GEOTECHNICAL AND ENVIRONMENTAL PERFORMANCE OF RESIDUAL LATERITIC SOIL STABILISED WITH FLY ASH AND LIME

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DOCTOR OF PHILOSOPHY

By

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CERTIFICATE

It is certified that the work contained in the thesis entitled "**Geotechnical and Environmental Performance of Residual Lateritic Soil Stabilised with Fly Ash and Lime**", by Rajib Kumar Goswami (Registration No. 994701), has been carried out under my supervision and that this work has not been submitted elsewhere for a degree.

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Dedicated to my parents

ABSTRACT

Application of fly ash for soil stabilisation has been the focus of many researchers due to its role in facilitating pozzolanic reactions and the possibility of bulk utilisation. Residual lateritic soils, with typical inherent properties, exhibit a number of constructional difficulties related to their workability, field compaction and strength that may be overcome with fly ash stabilisation. The present research was aimed at understanding the behaviour of a highly weathered residual lateritic soil modified with fly ash and lime, with respect to their geotechnical and environmental performance.

Preceded by the study of physical and chemical properties of the experimental soil and fly ash, a series of investigations were conducted to study the effects of particle aggregation upon drying as well as grain size distribution on the index and compaction properties in order to finalize appropriate methods for pre-test sample preparation and test procedure. Subsequently, based on information from the literature, extensive laboratory tests were performed on different soil-fly ash-lime mixtures.

The geotechnical performance was examined focussing on the site workability, compaction, strength and durability of the modified material. The site workability was studied by examining the changes in plasticity characteristics. Moisture-density relationships were established and examined for effect of additive ratios, progressive particle breakdown and delay in compaction. Stress-strain-strength characteristics were analysed through unconfined compressive test for short-term and long-term changes to understand the durability of the stabilised material. The effects of extreme environmental conditions like flooding or wetting were also considered. As the lime and fly ash modification process continues over time, the effects of conditioning or delay in

compaction on plasticity, compaction and strength were investigated. Specific studies were conducted by performing consolidated undrained triaxial tests on the mixes to understand the change in shear strength and to study the influence of compaction parameters, namely moisture content and dry unit weight. The environmental performance was assessed by estimating the potential and actual leaching of various metals with time.

Results show that compared to several other soils, residual lateritic soil has better potential for bulk utilisation of fly ash through geotechnical application and it can lead to considerable improvement of the properties of the soil as well. Advantages include improvement of site workability and durability, large and early strength generation, reduction of the collapse potential and less drying requirement. However, the degree of improvement can vary according to the nature and extent of fly ash and quantity of lime added, comparative grain size distributions of the soil and fly ash, and delay in compaction. In the present case, replacement of the soil with 20% fly ash caused negligible change in the density and strength of the material, except slightly lowering the workability of the soil. Increase in the replacement level till 50% required a minimum supplement of 2% lime to improve the workability and strength of the stabilised material to be equivalent of the soil. In terms of compressive strength, the replacement of the soil by 35% fly ash gives the best long-term results after lime treatment. For maximum strength, the stabilised soil must be compacted at optimum moisture content. The use of such admixtures will not have any significant impact on the environment, as most of the toxic metals present in the fly ash may not leach beyond threshold limits.

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NOTATIONS

A	Pore pressure coefficient
B	Pore pressure coefficient
c'	Cohesion intercept based on effective stress
d'	Intercept of K_f line with the Y-axis
G	Specific gravity
k	Hydraulic conductivity
k_{sat}	Saturated hydraulic conductivity
LL	Liquid limit
P'	Stress path parameter $(\sigma_1' + \sigma_3')/2$
q	Stress path parameter $(\sigma_1 - \sigma_3)/2$
P_s	Proportion of soil
P_f	Proportion of fly ash
P_l	Proportion of lime
PI	Plasticity index
t_{50}	Time for 50% consolidation
V	Volume of soil
V_s	Volume of solids
V_w	Volume of water
W_s	Weight of air-dried soil
W_f	Weight of soil replaced with fly ash
W_l	Weight of lime added
w_s	Water content of the air dried soil
w_f	Water content of the dry fly ash as obtained from the factory
α'	Angle that the K_f line makes with the horizontal
ε_d	Secant deformation modulus
σ_1'	Major principal stress based on effective stress
σ_3'	Minor principal stress based on effective stress
ϕ'	Angle of shearing resistance based on effective stress
ϕ_{cu}	Angle of shearing resistance based on total stress (consolidated undrained test)

ABBREVIATIONS

AAS	Atomic Absorption Spectrophotometer
BC	Black Cotton
CBR	California Bearing Ratio
CEC	Cation Exchange Capacity
CU	Consolidated Undrained
EPA	Environmental Protection Agency
FBC	Fluidised Bed Combustion
JCPDS	Joint Committee on Powdered Diffraction Studies
LL	Liquid Limit
LVDT	Linear Variable Displacement Transducer
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
PFA	Pulverised Fuel Ash
PI	Plasticity Index
PL	Plastic Limit
PZC	Point of Zero Charge
RSD	Relative Standard Deviation
UC	Unconfined Compression
UCS	Unconfined Compressive Strength
UU	Unconsolidated Undrained
VGA	Vapour Generation Assembly

CHAPTER
ONE

INTRODUCTION

1.1 THE CONTEXT

Residual lateritic soils exhibit geotechnical properties that are in many ways different from the soils of temperate and other regions. They are extensively used for the construction of roads, embankments, and dams, and to support foundations of buildings, bridges and load-bearing pavements. However, the optimum use of these soils has been limited by general problems that are often encountered while working with these soils.

Fly ash, on the other hand, is produced by thermal power stations and continues to call attention requiring alternative solution for their disposal in many parts of the world. Its contribution on the development of pozzolanic reactions has been known for quite some time and is the focus of many researchers currently for their potential use as admixtures in stabilisation of soils (Srinivasaraghavan et al., 1999; Joshi, 2000). Attempts are being made to study the behaviour of soils obtained from various parts of the world by mixing them with fly ash alone or along with some additives, in order to improve their properties and also to increase the utilisation of the waste material (Indraratna et al., 1991 and 1991b; Nicholson and Kashyap, 1993; Sivapullaiah, et al.1996; Consoli et al., 2001; Cokca, 2001). However, there have been very few efforts to understand the effect of fly ash and additives like lime on the behaviour of residually derived lateritic soil that are commonly encountered in tropical and similar climatic regions of the world. This fact assigns credence and relevance to the need of systematic study of the effectiveness of fly ash and

lime in changing the behaviour of residual lateritic soil, particularly in view of the growing demand for affordable construction materials and the increasing problems involved with disposal of fly ash.

1.2 RESIDUAL LATERITIC SOIL

Residual lateritic soils are products of in-situ weathering and partial or total decomposition of rocks. The characteristics of residual soils depend principally on the climate and parent rock material as well as localized influences such as drainage, topography and time. These soil-forming factors are optimized in tropical and other similar climatic regions where heavy rainfall and warm temperature under good drainage conditions accelerate chemical reactions that transform acid or basic rock minerals into clays, with removal of alkali-soluble bases and silica colloids. This process leads to the formation of thick horizons of reddish lateritic soil profiles rich in iron and aluminum, and kaolinite clays (Townsend, 1985). The entire process of formation of these soils consists of three major stages (Loughnan, 1969):

(i) *Decomposition*, characterised by physio-chemical breakdown of primary minerals and the release of the constituent elements SiO_2, Al_2O_3, Fe_2O_3, CaO, MgO, Na_2O, K_2O etc in simple ionic form.

(ii) *Laterization* that involves leaching of silica and bases followed by relative accumulation or enrichment of oxides and hydroxides of iron, aluminium and, titanium (known as sesquioxides) under appropriate climatic condition.

(iii) *Dehydration or desiccation* due to partial or complete loss of water leading to concentration and crystallisation of sesquioxide-rich materials and secondary materials.

The soils are available predominantly in tropical and sub-tropical regions in the form of laterites like lateritic soft rock, lateritic gravel, lateritic soil (a mixture of clay, silt, sand and gravel), and lateritic clay (soil with negligible amount of gravel) (Anirudhan, 1998).

1.2.1 Engineering Problems Associated with the Use of Residual Lateritic Soil

Residual soils develop a particular fabric, grain structure and grading-in-place, which is fundamentally different from transported soils (Vaughan, 1988). Being naturally formed at higher elevations, locations of residual lateritic soils provide excellent borrow areas for extensive use in diverse construction activities. However, effective use of this soil is often limited by a number of problems that are frequently encountered while working with them. Most common among them are difficulties experienced in handling or workability, compaction and loss of strength, particularly during the wet season. A summary of these engineering problems related to specific property or condition of the soil is presented in Table 1.1.

Table 1.1 Engineering problems of residual lateritic soil in relation to its property/ condition

Condition/Property of the soil	Related engineering problem
Variation in parent rock material and extent of weathering	Difficulty in quality control
Presence of sesquioxides	1. High plasticity 2. Difficult to obtain reproducible results of classification and index tests 3. Difficulty in handling/workability
Presence of friable micro-cluster soil structure and Al & Fe cementing bonds	1. Difficulty in obtaining reproducible results of classification tests 2. Difficulty in compaction
Collapse behaviour of the soil structure	Loss of strength and compressibility due to flooding/wetting
Wet field condition due to frequent seasonal and heavy rainfall	1. Difficulty in handling/workability 2. Effective compaction

In spite of these difficulties, lateritic soils are being used worldwide with the degree of success depending on the specific soil and site characteristics.

1.3 FLY ASH: A NEW RESOURCE MATERIAL

Fly ash is one of the most plentiful industrial by-products generated as a result of burning of coal by electric power plants. The fly ash particles are spherical and hollow with a glassy exterior surface, commonly known as cenospheres. The particles have large surface areas, low specific gravity, no plasticity and they generally fall into the grain size category of silt. It is an artificial pozzolan, that is, a siliceous or siliceous-aluminous material, which becomes cementitious when combined with an activator (lime, Portland cement, or kiln dust) in the presence of water.

1.3.1 Need for Bulk Utilization of Fly Ash

The current annual production of coal ash worldwide is about 700 million tonnes of which 75 to 85 per cent is fly ash (Joshi and Lohtia, 1997). Generation of fly ash in bulk quantities is a matter of serious concern, not only because of issues associated with disposal and utilization but also because of its threat to public health and ecology. The present day utilization of ash on worldwide basis varies widely from a minimum of 3% to a maximum of 100%, and the average utilization amounts only to about one tenth of the total ash production (Joshi, 2000).

In India, the problem of disposal of fly ash is more vulnerable and complex than many other countries due to the high ash content (30% to 50%) of the coal. There are about 72 coal based thermal power plants in India that produce about 100 million tons of fly ash every year (Sridharan, 2002). Out of this, only about 9% is utilized and the rest is disposed at a cost of more than Rs.700 crores (approximately $150 million) every year (Kumar et al., 1999).

Fly ash has been used in concrete since a long time. Nowadays, it is also being

used for manufacturing of cement, building blocks, bricks, and structural fills etc. Unfortunately, all these uses consume a relatively small part, still leaving most of it to be stored or disposed off. It has been realised that areas like geotechnical applications that do not demand stringent specifications, have vast scope for bulk utilisation of fly ash.

1.3.2 Fly Ash Stabilisation of Soils

Because of its pozzolanic character, fly ash is found to be effective in stabilization of many soils and it contributes to soil stabilisation in the following ways:

(i) Filling the voids left by the soil particles, thus giving a more compact structure to the soil

(ii) Acting as fine aggregate, when used with other cementing material such as cement, lime etc.

(iii) By participating in chemical reaction with soil particles and soil moisture thus acting as cementing material.

The pozzolanic materials present in the fly ash react with lime in presence of water to form cementitious materials by a process commonly known as hydration of fly ash. These hydration reactions for soil stabilization are (TRB 1987):

$$CaO + H_2O \Rightarrow Ca(OH)_2 \quad (1.1)$$

$$Ca(OH)_2 \Rightarrow Ca^{++} + 2\,[OH]^- \quad (1.2)$$

$$Ca^{++} + 2\,[OH]^- + \underset{\text{(silica)}}{SiO_2} \Rightarrow \underset{\text{(gel)}}{CSH} \quad (1.3)$$

$$Ca^{++} + 2\,[OH]^- + \underset{\text{(alumina)}}{Al_2O_3} \Rightarrow \underset{\text{(gel)}}{CAH} \quad (1.4)$$

The reaction products are very complex in nature, normally expressed as hydrated

calcium silicate gel or calcium aluminate gel and they can bind inert materials together. The rapid rates at which these hydration reactions occur depend on a number of factors such as the composition of fly ash, presence of other chemical constituents in the fly ash and the lime available for reaction (Edil et al., 1987). Lime present in the fly ash can also modify soils through flocculation and cation exchange changing its texture and other characteristics. Further, the pozzolanic materials, also present in soil to some extent, react in a similar manner when lime is added to soil for stabilization purpose.

1.4 THE PRESENT STUDY

Lateritic soils are conducive to cement stabilisation due to the high concentration of iron and aluminium sesquioxides in them (Townsend, 1985). The soils show limited response to lime stabilisation as most of the silica required for it has been either leached out or remains coated by the sesquioxide present in the soil. Lateritic soil is a good source of alumina but poor in free silica. The effects of lime treatment can be enhanced to a great extent if the apparent shortage of silica can be adequately supplemented by the addition of fly ash, which is high in silica content.

In recent times, attempts are being made to understand the behaviour of soils obtained from various parts of the world by mixing them with fly ash alone or along with some other additive in order to improve their properties while increasing the utilisation of this waste material as well (Thome et al., 1998; Sivapullaiah et al., 1996; Cokca, 2001; Consoli et al., 2001). However, very few attempts have been made to understand the effects of fly ash and lime on the behaviour of residually derived lateritic soils. This has been partly due to the relative lack of available equipment and technology for field mixing in tropical regions (Nicholson and Kashyap, 1993). These regions encompass many developing nations that are the major producers of fly ash and at the same time also possess significant deposits of residual lateritic soils.

India contributes approximately one tenth of the total world annual fly ash production. It also encompasses vast deposits of residual lateritic soil. Exploring the application of fly ash in making an otherwise problematic soil suitable for construction purposes is thus, a need of the hour. It is also essential to understand the behaviour of soil-fly ash mix supplemented by other additives like lime. This will not only help in optimising performance but also in overcoming the difficulties that may occur with the use of the soil under different conditions. The rationale behind the present study lies in this important premise. In this work an attempt has been made to investigate the behaviour of a highly weathered residual lateritic soil mixed with low calcium fly ash and high purity lime, emphasizing on their geotechnical application and examining the environmental implications.

The utilisation of fly ash in association with soil for construction purposes involves mixing and compaction. Hence, the entire investigation has been intended mainly to understand the plasticity, compaction, stress-strain-strength and leaching behaviour of the soil modified with fly ash and lime. All these properties have been studied with due consideration to the short-term and long-term modification, various effects of conditioning/delay in compaction, durability of the modified material and effects of flooding and wetting that is often prevalent in locations where residual lateritic soils normally develop. An important aspect of the present work have been that the entire investigation was carried out on the basis of recent developments on pre-test sample preparation and laboratory procedures appropriate for studying the behaviour of residual lateritic soils.

Considerable research has been carried out to improve the density of ash by different techniques (Gandhi et al., 2000). Both fly ash and lateritic soils are prone to particle breakage during compaction and the process is likely to change the density of the

stabilised material. Hence the effects of particle breakage on the compaction behaviour of the mixes were investigated by adopting a new method of successive re-compaction though using the same apparatus used for Indian Standard (IS) light compaction tests.

Soil for the present study was obtained from the northeastern region of India. Although there exists vast scope of utilisation of the large deposits of residually derived lateritic soil in this region, no study on the properties of the soil have been reported at all. The present study provided a scope to systematically investigate the geotechnical properties of the soil for the first time.

Another vital aspect of the study, in view of growing concerns, has been the investigation of the environmental behaviour of the modified materials, which was achieved by means of examining the leaching characteristics of the different mixes. The leaching characteristics were studied by further modifying the column apparatus recommended by the ASTM D4874-95 in order to facilitate application of higher operating pressure application.

1.5 OBJECTIVES OF THE PRESENT STUDY

The present research work is directed at developing an understanding of the behaviour of a highly weathered residual lateritic soil mixed with fly ash individually or in combination with a high purity lime by means of laboratory tests. The scope of the present work can be summarized as follows:

1. To investigate the physical, chemical and geotechnical properties of the selected soil and fly ash, particularly to understand the effects of sample pre-drying, moulding and gradation on the index and compaction properties of the soil.
2. To investigate the plasticity, compaction and stress-strain-strength characteristics of various mixes consisting of lateritic soil, fly ash and lime, particularly to identify:

i) the individual and combined effects of the addition of different proportions of fly ash and lime,

ii) the effects of conditioning/delay in compaction,

iii) the short-term and long-term changes with curing,

iv) the effects of progressive particle breakdown on the moisture-density relationship, and

v) the change in compressive strength due to extreme environmental conditions like flooding/wetting.

3. To study the shear strength behaviour of a selected soil-fly ash mix through triaxial compression test to understand (a) the effect of lime on mixture behaviour (b) the influence of compaction parameters– moisture content and dry unit weight

4. To study the flow and leaching characteristics of the raw and improved material, particularly to obtain

i) the potential and realistic estimates of leaching for metals,

ii) variation in leachate behaviour due to change in fly ash and lime content, and

iii) the effect of continuous permeation on the flow parameters and the leaching pattern.

1.6 THE APPROACH

Initial laboratory investigations were carried out on the experimental soil and fly ash to characterise their physical, chemical and engineering properties as well as to understand the effects of sample pre-drying, moulding and gradation on index and compaction properties of the soil. Based on the results of initial investigations, the soil was mixed with different percentages of fly ash and the changes in plasticity, compaction, strength and leaching behaviour were studied by conducting following laboratory tests: (a)

Liquid limit test (b) Plastic limit test (c) Compaction test (d) Unconfined compression test (e) Consolidated undrained triaxial test (f) Batch leaching test and (g) Column leaching test. Similar tests were also conducted on the soil-fly ash mixes with the addition of various percentages of lime, and the resulting changes in material behaviour were investigated.

A comprehensive literature review was undertaken that focussed on properties and engineering problems of residual lateritic soil, characteristics of fly ash and its production and disposal, soil stabilisation with fly ash and lime and leaching behaviour of fly ash stabilised soil mixes. This review is reported in Chapter 2. The physical and chemical properties of experimental soil and fly ash used in this investigation are detailed in Chapter 3. The testing programme, experimental details and results of liquid and plastic limit tests on soil, fly ash and its various mixes are presented in Chapter 4, and those for compaction tests are presented in Chapter 5. Chapter 6 and 7 deal with the testing programme, experimental details and results of unconfined compression and triaxial tests on soil, fly ash and its mixes. The test programme and results of the leaching studies are presented in Chapter 8. Finally, the summary and major conclusions of the study vis-a-vis the objectives are presented in Chapter 9.

CHAPTER
TWO

LITERATURE REVIEW

2.1 INTRODUCTION

Although substantial literature exists today concerning the use of fly ash and other additives in stabilisation of various types of soils, reports on the specific application of fly ash in improving residual lateritic soils have been rather scarce mainly due to the limited nature of work done so far in this area in view of inherent difficulties. Notwithstanding the fact, in this chapter, an attempt has been made to present a complete review of the existing literature pertaining to the present study, highlighting the following aspects:

(a) the characteristics of residual lateritic soil, its origin and formation, geotechnical properties and engineering problems,

(b) fly ash, its production and physical, chemical and geotechnical properties,

(c) use of lime as admixture for improvement of soil and fly ash,

(d) combined use of fly ash and lime for stabilisation of soils with particular reference to change in plasticity, compaction and strength behaviour of residual lateritic soils, and

(e) leaching characteristics of fly ash and lime stabilised soils

2.2 RESIDUAL LATERITIC SOIL

2.2.1 Origin and Formation

Oliver (1984) has described residual soils as created by the intensive and deep in-situ physical and chemical weathering of rocks resulting in a mineral structure that is in greater equilibrium with the rock's new physico-chemical environment. Usually chemical processes tend to predominate in the weathering of igneous rocks and physical processes dominate the weathering of sedimentary and metamorphic rocks. Both kind of processes are closely interrelated and one process never proceeds without some contribution of the other. In both the cases, weathering progresses from surface to downwards with the degree of weathering decreasing with depth (Mitchell, 1976).

Srtakhov (1967) from his study on influence of global climate on depth of weathering and weathering products observed that climate is the chief factor dominating the depth of weathering profile of residual lateritic soils. Figure 2.1 summarizes the effects of global climate on rock weathering and formation of the various weathering products.

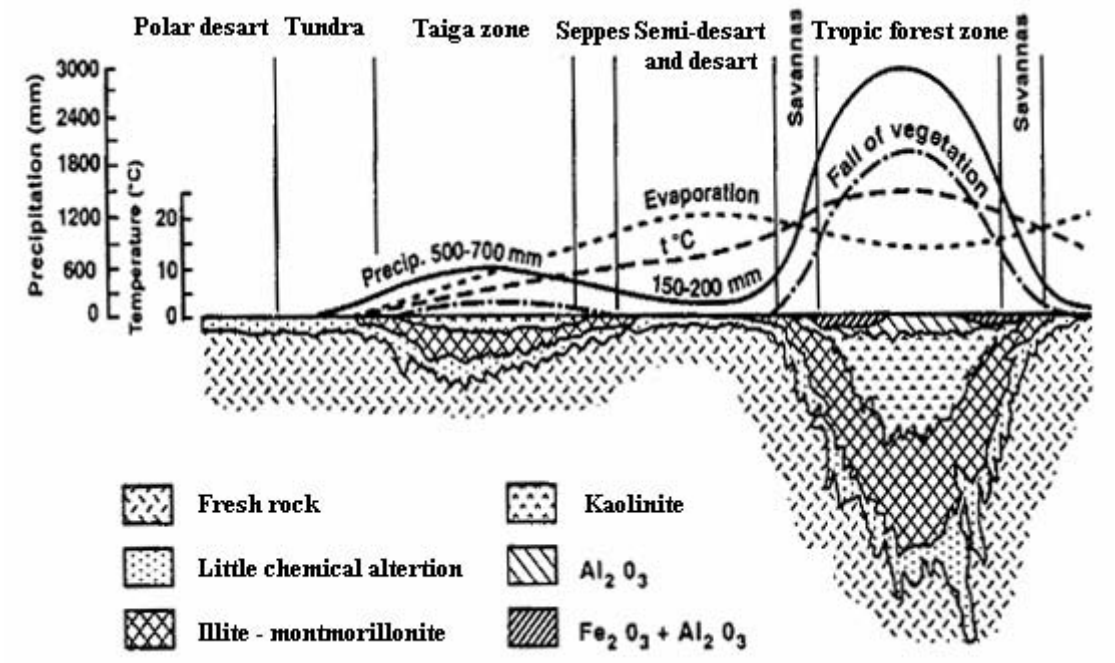

Figure 2.1 Influence of global climate on depth of weathering and weathering products (Srtakhov, 1967)

As shown in the figure, climatic conditions in tropics are favourable to the formation of thick horizons of reddish lateritic soils rich in iron and aluminium. As leaching decreases with depth, sufficient silica is retained to combine with alumina to form kaolinite.

Parent material significantly influences the soil formation because of their variation in weathering rate. Mohr et al. (1972) reported that the zone of alteration in acidic quartz rocks are quite thick because of the lower weathering rate of rock forming minerals (like quartz, feldspars, hornblende and micas) than that of the minerals that form basic rocks. Figure 2.2 shows the rock forming minerals of some principal igneous rocks. Fig. 2.3 presents the weathering sequence of these minerals.

Magma	Mode of origin	Name of rock	Principal Rock-Forming Minerals													
				Potash-feldspars			Plagioclases									
			Quartz	Orthoclase	Microcline	Sanidine	Albite-oligoclase	Andesine	Labradorite-Anorthite	Muscovite	Biotite	Hornblende	Hypersthene	Augite	Olivine	Volcanic glass
Granitic	P	granite	•	•	•		•			•	•	•				
	E	rhyolite	•			•	•				•	•				•
Grano-dioritic	P	quartz diorite	•				•	•			•	•				
	E	dacite	•				•	•			•	•				•
Dioritic	P	diorite						•	•		•	•	•	•		
	E	andesite						•	•		•	•	•	•		•
Gabbriod	P	gabbro							•					•	•	
	E	basalt							•					•	•	•

P= plutonic E= effusive

Figure 2.2 Constituents of principal igneous rocks (Mohr, 1972 as referred by Townsend, 1985)

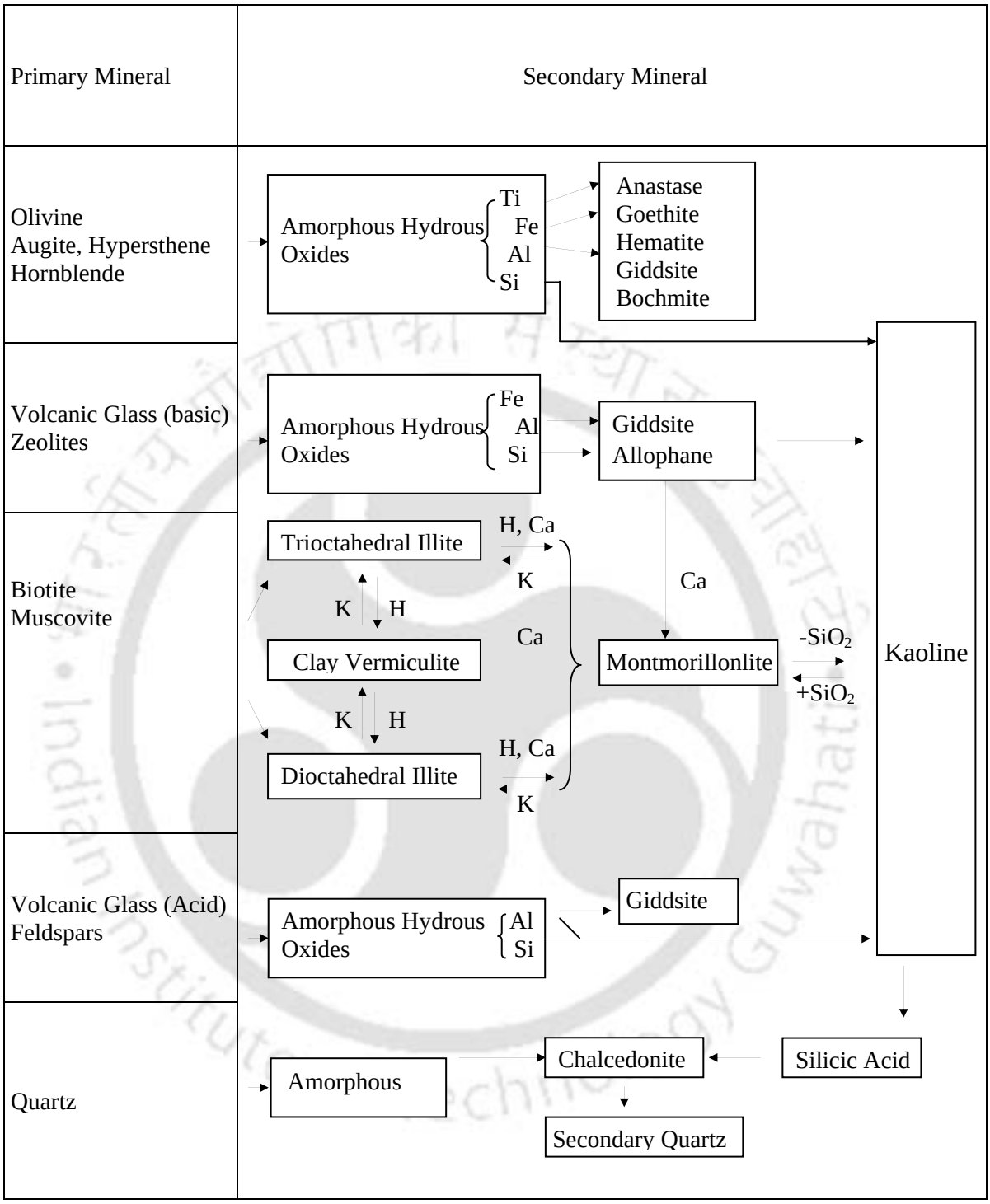

Figure 2.3 Weathering of primary rock forming minerals. (Fields and Swindale, 1954 as referred by Townsend, 1985)

Weathering products, as shown in Fig. 2.3 indicates that kaoline is the end product

of all the igneous rocks. Also, sesquioxides of iron and aluminium can occur due to weathering of practically all igneous rocks. Loughman (1969) reported that in acid leaching environment, as minerals weather, much of the silica and basic cation are dissolved and slowly leached from the soil profile with good drainage.

Less silica present in the remaining solution means the proportion of the alumina will be high enough to result in formation of minerals like kaolinite, halloysite (1:1 minerals) etc. Conversely, montmorillonite, illite (2:1 minerals) are most common in poorly drained soils that have little or no leaching. Table 2.1 summarizes the weathering products of various environments.

Table 2.1 Weathering products in relation to some specific environments (After Loughman, 1969)

Environment	Hydrogen ion con--centration	Oxidation/ Reduction potential	Metallic ions present	Mineralogy
Non leaching, hot Rainfall 0-305 mm	Alkaline	Oxidizing	Ferric iron	2:1 Clays, hematite, carbonates and salts
Non leaching , below Water table	Alkaline to neutral	Reducing	Ferrous iron	2:1 Clays, siderite, organics
Moderate leaching, temperate climate rainfall 305-1,270 mm	Acid	Oxidizing to Reducing	Sesquioxide iron and aluminum	1:1 Clays, hematite, organics
Intense leaching, hot Rainfall >1,270 mm	Acid	Oxidizing	Sesquioxide iron and aluminum	1:1 Clays, hematite, gibbsite
Intense leaching, cool Rainfall >1,270mm	Acid	Reducing	Alumina and silica	1:1 Clays, gibbsite, organics

Figure 2.4 illustrates typical residual soil profile. As shown, it consists of three indistinctly divided zones (Little, 1969). The upper zone consists of highly weathered and leached soil often reworked by burrowing animals and insects or by cultivation, and intersected by root channels. The intermediate zone also consists of highly weathered material but exhibits some features of the structure of the parent rock and may contain

core stones. This zone often contains pedogenic materials such as nodules of calcium and iron salts, which may give it a spotted or mottled appearance.

Depending on climate and ground water condition, laterization can occur within the upper zone. The laterization process results in relative accumulation or enrichment of sesquioxides (mainly Al_2O_3, Fe_2O_3, and TiO_2) (Loughman, 1969). Sometimes, secondary emplacements of these salts occur due to evaporation of iron or aluminium near surface ground water. This causes increased cementation of sesquioxides of these salts.

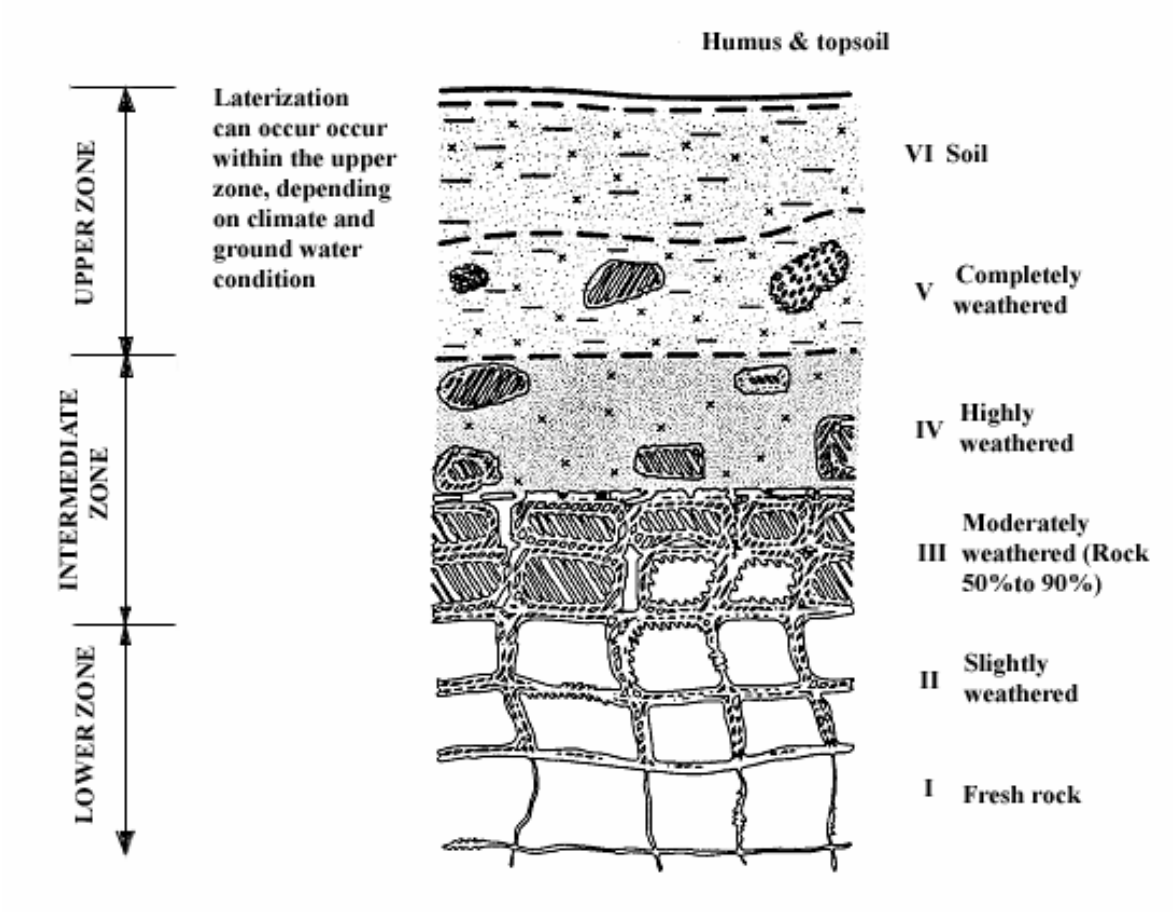

Figure 2.4 Schematic diagram of tropical residual soil profile (Little, 1969)

2.2.2 Geotechnical Properties and Engineering problems

The geotechnical properties of residual lateritic soil have been reported in detail by many authors like Gidigasu (1976), Mitchell and Sitar (1982), Uehara (1982), Desai (1985), Townsend (1985), Wesley (1988), Wesley (1990), and Blight (1997).

Residual lateritic soils exhibit variability both laterally and vertically due to the variation in parent rock material and extent of weathering. Hence quality control becomes difficult. Material must be mixed or selected or both to maintain homogeneity of the material in the soil structure (Blight, 1988).

In lateritic soils, it is difficult to obtain reproducible results of classification tests, as sesquioxides present in the soil tend to impart change in engineering properties upon wetting and drying as well as remoulding. The effect of drying has been attributed to increased cementation due to oxidation of the iron and aluminium sesquioxides (Townsend, 1985). Figure 2.5 shows the decrease in plasticity observed upon drying for several lateritic and allophanic soils.

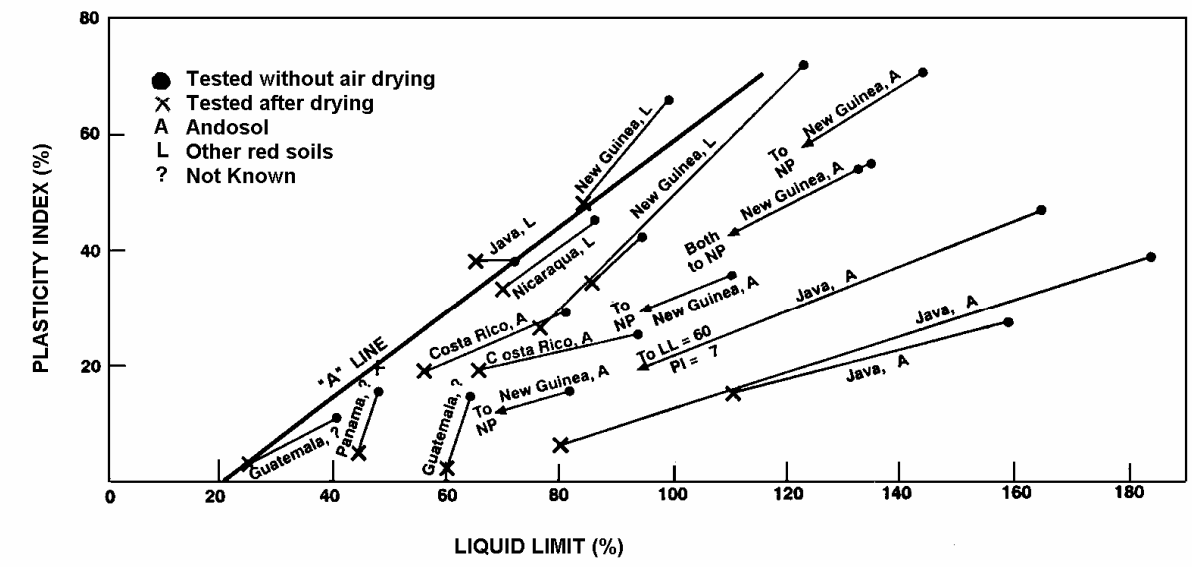

Figure 2.5 Effect of drying on plasticity of Volcanic soils (Morin and Todor, 1975 as referred by Fourie, 1997), (NP = Non Plastic)

The effect of remoulding is a trait of lateritic soils by which the friable aluminium

and iron cementing bonds between clay clusters are broken and finer fractions result (Townsend, 1985). Table 2.2 shows the effects of remoulding upon Atterberg limits. Because of these reasons, the soils need a different approach even at the initial stages of identification and classification.

Table 2.2 Remoulding effects on Atterberg limits of lateritic soils (as referred by Townsend,1985)

Soil type and location	Liquid Limit		Plasticity Index		Reference
	Natural	Remoulded	Natural	Remoulded	
Red clay, Kenya	74	84	36	45	Newill, 1961
Red clay, Kenya	77	91	16	32	Newill, 1961
Lateritic soil, Cuba	46	53	15	22	Winterkorn and Chandrasekharan, 1951
Lateritic soil, Panama	60	70	21	30	Townsend et al., 1969

Sesquioxides present in lateritic soils exhibit very high plasticity and tend to impose its plasticity to soil (Sridharan et al., 1991). They usually remain coated over the clay, quartz and other mineral particles forming clay clusters and aggregates of particles providing enormous surface for adsorption. As the environment, where the soils usually form, are characterised by frequent or seasonal heavy rainfall, the workability of the soils get considerably reduced and handling of the soils under these conditions may be difficult (Blight, 1997).

Townsend (1985) reported that the standard Proctor unit weight of lateritic soils could be quite low although its constituents have relatively high specific gravity due to the presence of porous micro-cluster structure of these soils. Typical values for lateritc soils (less lateritic gravel) range from 1.281 – 1.682 g/cm^3. Table 2.3 presents examples of dry unit weight and optimum moisture contents (OMC) for selected lateritic soils.

Table 2.3 Examples of Proctor dry unit weight and optimum moisture contents for

selected lateritic soils (as referred by Townsend, 1985).

Soil type and location	LL (%)	PI (%)	Dry unit weight (g/cm^3)	OMC (%)	Reference
Lateritic soil, Panama	58	18	1.37	35	Townsend (1970)
Lateritic soil Venezuela	45-39	14-13	1.65-1.71	20.5-17.8	Prusza (1983)
Granite saprolites, Brazil	35-20	9-18	2.01-1.79	7-11	Taiji (1979)
Gneiss saprolites, Brazil	35-31	8-20	1.82-1.61	12-20	Taiji (1979)
Basalt saprolites	45-98	10-43	1.49-1.22	28-42	Taiji (1979)

Similar to the effects of drying on index properties, drying of lateritic soil from insitu moisture content may also change the compaction characteristics of lateritic soils. Gidigasu (1974) studied the influence of sample preparation and laboratory procedure on compaction characteristics of a residual lateritic soil and observed that not only the OMC of the soil significantly altered by air or oven drying before compaction, the maximum dry density (MDD) was also considerably changed (Fig. 2.6).

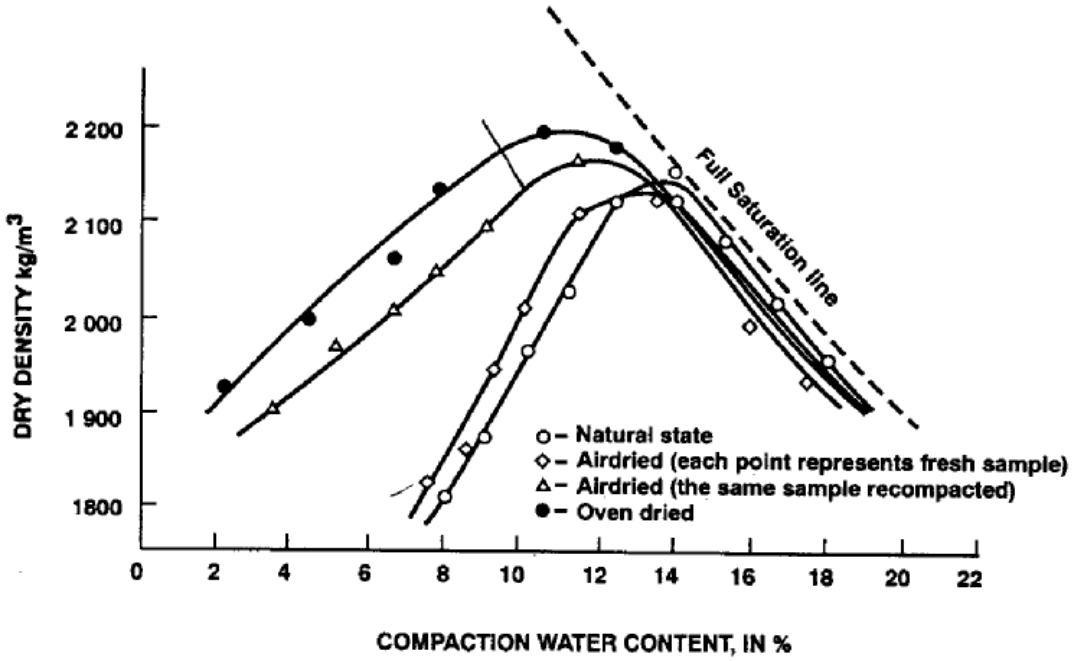

Figure 2.6 Influence of method of sample preparation and laboratory procedure on compaction characteristics of a lateritic soil from Ghana (Gidigasu, 1976)

Simmons and Blight (1997) observed that the compaction characteristics of residual soils might vary depending on the method of applying the compaction energy. Gidigasu and Dogbey (1980) reported that compaction often results in progressive breakdown of particles in residual soils that develop particularly over granite. The soil structures in many lateritic soils are often lightly cemented and the weathered soil particles tend to break down under the compaction effort.

The chief difficulties involved with using lateritic soils concern with field compaction and water content. As field conditions usually remain wet of optimum for a major part of the year, drying prior to compaction is often essential (Townsend, 1985). Moreover, the micro-cluster structure of the soil is quite friable and excessive field compactive effort in the presence of high water content can create a morass of red mud making compaction quite difficult. Because of all these complexities, Simmons and Blight (1997) emphasized the need for special consideration for the effective compaction of lateritic and other residual soils.

Residual soils develop a particular fabric, grain structure and grading in place which is fundamentally different from transported soils (Vaughan, 1988). Therefore, the insitu shear strength of residual soils are affected by stress history, grain/particle strength, bonding, relict structures and discontinuities, anisotropy and initial (insitu) void ratio or density as well as the degree of saturation of the soil (Brenner et al., 1997). Because of these complexities, difficulties have been reported by many researchers in recommending standard and consistent procedures for determination of insitu shear strength parameters (Gidigasu, 1976; Vaughan, 1988; Raju and John, 1975). In case of lateritic soils, shear strength parameters are higher than those suggested by plasticity index, with effective friction angles typically between 20 and 30 degrees for lateritic

clays, and between 30 and 40 degrees for lateritic gravels (Townsend, 1985). Table 2.4 presents the examples of strength parameters for lateritic soils.

Table 2.4 Typical strength parameters for lateritic soils

Soil type and location	ϕ' (degrees)	c' (kPa)	Reference
Granetic lateritic soil, Venezuela	21.5	20	Prusza (1983)
Lateritic clays, Africa	22.5	0-100	Horn (1982)
Lateritic gravels,Africa	37.5	0- 40	Horn (1982)
Lateritic soil, Panama	38.0	0	Townsend (1970)
Granetic laterite, Brazil	31.0	0	Vargas (1973)
Lateritic soil, India	19.0-27.8	25-50	Allam and Sridharan (1984)

Lateritic soils have been used worldwide for construction of earth dam embankments, with the degree of success dependent on the specific soil and site condition (Townsend, 1985). Good shear strength values are reported for compacted laterites from many parts of southern India (Vinod, 1997; Basavarajaiah et al., 1981; Chandrasekhara Rao and Rajeevalochanam, 1989).

Several authors (Foss, 1973; Vargas, 1973; Fooks, 1990; Barksdale and Blight, 1997; Rao and Revanasiddappa, 2002) have reported collapse behaviour of the soil structure in lateritic soils upon saturation. Typically, the consolidation curves of the soil exhibit an apparent preconsolidation pressure due to cementation and agglomeration of clays (or possibly due to soil suction). However, upon saturation, the apparent preconsolidation pressure decreases and large settlements occur (Townsend, 1985).

Residual soils, particularly developed from decomposition of granite, in its natural state and also when used as a construction material, often exhibit loss of stability and increased settlement during rainy seasons due to the loss of cohesive component of strength resulted from strong inter-particle bonds that develop due to repeated seasonal drying and wetting (Lee and Coop, 1995).

2.2.3 Distribution of Lateritic Soils in India

Large deposits of lateritic soil formations are found extensively in many places over peninsular India (Fig.2.7). There are also numerous occurrences over parts of Maharastra, Madhya Pradesh, Orissa, West Bengal, and parts of Northeast India. Most of these soil formations are of residual type showing wide variations in structure and texture as well as chemical, physical and other engineering properties both in vertical as well as lateral direction (Krishnan, 1982).

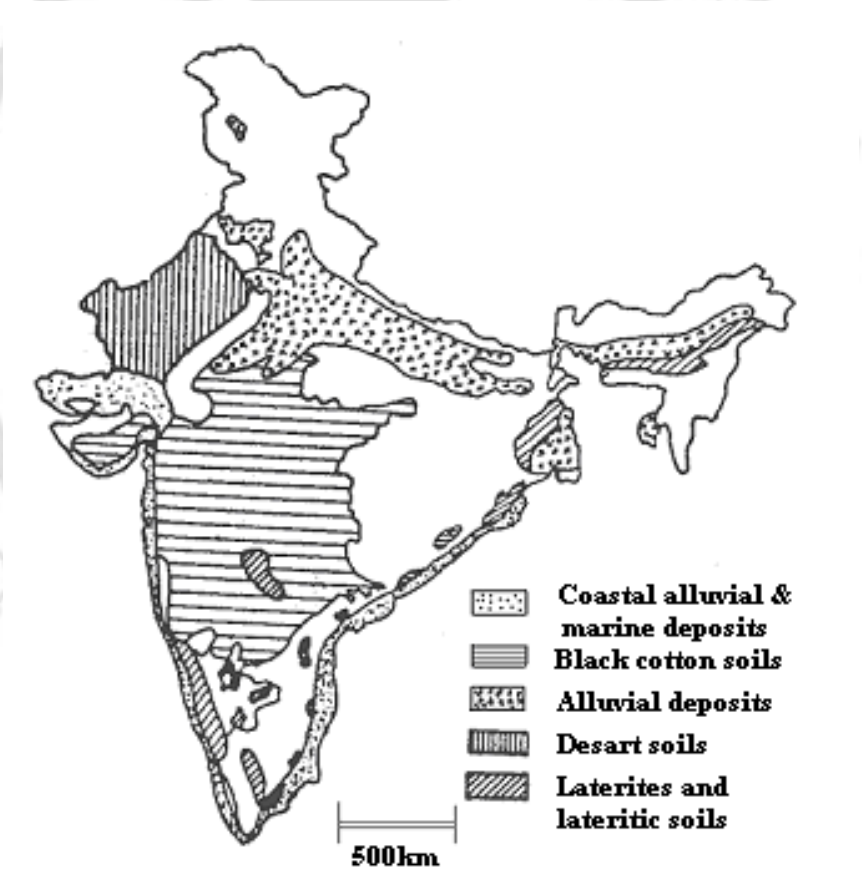

Figure 2.7 Distribution of soils in Indian mainland. (Katti et. al, 1975)

These deposits are derived mainly from dolerite, and basalt of Decan Trap formation as well as from feldspathic rocks containing a moderate amount of alumina and silica, such as gneisses and granites (Krishnan, 1982). Residual lateritic soils in northeastern part of India are found to confine mostly in the detached part of Indian peninsula forming the Shillong plateau, Mikir hills and their numerous extensions in the state of Assam. These soils are also extensively found as capping in numerous isolated inselbergs formed by the projection of the above formation scattered along north and south bank of river Brahmaputra in Goalpara, Kamrup, Darrang and Nowgaon districts. In this region, the soils are mostly formed by the weathering of granites and gneisses (Deka, et al.1998).

In the past, attempts have been made to investigate the problems associated with identification, classification and other engineering properties making use of all the natural laws known to be operative under the particular and specific conditions prevailing (Wooltronton, 1976). In India, few serious attempts have been made in this regard to characterize laterites and lateritic soils adopting this concept (Anirudhan, 1998). Most studies in India have been carried out in the southern states like Kerala, Karnataka, Tamilnadu, Madhya Pradesh and Maharastra. A summary of engineering properties of Indian Laterites is presented in Table 2.5.

It can be seen from Fig. 2.7 that large deposits of lateritic soils are found in the northeastern part of India. The average normal rainfall in the area is 2340 mm/year (Kalita, 1984). The hot and humid climate of this region is very much conducive to the formation of lateritic soil. Although they occur extensively, no systematic study has been reported so far.

Table 2.5 Engineering properties of Indian laterite deposits (after Anirudhan, 1998)

Region	Grain size	Specific gravity	Natural water content (%)	Index properties				Compaction MDD (gm/cc)	Shear strength properties (in kg/cm^2)	Reference
				Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Shrinkage limit (%)			
Goa	Clay 10 to 32% Silt 47 to 66% Sand 15 to 22%	2.72 to 2.96	37 to 51	48 to 61	34 to 40	14 to 21	34 to 37	OMC = 25 to 30% MDD = 1.4 to 1.48	ϕ = 30° to 32° c' = 0.25 to 0.5	Katti (1996)
Bombay	Clay 0 to 26% Silt 22 to 49% Sand 26 to 70%	2.76 to 2.92	18 to 44	31 to 51	22 to 35	9 to 19	18 to 28	OMC = 14 to 25% MDD = 1.5 to 1.82	ϕ = 36° to 42° c' = 0 to 0.1	Katti (1996)
Calicut	Gravel 16 to 44%		16 to 28	43 to 71	22 to 30	21 to 41			ϕ = 32.5° to 26.3° c = 0.25 to 0.6	Chandrakaran et al.(1993)
Calicut Rajamundry	Clay 18% Clay 39%		25 to 30 15 to 20	72 62	38.8 24.0	33.5 38.0				Seshagiri and Rao (1981)
Quilon Kerala		2.68	22	50 to 54	33 to 34	17 to 20			ϕ = 29° to 31° c = 1.0 to 1.25	Raju and John (1975)
Changanachery Kerala		2.75 to 2.8	22	50 to 54	33 to 34	17 to 20	26		ϕ = 30° c = 1.0	Raju and John (1975)
South Karnataka		2.4 to 2.65	10 to 20	40 to 50	30 to 40	10 to 12	15 to 20	OMC = 3 to 20% MDD = 1.65 to 1.8		Sreekantiah (1984)
Ranchi		2.76		43.0	26.5	17.0	21.2	OMC = 17.5% MDD = 1.85	ϕ = 23° (at OMC) c = 0.53	Sandrasekhara Rao (1989)
Jamshedpur	Silt 2.5% Sand 67.5% Gravel 30%	2.68	21	32.5	25.8	6.7	26	OMC = 17% MDD = 1.94 k= 1.4 × 10^{-4}cm/sec	(at OMC) ϕ = 29.5° c = 0.41	Khan and Pise (1996)
Bhubaneshwar Orissa	Clay+Silt 45 to 63% Sand 37 to 54% Gravel 0 to 2%	2.48 to 2.7	6 to 23	30 to 45	12 to 26	18 to 19			SBC from plate load test = 0.7 to 2.1 kg/cm^2	Das and Mahalik (1981)
Calicut laterite gravel	Gravel 80 to 90%	2.68 to 2.78		48	29	18		OMC = 28% MDD = 1.95		Nambiar (1992)

2.3 FLY ASH

2.3.1 Production, Storage and Disposal

Fly ash is the finely divided mineral residue resulting from combustion of ground or powdered coal at electricity generating plants (ASTM C 618 – 94a, 1995). During combustion, the volatile matter is vaporized and the majority of carbon is burned off. The mineral matter associated with the coal, such as clay, quartz, calcite, pyrite, and feldspar disintegrate and / or slag to varying degree. The finer particles of the slagged minerals are carried by flue gases from the flame zone and are subjected to quenching before being collected in the dust collectors. During quenching various alkali vapours condense on the particles. Therefore, the outer surface of most particles appears to be rough and also the degree of strain in the glass from surface of the interior reduces significantly (Joshi, 1970). The finer particles that escape with flue gases are collected as fly ash using cyclone separators, electrostatic precipitators or bag houses.

Fly ash from the electrostatic precipitators is periodically discharged into the hoppers. The fly ash from the hoppers is generally hydraulically sluiced into storage ponds or lagoons. Hopper ashes, as collected, contain very little moisture. But the moisture content of the ponded ashes can vary significantly. The fly ash, which is planned to be used as a pozzolanic material, is stored in silos from where it is loaded pneumatically or by gravity into tank trucks or similar rail road car for transportation to construction sites.

2.3.2 Classification of Fly Ash

Fly ash is typically classified by ASTM C 618 as class F or class C depending upon coal source. Class F fly ash is produced from burning anthracite and bituminous coals and contains only minor amounts of calcium oxide (CaO). This fly ash itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium oxide at ordinary temperature to form cementitious

compounds (Chu and Kao, 1993). Class C fly ash is normally produced from lignite and sub-bituminous coals, and usually contains a significant amount of calcium oxide (CaO), also known as lime. In addition to having pozzolanic properties, Class C fly ash also is cementitious (ASTM C 618-99). Colour is useful for estimating the calcium oxide content and organic content of fly ash. Lighter colour fly ash generally has a greater percentage of CaO.

2.3.3 Physical and Chemical Characteristics of Fly Ash

The fly ash particles are spherical and hollow with a glassy exterior surface, commonly known as cenospheres (Sharma and Krishnamoorthy, 1992). The particles have a large surface area, low specific gravity, no plasticity and generally fall into the grain size category of silt as per Unified Classification System (Kokabu, 1968; Sharma and Krishnamoorthy, 1992; Dayal et al., 1989). Typical range of these properties reviewed from different sources are tabulated in Table 2.6

It is a usual and standard practice to report the chemical composition of fly ash in terms of oxides. Fly ash is basically comprised of compounds of silicon, aluminum, iron and calcium with smaller amounts of other elements such magnesium, sodium, potassium, titanium and vanadium etc. Typical ranges of these compounds as reviewed from different sources are also shown in Table 2.6.

The inorganic oxides in fly ash may be in crystalline or glassy phases. The relative amounts of crystalline and glassy phase materials in fly ash depend largely on the boiler design and operation used at the particular power plant. When the maximum temperature of the combustion process is above 1200° C and the cooling time is short, the ash is mostly glassy phase material (McCarthy et al. 1987). When a more gradual cooling of the ash particles occurs, crystalline-phase calcium compounds are formed.

It can be seen from Table 2.6 that some of the fly ashes exhibit relatively high

specific gravity. Joshi and Marsh (1987) attributed this to the presence of the large amounts of iron rich particles in the material.

Table 2.6 Physical, and chemical characteristics of fly ashes

Parameter	Origin of fly ash		
	United States	Canada	India
Specific gravity	2.14-2.69	1.91-2.94	1.46-2.66
Specific surface area, cm^2/g	1579-5550	1700-5900	1000-6000
Grain size	3.55-36.90	9.8-44.8	16.8-92.5
	Percentage retained on #325 sieve	Percentage retained on 45μm sieve	Percentage retained on 75μm sieve
Chemical composition			
Silicon dioxide, %	30.9-62.8	31.7-57.6	53-63
Aluminium oxide, %	12.3-27.0	13.6-23.7	27-37
Iron oxide, %	2.8-24.4	3.5-42.2	3.3-6.1
Calcium oxide, %	1.1-30.5	0.8-20.0	0-3
Magnesium oxide, %	0.7-6.7	0.1-4.0	0-0.8
Sulphur oxide, %	0.3-3.9	0.1-1.0	---
Sodium oxide, %	0.2-2.0	0.1-5.8	0.1-0.4
Potassium oxide, %	0.2-3.0	1.0-2.9	0-0.9
Loss on ignition, %	0.0-16.6	0.1-6.3	0.4-3

Ref: United States – EPRI (1987)
Canada – Joshi and Marsh, (1987), Joshi and J Lohtia, (1997)
India – Pandian et al. (1998a), Sridharan (2002)

Composition and properties of a fly ash depend on a number of factors including the coal source, boiler design, and type of ash collection system (Torrey, 1978; Yudhbir and Honjo, 1991; Ferguson, 1993). The minerals present in the coal dictate the elemental composition of the fly ash, but the mineralogy and crystallinity of the ash are dictated by the boiler design and operation (Baker 1987).

The presence and availability of SiO_2 in fly ash is of great importance for its

reaction with the lime for pozzolanic reactivity. The silica, alumina, and ferric oxide contents of fly ash is responsible for production of cementitious compounds on hydration reaction with lime.

2.3.4 Geotechnical Properties

Digioria and Nuzzo (1972) reported that fly ash is nonplastic material and exhibits a compaction curve similar to that of cohesive soils. They observed that compaction properties have an apparent relationship with the specific gravity.

Gray and Lin (1972) observed that compressibility of fly ash depends on initial density, degree of saturation and pozzolanic reactivity and reported that fly ash has the requisite properties for use in load bearing fills or highway sub-bases.

Leonard and Bailey (1982) studied the geotechnical properties of pulverised coal ash to determine its suitability as a structural fill and reported that the unconfined compressive strength for fine ash is higher than those determined for the coarser ash specimens.

Toth et al. (1988) reported that the physical behaviour of fly ash is similar to that of silt, and because of its low unit weight, it is quite suitable for structural fills over soils with low consolidation pressure. Fly ash is less sensitive to variation in dry density with moisture content compared to the natural soils. Also, the apparent cohesion exhibited by fly ash vanishes upon saturation. They concluded that fly ash is quite suitable for structural fills over soils with low preconsolidation pressure.

Dayal et al. (1989) made an attempt to determine the liquid limit of six fly ash samples using Casagrange method and reported that fly ash is 100% non plastic and it is not possible to determine its plastic limit. They further noted that it is difficult to make a clean and straight groove even with the ASTM tool meant for low plasticity soil.

Martin et al. (1990) studied the compaction behaviour of six fly ashes and reported that compaction curves are flat compared to fine grained soils. Fly ash in a moist but

unsaturated condition displays an apparent cohesion due to the tensile stress of retained capillary water but this cannot be relied upon for long-term stability analysis as shearing resistance significantly influences the strength property of fly ash.

Das and Prakashi (1990) reported high value of water content (40%) for a fly ash and attributed the reason to the hollow shape of fly ash particles or cenospheres that can hold considerable quantity of water internally.

Indraratna et al. (1991a) studied the engineering behaviour of a low carbon, pozzolanic fly ash and reported complete loss of cohesion intercept due to saturation without significant change in friction angle. Based on comparison of saturated and unsaturated fly ash specimens, they concluded that shear strength of fresh fly ash results predominantly from frictional resistance.

Yudhbir and Honjo (1991) reported that high calcium fly ashes with practically no carbon give MDD values 17 to 18.5 kN/m^2 with water content ranging between 12 to 15%. Low calcium fly ashes on the other hand give MDD values as low as 9.3 kN/m^2 and OMC of about 45%. Large numbers of fly ashes have MDD values varying between 10.3 to 16 kN/m^2 with OMC ranging between 16 to 35% and specific gravity in the range of 2 to 2.4. they also noted that some fly ashes might achieve unconfined strength of 20 MN/m^2 in 28 days whereas other may only attain a maximum value of 0.4 MN/m^2 in 16 weeks.

Based on extensive laboratory and field research works on several Spanish fly ashes, Santayana and Mazo (1994) reported that compaction characteristics of fly ashes resembled fine-grained soils. They obtained low values of MDD for the fly ashes and observed that compaction curves were relatively flat. A high compaction effort was found to cause breakage of mostly spherical and hollow fly ash particles that increased the dry density but resulted in lower strength.

Dayal, U. (1995) examined the suitability of fly ash as a construction material. Singh and Panda (1996) reported that the shear strength of fly ash is primarily dependent

on its internal friction value and it is sensitive to moisture content changes. Dutta (1998) reported that fly ash exhibits self-hardening properties and its shear strength value is most significantly affected by its composition, moisture content and age.

Prashanth et al. (1999) studied the compaction behaviour of fly ashes and emphasized the difficulty in comparing the compaction curves due to difference in specific gravity of the fly ashes. They observed that finer fly ashes gave a lower solid volume occupation (V_s/V), higher range of water volume content (V_w/V_s) and higher optimum water volume content for the same compactive effort because of their larger surface area and smaller pores. Similarly rough-surfaced fly ashes gave a lower solid volume occupation and wider compaction curves for the same compaction effort, the size of the fly ash particles being the same. They also reported that on dry side of optimum, the compactive effort applied is resisted essentially by the shear resistance mobilised as a result of an increase in the effective negative pore water pressure and on the wet side of optimum by both shear resistance and the pore pressure developed. Based on a number of undrained triaxial strength tests, they concluded that shear strength on dry side of optimum at all water volume contents and for all fly ashes is essentially the same. Further, the strength of samples on the wet side of optimum is reduced due to built up of a positive pore pressure.

Boominathan and Hari (1999) investigated the static and cyclic shear strength characteristics of a fly ash. They observed that the static liquefaction resistance of fly ash was higher than that of cyclic liquefaction resistance. On monotonic loading following liquefaction, fly ash always responds in dilative manner even though it is contractive under pure monotonic loading.

Pandian et al. (1999) studied the effect of curing on the strength gain by fly ashes and reported that reactive silica and free lime content can have major influence on the

pozzolanic reactivity of fly ashes. A fly ash containing both reactive silica and free lime improves the strength even without addition of lime.

Sridharan et al. (2001), from a comparison of various compaction curves of a number of Indian fly ashes reported that these compaction curves resembled cohesion-less sands or sandy soil. They also observed that water content did not have an appreciable effect on dry unit weight, and the compaction curves were relatively flatter compared to that of the natural soils.

The shear strength characteristics of Indian fly ashes obtained under different test conditions, test procedures, densities, stress histories, and as studied by Shidharan et al. (1998); Srinivas et al. (1999) and Shidharan (2002) are summarised in Table 2.7.

Table 2.7 Typical ranges of shear strength parameters of Indian fly ashes obtained by different methods and test conditions

Method Test condition	Angle of internal friction (Degrees)	Internal cohesion (kPa)
<u>Direct shear test (Drained condition)</u>		
Loose dry	29 to 36 (ϕ_d)	---
Compacted	28 to 42 (ϕ_d)	9.8 to 39.2 (c_d)
Compacted and saturated	28 to 41 (ϕ_d)	---
<u>Triaxial test (Undrained conditon)</u>		
Compacted (at 95% proctor maximum density) and saturated	20 to 41 (ϕ_{cu})	0
Compacted (at 95% proctor maximum density) and saturated	26 to 39 (ϕ')	16 to 96 (c')

2.4 LIME MODIFICATION OF SOIL AND FLY ASH

2.4.1 Modification of Soil

Principles of soil stabilisation with admixtures have been widely dealt-with in texts by Kedzi (1979), Bell (1988), Hausmann (1990), Oweis and Khera (1990), Daniel (1993), Fell et al. (1993).

Lime either in the form of quicklime or as hydrated lime has been added to clay soils to improve physical properties for centuries. This process generally known as lime stabilisation, has been beneficial in improving the strength and stiffness characteristics of road foundations. Numerous literatures are available on lime stabilisation of soils. For the present investigation, however, only a few relevant ones, mainly concerned with the mechanism of lime stabilisation, are discussed in the following section.

The reaction between lime and clay can be divided into two processes (Sherwood, 1993): (a) modification that occurs typically within 24 hours, although it sometimes takes up to 72 hours depending on the clay mineral involved and (b) stabilisation that refers to a process which occurs far more slowly, producing a long-term strength gain by the progressive crystallisation of gels that are created once the lime has attacked the clay minerals.

The observed changes and effects on engineering properties are (Rogers and Glendinning, 1996): (a) a substantial reduction in the thickness, and thereby stabilisation of the adsorbed water layer resulting in reduced susceptibility of the clay to the addition of water (b) flocculation of clay particles, caused by greater attraction between the clay particles resulting in the reduction in the thickness of the diffused double layer (c) increase in internal angle of friction between agglomerates and hence greater aggregate shear strength (d) textural change from a plastic clay to a friable material that in granular is nature (f) the clay exhibits reduced plasticity as evidenced by a significant reduction in the

plasticity index that is usually caused by a considerable increase in the plastic limit of the clay. Clay minerals with a high cation exchange capacity (CEC), such as montmorillonite, undergo a large change in properties on mixing with lime whereas those with a low CEC, such as illite, undergo a small change.

Dumbleton (1962) investigated the change in plasticity of London clay and demonstrated that the effects of lime on plasticity vary with clay type, percentage lime addition and time and noted that the trends of variation in plastic limit are more consistent.

Based on the research carried out by Eades and Grim (1966), a methodology was derived (British Standards Institution, 1990) for determining the quantity of lime need to be added to soil in order to bring about pozzolanic reactions.

Diamond and Kinter (1965) observed that liquid limit determinations of lime stabilised soils produce greater variations due to the more sensitivity of liquid limit to cation exchange than the plastic limit.

Thomson and Eades (1970), however, observed that variation in the method of measuring pH, the freshness of lime used, and the clay mineral types might affect the test results suggested by Eades and Grim (1966).

Osula (1996) reported a study of Nigerian laterite modification to examine the practicality of use in road construction. The laterite was a highly weathered tropical soil having clay content of 13% consisting predominantly of kaolinite. It was observed that full benefit of lime modification is not achieved immediately after mixing. With higher lime content, modification was found to be quicker. It was also observed that the change in the properties of laterite are considerably greater to the liquid limit than the plastic limit to the extent that it is the considerable reduction in liquid limit that makes the material almost non-plastic after full modification.

Holt and Freer-Hewish (1996) studied the effects of mellowing duration on four

different types of lime treated soils and reported that MDD and OMC of the mixtures increased with mellowing duration, but changes were marginal after one day. Prolonged mellowing was deleterious to the strength gain process, although in certain cases it was found desirable to mellow the soil-lime mix up to one day. They suggested that dry mixed soils should be compacted immediately after mixing, with ideally not more than half a day delay for maximum strength and durability to be achieved. Wet mixed soils, however, required at least a half a day mellowing period, but not more than one day for maximum strength.

Perry et al. (1996) highlighted the variety of uses of quicklime and concluded that innovative use of lime has many benefits. They emphasized the need for proper investigation prior to taking up a project, due to complex nature of the lime stabilisation process.

Rogers and Glendinning (1996) examined the changes brought about to different soils by the addition of relatively small quantities of lime and reported that lime modification causes considerable change to the nature of a clay at lime contents below the fixation (i.e. ICL) level. The changes in plasticity are governed by the mineralogy of the clay and the quantity of clay contained in the soil. They also observed that the plastic limit is the best indicator of the lime content necessary to achieve the degree of modification. Rogers and Glendinning (1997) highlighted the various effects and suggested an alternative interpretation of the ICL test, and concluded that plasticity changes should be used to confirm the lime requirement for solidification, and that strength determination should be used for design.

Sivapullaiah et al. (1998) highlighted the importance of lime fixation point and observed the effects of compaction delay on the compaction and strength characteristics of expansive Indian black cotton soil. It was observed that below the lime fixation point, lime addition steeply decreased the MDD and marginally improved strength,

while lime addition beyond lime fixation point caused no further change in compaction characteristics except considerably improving the strength of lime treated soil. Below the lime fixation point, delay between wet mixing and compaction was found to be less critical to compaction and strength characteristics, while this was considerably altered for soil with lime content well above the lime fixation point.

Boardman et al. (2001) studied the time dependent effects of mineral structural chemistry on the lime-clay reaction using kaolinite and montmorillonite clays. They observed that the solidification mechanism due to lime addition for the two clay minerals is different. They noted that pozzolanic reactions involving kaolinite and montmorillonite occur owing to the dissolution of aluminium and silicon from the respective clay minerals. They attributed the short-term strength gain to flocculation and long-term strength gain to pozzolanic reaction. They noted no significant pozzolanic activity until 7 days curing time. Further, mineral dissolution takes place initially at the edges of the crystalline structure, leading to a difference in the reaction products between kaolinite and montmorillonite.

2.4.2 Modification of Fly Ash

Subbarao et al. (1995) investigated the effects of different curing environments on lime stabilised pond and fly ash samples and observed that high ambient temperature in tropical zones produces higher strengths under field curing conditions in summer than under laboratory curing condition.

Boominathan and Raju (1996) evaluated the behaviour of untreated and treated fly ash to be used as a structural fill material with an admixture of lime mixed at optimum lime content. They observed that lime treated fly ash could be effectively used as embankment material over soft clay as factor of safety increases due to considerable reduction in

settlement of foundation soil.

Prasad and Bai (1999) investigated the lime reactivity of some Indian coal ashes and reported that coal ashes with high silica content also contain higher lime content. High percentage of free lime in coal ash also play an important role in increasing its compressive strength and thereby its lime reactivity.

Nataatmadja and Morgan (1999) reported the engineering properties of two fine-grained class F fly ashes from Queensland, Australia and attempted to stabilize them with lime and cement. The strength of the fly ashes increased with the cement content but the ductility significantly reduced. They observed that cement and lime can be blended in proportion with fly ash to optimise the strength and ductility of the mix.

The effects of addition of sodium hydroxide and lime on the UCS of an Indian fly ash was investigated by Ramesh et al. (1999) and they reported that addition of sodium hydroxide alone was not beneficial for strength gain and lime could improve the strength only to a certain extent. Both lime and sodium hydroxide, when used together however, greatly enhanced the strength of the fly ash.

Ghosh and Subbarao (2001) examined the physiochemical and micro-structural developments of a fly ash modified with lime and gypsum. The modified material showed new peaks of low intensity, some of which were not prominent and indicated that the new phase formed might be in small quantity or in an amorphous state. They also established that a long curing period is necessary to achieve more compact material structure that comprise of interlocking network of reaction products, which increases the strength and durability of the material.

Kamon and Katsumi (2000) reported basic characteristics of coal fly ash derived from fluidised bed combustion system and evaluated their applicability as geo-material for

such as building of embankments, road base or as soil stabilizer. They reported remarkable strength gain by compaction and aging with or without admixture (CAS-carbonated aluminate salts) and the reason was attributed to lime and gypsum contents of the ash.

Hilmi and Lav (2000) analysed the micro-structural, chemical and mineralogical developments of a class F fly ash stabilised separately with cement and lime and observed that both the stabilisers produced similar hydration products. The amount, density, and the growth rate of the hydration products in lime-stabilised fly ash were less than that of cement-stabilised fly ash. The micro-structural improvement was found to initiate with fly ash particles serving as nucleation centres for pozzolanic reaction products. The voids between particles were filled by growing hydrates over time, which contributed to the strength gain.

Narendra et al. (2003) observed higher free swell volume, liquid limit and strength of fly ash-lime mixes when treated with water containing sodium chloride. The increased reactivity was attributed to formation of compound sodium calcium silicate hydrate (N-C-S-H), which is more voluminous and has a higher water holding capacity compared to that of calcium silicate hydrate (C-S-H) formed with lime alone.

2.5 STUDIES ON SOIL STABILISATION USING FLY ASH AND LIME

Several important studies on specific use of industrial waste for stabilizing soils have been reported by Ghosh et al. (1973), Joshi and Wright (1978), Gidley and Sack (1984), Lakshman Rao (1987), Kamon et al. (1989), Babu Rao and Reddy (1989), Jha (1989), Jain et al. (1990), Basavanna and Itagi (1990) and Patrudu and Rama Rao (1990).

Mateos and Davidson (1962) attempted to find an optimum amount of lime and fly ash admixture for stabilising various textured soils (dune sand, friable loess, alluvial and

gum botil) for unconfined compressive strength. Based on the study, they recommended 5% and 7% of lime for sandy soils and clayey soils respectively. The amount of fly ash recommended was between 10 to 25%.

Uppal and Dhawan (1968) reported that addition of fly ash-lime admixture to soil decreases its MDD and increases its OMC. They observed maximum compressive strength when lime and fly ash alone were mixed in the ratio of 1:1.5 to 1:2, and 25 to 30% of this mix was found to produce maximum compressive strength for soil stabilisation.

Ferguson (1993) examined the use of self-cementing fly ashes as a soil-stabilizing agent and discussed various aspects like factors influencing fly ash composition, stabilizing characteristics and applications of fly ash treatment.

Keshawarz and Dutta (1993) stabilised two types of expansive soils obtained from Texas, USA with fly ash and the results were compared with lime and cement stabilisation. They observed that fly ash, lime and cement stabilisation, were in general effective in ameliorating the texture and plasticity of the soils by reducing the amount of clay size particles and plasticity index. However, Portland cement imparted highest level of strength to the soil.

Nicholson and Kashyap (1993) carried out preliminary studies to understand the effects of a fly ash for stabilisation of a number of “poor to marginal” quality tropical soils of following origin: (a) highly expansive and high plasticity colluvial clay (b) residual volcanic soil (c) lateritic soil obtained from Hawaii. They observed that addition of 15% fly ash substantially reduced the liquid limit and plasticity index of the highly plastic clays and smaller additional decrease with higher fly ash content. For other silty soils, they observed gradual decrease in liquid limit and plastic limit. OMC increased and MDD gradually decreased with the addition of greater amounts of fly ash to the various soils. The swelling potential of all the soils tested with admixture decreased with

smectite clays indicating notable change. Addition of fly ash alone caused only a limited change in CBR value but it dramatically increased when a small percentage of lime was added to the soil fly ash mixtures. UCS, in general, increased with the addition of fly ash and subsequent curing. For some soils, particularly for the highly plastic ones, an initial loss of compressive strength was noticed with the addition of fly ash. Significant strength gain was noticed for soil mixes with increase in lime content and it was found to be better than the effect produced by lime alone.

The influence of active calcium, active silica and phospho-gypsum on the geotechnical properties of a high water content alluvial clay was investigated by Stepkowska (1994) and he concluded that the additives should be used both qualitatively and quantitatively as the active calcium is important factor determining the structure and behaviour of the stabilised material.

The strength gain of lime stabilised materials, such as soil-lime, lime-fly ash mixtures exhibit distinct difference with that of cement-stabilised materials (Jalali, 1994). The initial strength of the first group, at low curing temperature, shows an exponential dependence with time, approaching a linear relation as curing time increases. At high curing temperature, however, initially the strength-time curve is approximately linear for lime-stabilised materials.

Sinha et al. (1995) reported the results of geotechnical investigation of a 45m high earth–fly ash embankment of a fly ash pond in Uttar Pradesh in India. They noticed partial compaction of the mixed material (about 81% to 99% of MDD) due to movement of trucks, dumpers and tractors and also due to lapse of time. They also noticed variations in shear strength parameters in the field due to variation in density, grain size and moisture content as well as improper mixing of fly ash with soil in the field. Srivastava et al. (1995) investigated the geotechnical properties of fly ash and lime sludge stabilized alluvial soils and observed best stabilizing effect for the soil with 8% fly ash and 12% lime sludge.

Nettleton et al. (1996) presented a practical experience of constructing embankments of lightweight fill by mixing Pulverised Fuel Ash (PFA) and silt for sections of a new highway project in UK. The silt being slightly contaminated with 6% organic content, up to 100% water content and high sulphate content posed a serious problem for stabilisation. Extensive laboratory and field testing was done which proved its engineering capabilities and long-term chemical stability after treatment with quicklime and pulverised fuel ash.

Srivastava and Joshi (1997) examined the scope of utilising four different waste materials (lime sludge from two industries, red mud from alumina manufacturing industry and fly ash) by mixing them in different percentages with typical Indian alluvial soil and expansive soil. The liquid limit of the alluvial soil on interaction with lime sludge or fly ash or their combination was found to increase and in case of red mud negligible effects were noticed. An over all increase of about 82% was observed in UCS value, when 8% fly ash and 12% lime sludge were added to the alluvial soil. In case of the red mud, highest UCS value was observed when 16% of the material was added to alluvial soil. They also reported that 16% to 20% of waste, in general, could be added to the soil without significantly deteriorating its behaviour.

Sivapullaiah et al. (1996) showed that addition of fly ash significantly affects the index properties namely liquid and plastic limit and free swell characteristics of black cotton soil and results in considerable improvement of workability of the soil. They observed that pozzolanic reactivity of fly ash influences the property of soil by formation of gelatinous pozzolanic reaction compounds and the effect is dependent on the lime content. Liquid limit increases with flocculation and formation of hydration products but decreases with the reduction in thickness of diffused double layer. The plastic limit of BC soil increases due to the addition of fly ash but the effect of addition of lime to the mixes is considerably greater. The effects of grain size distribution, lime content and pozzolanic

reactivity on free swell volume are found to be similar to that of the effects noticed for liquid limit but considerably greater effect was observed for pozzolanic reaction compounds.

The effects of fly ash and lime on CBR, and unconfined compressive strength of alluvial soil, was studied by Singh et al. (1997) and they reported significant improvement in both the values (five times) when treated with the additives. The effects of addition of fly ash on soft marine soils were investigated by Chandrakaran and Nambiar (1997).

Sawa et al. (1998) investigated the effects of five different industrial wastes (two FBC fly ash, pulverised fly ash, ordinary fly ash, and paper sullage incinerated ash) for treatment of a low strength high water content muddy soil with a hardening agent [cement stabiliser with main composition CaO (57%), SiO_2 (21.6%), Al_2O_3 (7.7%), SO_3 (8%)]. Based on the results, they concluded that there exists an optimum mixing rate of hardening additives and in order to sustain the hardening activity of coal fly ash, quality control is very important.

Shrivastava et al. (1998) studied the scope of utilisation of different mixtures of fly ash and sludge from sugar factory for stabilisation of black cotton soil from India. They observed that liquid limit, plasticity index, MDD and OMC of the mixes reduced though marginally in case of MDD and OMC. The unconfined compressive strength, cohesion and CBR values improved while angle of internal friction decreased partially. They further observed that within the quantities of fly ash (up to 20%), and sludge (8 to 26%) used, optimum requirement for different proportions are different. Mixes containing 8% fly ash and 12% sludge showed optimum results for stabilisation.

Srivastava et al. (1997) assessed the engineering behaviour of an expansive soil stabilised with fly ash (amount used: 0 to 20%) and lime sludge (a waste from fertiliser

industry, amount added varying from 0 to 20%). In general, they observed that addition of 16% fly ash and 16% lime sludge to the soil was found to produce best stabilizing effect.

Thome et al. (1998) studied the effects of industrial by-products like fly ash, bottom ash and carbide lime, profusely available in Southern Brazil, for improving the properties of a sandstone residual soil and verified the influence of temperature in the development of pozzolanic reactions. Compaction tests were performed for characterisation and mix design purpose. The mechanical behaviour was evaluated through UC tests, and micro-fabric was observed through SEM. All tests were carried out on mixtures containing 65% of soil, 25% fly ash or bottom ash and 10% carbide lime. The influence of temperature was analysed for 6ºC, 28ºC and 44ºC. Results of UCS demonstrated that bottom ash is more effective than the fly ash for stabilisation of sandstone residual soil. The strength of the mixes increased in an exponential way with temperature.

Kaniraj and Havanagi (1999) studied the geotechnical characteristics of different soil-fly ash mix by using two types fly ashes (class F) collected from India and Germany. Based on results they concluded that degree of compaction of the soil-fly ash mixtures was not sensitive to water content. As fly ash amount in the mix was increased, MDD decreased, whereas OMC increased. The variation of MDD with fly ash content was more or less linear whereas that of OMC with fly ash content was slightly non linear. The actual values of MDD and OMC could be predicted from the specific gravity of the mixes. UCS of the soil-fly ash mixes increased with the increase in fly ash content. In case of UU triaxial tests, no consistent trends were observed for angle of internal friction and cohesion due to the variation of fly ash content. Consolidation of the compacted fly ash occurs very rapidly both in the consolidation test and drained triaxial test. Compression and re-compression indices for the sandy soil-fly ash mixes were found to be small.

Chandrakaran and Nambiar (1999) constructed a small-scale physical model to study the effectiveness of lime and fly ash, used in the form of columns to stabilise the soft

clay soil. They observed that insertion of lime fly ash columns to the clay bed resulted in a decrease in plasticity index of the clay due to reduction of liquid limit and slight increase in the plastic limit. The undrained strength of the soil was found to increase and its magnitude increases with the increase in contact period of the lime-fly ash column with clay. However, the effect was found to reduce with the increase in radial distance from the columns.

Srinivasaraghavan et al. (1999) studied the effects of lime and fly ash addition with regard to the strength gain and changes in permeability characteristics of sand. By comparing the strength variations for curing in air and curing under humidity, they observed that latter method of curing gave consistently higher strength than the earlier. The permeability character of the soil was appreciably influenced by the addition of lime fly ash combine.

Cokca (2001) evaluated the effectiveness of high calcium and low calcium fly ashes for stabilisation of an expansive soil and by comparing with the effect of lime and cement stabilization, concluded that expansive soils can be successfully stabilized by fly ashes. Lime and cement were added to the expansive soil at 0 – 8%. Fly ashes were added to the soil at 0 – 25%. Addition of these stabilizers and subsequent curing decreased the plasticity index, activity and swelling potential of the soil.

Consoli et al. (2001) evaluated the stress-strain-strength behaviour of sandstone residual soil improved through the addition of carbide lime and fly ash. They observed coupled effect of following two main contributing factors to the stress strain response:

a) Structure imparted by compaction, mainly density and packing, which predominates in the short term
b) Formation of a cementitions matrix, which predominates after pozzolanic reactions have been developed.

Further, factors such as curing temperature, compressive and tensile strength

mobilization rate, and compaction parameters affect the stress-strain-strength behaviour of the mixed material with time. Strength gain was observed after 90 days of curing, believed to be due to an induction period for pozzolanic reaction. Based on triaxial compression tests, they reported that after a short delay following compaction, the maximum stiffness occurred for specimens moulded on the dry side of the OMC and the maximum strength at the OMC occurs. After 28 days, pozzolanic reactions magnified brittleness and further increased triaxial peak strength and stiffness. Also, maximum strength and stiffness occurred on the dry side of the OMC at the end of 28 days of curing.

Tan and Iyisan (2001) investigated the change in behaviour of a mixture of clay and fly ash obtained from Turkey and reported significant improvement of mechanical properties of the mixed material. They concluded that more economical and environment friendly solutions and applications can be achieved by utilising fly ash in geotechnical engineering practice.

Kumar (2002) studied the effect of addition of fly ash, lime and Na_2CO_3 on the permeability of locally available fine sand and reported marginal reduction of permeability till 15% addition of fly ash alone. Kumar (2002) also reported that permeability can be further reduced only by the optimum addition of stabilizers. A combination of 70% soil, 28% fly ash, 1%lime and 1% Na_2CO_3 was found to give best results against the optimum use of fly ash in the soil.

de Brito Galvao et al. (2003) investigated the effects of kiln ash on engineering properties, particularly permeability and compressibility of a residual lateritic soil (66.5% clay, 11.3% silt and 21.3% sand; classified as ML) obtained from Brazil to explore the possibility of using the material as a road construction material. Kiln ash containing 61% CaO (available 26.43%) were mixed to the soil in natural state at an application rates of 0, 2, 4, 6 and 8% and liquid limit, plastic limit, proctor compaction, unconfined compression

and consolidation tests were performed. Test result showed substantial change in Atterberg limits with the addition of kiln ash. Liquid limit decreased 59% to 49% and plasticity index decreased from 27% to 15%. But, change of index properties with time was found to be marginal. MDD and OMC of the mixes for kiln ash content of 0 – 8% did not show substantial change. UCS varied from 339.57 to 422.8 kPa, an increase of about 25%. The coefficient of permeability also ranged from 1.3×10^{-7} to 7.3×10^{-8} cm/sec. One dimensional consolidation tests on untreated soil and soil treated with 2.5, 5, 10 and 25% kiln ash showed improvements in compressibility of the soil-kiln ash mixes that increased with curing time. With 20% kiln ash and samples cured for 177 days, strain reduced by about 19%.

2.6 LEACHING MECHANISM AND CONTROLLING FACTORS

Fundamentals of leaching and its controlling mechanisms have been elaborately dealt in texts by Conner (1990), Allen et al. (1993), McBride (1994), Stumm and Morgan (1996).

