0.16
95	0.05	0.21	0.05	0.43	0.18	0.17
100	0.05	0.21	0.05	0.43	0.18	0.17

LOC + WSA-PC 80:20

	UM			M		
DAYS	**10%**	**15%**	**20%**	**10%**	**15%**	**20%**
1	0.00	0.00	0.00	0.00	0.00	0.04
2	-0.01	-0.01	-0.01	0.11	0.29	0.04
3	-0.02	-0.02	-0.02	0.13	0.30	0.04
4	-0.03	-0.02	-0.02	0.14	0.32	0.05
5	-0.03	-0.03	-0.03	0.15	0.34	0.05
6	-	-	-	-	-	-
7	-	-	-	-	-	-
8	0.01	-0.02	-0.01	0.18	0.34	0.06
9	0.01	-0.02	0.00	0.24	0.35	0.06
10	0.01	-0.01	0.00	0.25	0.35	0.06
11	0.01	-0.01	0.00	0.26	0.35	0.08
12	0.01	0.00	0.00	0.28	0.36	0.08
13	-	-	-	-	-	-
14	-	-	-	-	-	-
15	0.02	0.00	0.00	0.29	0.36	0.10
16	0.02	0.00	0.00	0.29	0.36	0.10
17	0.02	0.00	0.00	0.29	0.36	0.10
18	0.02	0.00	0.00	0.30	0.36	0.11
19	0.02	0.00	0.00	0.30	0.36	0.12
20	0.02	0.00	0.00	0.30	0.36	0.13
21	0.02	0.00	0.00	0.30	0.36	0.15
30	0.02	0.01	0.01	0.31	0.36	0.15
35	0.02	0.01	0.01	0.33	0.37	0.15
40	0.02	0.01	0.01	0.34	0.37	0.15
45	0.02	0.01	0.01	0.35	0.38	0.15
50	0.03	0.02	0.01	0.35	0.38	0.16
55	0.03	0.02	0.01	0.35	0.38	0.16
60	0.04	0.02	0.01	0.38	0.41	0.16
65	0.04	0.02	0.02	0.38	0.41	0.16
70	0.05	0.04	0.02	0.39	0.42	0.16
75	0.05	0.04	0.03	0.39	0.42	0.16
80	0.05	0.04	0.03	0.40	0.42	0.16
85	0.06	0.05	0.03	0.42	0.44	0.16
90	0.06	0.05	0.03	0.42	0.44	0.16
95	0.06	0.05	0.03	0.43	0.45	0.16
100	0.06	0.05	0.03	0.43	0.45	0.16

B.6-4 LINEAR EXPANSION

Table B.6-4: Linear expansion value for mellowed (M) and unmellowed (UM) samples of WSA-GGBS-stabilized LOC.

LOC + WSA-GGBS 50:50

	UM			M		
DAYS	**10%**	**15%**	**20%**	**10%**	**15%**	**20%**
1	0.00	0.00	0.00	0.00	0.00	0.00
2	-0.04	-0.04	-0.02	0.08	0.12	0.13
3	-0.07	-0.01	-0.02	0.10	0.15	0.14
4	-0.10	-0.01	-0.03	0.12	0.20	0.14
5	-0.12	-0.01	-0.04	0.14	0.28	0.15
6	-	-	-	-	-	-
7	-	-	-	-	-	-
8	0.03	0.04	-0.01	0.18	0.34	0.15
9	0.05	0.04	-0.01	0.23	1.40	1.16
10	0.06	0.04	-0.01	0.27	1.40	1.16
11	0.06	0.04	-0.01	0.30	1.40	1.16
12	0.06	0.05	-0.01	0.35	1.41	1.16
13	-	-	-	-	-	-
14	-	-	-	-	-	-
15	0.06	0.05	-0.01	0.58	1.41	1.17
16	0.06	0.05	-0.01	0.65	1.41	1.17
17	0.06	0.05	-0.01	0.75	1.41	1.17
18	0.06	0.05	-0.01	0.85	1.41	1.17
19	0.06	0.05	-0.01	0.88	1.41	1.17
20	0.06	0.05	-0.01	0.90	1.41	1.17
25	0.06	0.05	-0.01	0.92	1.41	1.17
30	0.06	0.05	-0.01	0.93	1.42	1.17
35	0.06	0.05	-0.01	0.96	1.43	1.18
40	0.06	0.05	-0.01	0.96	1.43	1.18
45	0.06	0.05	-0.01	0.97	1.43	1.19
50	0.07	0.05	-0.01	0.97	1.44	1.19
55	0.07	0.05	-0.01	0.97	1.44	1.19
60	0.08	0.05	-0.01	1.00	1.45	1.21
65	0.08	0.05	-0.01	1.00	1.45	1.21
70	0.09	0.06	0.00	1.01	1.45	1.22
75	0.10	0.06	0.01	1.01	1.45	1.22
80	0.10	0.06	0.01	1.02	1.45	1.23
85	0.11	0.07	0.01	1.05	1.45	1.25
90	0.11	0.07	0.01	1.05	1.45	1.25
95	0.11	0.07	0.01	1.05	1.45	1.25
100	0.11	0.07	0.01	1.06	1.45	1.26

LOC + WSA-GGBS 70:30

	UM			M		
DAYS	**10%**	**15%**	**20%**	**10%**	**15%**	**20%**
1	0.00	0.00	0.00	0.00	0.00	0.00
2	-0.04	-0.02	-0.02	0.01	0.20	0.05
3	-0.06	-0.03	-0.02	0.01	0.25	0.05
4	-0.08	-0.03	-0.02	0.01	0.28	0.05
5	-0.10	-0.03	-0.02	0.01	0.33	0.05
6	-	-	-	-	-	-
7	-	-	-	-	-	-
8	0.02	0.02	0.01	0.01	0.34	0.06
9	0.03	0.03	0.01	0.21	1.34	0.06
10	0.04	0.04	0.01	0.31	1.34	0.06
11	0.04	0.04	0.01	0.41	1.34	0.06
12	0.04	0.04	0.02	0.46	1.35	0.06
13	-	-	-	-	-	-
14	-	-	-	-	-	-
15	0.04	0.04	0.02	0.61	1.36	0.08
16	0.04	0.04	0.02	0.64	1.36	0.08
17	0.04	0.04	0.02	0.66	1.36	0.10
18	0.04	0.04	0.02	0.67	1.36	0.10
19	0.04	0.04	0.02	0.67	1.36	0.10
20	0.04	0.04	0.02	0.68	1.36	0.10
25	0.05	0.04	0.02	0.68	1.36	0.12
30	0.05	0.05	0.02	0.68	1.36	0.13
35	0.05	0.05	0.02	0.70	1.37	0.15
40	0.05	0.05	0.02	0.70	1.37	0.15
45	0.05	0.05	0.02	0.71	1.38	0.15
50	0.05	0.05	0.02	0.71	1.38	0.15
55	0.05	0.05	0.02	0.71	1.38	0.15
60	0.06	0.06	0.02	0.73	1.40	0.18
65	0.06	0.06	0.02	0.73	1.41	0.19
70	0.07	0.07	0.03	0.73	1.42	0.19
75	0.08	0.08	0.04	0.75	1.42	0.19
80	0.08	0.08	0.04	0.75	1.43	0.19
85	0.09	0.09	0.05	0.78	1.45	0.22
90	0.09	0.09	0.05	0.78	1.45	0.22
95	0.09	0.09	0.05	0.79	1.45	0.22
100	0.09	0.09	0.05	0.79	1.45	0.22