"Leaching" is the permeation of contaminated pore water out of a porous matrix of waste material. The contaminated water generated is called the "leachate" and the capacity of the waste material to leach is called "leachability". The factors that affect leaching are:

a) Solubility of Metals
b) Adsorption of Metals
c) Chemistry of the Pore Water
d) Chemistry of the Solid Phase

Solubility of metals in water depends on hydrolysis, the presence of other organic and inorganic ligands, their coordination chemistry, and the pH of the solution. Solubility of different metal species is highly dependent on pH of the solution. Since the pH of pore water in soil-fly ash mixtures is expected to be very high, the release of heavy metals may

be solubility-controlled (Murarka et al.1991).

Adsorption of metals is found to depend on the properties of the solid (particle size, nature of inorganic oxide coating, organic carbon content, and zero point charge of the solid) as well as the properties of the liquid, (pH, total dissolved metal concentrations determined by the sum of free metal pool, inorganic ion pairs, as well as the concentration of the complexing ligands). Since pH has a major influence on solubility of most chemical species, its effect on adsorption is generally dominant, (Sauve et al., 2000).

The chemistry of the pore water is highly dependent on its pH, presence of dissolved ligands, total metal concentration, and background of electrolyte. pH is the most important parameter controlling the metal release during leaching (Brunori et al., 1999). It also strongly influences the adsorption of metal ions on the solid surface (Brunori et al., 1999, Sauve et al., 2000). This is caused primarily by a change in the surface potential of the adsorbent, and also by a change in the competition between protons and metal ions for adsorption sites. A change in surface potential affects metal adsorption through a change in electrostatic attraction or repulsion (Allen et al. 1993).

The factors that affect the pH of fly ash leachate was studied by Theis and Wirth (1982) and they observed that the relative amounts of lime and amorphous iron in the fly ash define the acidic or basic character of fly ash leachate. The iron content is a measure of the acid component, whereas soluble calcium, which is associated with the lime fraction, represents the basic component of fly ash.

Dissolved ligands are the anions or molecules with which a metal forms a coordination compound. Its concentration in pore water significantly affects leaching since it regulates solubility and adsorption of metals in pore water. The presence of soluble metal complexes often reduces metal adsorption (Reed and Nonavinakere, 1992), particularly when the metal complexes have less affinity for the sorption sites than the free metal ion.

The total metal concentration influences the amount of metal adsorbed on solid surfaces and the metal concentration in aqueous phase. An increase in total metal concentration causes a decrease in partition coefficient and leads to higher metal concentrations in the aqueous phase (Di Torro et al., 1986).

Background electrolyte composition and ionic strength can affect metal adsorption by altering the solution chemistry of the metal and the electric double layer associated with the particle surfaces. In systems with more than one adsorbable ion, competition between a metal ion of interest and other species for the same adsorption sites is an important factor that influences metal adsorption (Allen et al., 1993). A concentrated background electrolyte can "swamp" the surface of the solid, limiting metal access to the adsorption sites (Reed and Nonavinakere, 1992).

Leaching characteristics are also significantly influenced by the chemistry of the solid phase due to the variation in distribution of metals on the solid phase and the surface charge of the solid surface. In case of fly ash, leaching is affected by the total amount of metals in the fly ash and the relative amounts of metals present in the glassy and crystalline phases. Metals deposited on surface of the fly ash during combustion are more active chemically than the metals locked within the glassy phase that are released only through long-term weathering processes (Theis et al., 1982). The glassy spheres are relatively inert and immune to dissolution due to their elemental composition and glass-like structure.

The point of zero charge (pzc) is the pH at which the net surface charge of a solid is zero. Increasing the pH above the pzc results in a more negative surface charge, and increased adsorption of cations on the solid surface (Ricou et al., 1999). The pzc of fly ash typically varies between 2 and 4 (Ricou et al., 1999).

2.7 LEACHING STUDIES OF FLY ASH AND LIME STABILISED SOIL

A major concern associated with the use of fly ash for geotechnical application is that fly ash may contain different toxic contaminants. These contaminants in the leachate

emanating from fly ash or soil-fly ash may contaminate groundwater. Several investigators have studied leaching characteristics of metals from fly ash and soil-fly ash mixtures.

Theis et al. (1982) conducted a series of leaching tests with eleven different fly ash samples (six alkaline and five acidic) to investigate the sorptive behaviour of trace metals on fly ash in an aqueous system under several constant pH conditions (3, 6, 9, and 12). The fly ashes were also subjected to two extraction procedures involving dissolution in hydrofluoric acid and aqua regia (digestion) as well as with ammonium oxalate to determine the total metals in the ash and the fraction associated with the surface of the fly ash. Based on the results, they reported that the fraction of surface metals released to leachate is not higher than 20% of the surface associated metals at neutral pH. Also, the leaching of trace metals from fly ash in leachate follows a predictable pattern of decreasing release with increasing pH.

Das et al. (1989) evaluated leaching patterns of a number of acidic and alkaline fly ashes with the help of column leaching tests. Leachate obtained from permeation was analysed for Al, Na, K, Br, Ca, Cr, Cs, Mo, Sb, Se and Si, and based on the data obtained from the column leaching tests, following mass balance equation was proposed:

$$v (c_0 - c) + G F(t) = V \, dc/dt \qquad (2.1)$$

where, v is the average flow velocity,

c_0 = influent concentration (=0 for distilled water),

c = concentration after many pore volumes (long-term equilibrium),

G = mass of fly ash in the column,

V = volume of column expressed in pore volumes, and

F(t) is specific mass transfer function defined by the following:

(a) instantaneous dissolution upon wetting (F(t) = 0);

(b) exponentially decreasing release ($F(t) = ke^{\alpha n}$);

(c) slow dissolution (F(t) = k); and

(d) precipitation and adsorption governed dissolution

$$(F(t) = k_1 e^{\alpha n} - k_2 [c(t) - c]$$

where, n = number of free volumes led through, and k, k_1, k_2 and α are metal release rate constants. They reported that instantaneous dissolution occurs for Na, Cr, Se, Mo, and W.

Edil et al. (1992) reported an investigation of the effects of long-term permeation of inorganic leachate on sand-fly ash mixtures for evaluation of its feasibility as liner material. Coarse sand was mixed with class C fly ash in equal proportions by weight and compacted at two different dry densities (low and high) to get two different hydraulic conductivities. Column tests were conducted in flexible-wall permeameters with a constant head and using a synthetic leachate as the feed solution. The synthetic leachate consisted of Ca, SO_4 (as S), Cd, Zn, and B with Na, and Cl added to achieve comparable ionic strength to a leachate collected from a fly ash storage facility. The leachate was analysed for Cd, Sr, Se, As, Cr, Ca, Na, SO_4 (as S), Zn, Al, and B. Edil et al. (1992) proposed that the release pattern from fly ash follows two basic shapes. These release patterns are described as the first-flush response and the lagged flush response. It was also noticed that most metals show the first-flush leaching pattern. The exceptions were strontium and chromium for the test specimen having low hydraulic conductivity.

The effect of flow rate on leaching characteristics of compacted soil-fly ash mixtures was investigated by Creek and Shackelford (1992). Column leaching tests were conducted on soil-fly ash mixes prepared with class F fly ash, sand and bentonite (50% fly ash + 45% sand + 5% bentonite; 75% fly ash + 15% sand + 10% bentonite; 75% fly ash + 22.5% sand + 2.5% bentonite). Tests were conducted at four different hydraulic gradients (25, 50, 100, and 150) using tap water as the permeant fluid. The leachates were analyzed for the 13 trace metals (Al, B, Ba, Ca, Cd, Cr, Cu, Fe, Mn, Mo, Pb, Sr, and Zn). It was

noticed that an increase in fly ash content tends to increase the leaching of most of the metals and the flow rate had no noticeable effect on leaching of metals. Similar to Edil et al. (1992), Creek and Shackelford (1992) also noted early and delayed leaching patterns. Except barium, calcium, and strontium, which showed delayed leaching pattern, most of the leached metals showed the early release pattern.

Goh and Tay (1993) studied the effect of lime on leaching characteristics of MSWI fly ash. Concentrations of trace metals (Cd, Cr, Cu, Ni, and Zn) from the column tests on un-stabilised fly ash were found to decrease with time. The concentrations are even lower in the effluent from the mixture stabilised with lime (having higher pH).

Young and Phadungchewit (1993), from a study on pH influence on selectivity and retention of heavy metals in four different kinds of soils, reported that in evaluating the attenuation potential of the soil sub-grade (i.e. contaminant attenuation), emphasis need to be given to the selectivity of the soil –heavy metal contaminant system to the pH of the leachate and the competitive effect between heavy metals present in the leachate. Also, a change in the soil solution pH results in a corresponding change of the dominant retention mechanism of heavy metals in the soil. At high pH, precipitation mechanism (precipating as hydroxides and (or) as carbonates) dominates. As pH decreases precipitation become less important, and cation exchange becomes dominant. High amounts of heavy metals are retained in the soil if the soil buffer capacity remains high enough to resist a change in pH. Also, the selectivity order of heavy metals retention in soils depends on the pH of the soil solution.

Young and Phadungchewit (1993a) evaluated the role of various clay soil solids (clay minerals, organics, amorphous materials and carbonates) in heavy metal retention capability at different acidic conditions of the leachate by using selective sequential extraction method and the chemical speciation model MINTEQ (metal speciation

equilibrium model for surface and groundwater). The results showed that heavy metal could be retained in the four clay soils studied by several soil phases or mechanisms such as exchangeable, carbonate, hydroxide and organic phases. The retention of heavy metal in any phase depends on soil solution pH, soil constituents, and type of heavy metal. At high soil pH, retention by precipitation mechanism prevails, whereas at low pH, it is dominated by cation exchange. The capacity of the soils to retain high amounts of heavy metals as they receive increasing amounts of acid (i.e. as pH is reduced) depends directly on the initial pH values of the soil and on their buffer capacities.

Biliski and Alva (1995) evaluated the environmental impact of mixing fly ash with fine sand for agricultural uses. Leaching tests were conducted in 30 cm soil columns where only top 10 cm was amended with fly ash. Fly ash was mixed uniformly within the top 10 cm of soil in the column at application rates of 0, 10, 20, 30, 40, and 50%. Leachates were analysed for Cu, Mn, Fe, Zn, Cr, and Se and it was reported that the total mass leached increases significantly with increasing fly ash content. The highest concentrations of most elements were observed in the leachate corresponding to 1.5 pore volumes, except for Fe and Cu. The Fe and Cu concentrations reached a maximum within one pore volume. Beyond three pore volumes of flow, the concentration decreased considerably for all elements.

Chichester and Landsberger (1996) investigated the leaching dynamics of metals from municipal solid waste incinerator (MSWI) fly ash. A rigid-wall permeameter was used, the bottom plate of which was divided into two parts coaxially (inner and outer ring) to monitor for sidewall flow. The column was operated with deionized water (pH = 5.7) at a nearly constant water head having a hydraulic gradient of approximately 4. Chichester and Landsberger (1996) reported that the concentrations of Sr and B decrease substantially at higher pore volumes of flow.

Parsa et al. (1996) investigated the stabilisation/solidification of hazardous wastes from nitric acid processing of nuclear material using fly ash. The process involved mixing waste and fly ash and compacting the mixture for less than 3 seconds at 1.4 – 6.9 Mpa to form a monolith. A simulated mixed-waste, containing chromium, cadmium and lead, was used. Toxicity characteristics leachate procedure (TCLP) results showed that the chromium concentration in the leachate was well below current EPA limits. It was also observed that for the particular waste, TCLP results were not significantly affected by the waste fly ash ratio in the block due to the limited range of waste concentration.

Gustin and Thomes (1997) evaluated the leaching characteristics of fly ash stabilized soil by Synthetic Precipitation Leach Procedure (SPLP) tests following EPA Method 1312. A class C fly ash was mixed with 6 different soils types [poorly graded sand (SP), silty sand (SM), clayey sand (SC), silt (ML), low plasticity clay (CL), and high plasticity clay (CH)] at an application rate of 20%. Leachate was analysed for Ba, B, Cr, Th, Zn, S. Test results indicated that the concentration of most of the contaminants is higher for fly ash than the soils. The concentration of barium from all of the soil-fly ash mixtures was much lower than that from fly ash alone. The concentration of thallium and zinc from the soil-fly ash mixtures was also lower than from the fly ash alone, but the concentrations of boron, chromium, and sulphate were higher for most of the soil-fly ash mixtures than the soil or fly ash alone. Gustin and Thomes (1997) attributed the reason for lower barium concentration in soil-fly ash mixtures to dilution and adsorption on clay particles.

Ghose and Subbarao (1998) performed hydraulic conductivity tests on a low calcium class F fly ash stabilised with various percentages of lime (4, 6, and 10%) and gypsum (0.5, and 1%) to investigate the hydraulic conductivity and leaching characteristics. Stabilised fly ash mixes were compacted at standard proctor density in

rigid wall permeameter and allowed to cure for 7 and 28 days before tests. Tap water (pH = 7) was used as influent and a constant hydraulic gradient of approximately 10 was applied. The leachate was analysed for Cd, Cr, Cu, Fe, Mg, Ni, Pb, Zn, As, and Hg. Results showed that treatment of lime and gypsum reduced the hydraulic conductivity by approximately 500 times with respect to that of the unstabilised fly ash. Hydraulic conductivity of the stabilised material reduced with an increase in moulding water content. Amount of reduction was less on wet side of the optimum water content. Curing resulted in reduction of hydraulic conductivity of the stabilised material. It was found that the concentration of trace metals generally decreases with increasing pH of the effluent. Moulding water content on wet side of optimum was more effective in reducing leachate of metals in terms of total quantity per day compared to the dry side of the optimum.

Kamon and Katsumi (1999) examined the environmental impact of stabilised soil containing heavy metals like chromium and lead. Batch and column leaching tests were conducted on alluvial soil (LL = 63.7%, PL = 28.7%) stabilised with 15% cement and mixed with deionised water containing $K_2Cr_2O_7$ or $PbNO_3$ until 80% water content and 100mg Cr or Pb per Kg dry soil was obtained. The contaminated soil was mixed with 15 % cement, placed in moulds and cured for 28 days before tests. Results showed that concentrations of Cr in the effluent from polluted soil are below the regulatory level, while the Pb concentration is higher than the regulatory level. Long-term heavy metal leaching was estimated based on dilution by rainfall, and field hydraulic gradient. Stabilised Pb-soil and Cr-soil were found to be suitable for geotechnical applications because of their low hydraulic conductivity that reduced the mass flux to the ground water.

Woolard et al. (2000) investigated the lead adsorption by fly ash modified by hydrothermal treatment with NaOH solutions. During modification, the zeolites, Na-P1 and hydroxysodalites were synthesised. Adsorption experiments at pH = 5 revealed that all

modified ash samples adsorbed significantly more Pb than raw ash. The best adsorption was obtained for ash modified with 3M NaOH. The same 3M NaOH modified ash also proved the most effective sorbent when adsorption was determined as a function of pH.

Kanungo and Mohapatra (2000) investigated the leaching characteristics of two class F fly ashes collected from India through batch tests at different pH. A liquid-to-solid ratio of 1:25 was adopted; 2 g of fly ash sample was mixed with deionised water and shaken five times daily for 3 to 4 min. up to 30 days in a series of 125-mL polyethylene bottles. The solid contents of the bottles were separated by centrifugation and filtration through 0.45 μm membranes. Similar experiments were carried out at constant pH of 3.0, 5.5, and 8.0. The pH was adjusted using 0.1 M HNO_3 and 0.1 M NaOH. The filtrate was analysed for trace elements. As in Theis et al. (1982), Kanungo and Mohapatra also found that for most of the trace elements, aqueous phase concentrations increase at lower pH.

Pandian and Balasubramonian (2000) conducted leaching studies on two fly ashes (one with high lime content and the other with low lime content) using Jar Method to simulate the natural conditions of surface ponding in the ash ponds. 10 gm of samples of each fly ash was taken into four 100ml jars filled with solution of known pH's 3, 5, 7, and 9 and having no toxic metals. These samples were allowed to remain without any subsequent disturbance and representative fractions of supernatant solutions were sampled periodically at 5 hours, 1 day, 3 days, 1 week, 1 month and 2 months. pH was measured and leachate so collected were analysed for Zn, Cr, Ag, Mn, Mg, and Na. It was observed that leaching of trace elements from fly ash mainly depended on the pH of the leaching solution and duration of reaction time. Silver and Magnesium concentration in the leachate decreased with the increase in pH for fly ash due to formation of hydroxide and carbonate precipitates. Chromium could not be leached out at higher pH than the initial pH 5.0 from both the fly ashes. Lithium leached out from both the fly ashes.

Singh et al. (2001) proposed a refinement over the simple batch leaching to study the leaching behaviour of fly ash in ash ponds. The proposed sequential batch leaching tests were carried out on fly ash specimens till a cumulative liquid to solid ratio of 100 was achieved. The tests were conducted in association with column leaching tests using acidulated water (pH = 4) and the leachate was analysed for Ba, Co, Cr, Cu, Mo, Ni, V, and Zn. Due to limitation associated with the analysing instrument, results were interpreted against the background liquid to solid ratio and pH.

Canty and Everett (2001) evaluated the use of alkaline coal combustion by-products (CCB) produced in Oklahoma to treat acid mine drainage (AMD) through alkaline injection treatment (AIT) and also provided an insight into the suitability of CCBs for alternative uses: the neutralisation of acidic industrial waste and soil liming application. From the results, they observed that fluidised bed ash (FBA) or secondly flue gas desulfurisation ash (FGDA) performed better as neutralising material. Fly ash (FA) was found to be less suitable for this particular application. FA and FGDA was found to contain larger concentration of toxic metals and in general are more susceptible to cation exchange. FA in particular, has a significant amount of toxic metals and acidic condition prevailing in mines could promote these metals. From this study, Canty and Everett (2001) suggested that FBA and FGDA may be suitable for acidic waste neutralisation.

Heebink and Hassett (2001) carried out SPLP (EPA method 1312), water leach test (ASTM D 3987) and long term-leaching tests on several samples collected from sites prepared with soil stabilised with different fly ashes. From the study Heebink and Hassett (2001) concluded that use of fly ash for stabilisation of soil is not hazardous if the addition rate of fly ash is limited to less than 12% to 14%.

Pandian et al (2001) studied the possibility of using fly ash as pre-filter material to retain contaminant cations and reported that the capacity of fly ash to retain the

contaminant ions depend on type of fly ash, ion concentration, pH of the ionic solution and duration of reaction time. The capacity of fly ash to retain ions increases with the increase in initial concentration up to a maximum and remains constant thereafter. It was also observed that the duration of reaction time required for the fly ash to retain metal ions to its maximum capacity is 72 hours.

2.8 CONCLUSIONS

1. Literature revealed that residual lateritic soils are formed in hot and humid climates similar to that of tropical regions and its properties can differ significantly from those of the more familiar transported soils. Conventional test procedures adopted for analysing its geotechnical properties, particularly the index and classification test, may lead to misleading conclusions about the properties of some of these soils.
2. The properties of fly ash such as compaction, strength, permeability, effects of curing time, effects of delay in compaction, influence of lime and other additives, pozzolanic activity, leaching characterstics etc have been studied in detail by a large number of researchers.
3. Substantial literature is also available on stabilisation of various types of soils using fly ash and many other additives. However, reports on the specific application of fly ash in improving the properties residual lateritic soils, formed out of granites and gneisses have been rather scarce, mainly due to the limited nature of work done so far on the soil. This is possibly due to the locations of such soil development, the inherent difficulties as well as relative lack of available technology and equipment in the areas of tropical and similar climatic regions.
4. In India, limited attempts have been made to study the characteristics of the residual soils by adopting appropriate procedures relevant for analysis of residual soils. Large deposits of residual lateritic soils are available in north-eastern part of India and no

systematic study on the properties of the soil has been reported so far.

5. Leaching study on fly ash and fly ash stabilised materials has been the focus of many researchers due to the presence of a number of toxic contaminates in the fly ash. More importantly, it is due to the nature of some of the fly ash stabilised material, which has the capacity to retain such contaminants to leach out later under conducive conditions, that leaching studies have assumed significance.

CHAPTER
THREE

PHYSICAL AND CHEMICAL PROPERTIES OF THE FLY ASH AND THE SOIL

3.1 EXPERIMENTAL FLY ASH

Fly ash for the present study was procured from a thermal power plant located in Bongaigaon in western Assam of India. The plant has a total installed capacity of 240 MW manufactured in four units, each having capacity of 60 MW. It produces fly ash and bottom ash in the ratio of 78:22 with maximum ash production capacity of 50 tons/hr and 14 tons/hr respectively. Fly ash from the plant was collected in a dry form directly from the hoppers and transported in airtight double polythene bags.

3.1.1 Physical and Chemical Properties

The physical and chemical properties of the fly ash, characterized for the present study are shown in Table 3.1. Specific gravity, grain size distribution and plasticity properties were studied by methods recommended by the IS: 2720 (Part-3, 4 & 5). The specific gravity of the fly ash is 2.06, which is substantially low in comparison to that of soil and so, favours its application as a fill and embankment material. The specific gravity of Indian coal ashes varies from 1.46 to 2.66 (Pandian et al., 1998). The low specific gravity is because of the hollow nature of the fly ash particles, or the cenospheres. Moisture

content of the fly ash was measured immediately after its collection from the hoppers and it was found to be 0.5%.

Fly ash is non-plastic and difficulty was experienced in cutting a groove while determining its liquid limit using the Casagrande device. Liquid limit of fly ash was therefore, determined with the cone penetration method and was found to be 36.77%. Fig. 3.1 presents the grain size distribution of the fly ash. The experimental fly ash is predominantly fine-grained consisting of more than 80% materials finer than silt size fraction (75 μm). The fly ash can be classified as sandy silt (SM) as per the IS classification.

The specific surface area of the fly ash is 2019 cm^2/g. It was determined as per the method suggested by ASTM C–204 using Blaine's air permeability apparatus and Portland cement was taken as benchmark. The specific surfaces of the Indian coal ashes range from about 1000 cm^2/g to 6000 cm^2/g (Sridharan, 2002).

The chemical composition of the fly ash in terms of major oxides obtained by X-ray fluorescence technique (Phillips 1410, Holland) is presented in Table 3.2. The free lime content of the fly ash is very low. The pH of fly ash was measured in 1:20 fly ash-water ratio and the Cation Exchange Capacity (CEC) was determined at pH 7 as per IS: 2720 (Part 24) 1976 and it is found to be 3.1 meq/100g.

The mineralogical composition of the fly ash was obtained by X-ray diffraction studies (SEIFERT, Model-XRD 3003), using a graphite monochromator and Cu-Kα radiation (scanned from 2θ varying from 10˚ to 80˚). The minerals present were identified with the help of JCPDS search match data files. The X-ray diffraction pattern of the fly ash is depicted in Fig. 3.2, which shows that Quartz, Magnetite and Mullite is the dominant crystalline constituent with traces of Malilite, Sillimanite, Gehlanite and Hematite.

Table 3.1 Physical and index properties of the experimental fly ash

Properties	Value
Moisture content (%)	0.50
Specific surface area (cm^2/g)	2019
Specific gravity	2.06
Grain size distribution	
Gravel (%)	0
Sand (%)	17.2
Silt (%)	80.2
Clay (%)	2.60
Liquid limit (%)	36.7

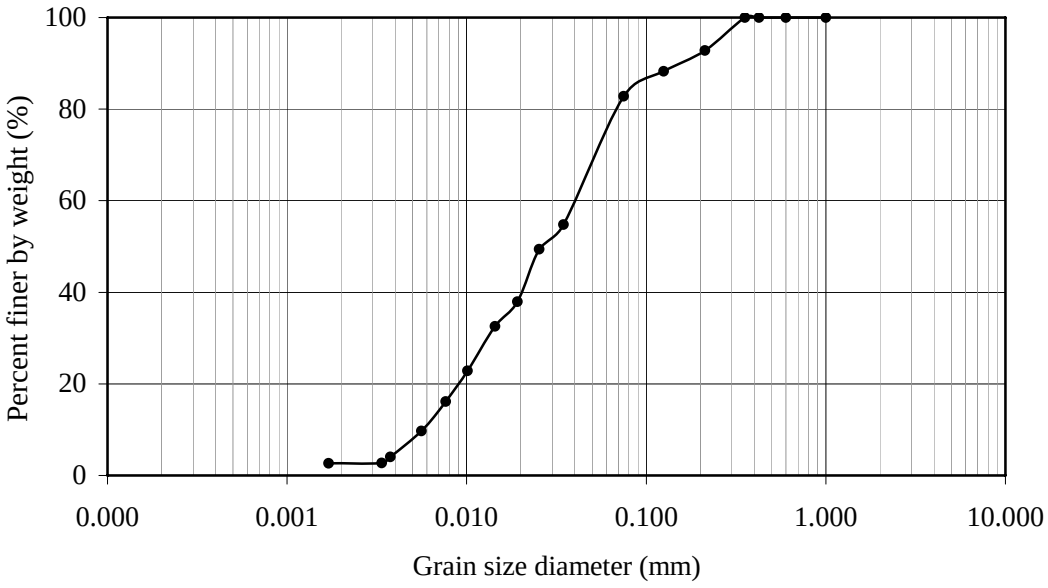

Figure 3.1 Grain size distribution curve of the experimental fly ash

Table 3.2 Chemical composition and major oxides present in the fly ash

Element	Value (%)	Element	Value (%)
Silica (SiO_2)	60.238	Manganese oxide (MnO)	0.038
Alumina (Al_2O_3)	22.895	Titanium oxide (TiO_2)	1.234
Iron oxide (Fe_2O_3)	6.350	Phosphorus oxide (P_2O_5)	0.566
Lime (CaO)	0.854	Barium oxide (BaO)	0.085
Magnesia (MgO)	1.239	Strontium oxide (SrO)	0.056
Soda (Na_2O)	0.037	Sulphur oxide (SO_3)	0.090
Potash (K_2O)	1.648		
SiO_2/ Al_2O_3	2.630		
CEC (meq/100g)	3.1		
Loss on ignition (%)	4.67		
pH (after collection)	6.28		

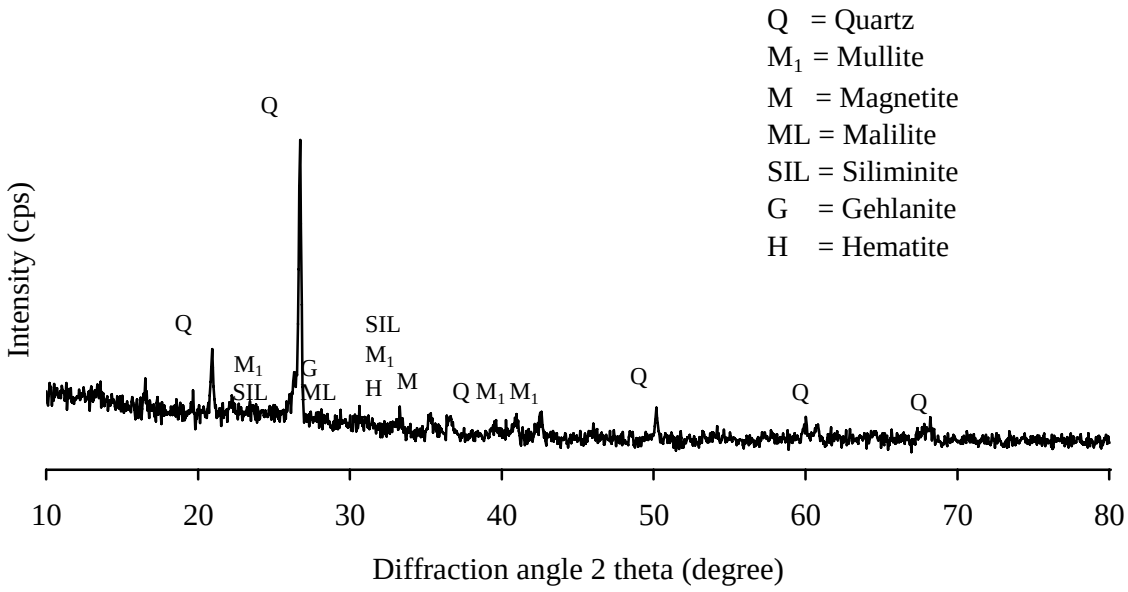

Figure 3.2 X-ray diffraction pattern of the experimental fly ash

3.2 EXPERIMENTAL RESIDUAL LATERITIC SOIL

The residual lateritic soil, used in the present investigation was sampled from a site near the present research laboratory on the northern outskirts of Guwahati city. The soil is found overlying Quartzo-feldspathic gneiss and porphyritic granite rocks.

Disturbed samples of the soil were collected from the completely weathered zone i.e. zone V and were first air-dried in the laboratory till the moisture content reduced approximately to 5%. Air-dried soil was then broken into smaller sizes using rubber mallet. Utmost care was taken to prevent breaking of soft stones or similar fraction in the soil. Disturbed samples of the soil in its natural state were also collected and stored in double packed polythene bags covering them with wet jute bags. Care was taken to ensure that all material came from the same location of the borrow area. Few block samples of size 20cm X 20cm X 20cm were also collected for unconfined compression and triaxial tests.

The physical, chemical and mineralogical properties of the soil were studied by the same procedures that were followed in case of the fly ash (section 3.1.1).

3.2.1 Physical Properties of the soil

Table 3.3 presents the physical properties of the soil. The soil has natural moisture content in the range of 22.1- 38.2%, obtained in three different seasons and its in situ dry unit weight is 1.28 gm/cm^3. Lateritic soils, in general, posses a high natural moisture content due to the presence of sesquioxides that tend to retain the natural moisture (Townsend, 1985). In case of highly weathered and leached soils residual from acid rocks such as granite that contain a large proportion of quartz, leaching and loss of material results in high void ratio and unstable collapsible grain structure (Barksdale and Blight, 1997).

Table 3.3 Physical properties of the experimental soil

Properties	Value
In situ dry unit weight (gm/cm^3)	1.28
In situ moisture content (%)	22.1-38.2
Specific gravity	
Natural	2.62
Air dried	2.63
Oven dried	2.66
Grain size distribution	
Gravel (%)	0
Sand (%)	9.8
Silt (%)	76.0
Clay (%)	14.2

The specific gravity of the soil was found to be 2.66. The specific gravity of Indian laterites and lateritic soils ranges from 2.4 to 2.96 with most of the values over 2.7 (Anirudhan, 1998). For determining specific gravity, IS: 2720 (Part III) recommends pre-drying of the soil in the oven before the experiment. Earlier investigators (Fourie, 1997; Pandian, (1998) have reported that specific gravity of residual soils is affected by the method pre-drying. Hence, additional tests on the soil at natural state and air-dried condition were conducted as per the method suggested by Fourie (1997). In both the cases, the dry mass of the soil was calculated by drying the soil specimen after the specific gravity test had been completed. Results (Table 3.3) indicate that the specific gravity of the experimental lateritic soil is not affected by air-drying of the soil. However, it shows a marginal increase with oven drying, which can be attributed to the aggregation of soil particles upon oven drying.

3.2.2 Chemical Composition

Table 3.4 presents the chemical properties of the experimental soil. The ratio of SiO_2 to Al_2O_3 for the soil is 2.65, which indicates the initial stage of laterization for the soil. The soil has a substantial proportion of aluminium and iron oxides (25.86%) indicating their extensive emplacement from the near surface ground water. The pH of the soil is 3.89 and it indicates the strong acidic environment of soil development. The soil has a low CEC (8 meq/100g) and refers to its limited potential for chemical stabilization.

Table 3.4 Chemical properties of the experimental soil

Element	Value
Major Oxides	
Silica (SiO_2)	47.070
Alumina (Al_2O_3)	17.768
Iron oxide (Fe_2O_3)	8.087
Lime (CaO)	0.070
Magnesia (MgO)	0.836
Soda (Na_2O)	0.102
Potash (K_2O)	1.655
Manganese oxide (MnO)	0.073
Titanium oxide (TiO_2)	1.030
Phosphorus oxide (P_2O_5)	0.055
Strontium oxide (SrO)	0.004
Cation Exchange Capacity (meq/100g)	8
pH (L/S= 20)	3.89

3.2.3 Mineralogical Composition

The X-ray diffraction pattern of the soil is shown in the Fig.3.3. Analysis of the diffraction pattern with the help of JCPDS search match data files, it was found that quartz is the dominant mineral in the soil with feldspar, mica and clay minerals (predominantly Kaolinite) present in smaller quantities.

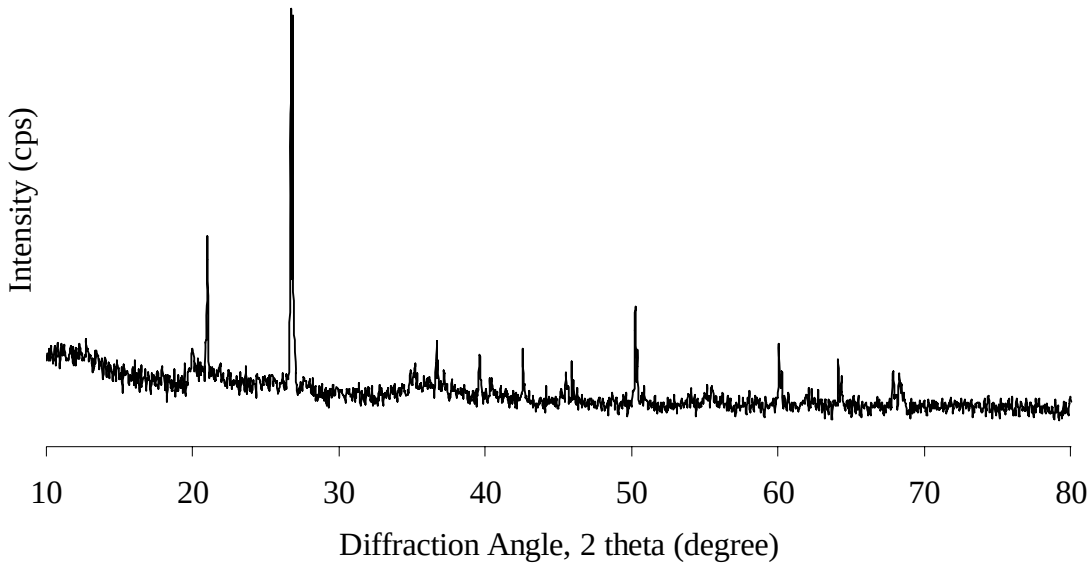

Figure 3.3 X-ray diffraction pattern of the experimental soil

3.2.4 Grain Size Distribution

Grain size analysis of the soil was done by wet sieving method, using un-dried soil in natural state. Dispersion agent used was a mixture of 33 gm of sodium hexametaphosphate and 7 gm of sodium carbonate, dissolved in 1 litre of water. A few additional tests were carried out to understand the effect of drying on grain size distribution of the soil. For this purpose, the coarse fraction (fraction retained by 75-micron sieve) was

pre-dried separately by air and oven and then analysed by dry sieving method. The tests were repeated and samples were also analysed by wet sieving methods.

The results of grain size distribution tests on the soil are shown in Table 3.5. The grain size distribution curves are presented in Fig. 3.4. The soil is predominantly fine-grained consisting of 14.2% clay that reflects the typical fine grained and well graded behaviour of highly weathered residual soil. The soil at its natural state can be classified as clayey silt (MH) according to IS Classification System (or Unified Soil Classification System).

It can be noted from Fig. 3.4 that for the same soil, the grain size distribution varies significantly due to variation in the method of testing. The soil, which was classified as silt on the basis of wet sieve analysis, changes to sand when analysed with dry sieving method indicating the extent of aggregation of fine particles upon drying.

Table 3.5 Results of grain size analysis on the soil for different sieving and pre-drying methods

Condition		Grain size			
		Gravel (%)	Sand (%)	Silt (%)	Clay (%)
Wet sieving:	Natural un-dried	0	9.8	76.0	14.2
	Air-dried	0	12.0	75.4	12.6
	Oven-dried	0	17.3	71.7	11.0
Dry sieving:	Air-dried	0	72.0	22.6	5.4
	Oven-dried	0	74.7	21.4	3.9

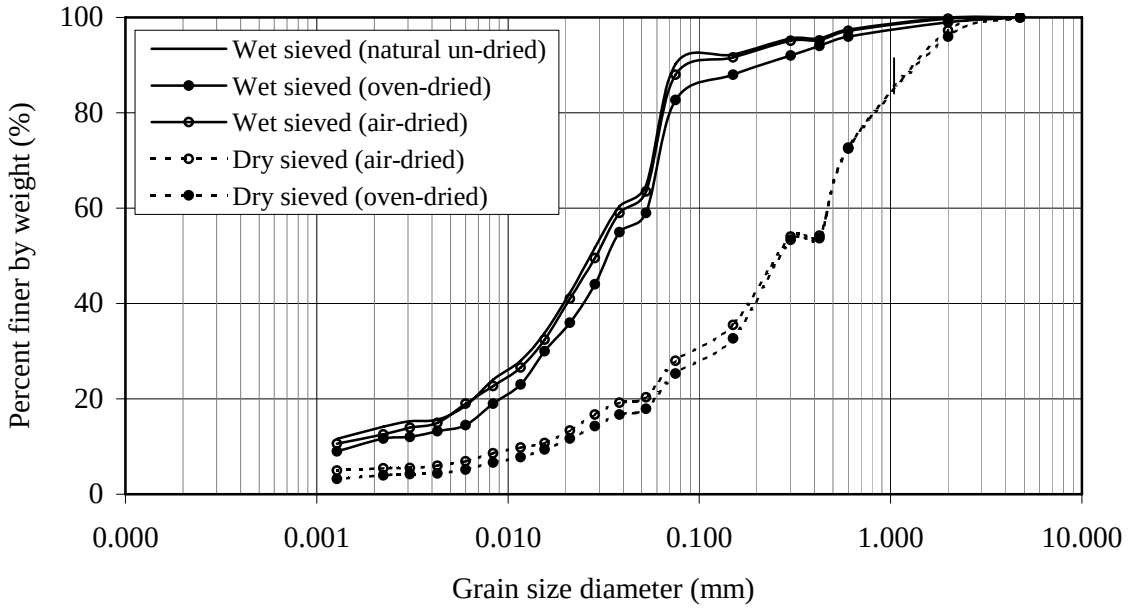

Figure 3.4 Grain size distribution curves of the soil obtained from different laboratory drying and testing conditions.

The oxidation process results in an increase of capillary stresses within the soil mass that causes the aggregation of fine particles to form larger particles. Earlier studies (Nambiar et al., 1992) have shown that the extent of aggregation depends on the environmental factors, particularly the cycles of wetting and drying. Pandian (1998), reported that aggregates formed upon drying are stable and the process is irreversible.

Although, the soil chosen for the present study shows aggregation of particles upon drying, most of the aggregate, so formed, are not strong and they easily de-floc under the action of dispersion agents and water. This is evident from the little change in gradation of the oven-dried soil by wet sieving method in comparison to the significant change in the gradation of the oven-dried soil by dry sieving method, when compared with the gradation of the natural soil as shown in the Fig. 3.4. It is also apparent from the figure that the effect of air-drying is less compared with oven drying as observed from the slight increase in the

sand size fraction and corresponding decrease in the silt and clay size particles with air-drying (Table 3.5).

3.3 EXPERIMENTAL LIME

Lime used in the present investigation was laboratory grade quick lime (CaO) in powdered state and it was obtained from the local market (manufacturer – Qualigens fine chemicals).

3.4 METHOD OF SOIL-FLY ASH-LIME MIXTURE PREPARATION

Soil collected for the present study was first air-dried inside the laboratory with the moisture content reduced to approximately 5%. Whenever sieving was required, air-dried soil was then broken into smaller sizes using rubber mallet with utmost care not to break of soft stones or similar fractions in the soil. The required fraction of the soil was then kept in double packed polythene bags for further investigation. The entire quantity of soil required for a particular series of investigation was prepared once at a time and the same was mixed thoroughly before packing in the polythene bags. For preparation of different mixes, fly ash was used in dry state and as stored after collection from the plant. Whenever lime was used, it was added only in the dry form.

In the preparation of all types of samples, first the required amounts of air-dried soil and fly ash were measured and mixed together in dry state. If no lime was added, the dry soil-fly ash mixture was mixed with required amount of water. Whenever lime was used, the dry soil-fly-ash mixture was first mixed with lime and then the soil-fly ash- lime mixture was further mixed with water. All mixing was done by hand and proper care was taken to prepare homogeneous mixtures at each stage of mixing.

3.5 MIX DESIGNATIONS FOR PRESENT INVESTIGATION

Table 3.6 shows the details of different soil-fly ash mixtures and their designation used for them in the present investigation. The designations used are: RLS for residual lateritic soil, BFA for Bongaigaon fly ash and FA for the soil-fly ash mixture. In the mix designation, the amount of soil replaced with fly ash is indicated by numeral attached before the symbols. Thus 35FA indicates that 35% by weight is fly ash and the remaining is soil. The percentage of lime added was indicated as percentage of the weight of soil-fly ash mixture.

Table 3.6 Designation of soil-fly ash mixes used in the present investigation

Designation of mix	Soil-fly ash mixture
RLS	Residual Lateritic Soil
20FA	80 % soil +20 % fly ash
35FA	65 % soil +35 % fly ash
50FA	50 % soil +50 % fly ash
BFA	Bongaigaon Fly Ash

3.6 CONDITIONING AND CURING

The experimental investigations outlined in various chapters include different period of conditioning and curing of samples before conducting various tests. Conditioning period is the time allowed to elapse between initial mixing of fly ash and lime into the soil and their final compaction/other tests. During this period, the mixed material was allowed to mellow and its effect on different geotechnical properties was studied.

Curing period, mentioned at various stages, on the other hand, is the time period allowed to elapse after final compaction of the mixed material until performing tests for determination of different strength parameters.

3.7 SUMMARY

The experimental fly ash has a large proportion of silt size particles (80.2%) and contains very small amount of lime (0.854%) indicating its very low hydration potential. From its chemical composition, the fly ash can be classified as class ‘F’ type as per ASTM, C-618. Low lime content also resulted in a near neutral pH of the fly ash. The natural moisture content present in the soil show relatively wet field condition and highlight need for drying before field compaction. Its low insitu dry density shows the porous micro-cluster structure typically observed in residual lateritic soils. The proportions of iron and aluminium oxides of present in the soil indicate its early stage of laterisation.

CHAPTER
FOUR

PLASTICITY CHARACTERISTICS OF THE SOIL-FLY ASH-LIME MIXES

4.1 INTRODUCTION

Lime has been used for centuries to improve the site workability and strength of soils. Only of late, it has started being used also in conjunction with fly ash due to the latter's excellent pozzolanic property. The inherent problems associated with site workability and difficulty in determining plasticity characteristics of residual lateritic soils have already been discussed elsewhere (section 2.2.2). The workability or ease of handling of a soil at site during construction operation is closely related with its plasticity characteristics. These characteristics for the present residual lateritic soil were studied particularly to understand:

a) the scope of lime stabilisation,
b) the effects of different proportions of fly ash alone and also when it is supplemented with different amounts of quick lime,
c) the effects of conditioning on short-term and long-term changes, and
d) the underlying mechanisms that control the change in plasticity and hence the workability.

A preliminary investigation was also carried out to understand the effects of sample pre-drying, moulding and gradation on plasticity properties of the soil in order to facilitate

selection of appropriate pre-test sample preparation technique and test procedure for the mixes. The test programme, experimental details and the results of this investigation, along with discussion on the mechanisms that is known to control the plasticity characteristic of different soils are presented.

4.2 TEST PROGRAM

Two series of tests were conducted. Table 4.1 shows summary of testing variables for Atterbarg Limit tests. In the first series, soil samples with different proportions of lime alone were tested after allowing them to condition for a period of 1 day. In the second series, samples with selected lime proportions of 2%, 4% and 8% were only considered and the conditioning period was increased to 28 days and 56 days.

Table 4.1 Summary of testing program for Atterberg limits on various soil–fly ash mixes

Designation of mix sample	Proportions of lime used (%)	Conditioning periods
RLS	0, 1, 2, 3, 4, 5 and 8	1 day
20FA	Same as above	
35FA	Same as above	
50FA	Same as above	
RLS	0, 2, 4, and 8	28 days and 56 days
20FA	Same as above	
35FA	Same as above	
50FA	Same as above	

4.3 EXPERIMENTAL DETAILS

4.3.1 Initial Investigation on the Soil

It has been widely reported that with most lateritic soils, standard tests for Atterberg limits do not yield reproducible results since the test results are influenced to a large extent by pre-test sample preparation conditions [Gidigasu (1976), Townsend (1985), Fourie (1997)]. As suggested by Fourie (1997), additional conditions related to drying temperature, duration of mixing and method of mixing have been adopted along with those specified in the prevailing standard practice as per ASTM D 4318-93 (1995 and 1995a).

Three drying conditions were adopted: undried (natural state), air-drying at 25˚-30˚C and oven-drying at 105˚-110˚C. The whole of the natural soil procured from the field was finer than 4.25 mm. The soil was broken down by soaking in distilled water to form a slurry, which was then washed through a 0.425 mm (No. 40) sieve until the water ran clear. The fraction passing the sieve was separated and kept in airtight plastic bags for 24 hours before testing. Another set of soil samples was obtained by wet sieving through sieve size of 0.075 mm (No. 200) and kept aside for 24 hours the same way.

Five equal parts of the fresh soil were separately mixed with different amounts of water by hand to give a range of moisture contents, and left for 24 hours for equilibration of moisture content before testing. The next day, after mixing further by hand for 5 minutes, plastic and liquid limit tests were conducted.

For studying the effect of duration and method of mixing, only air-dried soil finer than 0.425 mm was used, which constitute 95.6% of the total mass. After the conditioning period of 24 hours, two hand-mixing times of 5 and 25 minutes were adopted. A mixing time of 5 minutes only was adopted when a mechanical stirrer was used.

4.3.2 Sample Preparation and Test Procedure for Mixes

For each test, air-dried soil fraction finer than 0.425 mm was separated through dry sieving. Five separate samples were prepared by thoroughly mixing the required amounts of soil, fly ash and lime in dry state. Each of these samples were mixed with varying amounts of distilled water to give a range of moisture contents suitable for liquid limit determination. After thorough mixing, the specimens were packed in polythene bags, sealed and allowed to condition for stipulated periods as per the test program.

After conditioning, the samples were initially tested using the Casagrande's method for liquid limit determination as per ASTM D 4318-93 (1995a). As difficulty was experienced in cutting the required groove for certain samples, especially for 35FA and 50FA samples, the cone penetration method as per BS: 1377–1975, Test 2A (British Standard Institution, 1975) was adopted for all samples. The cone penetration values were maintained between 15 to 25 mm by adjusting the consistency of the samples. Whenever reported, the mean value of liquid limit was averaged from two separate flow curves.

Using parts of the above samples, the plastic limit was determined applying the standard method as per ASTM D 4318-93 (1995). The mean value of plastic limit was averaged from three tests.

4.4 MECHANISMS CONTROLLING PLASTICITY CHARACTERISTICS

Plasticity characteristics of soils are primarily controlled by the amount and type of clay minerals present. The relevance of clay minerals arises due to their fabric, small size, large surface area and effects of surface force interactions.

Clay minerals have a net negative charge and depending on the pore water environment, they adsorb cations in order to balance the charge deficiency. In a dry state,

adsorbed cations are tightly held by the negative force fields of the clay particles. In the presence of pore water, adsorbed cations tend to diffuse away from the clay surface to equalize its concentrations. However, the negative electrical field of the clay surface restricts this diffusion. The negative clay surface and the distributed charge adjacent to it, together, is called the diffused double layer. Distribution of ions adjacent to the charged surface or the thickness of the diffused double layer is sensitive to variations of surface potential, electrolyte concentration, types of exchangeable cations, temperature and nature of pore fluid.

Yong and Warkentin (1966) inferred from liquid limit data for kaolinite clays saturated with various cations that the inter-particle forces have an important role in determining the liquid limit. They also observed that the saturation with di-valent and tri-valent cations favours electrostatic attraction between positive edges and negative faces of the clay minerals leading to a flocculated particle arrangement. This results in a higher liquid limit because of the greater amount of water entrapped within the void spaces.

Mitchell (1976) has reported that the type of adsorbed cation has a much greater influence on the high plasticity minerals (e.g., montmorillonite) than on the low plasticity minerals (e.g., kaolinite). Further, increasing cation valence of adsorbed ions decreases the liquid limit value of expansive soils, but tends to increase the liquid limit values of non-expansive soils.

For montmorillonitic clay, Sridharan et al. (1986) observed that although the liquid limit was also governed by shearing resistance at particle level, the contribution due to diffused double layer overrides and primarily governs the liquid limit.

For kaolinitic soils, Sridharan et al. (1988) observed that the liquid limit is not a function of diffused double layer thickness and has no correlation with the percentage of clay fraction. Instead, the particle arrangement (clay fabric), which is regulated by inter-particle attraction and repulsive forces, play a dominant role in influencing the liquid limit. They also reported that the presence of divalent and trivalent cations and a low soil pH promotes positive edge-negative face flocculation, while the presence of monovalent sodium ions and a high soil pH creates a barrier to particle flocculation.

Lateritic soils contain a sizeable fraction of iron and aluminium oxides (termed as sesquioxides) besides clay minerals. In addition to the recognized function as a cementing agent, sesquioxides behave as a colloidal agent that also influences the plasticity characteristics. When iron hydroxide is suspended in an aqueous solution, the exposed hydroxyl (OH) surface layer of $Fe(OH)_3$ is capable of adsorbing and desorbing protons (H^+ ions) depending on the pH of the medium, typical of a variable surface charge mineral (Uehara and Gillman, 1981).

Adsorption of H^+ ions would impart a positive charge to the surface, while desorption would lead to a negative surface charge. It is also possible that at a certain pH value termed as point of zero charge (PZC), equal amounts of H^+ and OH- ions are adsorbed on to the hydroxylated surface so that the net surface charge is zero. If the actual pH of a system is less than PZC, then the surface has a predominant positive charge and vice versa (Fig.4.1). The PZC of iron and aluminium oxide minerals range between 7 and 9. Therefore, in a low pH environment, the negatively charged clay minerals will be heavily coated by the positively charged sesquioxides.

Addition of fly ash and lime to the soil not only changes the gradation of the resulting mix, but also brings about considerable change in the pore fluid condition thereby causing profound alteration of the physico-chemical properties.

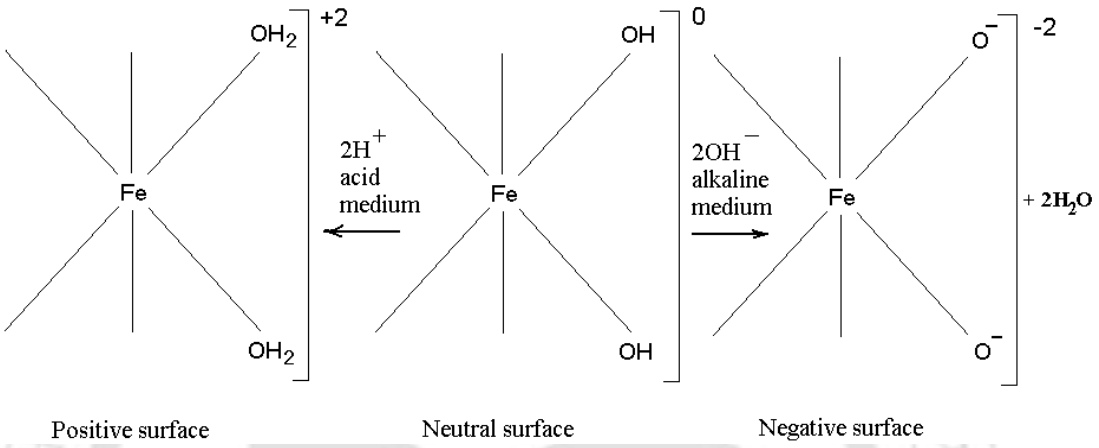

Figure 4.1 Charging of $Fe(OH)_2$ surface by protonation and deprotonation reactions (after Uehara and Gillman, 1981)

4.5 RESULTS AND DISCUSSION

4.5.1 Plasticity Behaviour of Experimental Soil

The summary of test results of preliminary investigation carried out on the soil under different initial conditions is presented in Table 4.2. The predominant clay mineral present in the soil is kaolinite. The experimental soil at its natural state (finer than 0.425 mm) has liquid limit of 58.2% and plastic limit of 39.5%. It can be noted that both liquid limit and plasticity index show a decreasing trend with the increase in drying temperature. The amount of reduction is observed to be greater for higher temperature, as noted for samples changed from an air-dried state to an oven-dried state.

In residual lateritic soils, sesquioxides present along with the clay minerals play a major role in controlling the consistency limits. Iron hydroxide gets adsorbed onto the clay mineral surface by hydrogen bonding between its exposed hydroxyl or oxygen atoms as well as by coulombic attraction between the positively charged $Fe(OH)_3$ and the negatively charged clay mineral surface (Greenland et al., 1968).

Table 4.2 Consistency properties of the experimental soil under different initial conditions

Condition	Finer than Sieve Size (mm)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Method of pre-drying				
Natural state	4.25	55.3	37.6	17.6
Natural state	0.425	58.2	39.5	18.7
Air-dried at 25˚C	0.425	54.9	37.1	17.8
Oven-dried at 105˚C	0.425	46.5	30.9	15.6
Air-dried at 25˚C	0.075	68.0	41.4	26.6
Oven-dried at 105˚C	0.075	51.8	31.3	20.5
Duration and type of mixing*				
5 minute hand mix	0.425	58.3	40.0	18.3
30 minute hand mix	0.425	64.8	42.3	22.5
Mechanical mixer	0.425	69.6	43.3	26.3

* Only air-dried soil was used

These coatings on the clay particles provide an enormous surface for adsorption of water. Dehydration of these coatings upon drying results in increased cementation of the clay particles and consequent decrease in specific surface area for water absorption, resulting in the consequent decrease in the consistency limits (Townsend, 1985).

Sridharan et al. (1991) have reported that colloidal $Fe(OH)_3$ particles exhibit very high liquid (390% - 580%) and plastic (240% - 275%) limits. On addition of the colloidal $Fe(OH)_3$ to kaolinite, $Fe(OH)_3$ imposes its plasticity on the clay thereby markedly enhancing the consistency limits of the mixes. Oven-drying leads to a marked reduction of the consistency limits caused by the loss of plasticity due to the transformation of the $Fe(OH)_3$ coatings to non-plastic hematite (α-Fe_2O_3) and magnetite/maghemite (Fe_3O_4/γ-Fe_2O_3).

The soil fraction finer than 0.075 mm size shows a greater reduction of the consistency limits upon oven-drying. From Table 4.2, it can be noted that the plasticity characteristics of the natural soil (finer than 4.25) mm are found to be similar to that of the air-dried soil finer than 0.425 mm. This can be attributed to the fact that 95% of the soil in natural state is finer than 0.425 mm.

It can also be observed from Table 4.2 that both the consistency limits increase with the increase in mixing duration. The plasticity index is also seen to increase. Also, the consistency limits become greater when the samples are mixed using a mechanical mixer. The change in plasticity behaviour upon remoulding is attributed to the breakage of friable aluminium and iron cementing bonds between clay clusters that result in the increase of finer fractions.

The plasticity properties of the soil with different drying conditions are plotted with respect to the A-line (Fig. 4.2) and they have been compared with the range given by Mitchell & Sitar (1982). The plots remaining below the A-line and the points moving

downwards are indicative of the extent of dehydration of sesquioxides and subsequent aggregation.

The initial investigation as per the first test series has shown that pre-test sample preparation conditions affect the plasticity behaviour of the soil. This necessitates a careful selection of these conditions for studying the effects of addition of fly ash and lime. At the same time, the preparation conditions should reflect actual field conditions where such mixtures are intended to be used.

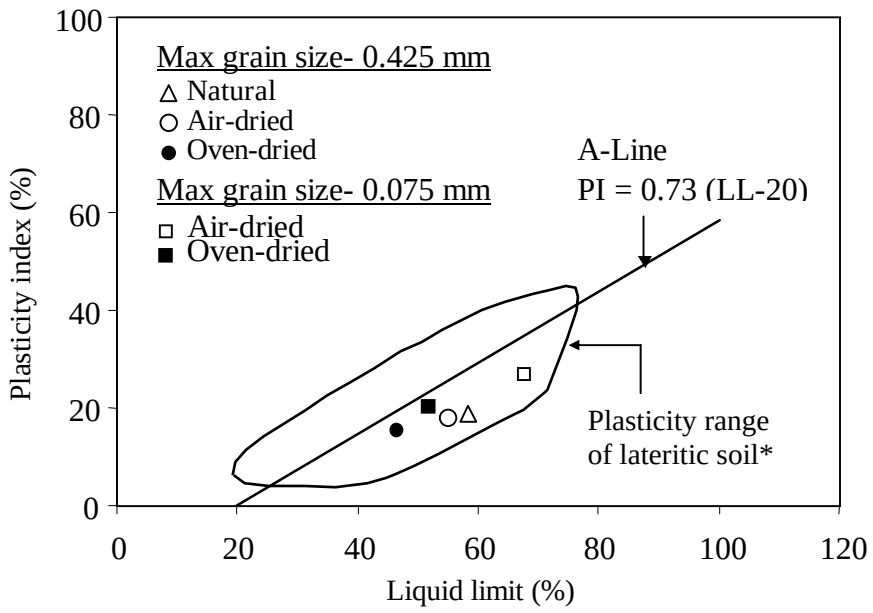

Figure 4.2 Variation of plasticity characteristics upon drying for the experimental soil (* Range as per Mitchell and Sitar, 1982).

During actual field applications, air-drying is anticipated. Therefore, it was decided to use air-dried soil finer than 0.425 mm for studying the aggregate plasticity behaviour of the soil upon addition of fly ash and/or lime. As the time of mixing was seen to affect the

gradation and the test results, it was decided to restrict the same to 5 minutes and do the mixing by hand only.

4.5.2 Plasticity Behaviour of Soil – Fly Ash – Lime Mixes

Figures 4.3(a)–(d) show typical relationships between cone penetration and moisture content for various mixes. Figure 4.3(a) examines the effect on index properties of the soil as a result of the addition of different proportions of fly ash alone, whereas Figures 4.3(b)-(c) illustrate the enhanced effects due to addition of different quantities of lime to the soil-fly ash mixtures. The measured values of liquid limit, plastic limit and plasticity index of the mixes are tabulated in Tables 4.3(a)–(d) for conditioning periods of 1 day, 28 days and 56 days.

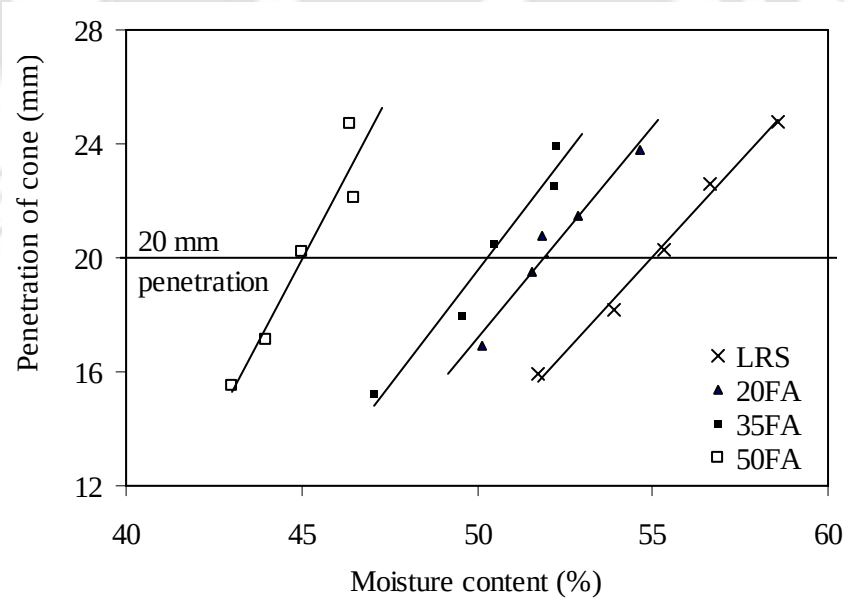

Figure 4.3(a) Cone penetration plotted against water contents showing the effects of fly ash on the soil

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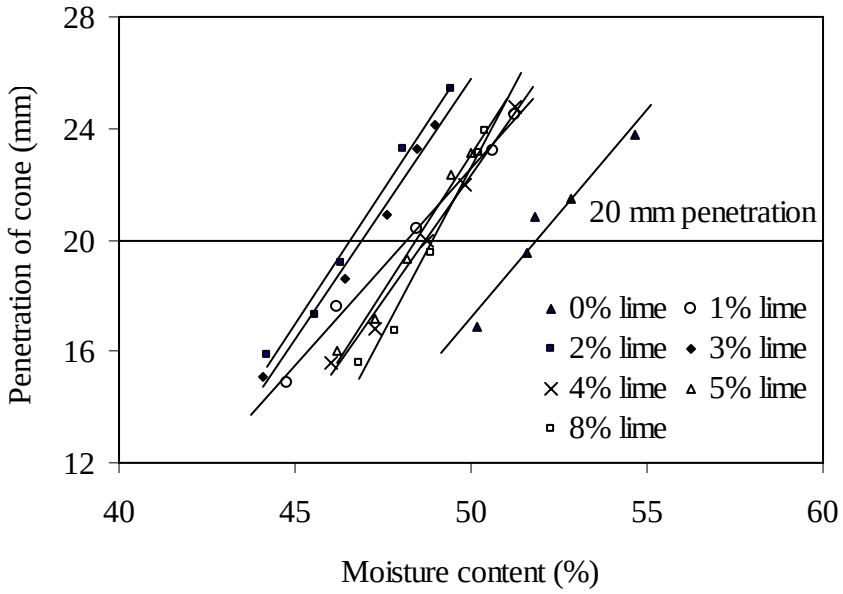


Figure 4.3(b) Typical plots of cone penetration against water content showing the effects of lime on 20FA mix

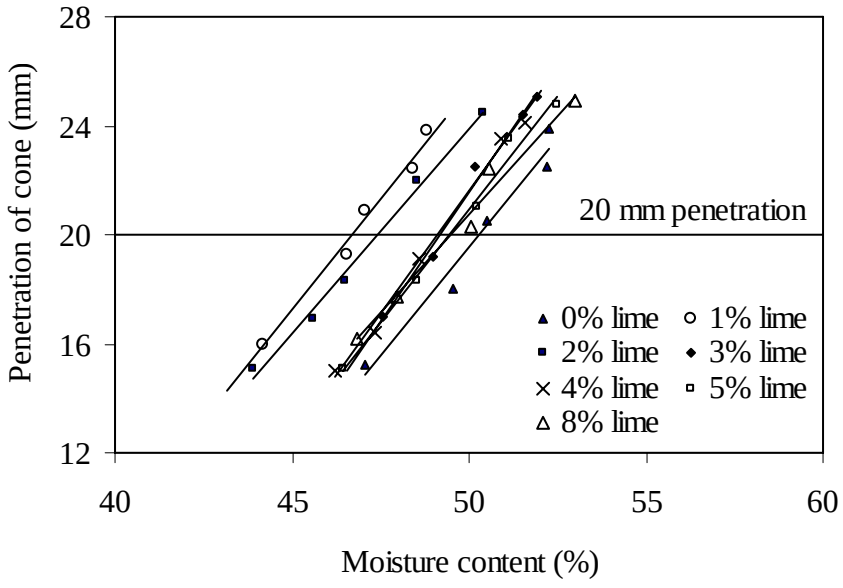


Figure 4.3(c) Typical plots of cone penetration against water content showing the effects of lime on 35FA mix

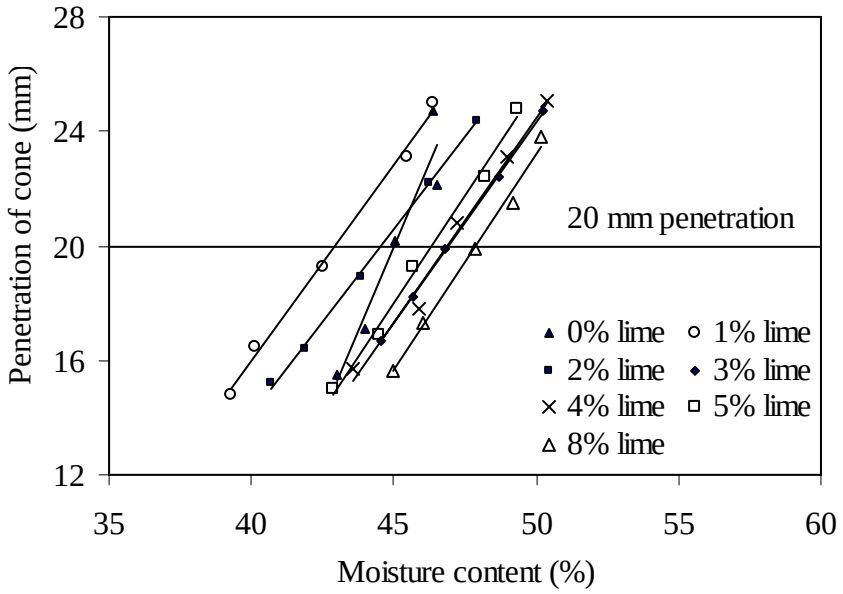


Figure 4.3(d) Typical plots of cone penetration against water content showing the effects of lime on 50FA mix

Table 4.3(a) Summary of plasticity changes in experimental soil caused by lime addition at different conditioning periods

Lime Addition (%)	Liquid Limit (%)			Plastic Limit (%)			Plasticity Index (%)		
	1day	28days	56days	1day	28days	56days	1day	28days	56days
0	54.9	54.7	54.8	37.5	37.3	37.6	17.4	17.3	17.2
1	52.1	---	---	39.4	---	---	12.1	---	---
2	50.1	50.3	50.5	40.2	42.3	42.6	9.9	8.0	7.9
3	50.1	---	---	40.7	---	---	9.5	---	---
4	50.3	51.0	51.2	41.3	43.5	44.8	9.0	7.5	7.3
5	50.3	---	---	41.3	---	---	8.9	---	---
8	50.2	51.1	51.5	41.4	43.9	45.2	8.8	7.2	6.0

Table 4.3(b) Summary of plasticity changes in the mix 20FA caused by lime addition at different conditioning periods

Lime Addition (%)	Liquid Limit (%)			Plastic Limit (%)			Plasticity Index (%)		
	1day	28days	56days	1day	28days	56days	1day	28days	56days
0	52.0	52.1	52.1	30.5	31.1	31.3	21.5	21.0	20.8
1	48.3	---	---	34.6	---	---	13.7	---	---
2	46.7	47.7	48.2	37.0	38.5	39.2	9.7	9.2	9.0
3	47.0	---	---	37.9	---	---	9.1	---	---
4	48.8	50.6	50.9	40.0	43.1	43.6	8.8	7.5	7.3
5	48.6	---	---	40.1	---	---	8.5	---	---
8	49.0	51.7	52.6	40.4	43.9	44.9	8.6	7.8	7.5

Table 4.3(c) Summary of plasticity changes in the mix 35FA caused by lime addition at different conditioning periods

Lime Addition (%)	Liquid Limit (%)			Plastic Limit (%)			Plasticity Index (%)		
	1day	28days	56days	1day	28days	56days	1day	28days	56days
0	50.3	50.5	50.6	26.0	27.2	27.7	24.3	23.3	22.9
1	46.8	---	---	32.1	---	---	14.6	---	---
2	47.5	48.7	49.1	34.9	37.0	37.8	12.6	11.7	11.7
3	49.3	---	---	38.7	---	---	10.6	---	---
4	49.3	51.5	51.7	39.6	43.4	44.0	9.7	8.1	7.7
5	49.5	---	---	41.1	---	---	8.4	---	---
8	49.6	51.9	52.1	41.2	45.0	45.9	8.4	6.9	6.2

Table 4.3(d) Summary of plasticity changes in the mix 50FA caused by lime addition at different conditioning periods

Lime Addition (%)	Liquid Limit (%)			Plastic Limit (%)			Plasticity Index (%)		
	1day	28days	56days	1day	28days	56days	1day	28days	56days
0	45.0	45.1	45.3	24.0	25.7	25.9	21.0	19.4	19.4
1	43.0	---	---	27.9	---	---	15.1	---	---
2	44.6	46.0	46.1	34.3	37.3	38.1	10.3	8.7	8.8
3	44.8	---	---	35.3	---	---	9.5	---	---
4	45.0	47.1	47.4	36.3	40.2	41.3	8.7	6.9	6.1
5	45.1	---	---	37.9	---	---	7.2	---	---
8	45.2	47.4	47.6	38.8	42.9	43.6	6.4	4.5	4.0

4.5.2.1 Variation of Liquid Limit

Addition of lime to a soil-water system results in a strong exothermic hydration reaction that increases the concentration of $Ca^{2+}_{(aq)}$ and $OH^{-}_{(aq)}$ ions in the pore water. The $OH^{-}_{(aq)}$ ions raise the pH of the pore fluid. The known effect of this on clay is: (a) decrease in liquid limit – due to reduction in the thickness of the diffused double layer of clay particles caused by the increase in the electrolyte concentration and also due to exchange of monovalent cations by divalent Ca^{2+} ions, or (b) increase in liquid limit – due to flocculation of clay particles.

The effect of lime content and conditioning period on the liquid limit of the soil is shown in Fig. 4.4. For a conditioning period of 1 day, the liquid limit of the soil without any additive is 54.9%. Addition of 1% lime reduces the liquid limit from 54.9% to 52.1%.

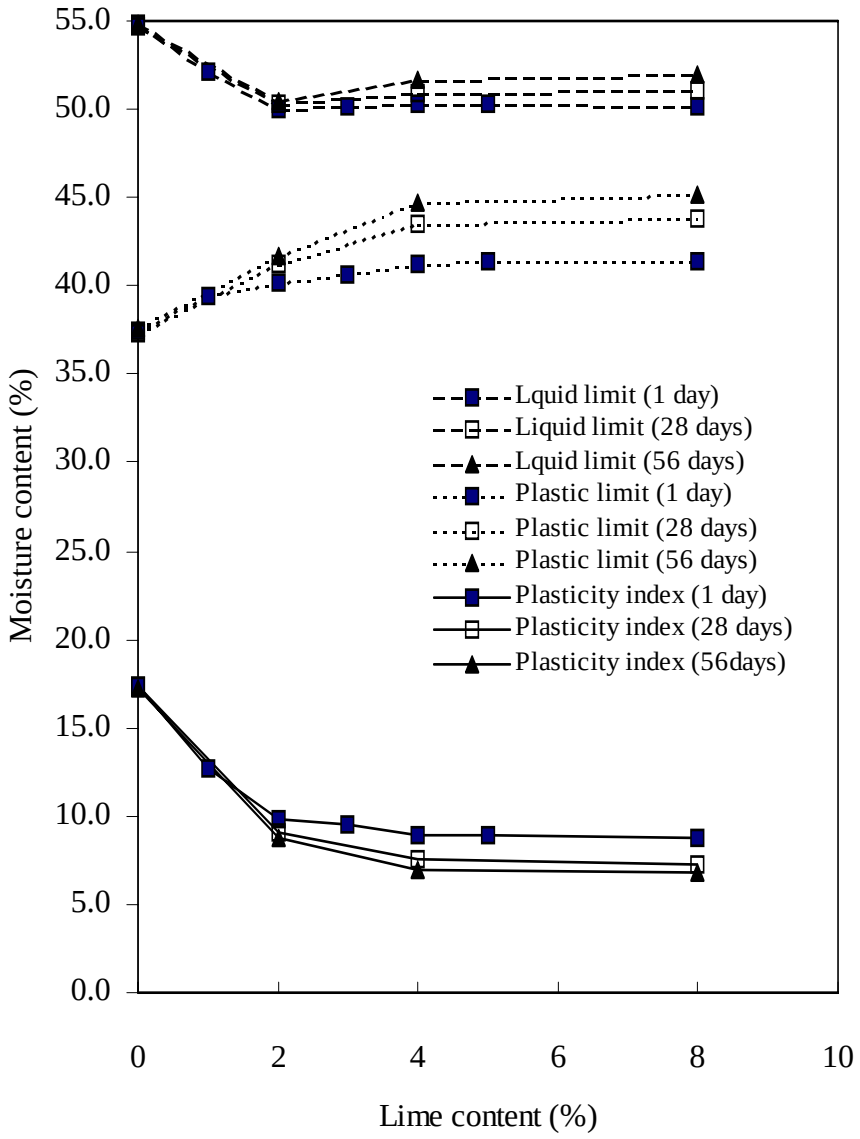


Figure 4.4 Variation of liquid limit, plastic limit and plasticity index of experimental soil with increasing lime content and conditioning period

Addition of 2% lime further reduces the liquid limit to 50.1%. However, addition of lime beyond 2% has a marginal influence on the liquid limit. The pH of all soil-lime mixes was observed to be greater than 11. The predominant clay mineral in this soil is kaolinite. As discussed earlier, the liquid limit of kaolinitic soils is governed more by particle arrangement (clay fabric), and although the presence of divalent Ca^{2+} should promote flocculation and thereby increase the liquid limit, the same was not observed for the mixes

due to the high pH level (pH >11) of pore fluid that creates a barrier to particle flocculation. Also, the high pH level of the pore fluid would have charged the associated $Fe(OH)_3$ predominantly with negative surface as discussed earlier, but its effect on the liquid limit of the mixes can be assumed to be minimum as the air-drying of the soil before testing may have caused the dehydration of colloidal $Fe(OH)_3$. It can also be noted from Fig. 4.4 that the increase of conditioning period marginally increases the liquid limit of the lime treated soil. The effect gradually increases with the increase in lime content. The high pH level of the pore fluid allows mineral dissolution of Al_2O_3 and SiO_2. The dissolved components from the clay minerals react with the Ca^{2+} ions present in the pore water, and lead to the formation of calcium silicate hydrate and calcium aluminate hydrate compounds (Sherwood, 1993). These cementitious compounds increase the water holding capacity resulting in increase in the liquid limit.

Figure 4.5 shows the effect of addition of fly ash and lime on the liquid limit of the soil after 1 day of conditioning period. It can be noted that addition of only fly ash decreases the liquid limit of the soil. The magnitude of decrease is greater with an increase in the proportion of fly ash. As the fly ash used has liquid limit of 36.8%, which is much lower than the liquid limit of the soil (54.9%), its addition has a diluting affect that tends to reduce the liquid limit of the mix.

The free lime content of the fly ash used is only 0.85% and its percentages are 0.17%, 0.30%, and 0.43% in 20FA, 35FA and 50FA mixes respectively. Even when no lime is added, the free lime should increase the liquid limit. However, as the amount of free lime in the mixes is very low, its influence is expected to be less. The observed continuous decrease in the liquid limit of the mixes confirms that the diluting effect of fly ash is dominant.

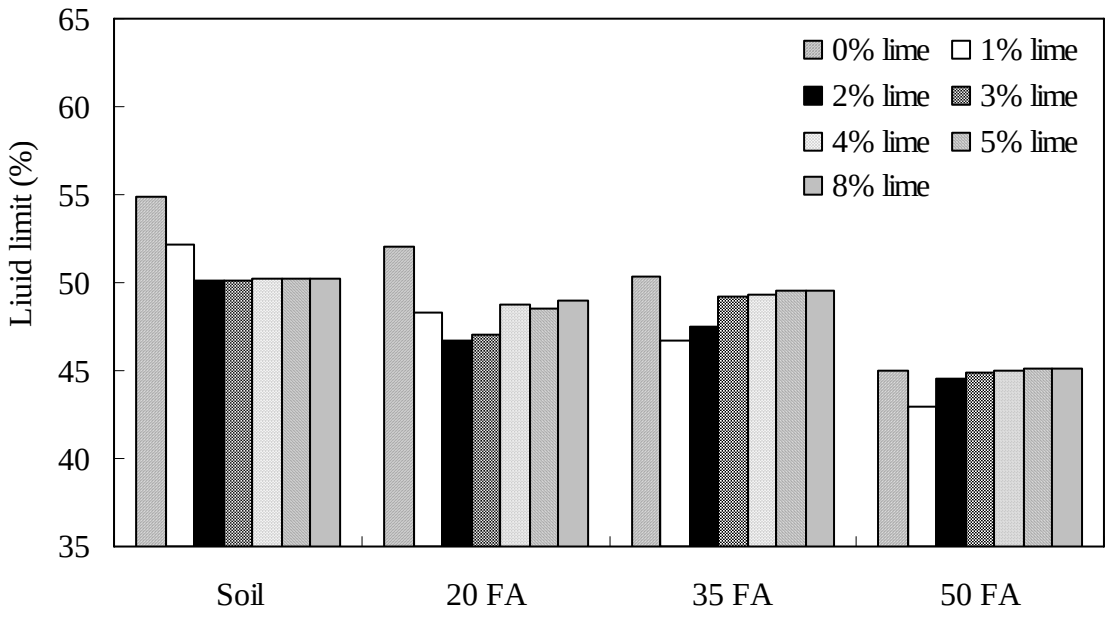


Figure 4.5 Variation of liquid limit of different soil-fly ash mixes with increasing lime content (for 1 day conditioning period)

It can be noted further from Figure 4.5 that addition of lime to the above soil-fly ash mixes significantly influences the liquid limit. With increasing lime content, the liquid limit initially decreases to a minimum value, after which it increases. The reduction in liquid limit can be attributed to cation exchange as discussed earlier. For higher proportions of fly ash, the minimum liquid limit value is achieved with a smaller quantity of lime. The quantities of lime are found to be 2% for 20FA and 1% both for 35FA and 50FA. This is possibly due to the fact that for higher percentages of fly ash, the amount of soil is less and hence the amount of exchangeable cations. Beyond these lime contents, the liquid limits of all the mixes increase, which can be attributed to flocculation. It can be noted that the increase becomes insignificant beyond lime contents of 4%, 3% and 2% for 20FA, 35FA and 50FA respectively. Figure 4.6 shows the effect of increasing conditioning time on the liquid limit of different soil-fly ash-lime mixes.

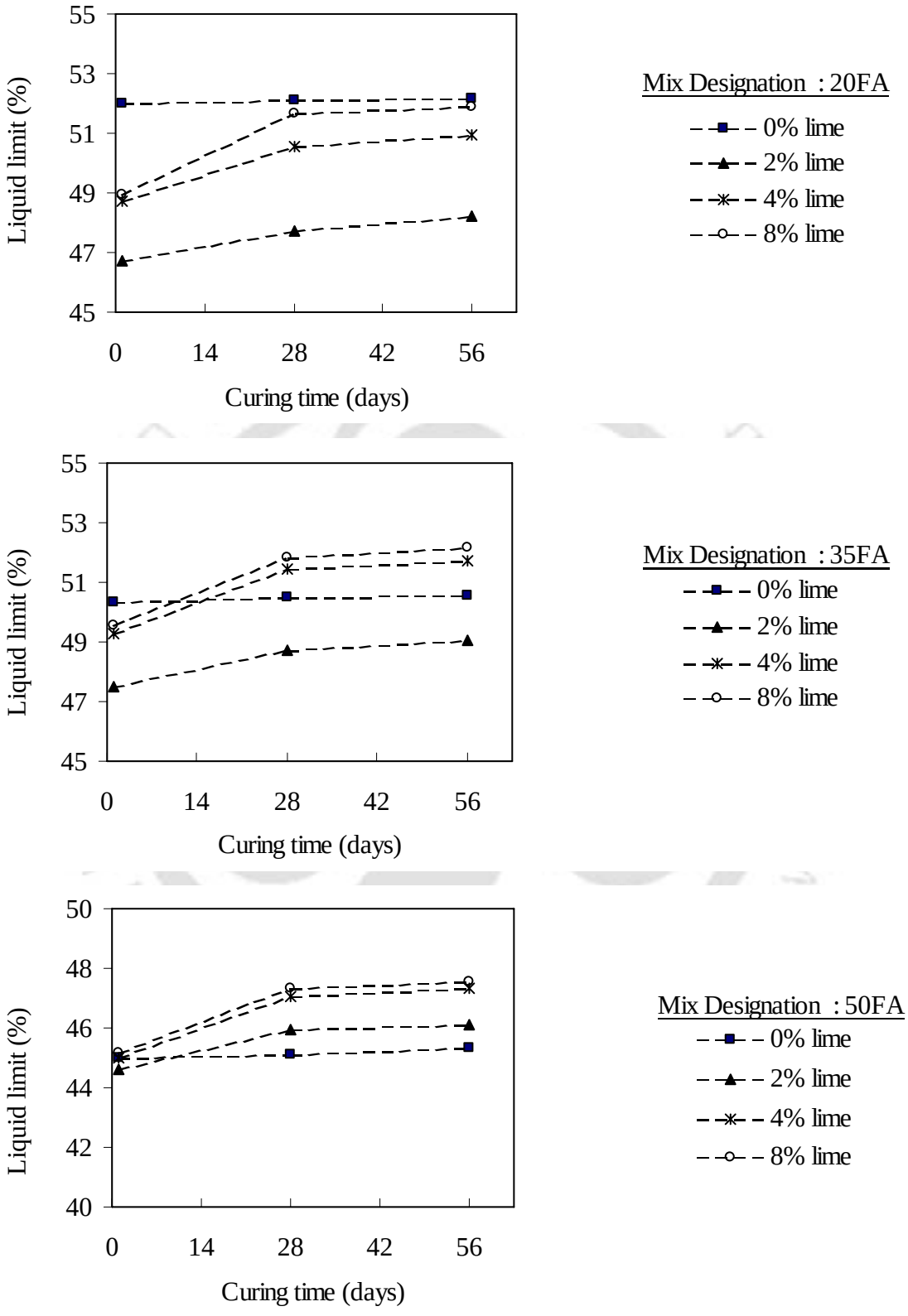


Figure 4.6 Variation of liquid limit of soil-fly ash mixes 20FA, 35FA and 50FA with increasing lime content and conditioning period

As conditioning time increases, the liquid limit of all the mixes increases, and the reason can be attributed to increased affinity of the mixes to water due to the formation of cementitious compounds as discussed earlier for soil-lime mixes. It can be observed from the plots that the change in liquid limit that occurs in the first 28 days of conditioning is greater than the change in the next 28 days, indicating stronger pozzolanic activity within the initial time period. For no lime addition, the mixes show practically no change in liquid limit values.

4.5.2.2 Variation of Plastic Limit

The plastic limit of soils is the reflection of resistance to shearing at the particle level. Plastic limit is mainly governed by the amount of coarser fraction, distribution of ions around the charged clay surface and the soil fabric (Sivapullaiah et al., 1996; Venkatappa Rao and Rekhi, 1977). Addition of fly ash and lime to soil changes its texture and gradation as well as pore fluid characteristics, resulting in considerable change in its plastic limit.

The effect of lime content and conditioning period on plastic limit of the experimental soil is shown in Fig. 4.4. For a conditioning period of 1 day, the plastic limit of the soil without any additive is 37.1%. Addition of 1% lime increases the plastic limit to 39.4%. Addition of 4% lime further increases the plastic limit to 41.3%. However, addition of lime beyond 4% has only a marginal influence on the plastic limit. As the conditioning periods are increased, similar trends are observed and the plastic limit is noted to increase further.

The short-term (one day) change in plastic limit can be attributed to reduction in thickness of the diffused double layer and flocculation of clay particles. The long-term

change is due to increase in shearing resistance at the particle level as a result of the crystallization and flocculation of hydration products.

Figure 4.7 depicts the effect of addition of fly ash and lime on the plastic limit of the soil after a 1 day conditioning period. Addition of only fly ash considerably reduces the plastic limit of the soil. The proportion of clay-size fraction in the fly ash is 9.27% (Fig. 4.8). Compared to this, the soil has a higher clay-size fraction by approximately 13%. In contrast, there is a marked difference in the grain-size distributions of the two materials for the sizes ranging between 0.012 mm and 0.10 mm, where the fly ash has a greater percentage of finer fraction. For example, for the grain size of 0.020 mm, the difference in size fraction is as high as 20%. When fly ash alone is added to the soil, the net effect is the replacement of coarser particles of the soil with finer particles of fly ash that results in the reduction of the plastic limit of the mixes.

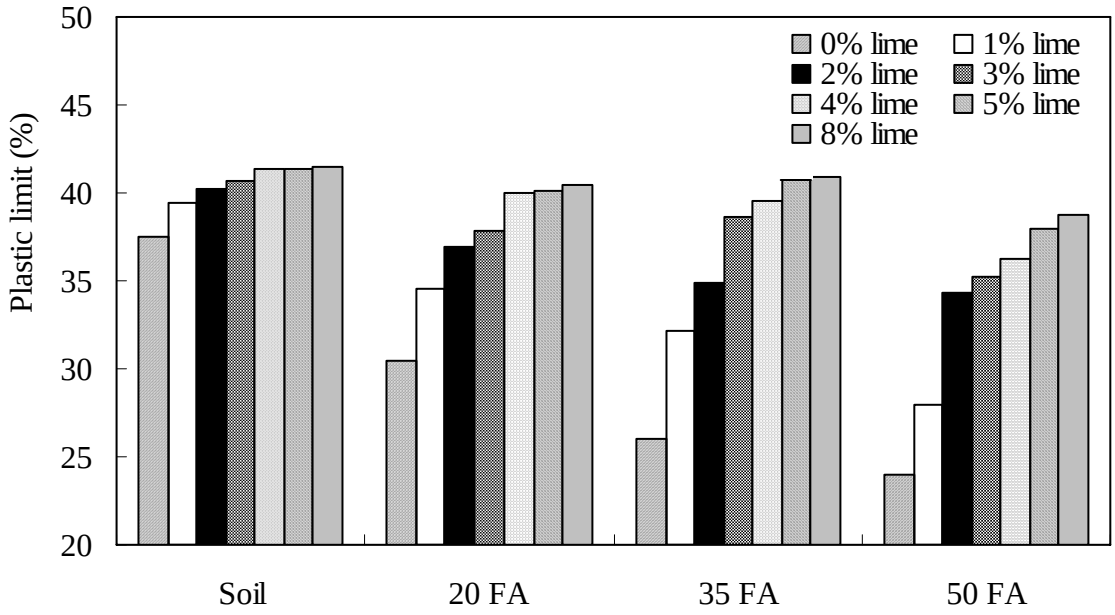


Figure 4.7 Variation of plastic limit of different soil-fly ash mixes with increasing lime content (for 1 day conditioning period)

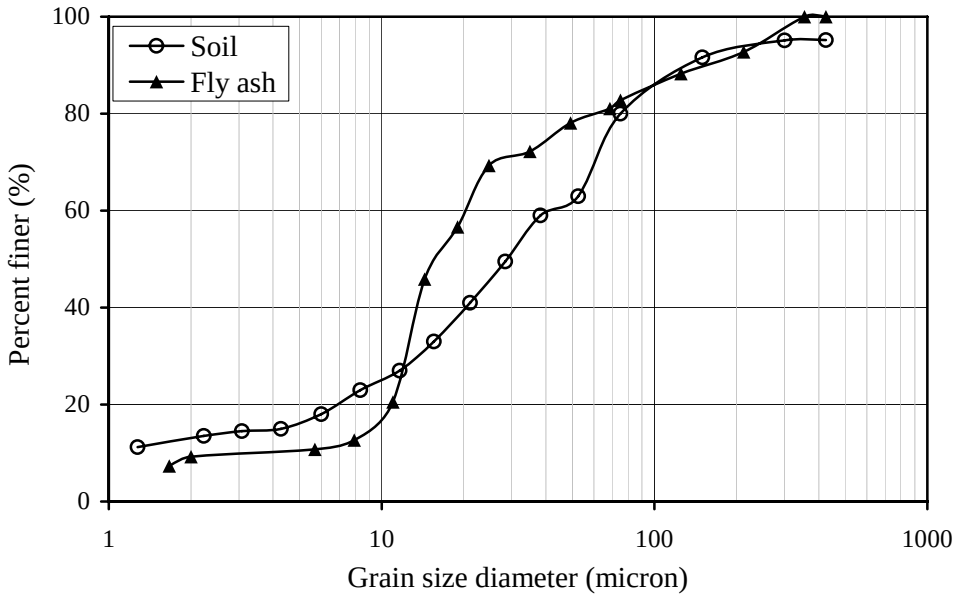


Figure 4.8 Comparison of the grain size distribution of the soil and fly ash

No significant effect due to the low free lime already present in the fly ash was observed. As noted from Figure 4.7, the continuous decrease of the plastic limit of the soil-fly ash mixes with no lime addition indicates the dominant role of the finer size fraction of fly ash in the reduction of plastic limit. These mixes show very little change in the plastic limit upon conditioning as indicated by Fig. 4.9.

It can be observed further from Fig. 4.7 that a considerable increase in plastic limit of all the mixes occurred after addition of lime, which can be attributed to the decrease in thickness of diffused double layer and/or flocculation of clay particles. It is interesting to note that the plastic limit continues to increase in a rapid manner up to a certain percent lime addition, depending on the proportion of fly ash in the mix. Beyond this amount, very little further change is noted.

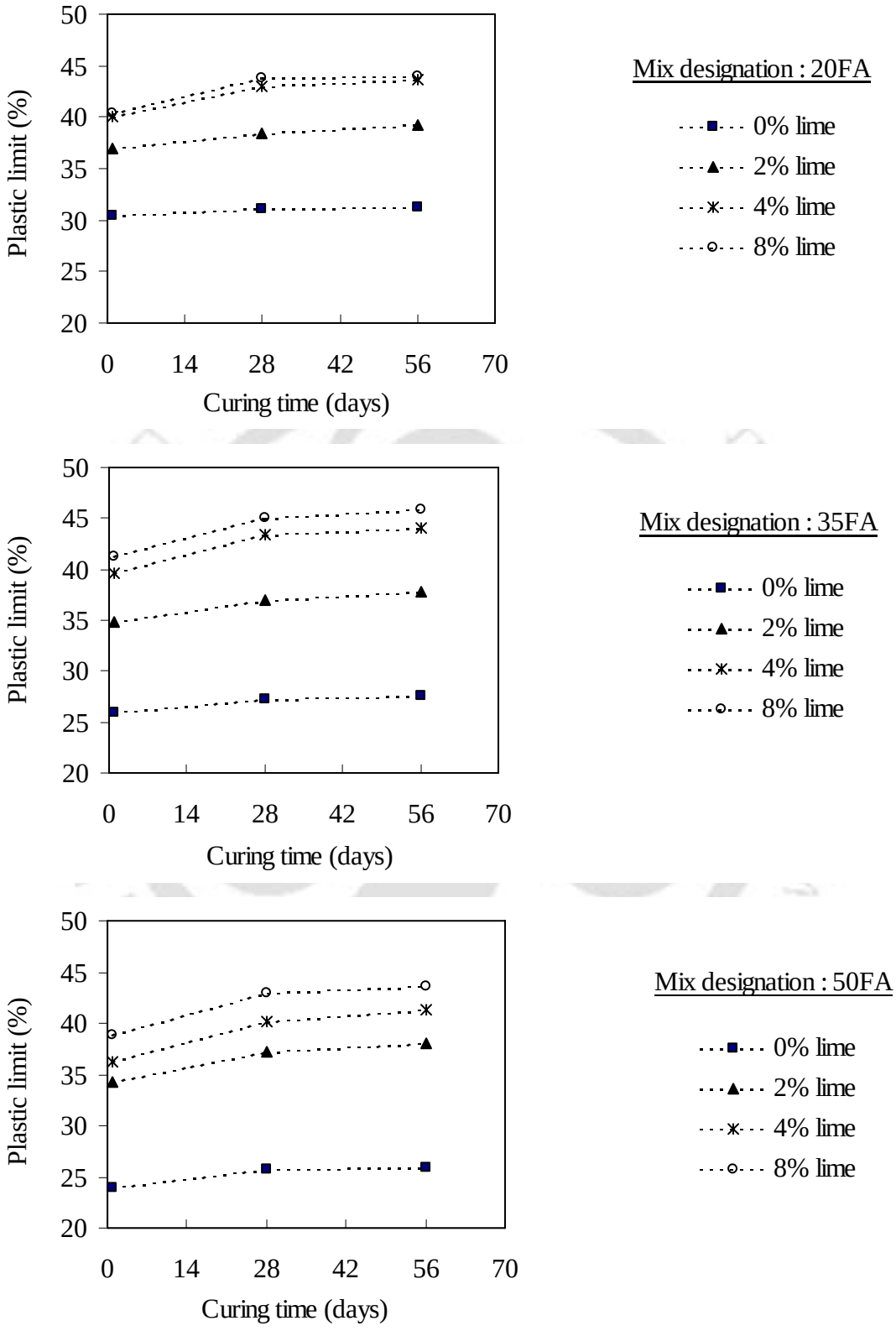


Figure 4.9 Variation of plastic limit of soil-fly ash mixes 20FA, 35FA and 50FA with increasing lime content and conditioning period

These threshold percent lime contents are 4%, 3% and 2% for the mixes 20FA, 35FA and 50FA respectively. In general, the increase in plastic limit of clays upon the addition of lime is considered significant in the sense that the plastic limit is the best indicator of the lime content necessary to achieve the degree of modification sought (Rogers and Glendinning, 1996). This is because of the fact that the variation pattern of plastic limit change is consistent for any clay. In contrast, liquid limit (and hence plasticity index) tends to produce a more variable pattern of changes. The quantity of lime required to be added to a clay to induce full modification of the material is known as the lime fixation percentage addition and it is determined by Initial Consumption of Lime (ICL) test (BS:1924-1990), in which the measurement of an absolute pH is used to determine the fixation value. Rogers et al. (1997) later proposed that the ICL value is likely to be conservative, and recommended the establishment of the lime fixation percentage from the full variation of plastic limit with lime addition up to ICL value. The threshold lime contents obtained from the variation of plastic limit with the lime addition, as observed in the present study (Fig. 4.7), therefore, can be regarded as the lime fixation percentages of the mixes.

Beyond the lime fixation percentage, a marginal increase in plastic limit is still observed for all the mixes. The increase is more for a higher percent of fly ash. The highly alkaline state created by excess lime facilitates dissolution of silica and alumina, which increases as fly ash percent increases. The resulting hydration products lead to a concomitant increase in plastic limit.

Figure 4.9 also shows the effect of increasing conditioning time on the plastic limit of different soil-fly ash-lime mixes. As observed for the liquid limit values, the increase in

plastic limit is also found to occur mostly within the first 28 days of conditioning, beyond which the rate tends to slow down.

4.5.2.3 Variation of Plasticity Index

The plasticity index data for different soil-fly ash-lime mixes are already presented in Tables 4.3(a)–(d). The residual lateritic soil shows immediate decrease in plasticity index upon addition of lime (Fig. 4.4). It is apparent that a lime addition of 2% is sufficient to enhance the workability of the soil by reducing the plasticity index. Increasing the lime content from 2% up to 8% showed marginal effect in reducing plasticity index. Keeping the lime content at 2%, increasing conditioning time was found to have a greater effect on the plasticity index.

The variation of plasticity index for different soil-fly ash mixes with the increase in amount of lime used is illustrated in Fig. 4.10.

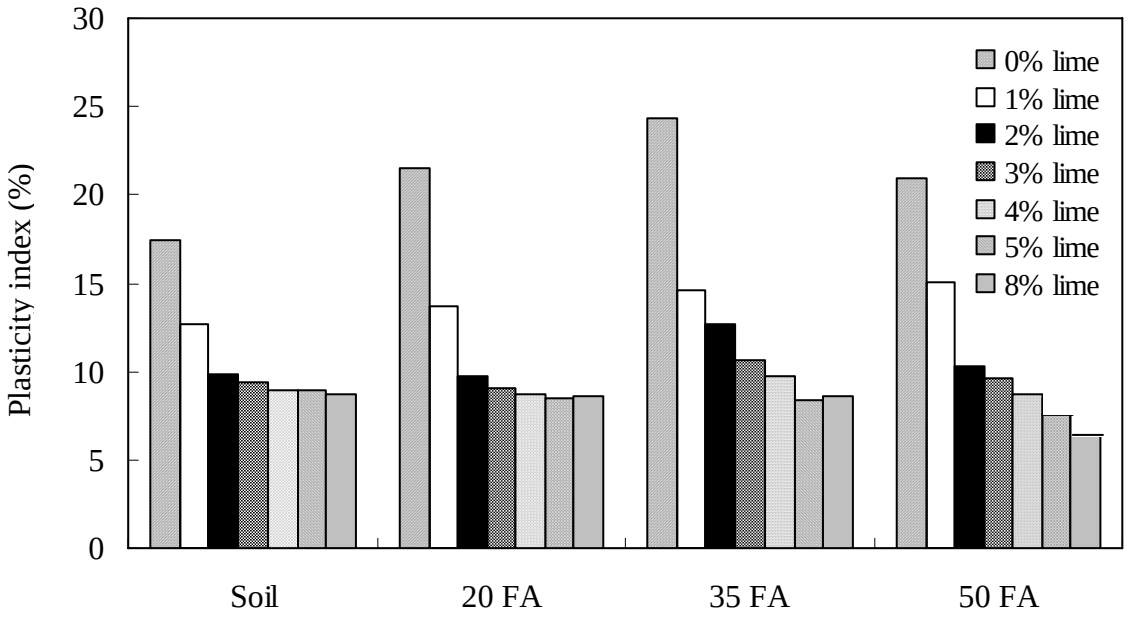


Figure 4.10 Variation of plasticity index of different soil-fly ash mixes with increasing lime content (for 1 day conditioning period)

The increase in plasticity index as a result of the reduction in plastic limit indicates that addition of fly ash alone to the soil is likely to reduce the workability of the mix to some extent. Soil-fly ash-lime mixes on the other hand, clearly exhibit reduced plasticity index upon the addition of just 1% lime, as indicated by a significant reduction in the plasticity index. With higher lime contents, the plasticity index decreases further indicating an improvement in the workability.

Conditioning further reduces the plasticity index and its influence is found to be more effective with higher proportions of fly ash and lime (Figure 4.11 and Tables 4.3(b) to 4.3(d)). For all the mixes, the reduction in plasticity index is mainly on account of the considerable increase in plastic limit.

Workability at site is closely related with the plasticity property of a soil. Generally, the decrease in plasticity index indicates an improvement in workability of the soil (Sivapullaiah, et al., 1996). Based on extensive research on the lime stabilisation, Rogers and Glendinning (1996) reported that changes in plastic limit alone are sufficient to judge the change in site workability. They further observed that in general, it is the increase in plastic limit that is more important for improvement in workability. Results obtained from the present study clearly indicates that addition of low calcium fly ash having grain size finer than the soil is likely to cause significant reduction in site workability of the soil. This is evident from the significant and continuous reduction in its plastic limit with the increase in fly ash proportion. However, the apparent loss of workability of the soil-fly ash mixes can be suitably recovered by addition of adequate quantity of lime. It is also clear that conditioning further improves the workability of the soil-fly ash-lime mixes, mainly due to significant increase in the plastic limit and concomitant reduction of plasticity.

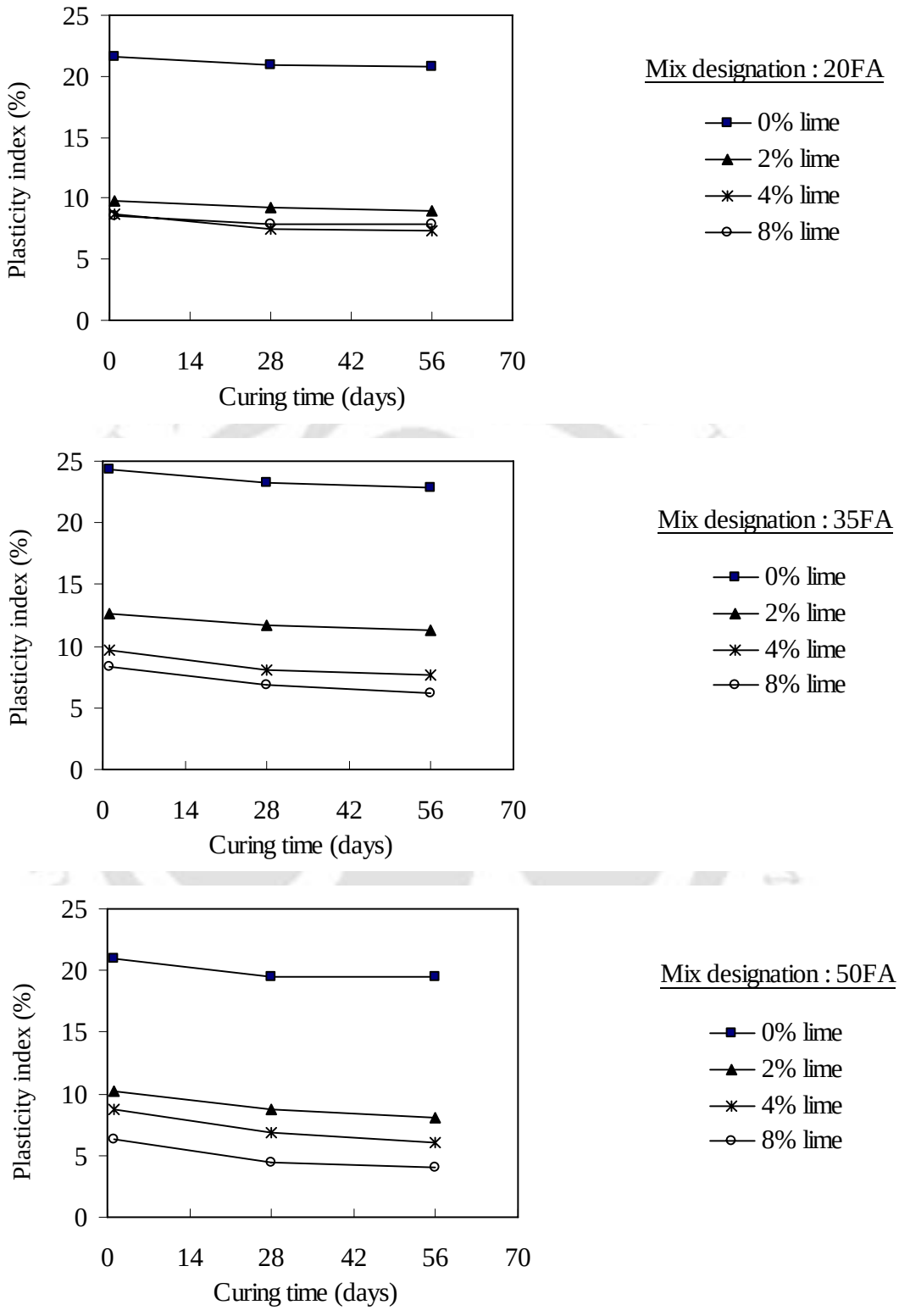


Figure 4.11 Variation of plasticity index of soil-fly ash mixes 20FA, 35FA and 50FA with increasing lime content and conditioning period

4.6 CONCLUSIONS

1. Pre-test sample preparation technique affects the plasticity properties of the residual lateritic soil. It is therefore essential to select an appropriate procedure in accordance with potential field application and also to better comprehend the responses of the modified material with respect to the additives used and the tests they are subjected to.
2. The plasticity properties of the residual lateritic soil can be changed considerably by the addition of lime. Although the independent effects of addition of lime are contrasting in nature (i e. reduction in liquid limit and increase in plastic limit), their combined effect considerably reduces the plasticity index, thereby improving its workability to a large extent. Most of the change is caused by cation exchange and/or flocculation.
3. Addition of fly ash alone to the soil significantly alters its plasticity characteristics. Increasing fly ash content in the mixes causes reduction of both plastic limit and liquid limit, but the observed increase of plasticity index is due to a relatively greater reduction of plastic limit. However, this effect to a great extent depends on the comparative grain size distributions of the soil and the fly ash, as well as the free lime content of the fly ash.
4. When lime is added to the soil in conjunction with fly ash, it results in appreciable reduction of the plasticity index. The change is on account of the considerable increase in plastic limit that makes the mix almost non-plastic after full modification.
5. Conditioning further improves the workability of the soil-fly ash-lime mixes, mainly due to significant increase in the plastic limit and concomitant reduction of plasticity

index. Most of the change is affected in the first 28 days of conditioning and beyond this period the rate considerably slows down.

6. The short-term change in plasticity characteristics of the soil-fly ash-lime mixes appear to be governed by both cation exchange and flocculation. However, in view of the complex nature of plasticity changes as well as early onset of pozzolanic activity in the present case, the specific contribution of each of these mechanisms is not clear at this stage. The long-term change, on the other hand, can be clearly attributed to flocculation.

CHAPTER
FIVE

COMPACTION BEHAVIOUR OF THE SOIL-FLY ASH-LIME MIXES

5.1 INTRODUCTION

Mixing and compaction are important steps for construction processes involving the use of admixtures. Study of plasticity properties of the stabilised soil (Chapter 4) has shown that addition of fly ash and lime causes fundamental changes in the properties of the soil due to short and long-term modifications and formation of various hydration products. Keeping this in view it was considered essential to examine the resulting changes of fly ash and lime addition in compaction behaviour of the soil and its mixes, which are discussed in this chapter.

Compaction often results in progressive breakdown of particles in residual soil and re-compaction results in change of maximum dry density (MDD) and optimum moisture content (OMC) of the soil (Gidigasu, 1974). Moreover, because of their hollow shape, the fly ash particles mixed with the soil are also prone to physical breakdown during compaction and the process is likely to affect the compaction behaviour of the mixes. Therefore, the effects of progressive particle breakdown on compaction behaviour of the mixes were examined by a new method involving successive re-compaction, immediately after completion of tests done by IS light compaction method. As the soil-fly ash-lime modification process continues for a long time, a series of tests were conducted by suitably delaying the compaction to examine how such delay is likely to influence the compaction

behaviour. A preliminary investigation was also carried out to understand the compaction behaviour of the experimental soil before undertaking the above tests.

5.2 COMPACTION TEST PROGRAMME

Table 5.1 summarizes the compaction test program. Three different series of tests were conducted. In the first series, compaction properties of all the soil-fly ash mixes and the effect due to the addition of lime were investigated after conditioning for 24 hours as per IS: 2720 Part (VII). In the second series, tests were conducted to investigate the effects of successive re-compaction and in the third series, the conditioning period was varied up to 3 days to study the effect of delay in compaction.

Table 5.1 Summary of compaction test program on soil-fly ash-lime mixes

Series	Designation of mixes	Lime added (%)	No. of repeat compactions per sample	Delay in compaction
1	RLS, 20FA, 35FA, 50FA	0, 2, 3, 4	0	24 hours
2	RLS, 20FA, 35FA, 50FA	0, 2, 3, 4	3	24 hours
3	RLS, 20FA, 50FA, BFA	0, 2, 4	0	Immediate, 12 hours, 24 hours, 2 days, and 3 days

5.3 EXPERIMENTAL DETAILS

5.3.1 Initial Investigation on the Soil

Earlier investigators on lateritic soils have reported that pre-test sample preparation and laboratory procedure adopted for analysing its compaction characteristics influence the test results to a great extent (Simmons and Blight, 1997). Hence, a series of initial

compaction tests were carried out by varying the pre-test drying condition, maximum grain size and conditioning period from that of standard specifications.

The samples were air-dried, oven-dried or kept un-dried at natural state. In case of air-dried and oven-dried samples, the maximum size of particles was restricted to 0.425 mm, 2 mm or 4.75 mm for different samples to ascertain the individual effects of different size fractions on compaction properties. For soil samples at undried natural state, sieving was not feasible and hence these were tested without size separation. Separation was done with the aim of maintaining uniformity of the grain size distribution of the soil part to be used for studying the variation of the compaction properties when mixed with fly ash and lime. A fresh sample was used every time to obtain each compaction point of the compaction curve. All the compaction tests were carried out using a mould of 100 mm nominal diameter and a 2.6 kg rammer with 310 mm free fall as per IS: 2720 Part (VII) for light compaction.

Based on the results of the preliminary tests, it was decided to air-dry the soil and restrict the maximum grain size to 2 mm.

5.3.2 Test Procedure for the Mixes

For each compaction test, about 6 to 10 kg of less than 2 mm fraction of air-dried soil was mixed with the required percentages of dry fly ash and lime as discussed in the method of preparation soil-fly ash-lime mixtures (Section 3.4). After thorough mixing, the dry soil-fly ash-lime mix was separated into six specimens, to whom different amounts of water were added to give a range of moisture contents suitable for determination of MDD and OMC. The samples were mixed by hand for 5 minutes to obtain a homogeneous mass. The samples were then packed in airtight polythene bags and kept for conditioning prior to

compaction. At the end of specific conditioning period, compaction tests were carried out as per IS: 2720 Part (VII).

5.3.3 Test Procedure for Successive Re-Compaction Test

Immediately after the first compaction done with 24 hours conditioning as per the IS: 2720 Part (VII), each sample was extruded from the mould, clods were broken by applying mild pressure with hand and an additional amount of moisture was added to compensate for the loss of moisture and then allowed to condition for a period of one hour before re-compaction. All the six samples prepared in this manner were compacted again to obtain the 1st re-compaction curve. This procedure was repeated to obtain the 2nd and 3rd re-compaction curves. Thus, each sample was compacted four times.

5.4 RESULTS AND DISCUSSION

5.4.1 Compaction Behaviour of the Soil

Figures 5.1(a) and 5.1(b) show the various compaction curves obtained from the soil under different initial conditions of sample pre-drying method and maximum grain size considered. The corresponding values of MDD and OMC are summarised in Table 5.2. The MDD values range between 1.49 gm/cm^3 to 1.55 gm/cm^3, while the OMC values range between 22.74% and 26.25%. The variation of MDD is less whereas that of OMC is more.

Lateritic soils generally exhibit low MDD due to their porous micro-cluster structure in spite of their constituents having relatively high specific gravity that ranges from 2.7 to 3.5 (Townsend, 1985). Typical values of MDD of these soils range from 1.28-1.68 gm/cm^3.

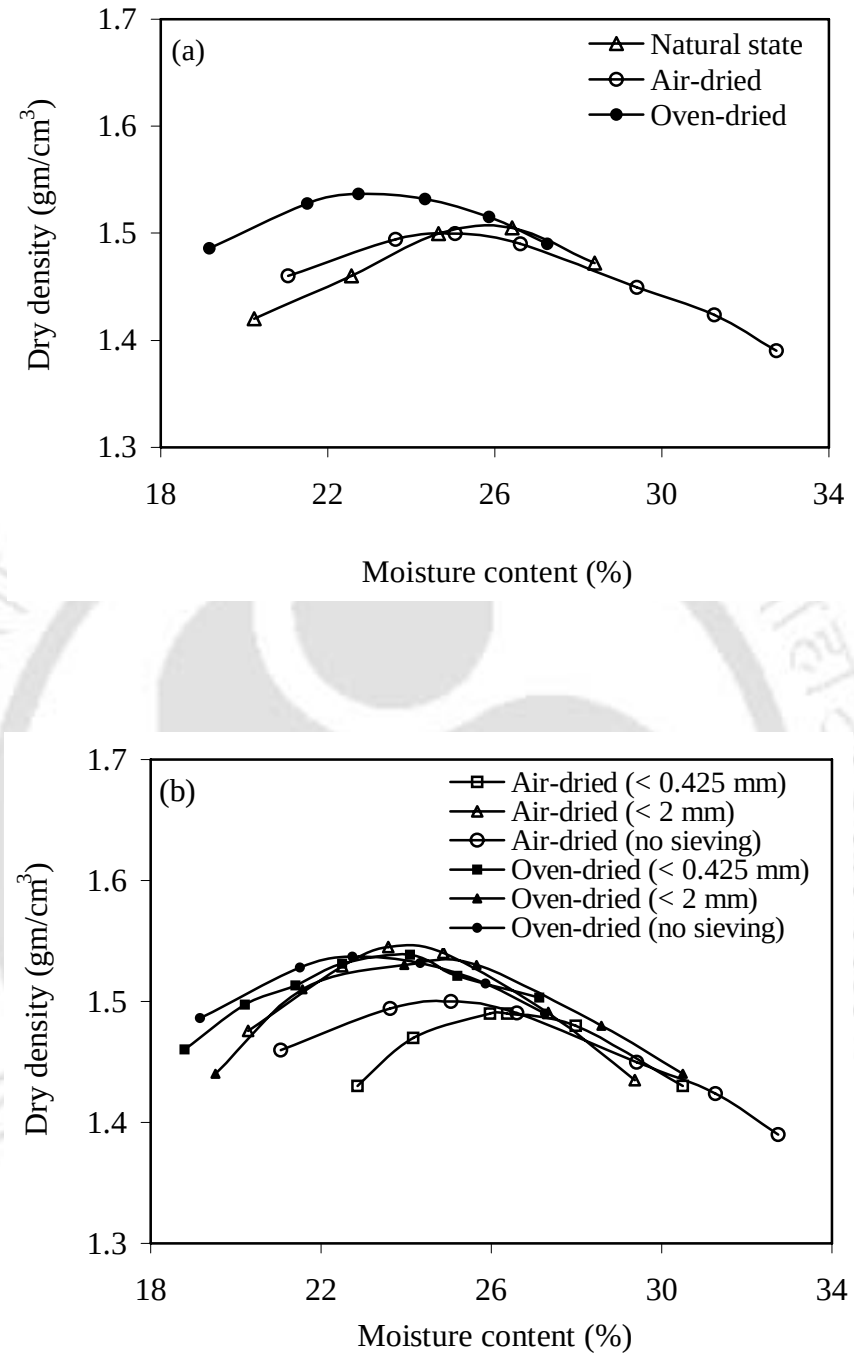


Figure 5.1 Moisture-density relationship of the soil with varying (a) pre-drying condition and (b) maximum grain size considered

Table 5.2 Summary of results of initial compaction tests on soil

Method of pre-drying	Maximum grain size	MDD (gm/cm^3)	OMC (%)
Natural	4.75 mm (As collected)	1.51	25.80
Oven-dried	4.75 mm (As collected)	1.54	22.74
Oven-dried	2.00 mm	1.53	24.52
Oven-dried	0.425 mm	1.55	23.60
Air-dried	4.75 mm (As collected)	1.50	25.05
Air-dried	2.00 mm	1.54	24.12
Air-dried	0.425 mm	1.49	26.25

Fig. 5.1(a) illustrates the influence of sample pre-drying method on the moisture–density relationship of the soil. The effect of oven-drying is more pronounced compared to that of air-drying. Air-drying causes marginal reduction of OMC, whereas oven-drying tends to increase the MDD and reduce the OMC of the soil. The change in properties upon drying is mainly attributed to the aggregation of particles caused by the dehydration of sesquioxides.

Sesquioxides present in lateritic soils coat the clay and other mineral particles. When the soil is dried before compaction, large capillary stresses are developed within the coatings due to dehydration, and the stresses bind the soil particles to form larger aggregates. Within these aggregates, the fine particles are already closely packed. Under normal compactive effort, these aggregates easily break down and the fine particles get closely packed thus increasing the MDD. Both Townsend (1985) and Gidigasu (1974) have reported earlier that lateritic soils when air-dried/oven-dried, usually show lower OMC and higher MDD.

Figure 5.1(b) illustrates the influence of particle gradation on the moisture-density relationship of the soil under different initial pre-drying conditions. For oven-dried samples, the maximum grain size appears to have no significant effect on the variation of MDD and OMC. However, for air-dried samples, the MDD for the soil finer than 2mm is observed to be more.

The above preliminary investigation clearly shows that every time the initial conditions are changed, different compaction curves are obtained. A close examination of the compaction curve of the air-dried soil with grain size less than 2 mm shows that its moisture-density relationship is very similar to that of the soil at un-dried natural state, which was observed to be totally finer than 4.75 mm (Fig. 3.4) This is possibly due to the fact that the size fraction coarser than 2 mm in the soil is considerably less (0.3%).

However, the percentage of this size fraction may differ considerably depending on the burrow area, particularly when the soil is procured in bulk quantities. This fraction may also interfere with the uniform distribution of additives such as fly ash and lime. Therefore, it was decided to use only soil samples finer than 2 mm after air-drying for subsequent investigations. This ensured a better control over the matrix of the composite material undergoing time-dependent pozzolanic reactions.

5.4.2 Effect of Fly Ash and Lime

Figure 5.2 shows all the compaction curves obtained as per IS method for the soil and various soil-fly ash-lime mixes. From these curves, the values of MDD and OMC have been obtained and are tabulated in Table 5.3. Their variations with percent fly ash and percent lime are presented in Fig. 5.3 and Fig. 5.4.

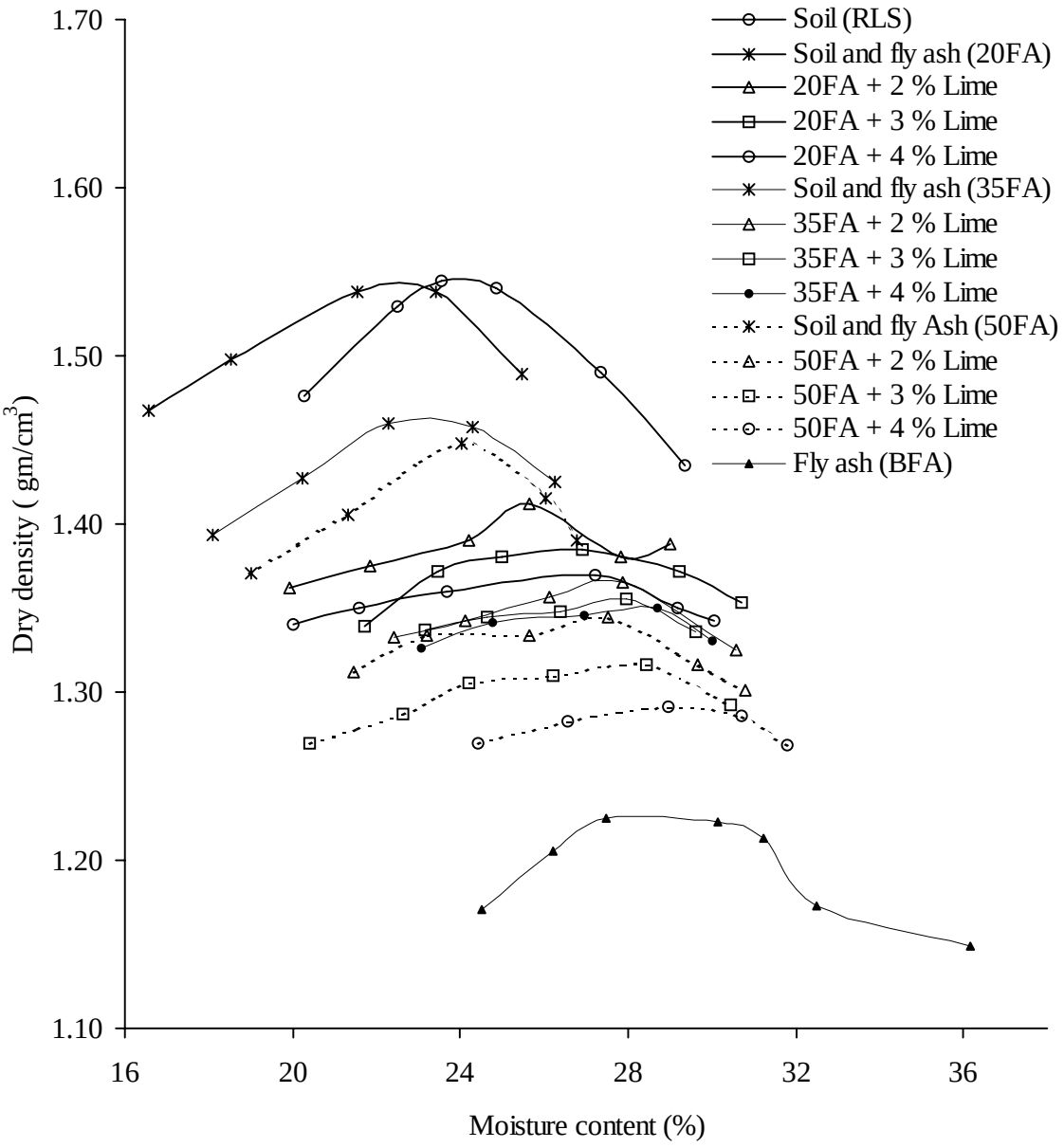


Figure 5.2 Compaction curves of soil, fly ash and various soil-fly ash-lime mixes

Table 5.3 Variation of MDD and OMC of various mixes with single and repeated compaction.

Mix	% Lime added	MDD (gm/cm^3)				OMC (%)			
		As per IS method	1st Repeat	2nd Repeat	3rd Repeat	As per IS method	1st Repeat	2nd Repeat	3rd Repeat
RLS	0	1.54	1.560	1.560	1.550	24.12	24.69	24.28	24.54
20FA	0	1.54	1.564	1.560	1.568	22.58	22.89	22.78	22.98
35FA	0	1.46	1.483	1.512	1.520	23.21	23.77	23.06	22.98
50FA	0	1.45	1.467	1.480	1.486	24.20	24.12	23.55	23.43
RLS	2	1.50	---	---	---	25.20	---	---	---
20FA	2	1.40	1.418	1.438	1.452	25.80	26.41	25.94	26.31
35FA	2	1.37	1.425	1.435	1.452	27.33	26.50	25.43	25.25
50FA	2	1.34	1.395	1.422	1.432	27.53	27.11	26.76	26.66
RLS	3	1.49	---	---	---	25.55	---	---	---
20FA	3	1.38	1.415	1.441	1.442	26.31	26.31	26.06	25.36
35FA	3	1.36	1.401	1.418	1.435	27.81	27.10	26.48	25.98
50FA	3	1.32	1.343	1.400	1.413	28.43	27.39	27.26	26.28
RLS	4	1.49	---	---	---	25.74	---	---	---
20FA	4	1.37	1.401	1.420	1.437	26.86	26.63	25.61	25.32
35FA	4	1.34	1.405	1.414	1.425	28.51	25.98	26.21	26.05
50FA	4	1.29	1.350	1.370	1.355	29.24	27.56	28.13	27.91

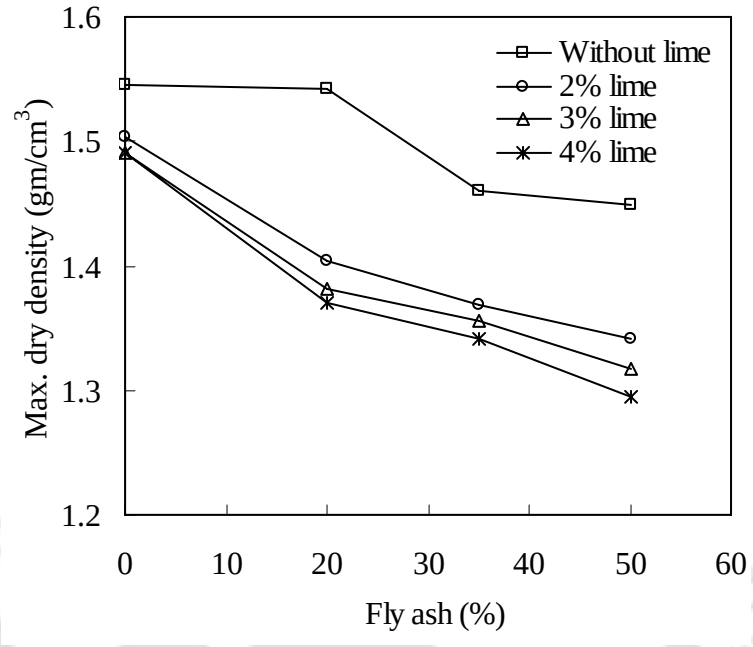


Figure 5.3 Variation of MDD of the mixes with the change in fly ash proportion and percent lime added

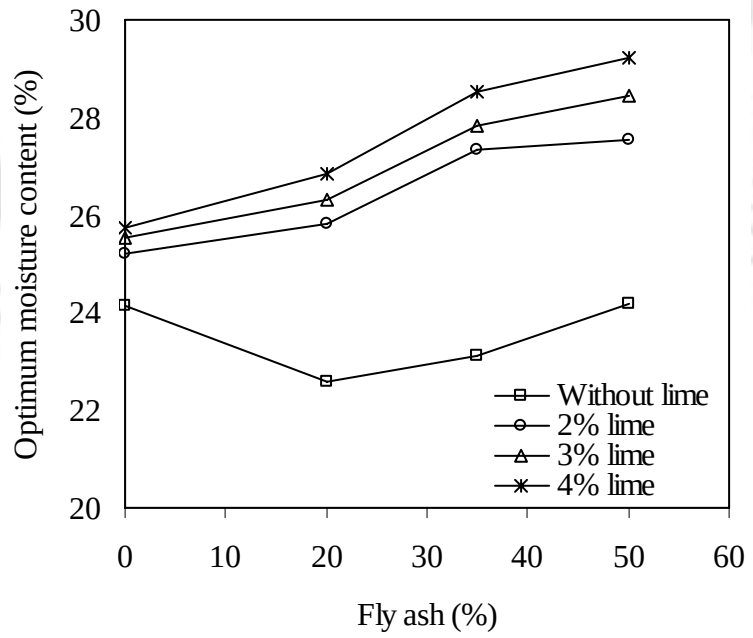


Figure 5.4 Variation of OMC of the mixes with the change in fly ash proportion and percent lime added

It can be observed from Fig. 5.3 that the MDD of the soil-fly ash mixes does not get significantly altered up to 20% replacement of soil with fly ash. With further addition of fly ash, the MDD decreases. The MDD of the fly ash is 1.23 gm/cm^3, which is much lower than that of the soil (1.54 gm/cm^3). Also, an examination of the grain size distributions of the soil and fly ash (Fig. 4.8) shows that the fly ash is much finer than the soil in between the size fractions of 0.012 mm to 0.10 mm. The lighter and finer particles of the fly ash initially fill up the voids between the relatively coarser particles of the soil at low proportions. After the voids have been filled up to the maximum extent possible, as the fly ash content increases, the MDD of the mixes decreases.

The OMC of the soil-fly ash mixes without lime decreases initially up to 20% fly ash content, and then gradually increases with higher fly ash content (Fig. 5.4). Although the fly ash has OMC of 28.30%, which is higher than that of the soil (24.12%), the initial decrease in OMC is due to the fine fly ash particles occupying some of the air voids that could have been filled up with water. The increase in OMC beyond 20% fly ash can be attributed to the higher water holding capacity of the porous fly ash particles.

As explained in section 4.5.2.1, the fly ash used has low free lime content and its effect on MDD and OMC of the soil-fly ash mixes can be assumed to be almost negligible. When extra lime is added to the soil-fly ash mixes, significant changes in its compaction characteristics are observed (Fig. 5.2). It can be seen from Fig. 5.3 that for any proportion of fly ash, MDD decreases more with the increase in lime percentage. The reduction is found to vary from 9.0% to 11.5% for 20FA, 6.4% to 8.2% for 35FA, and 7.5% to 10.7% for 50FA, as different quantities of lime are added (2%, 3% and 4% by weight). The

amount of reduction is significant compared to the addition of fly ash alone. This is due to the extensive modification of the mixes brought about by the lime addition. Due to these modifications, changes are observed in texture, thickness of adsorbed water layer, inter-particle forces /particle arrangement, and internal angle of friction.

It is also evident from Fig. 5.3 that for a given lime content, the MDD of the soil reduces continuously with the increase in the proportion of fly ash. The amount of reduction, when compared with the MDD of the soil, is found to be 9.5% to 13.2% for 2% lime addition, 10.6% to 14.8% for 3% lime addition and 11.3% to 16.2% for 4% lime addition. The reason for this change can be explained from the low MDD of the fly ash and its dilution effect.

The OMC of the soil is also significantly affected by the addition of lime and fly ash as clearly illustrated in Fig. 5.4. For a given proportion of fly ash, the OMC increases with the increase in the amount of lime. Addition of lime from 2% to 4% results in 14.3% to 19.0% rise in OMC for 20FA, 17.3% to 22.83% for 35FA, and 13.8% to 20.83% for 50FA, when compared with the OMC of the respective mixes without the addition of lime. Similarly, when the proportion of fly ash is increased from 0% to 50%, the OMC of the mix increases by 6.9% to 13.3% for 2% lime addition, 9.0% to 17.9% for 3% lime addition and 11.4% to 21.2% for 4% lime addition.

The patterns of decrease in MDD and increase in OMC with increasing fly ash percentage are similar to that of soils with increasing content of finer fraction. Further, it can be noted from Figs. 5.3 & 5.4 that the magnitudes of change in MDD and OMC of all

the mixes are greater with 2% of lime addition than 4% lime addition. This is due to the reduced demand for lime for short-term modification of the mixes.

As explained earlier (Section 4.5.2.2) the quantity of lime required for immediate modification is best understood from the study of changes of the plastic limit of the soil upon the addition of lime (Rogers and Glendinning, 1996). A comparison of the variation of plastic limit (Fig. 4.7) and MDD & OMC (Fig. 5.3 and 5.4) of the mixes due to the change in lime content shows a similar pattern of variation indicating that change in MDD and OMC of the mixes can be correlated with the variation of plastic limit. Similar to the changes in MDD and OMC, the plastic limit of the mixes exhibit greater incremental change with the initial 2% lime and beyond this lime content, considerably less modification is noticed.

5.4.3 Effect of Re-Compaction

The curves obtained from successive re-compaction of the soil are shown in Fig. 5.5. The corresponding MDD and OMC values obtained from these curves are also tabulated in Table 5.3. It can be noted from the figure and the table that remoulding and repeated compaction have very little influence on the MDD and OMC of the soil.

Figures 5.6(a) to (c) show re-compaction curves for different soil - fly ash mixes without the addition of lime. It can be seen from the curves that when 20% fly ash is added to the soil, the MDD of the mix shows minor variation. However, increase the fly ash content appreciably changes the re-compaction curves. In case of the 35FA mix, after each successive compaction, compaction points on both dry and wet sides of the OMC depict

greater dry densities; whereas in the case of the 50FA mix, the increase in dry density is observed only on the dry side of the OMC.

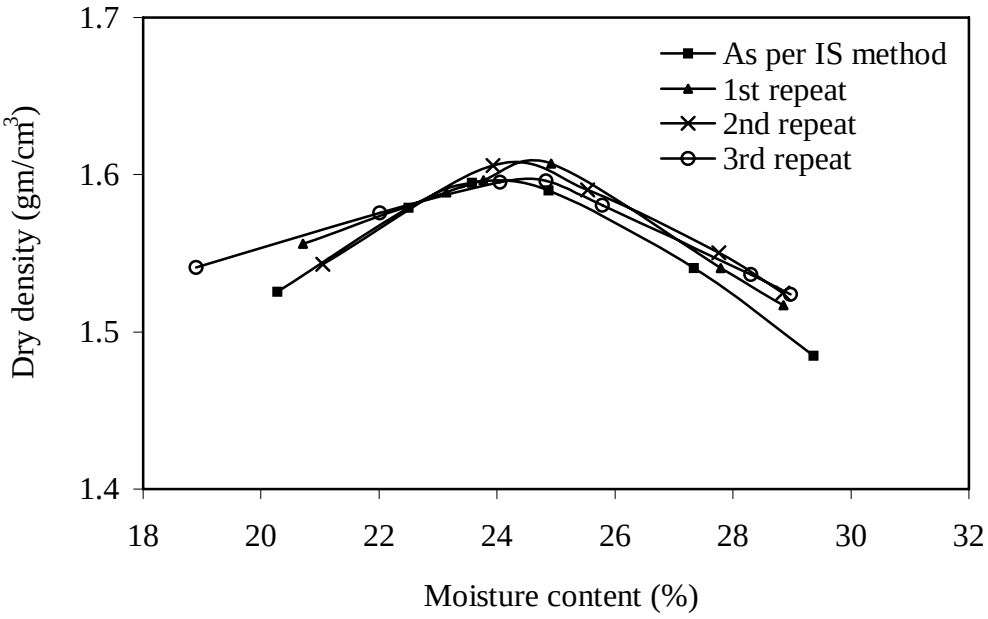
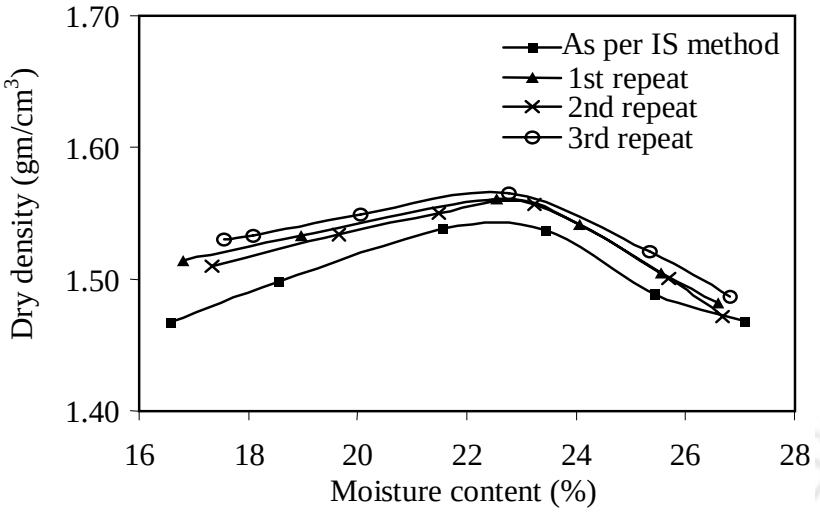
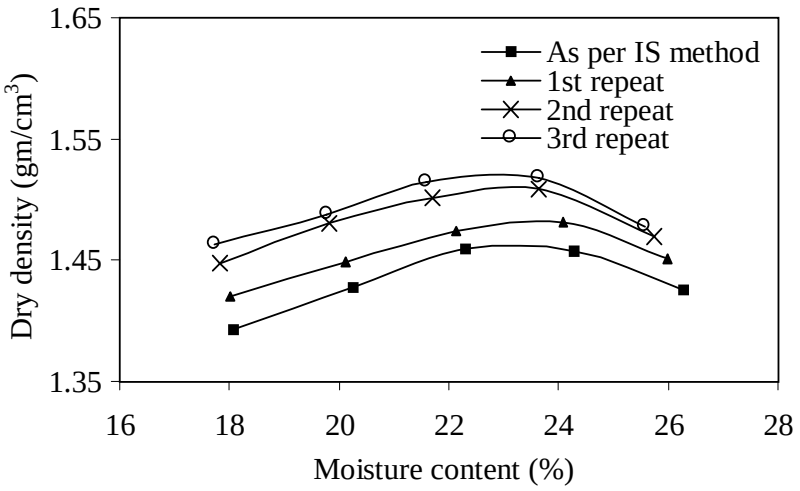


Figure 5.5 Compaction curves obtained from successive re-compaction of soil

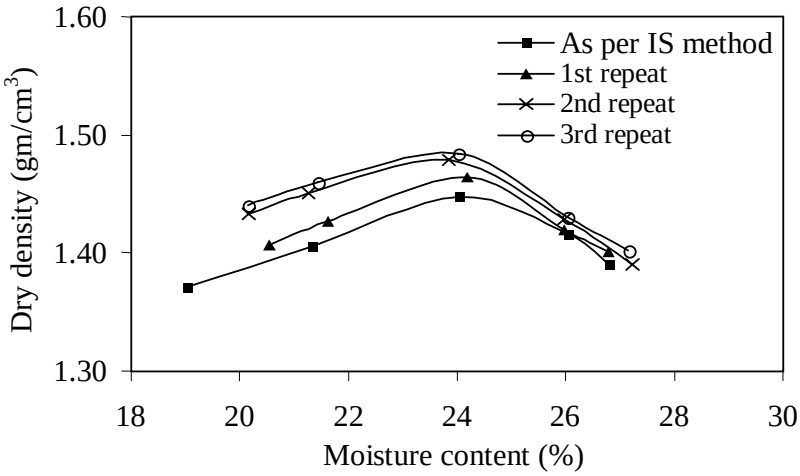
When lime is added, substantial change in the re-compaction characteristics is observed for all the soil-fly ash mixes as shown in Figs. 5.7 to 5.9. The variations of MDD and OMC of the soil-fly ash, and soil-fly ash-lime mixes due to each successive re-compaction are illustrated in Figs. 5.10 through 5.17, and the values are also presented in Table 5.3. For all the mixes, the patterns of variation of MDD and OMC values with the change in fly ash proportion for each re-compaction are somewhat identical with the pattern shown by the initial compaction.



(a)
Mix designation: 20FA
Lime content – nil

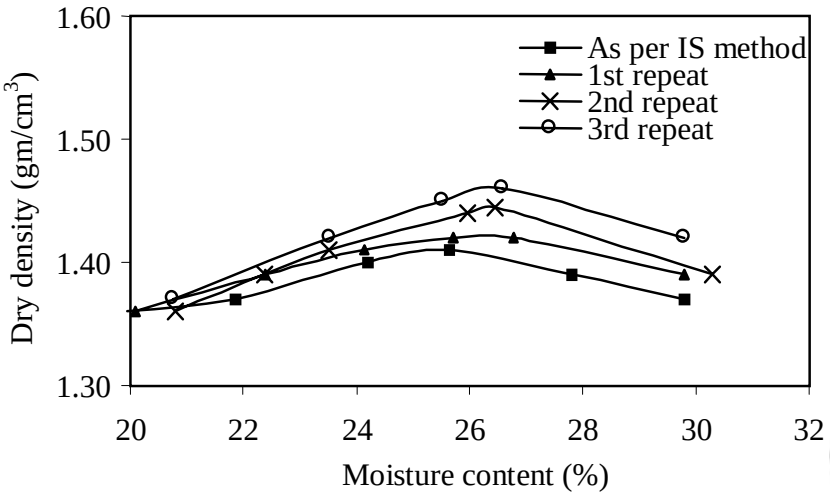


(b)
Mix designation: 35FA
Lime content – nil

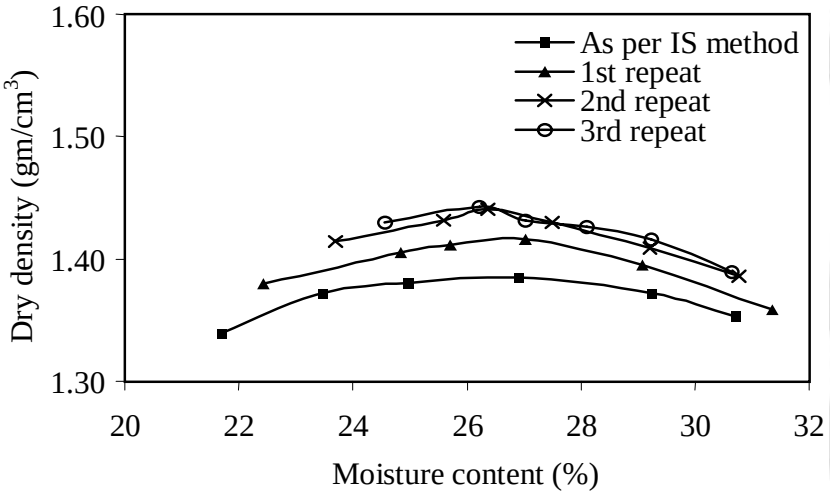


(c)
Mix designation: 50FA
Lime content – nil

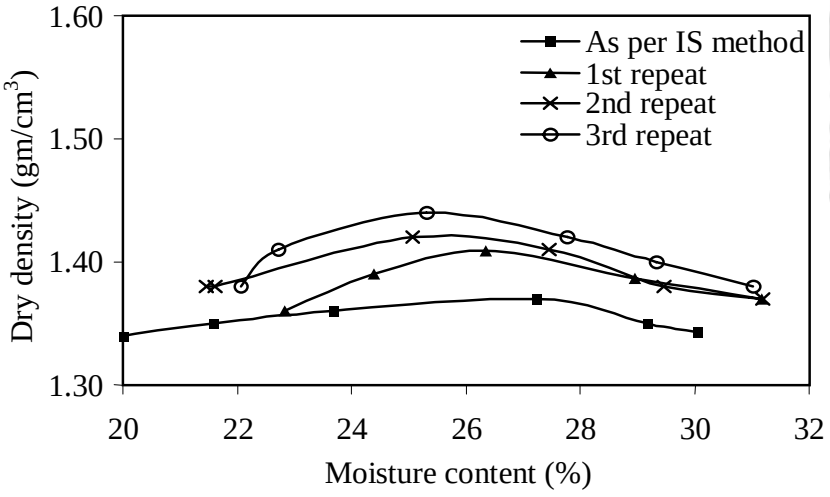
Figure 5.6 Effects of repeated compaction on dry density vs. moisture content relationship of different soil–fly ash mixes without addition of lime



(a)
Mix designation: 20FA
Lime content – 2%

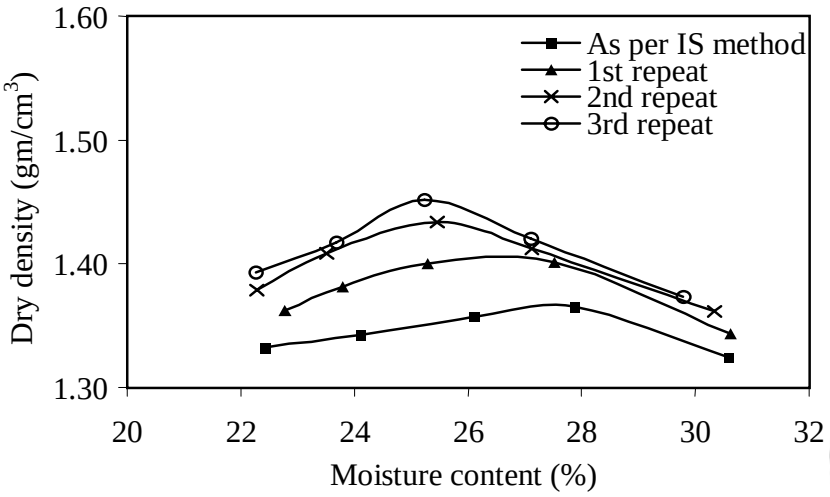


(b)
Mix designation: 20FA
Lime content – 3%

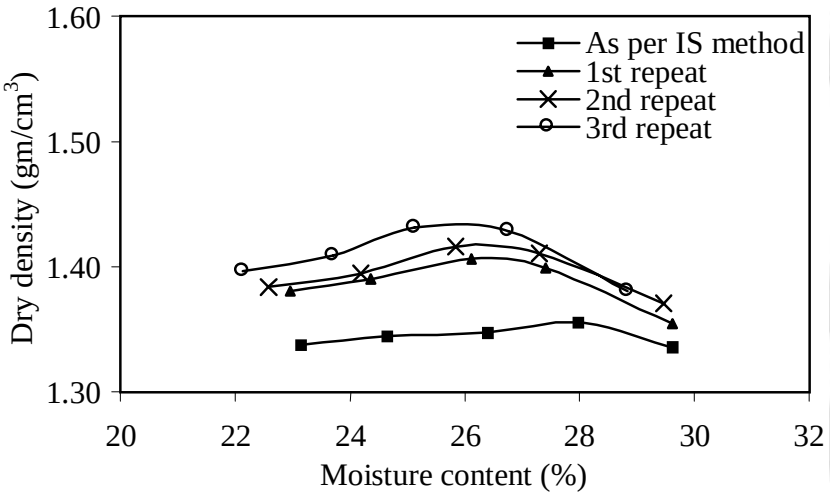


(c)
Mix designation: 20FA
Lime content – 4%

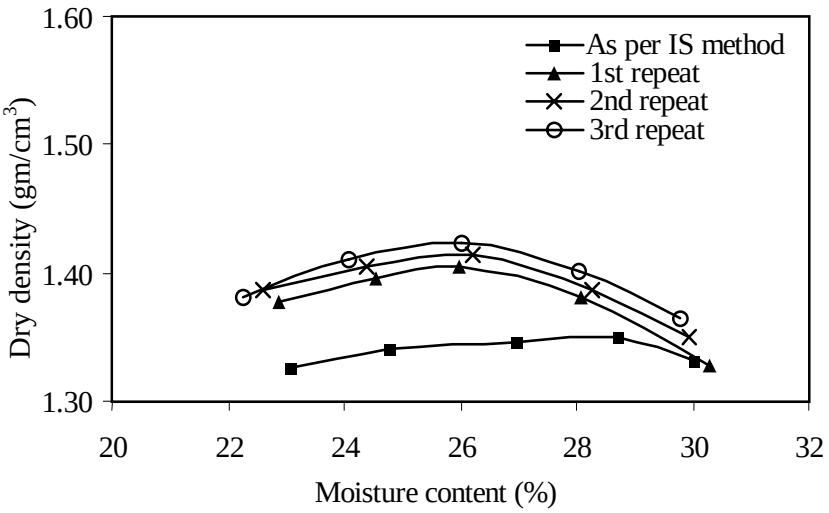
Figure 5.7 Effects of repeated compaction on dry density vs. moisture content relationship of 20FA mix with various amounts of lime



(a)
Mix designation: 35FA
Lime content – 2%

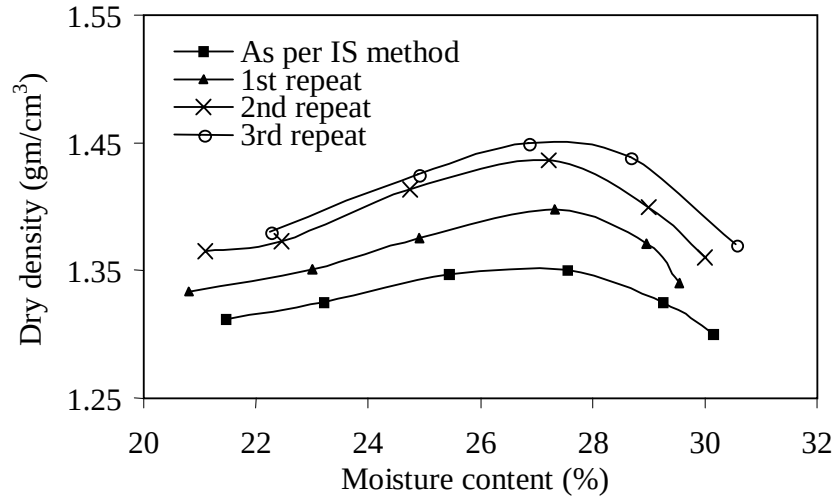


(b)
Mix designation: 35FA
Lime content – 3%

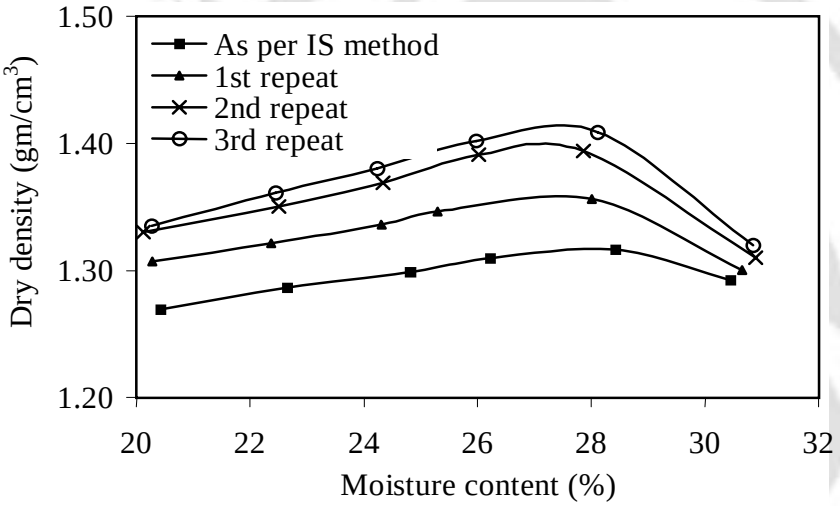


(c)
Mix designation: 35FA
Lime content – 4%

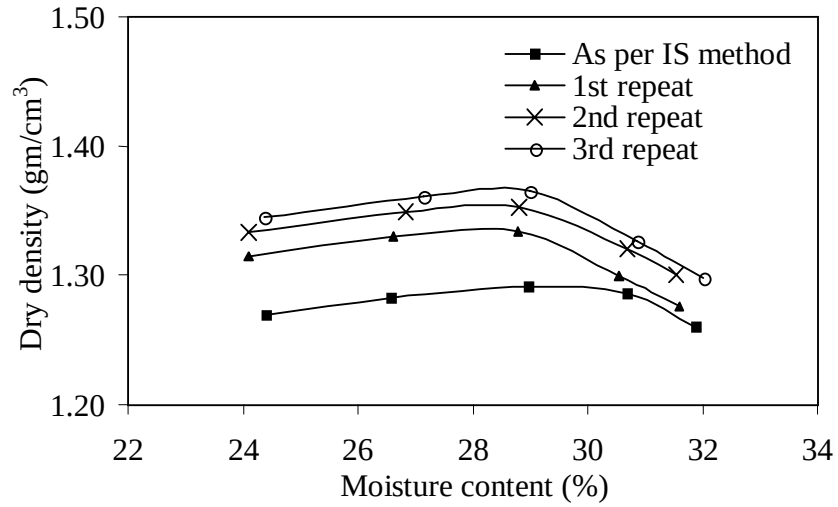
Figure 5.8 Effects of repeated compaction on dry density vs. moisture content relationship of 35FA mix with various amounts of lime



(a)
Mix designation: 50FA
Lime content – 2%



(b)
Mix designation: 50FA
Lime content – 3%



(c)
Mix designation: 50FA
Lime content – 4%

Figure 5.9 Effects of repeated compaction on dry density vs. moisture content relationship of 50FA mix with various amounts of lime

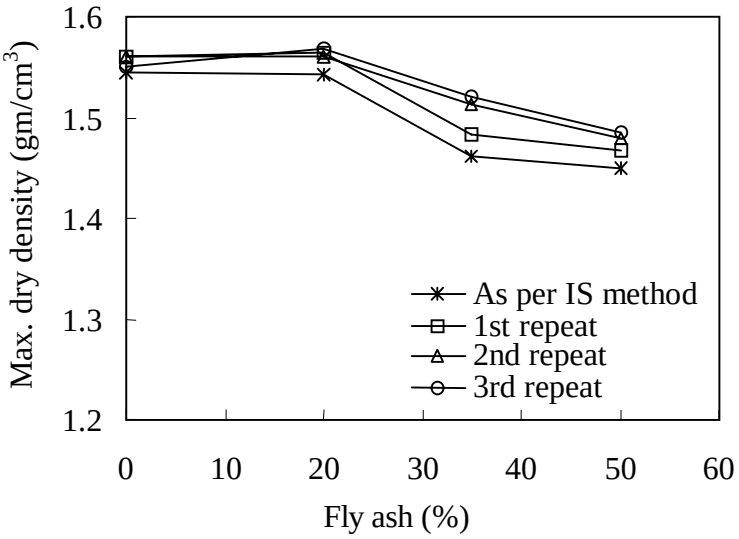


Figure 5.10 Variation of MDD of the soil due to the variation of fly ash proportion and repeat compaction (No lime added)

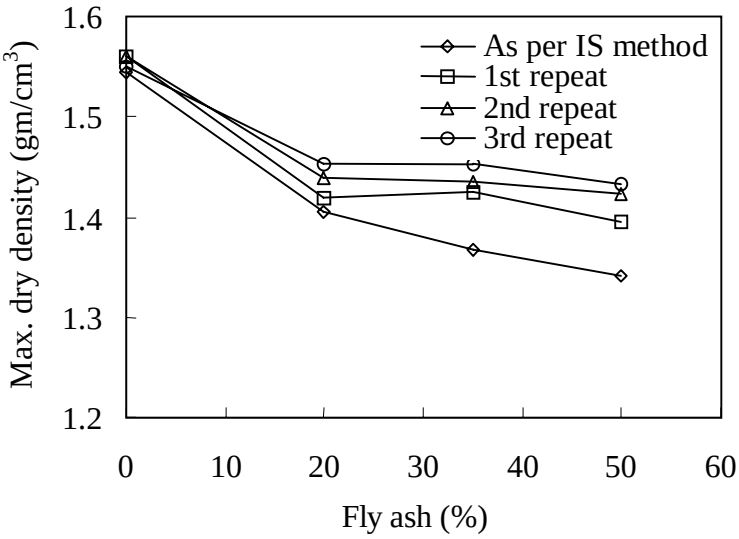


Figure 5.11 Variation of MDD of the soil due to the variation of fly ash proportion and repeat compaction (with 2% lime added)

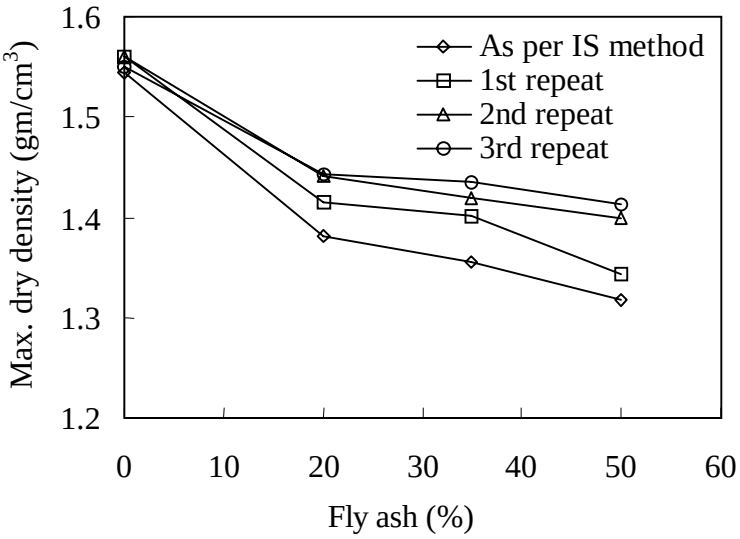


Figure 5.12 Variation of MDD of the soil due to the variation of fly ash proportion and repeat compaction (with 3% lime)

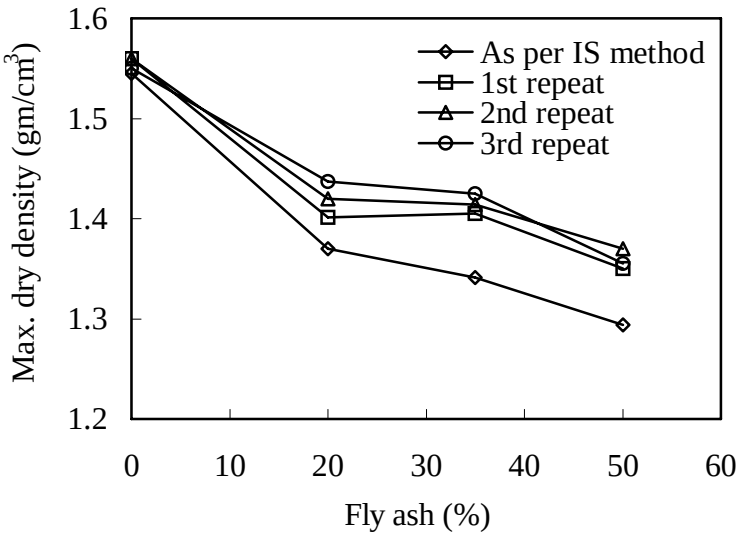


Figure 5.13 Variation of MDD of the soil due to the variation of fly ash proportion and repeat compaction (with 4% lime)

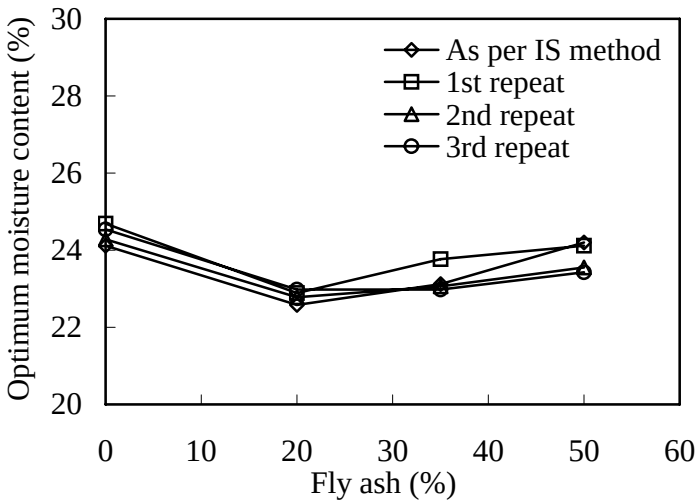


Figure 5.14 Variation of OMC due to the variation in fly ash proportion and repeat compaction of the soil (No lime added)

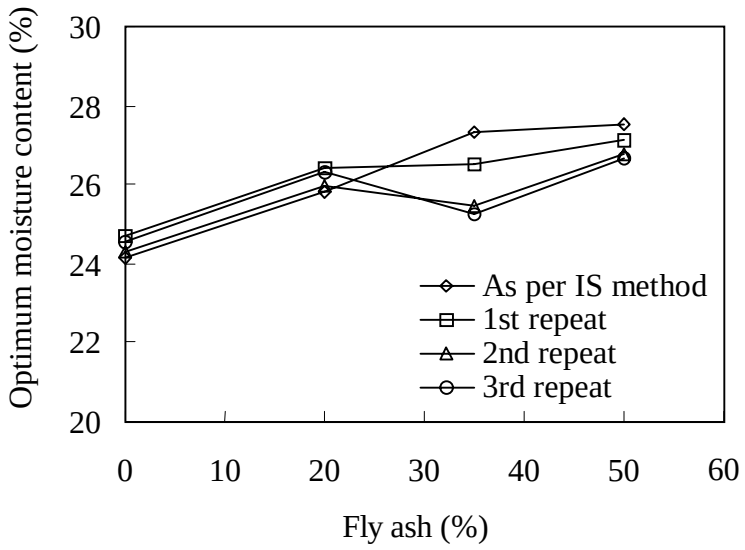


Figure 5.15 Variation of OMC due to the variation in fly ash proportion and repeat compaction of the soil (with 2% lime added)

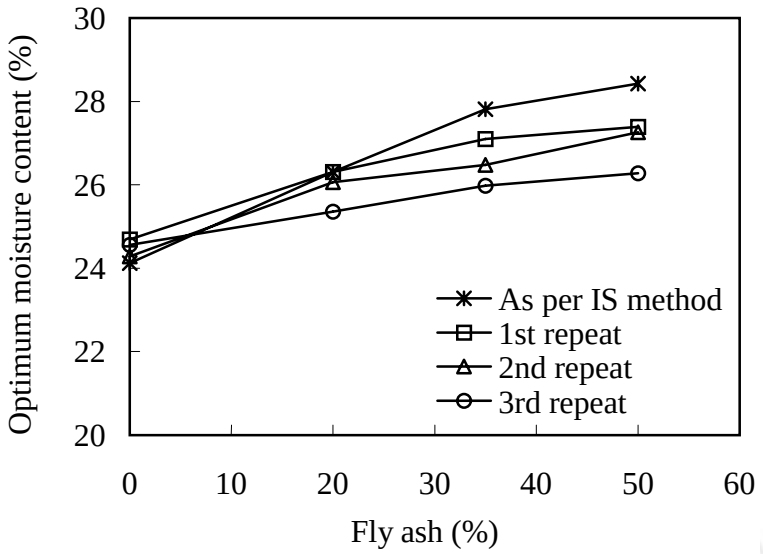


Figure 5.16 Variation of OMC due to the variation in fly ash proportion and repeat compaction of the soil (with 3% lime)

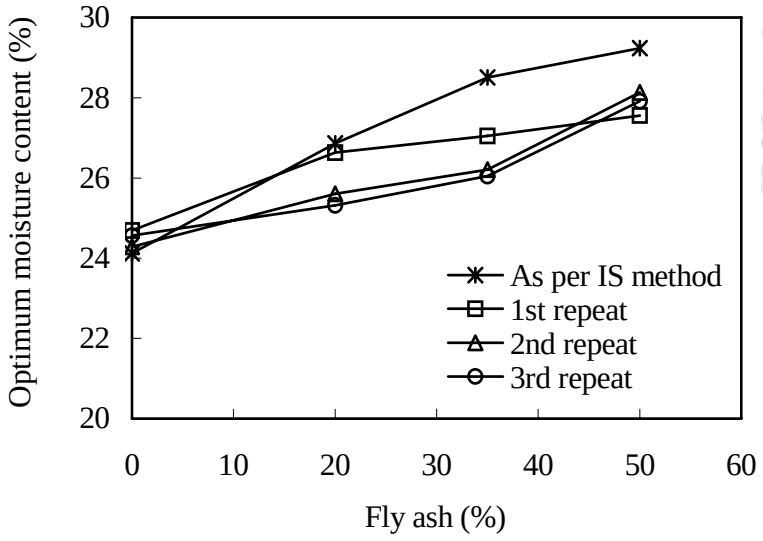


Figure 5.17 Variation of OMC due to the variation in fly ash proportion and repeat compaction of the soil (with 4% lime)

Irrespective of any fly ash content, the MDD of the mix increases with the increase in the number of successive re-compactions. For all the mixes, the changes of MDD obtained in the 1st re-compaction curve is larger compared to the compaction curves generated due to subsequent re-compactions.

Figure 5.10 shows that when no lime is added, the increase in MDD values obtained in the final repeat compaction, compared to that of the initial compaction, are found to be in decreasing order for 35FA, 50FA and 20FA mixes respectively, indicating that at 35% fly ash content, the step-wise increments are greater. On the other hand, the variations of OMC values for the successive re-compaction are found to be less and identical for all the fly ash contents (Fig. 5.14).

However, when lime is added to the mixes, the increase in MDD values is generally greater at a higher fly ash content (Figs. 5.11, 5.12, and 5.13). The OMC values, at any fly ash content, generally decrease with the number of repeated compaction (Figs. 5.15, 5.16, and 5.17). However, no specific trends could be delineated about the magnitude of change.

It is clear that increase in MDD upon successive re-compaction occurs only in the presence of higher amount of fly ash, and the addition of lime further enhances the effect. Unlike soil, the fly ash particles are spherical and hollow with a glassy exterior surface, commonly known as cenospheres. The glassy fraction in fly ashes usually varies from 70% to 80% depending on the type and coal source (Joshi and Lohtia, 1997). During compaction, a part of the compaction energy is spent in breaking these hollow particles and with each successive re-compaction, these broken particles get closely packed, thereby increasing the MDD of the mixes. Further, it appears that the majority of the particles seem

to break during the first re-compaction itself, as reflected by decreasing magnitude of increase in MDD beyond the first re-compaction.

The increased effect of change in the magnitude of increase in MDD upon re-compaction, with lime added, can be attributed to the modification of the clay fraction as already discussed. The modification process results in a textural change and the plastic soil changes to a friable material that is more granular in nature. This might have resulted in a better grain-to-grain transfer of compaction energy, leading to further breakdown of fly ash particles and consequent increase of MDD after the first repeat compaction. Further, during compaction, lime added to the mixes is more intimately mixed with soil and fly ash due to closer packing and exposure of fresh broken particle surfaces which results in further modification that is reflected in continued change in MDD and OMC of the mixes upon successive re-compaction. The decrease in OMC values upon successive re-compaction of mixes can be attributed to the change in gradation of the mixes after the addition of lime

The study of re-compaction characteristics indicate that the dry density of the soil – fly ash – lime mixes can be increased by as much as 7% through the process of re-compaction.

5.4.4 Effect of Delay in Compaction

Standard compaction methods specify a minimum delay period of about 16 hours to allow for uniform moisture distribution after mixing. The results of compaction tests performed to study the effect of compaction delay ranging up to 3 days are presented in Table 5.4.

Table 5.4 Change in MDD and OMC due to variation of delay in compaction of selected mixes

Mix	Lime added (%)	Maximum dry density (gm/cm^3)					Optimum Moisture Content (%)				
		No delay	12 hours	1 day	2 day	3 day	No delay	12 hours	1 day	2 day	3 day
RLS	Nil	1.51	1.54	1.56	1.54	1.53	26.32	25.12	24.12	24.31	24.5
20FA	0	1.52	1.54	1.54	1.51	1.52	23.21	22.74	22.58	22.78	22.68
35FA	0	1.45	1.46	1.46	1.46	1.46	23.45	23.13	23.21	23.31	23.00
50FA	0	1.44	1.43	1.43	1.42	1.42	24.36	24.51	24.2	24.26	24.22
20FA	2	1.41	1.40	1.40	1.38	1.39	25.28	25.83	25.8	26.04	25.96
35FA	2	1.38	1.37	1.37	1.33	1.32	25.36	27.04	27.33	26.79	26.58
50FA	2	1.35	1.34	1.33	1.31	1.28	25.01	26.65	27.53	27.09	27.48
20FA	4	1.38	1.38	1.37	1.37	1.37	25.90	26.75	26.86	26.95	27.11
35FA	4	1.36	1.35	1.34	1.33	1.31	26.35	27.46	28.51	28.60	28.62
50FA	4	1.31	1.30	1.29	1.28	1.27	27.35	28.00	29.24	28.88	29.20
BFA	Nil	1.23	1.22	1.23	1.21	1.22	28.050	27.65	28.32	27.36	28.34

Both the MDD and OMC of the soil show some variation with 24 hours of delay in compaction (Figs. 5.18 & 5.19). Beyond this period, however, little further change is observed. The MDD for the soil gradually increases with the progressive delay in compaction up to 24 hours, while the OMC shows a decreasing trend within the same period. This variation in MDD and OMC during the initial 24 hours delay in compaction can be attributed to the progressive distribution of moisture within the soil mass. An even distribution of the moisture appears to take place by 24 hours. The MDD and OMC of the fly ash on the other hand is less affected by the delay in compaction, which is due to the low free lime content of the particular fly ash used.

Figures 5.18 & 5.19 also show the changes in the MDD and OMC respectively due to delay in compaction of soil - fly ash mixes. Change in MDD due to delay is about the same for soil-ash mixes as for soil alone. The changes to the MDD due to compaction delay of the soil - fly ash mixes upon the addition of 2% and 4% lime are illustrated in Fig. 5.20. For all mixes, the MDD progressively decreases upon the addition of lime. However, for 2% lime addition, the rate of decrease is found to increase after 24 hours.

The immediate changes to the MDD upon the addition of lime are due to the short-term modification of the soil present in the mix due to flocculation and consequent textural change. The further progressive change is on account of pozzolanic reactions between lime, silica and alumina present in the mix that further modify the material due to the formation of hydration products. With the increase in the percentage of fly ash and lime, more of these products are formed with more delay. The hydration products bind the particles to form aggregates that resist a part of the compactive effort, without undergoing much deformation themselves.

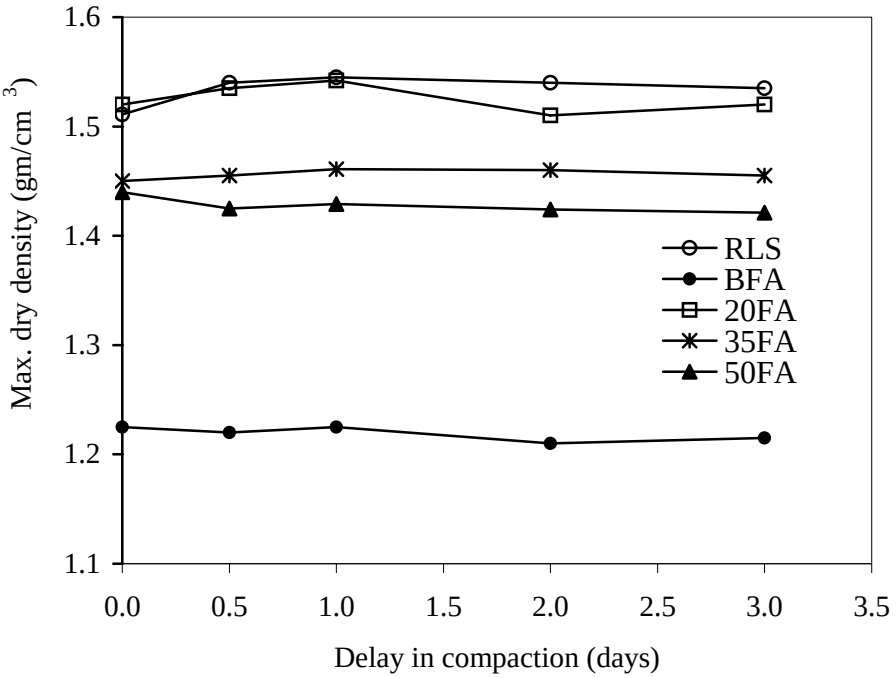


Figure 5.18 Variation of MDD with compaction delay for the soil, fly ash and their mixes without lime

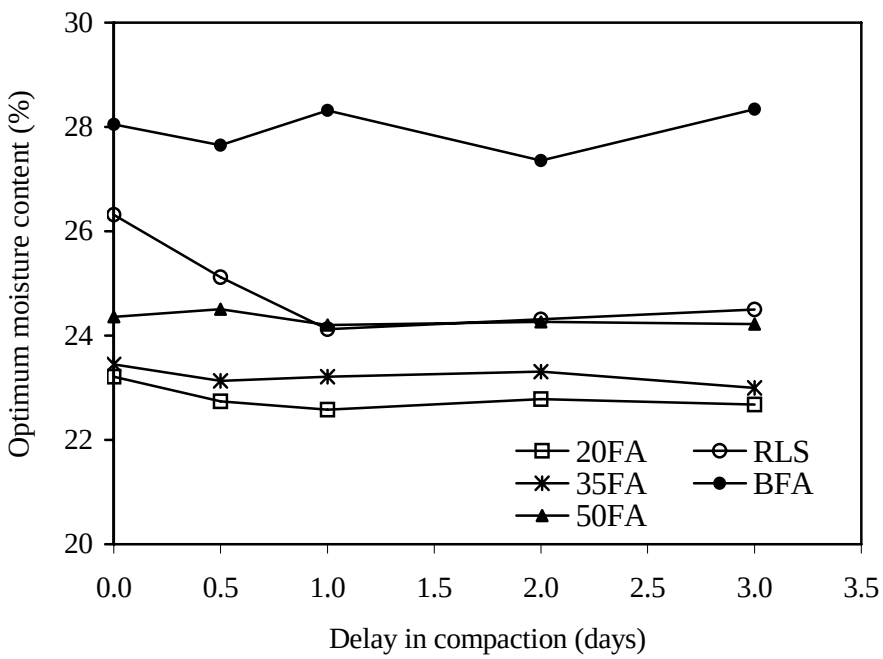


Figure 5.19 Variation of OMC with compaction delay for the soil, fly ash and their mixes without lime

It can be also noted from Fig. 5.20 that when compaction is carried out immediately without delay, mixes with 4% lime show lower MDD compared to mixes with 2% lime. However, the difference in MDD reduces beyond 24 hours of delay. Comparable values of MDD are obtained after a delay period of 3 days before compaction. This is due to the reason that more time is necessary for 2% lime to complete equivalent short-term modification of the soil, which otherwise can be achieved in less time with the addition of 4% lime.

The changes to the OMC due to compaction delay of the mixes upon the addition of lime are illustrated in Fig. 5.21.