B.6-5 LINEAR EXPANSION

Table B.6-5: Linear expansion value for mellowed (M) and unmellowed (UM) samples of lime-stabilized kaolinite and WSA-stabilized kaolinite

Kaolinite + LIME

	UM			M		
DAYS	10%	15%	20%	10%	15%	20%
1	0.14	0.02	0.11	0.10	0.08	0.11
2	0.14	0.03	0.11	0.10	0.08	0.11
3	0.14	0.03	0.11	0.10	0.08	0.10
4	0.14	0.02	0.11	0.09	0.07	0.08
5	0.14	0.02	0.11	0.09	0.07	0.05
6	-	-	-	-	-	-
7	-	-	-	-	-	-
8	3.09	3.14	3.71	2.32	1.61	2.51
9	3.11	3.16	3.75	2.36	1.63	2.58
10	3.15	3.18	3.78	2.42	1.64	2.63
11	3.19	3.22	3.80	2.49	1.65	2.72
12	3.21	3.25	3.85	2.52	1.69	2.76
13	-	-	-	-	-	-
14	-	-	-	-	-	-
15	3.24	3.28	3.87	2.54	1.70	2.80
16	3.24	3.29	3.87	2.55	1.71	2.80
17	3.26	3.29	3.89	2.56	1.71	2.81
18	3.26	3.29	3.89	2.57	1.72	2.81
19	3.26	3.30	3.89	2.58	1.73	2.83
20	3.28	3.30	3.90	2.61	1.76	2.84
25	3.30	3.31	3.90	2.62	1.77	2.85
30	3.30	3.31	3.90	2.64	1.78	2.86
35	3.31	3.32	3.91	2.67	1.81	2.88
40	3.31	3.32	3.91	2.69	1.82	2.89
45	3.32	3.33	3.92	2.71	1.84	2.90
50	3.33	3.33	3.92	2.73	1.86	2.92
55	3.33	3.34	3.92	2.74	1.87	2.94
60	3.34	3.34	3.92	2.75	1.87	2.94
65	3.34	3.36	3.90	2.76	1.88	2.95
70	3.32	3.34	3.90	2.78	1.89	2.96
75	3.32	3.34	3.90	2.79	1.90	2.97
80	3.33	3.34	3.90	2.81	1.91	2.98
85	3.34	3.34	3.90	2.81	1.91	2.98
90	3.35	3.34	3.90	2.81	1.92	2.98
95	3.36	3.34	3.90	2.82	1.93	2.99
100	3.37	3.34	3.90	2.84	1.94	3.00

Kaolinite + WSA

	UM			M		
DAYS	10%	15%	20%	10%	15%	20%
1	0.02	0.07	0.00	0.08	0.13	0.08
2	0.03	0.07	0.00	0.08	0.13	0.09
3	0.03	0.07	0.00	0.08	0.13	0.09
4	0.03	0.07	0.00	0.07	0.12	0.09
5	0.03	0.07	0.00	0.07	0.12	0.09
6	-	-	-	-	-	-
7	-	-	-	-	-	-
8	4.82	2.54	2.96	3.07	2.69	2.00
9	4.82	2.55	2.42	3.10	2.73	2.00
10	4.83	2.55	2.42	3.15	2.78	2.00
11	4.83	2.56	2.42	3.20	2.81	2.11
12	4.83	2.57	2.45	3.23	2.83	2.12
13	-	-	-	-	-	-
14	-	-	-	-	-	-
15	4.86	2.57	2.44	3.23	2.84	2.12
16	4.87	2.57	2.44	3.23	2.85	2.13
17	4.70	2.57	2.43	3.23	2.85	2.13
18	4.82	2.56	2.43	3.23	2.86	2.14
19	4.73	2.56	2.43	3.23	2.86	2.14
20	4.75	2.56	2.42	3.23	2.86	2.14
25	4.75	2.55	2.42	3.23	2.87	2.14
30	4.72	2.55	2.42	3.24	2.87	2.15
35	4.71	2.54	2.42	3.24	2.88	2.15
40	4.71	2.54	2.42	3.24	2.89	2.15
45	4.65	2.53	2.41	3.24	2.90	2.16
50	4.65	2.53	2.41	3.24	2.90	2.17
55	4.64	2.51	2.41	3.24	2.90	2.18
60	4.64	2.49	2.41	3.24	2.91	2.19
65	4.59	2.49	2.41	3.24	2.91	2.19
70	4.59	2.47	2.41	3.24	2.91	2.19
75	4.59	2.47	2.41	3.24	2.91	2.19
80	4.59	2.47	2.41	3.28	2.91	2.20
85	4.59	2.47	2.41	3.37	2.91	2.20
90	4.59	2.47	2.41	3.37	2.91	2.20
95	4.59	2.47	2.41	3.37	2.91	2.20
100	4.59	2.47	2.41	3.38	2.92	2.21

B.6-6 LINEAR EXPANSION

Table B.6-6: Linear expansion value for mellowed (M) and unmellowed (UM) samples of WSA-lime-stabilized kaolinite