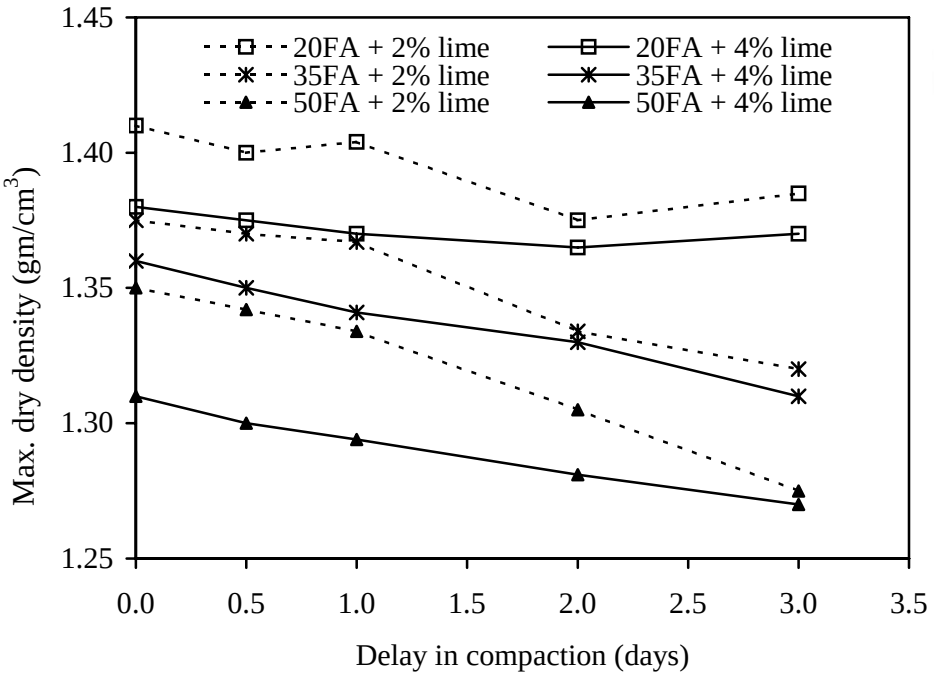


Figure 5.20 Variation of MDD with compaction delay for different soil–fly ash mixes with 2% and 4% lime

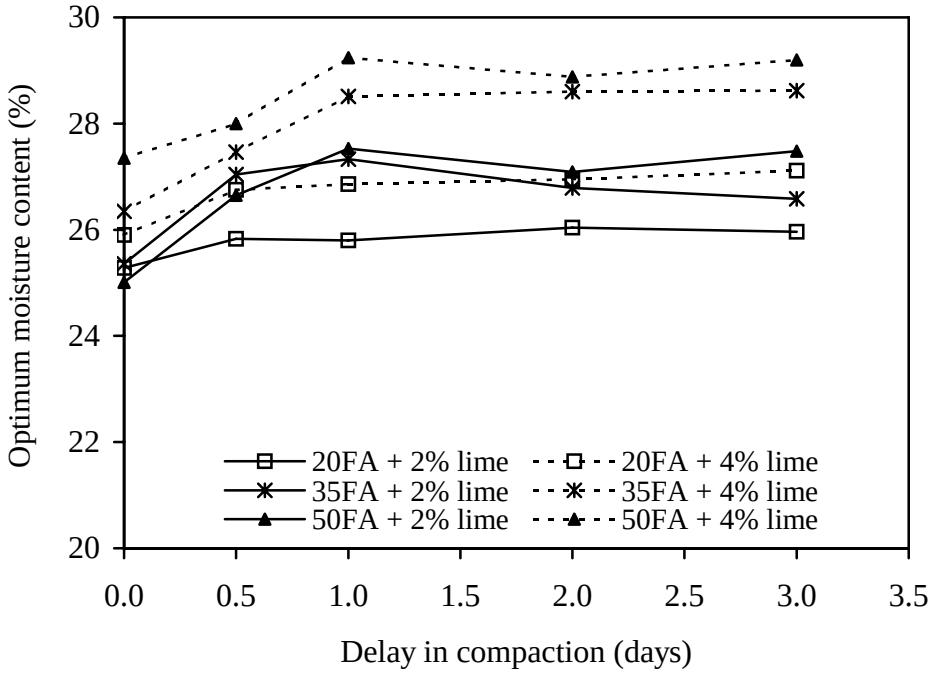


Figure 5.21 Variation of OMC with compaction delay for different soil–fly ash mixes with 2% and 4% lime

The OMC increases marginally for the 20FA mix and noticeably for 35FA and 50FA within 24 hours of delay. Beyond this delay period, very little change was noted. This is possibly on account of increasing requirement of water with the progress of hydration reactions and also due to the subsequent changes in plasticity and gradation of the mixes.

5.5 CONCLUSIONS

1. The MDD of the lateritic residual soil does not get significantly altered when replaced with fly ash up to 20%, but tends to decrease with further increase in fly ash content. In comparison, the OMC initially decreases up to 20% fly ash addition and then gradually increases with higher fly ash content.

2. The addition of lime to the soil-fly ash mixes leads to the reduction of MDD and the increase of OMC. For any lime percentage, the magnitude of change is observed to be proportional to the percentage of fly ash content indicating a marked influence of fly ash.
3. The above influence of lime on the compaction characteristics of the mixes correlates well with the variation of plastic limit that also exhibits greater incremental change up to 2% lime addition and lesser modification at higher lime contents.
4. Re-compaction can considerably change the MDD and OMC of soil-fly ash-lime mixes. The general effect of re-compaction is to increase the MDD and decrease the OMC, and a greater change occurs with higher amount of fly ash. The addition of lime further enhances the effect due to the material texture changing from plastic to friable. The dry density of the soil – fly ash – lime mixes can be increased by as much as 7% through the process of successive re-compaction.
5. For the soil, both MDD and OMC show no variation after 24 hours of compaction delay. Even when only fly ash is added to the soil, the MDD and OMC do not vary appreciably with delay.
6. When lime is added to the soil – fly ash mixes, the MDD progressively decreases with delay. After a compaction delay period of 3 days, comparable values of MDD are obtained for both 2% and 4% lime contents indicating that equivalent short-term modification is achievable with less lime by increasing the contact time between lime, fly ash and soil particles. The OMC increases noticeably within 24 hours of delay after which the increase is gradual.

CHAPTER
SIX

STRESS–STRAIN–STRENGTH CHARACTERISTICS (UNCONFINED COMPRESSION TEST)

6.1 INTRODUCTION

The compressive strength of a soil is an important factor in evaluating the suitability criteria for use as a construction material. Many early workers (Vasquez and Alonso, 1981; Indraratna and Kuganenthira, 1991, Nicholson and Kashyap, 1993, Keshawarz and Dutta, 1993; Srivastava et al., 1996; Consoli et al., 2001) have studied the compressive strength of stabilized and cemented soils using unconfined compression tests and have found the method to be quick and convenient for studying and predicting the strength of these materials. An extensive unconfined compression testing programme was carried out on the experimental soil, fly ash and their various mix proportions with and without admixture in order to examine:

a) the effect of different fractions of fly ash on immediate, short-term and long-term changes in material behaviour; particularly the stress-strain response, stiffness, ductility, and peak strength and also when the mixes are supplemented with different proportions of lime,

b) the change in material behaviour under extreme environmental conditions like flooding or wetting.

c) the effects of time-dependent solidification reactions and consequent changes in densities and on the unconfined compression strength of the mixes for different periods of delay in compaction,

Unlike the conventional process of oven drying, which is commonly adopted to pre-dry soils before specimen preparation, the present study was conducted entirely with air-dried soil, also taking into account, the water required for hydration of lime and loss of moisture due to drying as a result of increase in the plastic limit after lime addition.

In order to understand the effects of soil-structure on the strength of the soil and to study the suitability of the soil for compaction purpose, a few UC tests were conducted also on the undisturbed and remoulded specimens of the soil.

6.2 TEST PROGRAMME

Table 6.1 shows the summary of unconfined compression tests program selected to study the compressive strength of the soil, fly ash and different soil-fly ash-lime mixes. The first regime of tests comprising of five series was chosen to evaluate the immediate, short-term and long-term changes in the soil improved through the addition of fly ash and lime. The test specimens were prepared with static compactive effort after conditioning the hand-mixed specimens for 24 hours. They were then cured for 0, 3, 7, 14, 28, 56 and 480 days and then tested without any prior soaking. The first series was tested on the soil only. In the second series of tests, the properties of the fly ash alone were studied. The next three series of tests were conducted on the soil with the addition of fly ash alone and subsequently with lime. The amount of fly ash added was increased up to a maximum of 50% by weight. The addition of lime was limited to 4%

of the total weight of soil and fly ash due to limited short-term effect beyond this lime content on plasticity characteristics of the mixes (chapter 4) as well as considering economic viability for field application.

In the second regime of tests consisting of another five series, the specimens were prepared similar to that of the first regime except that the specimens were soaked for a further period of 2 days after curing for 0, 3, 28 and 480 days. This was done to simulate extreme environmental conditions involving wetting or flooding.

In the third regime of tests consisting of additional three series, the conditioning period of the un-compacted hand-mixed specimens was varied from 0, 12 hours, 24 hours and 72 hours.

Table 6.1 Summary of unconfined compression tests program

Test series no.	Fly ash used	Amount of lime used	Soaking condition	Conditioning period	Curing period
1	Nil	Nil	Un-soaked	24 hours	Immediate, 3, 7, 14, 28, 56 and 480 days
2	100%	Nil			
3	20%	0, 2%, 3% and 4%			
4	35%	0, 2%, 3% and 4%			
5	50%	0, 2%, 3% and 4%			
6	Nil	Nil	Soaked	24 hours	Immediate, 3,14, 28 and 480 days
7	100%	Nil			
8	20%	0, 2%, 3% and 4%			
9	35%	0, 2%, 3% and 4%			
10	50%	0, 2%, 3% and 4%			
11	20%	0, 2%, and 4%	Un-soaked	0, ½, 1, 2, and 3 days	28 days
12	35%	0, 2%, and 4%			
13	50%	0, 2%, and 4%			

6.3 EXPERIMENTAL DETAILS

6.3.1 Sample Preparation

All specimens for the test programme outlined in table 6.1 were prepared from air-dried soil. Lateritic soil collected for this purpose was first air-dried inside the laboratory until the moisture content reduced to approximately 5%. The air-dried soil was then broken into smaller sizes using rubber mallet with utmost care not to break the soft stones or similar fractions in the soil and then sieved through a 2 mm sieve. Only the fraction passing a 2 mm sieve was considered for the present investigation. The entire quantity of soil required for a particular series of investigation was prepared once at a time and then kept in double packed polythene bags for further investigation.

In general, all compacted test specimens for UC test on fly ash, soil, soil-fly ash and soil-fly ash-lime mixes were prepared at the optimum moisture content and maximum dry unit weight obtained as per IS: 2720 Part (VII) for light compaction test as discussed in chapter 5.

Since only air-dried soil was used, it was necessary to accurately determine the amount of air-dried soil and fly ash required to prepare each specimen at specified dry density. The general expression for the dry weight (W) of a soil-fly ash-lime mixture is given by

$$W = \frac{W_S}{(1+w_S)} + \frac{W_F}{(1+w_f)} + W_L \qquad (6.1)$$

Where, W_S = weight of air-dried soil

W_F = weight of soil replaced with fly ash

W_L = weight of lime added

w_s = water content of the air dried soil

w_f = water content of the dry fly ash as obtained from the factory

While preparing the mixes, first the required amounts of air-dried soil and fly ash were measured and mixed together in dry state. If no lime was added, the dry soil-fly ash mixture was mixed directly with required amount of water. Whenever lime was used, it was uniformly distributed and then water was added. All mixing was done manually and proper care was taken to prepare a homogeneous mix at each stage of mixing.

The amount of water required for a particular mix was calculated on the basis of following factors:

a) the chosen compaction moisture content,

b) water required for slaking of quick lime used, and

c) loss of water due to evaporation during mixing of the specimens.

Quick lime, when used for stabilization, immediately takes up 32% of its own mass of water thus significantly reducing the moisture content of the natural soil (Greaves, 1996). Addition of lime also causes an increase in the plastic limit thus causing an apparent drying. The slaking reaction of quicklime is highly exothermic and the heat generated causes further water loss due to evaporation. Hence, for the present study, whenever lime was used, an additional quantity of water equal to the 32% of the mass of lime and another 1% (based on laboratory trials) of the total water was added to take into account for slaking of lime and loss due to evaporation respectively.

After mixing with water, the specimens were packed in watertight polythene bags and allowed to condition for time periods ranging from 24 hours to 3 days as per the test requirements.

The compacted specimens were of 78 mm in length. After extrusion, the ends of the specimens were trimmed exactly flat to 76 mm length using a split mould. IS

standards for strength tests recommend that the specimen diameter should be at least 8 times greater than the largest particle size. The ratio of specimen diameter to the maximum particle size of the specimens in the present study is 19.

6.3.3 Curing

The unconfined compression tests were carried out after curing the specimens for required curing period. After preparation, the specimens were weighed, wrapped in moisture proof polythene, labelled and left for curing in desiccators at 100% humidity for the specific period of time ranging from 3 to 480 days. In case of tests without soaking, specimens were tested immediately after the completion of the curing period. In cases of soaked unconfined compression tests, specimens were soaked in distilled water after the completion of the specified curing period. For instance, 7 days cured specimens correspond to 7 days of curing in the desiccators followed by additional 2 days soaking in water.

6.3.4 Testing

The unconfined compression tests were carried out as per IS 2720 (Part X) with a constant strain rate of 1.25 mm/min. The load and deformation readings were taken up to the failure of the specimens. Using the test data, stress and strain were calculated. Graph of stress versus strain were plotted and the peak stress was obtained. A minimum of three specimens was tested for each variable and the average of the observed peak stresses was taken as the unconfined compressive strength (UCS).

6.3.5 Tests on Undisturbed and Remoulded Soil

Before carrying out UC test on the soil-fly ash and soil-fly ash-lime mixes, a few UC tests were conducted on undisturbed and remoulded specimens of the soil. Undisturbed specimens were prepared from block specimens collected from the field with the help of a soil lathe. Remoulded specimens were prepared by statically compacting the soil at field density (1.28 gm/cm^3) and field moisture content (28.2%) using the miniature static compaction tool and technique as discussed in section 6.3.2.

6.4 RESULTS AND DISCUSSION

The stress-strain curves of the soil for undisturbed state and different remoulded states are shown in Fig. 6.2. The stress-strain plots of the soil, fly ash and lime mixes for different curing periods are depicted in Figs. 6.3 (a - g), 6.4 (a - g) and 6.5 (a - g). The UCS values and the corresponding failure strains are summarized in Tables 6.2 to 6.6.

6.4.1 Behaviour of Soil and Fly Ash in Different States

From Figure 6.2 the UCS of the undisturbed soil was found to be 63.25 kPa. The remoulded soil at field moisture content and dry density has UCS value of 59.03 kPa, indicating strength sensitivity value of only 1.07. On the other hand, a considerably greater UCS of 520.7 kPa is observed when the soil is compacted at MDD and OMC. The strain at failure also increases with the increase in dry density.

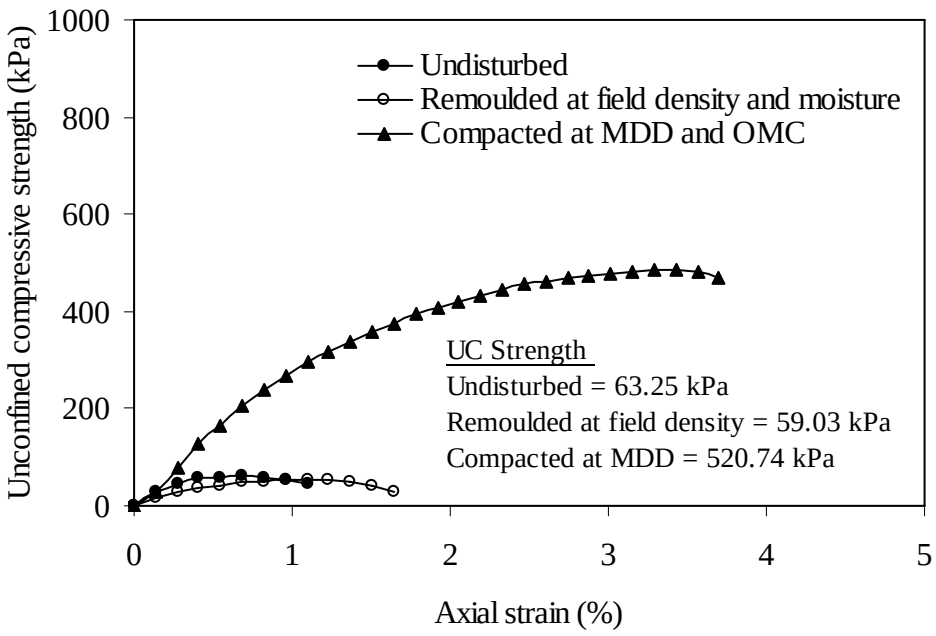


Figure 6.2 Typical curves of unconfined compression tests on the experimental soil.

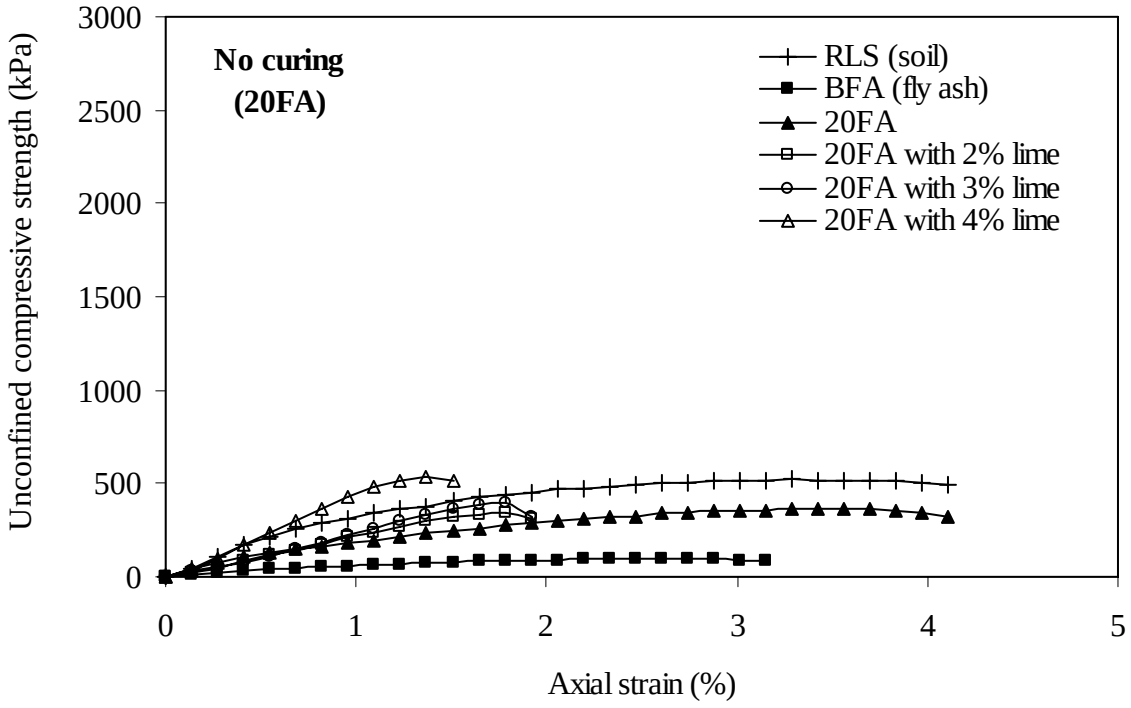


Figure 6.3 (a) 20FA mix - No Curing

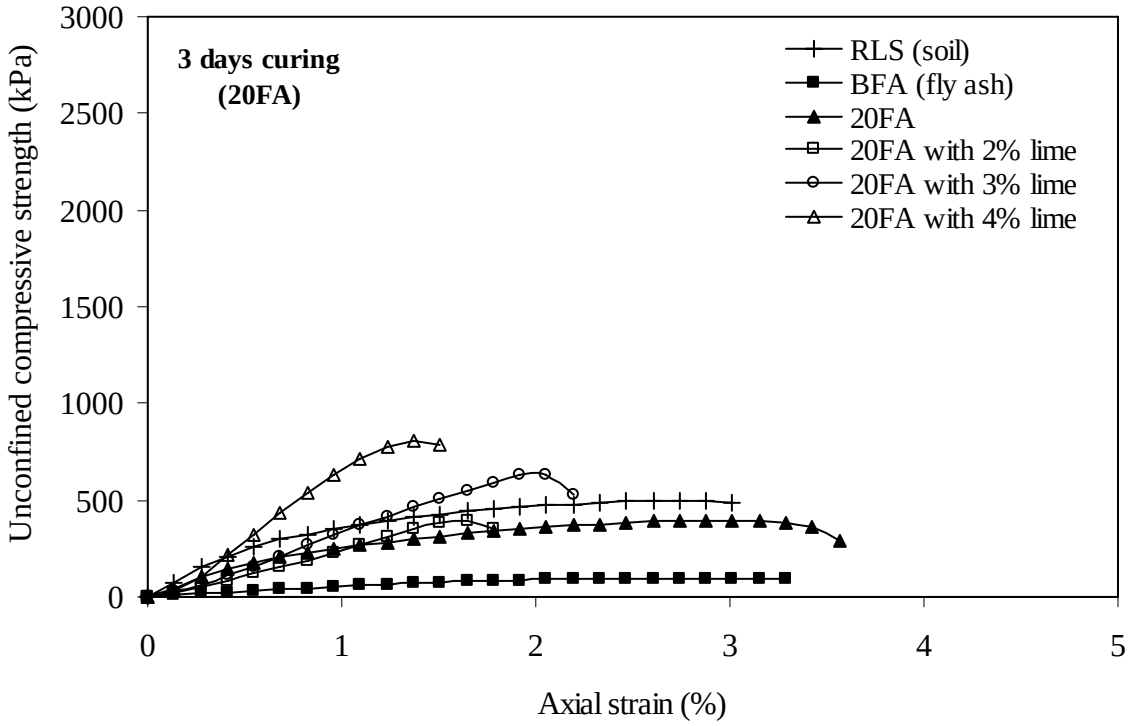


Figure 6.3 (b) 20FA mix –3 days curing

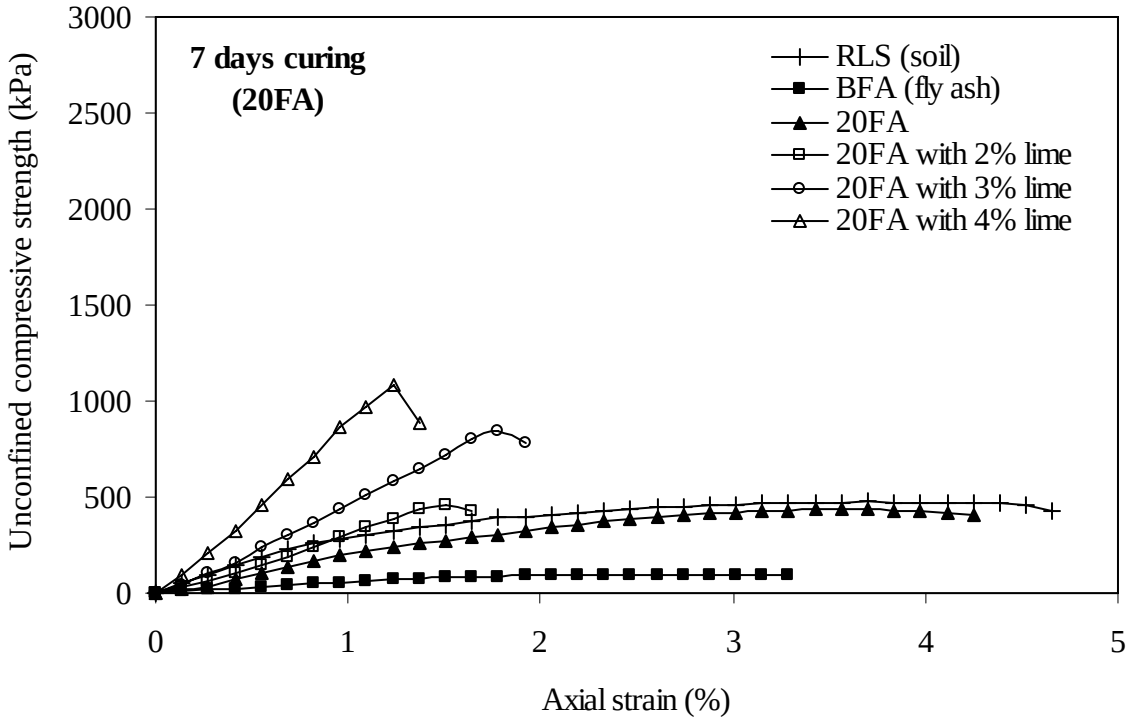


Figure 6.3 (c) 20FA mix –7 days curing

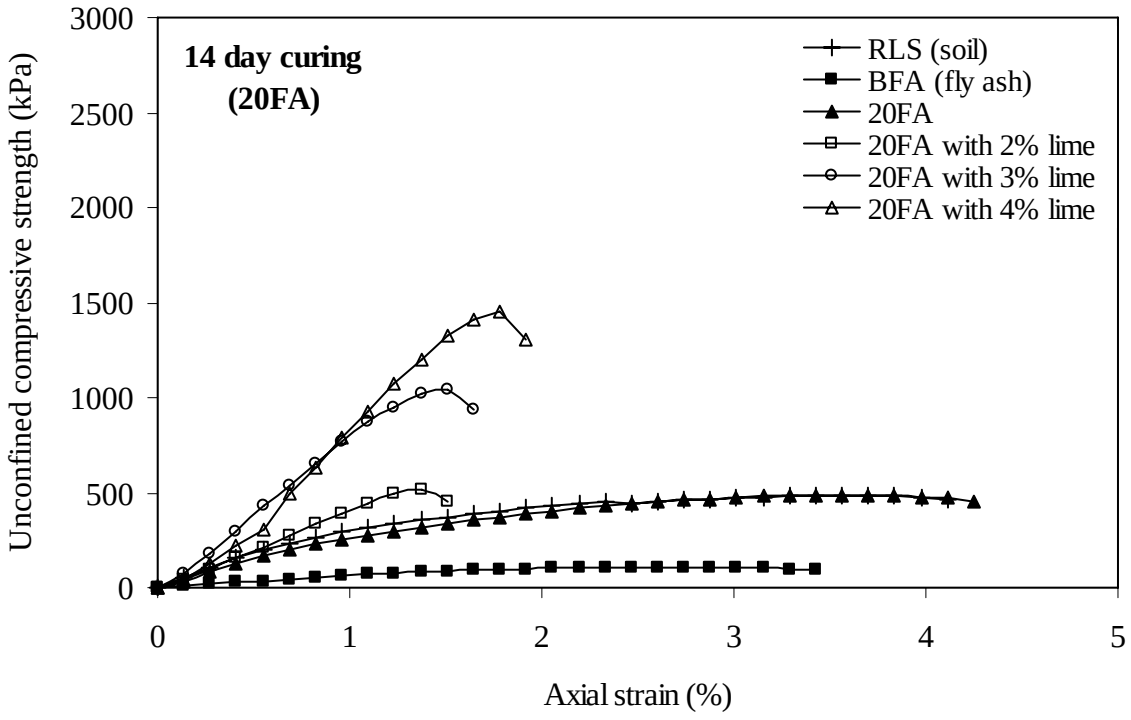


Figure 6.3 (d) 20FA mix –14 days curing

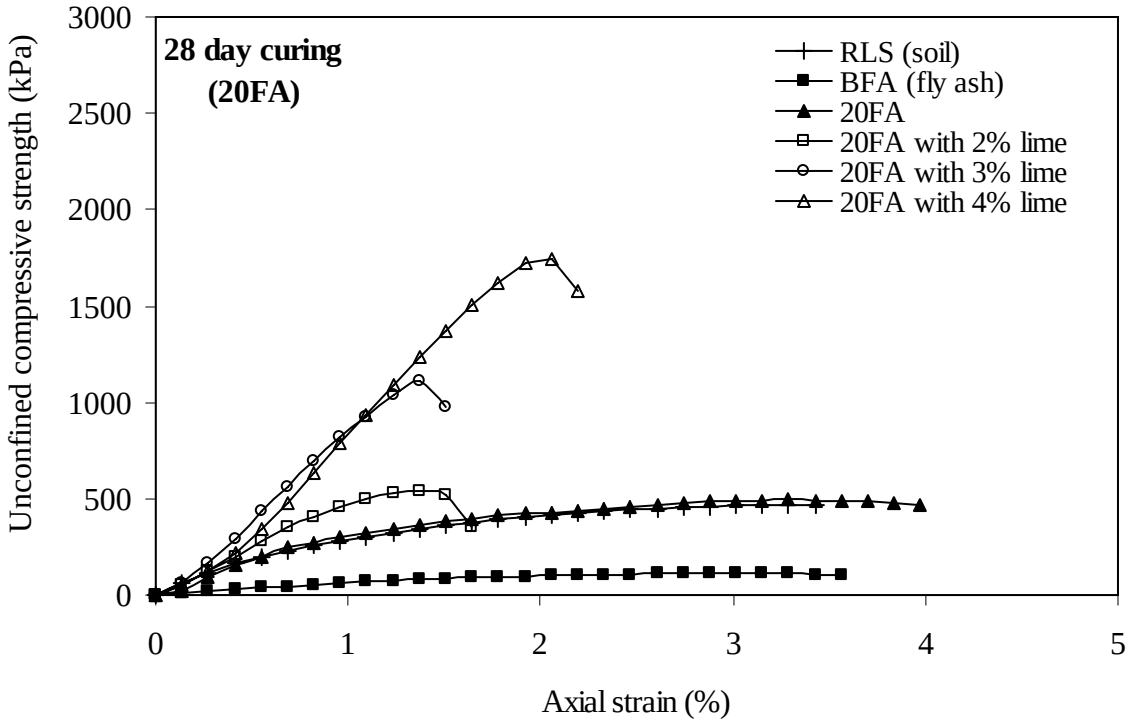


Figure 6.3 (e) 20FA mix -28 days curing

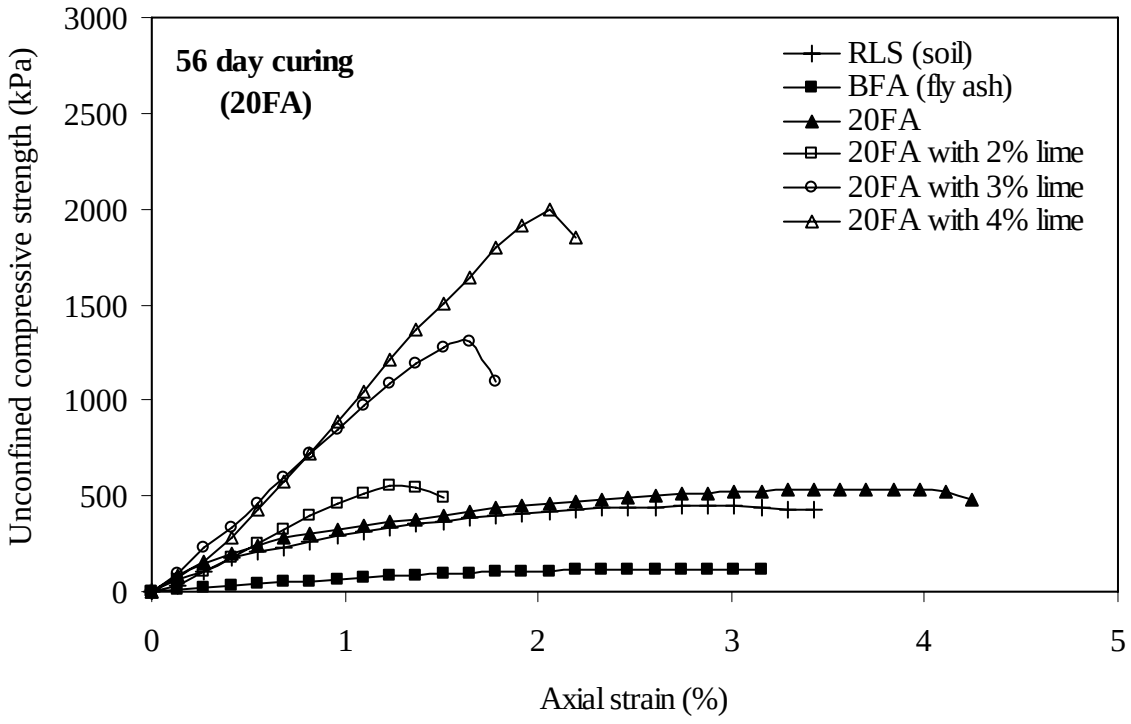


Figure 6.3 (f) 20FA mix - 56 days curing

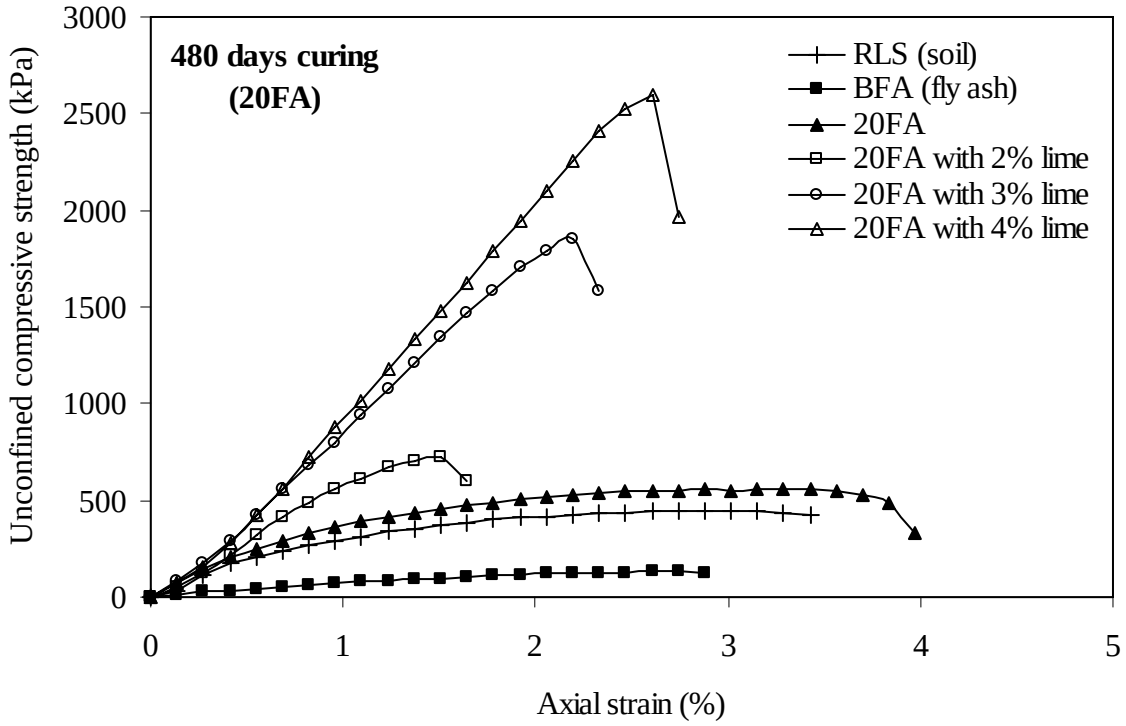


Figure 6.3 (g) 20FA mix – 480 days curing

Figure 6.3 (a-g) Typical stress-strain characteristics of 20FA mix at different period of curing

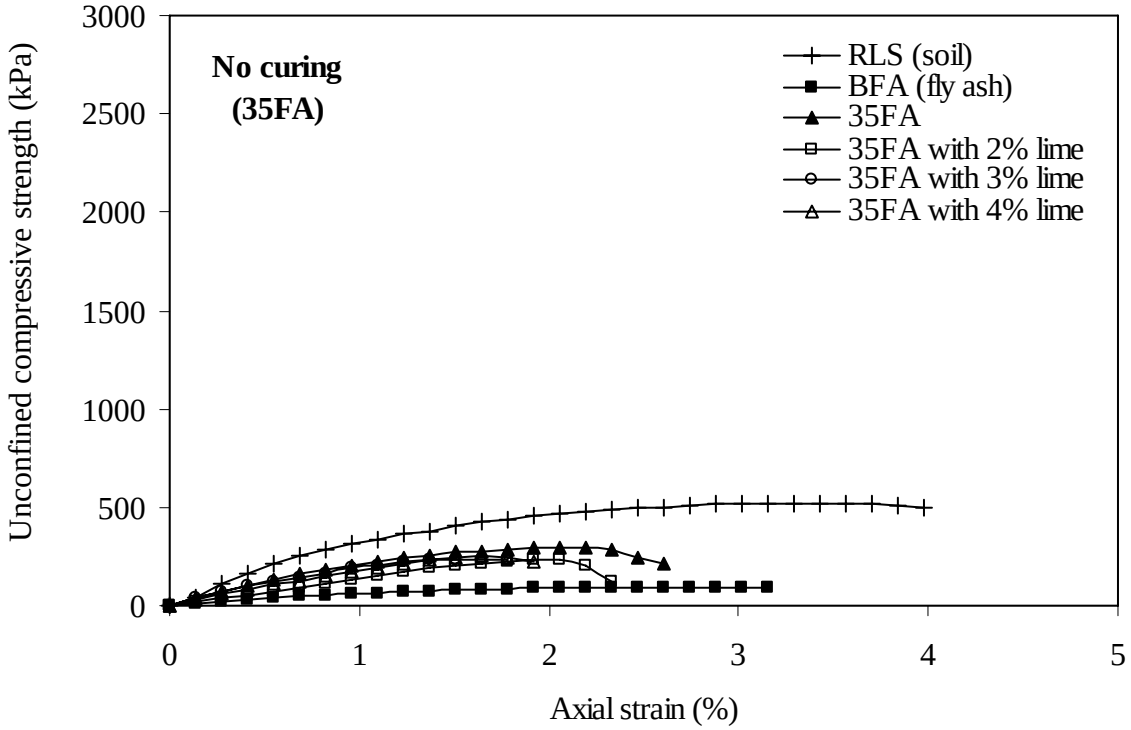


Figure 6.4 (a) 35FA mixes – No curing

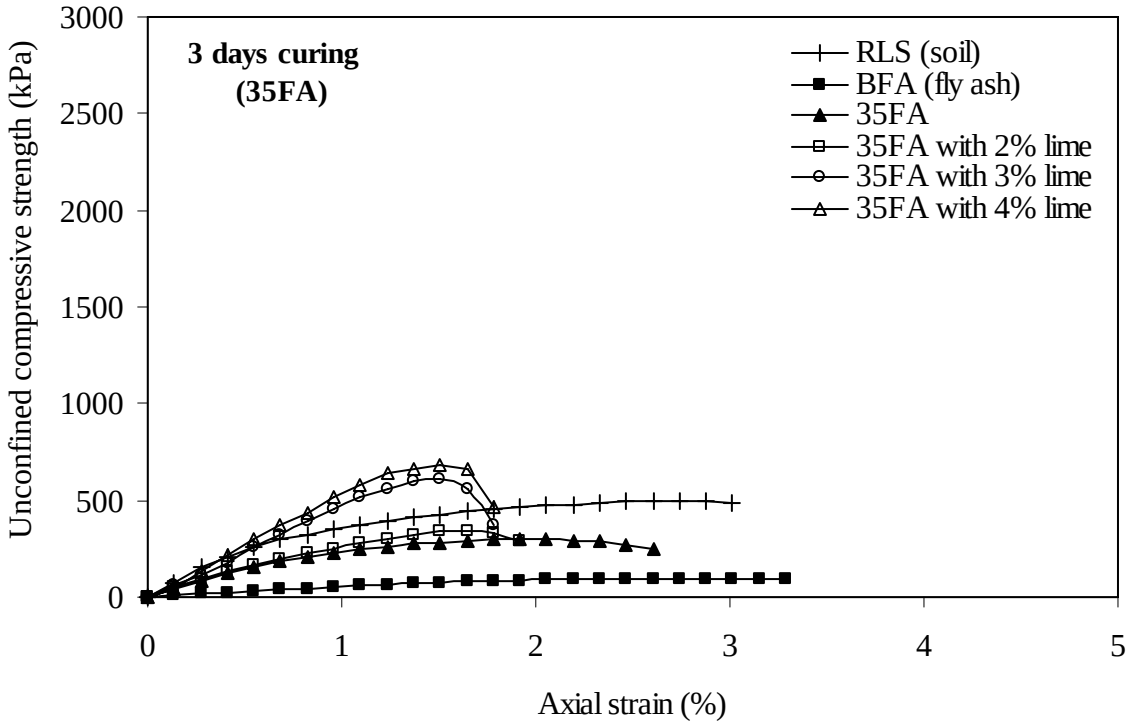


Figure 6.4 (b) 35FA mix – 3 days curing

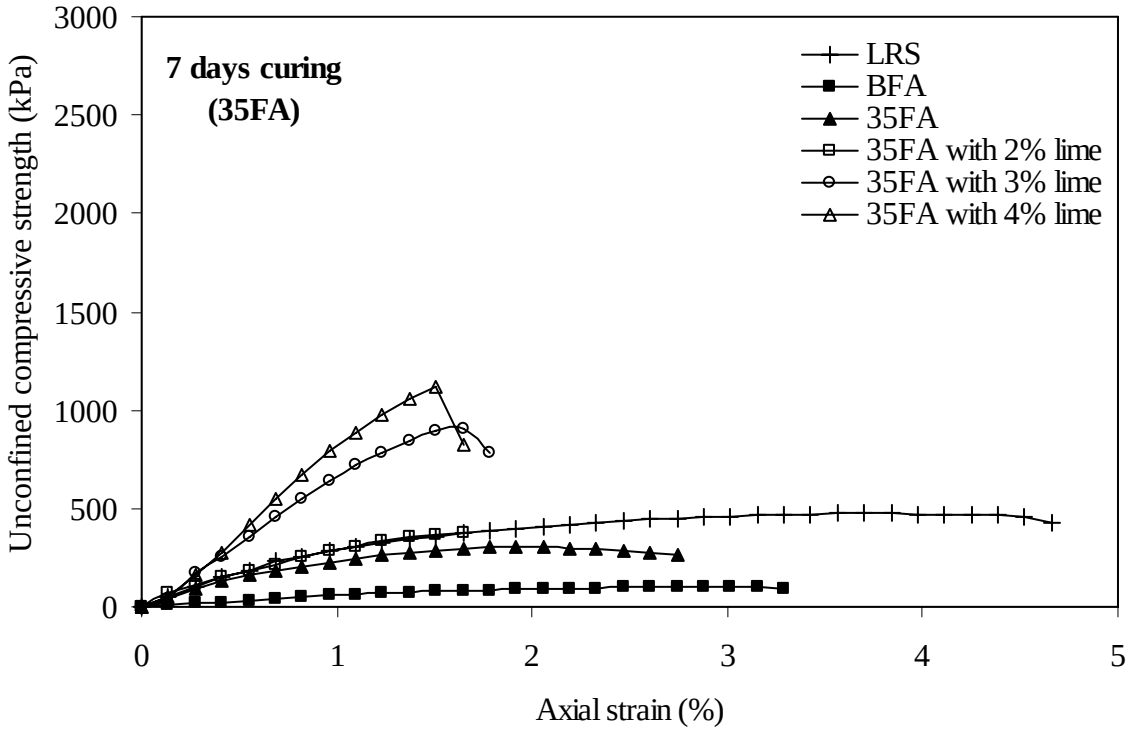


Figure 6.4 (c) 35FA mix – 7 days curing

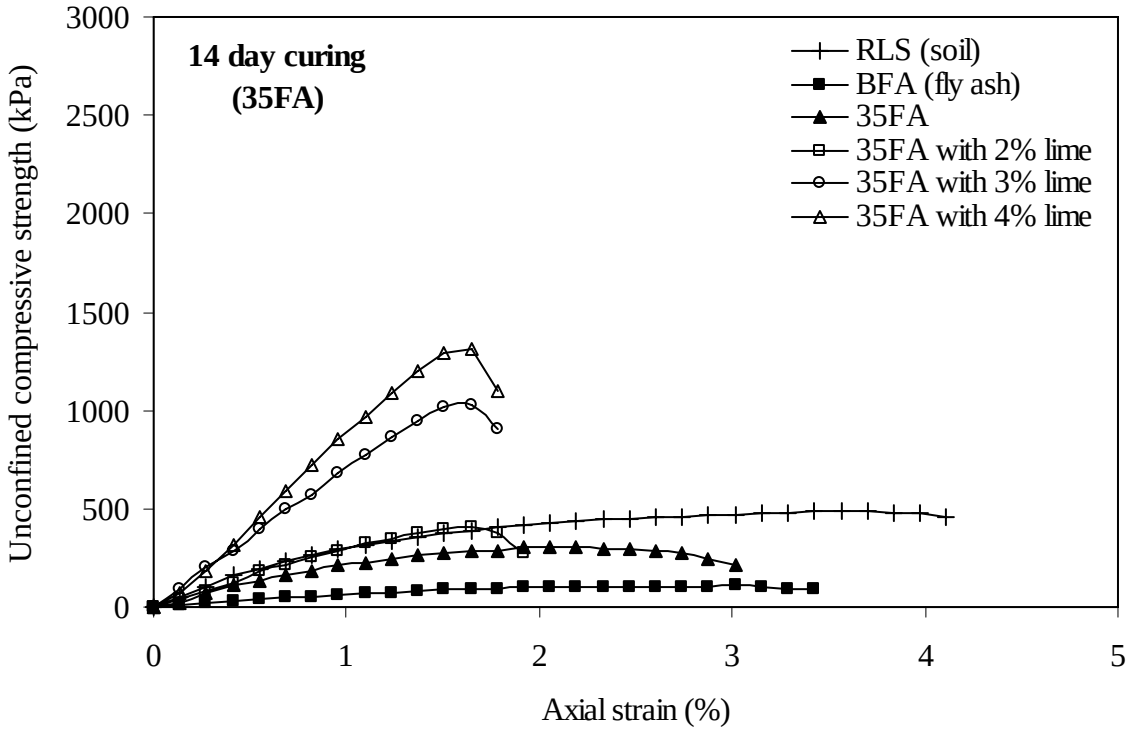


Figure 6.4 (d) 35FA mix – 14 days curing

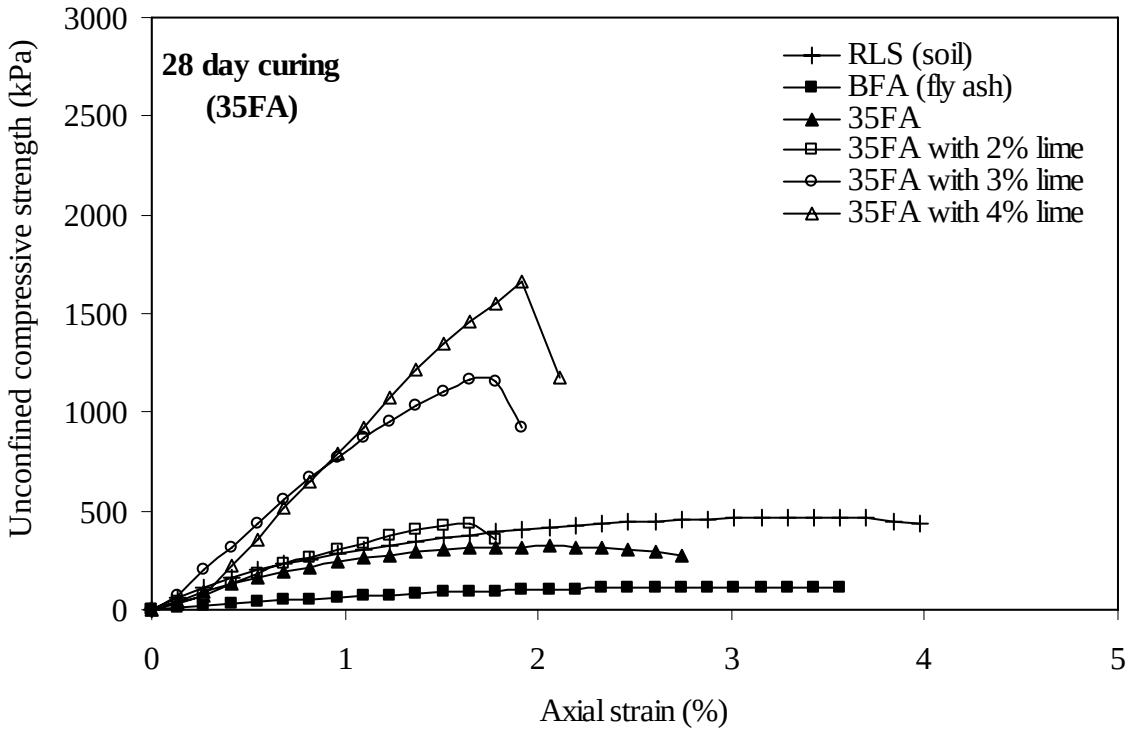


Figure 6.4 (e) 35FA mix – 28 days curing

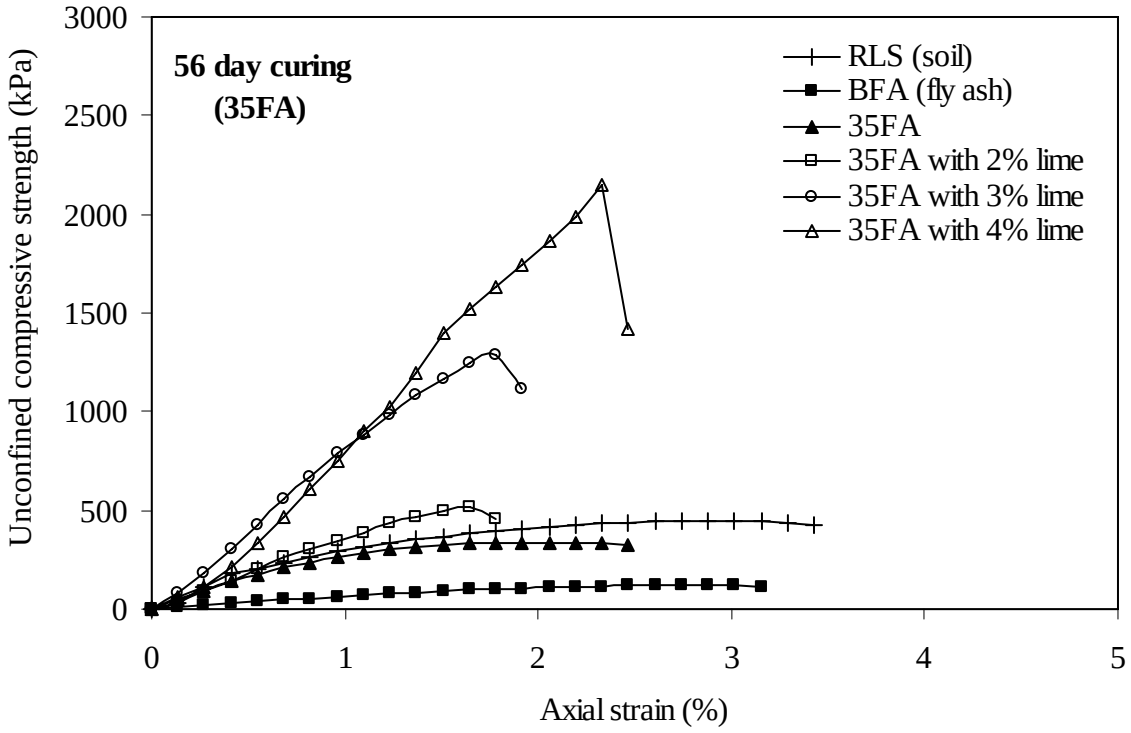


Figure 6.4 (f) 35FA mix – 56 days curing

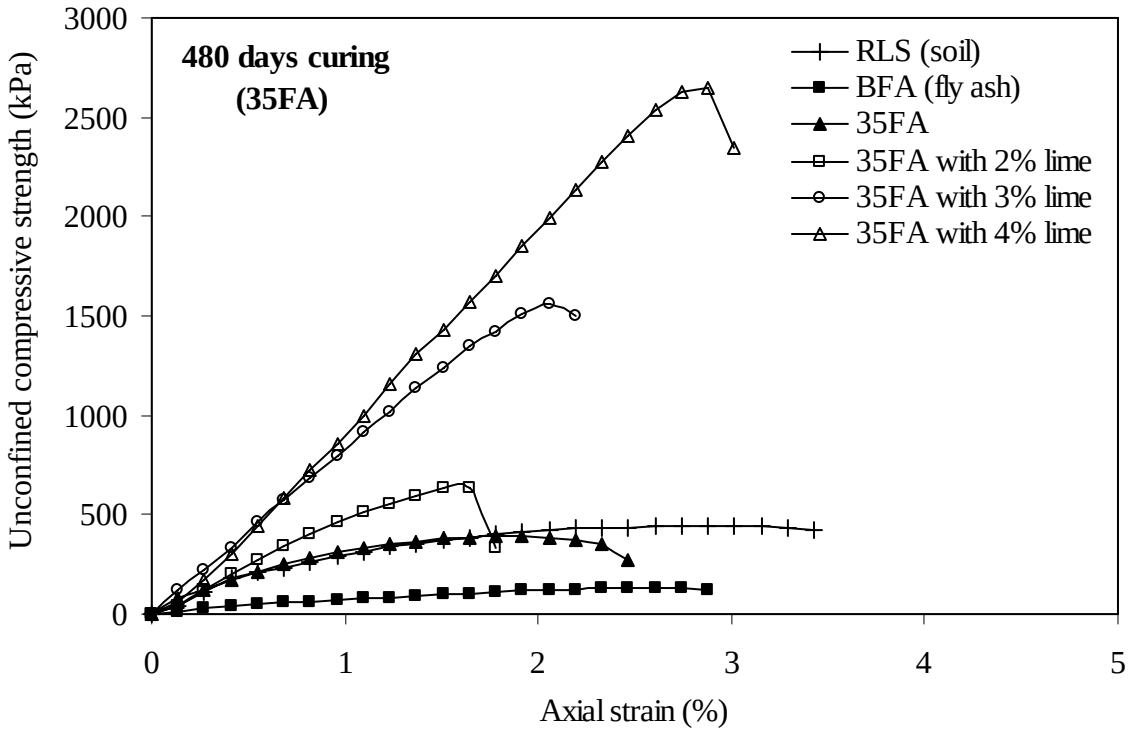


Figure 6.4 (g) 35FA mix – 480 days curing

Figure 6.4 (a-g) Typical stress-strain characteristics of 35FA mix at different period of curing

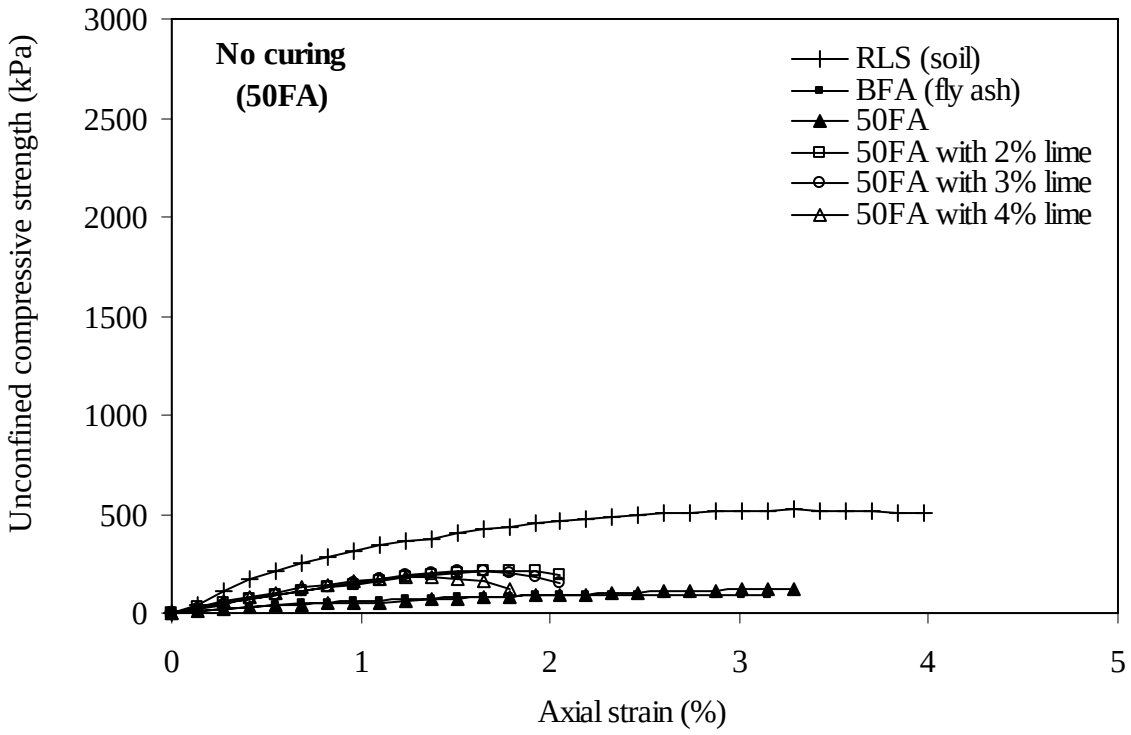


Figure 6.5 (a) 50FA mixes – No curing

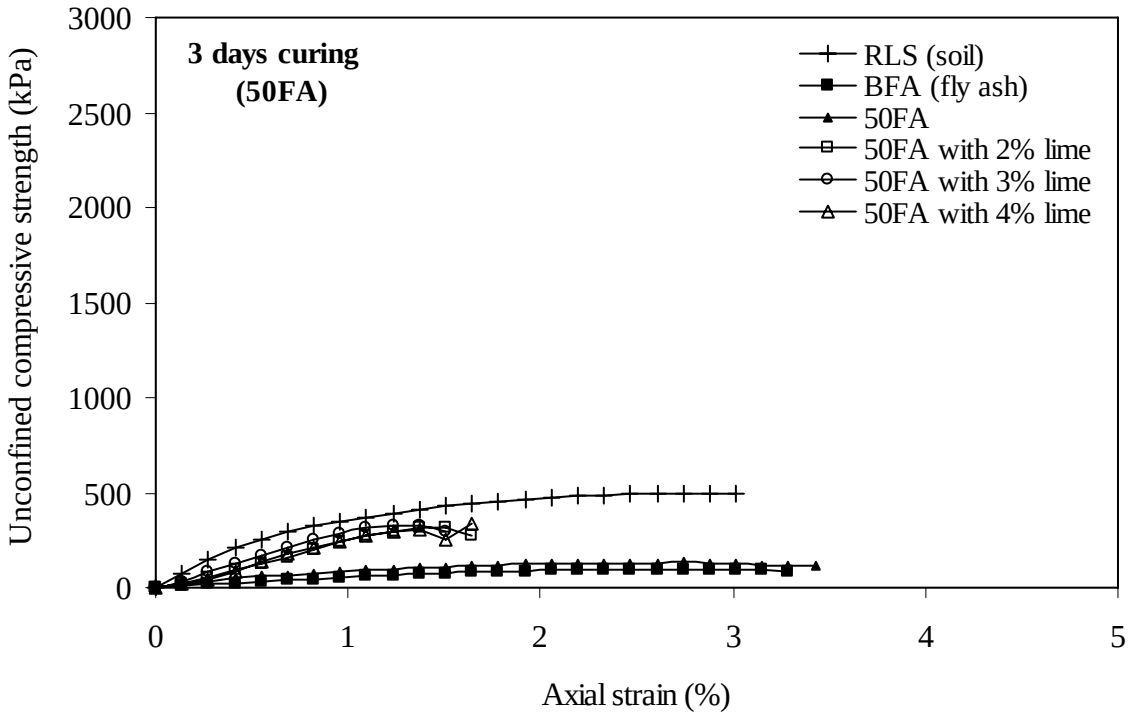


Figure 6.5 (b) 50FA mix – 3 days curing

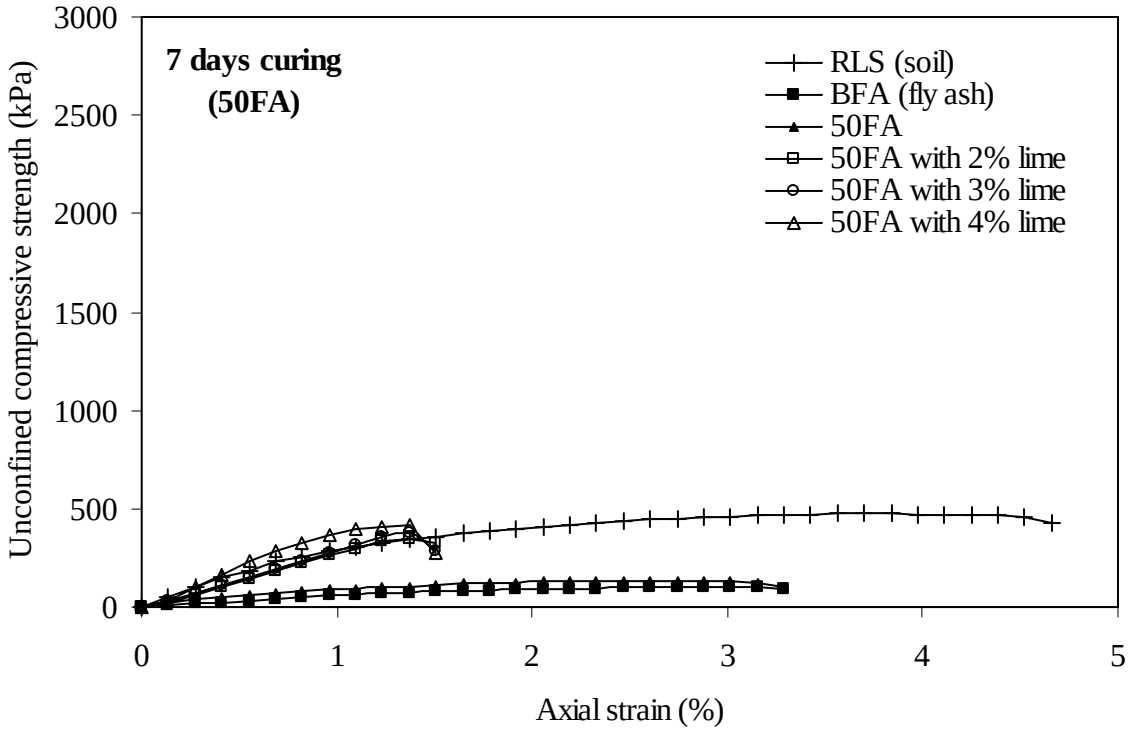


Figure 6.5 (c) 50FA mix – 7 days curing

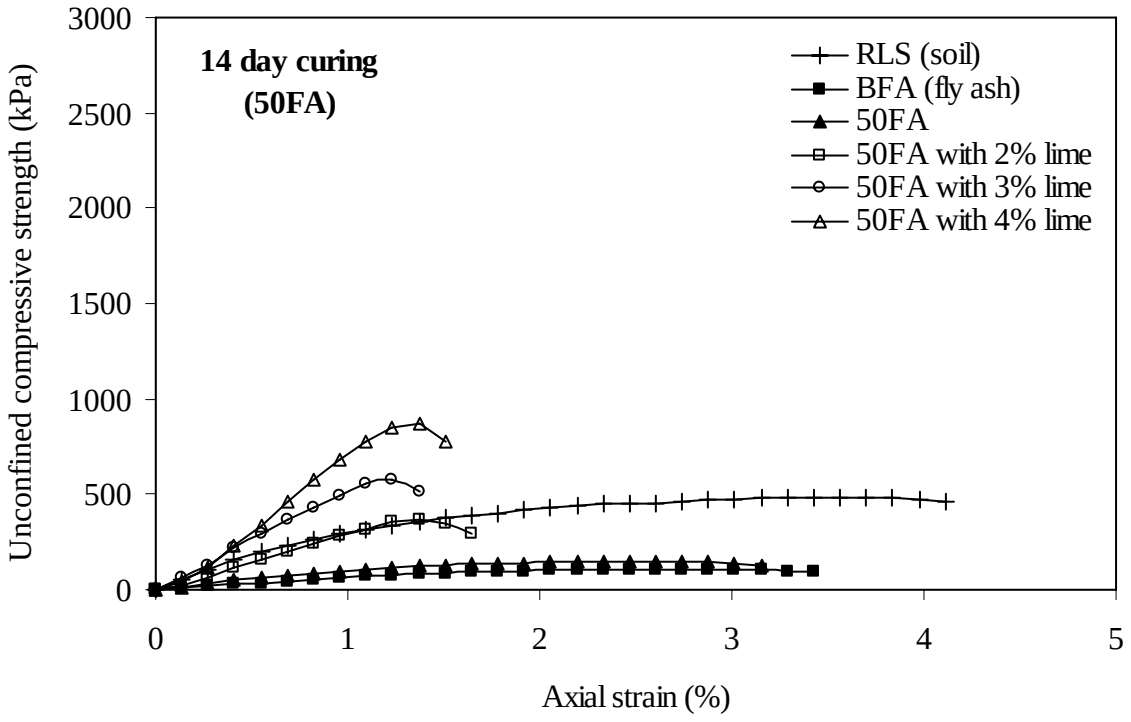


Figure 6.5 (d) 50FA mix – 14 days curing

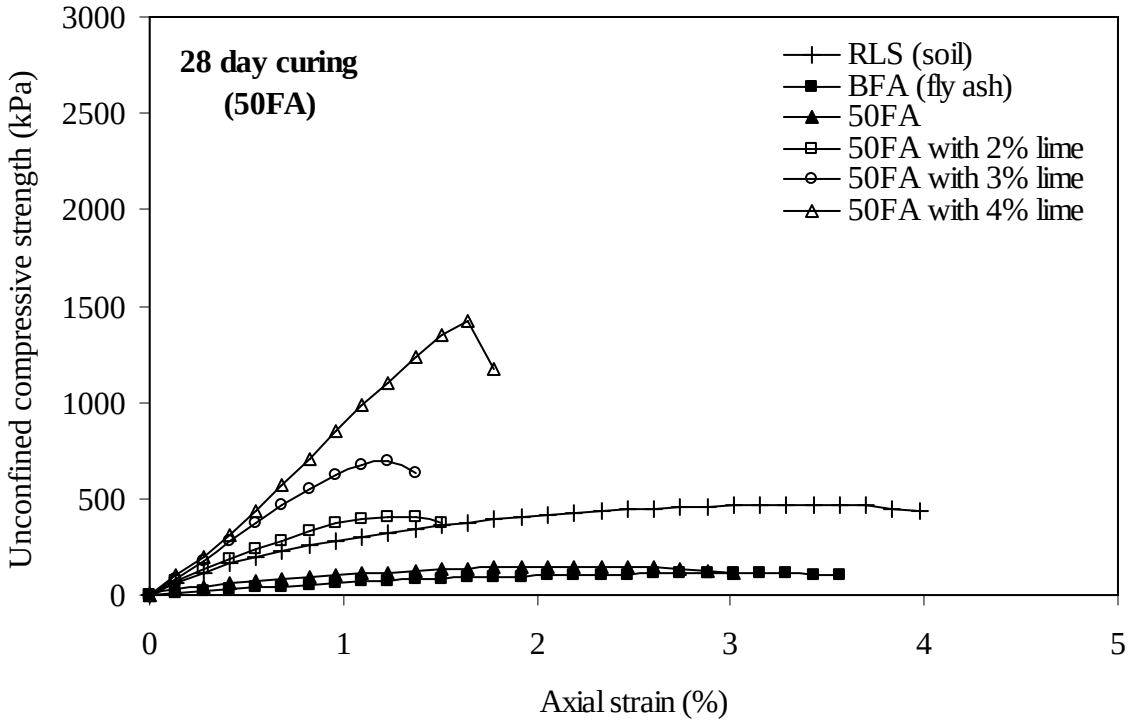


Figure 6.5 (e) 50FA mix - 28 days curing

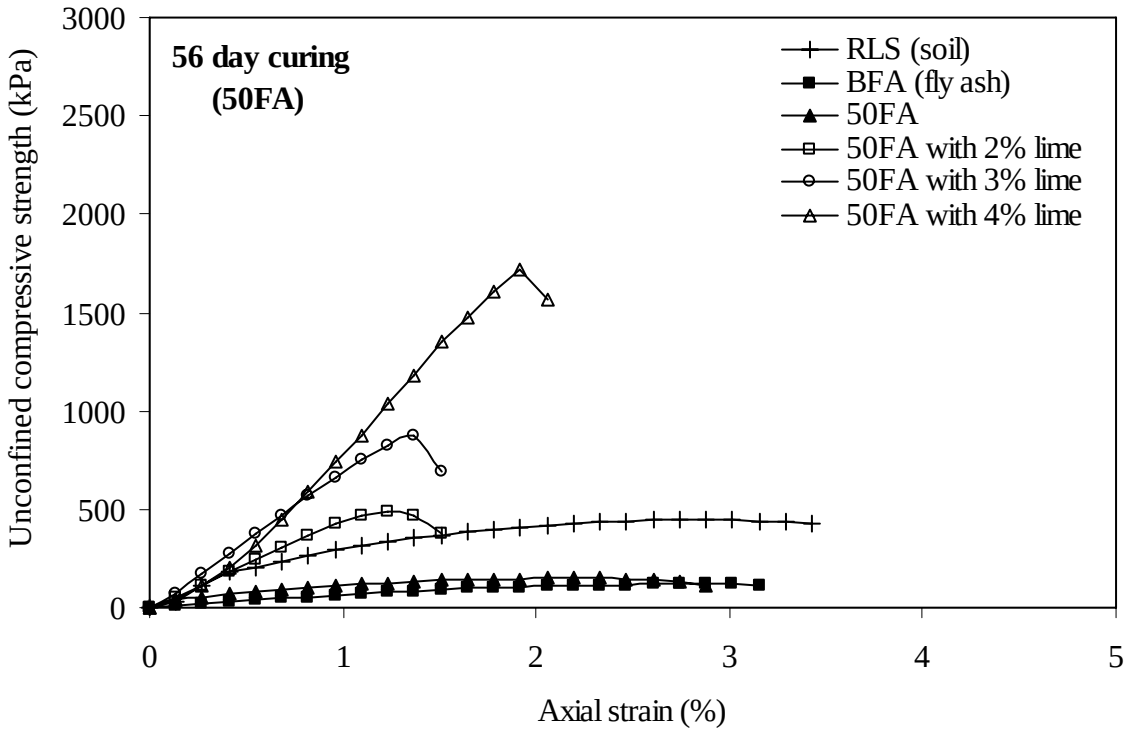


Figure 6.5 (f) 50FA mix – 56 days curing

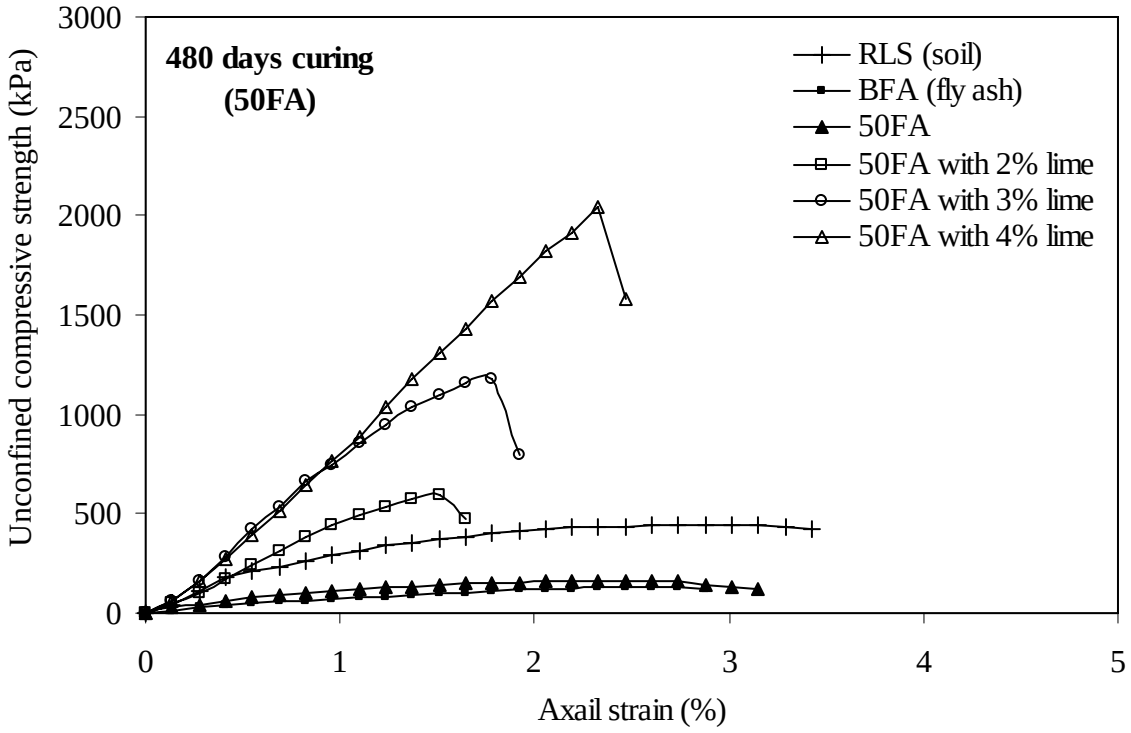


Figure 6.5 (g) 50FA mix – 480 days curing

Figure 6.5 (a-g) Typical stress-strain characteristics of 50FA mix at different period of curing

Table 6.2 Unconfined compressive strength and failure strains for experimental soil and fly ash compacted at MDD

Curing time (Days)	Experimental soil		Experimental fly ash	
	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}
0	520.7	3.29	94.1	2.33
3	495.2	2.60	96.1	2.74
7	572.7	3.70	98.1	2.60
14	485.4	3.43	105.9	2.60
28	467.8	3.15	111.8	2.74
56	447.2	2.74	120.6	2.88
480	447.2	2.74	132.4	2.60

σ_{ucf} = UC strength (kPa) ε_{ucf} = Strain at failure (%)

Table 6.3 Unconfined compressive strength and failure strains for various soil fly ash mixes with no lime addition

Curing time (Days)	20FA mix		35FA mix		50FA mix	
	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}
0	360.9	3.06	296.2	2.06	129.4	3.61
3	391.3	2.83	303.0	1.78	133.4	2.70
7	433.5	3.56	295.2	1.85	135.3	2.42
14	488.4	3.34	304.0	2.15	141.2	2.38
28	494.3	3.20	324.6	1.91	148.1	2.19
56	533.5	3.29	338.3	1.91	153.0	2.19
480	557.0	2.88	389.3	1.78	165.7	2.33

σ_{ucf} = UC strength (kPa) ε_{ucf} = Strain at failure (%)

Table 6.4 Unconfined compressive strength and failure strains for 20FA mix with various lime contents

Curing time (Days)	With 2% lime		With 3% lime		With 4% lime	
	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}
0	335.4	1.83	402.1	1.87	528.6	1.33
3	380.5	1.65	643.3	2.12	806.1	1.37
7	462.9	1.51	853.2	1.83	1082.7	1.51
14	499.2	1.23	1046.4	1.51	1401.4	1.65
28	523.7	1.33	1114.0	1.46	1757.4	2.10
56	539.4	1.23	1240.5	1.51	1984.9	2.06
480	719.8	1.51	1856.4	2.20	2596.8	2.60

σ_{ucf} = UC strength (kPa) ε_{ucf} = Strain at failure (%)

Table 6.5 Unconfined compressive strength and failure strains for 35FA mix with various lime contents

Curing time (Days)	With 2% lime		With 3% lime		With 4% lime	
	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}
0	243.2	1.83	243.2	1.55	257.9	1.60
3	325.6	1.71	607.0	1.46	678.6	1.51
7	376.6	1.65	905.2	1.65	954.2	1.51
14	408.0	1.65	1038.5	1.65	1301.3	1.65
28	430.5	1.60	1164.1	1.65	1656.4	1.92
56	500.1	1.65	1287.6	1.78	2145.7	2.33
480	635.5	1.65	1563.2	2.06	2651.7	2.88

σ_{ucf} = UC strength (kPa) ε_{ucf} = Strain at failure (%)

Table 6.6 Unconfined compressive strength and failure strains for 50FA mix with various lime contents

Curing time (Days)	With 2% lime		With 3% lime		With 4% lime	
	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}
0	212.8	1.65	211.8	1.60	176.5	1.37
3	318.7	1.33	321.7	1.33	303.0	1.37
7	346.2	1.33	372.7	1.33	384.4	1.19
14	369.7	1.28	574.7	1.23	882.6	1.33
28	412.9	1.28	690.4	1.23	1445.5	1.65
56	481.5	1.28	867.9	1.42	1720.1	1.92
480	593.3	1.51	1178.8	1.78	2047.6	2.33

σ_{ucf} = UC strength (kPa) ε_{ucf} = Strain at failure (%)

The improvement in the shear strength property by compacting to a certain density is an important aspect in determining its suitability for fill application. The soil shows considerable improvement in its UCS when compacted to MDD, which reflects its suitability for fill application. Many earlier workers in India have reported adequate increase in the shear strength of compacted lateritic soils (Vinod and Monoj, 1997; Basavarajaiah et al., 1981; Chandrasekhara Rao and Rajeevalochanam, 1989). However, the change in shear strength of the soil upon saturation will also be an important criterion in determining its suitability as fill material, as earlier investigators (Lee and Coop 1995,) have reported considerable reduction in shear strength due to flooding of the soil because of apparent loss of the cohesion that develop due to wet and dry cycles prevalent in areas dominated by residual lateritic soils.

On the other hand, the fly ash exhibits a much lower UCS (94.1 kPa) compared to that of the soil, but the value marginally increases up to 132.4 kPa at the end of 480 days of curing with strain at failure ranging from 2.3 % to 2.7% (Table 6.2).

6.4.2 Stress-Strain Behaviour of Soil-Fly Ash Mixes

Plot (a) of Figs. 6.3 to 6.5 shows stress-strain curves for specimens without curing. It can be seen that when no curing was done, the stress continues to increase with strain and no peak value is observed for both the soil and 20FA mix. The same observations regarding the 20FA mix can be made from Plots (b) to (g) of Figs. 6.3 for 3, 7, 14, 28, 56 and 480 days curing periods. At low curing periods, the stress-strain curve for the 20FA mix plots below that of the soil. As the curing period increases, the stress-strain curve for the 20FA mix approaches that of the soil, and at a curing period of 28 days, they almost overlap. On further curing, the plots for the 20FA mix lie above that of the soil.

In the case of 35FA mixes, peak stress value is observed for all curing periods (Fig. 6.4). The stress-strain characteristics of the 50FA mixes are very much similar to that of the fly ash alone for all curing periods (Figs. 6.5), and for both, no peak stress value was noticed.

The stress-strain curves for different mixes (20FA, 35FA and 50FA) for different curing periods are combined separately in Figure 6.6 for comparison. It can be readily observed from the figure that the process of curing does not significantly alter the overall stress-strain characteristics of the mixes, but the unconfined strength somewhat increases with curing period without much change in the stiffness and brittleness of the material. The failure strains remain within a narrow range varying between 2.83% to 3.56%, 1.60% to 1.83% and 2.19% to 3.61% for the three mixes respectively (Table 6.3).

6.4.3 Stress-Strain Behaviour of Soil-Fly Ash-Lime Mixes

Addition of lime to the soil-fly ash mixes causes a fundamental change in the stress-strain characteristics of the treated material. It can be readily observed from Figs. 6.3 to Figs. 6.5 that the overall behaviour is significantly influenced by the addition of lime to the soil-fly ash mixes and subsequent curing. Distinct changes are noticed in stiffness, brittleness and peak strength as a consequence of separate or combined effects of lime and fly ash, and as curing progresses, remarkable change in their characteristics takes place.

With the increase in lime content from 2% to 3%, the initial stiffness of the lime treated mixes is found to increase at all stages of curing (Fig. 6.3 to 6.5). But, when the lime content is increased further from 3% to 4%, the initial stiffness of the specimens increases only during the initial stages of curing up to 7 days. Beyond this curing period, the initial stiffness of the material remains more or less unchanged but both the failure strain and peak strength keep on increasing.

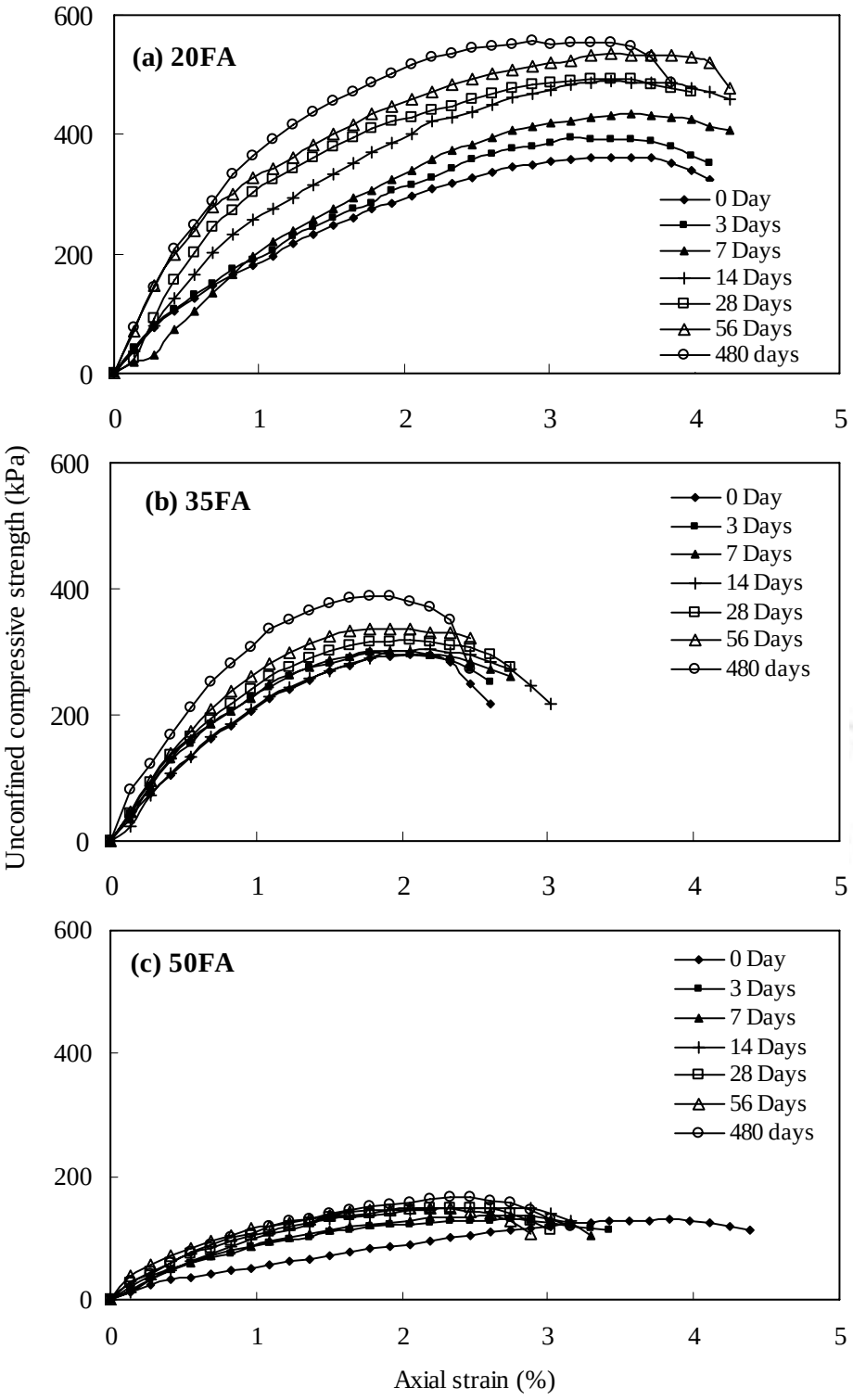


Figure 6.6 Typical stress-strain plots of different soil fly ash mixes without lime at various curing periods (a) 20FA mix (b) 35FA mix and (c) 50FA mix.

It is important to observe that the ductile nature of the lateritic soil as well as soil-fly ash mixes gradually changes to brittle nature with the addition of lime and with progress of subsequent curing. The brittleness increases with lime content and curing duration. The lime treated specimens during later stages of curing exhibit a highly brittle behaviour marked by rapid build-up of stress and sudden collapse after attaining the peak strength.

The failure strain of the mixes varies with lime content as well as with curing time. At the initial stages of curing upto 7 days, addition of lime has a reducing effect on the failure strain of the mixes. But at later stages of curing, the failure strain increases again and the increase is more for higher lime content (Fig. 6.3 to 6.5).

A comparison of the stress-strain characteristics of the soil-fly ash-lime mixes with the progress of curing period is presented in Figs. 6.7 to 6.9. It can be observed from the figures that for a given lime content, the stress-strain response of the mixes with different fly ash contents vary in an identical manner with the progress of curing. The failure strain, after an initial decline, tends to increase with the aging of the specimens. Addition of 2% lime to the mixes results in a continuous increase in stiffness of the material with curing and also gradually leads to higher peak strength without significant variation in failure strain. On further addition of lime, the stiffness of the specimens gradually ceases to change towards the later stages of curing. At 4% lime content, as curing progresses, the peak strength of the specimens shows remarkable increase with concomitant increase in failure strain without further change in the stiffness of the specimens. Particularly, the initial stiffness of the specimens towards the later stages of curing is practically independent of the fly ash content (Fig. 6.10 and Fig. 6.11).

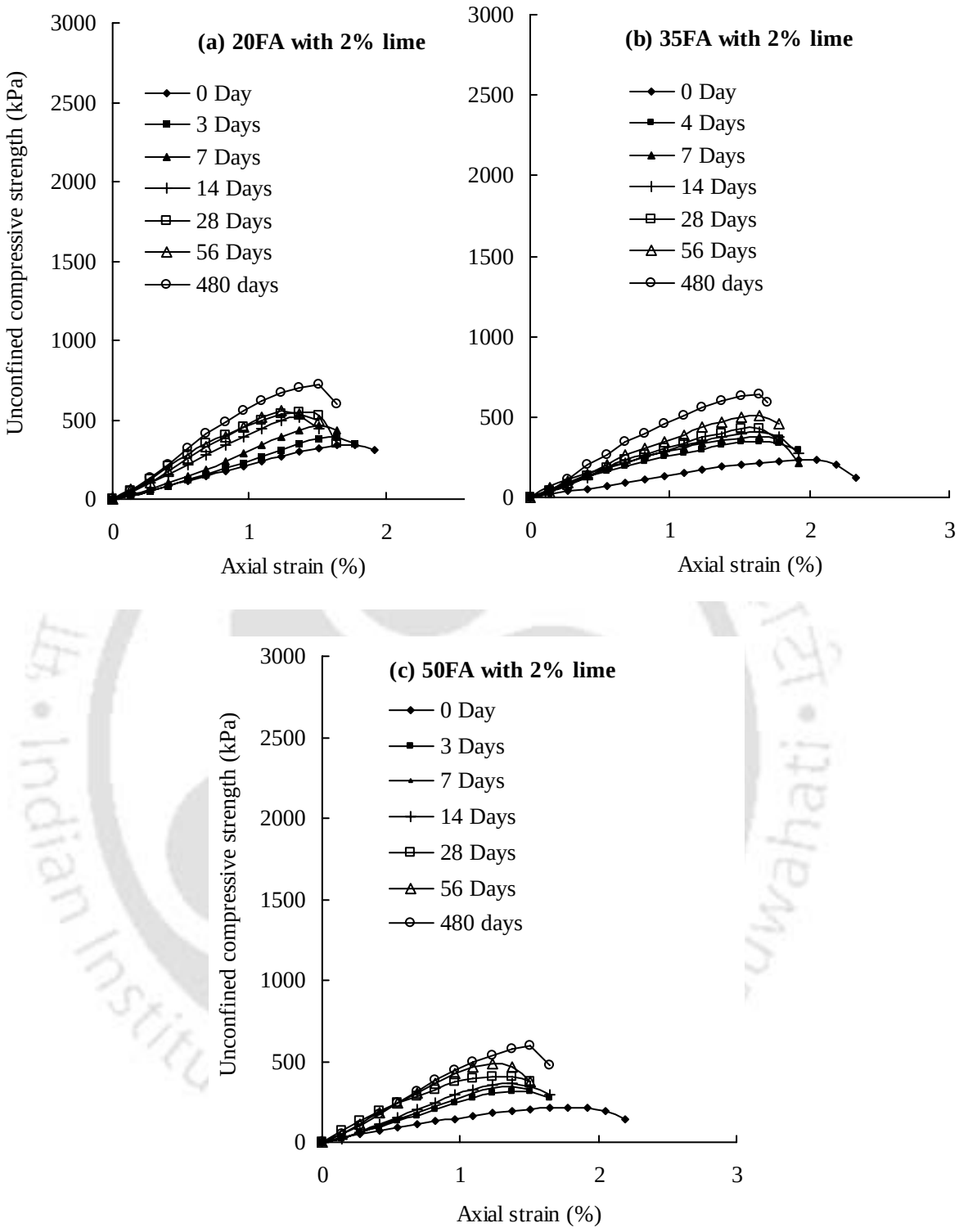


Figure 6.7 Typical stress-strain plots of various mixes with 2% lime after different periods of curing for (a) 20FA mixes (b) 35FA mixes (c) 50FA mixes

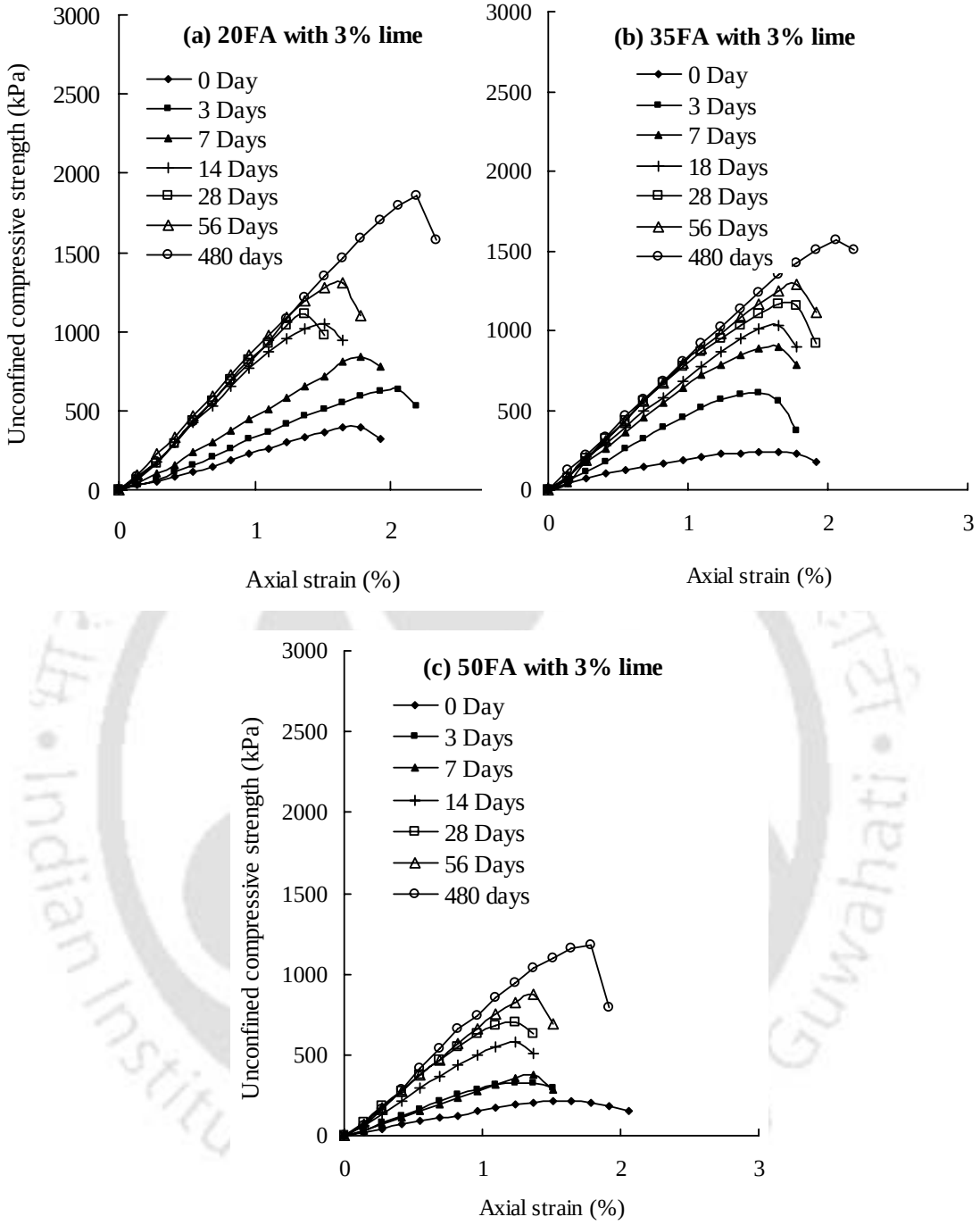


Figure 6.8 Typical stress-strain plots of various mixes with 3% lime after different periods of curing for (a) 20FA mixes (b) 35FA mixes (c) 50FA mixes

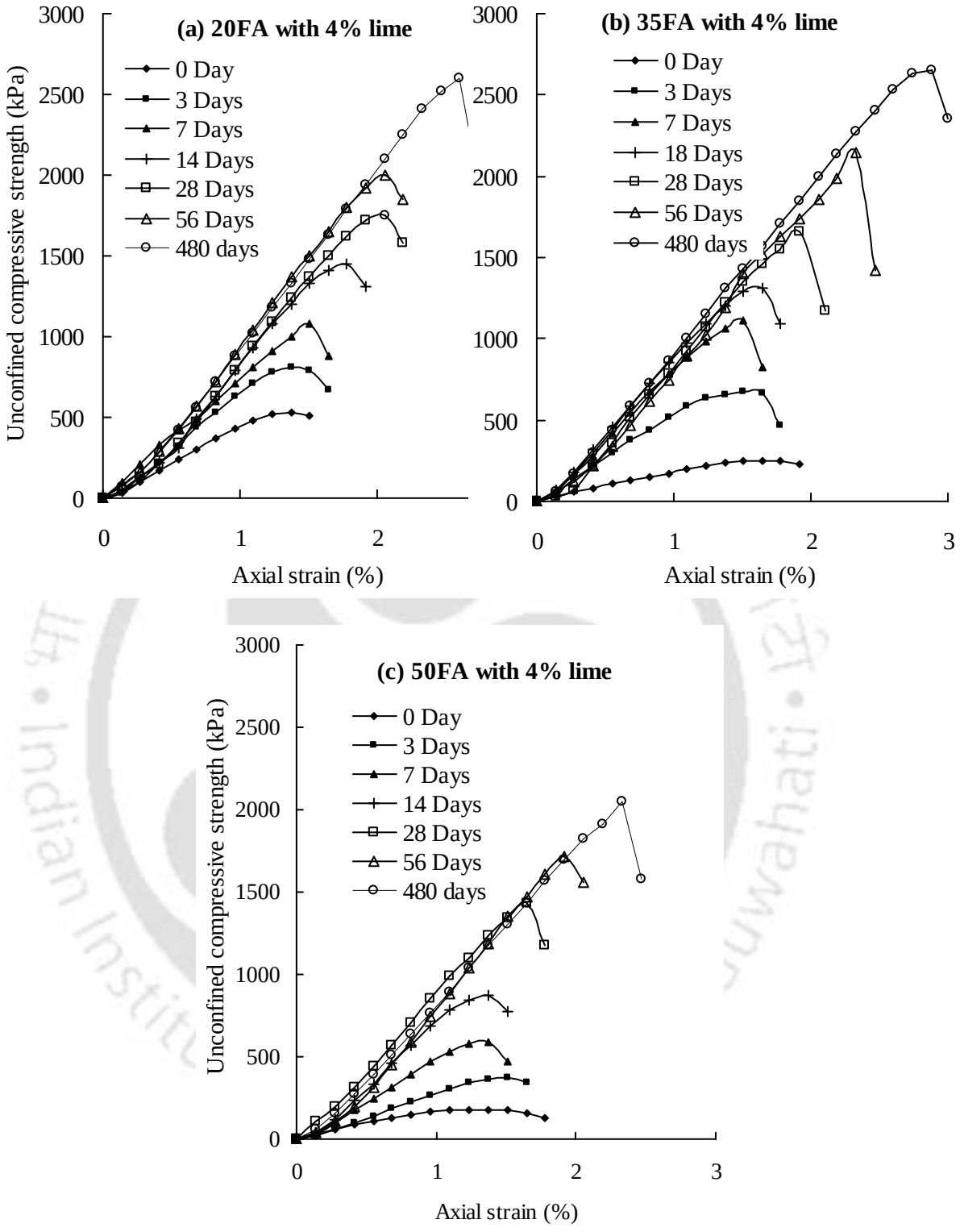


Figure 6.9 Typical stress-strain plots of various mixes with 4% lime after different periods of curing for (a) 20FA mixes (b) 35FA mixes (c) 50FA mixes

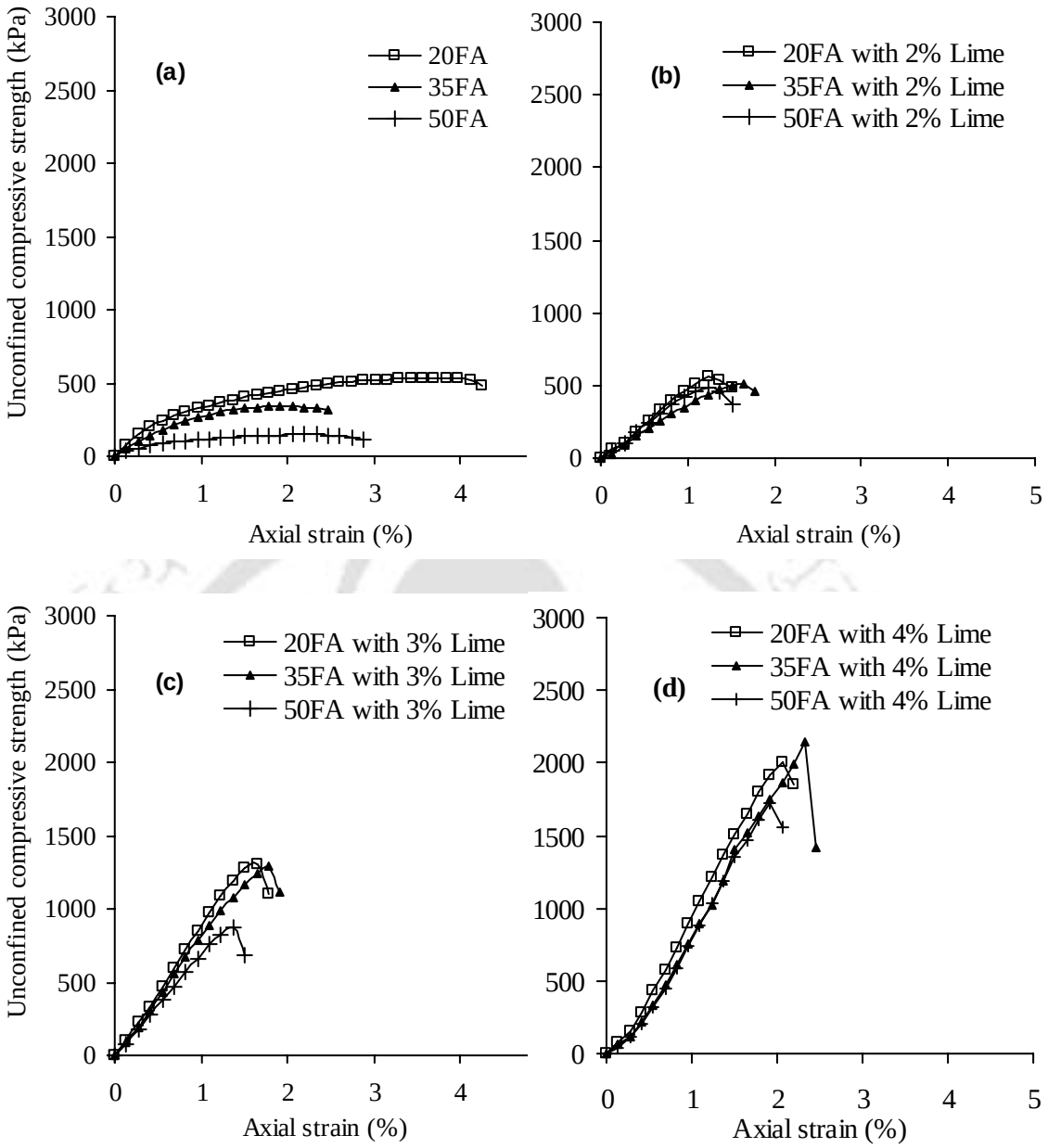


Figure 6.10 Typical stress-strain curves of various soil-fly ash mixes with (a) no lime (b) 2% lime (c) 3% lime (d) 4% lime, tested after 56 days of curing

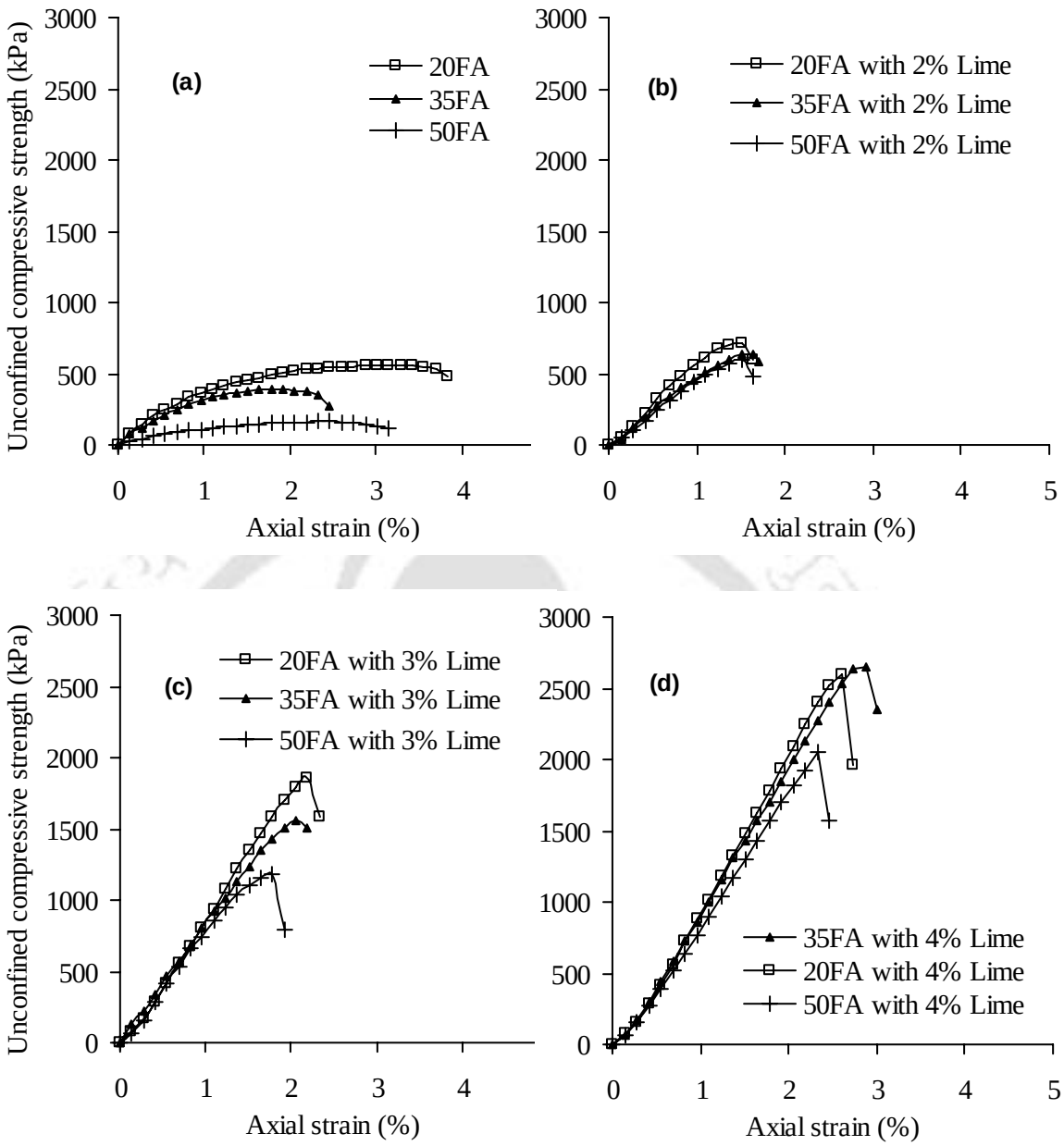


Figure 6.11 Typical stress-strain curves of various soil-fly ash mixes with (a) no lime (b) 2% lime (c) 3% lime (d) 4% lime, tested after 480 days of curing

6.4.4 Unconfined Compressive Strength (UCS) of Soil-Fly Ash Mixes

The UCS values and the corresponding failure strains of the soil–fly ash mixes are summarized in Table 6.3. For all percents of fly ash and curing periods used in the study, the UCS of the mixes ranges from 129.4 to 557.0 kPa with strain at failure ranging from 1.78% to 3.61%, when tested without lime. With the addition of lime, the UCS is found to vary between 176.5 to 2651.7 kPa with failure strain ranging from 1.19% to 2.88%.

The effects of curing time on UCS of different mixes are shown in Fig. 6.12. The results obtained for the soil are also plotted as a reference value for the 480 days curing period. The variation in UCS of the mixes with the increase in fly ash content and curing duration is shown in Fig. 6.13.

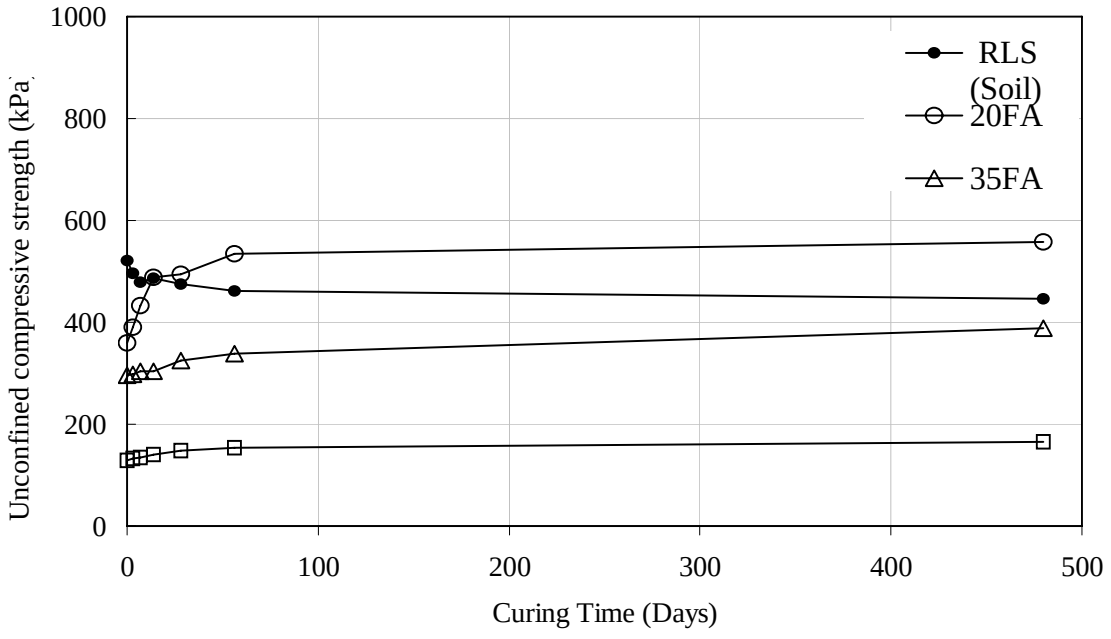


Figure 6.12 Variation of UCS of different soil-fly ash mixes with the increase in curing duration

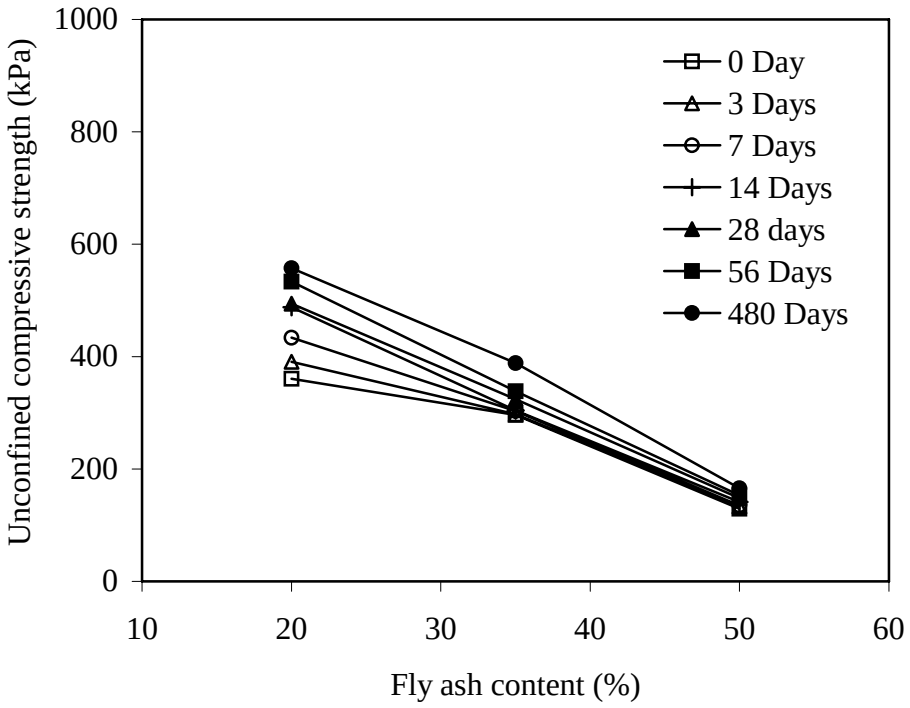


Figure 6.13 Variation of UCS of soil with the change in fly ash content and curing duration

It can be noted from Fig. 6.12 that the UCS of the soil decreases more within the first 7 days of curing period, after which the reduction is very marginal. It can also be noted that the addition of fly ash to the soil results in an immediate reduction of compressive strength (i.e. at zero curing period). Addition of 20% fly ash to the soil immediately reduces the UCS of the soil from 520.7 kPa to 360.8 kPa, and with 50% replacement of the soil with fly ash the UCS further reduces to 129.4 kPa.

It can also be noted that the loss of strength is nearly proportional to the amount of fly ash added to the soil (Fig. 6.13) that can be attributed to the change in gradation/texture of the soil. Nicholson & Kashyap (1993), while investigating compressive strength of a number of tropical soils reported similar behaviour and attributed it to the reduction in cohesive strength with concurrent increase in frictional strength due to the alteration of the soil into more friable and “less clayey” form.

It can be observed from Tables 6.3 and Figs. 6.12 & 6.13 that the compressive strength of the mixes steadily increases with the increase in curing time. However, the rate of strength gain decreases with the increase in amount of fly ash. A replacement of 20% soil with fly ash results in an increase in UCS of the 20FA mix from 360.9 kPa to 557.0 kPa (54.3%) over a curing period of 480 days. The amount of strength gain upon curing is lesser with a higher fly ash content. For the same curing period, the 35FA and 50FA mixes show only 31.5% increase (from 296.2 kPa to 389.3 kPa) and 28.0% increase (from 129.4 kPa to 165.7 kPa) respectively in the UCS.

Further, it can be noted from Fig. 6.12 that only the 20FA mix shows a higher UCS than the soil after 14 days curing. The UCS of the 35FA and 50FA mixes remain less than that of the soil even after a long curing period. Also, it can be observed that most of the strength gain occurs within the initial 56 days followed by marginal strength increase on further curing up to 480 days.

6.4.5 Unconfined Compressive Strength (UCS) of Soil-Fly Ash-Lime Mixes

It can be observed from Tables 6.4 to 6.6 and Figs. 6.3 to 6.5 that the UCS of the soil is appreciably affected by the combined use of fly ash and lime as well as by curing time.

The effects of curing time and lime content on UCS of the various mixes are shown in Figs. 6.14(a), 6.15(a) and 6.16(a). More detailed short-term variations in UCS of the respective mixes are depicted in Figs. 6.14(b), 6.15(b) and 6.16(b). It can also be readily observed that addition of lime to the soil-fly ash mixes and subsequent curing results in significant improvement of strength of the stabilized material. Earlier research (Consoli et al., 2001) on soil, fly ash and lime stabilization have shown that the rate of

strength development and the level of such strength depend on several factors including:

i) physical and chemical activity of clay minerals present in the soil,

ii) characteristics of fly ash,

iii) amount of replacement of soil with fly ash,

iv) mix proportions,

v) ambient emperature,

vi) curing environment, and

vii) nature and amount of other additives.

Further, the rate and level of strength gain initially and within a short-term period occurs due to the modification reactions between soil and lime, mostly caused by flocculation and cation exchange. These short-term modification reactions occur typically within a short time period depending on the clay minerals involved (Rogers and Glendinning, 1997). The long-term change in strength, on the other hand, is mainly controlled by pozzolanic reactions between fly ash and lime and also between soil and lime. The pozzolanic reactions starts after a relative time lag (also called the induction or delay period), and it results in the formation of new cementitious phase

It can be noted from Figs. 6.14(a), 6.15(a) and 6.16(a) that the UCS of the mixes increase at a much greater rate during the first 56 days of curing and the process gradually slows down with the increase in curing duration but continues to grow steadily with long-term curing.

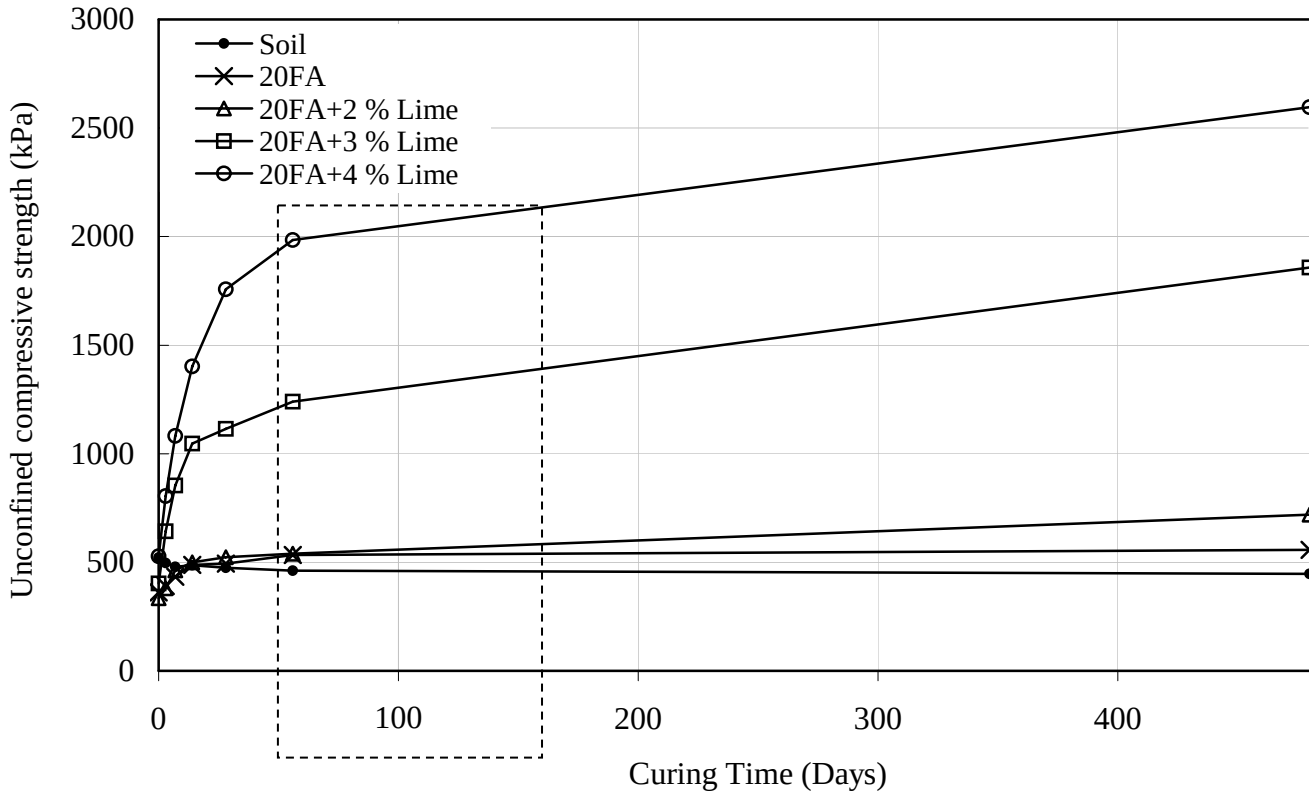


Figure 6.14(a) Variation of UCS of 20FA mix with increasing curing time

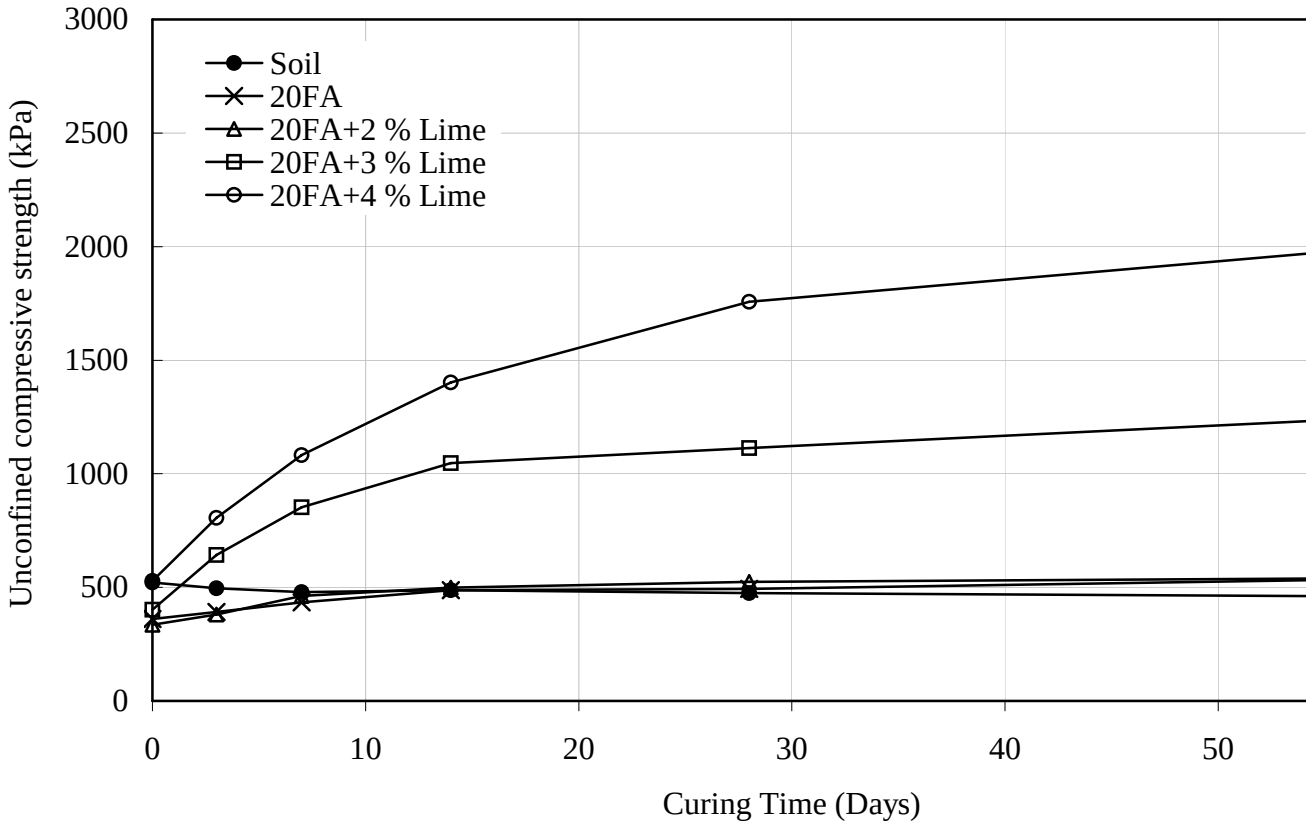


Figure 6.14 (b) Details of short-term variation of UCS of 20FA mix with increasing curing time as shown in the inset of Fig 6.14 (a)

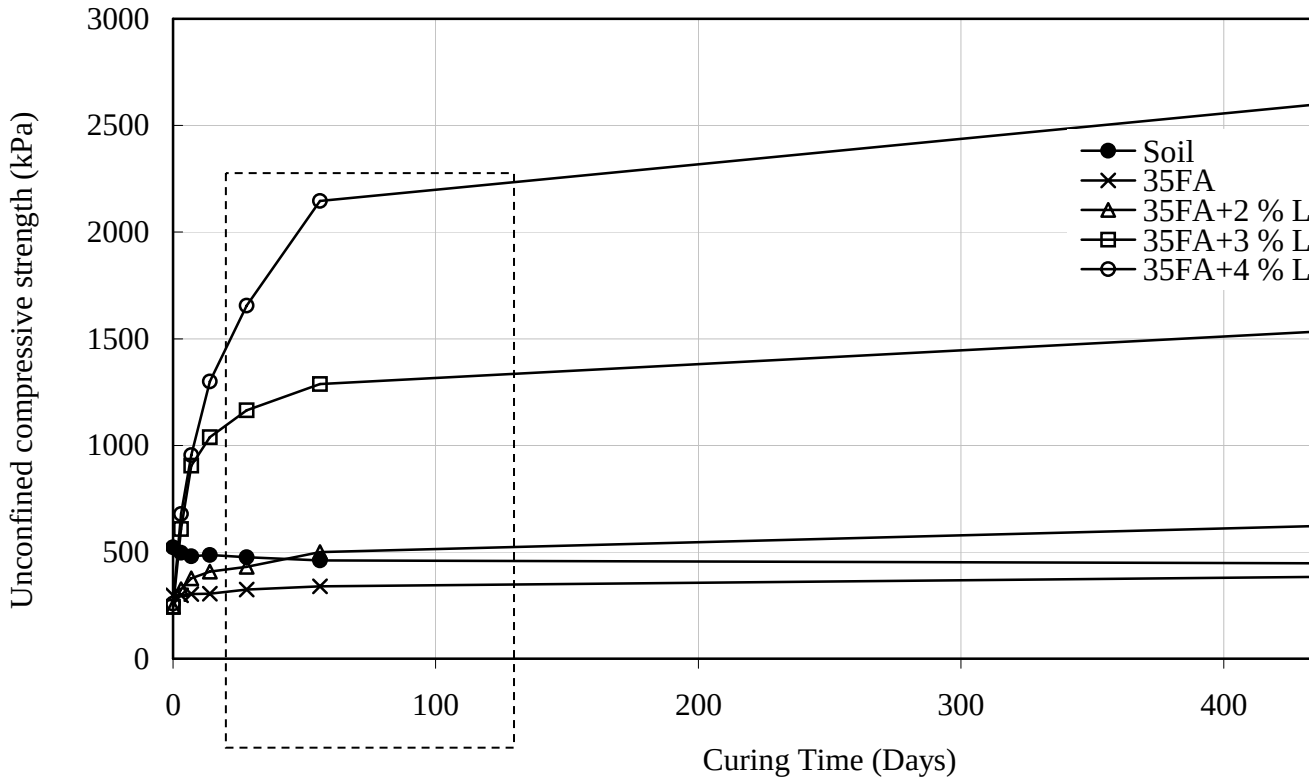


Figure 6.15 (a) Variation of UCS of 35FA mix with increasing curing time

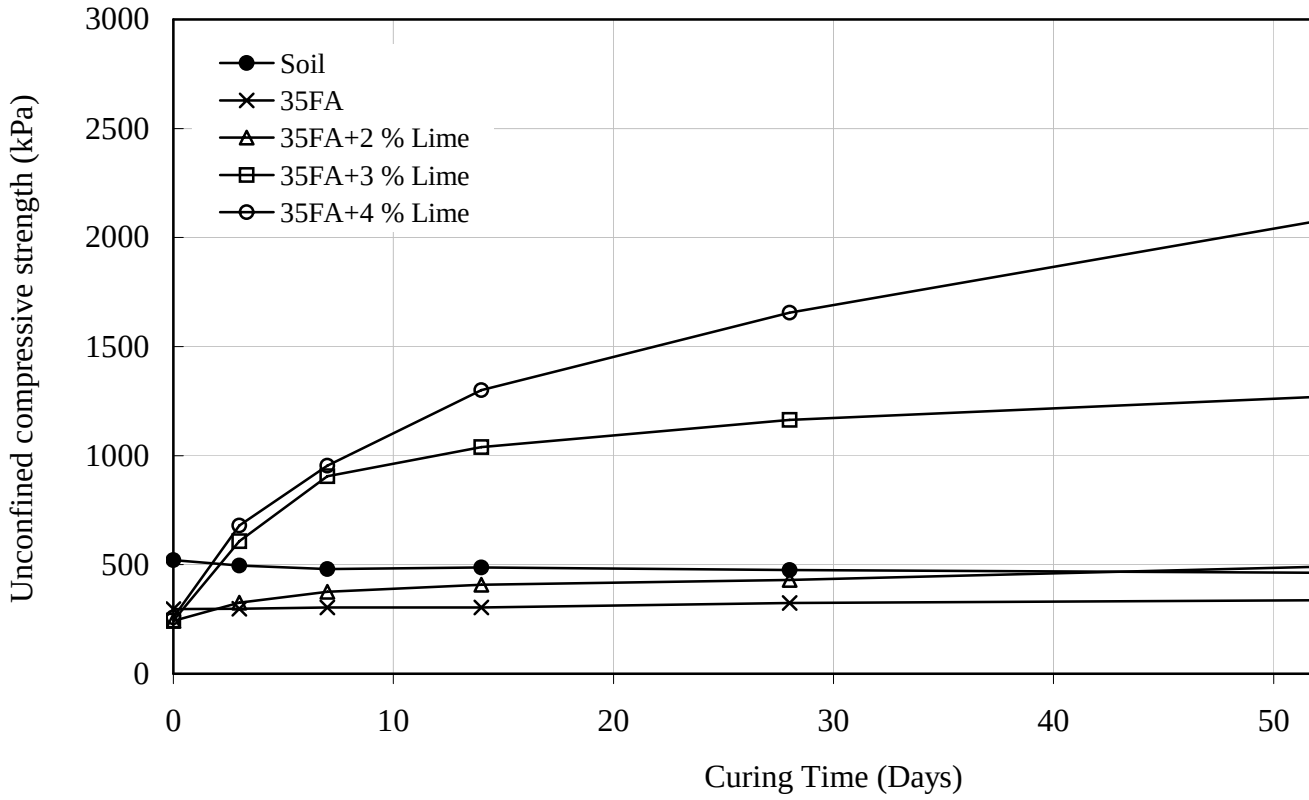


Figure 6.15 (b) Details of short-term variation of UCS of 35FA mix with increasing curing time as shown in the inset of fig 6.15(a)

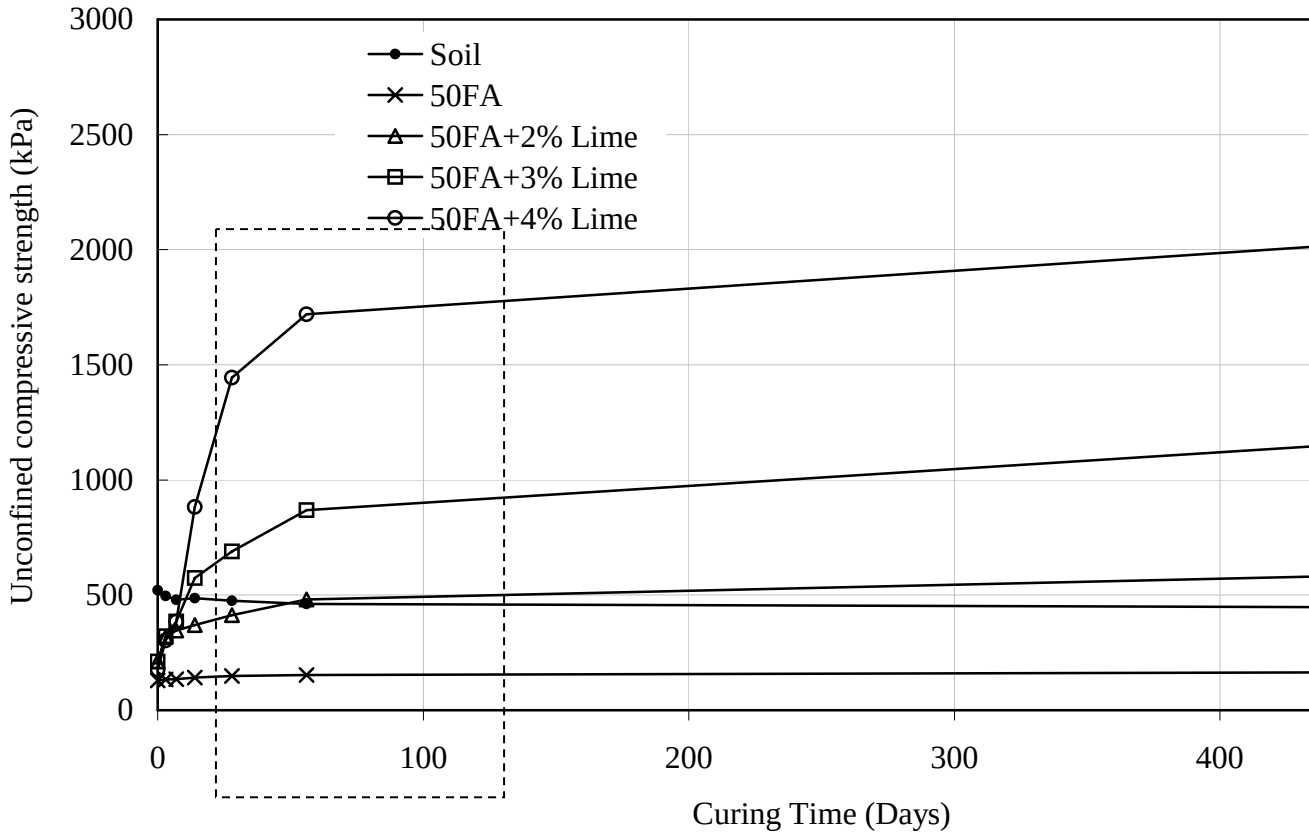


Figure 6.16 (a) Variation of UCS of 50FA mix with increasing curing time

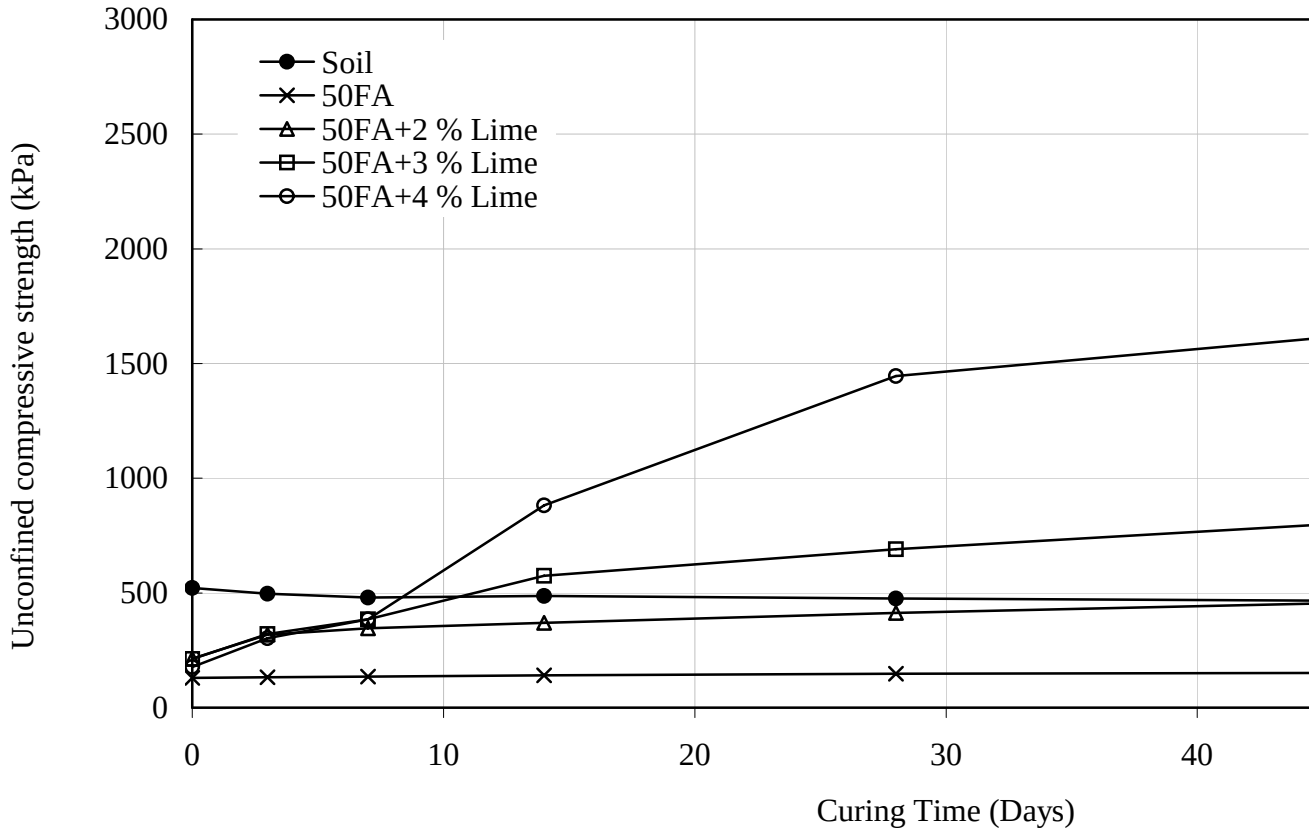


Figure 6.16 (b) Details of short-term variation of UCS of 50FA mix with increasing curing time as shown in the inset of fig 6.16 (a)

It can be further noted from Figs. 6.14(b), 6.15(b) and 6.16(b) that the rate and level of strength gain during the initial stage of curing up to 7 days due to the increase in lime content is not uniform and varies with the amount of fly ash present in the mix. In case of 50FA mix, it appears that the strength gain process is slightly delayed in the initial stage of curing [Fig. 6.16(b)] up to 7 days. The UCS initially increases at a lower rate up to 7 days and at a much higher rate up to 56 days. It again slows down to a steady rate on further curing over a long period.

Boardman et al. (2001), based on the research on lime-clay mixes reported significant pozzolanic activity only after a period of 7 days. In case of lime-fly ash mixes, Jalali et al. (1997) reported delay periods ranging from 4.8 h to 75 days with curing temperatures varying between 20ºC to 65ºC. Consoli et al. (2001) based on research on non-plastic sand, fly ash and carbide lime have reported sufficient strength gain only after a delay period of 90 days. A close examination of the short-term variation of UCS of the mixes, considered in the present study, indicates that significant pozzolanic activity starts after an induction period of 7 days.

A comparison of the lime fixation values, obtained from plastic limit tests (Chapter 4), with the rate and level of strength gain by mixes shows that they are closely linked with the induction period. The 50FA mix has lime fixation value of 2%. During the induction period, practically no effect on the UCS is seen, even when the amount of lime exceeds the lime fixation amount. Beyond the induction period, increasing lime content significantly improves the compressive strength. In the case of the 35FA mix, which has a lime fixation value of 3%, increasing the lime content from 2% to 3% noticeably increases the UCS also within the induction period. However, lime content exceeding 3% has no effect within the induction period. Beyond this duration, the increased lime content increases the compressive strength. The mix 20FA, whose lime fixation value is 4%, also shows significant improvement of strength within the induction period with the increase in lime content from 2% to 4%, but the effect of higher lime content could not be reported, as the use of lime was limited only to 4%.

The above observations clearly indicate that for lateritic soil-fly ash-lime mixes, significant pozzolanic activity starts after a curing period of 7 days and it continues for a long curing duration but at a slower rate beyond 56 days. The short-term modification process continues only up to the ICL value. Addition of lime beyond this lime content has little impact on the short-term change but greatly contribute to pozolanic reactions.

It is also important to note the high level of strength generated in the lateritic soil by the process of curing compared with other types of soils for similar treatment (Table 6.7). The presence of soluble silicon and aluminium ions is a major factor affecting the rate and extent of pozzolanic reactions produced by lime (Sherwood, 1993). In case of lateritic soils, high concentration of sesquioxides provides a good source of aluminum ions required for pozzolanic reactions. Most of the free silica in the soil, on the other hand, has been leached during laterite genesis and the clay minerals are the primary silica source for reaction with lime. However, if the clays are heavily coated and inhibited by the sesquioxides, then the effect of lime treatment would be minimal (Townsend, 1985). The apparent shortage of silica has been made up with the addition of fly ash, which is a very good source of silica and possibly explains the reason for an early onset of the pozzolanic activity. Further, as lateritic soils contain a high proportion of iron oxide, its agglomerates formed upon the addition of lime gives a better aggregate strength.

Table 6.7 Examples of the amount fly ash and lime added for achieving maximum value of unconfined compressive strength of various stabilised soils

Soil type	Fly ash + lime (%)	Compaction and curing details	UCS (kPa)	Reference

Expansive soil, LL = 42%, PI = 22%	20 + 0	Harvard min. compaction Cured for 28 days	29.5	Keshwars and Dutta (1993)
Lean clay LL = 44%, PI = 25%	16 + 0 (FBC)	Compated at MDD and OMC Cured for 7 days	275	Ferguson (1993)
Weathered shale LL = 52%, PI = 29%	18 + 0 (FBC)	Compated at MDD and OMC Cured for 7 days	1500	
Fat clay LL = 64%, PI = 40%	16 + 0 (FBC)	Compated at MDD and OMC Cured for 7 days	1990	
Fat clay LL = 53%, PI = %	18 + 0 (FBC)	Compated at MDD and OMC Cured for 7 days	2030	
Expansive soil, LL = 89%, PI = 47%	8 + 0	At MDD and OMC	359	Srivastava et al. (1995)
Decomposed granite, Non plastic	10 + 0	95% std. Proctor density, 28 days cured	770	Vesquez and Alonso (1981)
Weak tropical soil, LL = 30%, PI = 15%	15 + 0	95% std. Proctor density, 28 days cured	1100	Indraratna and Kuganenthira (1991)
Alluvial soil LL = 89%, PI = 47%	16 + 0	At MDD and OMC	228	Srivastava et al. (1997)
Alluvial soil LL = 27%, PI = 9%	10 + 5 12 + 7	At MDD and OMC 7 days cured	630 590	Singh et al. (1997)
Silty Clay LL = 29%, PI = 8%	35 + 2 35 + 7	Std. Proctor density, 28 days cured	1200 2000	Tsonis et al. (1983)
Expansive soil Highly plastic clay	15 + 0 (FBC)	Modified proctor density Cured for 28 days	651	Nicholson and Kashayap (1993)
Saprolite (Clay/sily soil)	15 + 0 (FBC)	Modified proctor density Cured for 28 days	397	
Residual volcanic soil Highly plastic	15 + 0 (FBC)	Modified proctor density Cured for 28 days	659	
Tropical soil Plastic clay	25 + 0 (FBC)	Modified proctor density Cured for 28 days	1993	
Expansive tropical Highly plastic clay	15 + 0 (FBC)	Modified proctor density Cured for 28 days	599	
Sandstone residual soil LL= 2%, PI= 7%	25 + 4 25 + 7 25 + 10	Compacted at MDD and OMC, cured for 28 days	1000 1247 1243	Consoli et al. (2000)
Silty soil	75 + 0	At MDD and OMC No curing	61.7	Kaniraj and Havanagi (1999)
Sandy soil	75 + 0	At MDD and OMC No curing	44.1	
Sandy soil	75 + 0	At MDD and OMC No curing	146.4	

Note : FBC = Fluidised Bed Combustion ash

Although the UCS of the soil decreases considerably with the addition of fly ash up to 50% (Tables 6.3 to 6.6 and Figs. 6.14 to 6.16), 2% lime was adequate to enhance the strength of the treated and cured material to a level equivalent to that of the soil. Beyond this amount, addition of further lime significantly improves the strength of the

treated material upon curing irrespective of the fly ash content. The long-term change shows that the rate of strength gain for 3% and 4% lime contents are almost similar. A closer look reveals that it is only because of the difference in the level of strength attained by the specimens during their short-term curing that a difference in the long-term ultimate strength of the mixes with increasing lime content occurs. For example, the somewhat higher long-term UCS shown by the 35FA mix compared to the 20FA mix, both with 4% lime content, is mainly due to the higher strength gained by the specimens during short-term curing.

Figure 6.17 shows the variations of UCS attained at different stages of curing by different soil-fly ash mixes for varying lime contents. The figures clearly show that the strength gain at different stages of curing changes with the amount of lime added. Specimens prepared with 2% lime shows only nominal increase in the UCS with curing. The strength gain shows a rapid and continuous increase with each additional percent lime. In case of the 20FA and 35FA mixes, addition of 2% lime marginally lowers their UCS, when tested without curing of the specimens. But increase in lime content or curing gradually increases the UCS. Also, both the mixes show a very sharp increase in long-term strength, when lime content was raised above 2%. When 2% lime was added, the 20FA mix shows an increase in UCS approximately by 29% (from 557.0 to 719.8 kPa). The same mix shows an increase of approximately 157% (from 557.0 to 1856.4 kPa) and 260% (from 557.0 to 2596.8 kPa) in the UCS, when the lime content is raised to 3% and 4% respectively.

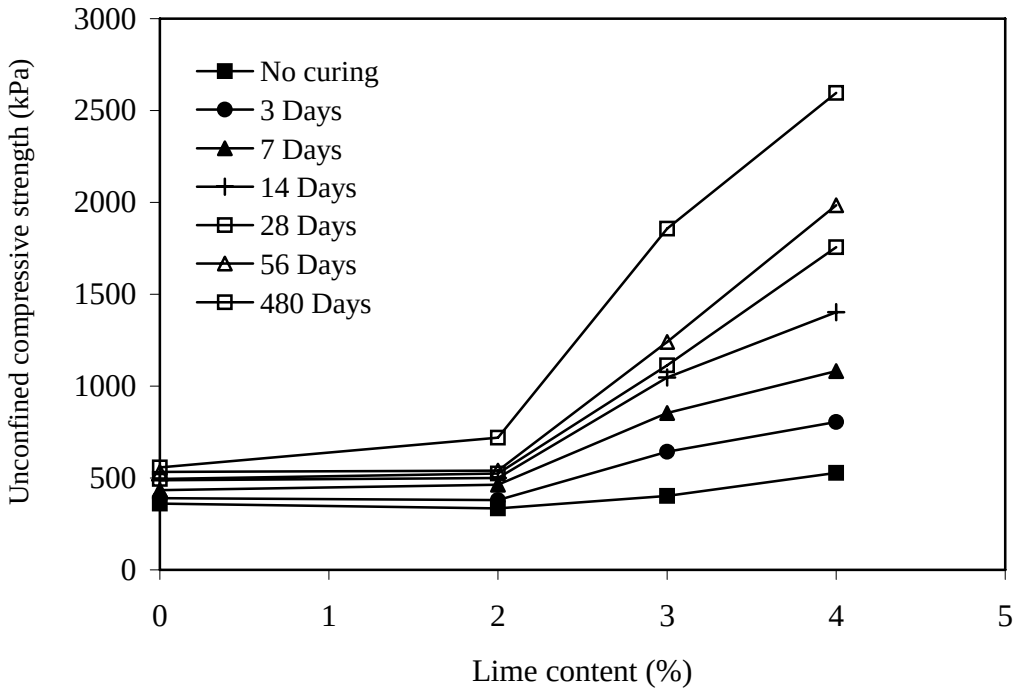


Figure 6.17 (a) 20FA mixes

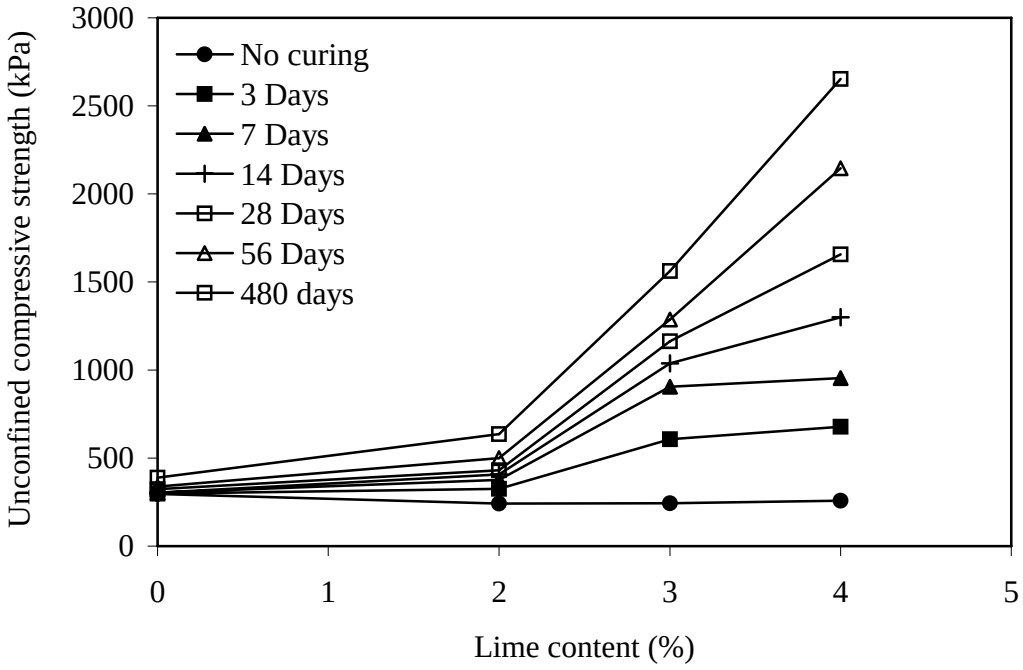


Figure 6.17 (b) 35FA mixes

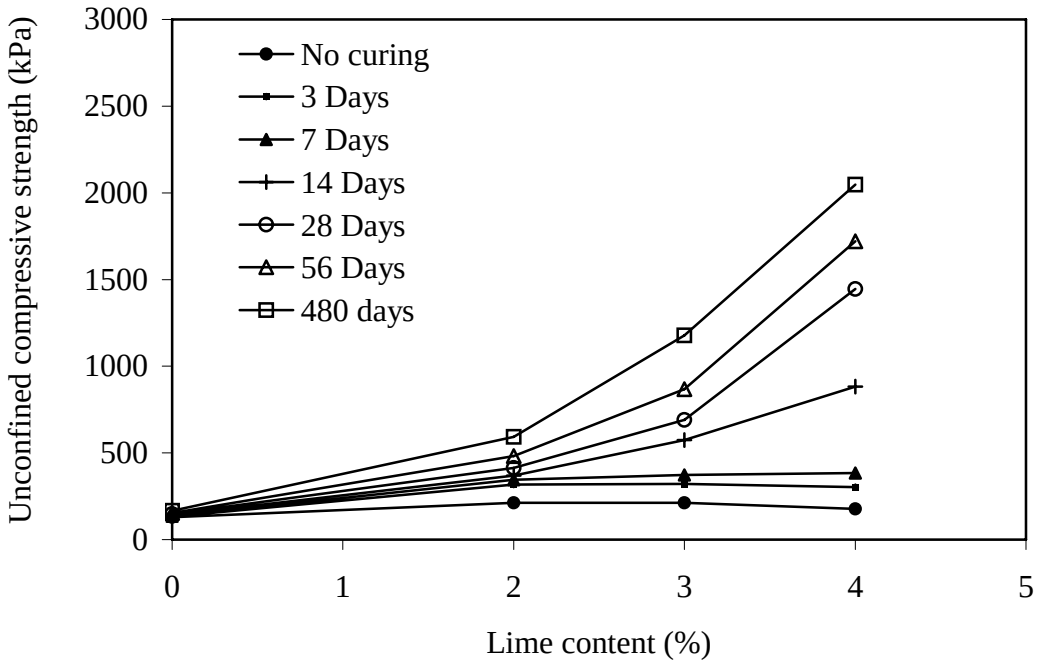


Figure 6.17 (c) 50FA mixes

Figure 6.17 Variation of UC strength of different mixes with lime content and curing period for (a) 20FA (b) 35FA (c) 50FA mixes

Similar observations are noted in the case of the 35FA mix, which shows an even higher increase of the UCS. For the same curing durations, the 50FA mix shows a uniform increase in the UCS.

6.4.6 Soaked Unconfined Compression Tests

The soil shows a significant reduction in the UCS upon soaking. The UCS of unsoaked soil immediately after compaction is 520.7kPa. Soaking before the UC test reduces this value to 211.2 kPa. All soil-fly ash specimens prepared without lime collapsed during soaking in water even after curing. Similar collapse was also observed in the case of lime treated specimens of 35FA and 50FA mixes without curing. The typical stress-strain curves of UC (soaked) tests on the mixes for 14, 28 and 480 days are presented in Fig. 6.18. The UCS values for soaked condition are presented in Tables 6.8 to 6.10.

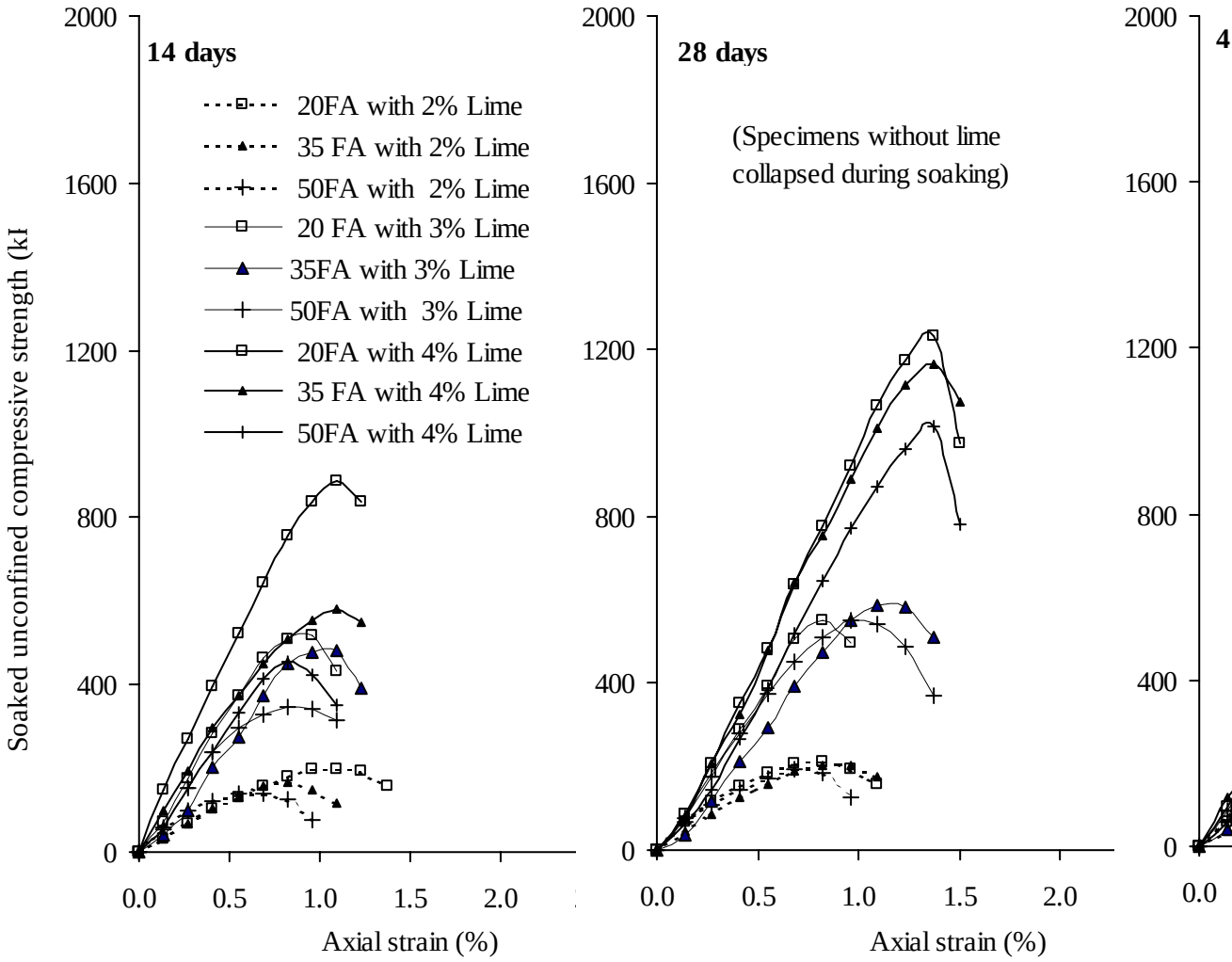


Figure 6.18 Typical stress-strain plots of different soil-fly ash-lime mixes tested after various curing periodTable 6.8 Soaked unconfined compressive strength and failure strains for the 20FA mixes.

Curing time (Days)	Without lime		With 2% lime		With 3% lime		With 4% lime	
	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}
0	Specimens collapsed during soaking		79.4	1.69	99.0	1.10	140.2	0.87
3			119.6	1.51	247.1	1.10	235.4	0.82
14			202.0	1.01	492.3	0.96	888.5	1.10
28			218.7	0.82	551.1	0.87	1219.0	1.37
480			475.6	0.87	1119.9	1.51	1700.5	1.65

σ_{ucf} = UC strength (kPa) ε_{ucf} = Strain at failure (%)

Table 6.9 Soaked unconfined compressive strength and failure strains for the 35FA mixes.

Curing time (Days)	Without lime		With 2% lime		With 3% lime		With 4% lime	
	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}
0	Specimens collapsed during soaking		Specimens collapsed during soaking					
3			72.6	1.33	115.7	1.10	121.6	0.87
14			166.5	0.82	482.5	1.10	579.6	1.10
28			204.0	0.87	583.5	1.28	1165.0	1.37
480			329.5	1.19	976.7	1.33	1747.6	1.70

σ_{ucf} = UC strength (kPa) ε_{ucf} = Strain at failure (%)

Table 6.10 Soaked unconfined compressive strength and failure strains for the 50FA mixes

Curing time (Days)	Without lime		With 2% lime		With 3% lime		With 4% lime	
	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}	σ_{ucf}	ε_{ucf}
0	Specimens collapsed during soaking		Specimens collapsed during soaking					
3			32.4	0.69	78.5	0.87	94.1	0.96
14			139.3	0.73	343.2	1.01	445.2	0.87
28			197.1	0.78	551.1	1.10	1000.3	1.33
480			304.0	0.82	790.4	1.23	1413.1	1.37

σ_{ucf} = UC strength (kPa) ε_{ucf} = Strain at failure (%)

It can be observed from the Fig. 6.18 that the peak strength, initial stiffness and brittleness of specimens increase with the amount of lime in the mix. Curing gradually transforms the ductile material to brittle nature, with its intensity increasing with the extent of curing. The variations of UCS obtained from soaked tests on soil-fly ash-lime mixes for various short and long-term curing period are presented in Fig. 6.19. It can also be noted from the Tables 6.8 to 6.10 that soaked UCS gradually increases with the increase in curing time of the specimens. For a specific curing period, the soaked UCS increases with the increase in lime content of the specimens. Further, the variation of UCS of the soaked specimens with curing time follow a trend that is very similar with the pattern obtained from un-soaked tests.

Similar to the un-soaked specimens, the long-term UCS of the mixes under soaked condition varies with lime and fly ash content. When low proportion of lime (2% and 3%) were added, mixes with low percentage of fly ash (20FA) showed maximum strength.

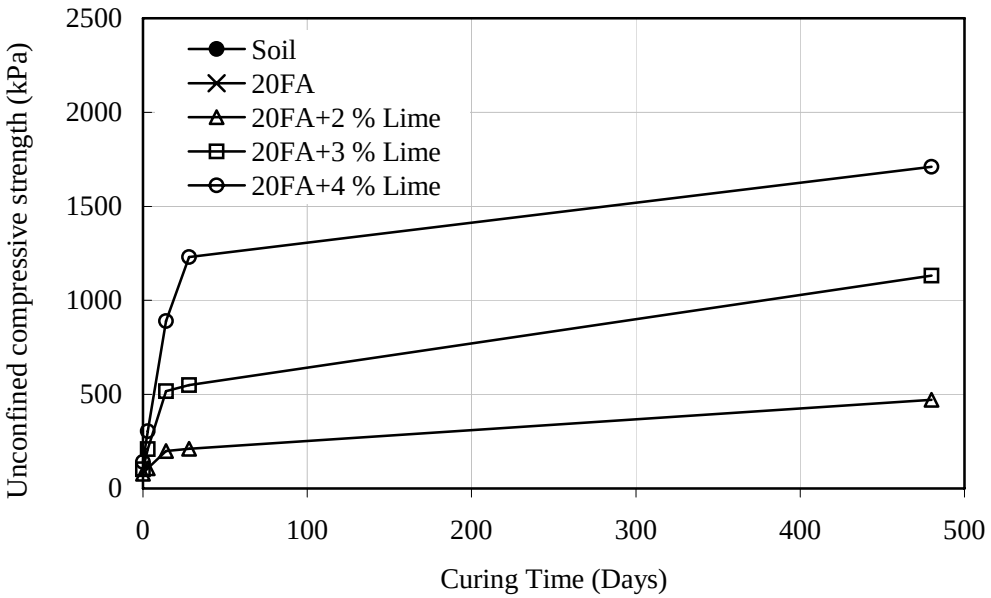


Figure 6.19 (a) 20FA mix (soaked)

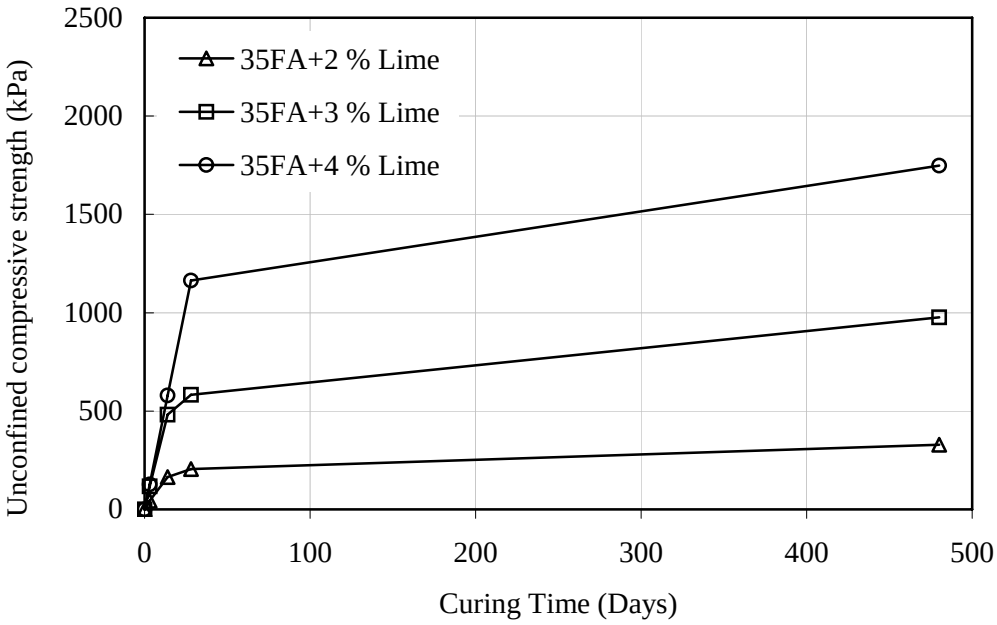


Figure 6.19 (b) 35FA mix (soaked)

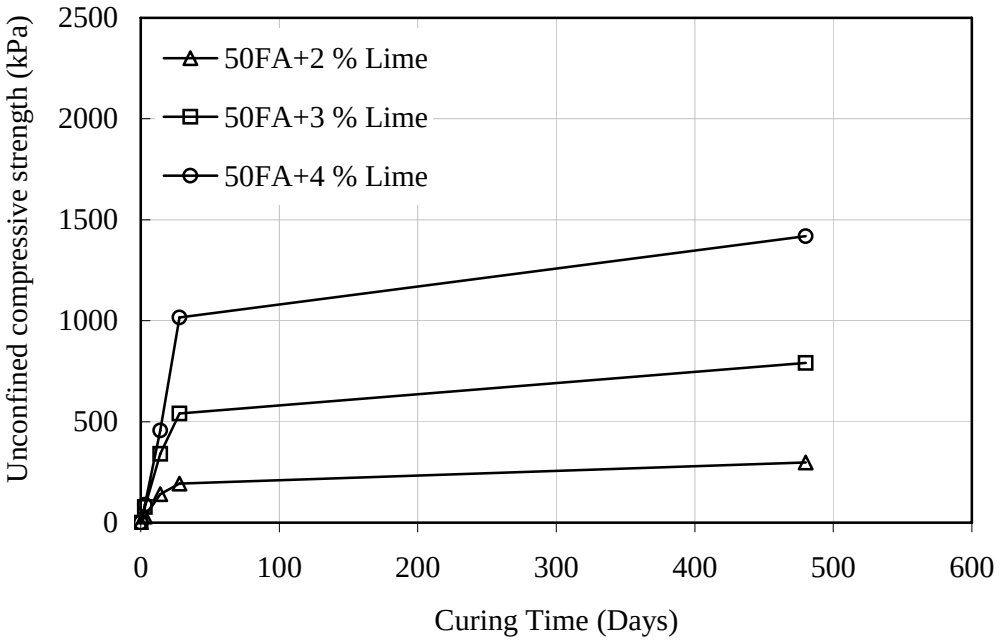


Figure 6.19 (c) 50FA mix (soaked)

Figure 6.19 Variation of UCS (soaked) with increasing curing time for (a) 20FA mix, (b) 35FA mix and (c) 50FA mix

Similarly, with high lime content (4%), mixes containing higher percentage of fly ash (35FA) showed maximum strength. A comparison of the UCS of the mixes under soaked condition with the corresponding values obtained without soaking, indicate significant reduction of UCS in the former case (Table 6.11). Although, the amount of reduction is significantly large in case of the specimens tested without curing, its magnitude considerably reduces with the increase in curing period. For example, the UCSs of specimens prepared from 35FA mix with 2%, 3%, and 4% lime get reduced by 4.5, 5.3, and 5.6 times respectively upon soaking at the end of 3 days of curing. The same specimens tested at the end of 480 days curing only have strength reduced by 1.9, 1.6, and 1.5 times on soaking. Similar trends are also observed in the case of the other two soil-fly ash mixes.

Table 6.11 Reduction in UC strength of different mixes due to soaking (expressed in terms of ratio of un-soaked strength to soaked strength)

Mix	Lime (%)	0 day	3 days	14 days	28 days	480 days
20FA	2	4.2	3.2	2.4	2.4	1.5
	3	4.0	2.6	2.1	2.0	1.7
	4	3.8	3.4	1.6	1.4	1.5
35FA	2	---	4.5	2.5	2.1	1.9
	3	---	5.3	2.2	2.0	1.6
	4	---	5.6	2.6	1.4	1.5
50FA	2	---	9.8	2.7	2.1	2.0
	3	---	4.1	1.7	1.3	1.5
	4	---	3.2	2.0	1.4	1.5

Lee and Coop (1995) from their study on the intrinsic behaviour of residually derived and decomposed granite soil reported that saturation of partially saturated soils has little effect other than the change in effective stress that may accompany flooding. The reduction in the UCS of the mixes upon soaking at the initial stages of curing, as observed in the present study, can therefore mainly be attributed to the change in their effective stress condition caused by the increased pore water pressure, when the soil is saturated/flooded.

Further, in the case of partially saturated residual soils, bonded structure plays an important role in their compression and strength (Vaughan, 1988; Fooks, 1990; Barksdale and Blight, 1997). From their studies on residually derived red soils from India, Rao and Revanasiddappa (2002) also reported that bonding plays an important role in the compression behaviour of residual soils and cementation bonds contribute largely to the yield stress. The reduction in the difference between unsoaked and soaked UCS with curing, observed in the present study is clearly due to the increased bonding developed due to the hydration reactions between lime and fly ash and also between lime and the soil.

6.4.7 Effect of Delay in Compaction

The UCS of the soil, fly ash, soil-fly ash mixes and soil-fly ash-lime mixes, obtained after different period of delay between wet mixing and compaction, are presented in Table 6.12.

Table 6.12 Unconfined compressive strength of the mixes obtained after different period of delay in compaction

Mix designation	Lime (%)	Delay in compaction				
		Immediate	12 hrs	1 day	2 day	3 days
RLS	0	442.3	460.5	467.8	463.7	460.3
BFA	0	121.3	120.6	120.3	118.0	117.0
20FA	0	492.3	507.3	494.3	471.2	455.6
35FA	0	341.4	336.6	324.6	320.2	316.3
50FA	0	158.0	155.6	154.2	151.3	148.6
20FA	2	552.0	542.0	523.7	492.0	475.2
35FA	2	457.0	442.3	430.5	390.4	348.1
50FA	2	455.3	433.9	412.9	366.6	298.8
20FA	4	1921.3	1902.3	1757.4	1602.0	1453.0
35FA	4	1862.0	1743.2	1656.4	1409.7	1202.5
50FA	4	1563.0	1489.6	1445.5	1365.0	1269.0

Figure 6.20 shows the variation of UCS of the soil, fly ash and their various mixes with and without delay in compaction. The soil, when tested without any additives, shows an increase in UCS within the 1 day of delay in compaction. The amount of increase in strength is about 5.7% and the reason can be attributed to the higher density of the soil (Section 5.4.4,) obtained as a result of better distribution of moisture within the soil. Beyond the one-day delay period, variation in UCS is very small and remains somewhat unchanged. The experimental fly ash on the other hand shows very marginal reduction in UCS due to delay in compaction over 3 days.

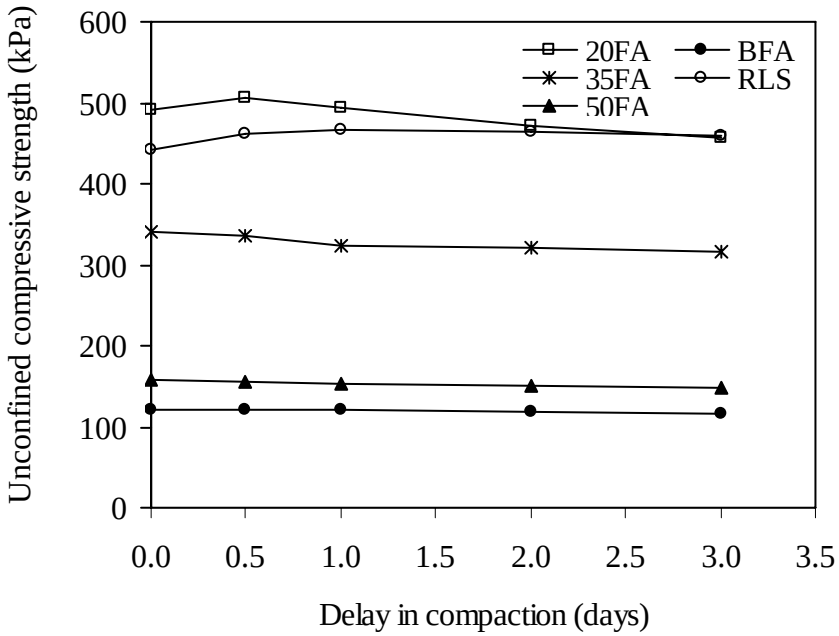


Figure 6.20 Variation of unconfined compressive strength of the soil, fly ash and their mixes with different period of delay in compaction

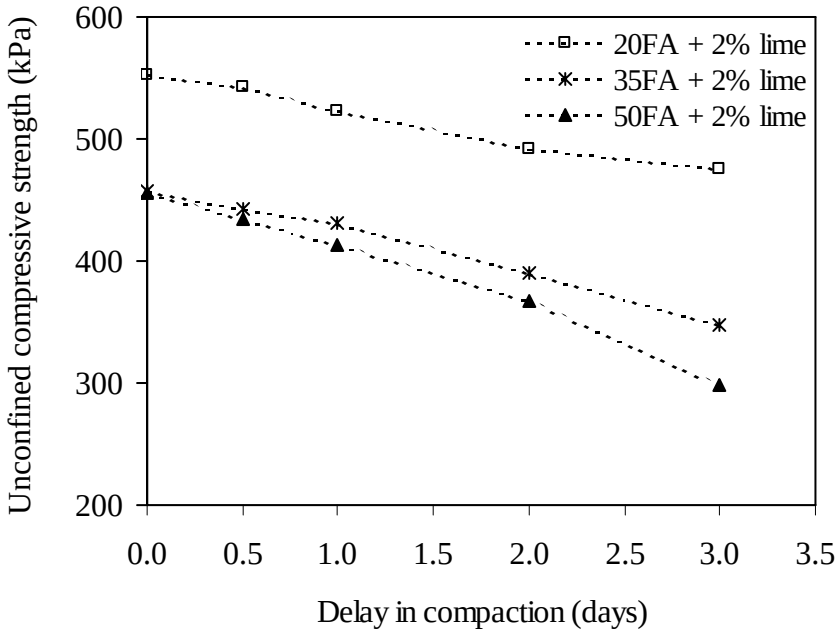
In case of the soil-fly ash mixes without lime, delay in compaction results in slight decrease in UCS. The amount of change is found to be more in case of 20FA mix and less in case of 50FA mix. In case of 20FA mix, three-days delay in compaction results in reduction of about 10% of UCS, when compared with the maximum value of UCS obtained after 12-hour delay period. The other two mixes 35FA and 50FA also show similar reduction in strength, but to a lesser degree (7.3% and 5.9% respectively), when compared with the maximum values obtained without any delay in compaction. In case of the 20FA mix, a slight increase in strength is noticed with 12 hours of delay in compaction, which can be attributed to the higher density of the material achieved because of the dominant effect of the soil fraction present in the mix.

The change in strength of the mixes with delay in compaction is due to the continued depletion of free lime present in the fly ash during the delay period. Available lime is consumed to satisfy the initial reaction of short-term modification that changes the density of the material. Also, as the delay period increases, the pozzolanic

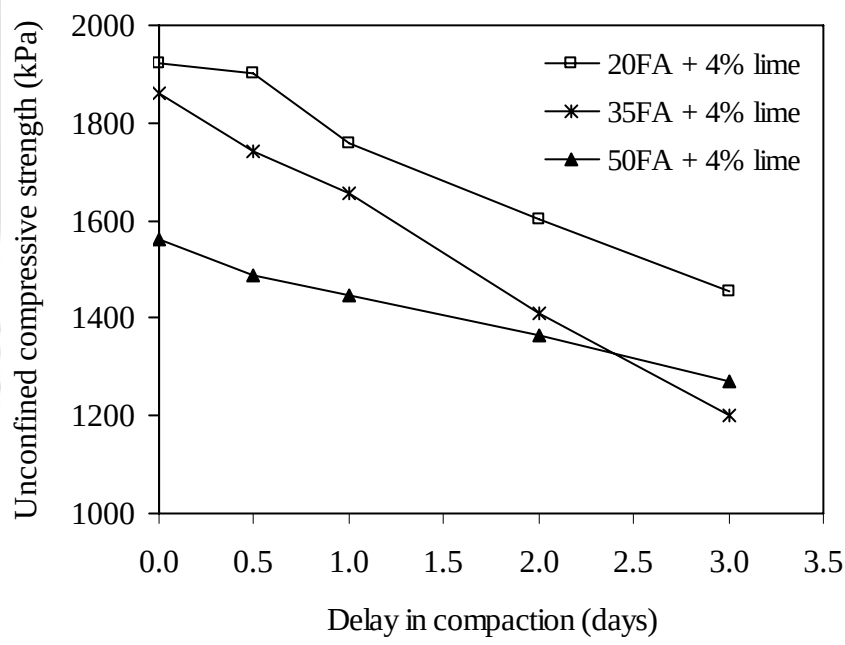
reaction between the soil, fly ash and lime continues. This continued reaction generates weak cementitious products that bond the inert soil and fly ash particles together producing a loose aggregate structure. Subsequent pulverisation and compaction break these relatively weak bonds, which cannot reform again.