Kaolinite + WSA-LIME 90:10

	UM			M		
DAYS	10%	15%	20%	10%	15%	20%
1	0.08	0.03	0.03	0.07	0.08	0.05
2	0.08	0.03	0.03	0.08	0.09	0.05
3	0.08	0.03	0.03	0.08	0.10	0.05
4	0.08	0.05	0.01	0.09	0.11	0.05
5	0.09	0.05	0.01	0.09	0.11	0.05
6	-	-	-	-	-	-
7	-	-	-	-	-	-
8	4.46	4.78	5.35	3.07	3.89	4.05
9	4.50	4.80	5.37	3.09	3.89	4.08
10	4.60	4.85	5.39	3.14	3.89	4.12
11	4.62	4.93	5.40	3.18	4.00	4.15
12	4.63	4.93	5.45	3.21	4.05	4.15
13	-	-	-	-	-	-
14	-	-	-	-	-	-
15	4.66	4.96	5.47	3.21	4.05	4.15
16	4.67	4.97	5.47	3.22	4.05	4.15
17	4.68	4.98	5.48	3.22	4.05	4.15
18	4.68	4.98	5.48	3.22	4.06	4.15
19	4.98	4.99	5.48	3.22	4.06	4.15
20	4.69	4.99	5.50	3.22	4.06	4.15
25	4.69	4.99	5.50	3.23	4.06	4.15
30	4.69	5.00	5.52	3.23	4.06	4.15
35	4.70	5.00	5.52	3.23	4.06	4.16
40	4.70	4.98	5.50	3.24	4.07	4.17
45	4.70	4.98	5.50	3.24	4.07	4.17
50	4.70	4.95	5.50	3.24	4.09	4.17
55	4.69	4.93	5.50	3.24	4.09	4.17
60	4.68	4.93	5.49	3.24	4.09	4.17
65	4.68	4.93	5.49	3.24	4.13	4.18
70	4.66	4.93	5.49	3.24	4.14	4.18
75	4.66	4.93	5.49	3.23	4.15	4.19
80	4.66	4.93	5.49	3.23	4.16	4.19
85	4.66	4.93	5.49	3.23	4.18	4.19
90	4.66	4.93	5.49	3.23	4.19	4.19
95	4.66	4.93	5.49	3.22	4.21	4.20
100	4.66	4.93	5.49	3.24	4.24	4.22

Kaolinite + WSA-LIME 80:20

	UM			M		
DAYS	10%	15%	20%	10%	15%	20%
1	0.17	0.07	0.03	0.11	0.05	0.08
2	0.17	0.07	0.03	0.11	0.05	0.08
3	0.17	0.07	0.03	0.11	0.05	0.08
4	0.15	0.07	0.02	0.11	0.05	0.08
5	0.13	0.08	0.02	0.10	0.05	0.09
6	-	-	-	-	-	-
7	-	-	-	-	-	-
8	4.77	4.81	5.54	3.92	4.57	5.43
9	4.80	4.83	5.60	3.96	4.60	5.45
10	4.82	4.85	5.62	4.00	4.63	5.45
11	4.84	4.86	5.64	4.05	4.66	5.45
12	4.86	4.88	5.68	4.07	4.69	5.45
13	-	-	-	-	-	-
14	-	-	-	-	-	-
15	4.94	4.94	5.72	4.07	4.69	5.45
16	4.96	4.96	5.75	4.07	4.69	5.45
17	4.98	4.98	5.76	4.07	4.69	5.45
18	4.98	5.00	5.76	4.07	4.69	5.45
19	4.98	5.01	5.78	4.07	4.69	5.45
20	4.99	5.02	5.78	4.07	4.69	5.45
25	5.00	5.02	5.78	4.07	4.69	5.45
30	5.00	5.01	5.78	4.07	4.69	5.45
35	5.01	5.01	5.78	4.10	4.69	5.45
40	5.01	5.02	5.78	4.11	4.70	5.45
45	5.02	5.02	5.78	4.13	4.70	5.45
50	5.02	5.02	5.78	4.15	4.71	5.45
55	5.02	5.01	5.78	4.15	4.71	5.45
60	5.02	5.01	5.78	4.15	4.71	5.45
65	5.02	5.01	5.79	4.16	4.72	5.45
70	5.02	5.01	5.79	4.17	4.72	5.45
75	5.02	5.01	5.79	4.18	4.72	5.45
80	5.02	5.01	5.79	4.18	4.72	5.45
85	5.02	5.01	5.79	4.18	4.72	5.45
90	5.02	5.01	5.79	4.18	4.75	5.45
95	5.02	5.01	5.79	4.18	4.75	5.45
100	5.02	5.01	5.79	4.19	4.77	5.45

B.6-7 LINEAR EXPANSION

Table B.6-7: Linear expansion value for mellowed (M) and unmellowed (UM) samples of WSA-PC-stabilized kaolinite

Kaolinite + WSA-PC 90:10

	UM			M		
DAYS	10%	15%	20%	10%	15%	20%
1	0.06	0.05	0.1	0.11	0.1	0.08
2	0.06	0.05	0.10	0.11	0.10	0.09
3	0.06	0.04	0.10	0.12	0.10	0.09
4	0.04	0.03	0.14	0.12	0.11	0.09
5	0.04	0.02	0.14	0.12	0.11	0.09
6	-	-	-	-	-	-
7	-	-	-	-	-	-
8	2.85	1.83	0.59	0.85	0.31	0.95
9	2.85	1.83	1.59	1.40	0.90	1.40
10	2.85	1.83	1.59	1.41	0.91	1.43
11	2.86	1.84	1.59	1.42	0.92	1.44
12	2.86	1.84	1.60	1.42	0.92	1.44
13	-	-	-	-	-	-
14	-	-	-	-	-	-
15	2.86	1.84	1.78	1.44	0.96	1.48
16	2.86	1.83	1.78	1.44	0.96	1.48
17	2.87	1.83	1.80	1.44	0.96	1.48
18	2.87	1.83	1.80	1.44	0.96	1.48
19	2.87	1.83	1.80	1.44	0.96	1.48
20	2.85	1.82	1.80	1.44	0.96	1.49
21	2.83	1.82	1.80	1.45	0.96	1.49
30	2.83	1.80	1.80	1.45	0.96	1.49
35	2.83	1.78	1.80	1.45	0.96	1.49
40	2.82	1.78	1.67	1.45	0.96	1.49
45	2.82	1.76	1.57	1.45	0.96	1.49
50	2.8	1.74	1.57	1.45	0.96	1.49
55	2.78	1.74	1.56	1.45	0.96	1.49
60	2.77	1.7	1.56	1.45	0.96	1.49
65	2.76	1.7	1.54	1.45	0.96	1.49
70	2.76	1.65	1.49	1.45	0.96	1.49
75	2.76	1.65	1.49	1.45	0.96	1.49
80	2.76	1.65	1.49	1.45	0.96	1.49
85	2.76	1.65	1.49	1.45	0.96	1.5
90	2.76	1.65	1.49	1.46	0.96	1.5
95	2.76	1.65	1.49	1.46	0.96	1.5
100	2.76	1.65	1.49	1.46	0.96	1.51