Figure 6.21(a) and Fig. 6.21(b) show the variation of UCS with the delay in compaction for the soil-fly ash mixes treated with 2% and 4% lime. It can be seen from the figures that compaction delay causes considerable change in UCS of the lime treated soil-fly ash mixes. For the addition of 2% lime, mixes 20FA, 35FA and 50FA show 13.9%, 23.8% and 34.4% reduction of UCS respectively (Table 6.12 and Fig. 6.21a) for a delay in compaction for 3 days. When compared with the UCS with no curing done, it is observed that all the three mixes with 2% lime show more or less the same percentage strength gain (42.7%, 43.1% and 44.3% respectively) with 28 days curing period (Table 6.4 to 6.6). But a close examination of the strength gain within 3 days curing period of the mixes shows that 50FA mix exhibits much higher level of short term-strength gain compared to that of the other two mixes. The 3 days strength gain in case of 50FA is 49.8%, while it is 13.4% and 33.9% respectively for 20FA and 35FA. Higher reduction of UCS with delay in case of the 50FA mix, therefore, can be attributed to higher rate of short-term strength gain by the mix.

The cementitious bonds produced by the hydration products formed during this short-term period of 3 days will be destroyed during remoulding and compaction and they are unlikely to contribute to the ultimate strength generation. Only the fraction of cementitious bonds that develop after the compaction is available for strength generation, which explains the overall reduction in UCS of the mixes with delay in compaction.



(a) with 2% lime



(b) with 4% lime

Figure 6.21 Variation of unconfined compressive strength with different period of delay in compaction for soil-fly ash mixes with (a) 2% lime (b) 4% lime

When the lime content of the mixes is increased from 2% to 4%, the amount of reduction in UCS with delay in compaction significantly increases (Fig. 6.21b). The amounts of reduction are found to be 24.4%, 35.4% and 18.8% for the mixes 20FA, 35FA and 50FA respectively. The 35FA mix shows higher reduction in UCS with delay, compared to that of the other mixes, possibly due to the greater pozzolanic reactivity of this mix within 3 days time as indicated from the study of strength generation with curing time (Table 6.4 to 6.6 and Fig. 6.14 to 6.16). The amount of strength increase for 0 to 3 days curing time for 35FA mix is found to be 420.7 kPa, while the other two mixes 20FA and 50FA shows an increase of UCS by only 277.5 kPa and 126.5 kPa respectively.

In view of the above observation, it is suggested that the delay between wet mixing and compaction should be kept as minimum as possible, particularly in case of lime treated mixes that exhibit early strength gain. Further, while the addition of fly ash and lime causes rapid change in cohesive and frictional strength of soils, there are however limitations in using only UCS to assess the frictional component of the strength (Nicholson and Kashayap, 1993). The results of short-term changes in UCS should, therefore, be used with caution and more comprehensive methods like triaxial test should be performed for better evaluation of strength of the mixes.

6.5 CONCLUSIONS

1. The soil shows considerable increase in its UC strength when compacted to maximum dry density. However, it reduces considerably upon soaking. The fly ash exhibits a much lower UC strength compared to that of the soil and soaking results in complete loss of strength.
2. Addition of fly ash to the soil immediately reduces the UC strength of the mixes with concomitant change in stress-strain response of the mixes. With smaller

fraction of fly ash, the mixes tend to behave like the soil alone. But, with higher proportion of fly ash, it behaves more akin to the fly ash. Curing does not significantly alter the overall stress-strain characteristics of the mixes, except that the UC strength slightly increases without much change in the stiffness and brittleness of the material. The amount of strength gain upon curing is lesser with a higher fly ash content.

3. The overall stress-strain-strength response is significantly affected by the addition of lime to the soil-fly ash mixes and their subsequent curing. At low lime content, the stiffness of the soil-fly ash-lime mixes increases at all stages of curing. But, at higher lime content, the stiffness increases only during the initial stages of 7 days, beyond which little further change is noticed. The stiffness towards the later stages is practically independent of the fly ash content. The brittleness increases with the increase in lime content, attaining an extreme state of brittleness in the later stage marked by rapid build-up of stress and sudden collapse after peak strength.
4. The failure strain of the soil-fly ash-lime mixes, after an initial decline, tends to increase with curing. At higher lime content and towards later stage of curing, it increases remarkably with peak strength.
5. The change in UC strength of the mixes for variation in lime content, tested immediately after compaction, is small. It is the curing that causes significant difference in the level of UC strength of the mixes. Mixes with higher lime content show much greater level of UC strength with long-term curing and show similar rate of long-term strength gain. Their difference in the level of long-term strengths occurs essentially due to their difference in short-term strength gain.
6. Among all the soil-fly ash-lime mixes, the mix containing 35% fly ash and 4% lime shows highest long-term UC strength. When the lime addition is limited to 3%, mix containing 20% fly ash shows the maximum long-term UC strength. The

long-term UC strength of the soil-fly ash-lime mixes containing 50% fly ash remains much below the level than shown by the other two mixes. However, considering the reported inadequacy of UC test, the results may be validated by conducting triaxial tests.

7. The soil-fly ash-lime mixes show an early strength generation due to early onset of pozzolanic reaction that largely occur after an induction period of 7 days due to the presence of naturally available free alumina and iron oxide in lateritic soils. The rate of strength gain during the induction period is dependent on the lime fixation values of the soil-fly ash-lime mixes.
8. The loss of strength due to soaking or flooding of the soil can be significantly reduced by the combined addition of fly ash and lime and subsequent curing after compaction. The reason for the improvement is attributed to the increased bonding developed due to the hydration reactions between lime and fly ash.
9. Delay in compaction (3 days) of soil-fly ash mixes, without lime, causes marginal reduction in UC strength. The magnitude of reduction decreases with the increase in fly ash content but considerably increases with the addition of lime due to modification of texture and gradation as well as formation of cementitious products. The effect considerably increases with the increase in lime content and the UC strength decreases even further with the delay between wet mixing and compaction. Mixes showing higher short-term strength gain on curing show greater reduction in UC strength with 3 days delay in compaction.
10. The 20FA mix without lime shows maximum reduction of UC strength with 3 days delay in compaction. For the same delay, 50FA and 35FA mixes show maximum reduction in UC strength when tested with 2% and 4% lime respectively.

CHAPTER
SEVEN

SHEAR STRENGTH CHARACTERISTICS

7.1 INTRODUCTION

The use of stabilised soils for geotechnical applications requires a thorough understanding of their shear strength behaviour. Limited studies are available on the shear strength behaviour of soil stabilised with fly ash and lime (Consoli et al., 2001). Unconfined compression tests showed that combined addition of fly ash and lime resulted in significant improvement of the above properties of the residual lateritic soil (Chapter-6). This necessitates a detailed study of the shear strength response of the stabilised material as well. Consolidated undrained (CU) triaxial tests were carried out on the soil and fly ash and selected mixes with the addition of different quantities of lime to understand:

i) the overall stress-strain-strength characteristics of the soil and changes in its behaviour due to the addition of fly ash and lime, and

ii) the influence of compaction parameters, namely moisture content and dry unit weight, on the mixture behaviour of a selected soil-fly ash mix with varying lime content.

A few CU triaxial tests on the soil at undisturbed and remoulded states were also conducted to understand the effects of soil structure on the shear strength behaviour of the soil.

7.2 TEST PROGRAMME

Table 7.1 shows the summary of all CU triaxial tests carried out. As shown in the Table, the first series of tests were conducted to understand the strength characteristics of the fly ash alone by performing tests on specimens prepared at MDD obtained as per IS Standard Light Compaction Test.

In the next series, the characteristics of the soil alone, both in remoulded and undisturbed state, were studied. The remoulded specimens were prepared both at the field density as well as at MDD by statically compacting them at specific moisture contents as listed in Table 7.1.

Table 7.1 Summary of consolidated undrained triaxial testing program

Series	Test no.	Mix	Lime added (%)	Test /Compaction condition	Dry unit weight (gm/cm^3)	Moisture content (%)
1	1-3	BFA	Nil	Optimum moisture	1.23	28.30
2	4-6	RLS	Nil	Undisturbed	1.28	27.66
	7-9			Remoulded at field density	1.28	27.66
	10-12			Compacted at MDD	1.52	25.12
3	13-15	35FA	Nil	Optimum moisture	1.46	23.21
4	16-18	35FA	2	Optimum moisture	1.37	27.33
	19-21	35FA	2	Dry side of optimum	1.34	24.10
	22-24	35FA	2	Wet side of optimum	1.34	29.60
5	25-27	35FA	4	Optimum moisture	1.34	28.51
	28-30	35FA	4	Dry side of optimum	1.31	22.98
	31-33	35FA	4	Wet side of optimum	1.31	30.52

The changes in the characteristics of the soil with the addition of fly ash and the subsequent effect due to the addition of lime under different compaction conditions were investigated in the remaining three test series. The tests were conducted only on the 35FA mixes with varying lime contents (no lime, 2% and 4%) as this particular mix

had exhibited the highest pozzolanic activity compared to the others. The specimens were compacted at their respective MDD and OMC as tabulated in Table 7.1.

The influence of compaction parameters– moisture content and dry unit weight – on the mixture behaviour was evaluated by comparing the stress-strain response of specimens exhibiting the same dry density (98% of MDD) but having different water contents (one at dry of optimum and the other at wet of optimum) (Table 7.1).

7.3 EXPERIMENTAL DETAILS

7.3.1 Specimen Preparation

All remoulded/compacted specimens were prepared in the same manner as for UC tests as elaborated in section 6.3.1. In the case of tests on the soil at field density, samples as obtained from the field were air-dried and compacted by the same procedure after adding required quantity of water to achieve the density.

For soil and fly ash specimens, the tests were carried out immediately after their preparation. For the remaining mixes, all the compacted specimens were cured for 28 days before carrying out the tests.

7.3.2 Experimental Set-up

The complete electronic triaxial set-up used for the present investigation is shown in Fig. 7.1. The set-up uses a pneumatic system for application of confining and back pressures. Axial load, axial deformation and pore water pressure were measured with electronic load cell, linear variable displacement transducer (LVDT) and pressure transducer respectively. The values were recorded through an electronic data acquisition system connected to a computer. Specimen volume changes were measured using a twin-burette volume change indicator.

(a) A close up view of the triaxial cell

(b) A view of a test in progress

Figure 7.1 Experimental set-up for triaxial tests

7.3.3 Saturation, Consolidation and Testing

The triaxial tests were carried out under full saturation, for effective confining pressures of 100, 150 and 200 kN/m^2. This range of effective confining pressures is generally applicable in engineering applications such as improved soil layers bearing pavement structures and shallow foundations.

The specimens were saturated using back pressure. For fly ash specimens, back pressure of 100 kN/m^2 was found to be sufficient. For soil and its mixes with fly ash and lime, a higher pressure of 600 kN/m^2 was required. The back pressure was applied in increments as per the method of BS 1377:Part 8 (British Standards Institution, 1990a). Throughout the process of saturation, a differential pressure of 20 kN/m^2 was maintained. For fly ash specimens, the 'B' parameter value calculated was almost one. For the other specimens, 'B' values of at least 0.9 were ensured. Thereafter, the specimens were consolidated.

After consolidation, the specimens were sheared in undrained condition. For CU tests, it is recommended that the time to failure should be five times of t_{50} to ensure pore pressure equalization throughout the specimen (Baxter, 2000), where t_{50} is the time to 50% consolidation as observed from the consolidation stage. For the present study, a time to failure between five to seven times of t_{50} was selected. The strain at failure was estimated as 5% of axial strain (as observed from UC tests).

In case of fly ash, the specimens were loaded at a constant strain rate of 0.6 mm/min. For all other specimens, a strain rate of 0.06 mm /min was maintained throughout the experiment.

The observed data were corrected for area and membrane as per the method of Head (1998). In case of soil-fly ash mixes without lime, the area and membrane corrections were made for barreling type of failure. Mixes with lime exhibited well-

defined single-plane slip failures, and the area and membrane corrections were made accordingly.

7.4 RESULTS AND DISCUSSION

7.4.1 Behaviour of Experimental Fly Ash

The results of consolidated undrained tests on fly ash specimens (CU_1 to CU_3) are presented in Table 7.2. The variations of deviator stress, pore water pressure and ratio of effective principal stresses with the axial strain are shown in Figs. 7.2(a) – (c).

The stress-strain curves (Fig. 7.2a) are similar to that of loose sand or normally loaded clay of low sensitivity, exhibiting failure at comparatively large strains (6.37% to 7.17%) without a definite peak deviator stress. The specimens also show significant residual strength, which may be attributed to the silty nature of the material with small clay fraction. The pore water pressure increases initially and then decreases gradually (Fig. 7.2b) thus exhibiting dilatancy behaviour that is somewhat similar to compacted/dense sand and overconsolidated clays.

Table 7.2 Results of CU tests on experimental fly ash

Test	Confining pressure (kPa)	Axial strain at failure (%)	Pore water pressure at failure (kPa)	A-factor at failure	σ_1' at failure (kPa)	σ_3' at failure (kPa)	σ_1'/σ_3' at failure
CU_1	100	6.37	-10	-0.03	379	110	3.45
CU_2	150	7.17	-04	-0.01	482	154	3.14
CU_3	200	7.15	07	0.02	573	193	2.97

Notes: A-factor Δ' = Change in excess pore pressure/change in deviator stress
σ_1' and σ_3' = Principal effective stresses

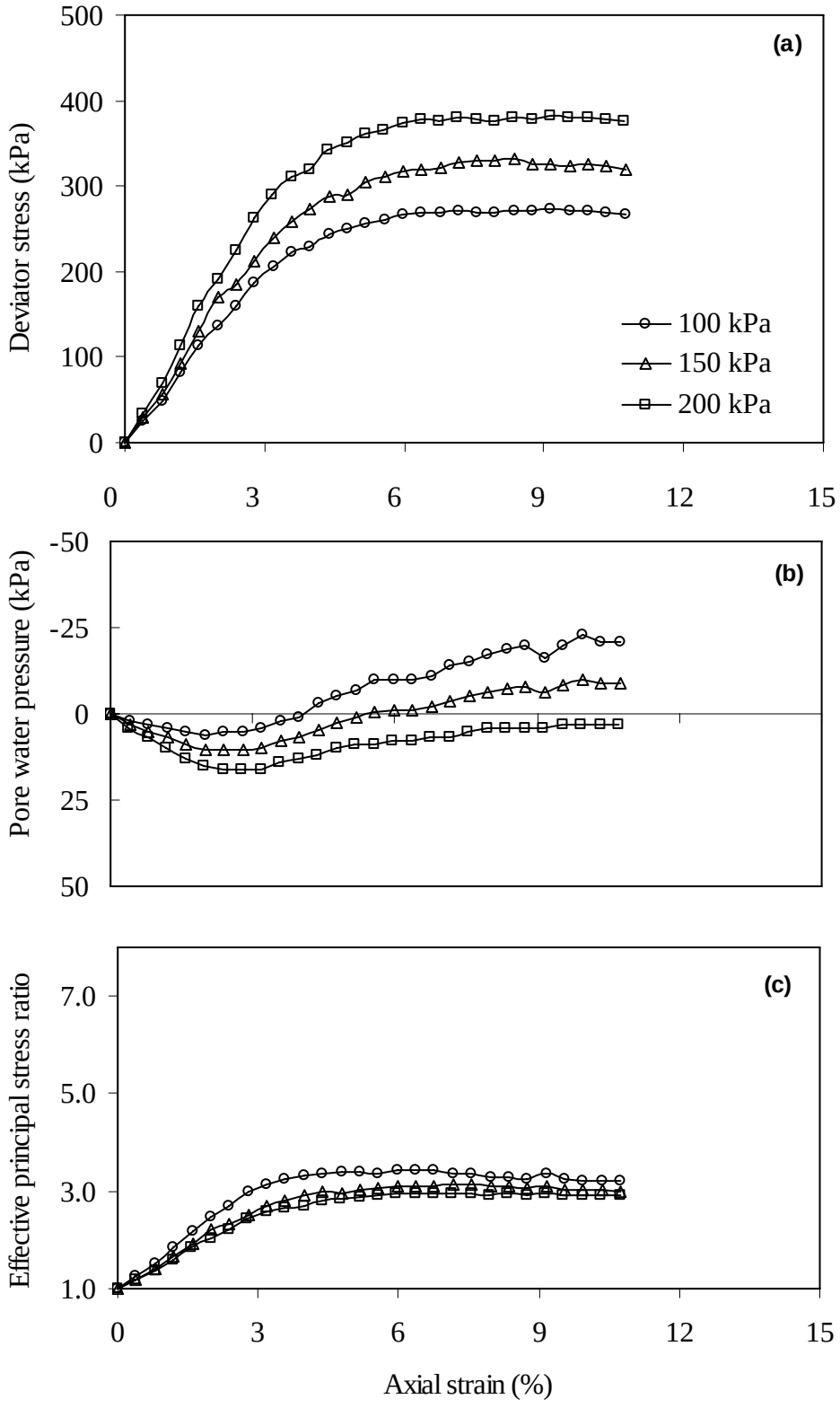


Figure 7.2 CU tests on the experimental fly ash showing the variations of (a) deviator stress, (b) pore water pressure, and (c) principal effective stress ratio with axial strain.

The results of the tests on fly ash specimens are also plotted as stress paths in Fig. 7.3. The effective stress paths for all confining pressures remain inclined towards right throughout the test, indicating very little development of pore pressure.

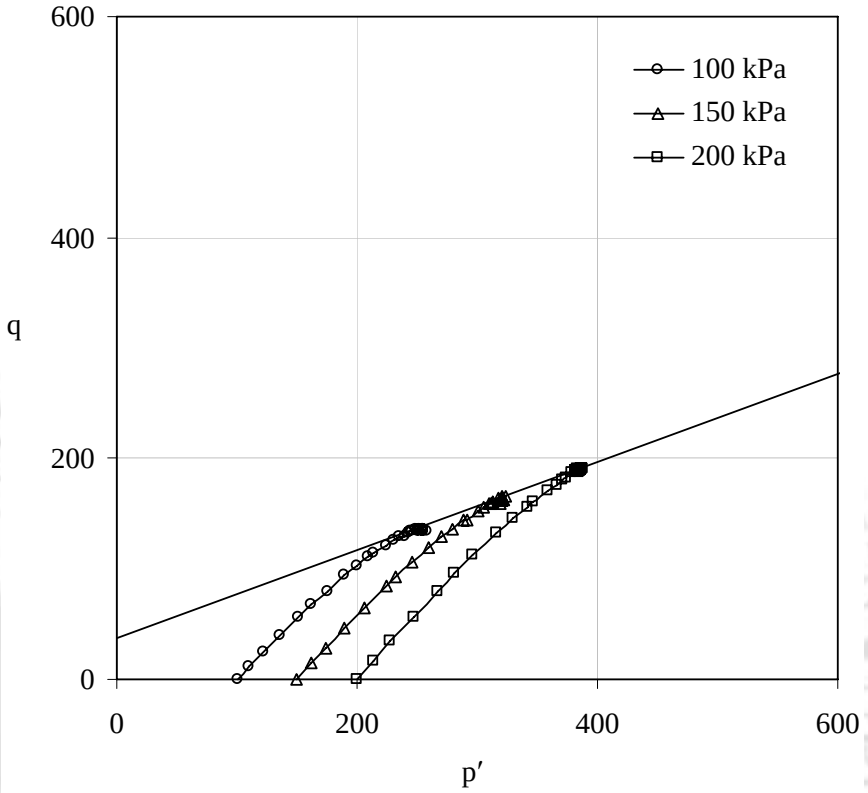


Figure 7.3 Stress paths from CU tests performed on fly ash specimens

The triaxial compression results are also plotted as stress paths. The variables p′ and q are defined below:

$$p' = (\sigma_1' + \sigma_3') / 2$$

$$q = (\sigma_1' - \sigma_3') / 2$$

where, σ_1' = major principal effective stress

σ_3' = minor principal effective stress

The failure line in the p′-q space is called the K_f line and is easily related to c′ and ϕ′. The effective undrained strength parameters for the soil at different testing conditions were computed using the following relationships:

$$\tan \alpha' = \sin \phi' \tag{7.1}$$

$$c' = \frac{d'}{\cos \phi'} \tag{7.2}$$

where, α' = angle that the K_f line makes with the horizontal

d' = intercept of K_f line with the Y-axis

The effective strength parameters ϕ' and c', computed from the equations 7.1 and 7.2 are found to be 22.44° and 52 kPa respectively. Srinivas et al. (1999) investigated the shear strength properties of seven different Indian fly ashes and reported that the ϕ' and c' values for compacted and saturated fly ash range from 25° to 39° and 17 to 96 kPa respectively. On the other hand, the ϕ_{cu} value varies from 19° to 38° without any cohesion component. McLaren and DiGioia (1987), based on values of 51 class F fly ashes reported ϕ' range as 34° ± 3.3°. The c' value of the present fly ash lies within the reported range, but ϕ' value is slightly lower than the minimum value reported in the range. Lower value of ϕ' is attributed to very low free lime content of the fly ash, as among many factors, the reactive silica and free lime content of fly ashes significantly influence the strength development (Pandian et al., 1999).

7.4.2 Behaviour of the Soil

Figure 7.4 shows the test results of CU tests done on the undisturbed, remoulded (at field density and water content) and compacted (at MDD) soil specimens (CU_4 to 12) in the form of deviator stress, pore water pressure and principal effective stress ratio plotted versus axial strain. The results of all these CU tests are summarized in Table 7.3, considering maximum principal effective stress ratio as the failure criterion.

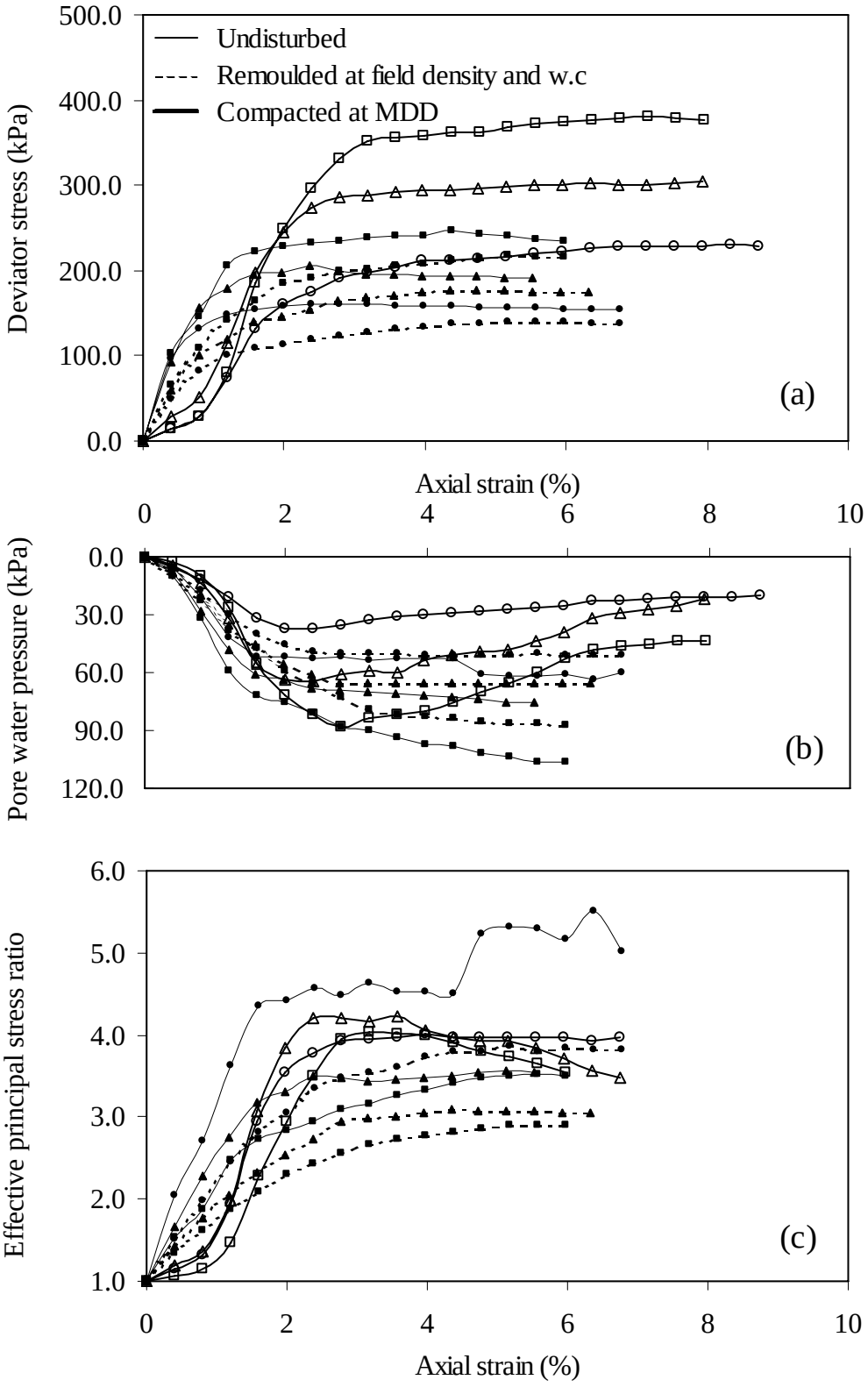


Figure 7.4 CU triaxial tests on the soil at undisturbed, remoulded (insitu) and compacted (at MDD) state showing the variation of (a) deviator stress, (b) pore water pressure, and (c) principal effective stress ratio with axial strain.

[Confining pressures: 100 kPa (circle), 150 kPa (triangle), 200 kPa (square)]

In the case of specimens at undisturbed state (CU_4 to 6), the principal effective stress ratio is found to be maximum at the peak deviator stress (Fig. 7.4). In contrast, in the case of remoulded insitu (CU_7 to 9) and compacted specimens (CU_10 to 12), the deviator stress continues to increase even after the principal stress ratio becomes maximum.

The increase in deviator stress occurring after this point is almost entirely the consequence of the drop in pore pressure due to the tendency of the soil to dilate when sheared (Bishop and Henkel, 1962). It shows that maximum values of c′ and ϕ′ are mobilized at lesser stress required to produce the maximum deviator stress.

Table 7.3 CU triaxial test results for experimental soil at different condition

Test	Condition	Confining pressure (kPa)	Axial strain at failure (%)	Pore water pressure at failure (kPa)	A-factor at failure	σ_1' at failure (kPa)	σ_3' at failure (kPa)	σ_1'/σ_3' at failure
CU_4	Undisturbed	100	2.40	54	0.33	210	46	4.58
CU_5		105	2.37	68	0.33	287	82	3.50
CU_6		200	4.37	88	0.39	349	102	3.42
CU_7	Remoulded (insitu)	100	5.18	51	0.36	189	49	3.86
CU_8		105	4.34	65	0.37	263	85	3.09
CU_9		200	5.17	86	0.39	332	114	2.91
CU_10	Compacted at MDD	100	3.97	30	0.14	282	70	4.02
CU_11		105	3.57	60	0.21	381	90	4.24
CU_12		200	3.18	84	0.24	467	116	4.03

Notes: A-bar = change in excess pore pressure/change in deviator stress
σ_1', σ_3' = principal effective stresses.

In general, the stress-strain curves are initially steep and then slight strain softening occurs until the specimens fail. The undisturbed specimens generally show much greater strength and stiffness than the remoulded specimens for all consolidation pressures. For undisturbed specimens, the peak deviator stress is obtained at low strains particularly for low consolidation pressures. This may be due to destruction of weak inter-particle bonds that normally develop in residual soils during their process of formation. These bonds impart a brittle behaviour to the material in its undisturbed state and as consolidation pressure increases, the structural effect of these bonds reduces.

On the other hand, specimens remoulded at field density and water content behaves in a ductile manner. The volume changes measured for these specimens after consolidation are similar to that of the undisturbed specimens, and the consequent densities result in comparable deviator stresses at high strains.

With specimens compacted at MDD and OMC, significant increase in the deviator stress for all consolidation pressures is observed [Fig. 7.4(a) and Table 7.3]. At large strains, the specimens exhibit dilatant behaviour at most stress levels [Fig. 7.4(b)]. The stiffness of the specimens increases with the increase in the amount of compaction.

The stress paths for CU tests for undisturbed and remoulded specimens as well as for the compacted specimens are shown in Figs. 7.5 and 7.6 respectively. The resulting undrained strength parameter values are listed in Table 7.4.

The stress paths for the undisturbed specimens are initially steep and then they gradually curve to the left due to the formation of excess pore water pressure. Towards the end, slight strain-softening occurs until the stress paths hit the K_f line due to generation of excess pore water pressure. The stress paths for the remoulded specimens also show a similar trend except towards the end, where it either tends to travel up the K_f line or stops at the K_f line with the increase in consolidation pressure.

The stress path for the compacted specimens, on the other hand, behaves in a different way. The stress paths remain steep and inclined slightly towards the right until they hit the K_f line at the point of maximum principal effective stress ratio. At low confining pressure, the stress path simply moves up the K_f line indicating slight strain-softening with marginal reduction in pore water pressure. For higher consolidation pressures, the stress paths after touching the K_f line gradually curve away from the K_f line towards the right due to mild strain hardening of the material associated with gradual reduction of pore water pressure. The deviator stress, even after the maximum principal effective stress ratio is attained, keeps on increasing marginally.

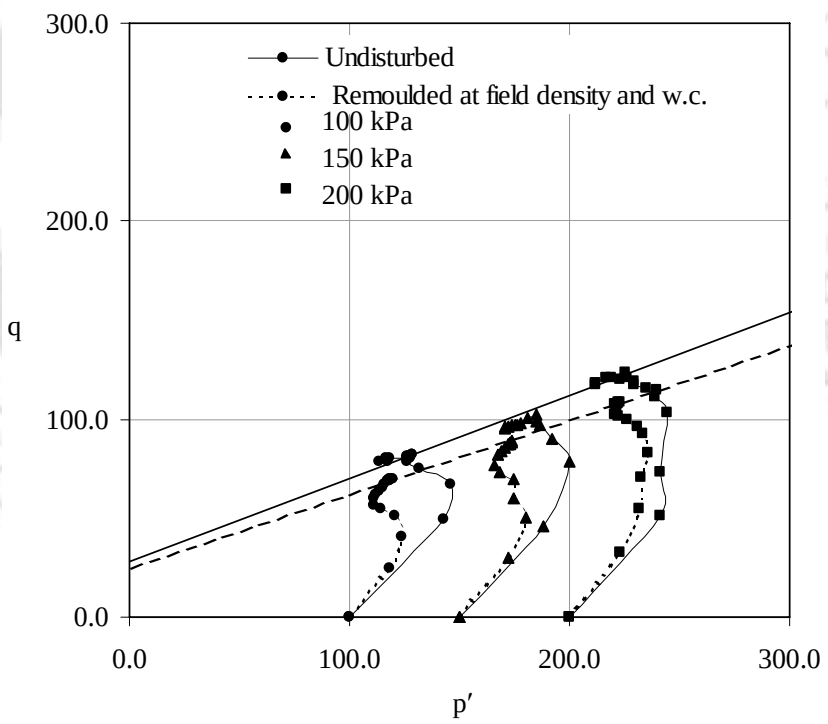


Figure 7.5 Effective stress paths from CU tests on undisturbed and remoulded (at field density and water content) specimens

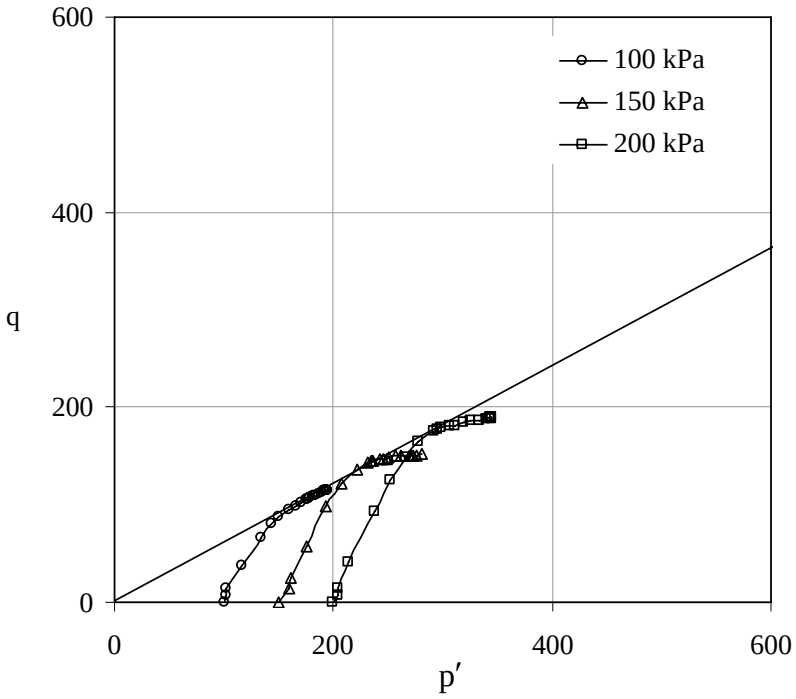


Figure 7.6 Effective stress paths from CU tests on compacted (at MDD) specimens

Table 7.4 Undrained strength parameter values for soil under different test conditions

Test condition	ϕ′ (degrees)	c′ (kPa)	α′ (degrees)	d′ (kPa)
Undisturbed	22.43	30	20.89	28
Remoulded (in-situ)	21.92	27	20.47	25
Compacted at MDD	37.34	—	31.24	—

The increase in deviator stress occurs due to increase in density, which increases the stiffness at low strain level and ductility of the material beyond the yield point.

In the case of lateritic soils, shear parameters are higher than those suggested by plasticity indices, with effective friction angles typically between 20° and 30° for lateritic clays, and between 30° to 40° for lateritic gravels (Townsend, 1985). The

effective friction angle for the undisturbed soil used for the present investigation (Table 7.4) is found to be 22.43˚, which is rather on the lower side of the range of values normally found for lateritic soils of tropical and other similar climatic regions (Table 2.4).

The experimental soil exhibits cohesion intercepts 30 and 27 kPa for undisturbed and remoulded specimens respectively. Vaughan (1985) highlighted that weak inter-particle bonds present in residual soils results in a definite c′ value. Sridharan (1988), while discussing the engineering properties of tropical soils also reported that inter-particle repulsive and attractive forces are significant for cohesion intercept. However, Wesley (1990) debated whether a real c′ value is necessarily related to the presence of bonds and reported that the c′ value exists even after remoulding.

The results of triaxial tests on the undisturbed soil and their presentation in stress paths show a behaviour that is more alike normally consolidated materials. Specimens remoulded at field density, on the other hand, exhibit a behaviour that more akin to “quasi-overconsolidated”. When specimens are compacted at MDD and OMC, their behaviour is similar to overconsolidated materials.

7.4.3 Behaviour of Soil-Fly Ash-Lime Mixes

The results of CU tests on 35FA mix with varying lime contents (CU_13 to 15, CU_16 to 18 and CU_25 to 27) compacted at MDD and OMC are presented in Figs. 7.7 to 7.9. In cases of the specimens without lime, the maximum principal effective stress ratio and peak deviatoric stress occur at the same strain. When lime is added, they yield significant difference, and the maximum principal effective stress ratio generally occurs at a strain smaller than the strain corresponding to peak deviator stress.

Since maximum principal effective stress ratio is referred to as the maximum obliquity of the failure plane, this criterion is used to define failure. The results of the above CU tests defined by this failure criterion are summarized in Table 7.5.

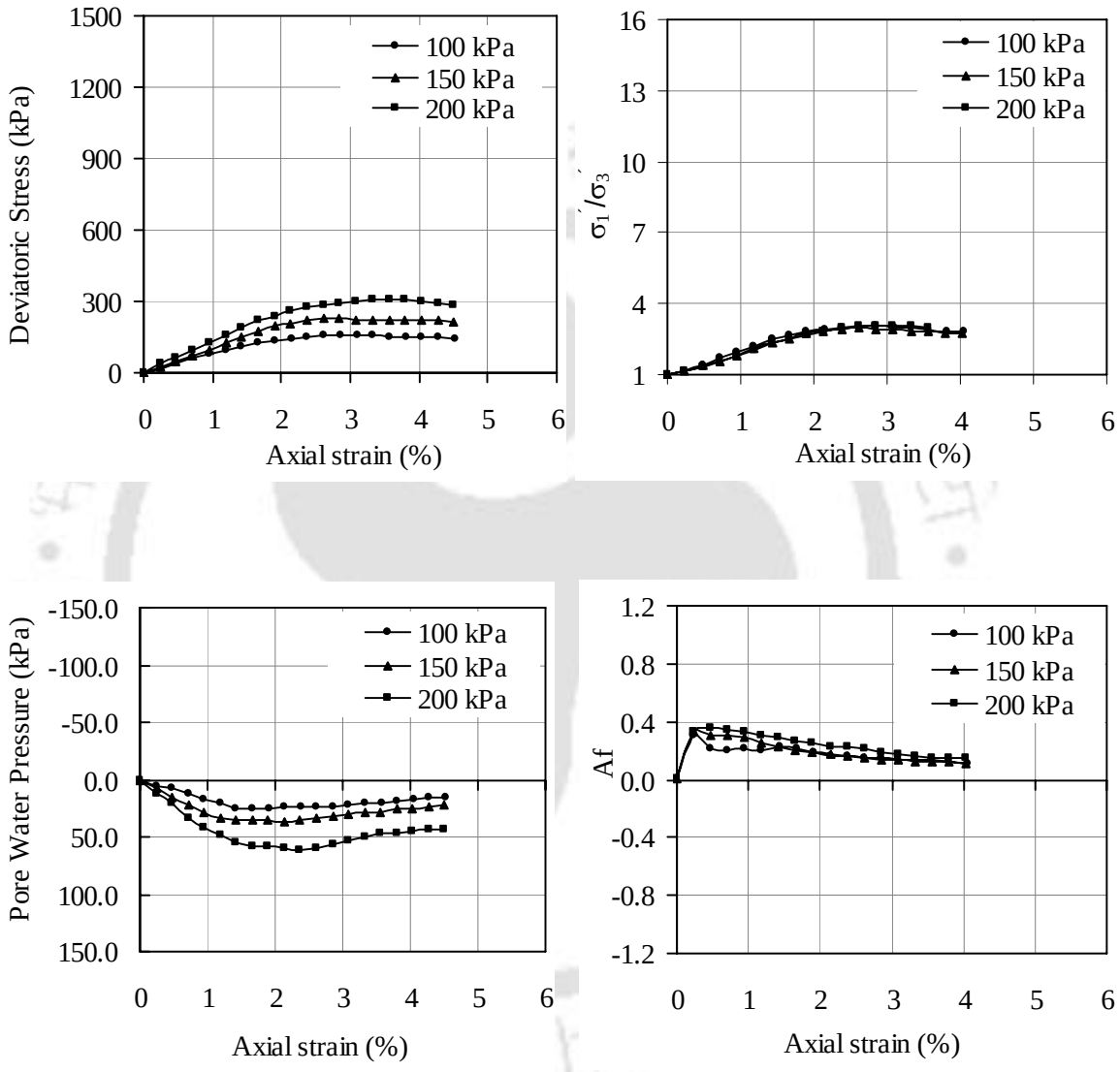


Figure 7.7 Results of CU tests (CU_13 to 15) performed on 35FA mix without the addition of lime.

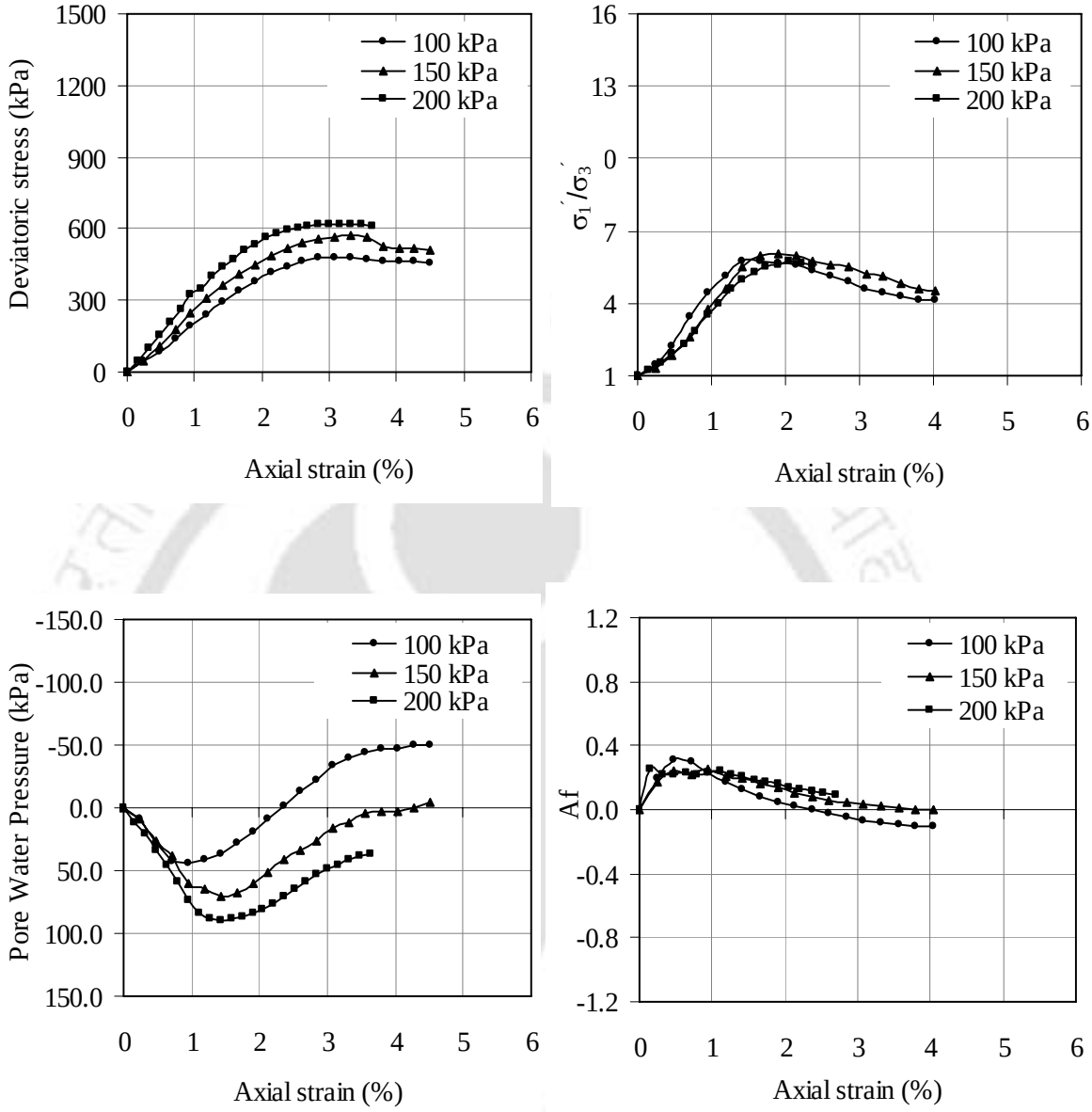


Figure 7.8 Results of CU tests (CU_16 to 18) performed on 35FA mix with 2% lime.

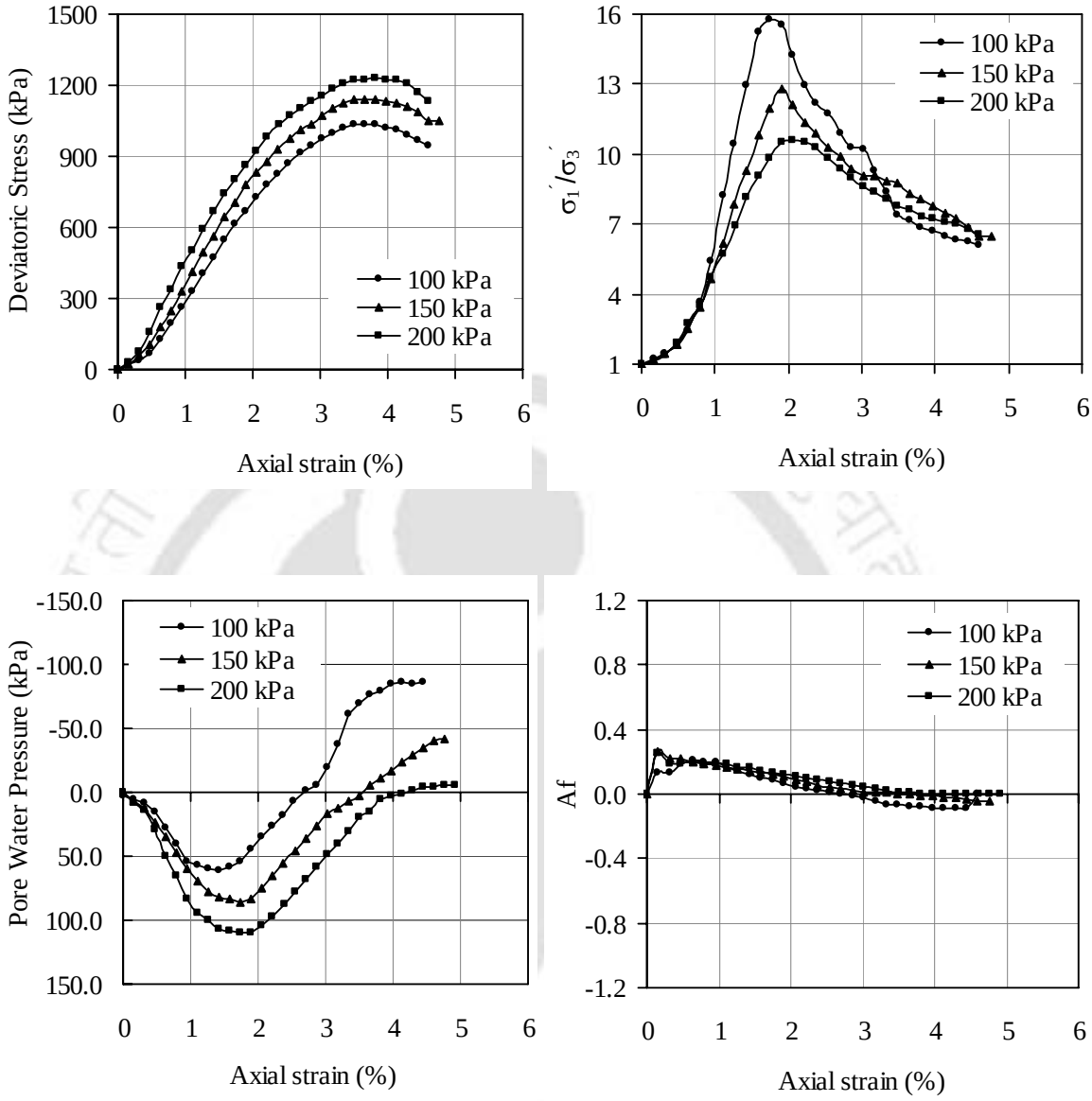


Figure 7.9 Results of CU tests (CU_25 to 27) performed on 35FA mix with 4% lime.

Table 7.5 Results of CU triaxial tests on 35FA mix with different lime contents.

Test no.	Lime content (%)	Effective confining pressure (kPa)	Axial strain at failure (%)	A-factor at failure	σ_1' at failure (kPa)	σ_3' at failure (kPa)	σ_1'/σ_3' at failure
CU_13	0	100	2.86	0.15	236	77	3.063
CU_14	0	150	2.61	0.15	345	116	2.98
CU_15	0	200	3.09	0.18	449	146	3.08
CU_16	2	100	1.66	0.08	410	071	5.75
CU_17	2	150	1.90	0.14	539	89	6.05
CU_18	2	200	2.06	0.14	634	119	5.74
CU_25	4	100	1.75	0.09	653	41	15.81
CU_26	4	150	1.91	0.11	843	66	12.78
CU_27	4	200	2.07	0.11	1017	96	10.56

Notes: A-factor = change in excess pore pressure/change in deviator stress
σ_1', σ_3' = principal effective stresses defined at maximum principal stress ratio

The deviatoric stress and pore water pressure-axial strain response obtained from the CU tests on 35FA mix for 100 kPa confining pressure with different lime contents is shown in Fig. 7.10. Specimens for these tests were compacted at optimum moisture content and maximum dry density and were cured for 28 days. It can be readily observed from the figure that addition of fly ash and lime significantly influences the overall behaviour of the material. Peak strength, stiffness, and brittleness of the soil show remarkable change as a consequence of the combined effects of fly ash and lime.

From the comparison of the curves, it can be noted that replacement of soil by 35% of fly ash with no lime addition does not affect the stress-strain characteristics much. The peak strength and strain at failure in fact is slightly reduced while the

brittleness is marginally altered accompanied by a noticeable change in the initial stiffness of the mixed material. Although the grain size distribution curves of the soil and fly ash are different, the replacement of soil by 35% fly ash causes only limited fundamental change in the mixture behaviour. In comparison, addition of lime causes a remarkable change in the behaviour of the mixed material.

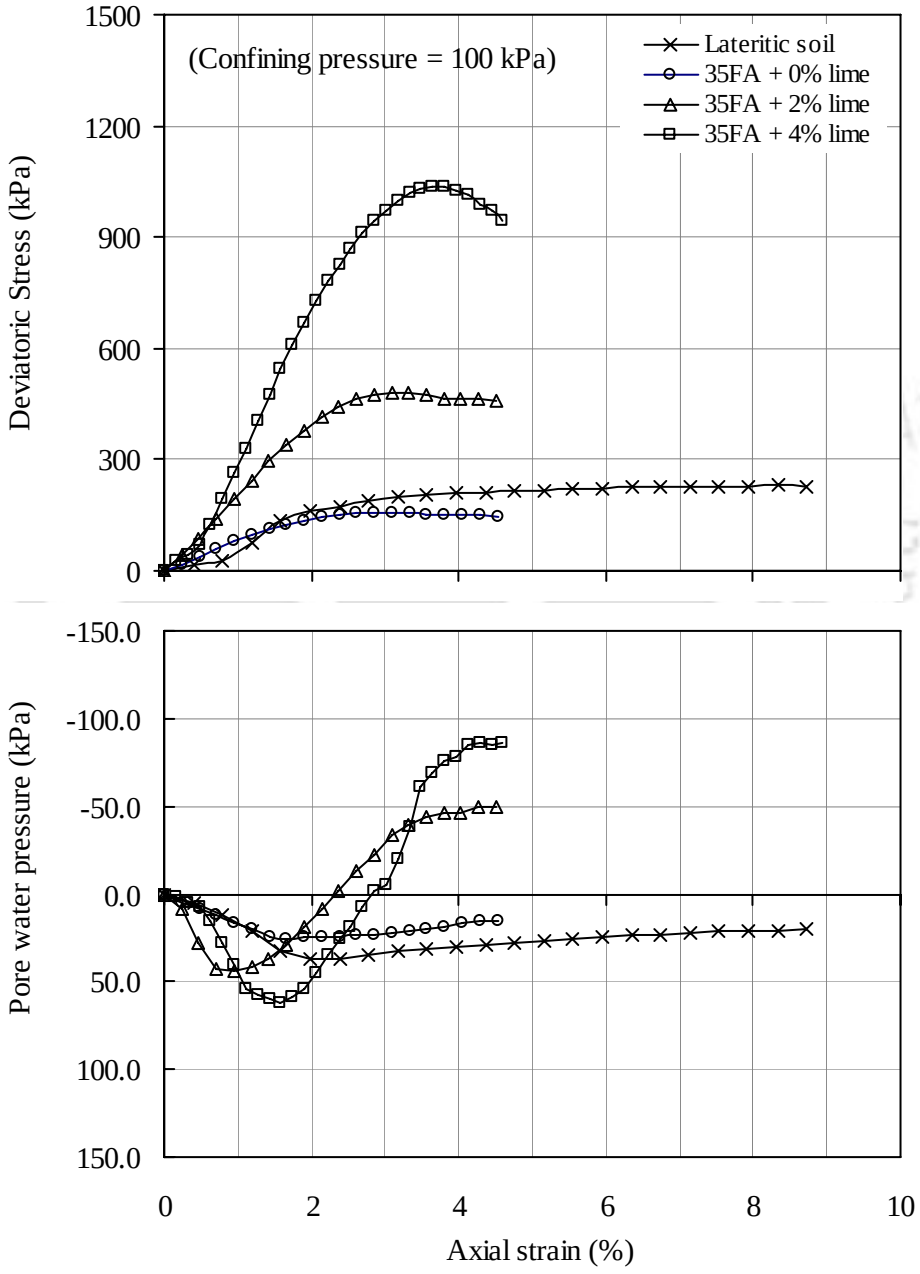


Figure 7.10 Stress-strain and pore pressure response of specimens compacted at MDD and OMC for 35FA mix with varying lime contents (Confining pressure = 100 kPa)

A comparison of the curves relating to compacted soil, soil-fly ash mix without lime and mixes containing 2% & 4% lime depicted in Fig. 7.10, shows that the peak strength and stiffness of the new material is significantly increased with the increase in amount of lime in the mix. Further, addition of fly ash and lime to the soil, transform the ductile nature of the soil into an extremely brittle one, accompanied by a marked reduction in the failure axial strain.

The pore pressure curves plotted in Fig. 7.10 show somewhat different behaviour for the untreated soil and soil treated with fly ash and lime. With the increase in strain, the pore pressure in the untreated soil initially increases and reaches the maximum value just before attaining the maximum effective principal stress ratio and then start declining gradually and remain on the positive side.

The soil treated with only fly ash exhibits similar behaviour except with a reduced peak pore pressure compared with the untreated soil. When the soil is treated with fly ash and lime, the pore pressure shows a rapid increase at the initial stage of loading and after attaining the peak value, also decreases sharply. The increased rate of development or fall of pore pressure in case of fly ash and lime treated specimens can be understood from the increased stiffness of the material developed due to cementation caused by the addition of lime. With further increase in strain, the pore pressure becomes negative and reaches the minimum near the peak deviator stress. Subsequently, the rate decreases as the soil presumably approaches an ultimate or critical state.

The above pore pressure behaviour can be interpreted from the point of relative contribution of the cohesive and frictional components of the stress-strain response. As loading started, first the cementation bonds are gradually broken that causes the localized structures to collapse. But as drainage is prevented, this results in an increase in the pore water pressure. With further increase in strain, inter-particle friction is

progressively mobilized that creates a tendency for the soil to dilate. As a result, the pore water pressure decreases as the soil approaches failure.

It can be noted from Fig. 7.10 that the large range of pore water pressure generated for the soil-fly ash-lime mixes depends on the quantity of lime added to the mixes. The magnitude of change is found to be greater when more lime is added to the mix. This can be attributed to the fact that as the material becomes more cemented and resistant to deformation, the tendency for both the initial compression and dilation increases and hence the effect on pore water pressure results.

The stress paths for the CU tests on specimens compacted at MDD and OMC for 35FA mix without lime are shown in Fig. 7.11. The stress paths remain steep and curve gradually towards the right until they hit the K_f line at the point of maximum principal effective stress ratio.

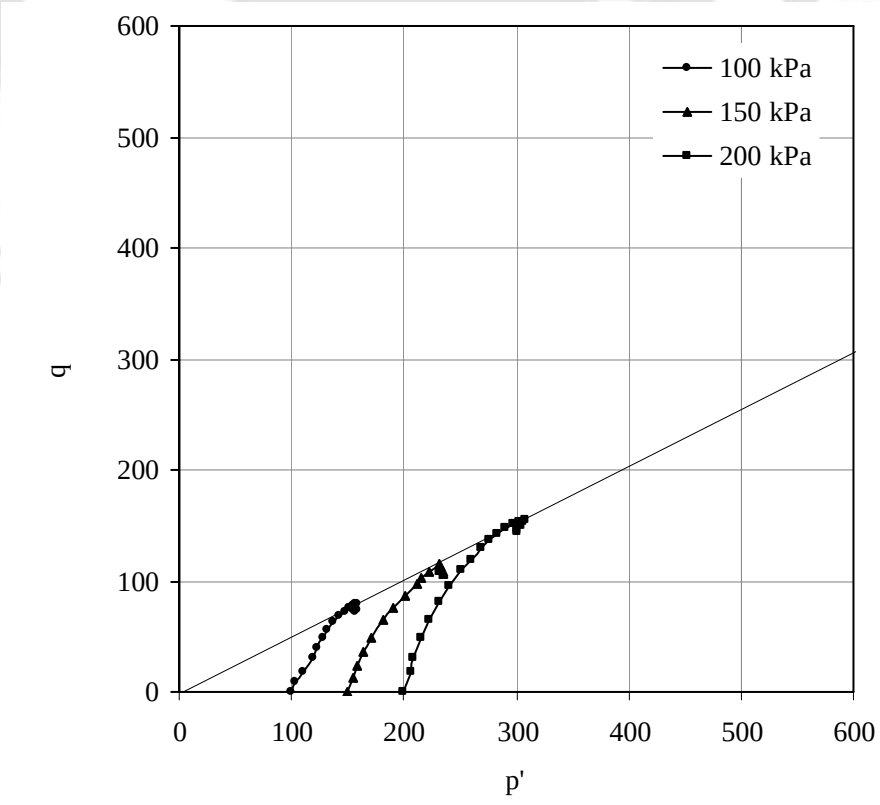


Figure 7.11 Stress paths from CU tests on specimens compacted at MDD and OMC for 35FA mix with no lime

The peak deviatoric stress and maximum principal effective stress ratio occur at the same strain level. Figures 7.12 & 7.13 show the stress paths for the CU tests on specimens compacted at MDD and OMC for 35FA mix with 2% and 4% lime respectively. The stress paths remain steep and inclined slightly towards the right until they are just about to hit the K_f line and slowly converge with the K_f line at the point of maximum principal effective stress ratio. With the increase in strain, the stress paths move up the K_f line and gradually move away from the K_f line to turn back after reaching the peak deviatoric stress. The deviator stress, even after the maximum principal effective stress ratio is attained, keeps on increasing till it reaches the peak.

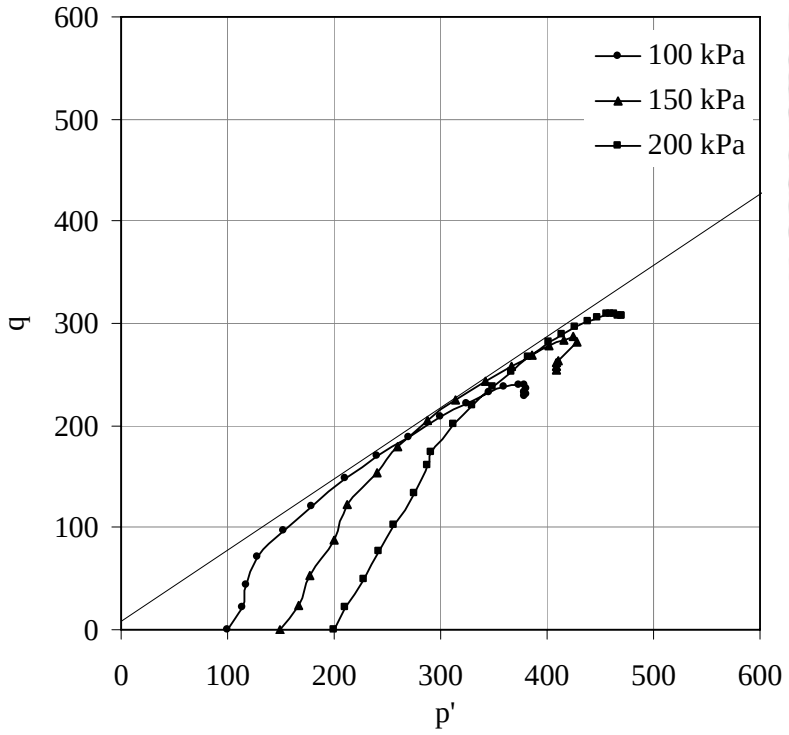


Figure 7.12 Stress paths from CU tests on specimens compacted at MDD and OMC for 35FA mix with 2% lime

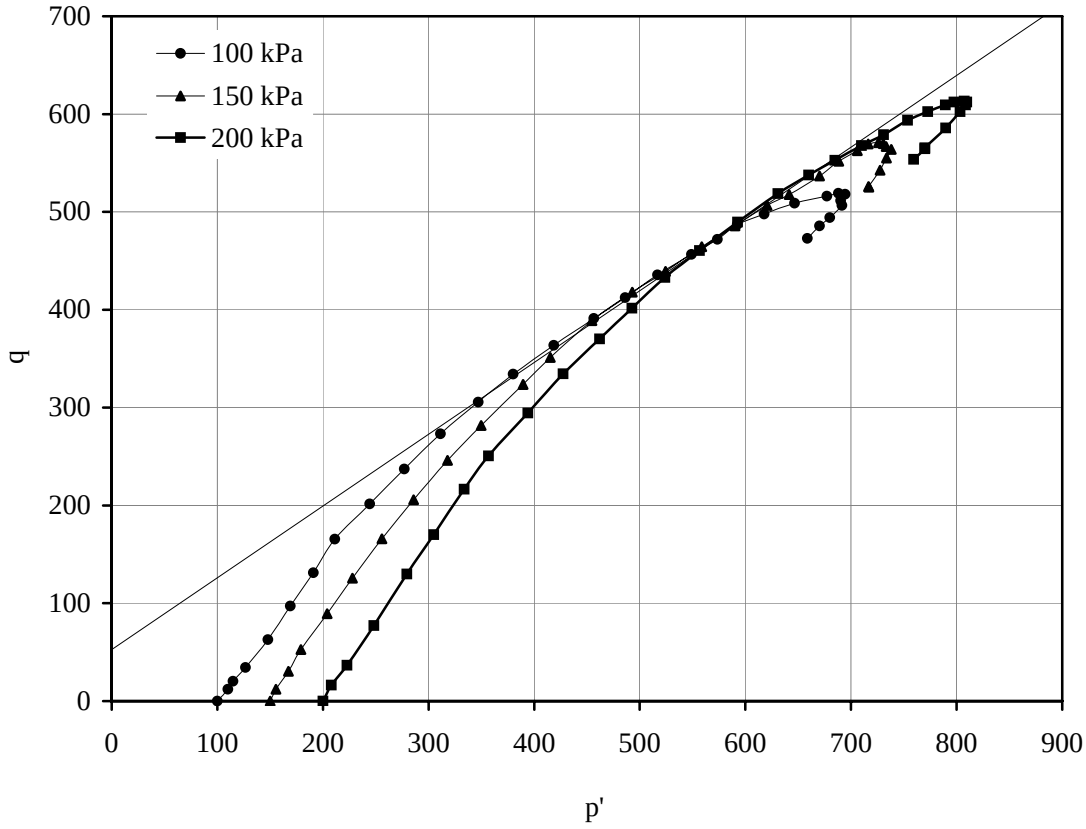


Figure 7.13 Stress paths from CU tests on specimens compacted at MDD and OMC for 35FA mix with 4% lime

The resulting effective strength parameter values are listed in Table 7.6. A comparison of the peak strength envelopes of the soil and the 35FA mix with varying lime contents is presented in Fig. 7.14. It can be observed from the figure that treatment of soil with fly ash without lime lowers the peak friction angle from 37.6º to 33.4º.

Table 7.6 Peak strength parameter values of 35FA mix compacted at MDD and OMC with varying lime contents

Lime contents (%)	ϕ' (degrees)	c' (kPa)	α' (degrees)	d' (kPa)	Secant Young's modulus for ε_a = 1% (MN/m^2)
No lime	33.4º	0	28.8º	0	7.5 – 13.5
2% lime	39.6º	29	32.5º	22	21.5 – 34.6
4% lime	46.8º	80	36.1º	55	43.1 – 54.4

Addition of 2% lime to the soil-fly ash mix again improves the peak friction angle to 39.6º with the emergence of a peak cohesion intercept of 29 kPa after 28 days of curing, possibly resulting from cementation due to the onset of pozzolanic reactions. With the increase in percent lime addition to 4%, the peak friction angle further increases to 46.8º accompanied by the development of a large cohesion component of 80 kPa indicating even stronger pozzolanic activity in the mix. The results clearly demonstrate that simple addition of fly ash to the soil lowers the peak strength parameters and indicate the role of lime in the improvement process.

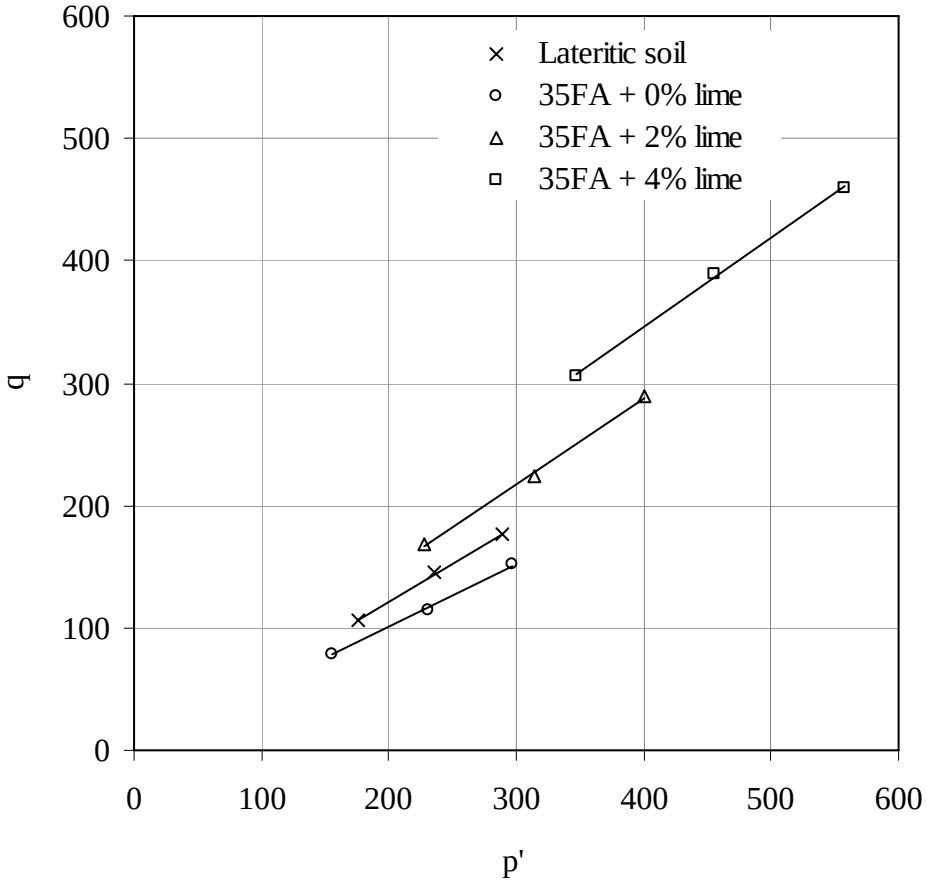


Figure 7.14 Peak strength envelopes for specimens compacted at MDD and OMC for the soil and 35FA mix with different lime contents after 28 day of curing

Addition of 2% lime to the soil-fly ash mix, which is below the ICL value (3%), is most likely to have resulted in the change in texture of the material due to cation exchange and/or flocculation-agglomeration mechanism, accompanied by weak pozzolanic activity essentially caused by the lime stabilization. This is well reflected by the increase in peak friction angle and appearance of a small peak cohesion intercept for the soil-fly ash mix upon the addition of 2% lime. The increase in peak cohesion intercept along with the peak friction angle with the addition of 4% lime to the mix (which is above the ICL value), on the other hand, clearly reflects the increase in cementation caused by the pozzolanic reactions.

A comparison of the secant deformation modulus of the soil and 35FA mix with varying lime contents, plotted against the axial strain on a logarithmic scale for a confining pressure of 100 kPa is presented in Fig. 7.15. The values of secant deformation modulus of the mixes presented in Table 7.6 pertain to 1% axial strain and confining pressures ranging from 100 to 200 kPa.

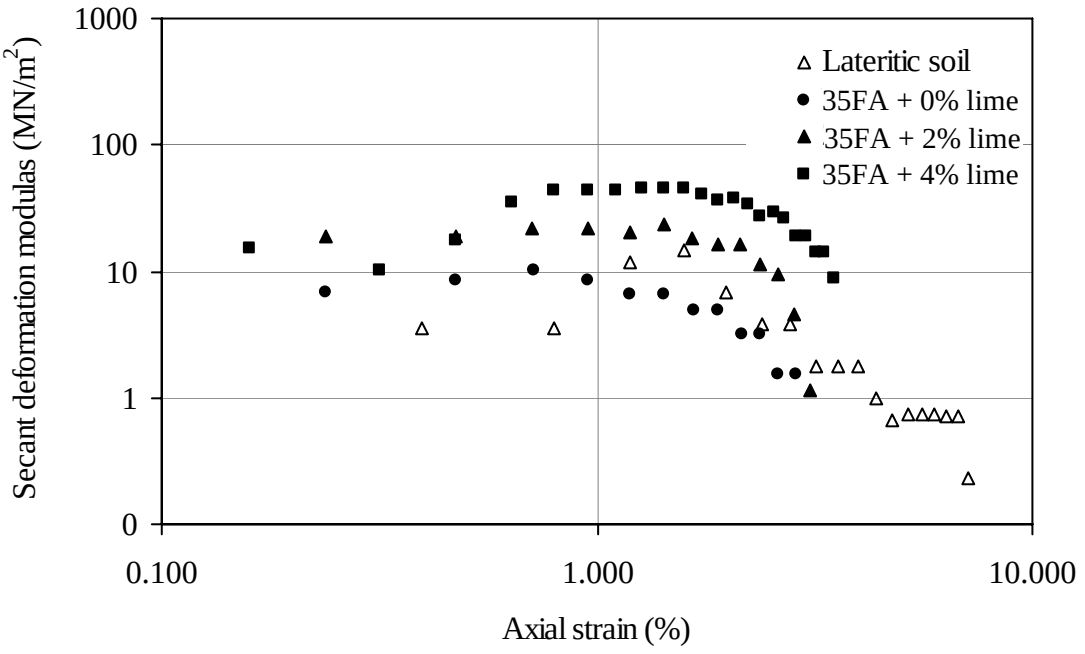


Figure 7.15 Secant deformation modulus versus axial strain for specimens compacted at MDD and OMC for 35FA mix with varying lime contents (confining pressure of 100 kPa)

At the initial stage of loading, deformation modulus of compacted soil is less than that of the soil-fly ash mix without lime. But, with increasing strain, the value tends to increase and exhibits slightly higher value than the soil-fly ash mix. At 1% strain, the values of the moduli for both compacted soil and the soil-fly ash mix are found to be similar with an average value of 10.7 MN/m^2. The addition of lime to the soil-fly ash mix significantly increases the modulus of deformation. Addition of 2% lime increases the average value of the modulus to 28.0 MN/m^2 and with the addition of another 2% lime, it further increases to 49.3 MN/m^2.

The shear strength behaviour observed in the present investigation was compared to the reported triaxial test results for residual and other stabilised soils (since comparable data on lime and lime-fly ash stabilised *lateritic* soils are not reported in literature) and found to be qualitatively in agreement [Townsend (1985), Consoli et al. (1998, 2001)]. Table 7.7 summarizes a comparison of typical deformation and strength characteristics of a few lime and fly ash stabilized soils [modified after Consoli et al. (2001)].

Table 7.7 Typical deformation and strength characteristics of lime-fly ash stabilized soils

Material	Peak friction angle (degrees)	Peak cohesion intercept (kPa)	Deformation modulus at ε_d = 0.1% (MN/m^2)	Reference
Lime Stabilized fine grained soils	25º-35º	-	-	Brown (1996)
Lime Fly ash stabilized gravels	49º-53º	-	-	Brown (1996)
Carbide Lime stabilized sand (at 28 days)	38º	42	69-88	Consoli et al. (2001)
Carbide Lime-fly ash stabilized sand (at 28 days)	46º	122	304-391	Consoli et al. (2001)

A comparison with the values obtained in the present study (Table 7.6) shows that the peak friction angle matches with the values presented in Table 7.7, while the cohesion intercept and deformation modulus fall somewhat below the typical range. The reason for this may be attributed to the varying degree of cementation and the resulting stiffness attained by the soil after lime-fly ash treatment. However, caution should be exercised while comparing the values, as strength envelopes are not actually linear and the values of friction angle and cohesion intercepts will depend on the confining stress range considered. Further, the values of deformation modulus are strongly influenced by the shear strain level, which also limits the scope for comparison of the values.

7.4.4 Effects of Compaction Condition on Shear Strength

The results of CU tests on 35FA mix with 2% lime (CU_19 through CU_24) and 4% lime (CU_28 through CU_33) under variable compaction conditions are presented in Figs. 7.16 to 7.19. Similar to the specimens prepared at their respective MDD and OMC, specimens of 35FA with 2% and 4% lime, compacted on dry and wet side of the optimum moisture also attained the maximum principal effective stress ratio at a strain lower than the peak deviatoric stress. Hence, the same failure criterion was adopted and the results are summarized in Table 7.8.

The deviatoric stress and pore pressure-axial strain responses, obtained for 35FA mix with 2% and 4% lime under different compaction conditions and cured for 28 days, are presented in Figs. 7.20 & 7.21. The curves shown in the figures relate to confining pressure of 100 kPa. Similar patterns were noticed for confining pressures of 150 and 200 kPa and hence these curves are not presented for discussion.

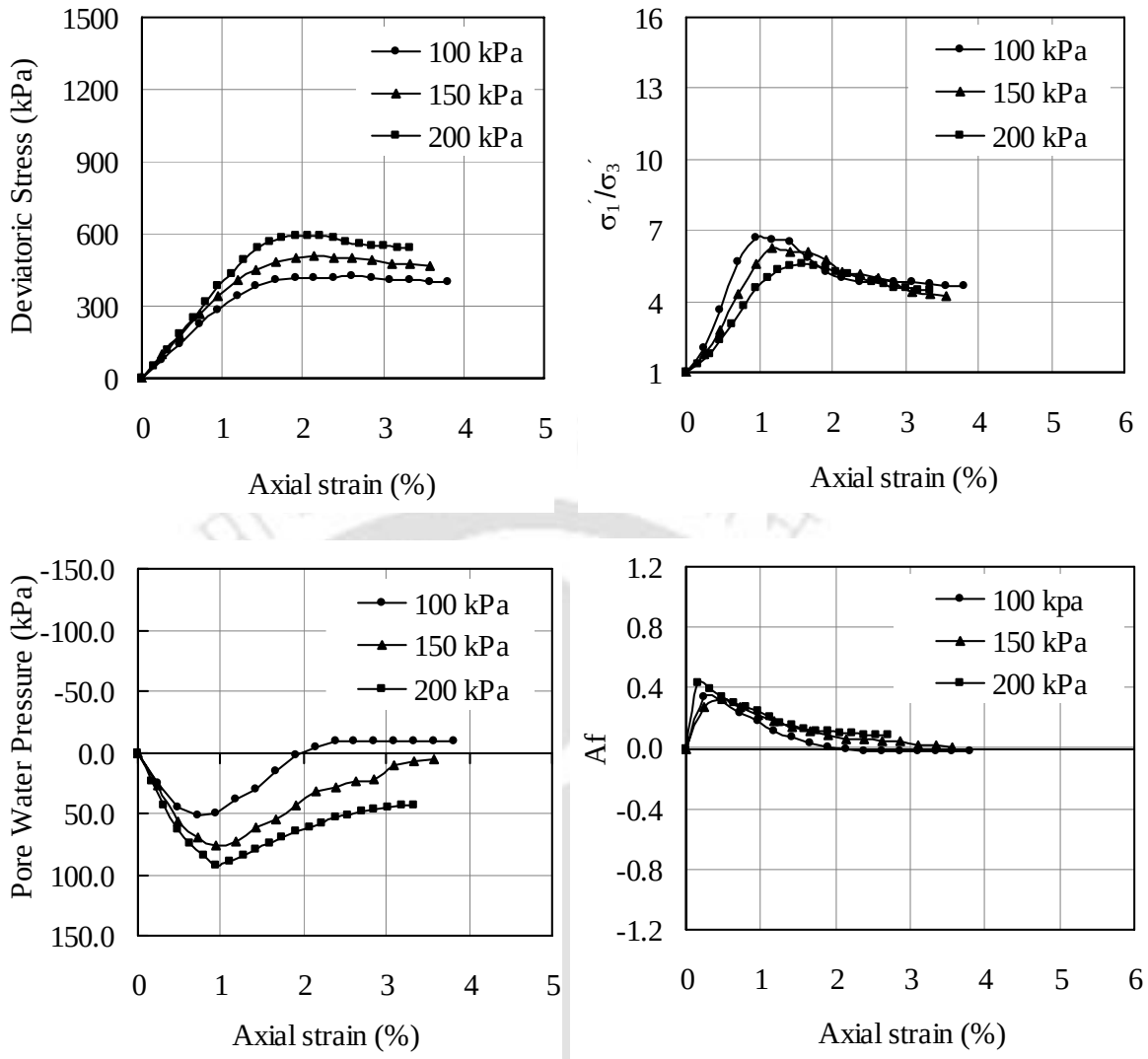


Figure 7.16 Results of CU test performed on 35FA mix with 2% lime for compaction condition - dry side of optimum moisture content.

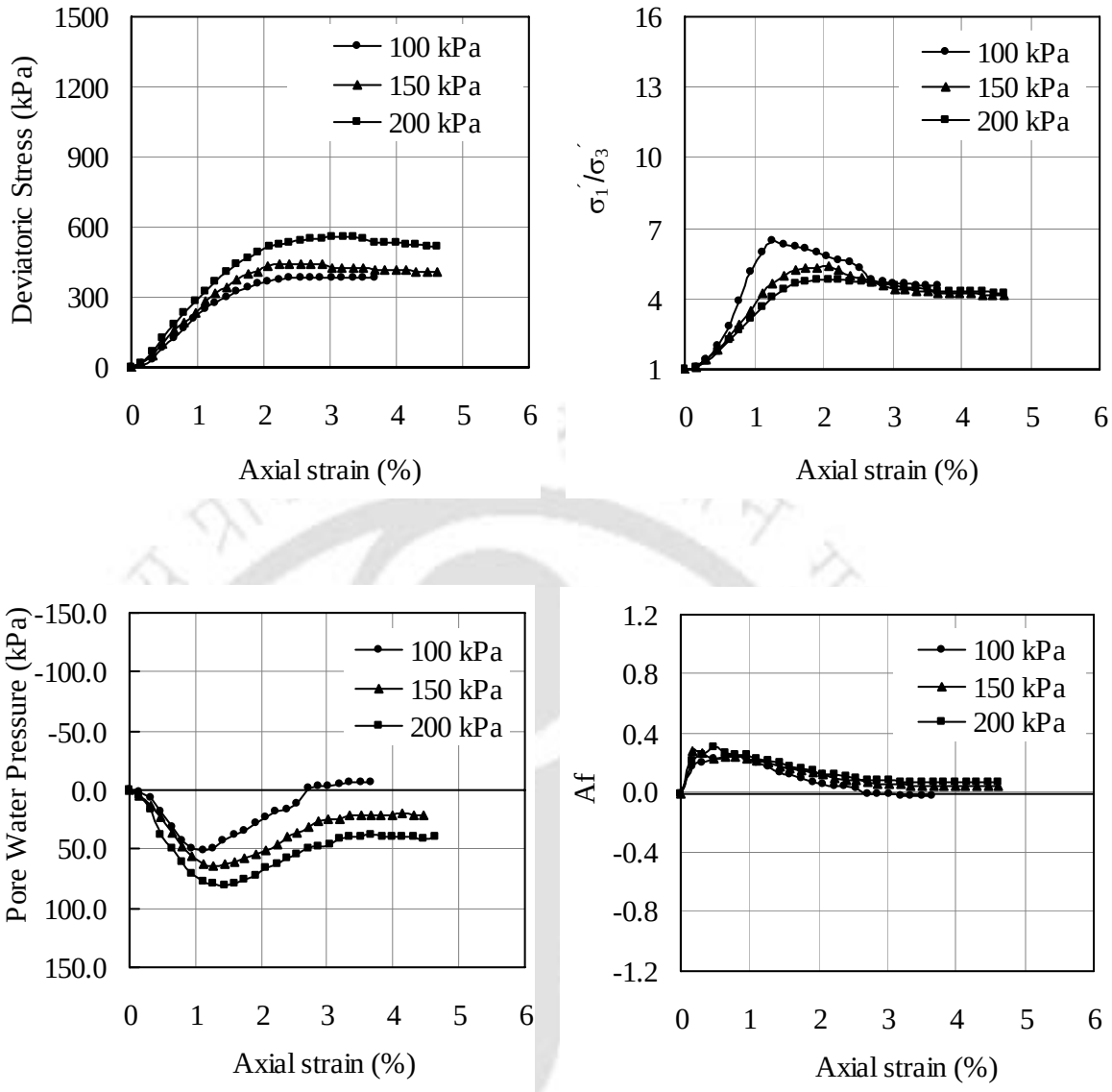


Figure 7.17 Results of CU test performed on 35FA mix with 2% lime for compaction condition - wet side of optimum moisture content.

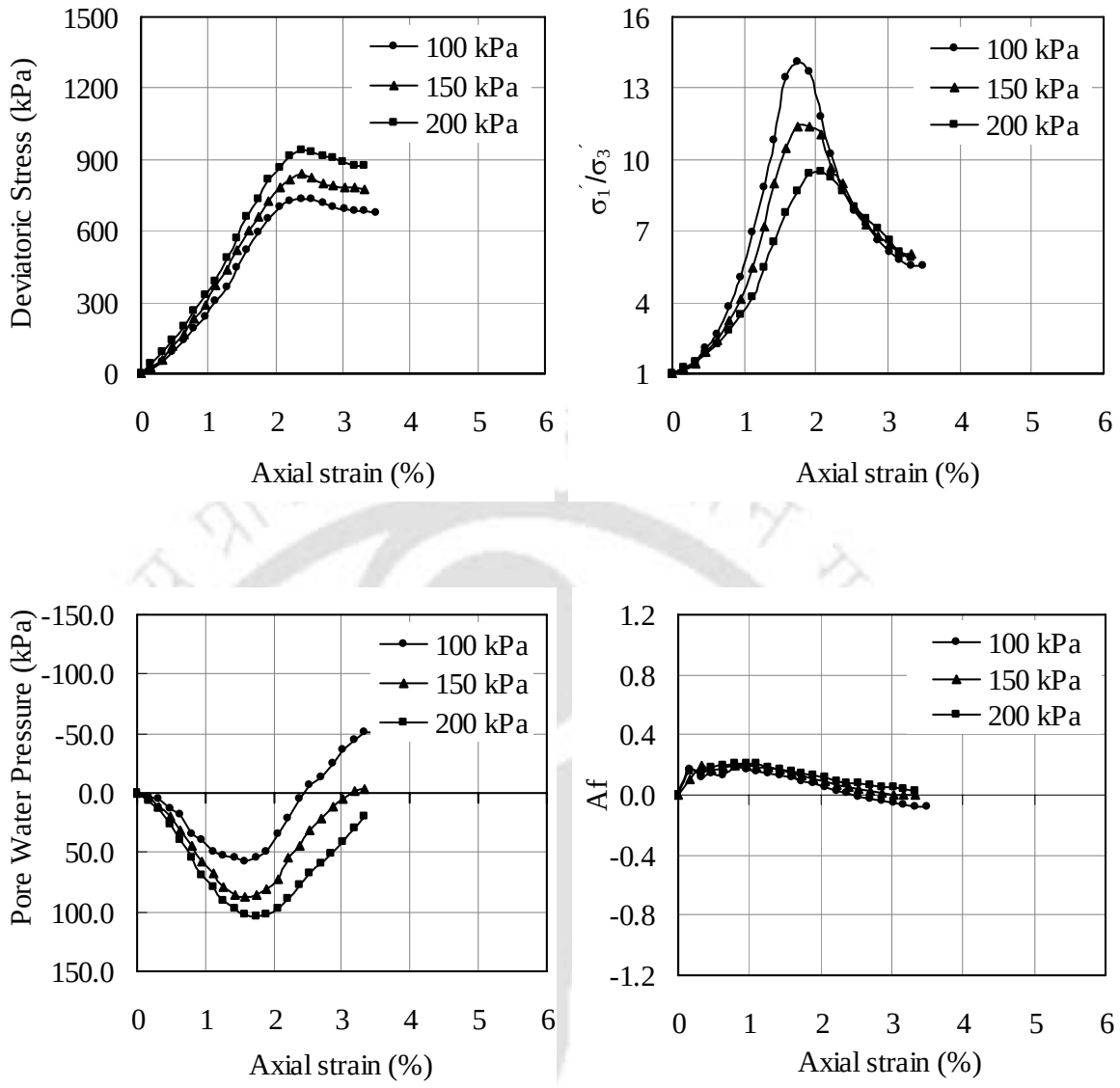


Figure 7.18 Results of CU test performed on 35FA mix with 4% lime for compaction condition - dry side of optimum moisture content.