Kaolinite + WSA-PC 80:20

	UM			M		
DAYS	10%	15%	20%	10%	15%	20%
1	0.02	0.00	0.01	0.17	0.08	0.04
2	0.02	0.00	0.01	0.17	0.10	0.05
3	0.02	0.00	0.01	0.17	0.11	0.06
4	0.01	0.00	0.01	0.17	0.12	0.06
5	0.01	-0.03	0.00	0.17	0.12	0.06
6	-	-	-	-	-	-
7	-	-	-	-	-	-
8	0.96	1.88	0.01	0.32	1.49	0.09
9	1.84	1.88	0.28	0.68	1.79	0.10
10	1.84	1.89	0.28	0.70	1.80	0.16
11	1.84	1.90	0.28	0.71	1.81	0.16
12	1.85	1.91	0.28	0.72	1.81	0.16
13	-	-	-	-	-	-
14	-	-	-	-	-	-
15	1.87	1.93	0.29	0.76	1.86	0.16
16	1.87	1.93	0.29	0.76	1.86	0.16
17	1.88	1.94	0.29	0.76	1.86	0.16
18	1.88	1.94	0.30	0.76	1.86	0.16
19	1.89	1.95	0.30	0.76	1.86	0.16
20	1.89	1.95	0.31	0.76	1.86	0.16
21	1.90	1.95	0.31	0.76	1.86	0.16
30	1.90	1.94	0.31	0.76	1.86	0.16
35	1.89	1.94	0.31	0.76	1.86	0.16
40	1.88	1.93	0.31	0.76	1.86	0.16
45	1.87	1.93	0.31	0.76	1.84	0.16
50	1.86	1.92	0.31	0.76	1.84	0.16
55	1.86	1.92	0.31	0.76	1.84	0.17
60	1.84	1.92	0.31	0.76	1.84	0.17
65	1.82	1.89	0.31	0.76	1.84	0.17
70	1.8	1.89	0.31	0.76	1.84	0.17
75	1.79	1.89	0.31	0.76	1.84	0.18
80	1.79	1.89	0.31	0.76	1.83	0.18
85	1.79	1.89	0.31	0.76	1.83	0.19
90	1.79	1.89	0.31	0.76	1.83	0.20
95	1.79	1.89	0.31	0.76	1.83	0.21
100	1.79	1.89	0.31	0.76	1.82	0.25

B.6-8 LINEAR EXPANSION

Table B.6-8: Linear expansion value for mellowed (M) and unmellowed (UM) samples of WSA-GGBS-stabilized kaolinite.

Kaolinite + WSA-GGBS 50:50

	UM			M		
DAYS	10%	15%	20%	10%	15%	20%
1	0.10	0.02	0.06	0.10	0.14	0.04
2	0.11	0.02	0.06	0.10	0.14	0.05
3	0.11	0.02	0.05	0.10	0.14	0.05
4	0.10	0.02	0.05	0.10	0.14	0.05
5	0.10	0.02	0.05	0.10	0.15	0.05
6	-	-	-	-	-	-
7	-	-	-	C	-	-
8	4.27	3.95	4.44	O		
9	4.30	3.95	4.44	L		
10	4.30	3.95	4.44	L		
11	4.30	3.95	4.43	A		
12	4.29	3.95	4.43	P		
13	4.29	3.95	4.42	S		
14	4.29	3.95	4.40	E		
15	4.29		4.39	D		
16	4.29		4.39			
17	4.29		4.38		C	
18	4.27		4.38		O	
19	4.26	C	4.37		L	
20		O	4.37		L	
25		L	4.37		A	
30		L	4.35		P	
35	C	A	4.34		S	
40	O	P	4.34		E	
45	L	S	4.32		D	
50	L	E	4.32			
55	A	D	4.28			C
60	P		4.3			O
65	S		4.28			L
70	E		4.25			L
75	D		4.22			A
80			4.22			P
85			4.22			S
90			4.22			E
95			4.22			D
100			4.22			

Kaolinite + WSA-GGBS 70:30

	UM			M		
DAYS	10%	15%	20%	10%	15%	20%
1	0.02	0.05	0.02	0.10	0.08	0.09
2	0.02	0.05	0.02	0.12	0.09	0.09
3	0.02	0.05	0.02	0.13	0.10	0.09
4	0.02	0.03	0.02	0.13	0.10	0.09
5	0.02	0.03	0.02	0.13	0.10	0.09
6	-	-	-	-	-	-
7	-	-	-	-	-	-
8	3.92	4.49	3.05			
9	3.92	4.49	3.06	C		
10	3.92	4.49	3.06	O		
11	3.93	4.49	3.06	L		
12	3.93	4.49	3.06	L		
13	3.93	4.49	2.98	A		
14	3.93	4.49	2.95	P		
15	3.93	4.49	2.98	S		C
16	3.93	4.49	2.98	E		O
17	3.93	4.49	2.98	D		L
18	3.93	4.49	2.98			L
19	3.93	4.49	2.97			A
20	3.93	4.49	2.97			P
25	3.93					S
30					C	E
35					O	D
40	C				L	
45	O				L	
50	L	C			A	
55	L	O			P	
60	A	L	C		S	
65	P	L	O		E	
70	S	A	L		D	
75	E	P	L			
80	D	S	A			
85		E	P			
90		D	S			
95			E			
100			D			

B.7 SULPHATE ANALYSIS - LOWER OXFORD CLAY

Table B.7: Value of SO3 per unit LOC for stabilized LOC

SULPHATE ANALYSIS

LOWER OXFORD CLAY

Unmellowed LOC cylinder **1.13**

Samples are from UCS and dried under silica gel prior testing

Mellowed LOC cylinder **1.18**

Average of 3 samples

			28 days		180 days		365 days		0 days		Linear	Exp.
STABILISERS		%	M	UM	M	UM	M	UM	M	UM	M	UM
			%	%	%	%	%	%	%	%	%	%
LIME		2%	1.96	1.73	2.11	1.68	1.40	2.04	1.38	1.21	1.80	1.55
		4%										
		6%	2.01	1.79	2.28	1.41	1.01	1.64	1.29	1.01	1.80	1.51
WSA	100%	10%	1.97	1.63	2.04	1.40	0.92	1.33	1.18	1.01	1.64	1.31
		15%										
		20%	2.24	2.04	2.19	1.84	1.35	1.81	1.40	1.04	1.98	1.64
WSA-LIME	90:10	10%	1.96	2.01	1.71	1.86	1.78	1.89	1.17	1.24	1.56	1.51
		15%										
		20%	2.29	2.15	2.18	2.10	2.20	1.95	1.48	1.26	1.81	1.68
	80:20	10%	2.06	1.88	2.20	1.91	1.90	2.08	1.38	1.26	2.03	1.59
		15%										
		20%	2.20	1.89	2.25	2.00	2.01	1.65	1.61	1.31	1.56	1.50
WSA-PC	90:10	10%	1.72	1.92	1.69	1.46	1.52	1.83	1.00	1.59	1.61	1.49
		15%										
		20%	2.28	2.21	2.01	2.09	1.96	1.85	1.36	1.45	1.86	1.78
	80:20	10%	2.02	2.07	1.82	1.43	1.83	1.71	1.38	1.52	1.88	1.47
		15%										
		20%	2.33	2.15	2.16	1.88	2.23	1.94	1.53	1.66	1.89	1.80
WSA-GGBS	50:50	10%	1.61	1.22	1.86	1.66	1.40	1.60	1.34	1.41	1.62	1.29
		15%										
		20%	1.99	1.59	2.01	1.85	1.61	1.73	1.31	1.43	1.94	1.53
	70:30	10%	1.73	1.53	1.79	1.48	1.46	1.49	1.43	1.47	1.61	1.22
		15%										
		20%	1.90	1.90	1.78	1.80	1.64	1.55	1.44	1.60	1.85	1.44

APPENDIX C

Pavements Unbound – Dawson (ed.)
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Strength and swelling properties of Oxford Clay stabilized with wastepaper sludge ash

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ABSTRACT: The potential of Wastepaper Sludge Ash (WSA), an industrial by-product of the recycling of paper, as a soil stabilizer is reported. Lower Oxford Clay (LOC), a sulfate-bearing clay, was stabilized with quick-lime ((CaO) at typical dosages of 2, 4 and 6%) and with various blended stabilizers incorporating WSA (at 10, 15 and 25% stabilizer dosages). Compacted cylinder specimens were made with and without allowing for a 3-day mellowing period prior to compaction. They were then cured for up to 365 days and then tested for compressive strength development. Linear expansion measurements were also taken for a period of up to 100 days. The results indicate that the strength values of systems incorporating WSA are generally higher than those utilizing traditional quick-lime. For all the stabilizers in the current investigation, strength values of the unmellowed specimens are also higher than for the mellowed specimens at all curing periods.

1 INTRODUCTION

As the environmental crises deepen and valuable resources continue to be depleted, it is important that resources be utilized at rates approximately equal to the natural rate of regeneration. This will involve optimization of recycling and use of renewable resources, subject to advances in technology. In the paper industry, the environmental impact of paper manufacturing may be reduced by increasing the quantities of paper recycled (Frederick et al. 1996), and by utilizing ash from combusted wastepaper sludge.

The composition of wastepaper sludge is a function of the type, grade and quality of the recycled paper, and also of its thermal history. The sludge comprises approximately equal amounts of organic and inorganic components, the latter consisting principally of limestone and kaolin. Péra & Amrouz (1998) have shown that combusted wastepaper sludge used as a mineral admixture in high strength concrete is effective as a pozzolan. In the UK, one of the principal wastepaper recycling companies, Aylesford Newsprint Ltd. (ANL), combusts wastepaper sludge in a fluidized bed, resulting in ash that is currently dumped to landfill (~700 tonnes/week). Research work by Kinuthia et al. (2001) and by Bai et al. (2003) has established the principal crystalline components in ANL's WSA as typically calcium oxide (about 5 wt.% of which is free quick-lime with traces of calcium hydroxide), together with non-hydraulic, latently hydraulic and hydraulic components (gehlenite, bredigite, α-C_2S, small amounts of anorthite, ~5 wt.% calcium carbonate, and quartz). The ash is highly alkaline (pH 11–12) probably as a result of the residual free CaO. The potential of the utilization of this ash in concrete has been investigated as reported by Kinuthia et al. (2001), Bai et al. (2003), and by Veerapan et al. (2003). The objective of the current investigation is to establish WSA's potential for soil-stabilized pavement material, with or without blending it with CaO, Portland Cement (PC), or with ground granulated blastfurnace slag (GGBS).

2 EXPERIMENTAL PROCEDURE

2.1 *Materials*

Lower Oxford Clay was the target soil for stabilization. It was supplied by Hanson Brick Ltd. from their brick works at Stewartby, Bedford. Mineralogical studies by Hanson Brick Ltd. have established the LOC to contain illite (23%), kaolinite (10%), chlorite (7%), calcite (10%), quartz (29%), gypsum (2%), pyrite (4%), feldspar (8%) and organics (7%). Pyrites and sulfates in clay soils form expansive minerals such as ettringite and thaumasite when such soils are stabilized using CaO, making the stabilized layer volumetrically unstable (Snedker 1990, 1996, Higgins et al. 1998, 2002, Wild et al. 1999). LOC is therefore an excellent challenge for investigative work on soil stabilization.

WSA was supplied by Aylesford Newsprint Ltd. in the form of a dry coarse to fine powder with a small percentage (less than 10%) of sand-sized particles. Quick-lime (CaO) was supplied by Buxton Lime Industries Ltd. in the form of a fine white powder of cement size fineness, while Portland Cement (PC) was supplied by Blue Circle Ltd. Ground granulated blastfurnace slag (GGBS) was supplied by Civil and Marine Slag Cement Ltd., Llanwern, Newport. The oxide composition of LOC and these stabilizers are given in Table 1.

WSA was the key stabilizer used, with and without blending with CaO, PC or GGBS. The control mixes were LOC stabilized with 2, 4 and 6% CaO. The dosages of WSA, and WSA blends were 10, 15 and 20%. These dosages had been established in a previous unpublished research study as the levels likely to achieve the minimum CBR value of 15% stipulated by Department for Transport (DfT) for a CaO-stabilized capping layer (Highway Agency (HA) 2000, MCHW 1). For the blended binders, two mix proportions were investigated (90:10 and 80:20 WSA: CaO, PC or GGBS).