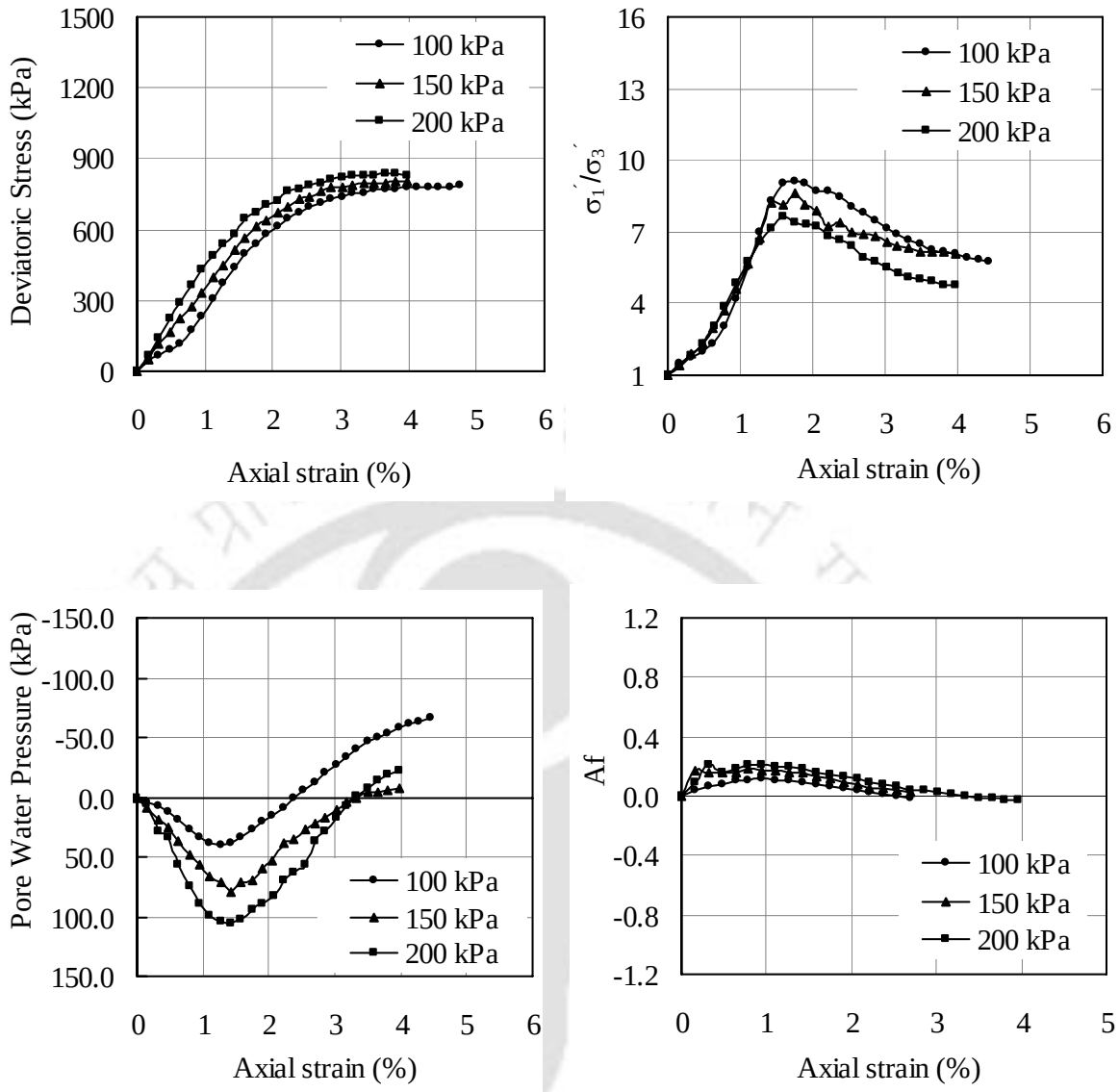


Figure 7.19 Results of CU test performed on 35FA mix with 4% lime for compaction condition - wet side of optimum moisture content.

Table 7.8 Results of CU tests for 35FA mix with 2% lime under different compaction conditions

Test	Compaction condition	Effective confining pressure (kPa)	Axial strain at failure (%)	A-bar at failure	σ_1' at failure (kPa)	σ_3' at failure (kPa)	σ_1'/σ_3' at failure
CU_19	Dry side	100	1.91	0.11	404	61	6.63
CU_20		150	1.43	0.14	541	89	6.01
CU_21		200	1.43	0.15	666	120	5.51
CU_22	Wet side	100	1.27	0.18	329	51	6.45
CU_23		150	2.07	0.12	530	98	5.40
CU_24		200	2.07	0.13	647	133	4.85

Notes: A-bar = change in excess pore pressure/change in deviator stress
σ_1', σ_3' = principal effective stresses defined at maximum principal stress ratio

Table 7.9 Results of CU test for 35FA mix with 4% lime under different compaction conditions

Test	Compaction condition	Effective confining pressure (kPa)	Axial strain at failure (%)	A-bar at failure	σ_1' at failure (kPa)	σ_3' at failure (kPa)	σ_1'/σ_3' at failure
CU_28	Dry side	100	1.75	0.09	635	45	14.10
CU_29		150	1.91	0.11	800	70	11.42
CU_30		200	1.75	0.11	969	102	9.50
CU_31	Wet side	100	1.75	0.06	608	67	9.12
CU_32		150	1.75	0.12	690	80	8.66
CU_33		200	1.59	0.16	741	97	7.64

Notes: A-bar = change in excess pore pressure/change in deviator stress
σ_1', σ_3' = principal effective stresses defined at maximum principal stress ratio

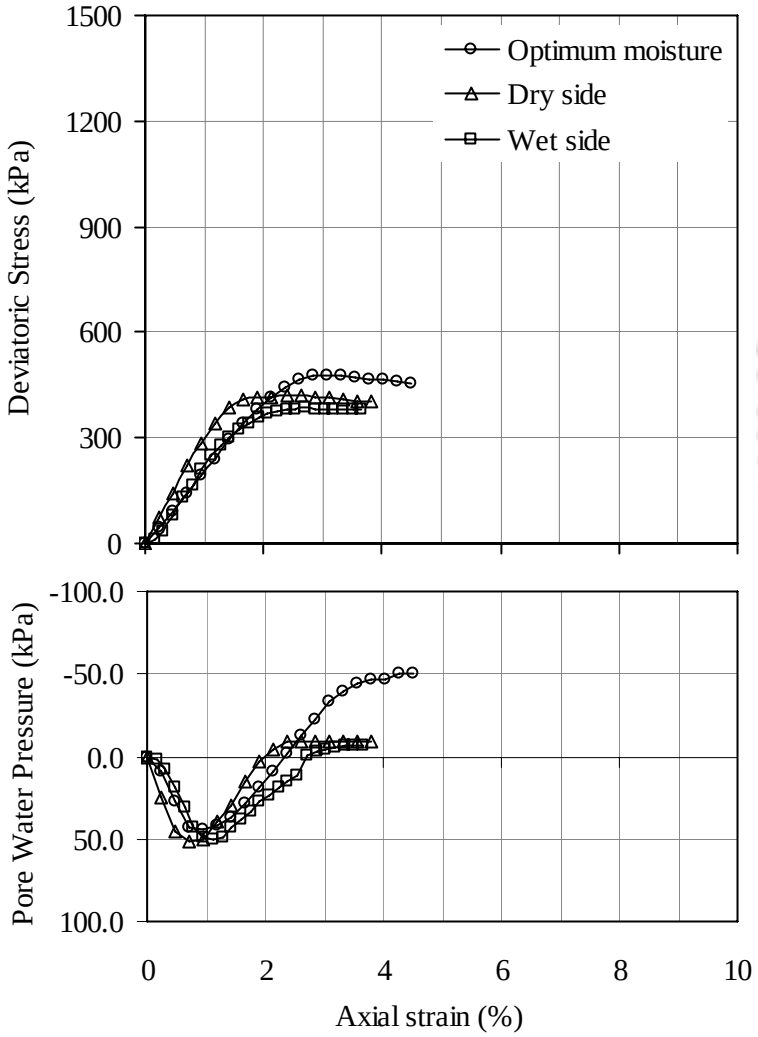


Figure 7.20 Stress-strain and pore pressure response of 35FA mix with 2% lime for all compaction conditions

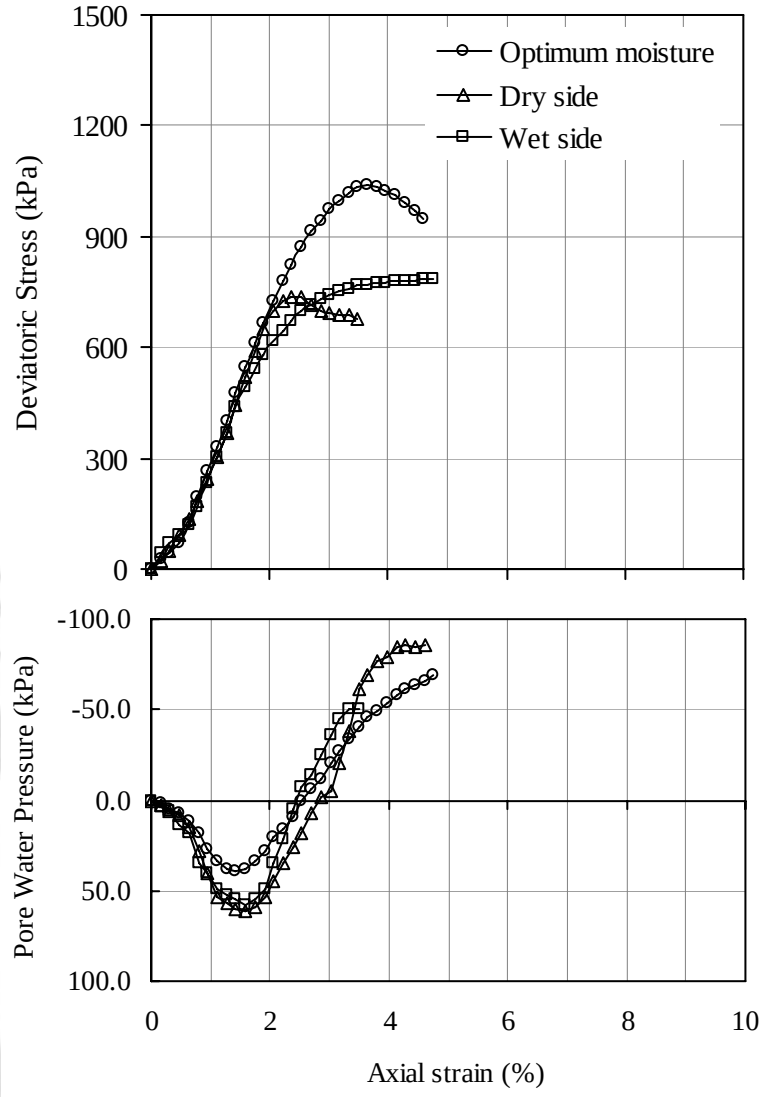


Figure 7.21 Stress-strain and pore pressure response for 35FA mix with 4% lime for all compaction conditions

It can be observed from the Fig. 7.20 & 7.21 that, except for a marginal increase in case of specimens compacted at dry side of optimum with 2% lime, the stiffness of the material does not change significantly due to the variation of compaction conditions. The above observations indicate that structures imparted by compaction have limited influence on the stiffness of the modified material, particularly at higher lime contents.

Changes in compaction condition and the resulting material structure, on the other hand, significantly influence the peak deviator stress and brittleness as shown in

Fig. 7.20 & 7.21. The peak deviator stress, for specimens compacted on both dry as well as wet side of the optimum moisture, is reduced with wet side showing significant reduction in peak strength. The brittleness remained somewhat similar for specimens compacted on dry of optimum moisture while, it decreases in case of specimens compacted on wet side of optimum moisture. The pore pressure-axial strain response of both the soil-fly ash-lime mixes are found to be the same irrespective of the compaction condition.

Figures 7.22 & 7.23 show the stress paths for the CU tests on specimens compacted on dry and wet of optimum moisture for 2% lime. The stress path generated under the same compaction condition, when lime content was increased to 4%, is illustrated in Figs. 7.24 & 7.25. No significant change in the pattern of the stress paths is observed due to the variation in compaction condition except that they hit the K_f line at lower strain for specimens compacted dry of optimum, compared to the specimens compacted at wet of optimum. The peak strength parameters for the soil-fly ash-lime mixes under different compaction conditions were computed using equations 7.1 & 7.2 and the values are listed in Table 7.10. A comparison of the peak strength envelopes of the soil-fly ash-lime mixes under different compaction conditions is depicted in Fig. 7.26.

Data presented in Tables 7.6 & 7.10 and Fig. 7.26 show that the effects of variations in compaction condition are considerably more on the wet side compared to the dry side of the optimum moisture content. The peak friction angle, in general, tends to reduce. The reduction is marginal for specimens compacted dry of optimum, while, significant for specimens compacted on wet side of optimum moisture, particularly with high lime content. It can be noted that the peak cohesion intercept does not significantly vary with the variation of compaction condition.

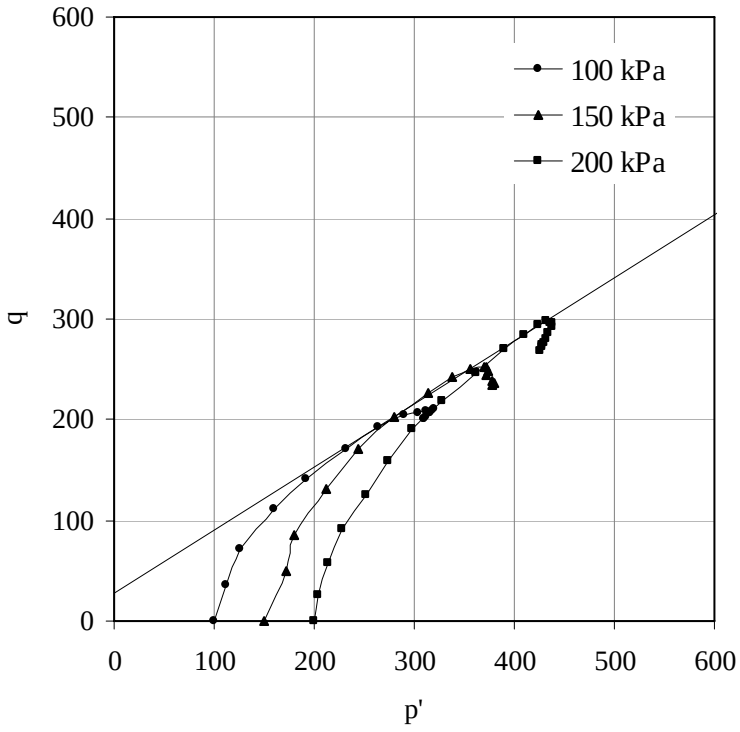


Figure 7.22 Stress paths from CU tests performed on specimens compacted on dry side of optimum moisture content for 35FA mix with 2% lime

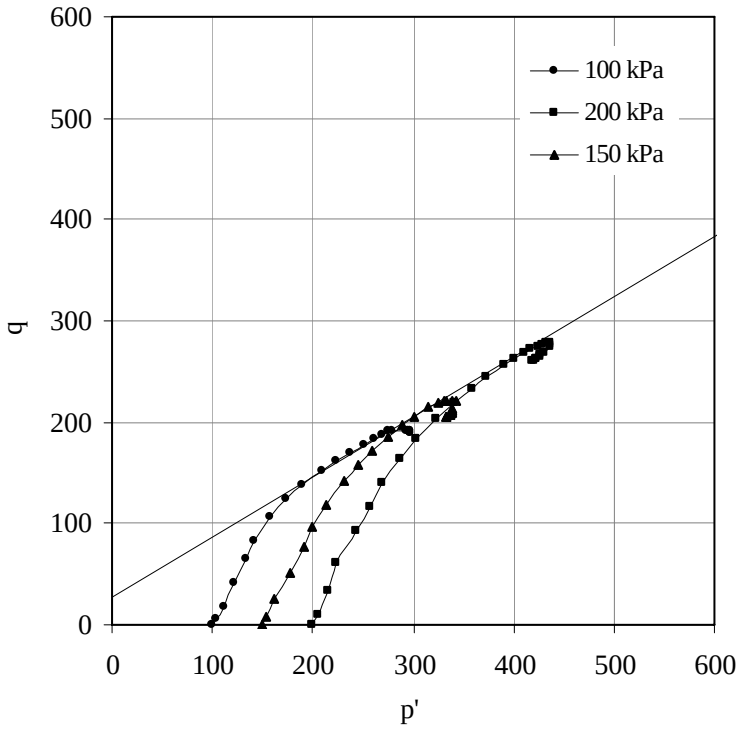


Figure 7.23 Stress paths from CU tests performed on specimens compacted on wet side of optimum moisture content for 35FA mix with 2% lime

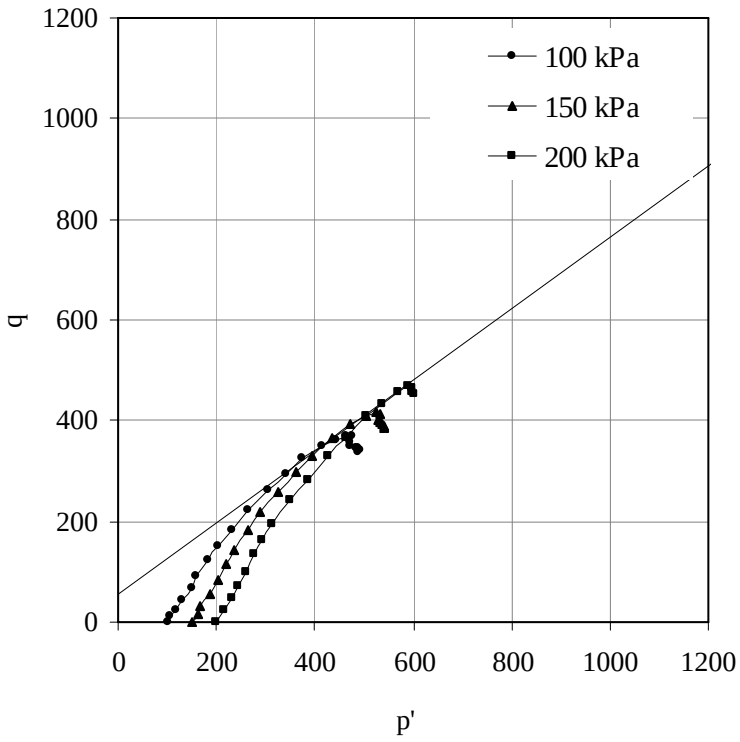


Figure 7.24 Stress paths from CU tests performed on specimens compacted on dry side of optimum moisture content for 35FA mix with 4% lime

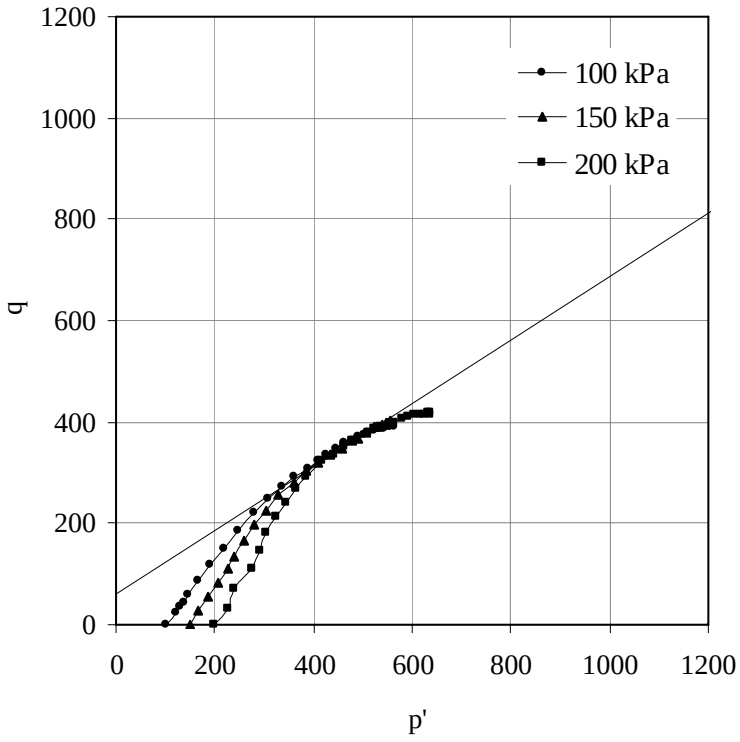


Figure 7.25 Stress paths from CU tests performed on specimens compacted on wet side of optimum moisture content for 35FA mix with 4% lime

Table 7.10 Peak strength parameter values for the 35FA mix with varying compaction conditions and lime contents

Lime contents (%)	ϕ' (degrees)	c' (kPa)	α' (degrees)	d' (kPa)	Secant Young's modulus for $\varepsilon_a = 1\%$ (MN/m^2)
Dry side/ 2% lime	38.56º	34	31.93º	25	25.3- 40.5
Wet side/ 2% lime	36.24º	30	30.59º	24	24.8 – 30.0
Dry side/ 4% lime	44.93º	78	35.23º	55	36.5 – 41.1
Wet side/ 4% lime	38.68º	77	32.00º	60	34.6 – 39.1

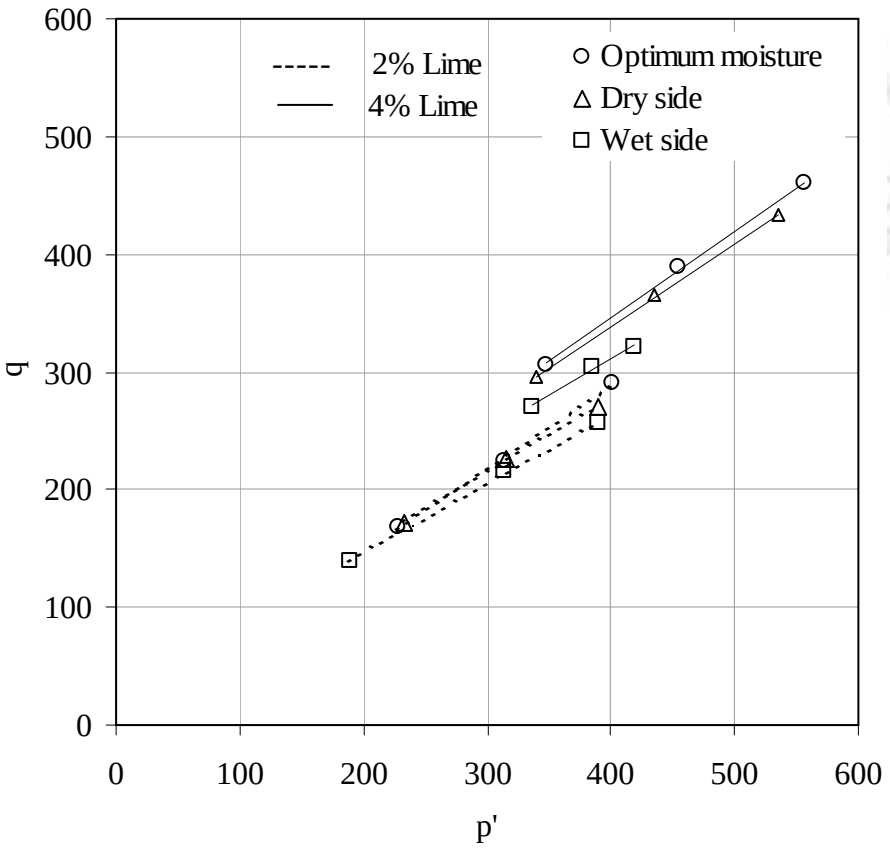


Figure 7.26 Peak strength envelopes for 35FA mix with 2% and 4% lime under different compaction conditions.

For the addition of 2% and 4% lime, specimens compacted at optimum moisture content show peak friction angle of 39.6º and 46.8º respectively. The cohesion intercepts for the same compaction condition are 29 kPa and 80 kPa respectively.

In case of 2% addition of lime, compaction on dry side marginally reduces the peak friction angle to 38.6º, while the cohesion slightly increases to 34 kPa. When the samples were compacted on wet side, friction angle was further reduced to 36.24º, but no significant change was noticed for cohesion intercept (30 kPa). In case of 4% lime, marginal reduction of peak friction angle from 46.8º to 44.9º is noticed when specimens are compacted on dry side. When the specimens are compacted on wet side, considerable reduction in friction angle from 46.8º to 38.68º is observed. However, no significant change is noted for cohesion intercepts due to the variation in compaction condition and it remains within the range from 78 to 80 kPa.

Consoli et al. (2001), on the basis of investigation on stress-strain behaviour of sandstone derived residual soil, carbide lime and fly ash mixes, reported maximum strength and stiffness for specimen compacted at dry side of the optimum moisture content at the end of 28 days of curing. They attributed the higher strength and stiffness to increased pozzolanic activity due to lower water-to-binder ratio in dry side of optimum. They also emphasized the role of water-to-binder ratio in determining the efficiency of a soil-fly ash-lime matrix, similar to its role prevalent in concretes and mortars.

The present investigation, however, shows that for lateritic residual soil mixed with fly ash and lime, the maximum strength is realised for specimens compacted at optimum moisture content. Also, the structure imparted by compaction has limited effect on the stiffness of the specimens and development of cementation, as indicated by little or no variation of the cohesion intercept and less variation in stiffness (Table 7.10 and Figs.7.20 & 7.21). This is possibly due to the fact that the soil-fly ash mixes

with 2% and 4% lime have a low dry unit weight (1.37 and 1.34 gm/cm^3, respectively) and their variation with change in compaction condition is also less. Also, the mixes 35FA with 2% and 4% lime exhibit considerably higher optimum moisture contents (27.33% and 28.51% respectively) and specimens prepared with the mix show a high initial degree of saturation (about 84%). Compaction on the dry side of optimum slightly decreases the degree of saturation to about 65-70%, while compaction on the wet side of optimum only marginally increases the degree of saturation to about 86-87%. Apparently, this results in less variation of required moisture for cementation reaction within the matrix of the modified material, also causing less variation of water-to-binder (fly ash and lime) ratio at different compaction conditions resulting in less variation in cohesion intercept.

Figures 7.27 and 7.28 illustrate the plots on a logarithmic scale for secant deformation modulus versus axial strains, obtained for the soil-fly ash-lime mixes pertaining to the confining pressure of 100 kPa and tested under different compaction conditions. The values of secant deformation modulus, presented in Table 7.10, were obtained at an axial strain of 1% and pertain to confining stresses ranging from 100 to 200 kPa.

It can be seen from Fig. 7.27 and 7.28 that variation in compaction conditions, in general, does not cause significant variation of the secant deformation modulus of the material. Throughout the entire analysed axial strain range, only the initial and final stages of axial strains show some amount of variation between the moduli of the specimens caused by the difference in compacted conditions without showing any specific trend. At the intermediate stages of axial strain, variation between the moduli gradually reduces and it is observed that the moduli of the specimens compacted on optimum moisture content are slightly higher than the moduli observed for the specimens compacted on dry or wet side.

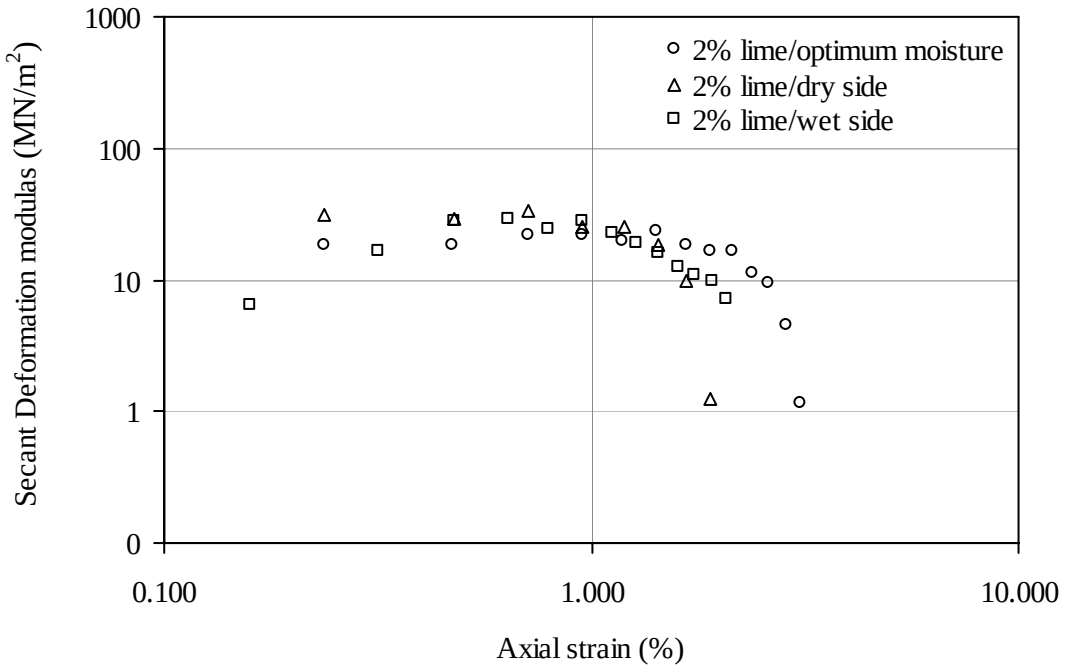


Figure 7.27 Secant deformation modulus versus axial strain for 35FA mix with 2% lime under different compaction conditions

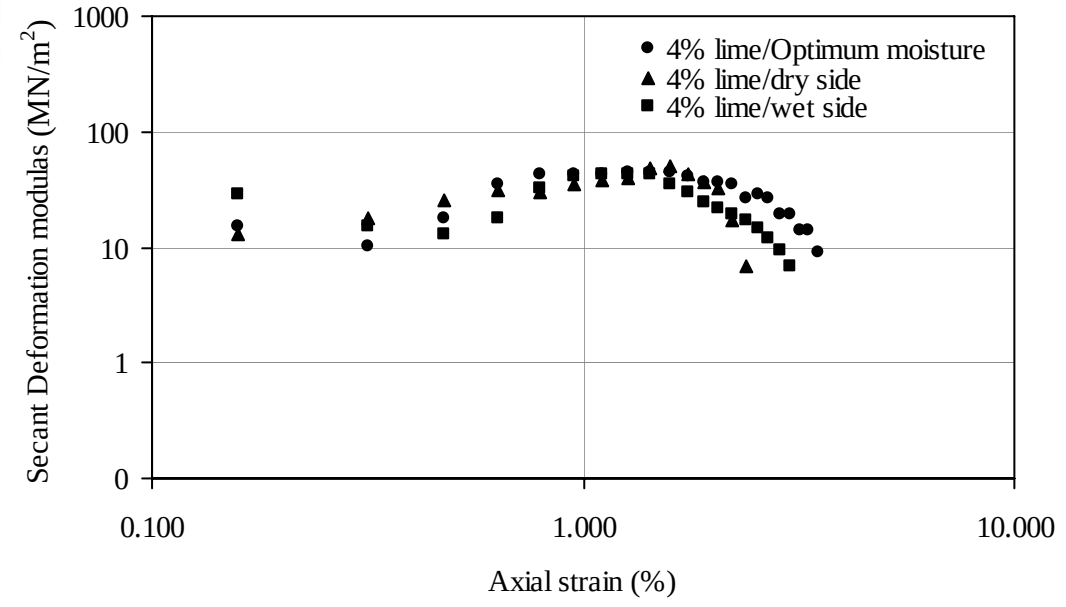


Figure 7.28 Secant deformation modulus versus axial strain for 35FA mix with 4% lime under different compaction conditions

Further, specimens compacted on dry side show higher values of secant deformation modulus compared with the specimens compacted on wet side of the optimum (Table 7.10). It can be noticed that around certain intermediate axial strain, the secant deformation modulus of the mixes, obtained under different compaction condition tends to be the same. These axial strains, corresponding to 2% and 4% lime, are 0.7% and 1.4% respectively and the corresponding values of the secant deformation modulus are 30.4 – 28.2 MN/m^2 for 2% lime and 42.7- 47.8 MN/m^2 for 4% lime.

The overall stress-strain-strength characteristics described in the foregoing paragraphs clearly suggest that addition of fly ash alone to the soil will result significant loss of strength mainly due to reduction of peak friction angle. However, the apparent loss can be recovered by the addition of lime to the soil-fly ash mix. This will result in significant increase in peak strength, stiffness, and brittleness of the material due to the increase of both peak friction angle and cohesion intercept. A slight variation in compaction condition will reduce the peak strength of the lime treated soil-fly ash mixes. Compaction on dry side will result marginal reduction, whereas in case of wet side, the reduction will be substantial. The effect of structure imparted by compaction has negligible influence on the cementation process. The variation in strength of the material occurs only because of the change in peak friction angle due to variation in density and packing. This density and packing tends to dominate the strength generation even after the development of cementation. For the present soil-fly ash-lime mix, to obtain maximum strength, compaction should be done at optimum moisture content derived from IS light compaction test.

7.5 CONCLUSIONS

1. The experimental fly ash has a low effective internal friction angle with moderate cohesion intercept. The experimental soil exhibits marginal influence of structure. Remoulding causes little variation in the effective strength parameters but compaction significantly increases the internal friction angle value.
2. Treatment of soil with only fly ash slightly lowers the peak friction angle with cohesion intercept remaining at zero value. As a result, the peak deviator stress decreases without much change in the stress-strain characteristics of the mix.
3. Addition of a small amount of lime (below lime fixation amount) to the soil-fly ash mix significantly increases the peak friction angle with the appearance of a cohesion intercept. As a result, peak deviator stress and brittleness increase considerably accompanied by reduction in failure strain.
4. Addition of lime above the lime fixation amount value results in significant increase in the cohesion intercept along with the peak friction angle. The substantial increase in the cohesion intercept clearly reflects the increase in cementation due to high pozzolanic activity. The increase in friction angle, on the other hand, can be attributed to the change in texture and gradation of the material as a result of lime modification.
5. Specimens compacted at optimum moisture content exhibit maximum strength. Compaction of specimen at moisture content other than the optimum causes reduction in peak strength with wet side showing greater reduction than the dry side.
6. Specimens compacted on dry side of the optimum show marginal reduction of the peak friction angle, while it reduces substantially in case of specimens compacted

on wet of optimum. On the dry side, the material tends to behave in a more brittle manner than the wet side.

7. The structure imparted by compaction condition has limited influence on stiffness of the specimens as well as cementation process. This is evident from only marginal change in cohesion intercept even with the increase in pozzolanic activity resulting from increased lime addition. The behaviour is mainly attributed to the relatively high initial degree of saturation of the specimens that causes very little change in water-to-binder ratio.

CHAPTER
EIGHT

LEACHING CHARACTERISTICS

8.1 INTRODUCTION

Recent emphasis on studies of leaching behaviour of waste materials call for due consideration to chemical, physical, and mineralogical characteristics of the material as well as site characteristics, interaction of the leachate and disposal environment (Hassett et al. 1992). Leaching studies for the present investigation were conducted, mainly to determine the flow and leaching characteristics of the raw and stabilised material, particularly to obtain:

1. the potential and realistic estimates of leaching for metals as well as its variation due to change in fly ash and lime content, and
2. the effect of continuous permeation on the flow parameters and the leaching pattern of the mixes.

8.2 EXPERIMENTAL DETAILS

Recent studies emphasize the need to involve multiple test procedures to examine the various aspects of leachability from fly ash and similar other stabilised material (Hassett et al. 1992). For the present study, two leaching test procedures were adopted, single batch leaching test and column leaching test.

8.2.1 Single Batch Leaching Test

Single batch leaching tests, henceforth called the batch tests, were conducted on the experimental soil, fly ash, and their various mixes to determine their potential leaching behaviour and to assess the possible toxicity of the fly ash when used for geotechnical applications. The batch tests were conducted following the standard test method for single batch extraction for wastes as per ASTM D-5233 – 92 (Re-approved 1999). Additional batch tests were conducted on the soil-fly ash mixes as well as after the addition of lime, in order to understand the change in potential leaching pattern under highly alkaline conditions.

8.2.1.1 Extraction Equipment

Leaching fluid was extracted using extraction equipment having an end over end rotation of 30 ± 2 rpm as per ASTM D-5233 – 92 (Re-approved 1999). As the specific equipment for this experiment was not available, it was designed and fabricated at the mechanical workshop of the Institute. The equipment has the capacity to hold four containers each having maximum capacity of 2 litres. A view of the extraction equipment is shown in Fig. 8.1. The equipment operates with the help of an electric motor with speed of 1440 rpm. A reduction gearbox was used to obtain the required rotation speed of 30 ± 2 rpm.

8.2.1.2 Sample Preparation and Testing

The soil-fly ash mixtures were prepared by adopting the same procedure that was used for studying their compaction and shear strength properties. The specimens used for the batch tests were initially compacted at 95% of MDD and corresponding moisture content of the respective mixes (as discussed in Section 6.3.1). Prior to testing, the compacted specimens (38mm diameter and 76mm long) were cured for 7 days and only the required amount (70g) was used for the batch tests.

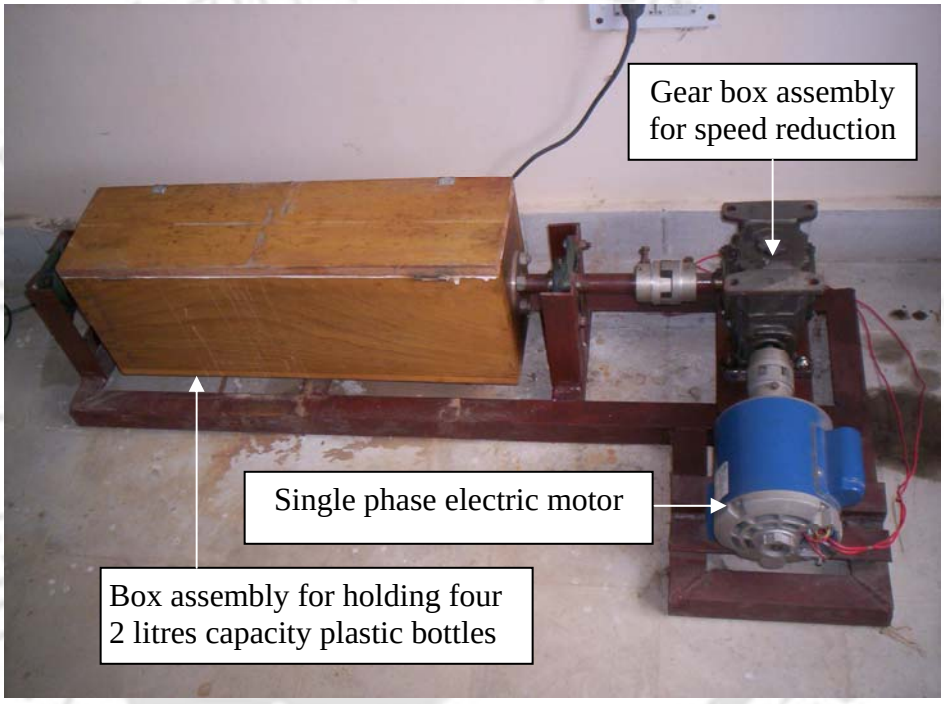


Figure 8.1 The fabricated extraction equipment used for batch leaching tests

To prepare the samples, the soil was air-dried and sieved to pass a 2 mm sieve. Required amounts of air-dried soil and fly ash, calculated on the basis of the equations 6.1 to 6.5 as described in section 6.3.1, were measured and mixed together in dry state. When no lime was added, the dry soil-fly ash mixture was mixed with required amount of water. Whenever lime was used, the dry soil-fly-ash mixture was first mixed with lime and then water was added. All mixing was done manually and proper care was taken to achieve homogeneity at each stage of mixing. After mixing with water, the samples were packed in watertight polythene bags and allowed to condition for 24 hours. Following this, the samples were compacted using the miniature static compaction tool (Fig. 6.1) and allowed to cure in 100% humidity inside desiccators before carrying out the batch tests.

The extraction fluid, used for all the batch tests, was prepared by mixing 5.7 ml of glacial acetic acid to 500 ml of ASTM type IV reagent water prepared by distillation [ASTM D-5233 – 92 (Reapproved 1999)]. To the mixture, 64.3 ml of 1N NaOH was added and the solution was diluted to 1L.

The batch tests were conducted by adding 70 g of raw test specimens to 1400 ml of extraction fluid in a 2 L sealed plastic container. The mixture was then agitated continuously for 18 hours at room temperature (22º C) using the extractor with end over end rotary movement at a rate of 30 rotations/min (Fig. 8.1). At the end of each extraction, the aqueous phase was separated, pH was measured and the leachate samples were preserved for further chemical analysis. Preservation of the leachate samples and subsequent chemical analysis of leachates are described in section 8.2.3.1

At first, tests were conducted on experimental soil and fly ash alone. This was followed by a series of tests involving different soil-fly ash mixes without the addition of

lime. In the next two series of tests, each of the soil-fly ash mixes were combined with low (2%) and high (4%) percentages of lime and then tested.

8.2.2 Column Leaching Tests

Column leaching tests were conducted to obtain more realistic assessment of leaching from soil-fly ash and soil-fly ash-lime mixes. The tests were conducted to determine leaching parameters, such as initial effluent concentration and effect of continuous permeation on the leaching pattern. The flow data obtained from the column leaching tests were used for assessment of hydraulic conductivity and its variation with the progress of pozzolanic activity and permeation. The column leaching tests were conducted following ASTM D 4874-95 (Reapproved 2001) on the experimental soil, fly ash and mixes as shown in Table 8.1.

Table 8.1 Test programme for single batch leaching test

Series	Mix Designation	Lime (%)
1	BFA RLS	Nil Nil
2	20FA, 35FA, 50FA	Nil
3	20FA, 35FA, 50FA	2%
4	20FA, 35FA, 50FA	4%

8.2.2.1 Leaching Column Apparatus

Column leaching tests were conducted to determine leaching parameters, namely initial effluent concentration and effect of continuous permeation on the leaching pattern.

The leaching column apparatus used for the present study was designed as per ASTM D4874-95 and fabricated at the mechanical workshop of the Institute. Since only inorganic species were investigated, the leaching column body was constructed of PVC, instead of glass, for the convenience of proper compaction. Further, low hydraulic conductivity of the experimental soil and some of its mixes with fly ash and lime required higher range of working pressure, which would not be possible with glass for safety reasons. With PVC, it was possible to maintain a high working pressure that assisted in saturation of the specimens prior to the tests, as well as facilitated convenient effluent collection. A photograph, along with a line sketch of the experimental set up, used for the present study is shown in Fig. 8.2 (a) and (b)

8.2.2.2 Sample Preparation

All column leaching tests were conducted on specimens compacted at 95% of MDD and corresponding water content. A comparatively lower dry unit weight was selected so that each pore volume of effluent could be collected within 24 ± 3 h with a reasonable working pressure. For preparation of various mixes, similar procedures were followed like the batch tests. For each column test, the precise amounts of the constituents of a mix (soil, fly ash, and lime) and the amount of distilled water, required to achieve the desired moisture content were determined from equations 6.1 to 6.5 as described in Section 6.3.1. The constituents were first measured and then mixed thoroughly by hand in dry form on a tray. The measured quantity of distilled water was then sprayed over it and mixed further until it appeared homogeneous. The mixture was then conditioned in a moisture-proof plastic bag for a period of 24 hours for equilibration of moisture.

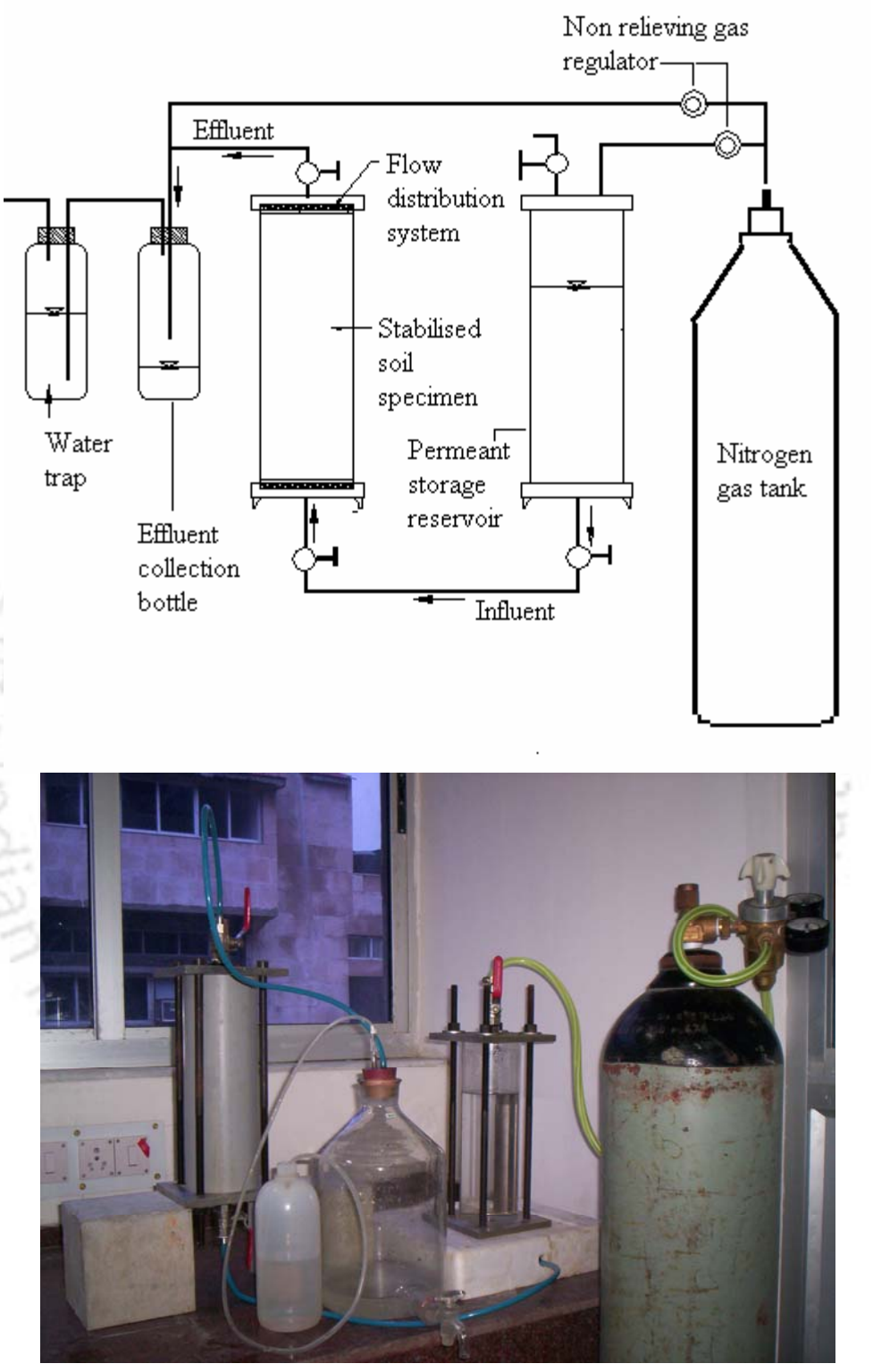


Figure 8.2 Line sketch and photograph of the column leaching test apparatus

The conditioned mixture was compacted into the leaching column in five approximately equal layers by applying standard compaction effort equivalent to light compaction. Extreme care was taken, so that the entire amount of already measured material is accommodated into the column at the end of compaction to give the desired density. The final compacted specimen was 98 mm in diameter and 300 mm in length. The compacted leaching column was then sealed in moisture proof plastic bag and allowed to cure for 7 days in 100% humidity before testing.

8.2.2.3 Testing

At the end of curing, the leaching column was assembled and allowed to saturate with reagent water. Certain specimens like that of the soil and the mix 20FA had to be saturated under pressure higher than 2 kg/cm^2. The degree of saturation of all the specimens at different stages of saturation process was checked from the difference of weight of the leaching column assembly before and after saturation.

Permeation through the leaching column was done in an up-flow mode. Hydraulic gradients between 3 and 72 were required to be applied to make sample collection convenient. These gradients are larger than those that exist in the field (which is typically about 1). However, Creek and Shakelford (1992) reported that metal leaching from fly ashes is independent of flow rate. Thus, the larger gradients are believed to have little impact on the test results.

Effluents from the column tests were collected in glass vessels. In order to minimize contact with air, the collection vessel headspace was filled with purified nitrogen gas and protected with an air trap as shown in Fig. 8.2. Pore volumes were sampled by collecting all column effluent generated over a period of 24 ± 3 h. Leachate was removed from the collection vessel after collection of each pore volume of flow, in high density polyethylene bottles and a portion (about100 ml) was set aside for chemical analysis. Each

leaching test was continued until the collection of 8 pore volumes. The leachate samples were preserved and subjected to chemical analysis following the method described in section 8.2.3.1

8.2.3 Chemical Analysis

Leachate samples collected from batch leaching tests and column leaching tests on experimental soil, fly ash and various soil-fly ash-lime mixes were stored in the laboratory and analysed suitably. Concentrations of Cd, Cr, Hg, Fe, Ni, Pb, Zn, Mn, Cu, Mg, Mn, Al, were determined by flame and furnace atomic absorption spectrophotometer (AAS) following EPA standard methods 213.2, 218.2, 270.2 and 272.2. All analysis was conducted using a Varian SpectrAA-220 Atomic Absorption Spectrophotometer using a computerized Data Station. The SpectraAA-220 was equipped with a Vapour Generation Assembly (VGA), which was used for analysis of Hg. Concentrations of Na and K was determined by flame photometer (Model- Elico-330) as AAS lamps for those two elements were not available.

8.2.3.1 Sample Collection, Filtration and Preservation

Sample collection, filtration and preservation were done following the EPA Method 1969. To prevent contamination of leachates, several measures were taken according to Part 4.0 of the Method, such as minimizing exposure, wearing gloves, using metal free apparatus and ensuring clean environment. Immediately after collection, leachate samples were filtered through 0.45 μm filter paper and acidified with concentrated 0.5% HNO_3 to bring the sample pH below 2. Samples, set aside for chemical analysis, were collected in new 100ml high-density polyethylene bottles that were washed with acid solution (2% HNO_3) before use. The sample bottles were sealed to prevent volume change and evaporation. All samples were preserved at 4^{o}C before chemical analysis.

8.2.3.2 Preparation of Standard Solution

1000mg/L Stock solutions obtained from Merck Chemical Company were used for preparation of all standard solutions. The stock solution was diluted using quartz double distilled water containing 0.5% HNO_3. Calibration standards were prepared fresh each time. A blank and at least three standards were prepared within the appropriate range.

8.2.3.3 Instrument parameters

Instrument parameters such as drying time, ashing time and wavelength were adopted from the standard method for each element. Other operating parameters, such as atomizing time and temperature and purge gas atmosphere, were chosen as recommended in the operation manual of manufacturer.

8.2.3.4 Analysis

Chemical analysis was performed according to methods corresponding to specific elements. Before each use of the AA spectrophotometer, the instrument was configured, tuned and calibrated with standards pertaining to the metals of interest. Laboratory blanks were tested along with standard solutions. Equipment blanks and bottle blanks were also tested. Three replicates of each sample were measured for accuracy and precision.

8.2.3.5 Calibration Method for Metal analysis

The instrument was optimized and initiated at zero before calibration. A standard solution was measured for five times for a stability check of the instrument. The instrument was assumed to be stable when the relative standard deviation (RSD) of the absorbance signals was less than 5% for all 5 replicates. The calibration points were obtained from the calibration blank and calibration standards and a new rational curve was fit (automatically by the data station) to the data, which was used as a calibration curve. A calibration curve was created each time after measuring 20 samples.

Three replicates of each sample were measured. The RSD of the concentrations estimated from the calibration curve for replicates, was calculated automatically by the data station. If the RSD was higher than 10%, the measurement was discarded and another measurement was made. After measuring two samples, a calibration blank was analysed to ensure no carryover of the metal of interest, and to check whether the analytical system was free from contamination. The calibration curve was also verified with a standard solution after analysing four samples. After measuring 10 samples, the calibration curve was re-sloped by analysing the calibration blank and the mid-point calibration standard. A new calibration curve was created if the re-slope varied by10% of the original slope.

8.3 RESULTS AND ANALYSIS

8.3.1 Single Batch Leaching Test

Aqueous phase concentrations of Cd, Cr, Hg, Fe, Ni, Pb, Zn, Mn, Cu, Mg, Mn, Al, Na and K obtained from single batch leaching tests along with allowable and threshold limits of the respective metals are summarized in Table 8.2. None of the metals exceed the threshold limits for any combination of soil, fly ash and lime. When compared with the allowable limits, it was noticed that except Cd and Cr, concentrations of all other metals are below the allowable limits for all the mix proportions. Concentration of Cd from experimental fly ash is 0.009 mg/L and it is about 1.8 times higher than the allowable limit (0.005 mg/L). The experimental soil also showed the presence of Cd in the aqueous concentration and its level (0.010 mg/L) is similar to that of the fly ash. The soil also exhibits significant concentration of Cr (0.125 mg/L) and its level is 2.5 times higher than the allowable limit for the metal.

It can be seen from the Table 8.2 that the leaching potential of Cr, Fe, Zn, Mn, and Al from the fly ash are less than that of the soil. In contrast, Hg, Ni, Mg, Ca, Na, and K

from fly ash showed higher leaching potential compared to that of the soil. Metals like Cr, Fe, Pb, and Cu could not be detected in the fly ash and among them, Pb and Cu were absent in the soil as well.

8.3.1.1 Effect of Fly Ash Content on leaching potential

Aqueous concentrations of various metals for different soil-fly ash mixes, presented in Table 8.2 shows that the leaching potentials of metals are affected by the addition of fly ash to the soil. It can be noted that the extent of change in leaching potential depends on the quantity of fly ash added to the soil. Further, the direction of change is not constant and varies with the type of metal. It can be observed from Table 8.2 that the leaching potential of almost all the metals, except Hg and Ni, in general, tend to reduce with the addition of a low volume (20%) of fly ash, irrespective of their original concentration. With further increase in fly ash content however, the concentration gradually increases.

In case of Ni and Hg, the leaching potential of fly ash is higher than that of the soil. It can be seen from Table 8.2 that leaching potential of both the metals increased with the addition of 20% fly ash, but decreased with further increase in fly ash content. In case of Zn and Mn, an opposite trend was observed. Addition of 20% fly ash to the soil lowered the leaching potential of these metals from the soil-fly ash mix (Table 8.2). Although fly ash itself has relatively lower leaching potential for the two metals, the concentration of both in the mix increased with the increase in fly ash content.

The concentrations of Ca, Mg and K in leachate from the batch test carried out with fly ash alone are higher than that of soil alone (Table 8.2). Addition of fly ash to the soil, at first lowers the concentration of Ca, Mg and K, but increases the concentration of these metals with the increase in the fly ash proportion, which reflects the effect of dilution and adsorption.

Table 8.2 Concentration of metals obtained from batch leaching test conducted on experimental soil, fly ash and their mixes with different lime contents.

Mix	Lime (%)	Metal Concentrations (mg/L)													
		Cd	Cr	Hg	Fe	Ni	Pb	Zn	Mg	Cu	Mn	Al	Ca	Na	K
BFA	0	0.009	ND	0.0008	ND	0.020	ND	0.162	3.592	ND	0.204	1.304	58.67	959.7	8.600
RLS	0	0.010	0.125	0.0005	0.034	0.005	ND	0.298	3.226	ND	0.774	1.710	2.630	891.1	6.200
20FA	0	0.004	ND	0.0007	ND	0.009	ND	0.063	2.629	ND	0.010	0.970	ND	939.0	5.700
35FA	0	0.008	ND	0.0005	ND	0.005	ND	0.104	3.134	ND	0.280	1.151	11.00	939.0	6.000
50FA	0	0.010	ND	0.0004	ND	ND	ND	0.143	3.447	ND	0.737	1.240	17.08	939.0	6.700
20FA	2	0.001	ND	ND	0.134	0.029	ND	0.036	9.951	ND	0.665	8.320	456.4	495.0	5.700
35FA	2	0.005	ND	ND	0.094	0.030	ND	0.021	10.58	ND	0.530	7.995	406.0	491.0	5.700
50FA	2	0.009	ND	ND	0.044	0.037	ND	0.016	11.25	ND	0.445	5.368	389.3	481.0	5.900
20FA	4	0.007	ND	ND	0.230	0.094	ND	0.043	12.35	ND	0.621	10.45	538.0	487.0	7.800
35FA	4	0.008	ND	ND	0.129	0.091	ND	0.041	12.01	ND	0.511	8.681	559.0	488.0	7.500
50FA	4	0.008	ND	ND	0.052	0.088	ND	0.017	12.32	ND	0.389	4.250	768.0	483.0	7.400
Allowable Limits	--	0.005	0.050	0.0010	0.300	--	0.050	5	150.0	1.000	---	---	---	---	---
Threshold Limits	--	0.050	5.000	0.1000	30.00	--	5.000	500	1500	100.0	---	---	---	---	---

Notes: 1. ND = Not Detected
2. Allowable and threshold limits are as per drinking water for World Health organization (WHO 1984), Guidelines for Canadian Drinking Water Quality (1979), and National Primary Drinking Water standards U.S. EPA (1986).
3. Threshold Limit = 100 × (allowable limit)

The soil, fly ash and soil-fly ash mixes showed high concentration of Na in the leachate obtained from the batch test, which is due to the addition of NaOH to the leaching fluid during the extraction process.

It is interesting to notice that the concentrations of metals in the leachate from soil-fly ash mixes are less dependent on the metal content of the soil itself. Soil is the major component (50-80% on weight basis and about 45% to 75% on volume basis) of the soil-fly ash mix, but the amount of metal in soil seems to have limited contribution to the metal concentration in leachate from the mixes. Fly ash, which is a minor constituent in the mix, on the other hand, appears to have more contribution to the metal concentration in the leachate derived from soil-fly ash mixes.

8.3.1.2 Effect of Lime on leaching potential

Aqueous concentrations of various metals obtained from the batch leaching test on lime treated soil-fly ash mixes reveal that the leaching potentials of most of the metals are significantly affected by the lime treatment of the soil-fly ash mixes (Table 8.2). When compared with soil-fly ash mixes without lime, leaching potential, in general, increases for Fe, Ni, Mg, Mn, and Al; decreases in case of Hg, Zn and Na; and remains somewhat similar in case of Cd, Cr, and K upon lime treatment. The concentration of Cu and Pb remained at a level below detection limit. The quantum of change and its direction depends on the amount of soil replaced with fly ash and it varies from species to species.

In case of Hg, its leaching potential substantially reduces upon lime treatment and the concentration decreases beyond the detection limit. Addition of lime reduces the leaching potential of Zn and it decreases further with the increase in fly ash content.

With 2% lime, the leaching of Cd reduces significantly in case of the mix with 20% fly ash. But, it slowly increases with the increase in fly ash content. The change in leaching

potential with 4% lime, however, is very marginal and appears to be independent of fly ash content, even up to 50% replacement of soil with fly ash.

Iron, which was earlier absent in the mixes, starts leaching in alkaline environment created by lime. But, its leaching potential tends to reduce with the increases in fly ash content for both 2% and 4% lime, suggesting predominant contribution from the soil.

8.3.2 Column Leaching Tests

Column leaching tests were conducted on soil-fly ash mixes to obtain more realistic estimates of leaching for metals. Column leaching tests were also conducted on lime stabilized soil-fly ash mixes to estimate the change in metal concentration in the leachate as a function of different percentages of lime in the mix. In addition, column leaching tests were also conducted on the experimental soil and fly ash alone separately to understand their individual responses and other flow parameters required for evaluating impacts on ground water. Concentrations of metals in the effluent from the column leaching tests were analysed following EPA Standard methods 213.2, 218.2, 270.2 and 272.2. Flow data, obtained from the column leaching tests, were used for assessment of hydraulic conductivity and its variation with permeation of the influent. A summary of the leachate composition and physical parameters obtained from the column leaching tests is presented in Tables 8.3 & 8.4.

8.3.2.1 pH of the Effluents

pH of the effluents from the column leaching tests was measured immediately after collecting the leachate samples. pH was measured with an Digital pH meter (Model EI–161E) equipped with a glass body electrode. Variation of pH of effluents obtained from column leaching tests on soil, fly ash, various soil-fly ash and soil-fly ash-lime mixes are presented as a function of pore volume of flow in Figs. 8.3(a-f).

Table 8.3 pH and initial effluent concentrations from column leaching tests conducted on experimental soil, fly ash and their mixes with different proportion of lime

Mix	Lime (%)	Effluent pH	Initial Effluent Metal Concentrations (mg/									
			Cd	Cr	Hg	Fe	Ni	Pb	Zn	Mg	Cu	
BFA	0	6.71	0.005	0.037	0.0003	0.031	0.177	ND	0.054	2.560	ND	0
RLS	0	3.70	0.010	0.027	0.0004	0.543	0.072	ND	1.354	13.65	ND	1
20FA	0	5.83	0.004	0.029	ND	ND	0.020	ND	0.624	2.100	ND	0
35FA	0	5.91	0.007	0.032	ND	ND	0.043	ND	0.993	3.132	ND	0
50FA	0	5.93	0.007	0.036	ND	ND	0.058	ND	1.545	3.102	ND	1
20FA	2	11.73	ND	0.033	0.0002	0.043	0.037	ND	ND	0.925	ND	
35FA	2	11.68	ND	0.048	ND	0.035	0.076	ND	ND	0.490	ND	
50FA	2	11.59	ND	0.087	ND	0.033	0.092	ND	ND	0.386	ND	0
20FA	4	11.86	ND	ND	0.0008	0.028	0.085	ND	ND	0.807	ND	0
35FA	4	11.65	ND	ND	ND	0.042	0.096	ND	ND	0.632	ND	0
50FA	4	11.64	ND	ND	ND	0.057	0.099	ND	0.135	0.536	ND	0

Notes: 1. ND = Not Detected

Table 8.4 Porosity, hydraulic conductivity and other related properties of specimens used for column leaching test

Mix	Lime (%)	Dry Unit Weight (gm/cm^3)	Water Content (%)	Volume of Void (cm^3)	Porosity	Seepage Velocity (cm/sec)	k (cm/sec)
BFA	0	1.16	24.0	978	0.44	4.47×10^{-4}	5.71×10^{-5}
RLS	0	1.48	20.3	993	0.44	3.57×10^{-4}	2.20×10^{-6}
20FA	0	1.47	16.3	948	0.42	3.39×10^{-4}	2.10×10^{-6}
35FA	0	1.39	18.0	953	0.43	3.56×10^{-4}	5.53×10^{-6}
50FA	0	1.38	19.7	941	0.42	3.82×10^{-4}	7.81×10^{-6}
20FA	2	1.33	18.0	1062	0.47	3.90×10^{-4}	1.80×10^{-5}
35FA	2	1.30	19.1	1031	0.46	3.30×10^{-4}	2.22×10^{-5}
50FA	2	1.27	18.0	1039	0.46	4.01×10^{-4}	5.43×10^{-5}
20FA	4	1.30	17.4	1075	0.48	3.67×10^{-4}	1.62×10^{-5}
35FA	4	1.27	18.2	1054	0.47	3.51×10^{-4}	2.15×10^{-5}
50FA	4	1.23	22.0	1077	0.48	3.51×10^{-4}	5.38×10^{-5}

Note: Values of k correspond to the initial effluent or 1^{st} pore volume

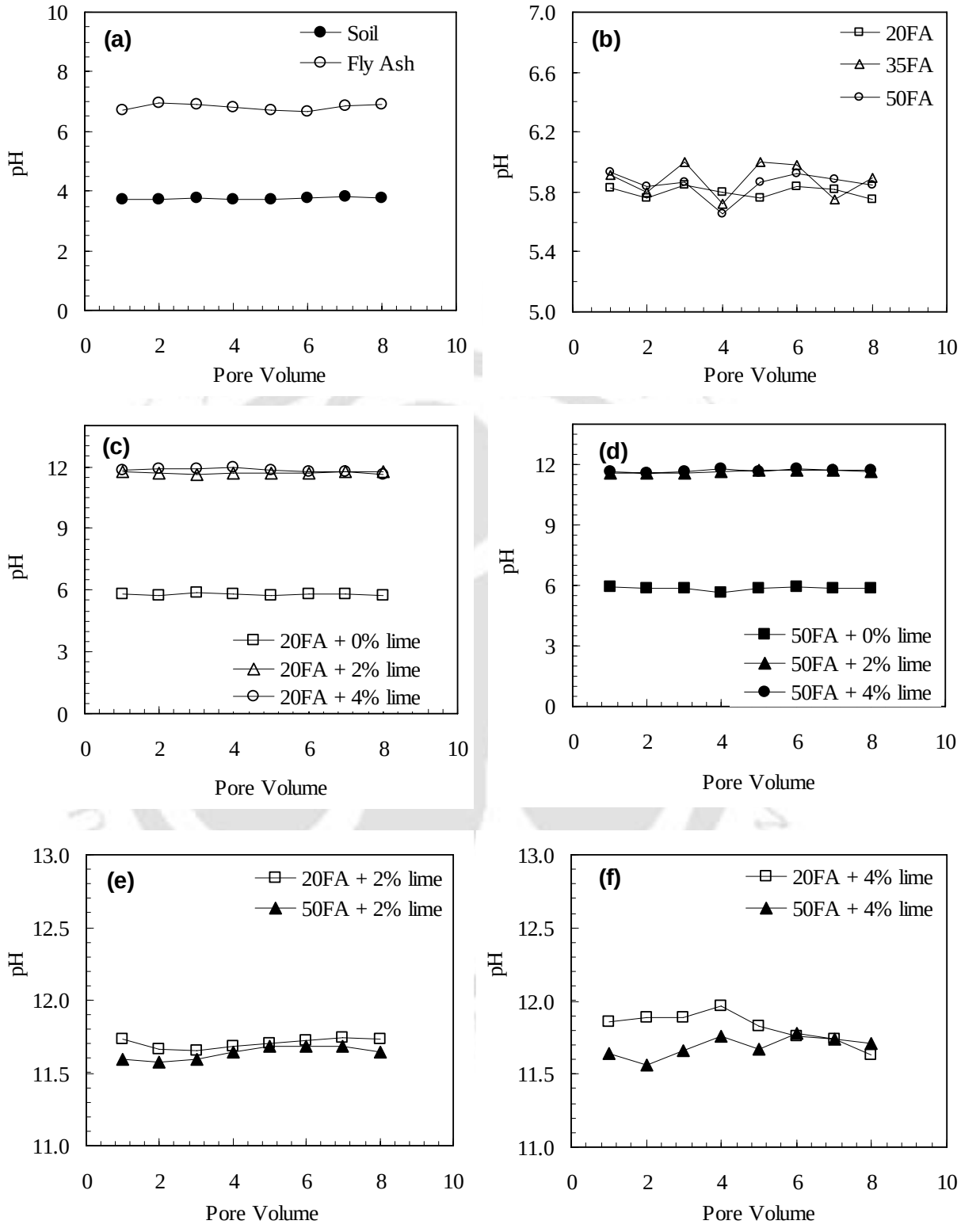


Figure 8.3 pH of effluents from column leaching test versus pore volume for (a) experimental soil and fly ash (b) soil-fly ash mixes (c-f) soil-fly ash-lime mixes

All the column leaching tests were allowed to run for 8 pore volumes of flow and in all cases, the pH of the flow remained essentially unchanged throughout the tests. The average pH of leachate collected from soil is 3.75, which indicates the strong acidic nature of the residual lateritic soil due to the decomposition of aluminosilicate minerals and accumulation of Al^{3+}. The average pH of leachate obtained from the fly ash is 6.82. The near neutral nature of the effluent from the fly ash is due to low hydration potential indicated by low percentage of CaO present in the fly ash. Addition of fly ash to soil raises the pH of soil-fly ash mixes. The variation in pH of the soil-fly ash mixes with permeation is shown in Fig. 8.3(b). Throughout the tests, the pH of the effluent from different soil-fly ash mixes remained within a close range of 5.7 to 6.0. The average pH of the effluent derived from soil-fly ash mixes with varying lime content is shown in Fig. 8.4(a) as a function of fly ash content.

It can be noticed from the figure that average pH of the leachate obtained from soil increases rapidly from 3.75 to 5.80 as the fly ash content increases from 0% to 20%. However, further increase in fly ash content only marginally increases the pH. For example with 50% replacement of soil with fly ash, the pH further increases only to 5.85. This is due to the buffering capacity of the soil, produced by the variable surface charge minerals like iron and aluminum sesquioxides present in the soil. The variable surface charge minerals present in lateritic soils produce a non-exchangeable form of reserve acidity that built up at low pH and contributes to soil buffering capacity (McBride, 1994).

Addition of lime to the soil-fly ash mixes appreciably increases the pH of the leachate as shown in Fig. 8.3(c-d). The average pH of the 20FA mix without lime is 5.80,

which increases to 11.7 with the addition of 2% lime to the mix. Fig. 8.4(b) illustrates the variation of average pH of the soil with the change in fly ash and lime content.

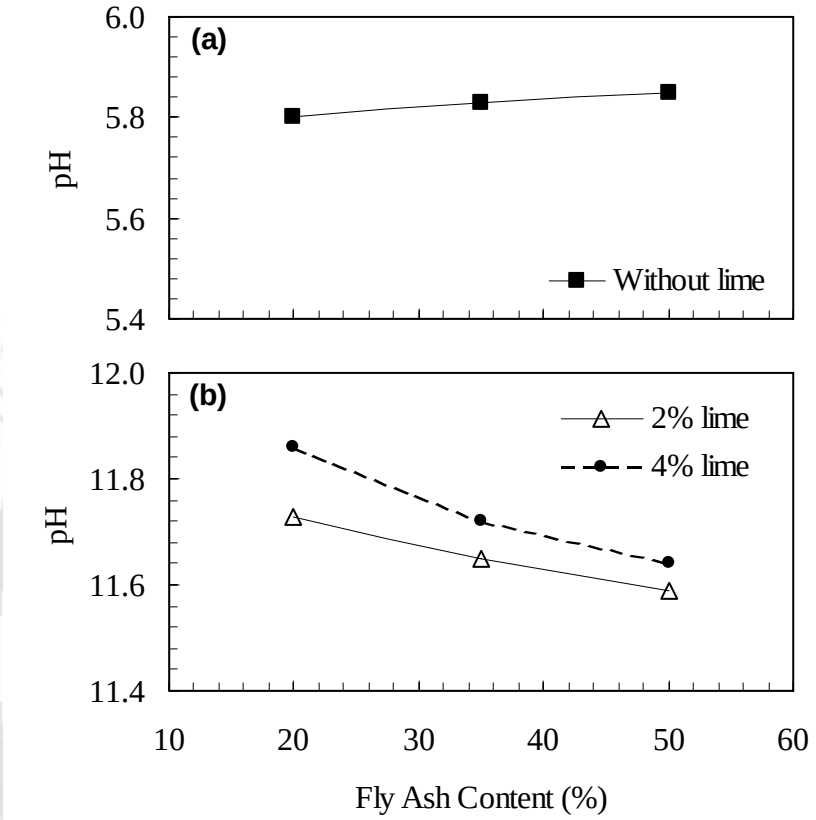


Figure 8.4 Change in pH of initial effluents from column leaching test on soil with varying fly ash content (a) Without lime (b) With 2-4% lime content

For a given lime content, it is observed that the average pH of leachate collected from soil-fly ash-lime mix marginally reduces with increasing fly ash content (Fig. 8.4). on the other hand, for a given fly ash content, pH of leachate from soil-fly ash-lime mixes increases marginally with lime content. However the rate of increase in pH values with higher lime content reduces with the increase in fly ash content (Fig. 8.4-b).

8.3.2.2 Hydraulic Conductivity

Flow data obtained from the column leaching tests were used for estimation of the hydraulic conductivity and its variation with permeation with respect to the soil, fly ash and their various mixes with or without lime. The hydraulic conductivity presented in Table 8.4 corresponds to the 1st pore volume. Porosity and other related properties of specimens as shown in the Table 8.4 are computed from the phase relationship.

Hydraulic conductivity of a material is an important parameter from the point of landfill construction. In accordance with RCRA (Resource Conversion Recovery Act) (Subtitles C and D) of USA, compacted liners and covers for hazardous and municipal solid waste landfills must have a hydraulic conductivity less than 1×10^{-7} cm/sec. Tests on compacted specimens at confining pressure of 0.10 kg/cm^2, performed earlier (Anderson et al., 2000), shows that partially air-dried lateritic soil has K_{sat} values from 3.1×10^{-5} to 8.7×10^{-7} cm/sec, depending on its dry density and compacted water content. It was also observed that hydraulic conductivity of lateritic soil could be lowered below 1×10^{-7} cm/sec by the use of small amount of admixture (5% bentonite).

The lateritic soil, chosen for the present investigation, was compacted at 95% relative to MDD and had hydraulic conductivity (2.20×10^{-6} cm/sec) similar to the values observed by Anderson et al. (2000). Fly ash, on the other hand is much more permeable (5.71×10^{-5} cm/sec) compared to the soil. It can be seen from Table 8.4 that addition of fly ash and lime to the soil increases the hydraulic conductivity of the material. Fig. 8.5 presents the variation in hydraulic conductivity of the soil-fly ash-lime mixes as a function of fly ash and lime content.

Addition of fly ash to the soil, initially, marginally reduced the hydraulic conductivity of the soil-fly ash mix and then it increased rapidly with the increase in fly ash

content. Addition of 20% fly ash to the soil marginally reduced the hydraulic conductivity of the soil from 2.2×10^{-6} cm/sec to 2.1×10^{-6} cm/sec. With further increase of fly ash to 35% and 50%, the hydraulic conductivity increased to 5.53×10^{-6} cm/sec and 7.81×10^{-6} cm/sec respectively.

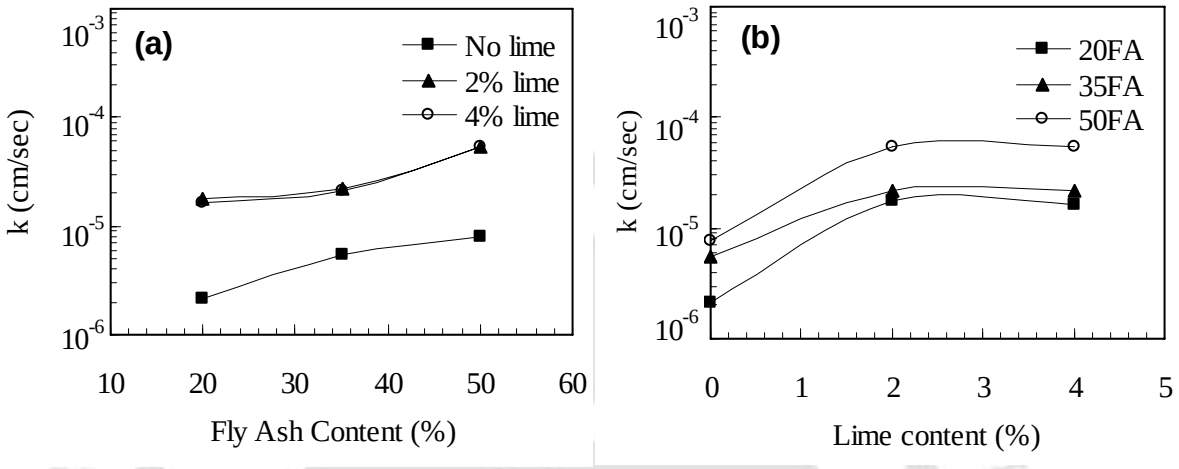


Figure 8.5. Change in hydraulic conductivity of the soil-fly ash-lime mixes with the variation of (a) fly ash content (b) lime content

The change in hydraulic conductivity can be attributed to the variation in dry density of the material resulting from the addition of fly ash. As explained earlier (Section 4.5.2.2), the fly ash has a greater fraction of fine particles than the experimental soil and addition of a small quantity of fly ash fills up the voids between the relatively coarser particles of the soil. As a result, the interconnectivity between the pores gets reduced, and hydraulic conductivity marginally reduces for the 20FA mix. With further addition of fly ash, however, the lighter fly ash particles tend to interfere with the compaction, thus reducing the MDD of the mixes, which is reflected in the increase in hydraulic conductivity of the soil-fly ash mixes with more than 20% of fly ash as shown in Fig. 8.5(a).

Addition of lime appreciably changes the hydraulic conductivity of the soil-fly ash mixes, with the extent and direction of change depending on the amount of lime added. It

can be noticed from Fig. 8.5(b) that addition of 2% lime to the mixes immediately increased the hydraulic conductivity of all the mixes. It got slightly reduced when the lime content was increased to 4%. The increase in hydraulic conductivity with a relatively smaller amount of lime can be attributed to the resultant change in dry density due to short-term modification of the texture and gradation of the mixes. The decrease in hydraulic conductivity, with further addition of lime, on the other hand, can be attributed to reduction in interconnectivity of the pores caused by the formation of pozzolanic reaction products.

The variation of hydraulic conductivity of the soil, fly ash with permeation is shown in Fig. 8.6 & 8.7. Throughout the tests, the hydraulic conductivity of soil, fly ash and their various mixes essentially remained unchanged. The soil-fly ash-lime mixes, on the other hand, show some variation with permeation (Fig. 8.8).

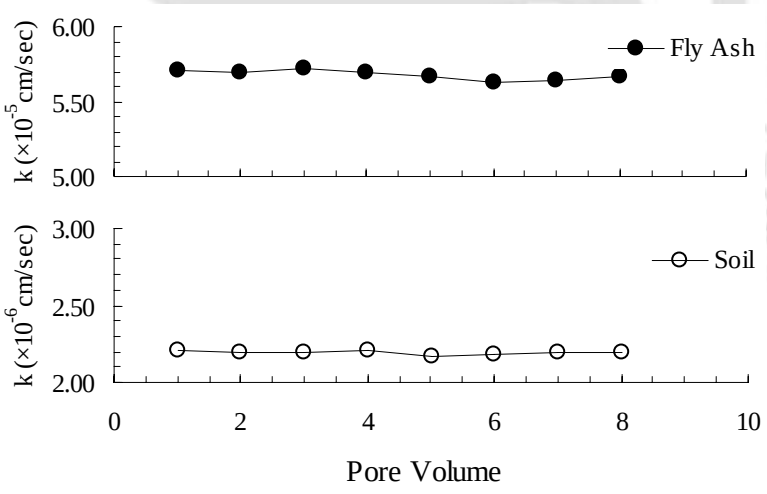


Figure 8.6 Hydraulic conductivity versus effluent pore volume for experimental soil and fly ash

Hydraulic conductivity of all the soil-fly ash-lime mixes gradually decreases with the progress of permeation due to the formation of hydration products that reduces the interconnectivity between the pores. Further, the 20FA mix shows somewhat greater

degree of reduction with the increase in lime content, which is possibly due to increased pozzolanic activity compared to the 50FA mix.

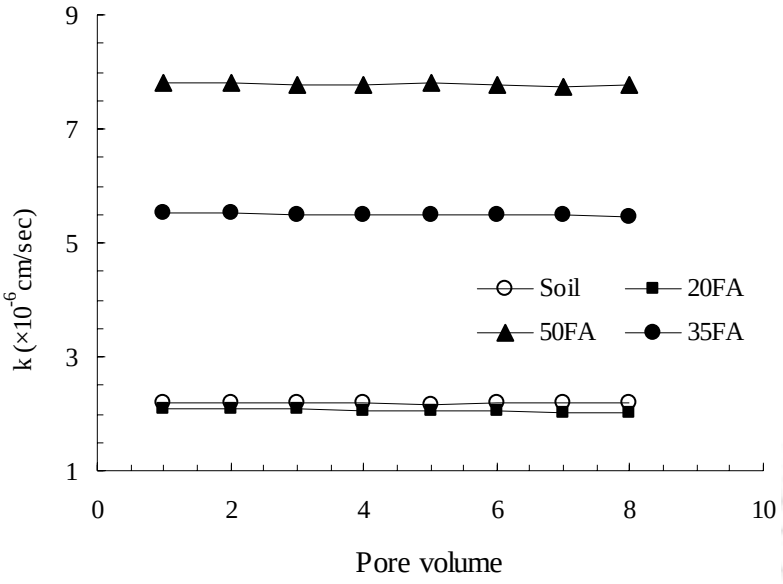


Figure 8.7 Hydraulic conductivity versus effluent pore volume for various soil-fly ash mixes without the addition of lime

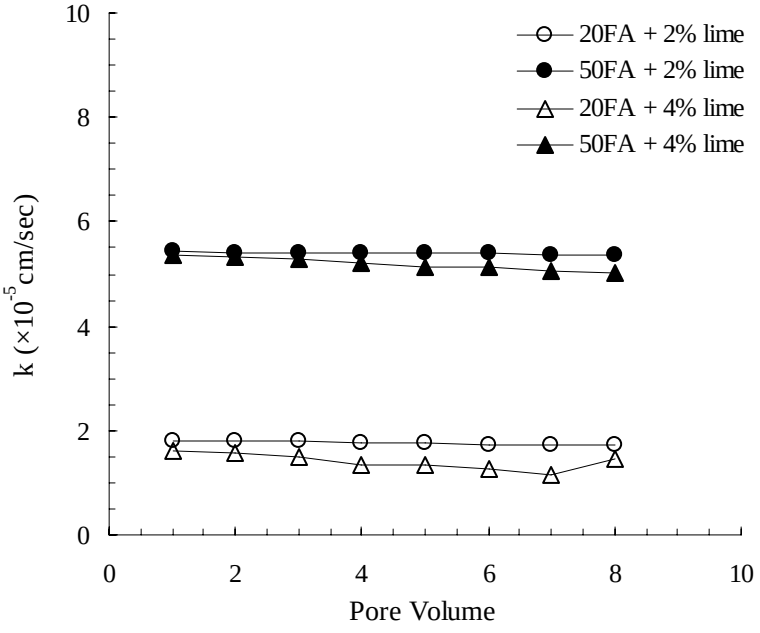


Figure 8.8 Hydraulic conductivity versus effluent pore volume for soil-fly ash mixes with 2% and 4% lime

8.3.2.3 Initial Instantaneous Effluent Concentration

The knowledge of concentration of different species in the initial pore water present in the matrix of a material is important for estimating the release of contaminants to ground water and it is represented by the initial instantaneous effluent concentration. The concentration of a species in the first leachate sample collected from the column leaching test can be assumed to be the initial instantaneous pore water concentration of that species. A summary of the initial effluent concentration is shown in Table 8.3. The change in initial effluent concentration of various elements with the variation in fly ash and lime content are discussed in the following sections.

Cadmium : The initial effluent concentration of Cd for soil is higher than the fly ash (Table 8.3), although batch leaching test showed that both the materials have the same leaching potential. This is possibly due to higher mobility of the metal in soil at low pH compared to that in fly ash. The Cd concentrations in all the leachates of soil-fly ash mixes were lower than in the soil alone and varied only marginally with the increase in fly ash content (Fig.8.9). In batch leaching tests, a similar trend was noticed for the soil-fly ash mixes (Table 8.2).

Addition of lime to the soil-fly ash mixes decreases the concentration of Cd (Table 8.3) to beyond detection limit. At pH lower than 6, Cd shows medium to high mobility due to weak adsorption on organic matter, silicate clays, and oxides. At higher pH (above 7), Cd can co-precipitate with $CaCO_3$, or precipitate as $CdCO_3$, and Cd phosphate may limit solubility (McBride, 1994). At still higher pH of slightly more than 10, precipitation completes (Table 8.5). Addition of lime to the soil-fly ash mixes considerably increases the pH of the effluent from 5.8-5.9 to 11.6-11.9 and the high pH condition might have reduced the mobility of cadmium.

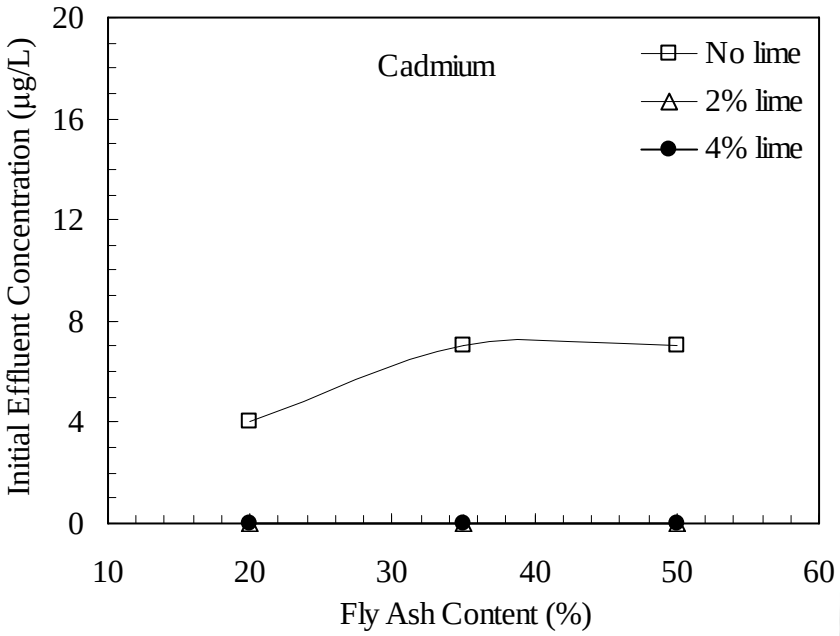


Figure 8.9 Variation of Cd concentration in initial effluents from column leaching test on soil for varying fly ash and lime content

Table 8.5 pH required to form and complete the hydroxide precipitation of some metal ions and also the completion of carbonate precipitation

Metal Ions	Hydroxide Precipitation begins at pH	Completion of Hydroxide Precipitation at pH	Completion of Carbonate Precipitation at pH
Ca^{++}	11.0	13.5	12.9
Fe^{++}	5.5	9.3	11.3
Zn^{++}	7.9	8.5	11.6
Mg^{++}	10.0	11.3	14.0
Mn^{++}	8.5	10.3	11.1
Ni^{++}	6.7	9.4	11.2
Cu^{++}	5.3	7.4	11.7
Pb^{++}	6.5	9.0	10.6
Cd^{++}	6.7	10.0	10.1
Ag^{++}	9.0	12.7	11.4
Co^{++}	6.0	9.2	10.1

Source: (Pandian et al., 1996)

Zinc : The initial effluent concentration of Zn in soil leachate is higher than the fly ash (Table 8.3). This is due to the higher mobility of the species in acidic environment of the effluent from the soil. The pH of the effluent from fly ash is near neutral and Zn shows limited mobility in such condition due to chemisorption on oxides and aluminosilicates (McBride, 1994). The concentrations of Zn in effluent from the soil-fly ash mixes are higher than that from fly ash alone and it increases with the increase in amount of fly ash in the mix (Fig. 8.10).