2.2 *Specimen preparation*

It was necessary to establish a common dry density and moisture content for specimen preparation. Therefore, several BS (BS1377, 1990) Proctor compaction tests were conducted in order to establish mean values of the density and moisture content to be adopted for the preparation of test specimens. In all the stabilized systems, the maximum dry density (MDD) ranged from 1.20–1.36 Mg/m^3 and a mean dry density value of 1.30 Mg/m^3 was adopted. The optimum moisture content (OMC) range was wide, from 27 to 33%, and two mean OMC values of 28% and 31% were adopted. Using the mean dry density and the nearest mean moisture content value, dry material (blended LOC + stabilizer) intended for mellowing was thoroughly mixed with the water required to achieve OMC and stored in polythene bags to mellow for 3 days at 20 ± 1°C and 100% relative humidity. During subsequent compaction, an extra amount of water (equivalent to 20% of OMC) was added so as compact the material wet of OMC as in common practice. For the unmellowed material, all the water required to achieve this condition was added during compaction, which was achieved using a cylindrical steel mould and a hydraulic jack. The specimens were then wrapped in several runs of cling film and several layers of polythene bags, before being placed on a perforated Perspex platform below which water was always maintained to ensure high relative humidity. This is referred to as moist

Table 1. Oxide composition of LOC, CaO, WSA, PC and GGBS.

	CaO	SiO_2	Al_2O_3	MgO	Fe_2O_3	FeO	$CaCO_3$	MnO	TiO_2	K_2O	P_2O_5	Na_2O
LOC[1]	6.15	46.73	18.51	1.13	6.21	0.80	–	0.07	1.13	4.06	0.17	0.52
Lime[2]	95.9	0.9	0.15	0.46	0.07	–	2.2	–	–	–	–	–
WSA[3]	37.0	34.0	18.39	5.04	1.77	–	–	–	–	–	–	–
PC	63.0	20.0	6.0	4.0	3.0	–	–	0.03–1.11	–	–	–	–
GGBS[4]	42.0	35.5	12.0	8.0	0.4	–	–	0.4	–	–	–	–

Note: Data supplied by [1] Hanson Brick Ltd. [2] Buxton Lime Industries Ltd. UK; [3] Southern Water Services Ltd. for Aylesford Newsprint Ltd.; and [4] Civil and Marine Slag Cement Ltd. UK.

curing, and was carried out for 7, 28, 90, 180 and 365 days in a temperature controlled chamber at 20 ± 1°C, before testing for Unconfined Compressive Strength (UCS). Three specimens were used for each curing period and for each mix composition, and the average strength value determined.

For specimens used to monitor linear expansion, approximately 10 mm of the bottom of the samples was exposed immediately after specimen fabrication, by cutting and removing the cling film. The specimens were placed on porous discs and then placed on a Perspex platform which was in turn placed in Perspex containers. The lids to the containers were fitted with dial gauges. A layer of water was always maintained below the Perspex platforms to provide a high humidity thus minimizing evaporation from the samples. After moist curing for 7 days, the samples were partially immersed in water to a depth of 10 mm by increasing the water level using a siphon. This ensured minimal disturbance of the specimens. The containers were kept in a temperature controlled chamber, maintaining temperatures at 20 ± 1°C and to 65 ± 5% relative humidity. Within the container the r.h. would be expected to be approaching 100%. Linear expansion during moist curing and subsequent soaking was monitored daily for about 100 days.

3 RESULTS

3.1 *Unconfined Compressive Strength (UCS)*

Figures 1(a) and (b) show the unconfined compressive strength of the CaO- and WSA-stabilized LOC for both mellowed and unmellowed conditions. In both systems, the performance of the unmellowed stabilized material is superior to that mellowed for 3 days, not only at 7 days but also after prolonged moist curing. At the low CaO level of 2%, no significant improvement in the strength with increasing curing time was observed for either mellowed or unmellowed conditions. The unmellowed material shows marginally higher strength values, relative to the mellowed material, throughout the one year of moist curing. At higher CaO levels of 4% and 6%, there is some improvement in strength upon prolonged moist curing, for both mellowed and unmellowed systems. After moist curing beyond 180 days, the unmellowed CaO-stabilized material shows superior strength development relative to the mellowed material. At all WSA levels (10%, 15%, and 20%), there is no noticeable improvement in the strength of the mellowed material with increasing curing time. This pattern is identical to the CaO-stabilized LOC at 2% CaO.

For the unmellowed WSA-stabilized material, there is also significant increase in strength throughout the period of moist curing with WSA levels above 10% resulting in a rapid strength increase after 28 days. In both CaO-LOC and WSA-LOC systems, it is apparent that the mellowing stage has a profound and long-term impact on both increase and rate of increase in strength. It is also apparent that in the unmellowed condition, WSA performs better than CaO, at the dosage levels investigated.

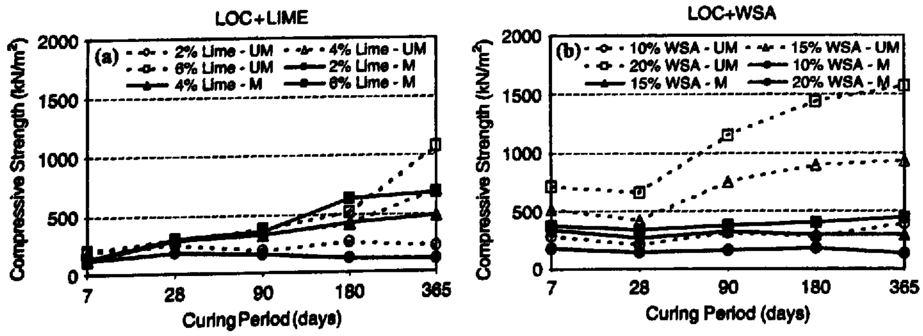

Figure 1. Compressive strength of LOC stabilized with (a) CaO, and (b) WSA, for mellowed and unmellowed conditions.

Figures 2(a) and (b) illustrate the strength development when LOC is stabilized with two WSA-CaO blends (90:10 and 80:20 WSA:CaO). As was the case for CaO-LOC and WSA-CaO-LOC systems, the unmellowed stabilized material performs better than the mellowed one. In contrast to the case when WSA is used on its own, as seen in Fig. 1 (b), by blending WSA with CaO, the mellowed material shows some slight strength increase with curing time at all dosage levels of the blended stabilizer. Better strength improvement is achieved with the unmellowed material stabilized with the blend richer in WSA (90:10). Therefore, there is no apparent advantage of increasing the CaO content beyond that present in the 90:10 (WSA:CaO) ratio.

The effects of blending WSA with PC, rather than with CaO, are illustrated in Figures 3(a) and (b). The blending ratios were identical to those used in the WSA-CaO blends. The benefits to strength enhancement of blending WSA with PC are very similar to, but marginally greater than, those of blending WSA with CaO. There is however a noticeable enhancement of strength development of the mellowed material when the level of PC is increased in the WSA-PC blends from 90:10 to 80:20

Figures 4(a) and (b) illustrate the effects of blending WSA with GGBS, a readily available material in the UK as a by-product from steel manufacture. In a previous research study by the authors on the use of WSA-GGBS blends as binder in concrete (Kinuthia et al. 2001, Veerapan 2003), a 50:50 (WSA-GGBS) blend was observed to show optimal strength and durability performance. This is the basis of the 50:50 (WSA-GGBS) ratio in the current investigation. In order to improve on the economics of this blend for soil stabilization, a blend with a higher proportion of the cheaper WSA was also investigated (70:30 (WSA-GGBS)). Dosage levels adopted were similar to those of the other WSA blends using CaO or PC (i.e. 10, 15 and 20%).

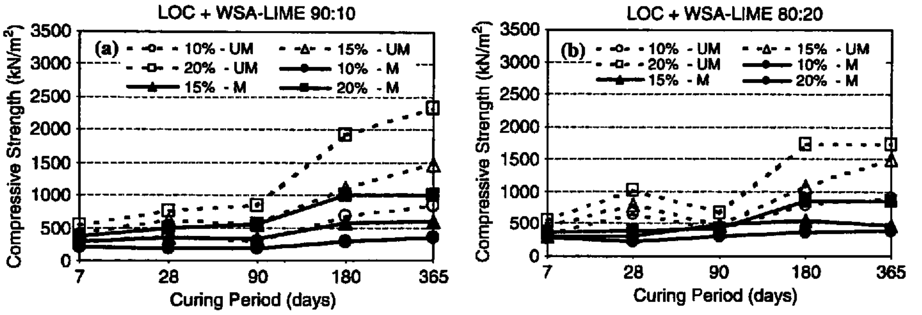

Figure 2. Compressive strength of LOC stabilized with two WSA-CaO blends (a) 90:10 WSA:CaO and (b) 80:20 WSA:CaO for mellowed and unmellowed conditions.

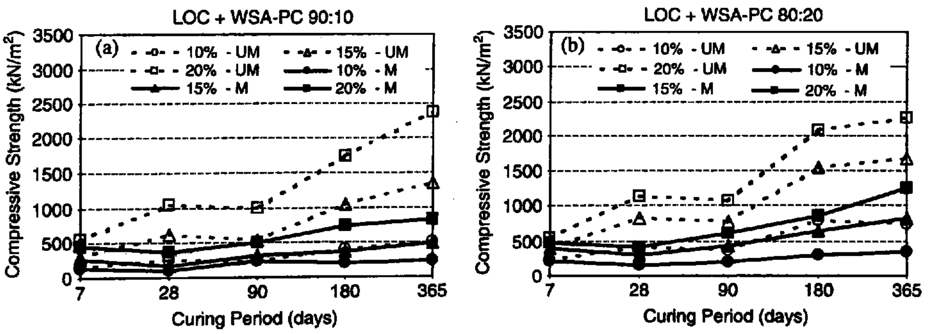

Figure 3. Compressive strength of LOC stabilized with two WSA-PC blends (a) 90:10 WSA:PC and (b) 80:20 WSA:PC for mellowed and unmellowed conditions.

As with the other stabilizers, WSA-GGBS blends also showed better strength development when the stabilized material was compacted without mellowing. The strength development was comparable with that achieved with WSA-CaO and WSA-PC blended stabilizers, with the 50:50 blends showing better performance in the case of the mellowed material, especially upon moist curing beyond 90 days (Fig 4(a)). Although the highest long-term strength in the entire LOC stabilization system investigated was observed on the unmellowed 50:50 WSA-GGBS system (2883 kN/m^2, at 365 days), the performance of the 50:50 blend is still very close to that of the cheaper 70:30 blend.

3.2 *Linear expansion*

The linear expansion of all the stabilized LOC systems under investigation was monitored for a period of 100 days. Over this period all the systems either attained terminal linear expansion or continued to expand at a negligible rate of increase. Figure 5 shows a typical plot of linear expansion with increasing soaking time, for the most expansive – LOC-CaO – system. The Figure illustrates that the expansion reaches a stable level after about 40 days of soaking. For the lesser expansive systems, the stability was achieved much earlier and due to limitations in space on this paper, Table 2 shows only the terminal linear expansion after 100 days of soaking.

It is evident that at the stabilizer dosages investigated, LOC stabilized with WSA on its own or with WSA-blends recorded significantly lower expansion at 100 days, compared with the CaO-stabilized one, for both mellowed and unmellowed systems. With all the stabilizers, the mellowed

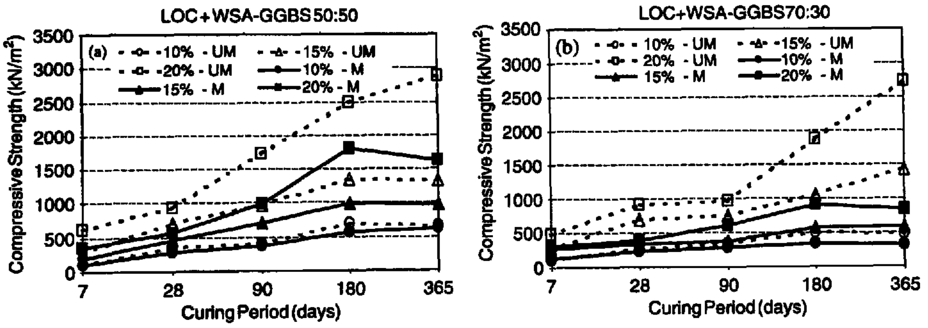

Figure 4. Compressive strength of LOC stabilized with two WSA-GGBS blends (a) 50:50 WSA:GGBS and (b) 70:30 WSA:GGBS for mellowed and unmellowed conditions.

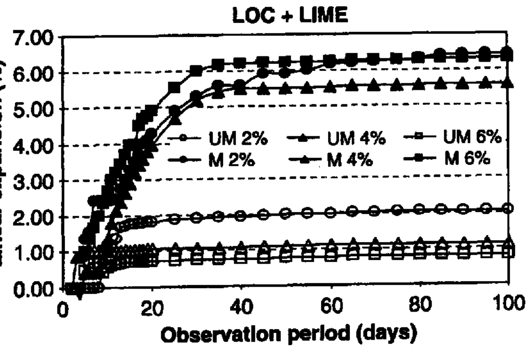

Figure 5. Linear expansion at 100 days of LOC stabilized with CaO and WSA (UM – Unmellowed; M – Mellowed for 3 days).

Table 2. Linear expansion at 100 days of LOC stabilized with two WSA-CaO blends, 90:10 and 80:20 (UM – Unmellowed; M – Mellowed for 3 days).

	WSA		WSA-CaO				WSA-PC				WSA-GGBS			
			90:10		80:20		90:10		80:20		70:30		50:50	
	M	UM	M	UM	M	UM	M	UM	M	UM	M	UM	M	UM
10%	0.92	0.47	0.24	0.15	0.67	0.09	0.43	0.05	0.43	0.06	0.79	0.09	1.06	0.11
15%	0.62	0.81	0.32	0.28	0.32	0.14	0.18	0.21	0.45	0.05	1.46	0.09	1.51	0.07
20%	0.32	0.25	0.29	0.10	0.38	0.29	0.17	0.05	0.00	0.03	0.22	0.05	1.26	0.01

specimens generally expanded significantly more than the unmellowed ones. This contradicts previous observations by Thomas (2000), who observed that mellowing CaO-LOC mixtures reduced linear expansion. The current investigation appears to suggest that mellowing does not always result in reduction in expansion.

Table 2 shows that both WSA-GGBS blends showed remarkable volume stability in the unmellowed condition. This confirms previously observed GGBS-induced suppression of linear expansion in unmellowed CaO-stabilized soil system (Thomas 2000, Higgins et al. 1998, 2002, Wild et al. 1999). It also suggests that the amount of CaO consumed during mellowing (by GGBS and by cationic reactions) has a big impact on subsequent material improvement during curing. Considering that CaO at the dosage levels investigated also performed poorly on strength development compared to WSA/WSA-blends, it is further clear that there are technological, economic as well as environmental advantages with the systems containing WSA.

4 DISCUSSION

It is well-established that during the period of mellowing CaO is consumed in a soil modification process in which the soil properties are changed (Kinuthia et al. 1999). From the current work it is evident that the changes in soil properties also include long-term effects on the strength properties of the mellowed material. When the CaO level originally present in the stabilized target material is low, there is no significant pozzolanic activity after mellowing and hence no enhanced strength development upon prolonged curing time.

There are numerous mechanisms that would result in delayed strength enhancement that is witnessed in the current work. The availability and rate of consumption of CaO above the ICL value, could be controlled by other factors besides the mere presence of excess CaO, such that the conditions are not conducive for enhanced strength in the unmellowed system until after some period. If the unmellowed material has the capability to produce sulfate via oxidation, then the consumption of residual CaO will gradually increase resulting in enhanced strength development in a sulfate-induced ettringite formation and strength enhancement and/or expansion (Kinuthia & Wild 2001). However, strength development especially in the presence of sulfate is not simply related to CaO content. For the mellowed material, the oxidation of LOC takes place early during the mellowing period, when the material has an easy access to CaO (Thomas 2000). As more hydration products form, the porous nature of the mellowed material (as evidenced by its lower density relative to the unmellowed material (Thomas 2000)) becomes a disadvantage, due to the more porous structure. In contrast, the more compact system of the unmellowed system benefits from the increase in hydration products.

When LOC is stabilized with WSA, the lack of strength increase in the mellowed system even at high WSA dosage levels (20%) suggests that the amount of CaO initially available in the system is low. It also suggests that after the initial free CaO present in the WSA is consumed, the WSA is incapable of further hydration during prolonged moist curing. In the unmellowed WSA-LOC system, the CaO initially present in the WSA is utilized in both modification and stabilization processes whereas in the mellowed system, the modification process take place when the soil is not compacted. The system is therefore susceptible to carbonation of lime, although this was minimized by the sealing in a polythene bag. The result is that there is little or no more CaO to further the stabilization process upon compaction. The situation is exacerbated by the porous nature of the material as already mentioned. In the unmellowed system, the process of modification and stabilization takes place in a more compact material and without interference. This has beneficial consequences as evidenced by the strength results. Carbonation of CaO and stalled hydration of WSA due to reduction of pH all add to the complexity of the hydration systems investigated.

Blending WSA with CaO or PC results in good strength development with increasing curing time, and better performance in volume stability in both mellowed and unmellowed conditions. On the other hand, blending WSA with GGBS shows the best performance for the unmellowed system. Previous work on CaO-GGBS systems has established the consumption of CaO by GGBS during curing. It is also well-established that the hydration of slag is also enhanced by the presence of sulfate (Higgins et al. 1998, 2002, Kinuthia et al. 1999), and the volume stability of the unmellowed WSA-GGBS system is only comparable to that of the WSA-PC. The closely comparable performance of WSA blends suggests that a decision on the preferred material (CaO, PC or GGBS) for blending with WSA will also be determined by other considerations besides strength development, such as relative cost and availability of the materials, volume stability, durability and site or other considerations.

5 CONCLUSIONS

From the work carried out on strength and linear expansion properties of stabilized LOC, it may be concluded that it is possible to utilize WSA for the stabilization of a sulfate-bearing clay. In the current research, the strength and expansion properties of WSA-stabilized LOC were superior to those achieved using the traditional CaO-stabilization. Therefore there is potential for technological, economic as well as environmental advantages of utilizing WSA and similar waste in pavement construction.

By blending WSA with a controlled amount of small quantities of CaO, the performance of WSA is greatly enhanced, resulting in improvement of both strength and volume stability. Both PC and GGBS may also be used to blend WSA, leading to systems of comparable advantageous effects, particularly when the material is compacted without mellowing. However, the high dosages of GGBS required are likely to be uneconomical.

Mellowing was not beneficial in the systems investigated in the current study. Whether or not to mellow will depend primarily on stabilizer used, other variables including the period of mellowing, the target material and site condition, besides possibly other factors. It is possible that the disadvantages of mellowing, such as reduced long-term strength, may be mitigated by compensating effects such as those of improved volume stability. Thus, more research on a wider range of soils and conditions prior to compaction is needed in order to fully establish the circumstances under which mellowing is likely to be beneficial.

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