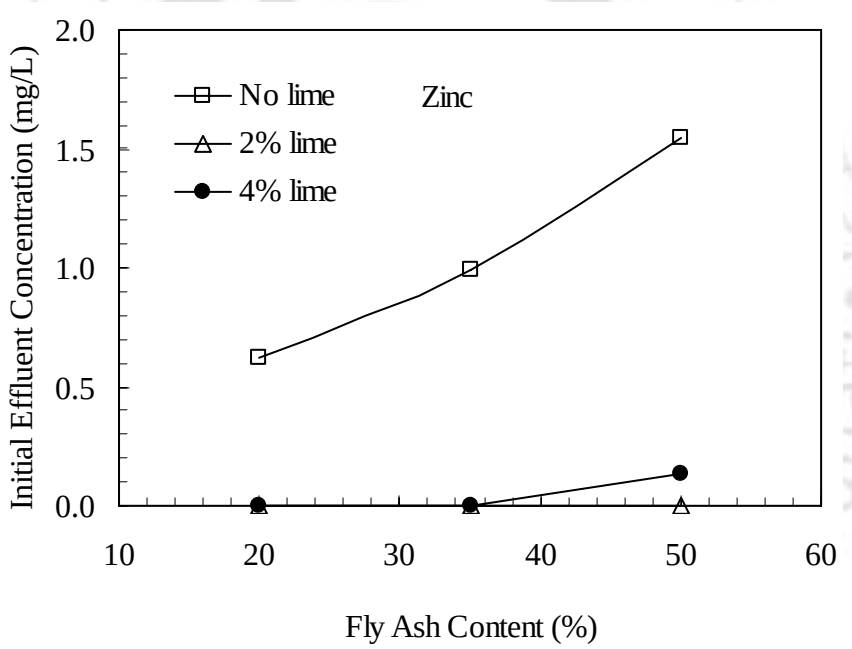


Figure 8.10 Variation of Zn concentration in initial effluents from column leaching test of soil for varying fly ash and lime contents

The increase in Zn concentration may be due to increased leaching from fly ash caused by higher mobility of Zn in decreased pH condition relative to fly ash alone. Addition of lime to the soil-fly ash mixes immediately reduces the concentration of Zn in the initial effluents from all soil-fly ash mixes and it falls below the detection limit, which is possibly due to the low solubility of Zn at higher pH.

Chromium: In case of Cr, the initial effluent concentration from column leaching test with respect to fly ash alone is slightly higher than soil alone (Table 8.3). It can also be seen from Table 8.3 that concentration of Cr in leachate from all the soil-fly ash mixes are lower than that from fly ash alone. The variation of Cr concentration in the effluents soil-fly ash mixes with the change in fly ash and lime content is presented in Fig. 8.11. It can be seen from the figure that initial effluent concentration of Cr from the soil-fly ash mixes, when used without lime, increases marginally with the fly ash content.

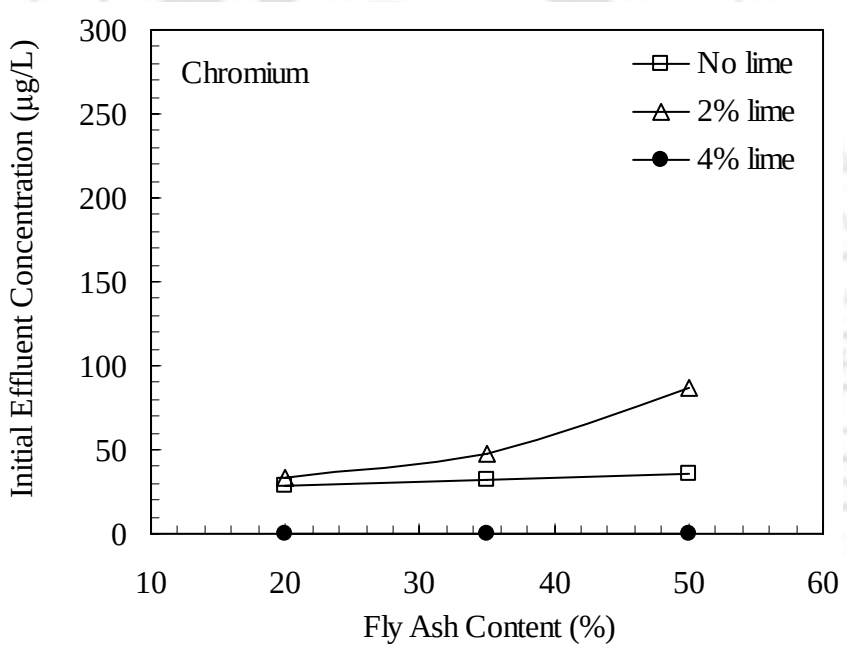


Figure 8.11 Variation of Cr concentration in initial effluents from column leaching test on soil for varying fly ash and lime content

With the addition of a small amount of lime (2%), the Cr concentration in leachate increases and shows concentration higher than in case of the soil or fly ash alone. However, further increase in lime content lowers the concentration to below detection limit. Reduction in the concentration of Cr is possibly due to the precipitation of its oxyanions during the formation of hydration product upon lime addition (Hassett et al., 2003).

Iron: The initial effluent concentration of Fe for the soil alone is much higher than the fly ash (Table 8.3). Lateritic soils are naturally rich in Fe due to weathering of ferromagnesian minerals. The low pH of the effluent (less than 4) keeps Fe in sufficiently soluble state. Low initial effluent concentration of Fe from fly ash alone is due to lower leaching potential (availability) as well as higher pH of the effluent, since hydroxide precipitation of Fe begins at about pH of 5.5 (Table 8.5). Addition of fly ash alone to the soil significantly lowers the Fe concentration and its level reaches below the detection limit for all percentages of fly ash used in the experiment (Fig. 8.12).

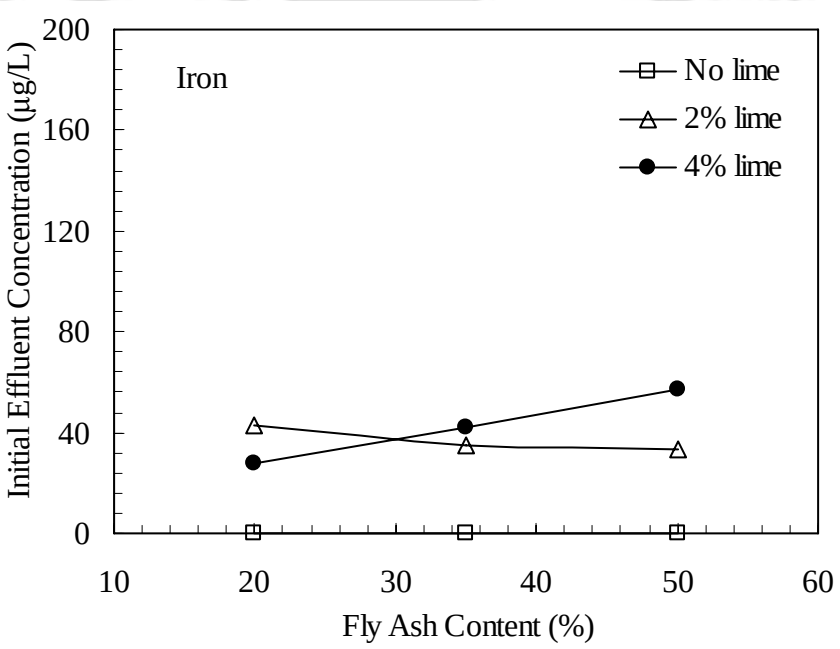


Figure 8.12 Variation of Fe concentration in initial effluents from column leaching test on soil for varying fly ash and lime content

Along with Al and Si, Fe gets released in highly alkaline medium. The lime treated soil-fly ash mixes show Fe concentrations in the initial effluents, which slightly increase or decrease depending on the fly ash and lime content (Fig 8.12).

Nickel: The initial effluent concentration of Ni from the column leaching test carried out on fly ash alone is much higher compared to the soil tested alone (Table 8.3). The mobility of Ni is rated as medium in acid soil and it becomes very low in neutral to alkaline soils (McBride, 1994). Higher concentration of Ni in fly ash than the soil is possibly due to the higher leaching potential of the metal from fly ash, noticed from the batch leaching test (Table 8.2). It can be noted from Table 8.3 that addition of fly ash to soil significantly reduces the concentration of Ni than soil alone. Further, Ni concentration increases slowly with the increase in fly ash content, which is due to higher initial concentration of the metal in fly ash than in the soil (Fig. 8.13).

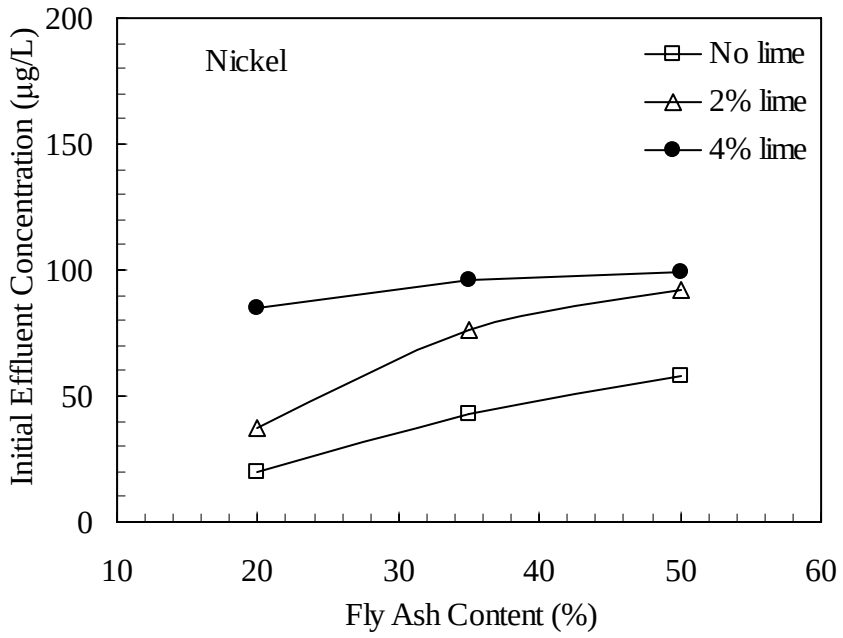


Figure 8.13 Variation of Ni concentration in initial effluents from column leaching test on soil for varying fly ash and lime content

A similar but somewhat enhenced effect is noticed when 2% lime is added to the soil-fly ash mixes. Further increase in lime content, however, minimizes the variation of

the metal concentration with change in fly ash content. With 4% lime addition, Ni concentration of the soil-fly ash-lime mixes ranges between 0.085 to 0.099 mg/L.

Magnesium: Although Mg has similar leaching potential from fly ash and soil (Table 8.2), the initial effluent concentration of the metal in leachate from column leaching test performed on fly ash alone is much less than that of soil alone, mainly due to the difference in pH of the effluents (Table 8.3). The effluent from soil alone has pH value of 3.7, at which Mg has higher mobility. When fly ash is added to the soil, the pH value increases to 5.8 – 5.9, which decreases the mobility of Mg ions from the soil. Figure 8.14 shows the initial effluent concentration of Mg from various soil-fly ash mixes as a function of fly ash and lime content.

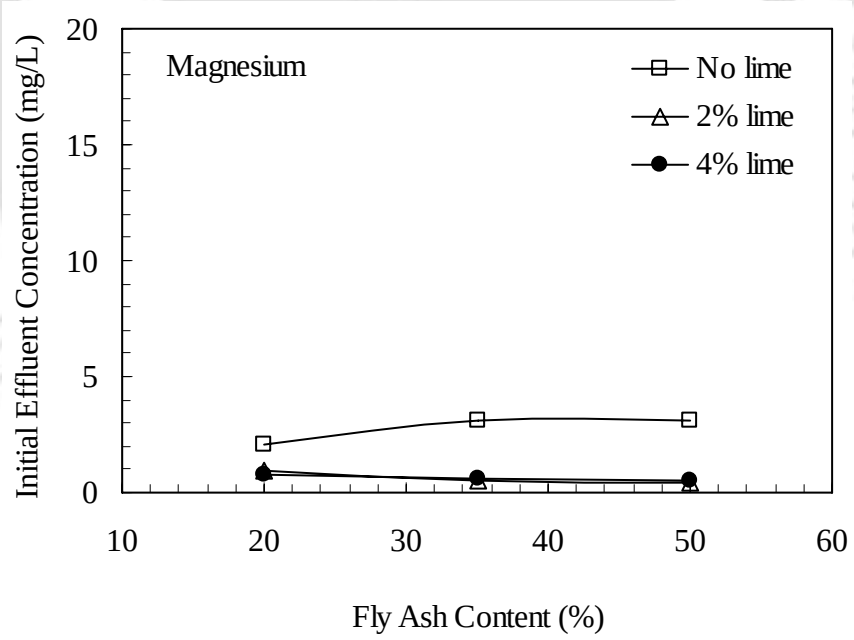


Figure 8.14 Variation of Mg concentration in initial effluents from column leaching test on soil for varying fly ash and lime content

The soil-fly ash mix shows some concentration of the metal that reduced significantly with the addition of lime due to formation of insoluble $Mg(OH)_2$ and $MgCO_3$. This is observed for both 2% and 4% lime addition.

Manganese: The variation of initial effluent concentration of Mn from soil, fly ash, soil-fly ash, and soil-fly ash-lime mixes are presented in Table 8.3. The soil has much higher concentration of Mn than the fly ash because of increased solubility of the metal species at low pH that favours the reduction of insoluble Mn oxides to soluble Mn ions. It is well known that manganese solubility is controlled by the redox potential and pH of the soil. The manganese ion is very soluble in water and forms hydroxide and carbonate precipitate only at high pH (>7). However, as the pH is raised above 6 in soils, Mn^{2+} bonds with organic matter, oxides, and silicates and its solubility decreases.

Further, small changes in the soil redox potential or pH can shift the Mn^{2+} – Mn oxide reaction (McBride, 1994). Fig. 8.15 shows the variation of Mn concentration with the change in fly ash and lime content of various soil-fly ash mixes. Replacement of the soil with fly ash up to 35% reduces the Mn concentration in effluents from the soil-fly ash mixes, possibly due to decrease in solubility of the metal with increase in pH after the addition of fly ash. However, it can be noted from Table 8.3 and Fig. 8.15 that increase in fly ash content to 50% significantly increases the Mn concentration, which is possibly due to change in redox potential and associated reduction of Mn-oxide to soluble Mn ions as explained earlier. It can also be noticed from Fig. 8.15 that addition of lime and subsequent increase in pH (>11) of the soil-fly ash-lime mixes significantly reduces the Mn concentration due to formation of insoluble Mn-hydroxide and carbonates.

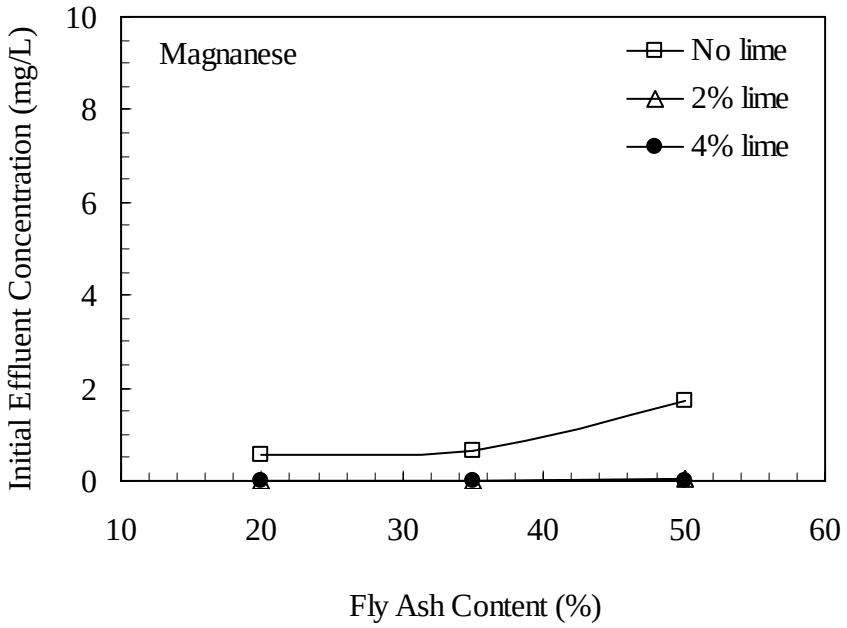


Figure 8.15 Variation of Mn concentration in initial effluents from column leaching test on soil for varying fly ash and lime content

Aluminium: The initial Al concentrations in leachate from the column leaching tests are shown in Table 8.3. The solubility and chemical form of Al in pure water is determined by a sequence of hydrolysis steps that ultimately results in the precipitation of $Al(OH)_3$ above pH 5, and the dissolution of $Al(OH)_3$ as the aluminate anion above pH 8 (McBride 1994). Although the soil has marginally higher leaching potential than the fly ash for Al (Table 8.2), column leaching test shows much higher concentration of the metal in soil leachate compared with that of the fly ash due to low pH of the effluent from soil (3.7). The pH of the effluent from fly ash, on the other hand, lies in the range 6 to 7, at which Al has minimum solubility and results in low Al concentration in fly ash. It can be noticed from Table 8.3 that replacement of even a fraction of the soil with fly ash significantly reduces the Al concentration of the soil-fly ash mixes. This is due to the increase in pH value of the mixes (5.8-5.9) from the pH value for the soil alone. Also, the Al concentration in soil-fly ash mixes varies only marginally with the increase in fly ash content (Fig.8.16).

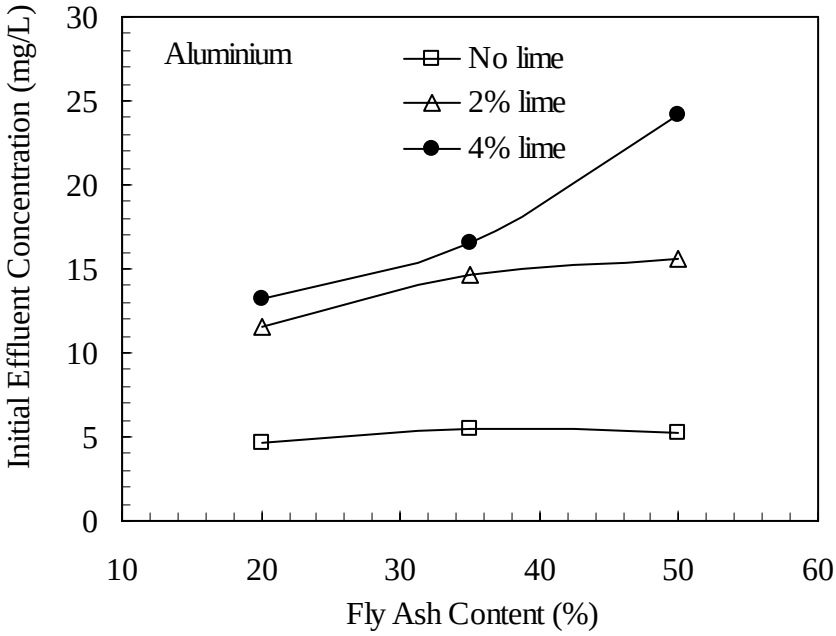


Figure 8.16 Variation of Al concentration in initial effluents from column leaching test on soil for varying fly ash and lime content

The reason for poor response of the variation of fly ash content to the Al concentration possibly lies with the fact that both the materials are good source of aluminum (Table 3.2 & 3.4) and replacement of one with the other does not make significant difference in the source of aluminum in the resultant mix. With the addition of lime, the pH of the effluents rises to very high level and consequently solubility of the metal also increases, thus raising the Al concentration in the lime treated soil-fly ash mixes. Further, at consistently higher pH, Al from the aluminum bearing minerals undergoes dissolution, which also increases the concentration of the metal. It is also interesting to note from Fig. 8.16 that the Al concentration of the soil-fly ash-lime mixes increase with the increase in fly ash content. This is due to higher pozzolanic activity (after 7 days of curing) in case of the mixes 20FA and 35FA than 50FA that produces insoluble hydration products by consuming some of the available Al ions from the solution. In case of 4% lime,

Al concentration increases further in fly ash content due to enhanced pozzolanic activity of the 20FA and 35FA mixes and formation of more hydration products.

Sodium and Potassium: The initial concentrations of Na and K in the effluent from fly ash are considerably higher than that of soil (Table 8.3). Although, both Na and K are highly soluble in water, their mobility is greatly affected by cation exchange. Since lateritic soils are formed in well-drained areas, most of the freely available Na and K have already been leached out leaving only the fraction present at the exchange sites and fixed in the clays. In case of the fly ash, which had never been subjected to permeation, most of the soluble Na and K was flushed out in the initial effluent and show increase in their concentration. With the replacement of 20% of soil with fly ash, the higher Na and K concentration in the ash tends to increase the concentration of Na and K in the soil-fly ash mixes as well. However, the increase in concentration is not proportionate with the amount of soil replaced with the fly ash, possibly due to cation exchange taking place (Figs. 8.17 & 8.18).

Addition of lime to the soil-fly ash mixes does not seem to change the Na and K concentration significantly. It can also be noted from the Figs. 8.17 & 8.18 that the concentrations of Na and K in the soil-fly ash-lime mix are somewhat independent of the fly ash content of the mix.

Calcium: The initial concentration of Ca in effluent from the column leaching test conducted on fly ash is higher than the soil tested alone (Table 8.3). The change in Ca concentration with the variation of fly ash percent in the soil-fly ash mix is shown in Fig. 8.19. It shows that addition of fly ash to the soil tends to proportionately increase the concentration of Ca in the soil-fly ash mixes.

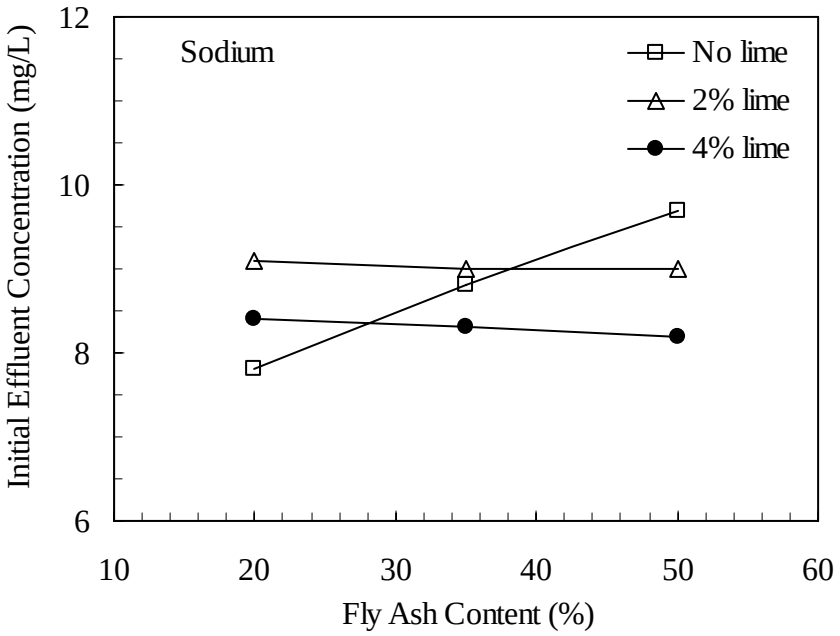


Figure 8.17 Variation of Na concentration in initial effluents from column leaching test on soil for varying fly ash and lime content

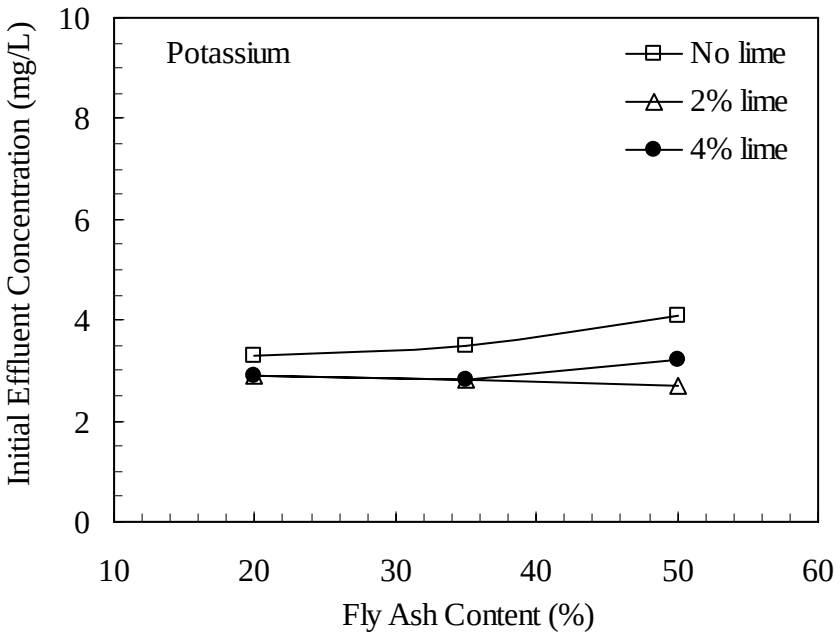


Figure 8.18 Variation of K concentration in initial effluents from column leaching test on soil for varying fly ash and lime content

When equal amount of calcium in the form of lime was supplemented to each of the soil-fly ash mixes, similar increase in Ca concentration is noticed for 2% lime (Fig. 8.19.). With the increase in lime content to 4%, mixes with higher fly ash content show much higher increase in Ca concentration. This is possibly due to more consumption of calcium by the mixes with low fly ash content for their higher pozzolanic activity resulting in the formation of insoluble complex calcium compounds.

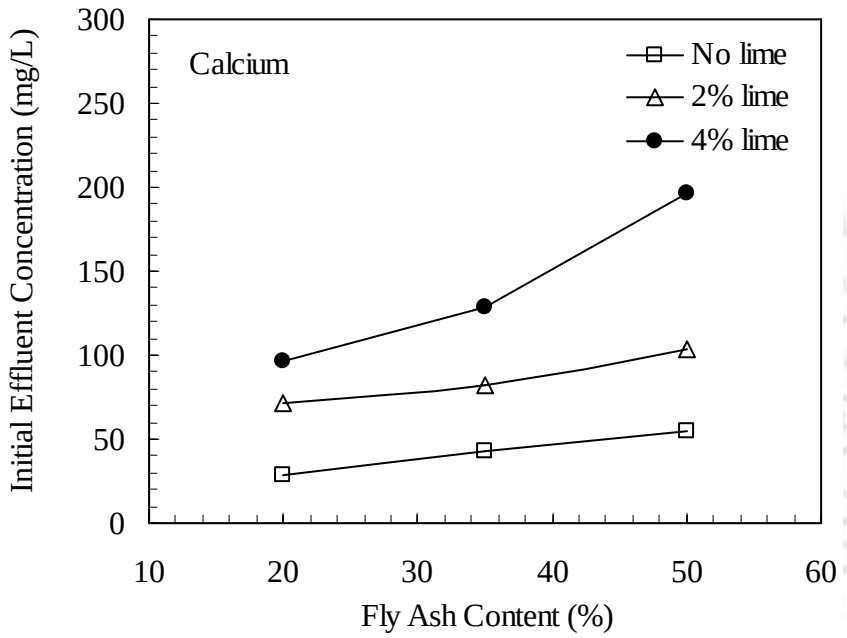


Figure 8.19 Variation of Ca concentration in initial effluents from column leaching test on soil for varying fly ash and lime content

8.3.2.4 Release Pattern

Garavaglia and Caramuscio (1994), on the basis of study on leaching behaviour of coal fly ash, distinguished three types of releases: (a) rate-limited release (b) solubility controlled release (c) adsorption controlled release. When the release is controlled by the dissolution rate of the precipitated metal compounds, then it is called rate-limited release. Rate limiting occurs in non-equilibrium situations. For rate-limited release, the

concentration of the leachate depends on the rate of dissolution, and leachate concentration remains unchanged over time unless the precipitated compounds are fully dissolved or the rate of release changes.

Solubility-controlled release also occurs in non-equilibrium situations, when the dissolution rate is infinite and dissolution is controlled by solubility. The concentration of the leachate depends on the solubility of a compound in pore water and remains at the solubility limit of the compound.

For adsorption-controlled release, the concentration of metals in pore fluid continually changes, because the metal concentrations in the solid and liquid phase are controlled by the partition coefficients. Adsorption-controlled release corresponds to an equilibrium condition where desorption occurs instantaneously.

The release patterns can be classified into two basic shapes (Edil et al., 1992). Fig. 8.20 shows two general types of responses that were observed for species (not originally present in the influent) that were released from fly ash/ soil matrix.

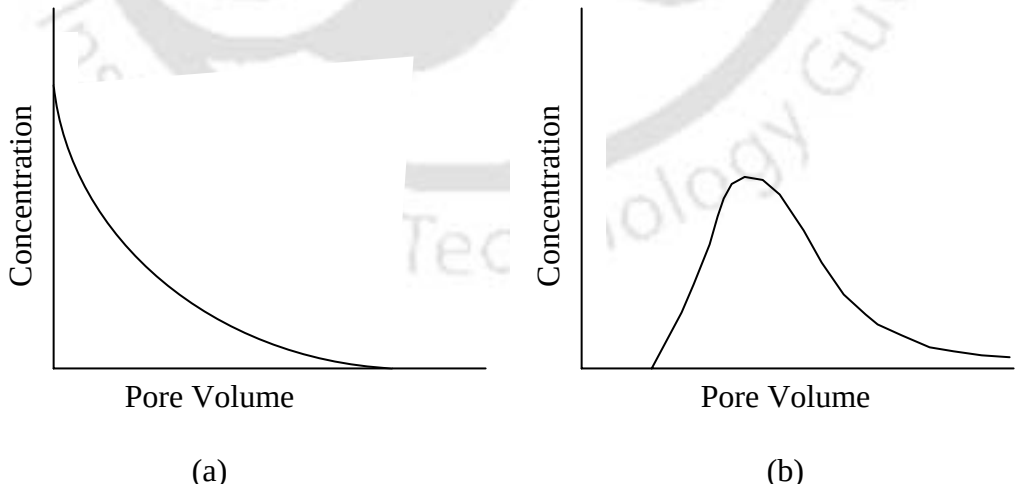


Figure 8.20 Typical effluent response curves of elements released from fly ash (a) first-flush response (b) lagged-flush response

These responses are called first-flush response and lagged-flush response. The first-flush response occurs when a chemical species is soluble under the initial permeation conditions (high pH, high soil matrix concentration, or low solution concentration). The decrease in concentration with time could be due to drop in solubility as the pH and alkalinity decrease or a drop in the species concentration in the soil matrix. The lagged response represents the opposite initial response of the first flush. Initial condition foster retention of species that becomes more soluble as the pH and alkalinity decrease.

The release pattern of each metal species from the soil, fly ash, and their various mixes with lime obtained from the column leaching tests are shown in Fig. 8.21 through 8.31. The same figures also present comparisons of the release pattern obtained from the soil-fly ash-lime mixes for variation in fly ash percentages with fixed lime content as well as release pattern obtained from the variations in lime content at different fly ash percentages.

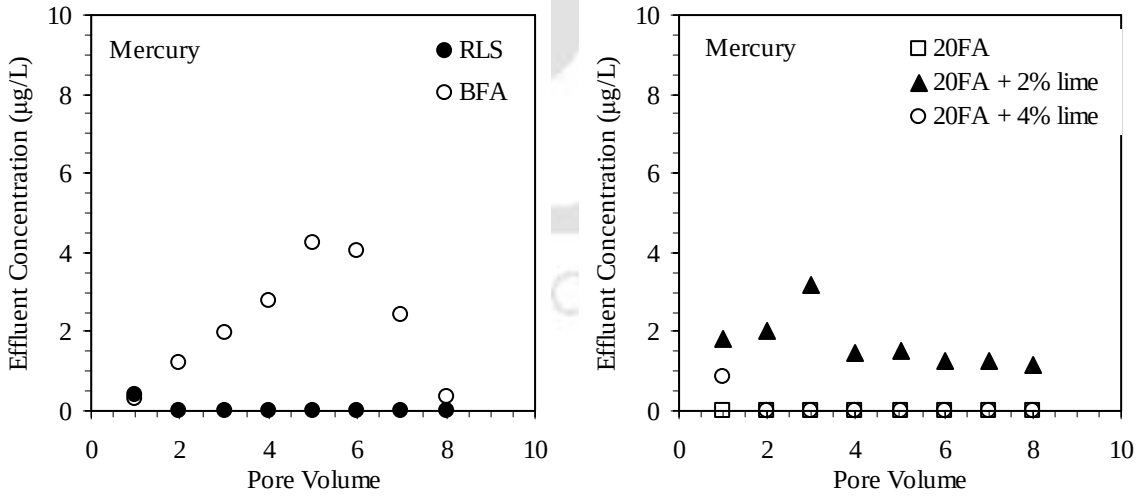


Figure 8.21 Change in concentration of mercury with permeation from column leaching test on soil, fly ash and its mixes with various amount of lime

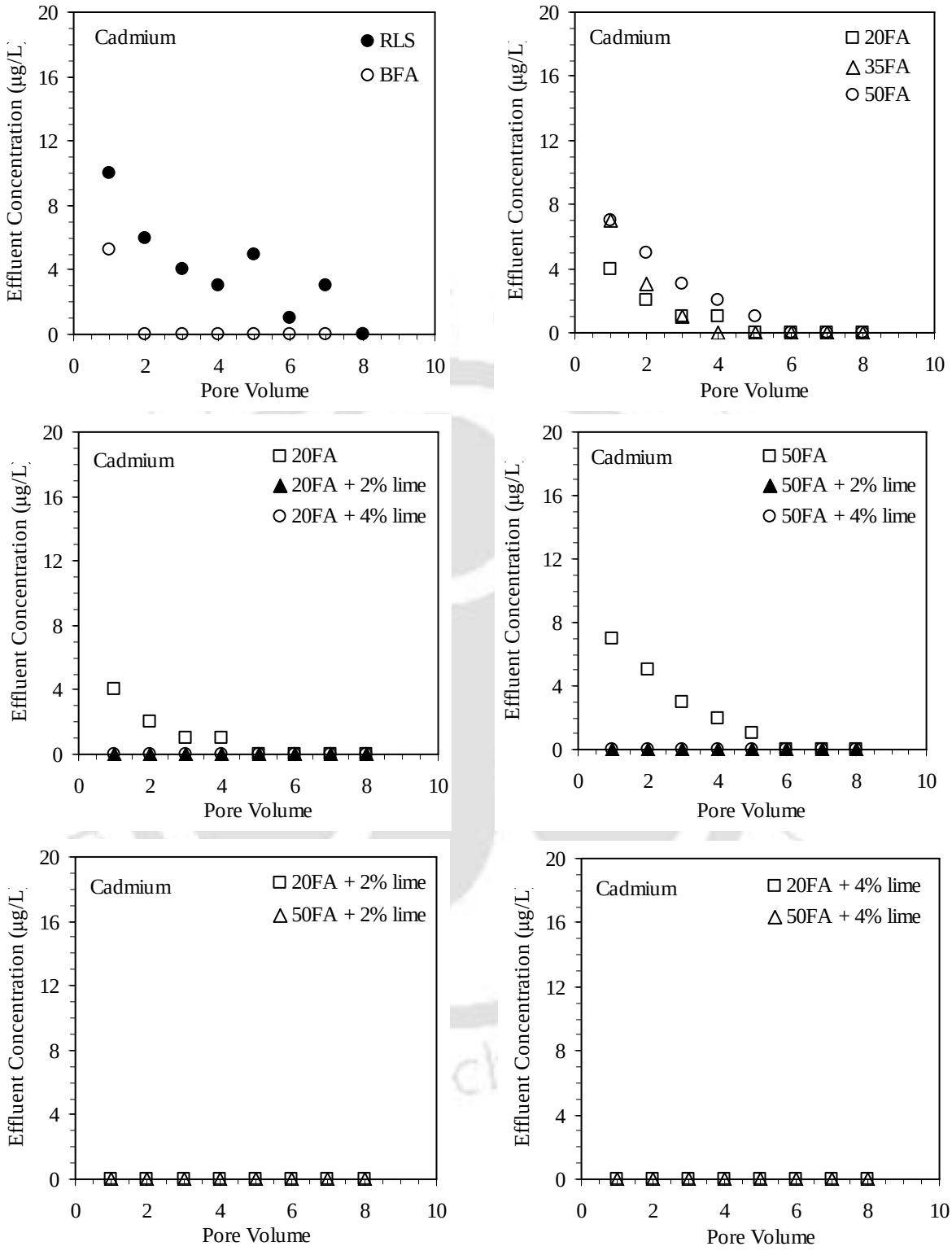


Figure 8.22 Change in concentration of cadmium with permeation from column leaching test on soil, fly ash and its mixes with various amounts of lime

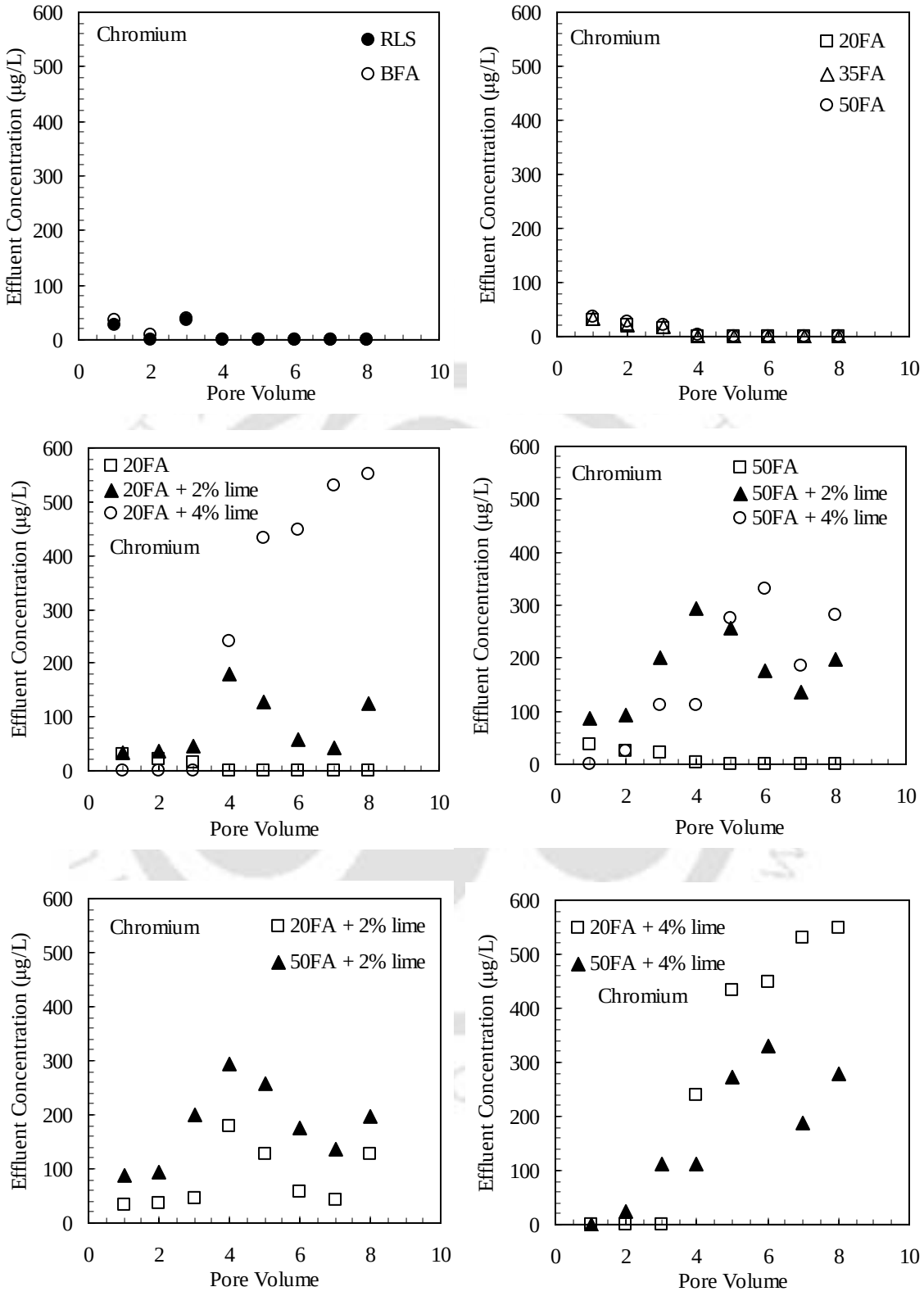


Figure 8.23 Change in concentration of chromium with permeation from column leaching test on soil, fly ash and its mixes with various amounts of lime

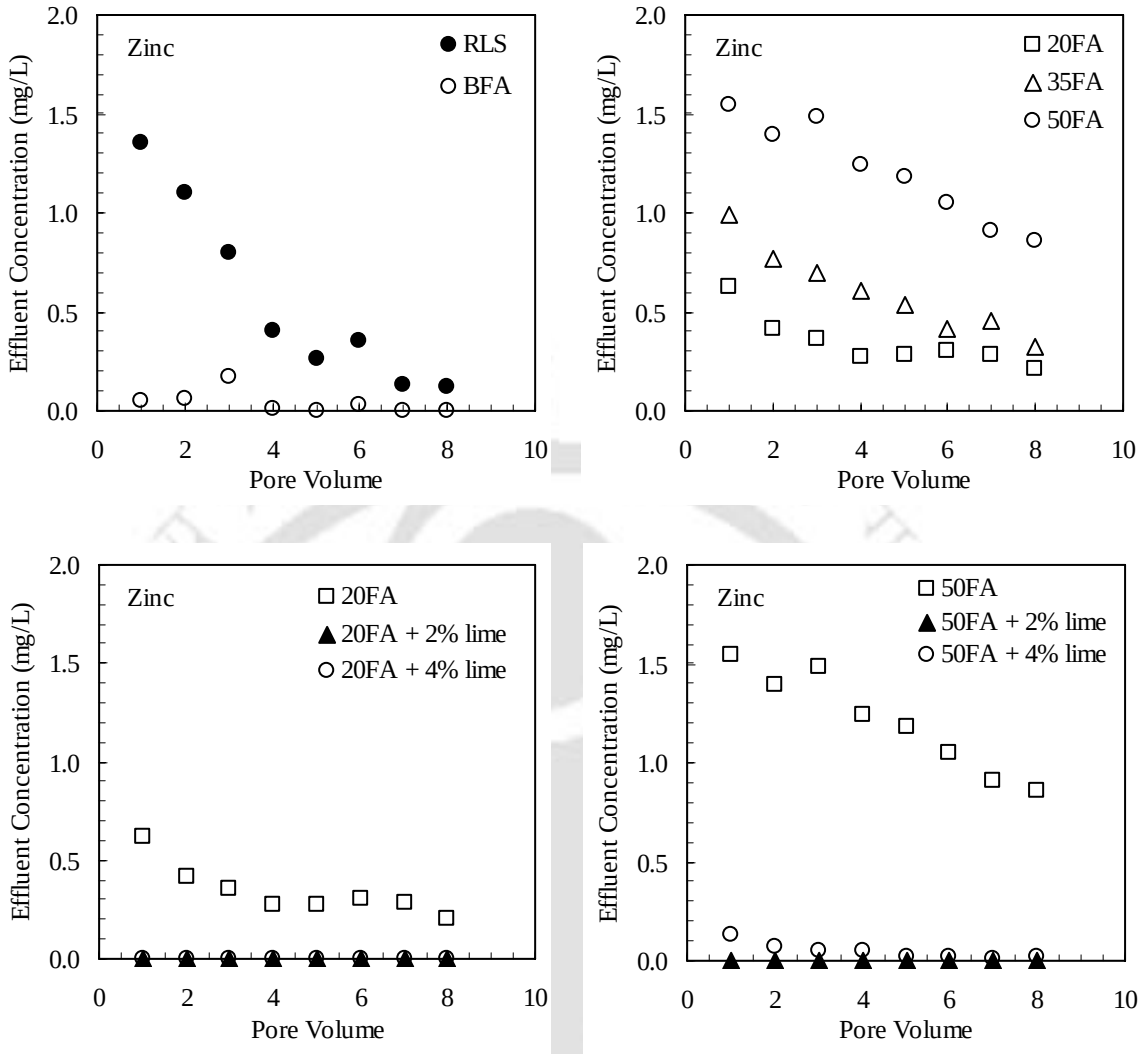


Figure 8.24 Change in concentration of zinc with permeation from column leaching test on soil, fly ash and its mixes with various amounts of lime

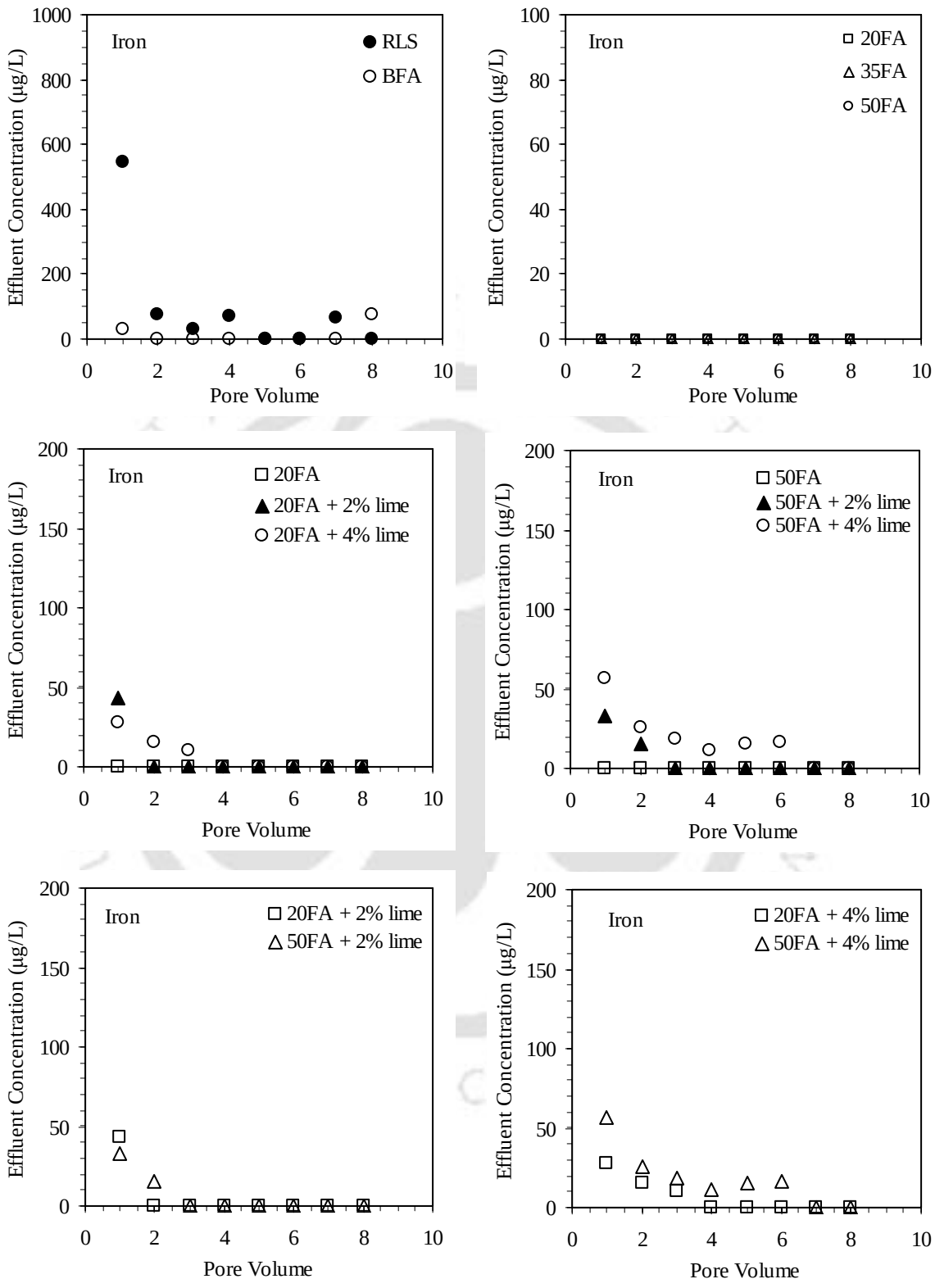


Figure 8.25 Change in concentration of iron with permeation from column leaching test on soil, fly ash and its mixes with various amounts of lime

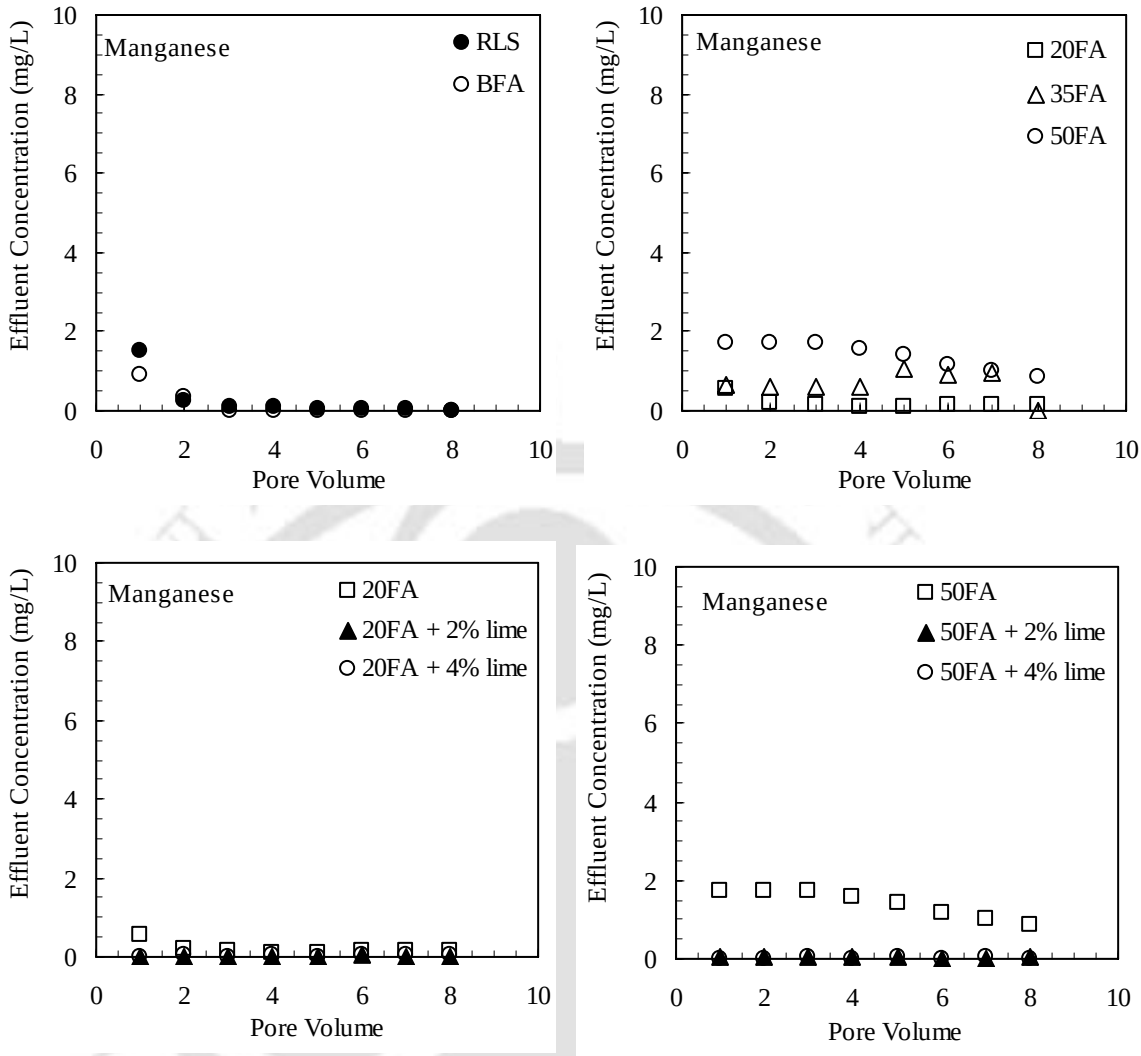


Figure 8.26 Change in concentration of manganese with permeation from column leaching test on soil, fly ash and its mixes with various amounts of lime

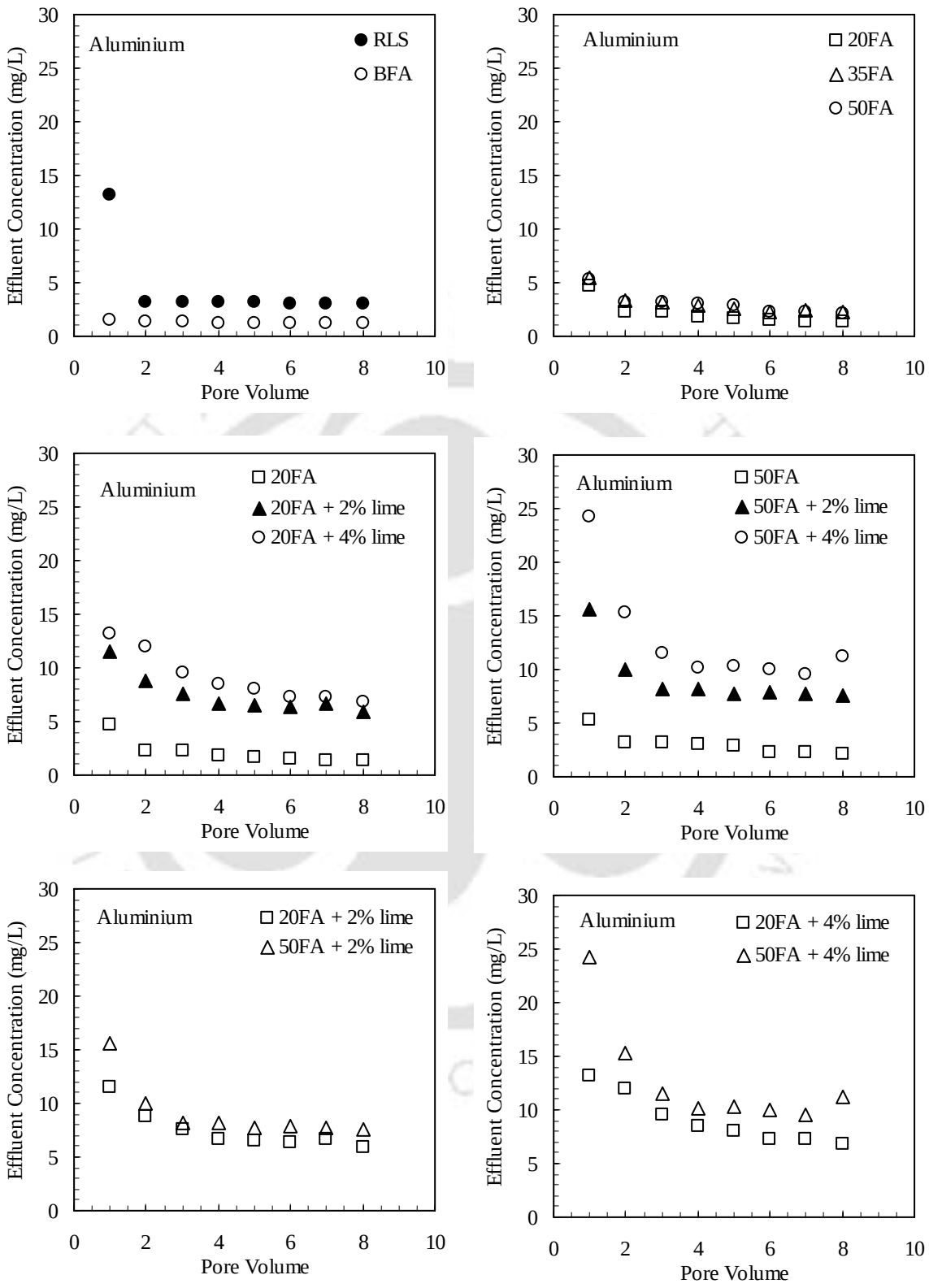


Figure 8.27 Change in concentration of aluminium with permeation from column leaching test on soil, fly ash and its mixes with various amounts of lime

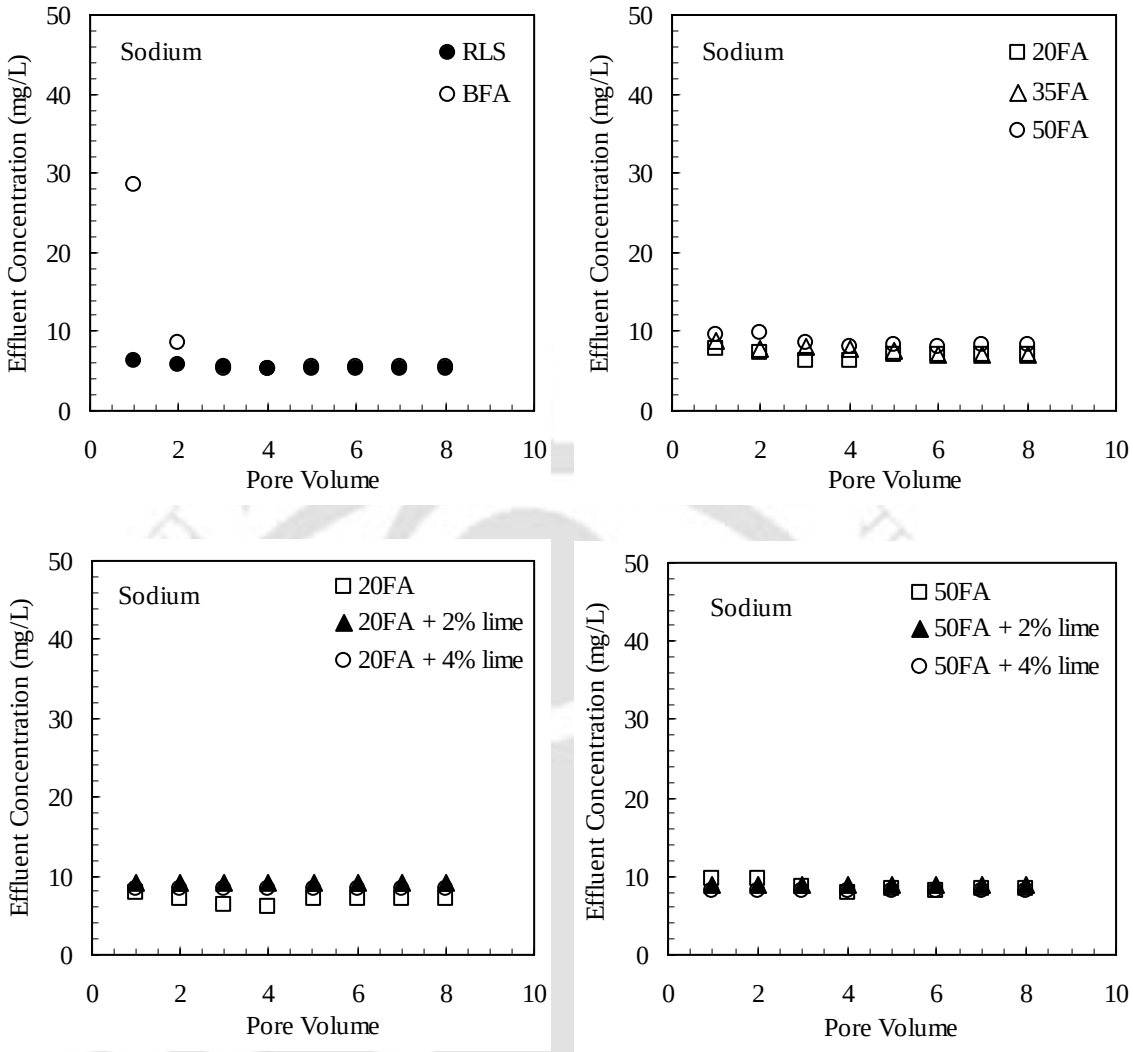


Figure 8.28 change in concentration of sodium with permeation from column leaching test on soil, fly ash and its mixes with various amount of lime

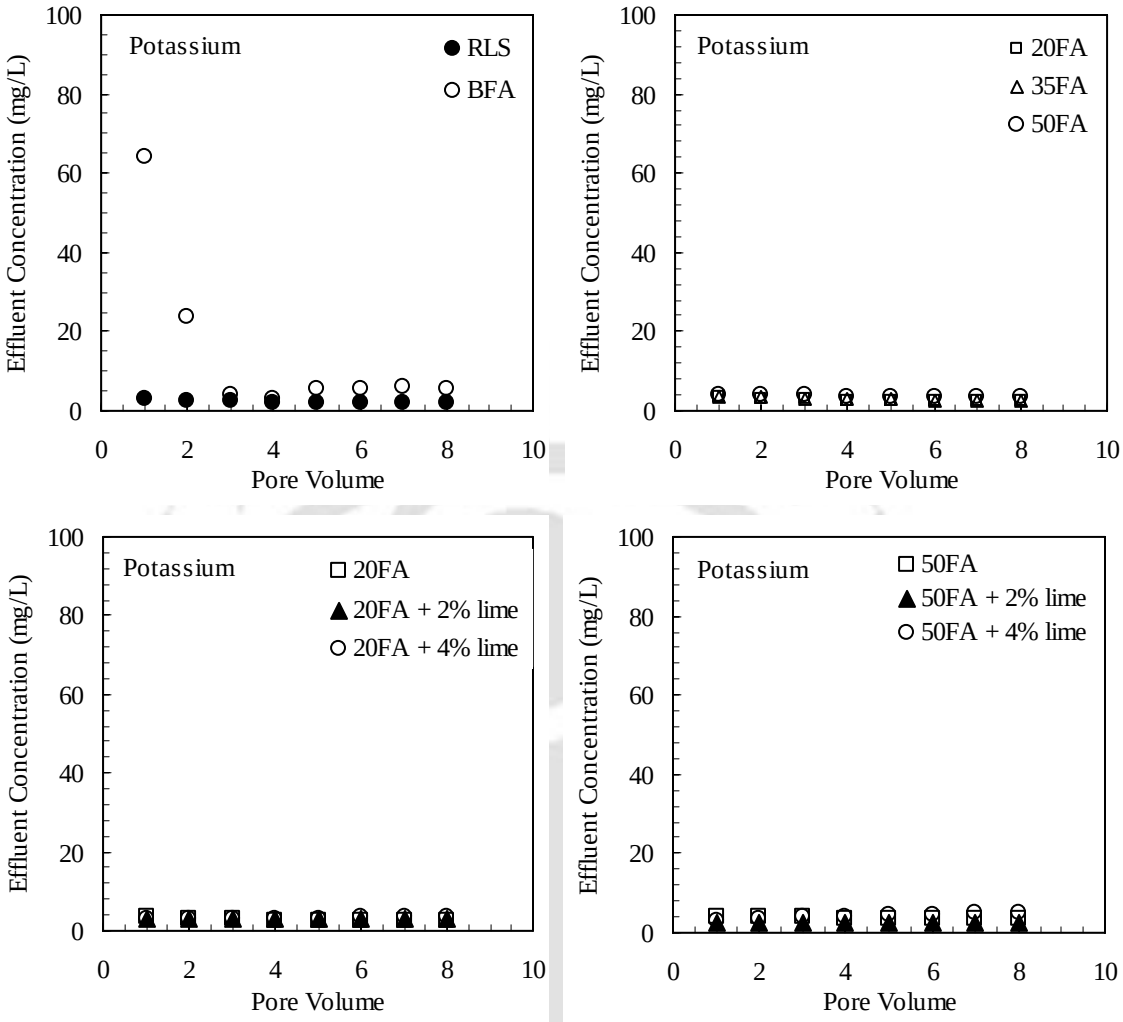


Figure 8.29 Change in concentration of potassium with permeation from column leaching test on soil, fly ash and its mixes with various amounts of lime

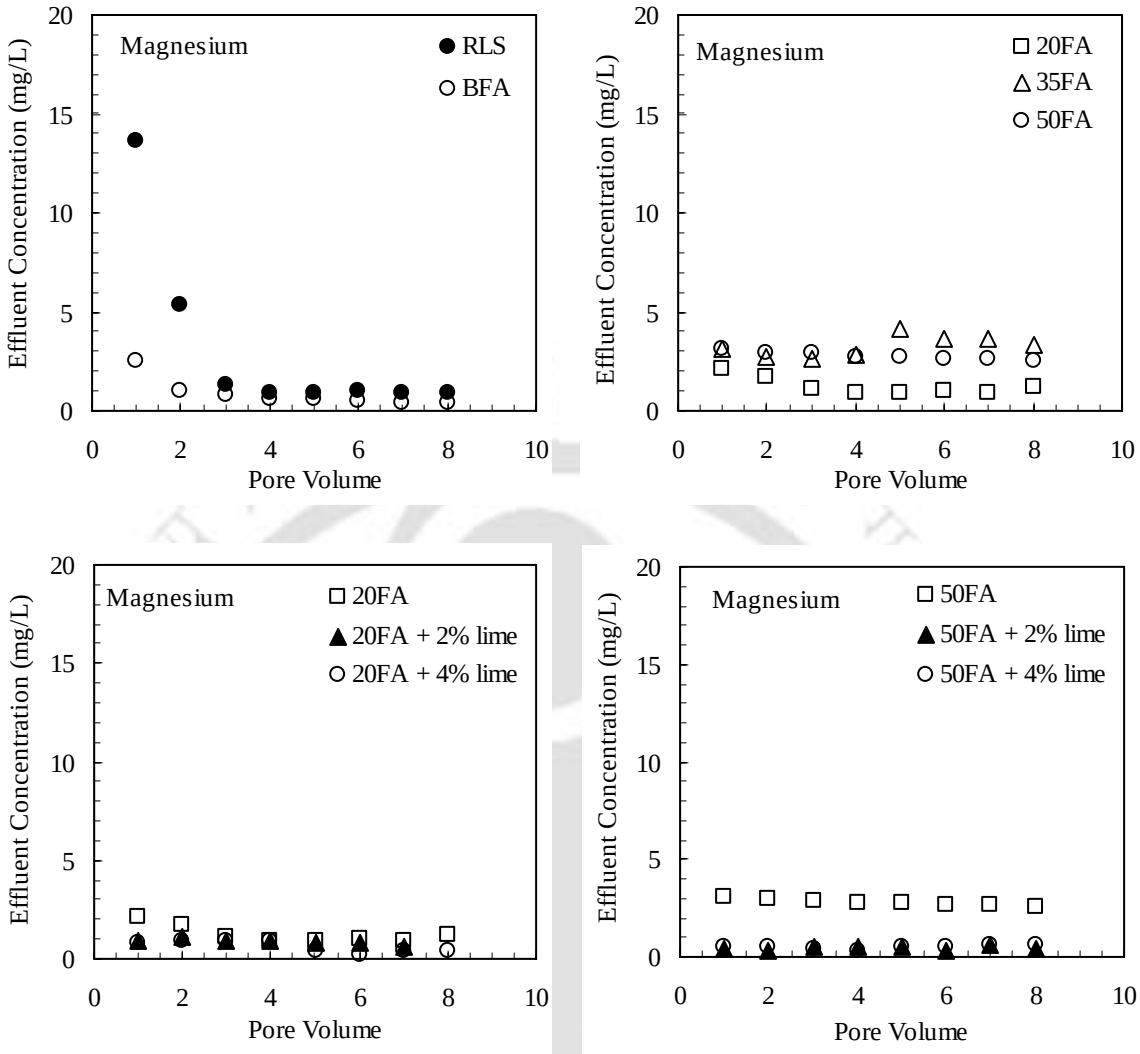


Figure 8.30 Change in concentration of magnesium with permeation from column leaching test on soil, fly ash and its mixes with various amounts of lime

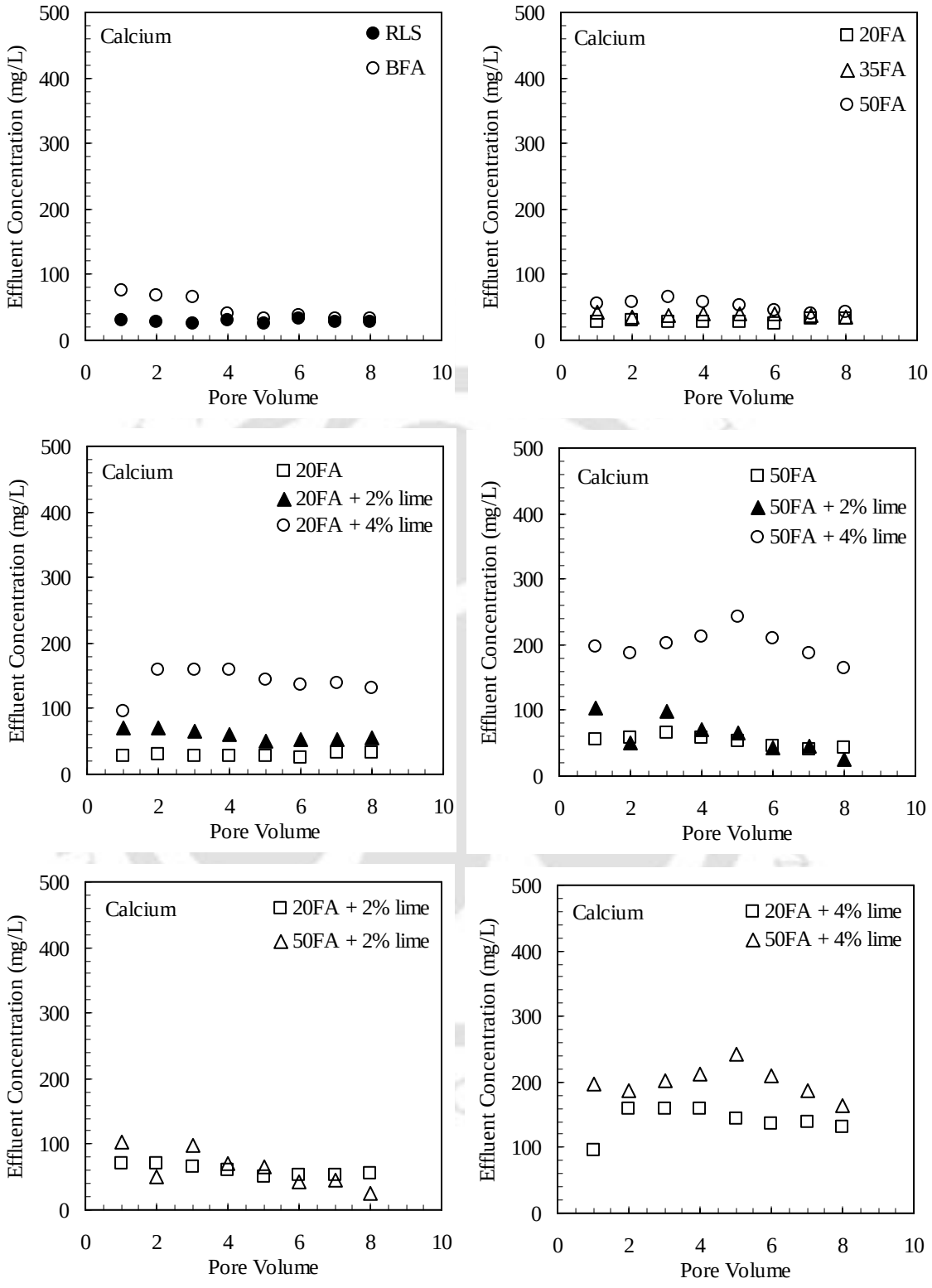


Figure 8.31 Change in concentration of calcium with permeation from column leaching test on soil, fly ash and its mixes with various amounts of lime

Release Pattern from Experimental Soil and Fly Ash:

Except Zn, Hg, and Al, the concentrations of most of the metal species, released from the fly ash decrease with time and exhibit a release response similar to the first-flush type, as discussed earlier. Metals like Cd, Cr, Fe, and Mn released from the fly ash depict typical first flush type of response and their concentrations decrease to a level beyond the detection limit after permeation for a few pore volumes (Figs. 8.22, 8.23, 8.25, and 8.26).

In case of the highly soluble species like Na, K and Mg, the concentrations from fly ash rapidly decrease with permeation and reach an equilibrium concentration after a few pore volumes (Figs. 8.28, 8.29, 8.30).

The concentration of Hg release from fly ash, on the other hand, gradually increases with time and decreases after reaching a peak value showing a response somewhat similar to the lagged-flush type (Fig. 8.21). Zn and Al show no significant change in concentration and it appears that equilibrium is achieved at the very initial stage of permeation (Figs. 8.24 and 8.27).

In case of the soil, release pattern of most of the metal species differ from the fly ash during initial few pore volumes. Concentrations of Zn, Cd, Fe, Mg, Mn, Hg, Al, and Cr decrease with permeation and resemble with first flush type of response. Among them, Zn and Cd concentrations decrease slowly but continuously, while for the rest of the species, the concentrations decrease rapidly. The pH of the effluent released from the soil is 3.7 and it remains unchanged with permeation. Solubility of most of the metals at such low pH is high and so, decrease in concentration can be attributed to drop in the species concentration in the soil.

As described earlier, most of the freely available Na and K along with Ca in the soil have already leached out due to their high solubility and good drainage condition during soil genesis. Therefore, the effluent concentrations of these species, released from the soil are small but continuous and possibly in equilibrium with the permeant concentration.

Release Pattern from Soil-Fly Ash Mixes without Lime:

Except for Cd, Mg, Mn, Hg and Fe, no significant change in release pattern of the metal species was observed with the replacement of the soil with various amounts of fly ash. In the case of Cd, its concentration, released from a given soil-fly ash mix, do not vary substantially with permeation (Fig. 8.22). For Mg and Mn, difference in their release pattern with respect to the soil alone was noticed only for the 50% fly ash replacement, where, decrease in species concentration is somewhat delayed and occurs only after the passage of a few pore volumes.

In case of Hg and Fe, initial effluent concentrations were beyond detection limit after the addition of fly ash to the soil and it remained so throughout the test. Effluent concentrations of Cr, Zn, and Al released from the soil-fly ash mixes resemble the release pattern of the soil alone and their concentration decreased with permeation. The concentrations of Na, K, and Ca released from the soil-fly ash mixes, on the other hand, remained constant throughout the test.

Release Pattern from Soil-Fly Ash-Lime Mixes:

Comparisons of the effect of increasing lime content on the release pattern from the soil-fly ash-lime mixes at various fly ash percentages for each element, presented in Figs. 8.21 though 8.31 shows that metal species like Zn, Mg, Mn, Na, and K do not exhibit significant difference in release pattern on addition of lime to the mixes. The effluent

concentrations either remain the same or slightly decrease with permeation, as shown in the figures. In case of Cd, whose concentration dropped below detection limit, also remains unchanged even with permeation (Fig. 8.22).

As described earlier in Section 8.3.2.3, lime treatment increases the initial effluent concentration of Fe and Al released from the soil-fly ash-lime mixes. The concentrations, however, decrease with permeation and the release patterns are similar for all the fly ash content, varying only in intensity. Mixes with higher fly ash content show more reduction due to higher initial effluent concentrations.

The effluent concentrations of Cr released from the soil-fly ash-lime mixes depict a completely different release pattern with the addition of lime. Comparisons presented in Fig. 8.23 show that release of Cr from the lime-treated mixes is somewhat delayed and its concentration increases with permeation. The extent of change, however, depends on the fly ash and lime contents. In case of the mix with 50% fly ash (50FA), the release pattern appears to be independent of the lime content. But, with 20% fly ash (20FA), 4% lime shows much higher increase in effluent concentration compared to the 2% lime addition. At 4% lime, mixes with low fly ash content results in higher release of Cr with permeation; but, with 2% lime, the release pattern appears to be independent of fly ash content.

Hg was found to be present only in 20FA with 2% and 4% lime. With 2% lime, the Hg concentration, in general, gradually decline with permeation. With increase in lime content, however, its concentration rapidly decreases beyond the detection limit immediately after 1st pore volume (Fig. 8.21).

Comparisons of the release pattern of Ca from the soil-fly ash lime mixes (Fig. 8.31) shows that its concentration for 2% lime addition marginally but gradually reduce

with permeation, possibly due to decline in species concentration as well as its consumption in pozzolanic reactions. With increase in lime content, however, Ca concentration tends to remain constant with permeation and its release appears to be controlled by the solubility of the species.

8.5 CONCLUSIONS

1. Aqueous phase concentrations of the metals obtained from single batch leaching tests show that none of the metals exceed the threshold limits for any combination of soil, fly ash and lime. Except Cd and Cr, concentrations of all other metals are below the allowable limits for all the mix proportions.
2. Leaching potential of a metal from soil-fly ash mixture depends on the metal concentration in the fly ash and soil as well as pH of the leachate. Irrespective of their original concentration in fly ash and soil, the leaching potential of almost all the metals, except Hg and Ni, in general, tend to reduce with the addition of a low volume (20%) of fly ash to the soil. With the increase in fly ash content, however, the concentration gradually increases barring Ni and Hg.
3. When lime is added to the mixes, leaching potential, in general, increases for Fe, Ni, Mg, Mn, and Al; decreases in case of Hg and Zn; and remains almost similar for Cd, Cr, Na and K along with Cu and Pb. However, the quantum of change and its direction depends on the amount of soil replaced with fly ash and it varies from species to species.
4. The pH of leachates obtained from column leaching test on soil shows strong acidic nature of the residual lateritic soil due to the decomposition of aluminosilicates and

accumulation of Al^{3+}. The pH of leachate collected from the fly ash is near neutral in nature.

5. Throughout the tests, pH of effluents obtained from different soil-fly ash mixes remained within a close range of 5.7 to 6.0 due to the high buffering capacity of the soil, produced by the variable surface charge minerals like sesquioxides of iron and aluminum present in the soil. Addition of lime to the soil-fly ash mixes appreciably increases the pH of the leachate (more than 11.7) and does not vary significantly with fly ash content.
6. Addition of small amount of fly ash to the soil marginally reduces the hydraulic conductivity. However, it increases rapidly with the increase in fly ash content. Addition of 2% lime to the mixes immediately increases the hydraulic conductivity further though it gets reduced marginally at higher lime content.
7. Permeation causes little change to the hydraulic conductivity of soil, fly ash and their various mixes. However, addition of lime and resulting hydration products gradually decreases the hydraulic conductivity with progress of permeation.
8. The initial effluent concentration from the column leaching tests of all the mixes varies non-linearly with fly ash content. The non-linearity exists due the variation in pH of the mixes with fly ash and lime content, which affects the solubility and adsorption of metals.
9. Study of release pattern of metal shows that except for Cr and Hg, release patterns of most of the metals resemble with the first flush type. The release pattern of Cr and Hg shows a release pattern similar to the lagged type of response.

CHAPTER
NINE

SUMMARY AND CONCLUSIONS

Due to their inherent properties discussed in the previous chapters, residual lateritic soils exhibit a number of constructional difficulties that chiefly relate to workability, field compaction and strength particularly under wet and moist climate. On the other hand, very little is known about the role of fly ash in changing the properties of residual lateritic soil, particularly about those derived from the decomposition of granites and gneisses. This has been emphasized and examined in detail in the present work with specific reference to the plasticity, compaction and strength behaviour of a highly weathered residual lateritic soil modified with fly ash alone, as well as in conjunction with lime. These properties were investigated by suitably varying the proportions of fly ash (up to 50%) and lime (up to 8%), focusing upon the short-term and long-term modifications, various effects of conditioning/delay in compaction, durability of the modified material and effects of flooding or wetting as well as the effect on the environment due to leaching from the stabilized material. All investigations were carried out with pre-test sample preparation technique and test procedure selected based on a series of initial investigations carried out on the soil taking into account the recent findings about the behaviour of these soils. The experimental investigations undertaken and results obtained, followed by analysis of the results, have already been discussed in

detail in different chapters. Summary of the results and the major conclusions are presented in the current chapter.

Keeping in view the observations made in the literature, the effects of pre-test sample preparation technique and that of the test procedure on the index and compaction properties of the residual lateritic soil were studied by varying the sample pre-drying conditions, soil gradation and the test procedure. The specific gravity of the soil increased marginally with oven drying but remained almost unaffected by the air-drying process. Considerable change in grain size distribution occurred for wet and dry sieving due to aggregation of soil particles upon drying. Oven drying caused greater aggregation than air-drying, the latter also resulting in considerable decrease in liquid limit and plasticity index. The finer fractions of the soil produced more pronounced effects on drying. Increase in sample mixing duration, on the other hand, increased the consistency limits. The particle aggregation due to oven drying also resulted in marginal increase in MDD and reduction of OMC. Except marginal reduction of OMC, air-drying had no significant effect on compaction characteristics. The variation in MDD and OMC due to the change in maximum grain size of the soil was also negligible. In its undisturbed state and at low confining pressure, the soil exhibited little influence of the structure. Remoulding caused negligible variation in the effective strength parameters but compaction significantly increased the ϕ' value. Based on the above observations, only air-dried soil was used for the investigation and a fresh sample was used every time to obtain each point of the flow and compaction curve.

In order to improve the workability of the soil, independent and combined effects of various percentages of fly ash and lime on the plasticity properties of the soil were studied for different sample-conditioning periods. Test results showed that after a one-day

conditioning period, the soil exhibited limited response to lime stabilisation. The independent effects of addition of lime on plastic limit and liquid limit of the soil were limited and contrasting in nature (reduction in liquid limit and increase in plastic limit). Their aggregate effect, however, resulted in a net reduction of plasticity index that improved the workability of the treated material to some extent. Conditioning further reduced the plasticity index, thereby considerably improving the workability of the lime treated soil.

Addition of fly ash alone to the soil significantly changed its plasticity characteristics. Increasing fly ash content in the mixes caused reduction of both plastic limit and liquid limit. Their net effect increased the plasticity index mainly due to a relatively greater reduction of plastic limit. However, this effect, to a large extent, depended on the comparative grain size distributions of the soil and fly ash as well as free lime content of the fly ash. Increasing sample conditioning period resulted in little change in the above properties.

Lime added to the soil in conjunction with fly ash resulted in appreciable reduction of plasticity index. The change was due to the considerable increase in the plastic limit that rendered the mix almost non-plastic after full modification. Conditioning further improved the workability of the soil-fly ash-lime mixes, mainly due to significant increase in the plastic limit and concomitant reduction of the plasticity index. The modifications were mainly caused by cation exchange and/or flocculation. Most of the changes were affected in the first 28 days of conditioning, and beyond this period, the rate considerably slowed down.

The compaction characteristics of the soil, fly ash and their various mixes, both in absence and presence of lime, were studied. The MDD of the soil did not get significantly

altered when replaced with 20% fly ash by weight, but decreased with further increase in fly ash content. The OMC initially decreased at 20% fly ash content and then gradually increased with higher fly ash content. In general, addition of lime to the soil-fly ash mixes resulted in marked reduction in MDD and increase in OMC. The change in MDD of the soil-fly ash mixes with the variation in quantity of lime correlates well with the respective change in plastic limit. In general, greater incremental change up to 2% lime addition, and lesser modification at higher lime contents, was noticed. For any lime percentage, the magnitude of change was observed to be proportional to the percentage of fly ash content, thereby indicating a marked influence of fly ash addition.

The effects of progressive particle breakage and successive re-compaction on the compaction behaviour of the mixes were studied by a new procedure using the compaction tool used for light compaction test. Progressive particle breakdown and successive re-compaction considerably increased the MDD and decreased the OMC of soil-fly ash-lime mixes. A greater change occurred with higher fly ash content, suggesting greater impact of the hollow fly ash particles. Addition of lime further enhanced the effects due to the change in material texture from plastic to friable. It was indicated that the dry density of the soil – fly ash – lime mixes can be increased by as much as 7% through the process of successive re-compaction.

In order to evaluate the suitability of the soil and its mixes with fly ash and lime as a construction material, the stress-strain-strength characteristics were studied by unconfined compression test performed on raw and stabilised soil. The soil showed considerable improvement in its UC strength when compacted to MDD. But, it reduced significantly upon soaking. Addition of fly ash to the soil immediately reduced the UC strength of the mixes and changed their stress-strain responses. The overall stress-strain

characteristics of the soil-fly ash mixes remained unchanged with curing, except that the UC strength slightly increased with little change in the stiffness and brittleness of the material. Increased percentage of fly ash produces less strength gain with curing. At the end of long-term (480 days) curing, only the mix containing 20% fly ash showed slightly higher UC strength than the soil alone.

Addition of lime and subsequent curing resulted in remarkable change in the overall stress-strain-strength response of the soil-fly ash mixes. At 2% lime content, the stiffness of the soil-fly ash-lime mixes increased at all stages of curing. However, at slightly higher lime content (4%), the stiffness increased only during the initial stages of 7 days, beyond which, little further change was noticed. Also, the stiffness towards the later stages of curing was found to be practically independent of the fly ash content. The brittleness increased with the increase in lime content, and with curing the mixes behaved in an extremely brittle manner marked by rapid built up of stress and sudden collapse. The failure strain, after an initial decline, also increased with curing, and towards later stages it increased remarkably with peak strength. Although the difference in UC strength of the mixes was small when tested immediately after compaction, curing resulted remarkable gain in UC strength. Particularly at higher lime contents, soil-fly ash mixes exhibited similar rate of long-term strength gain. Their differences in the level of long-term strengths resulted only due to the difference in short-term strength gain. Highest long-term UC strength of the mixes resulted in case of the addition of 35% fly ash and 4% lime. When the lime addition was limited to 3%, mix containing 20% fly ash showed the maximum strength.

The soil-fly ash-lime mixes showed a relatively early strength generation due to the onset of pozzolanic reaction that largely occurred after an induction period of 7 days.

This was due to the presence of naturally available free alumina and iron oxide in lateritic soils. Although the necessary free silica required for pozzolanic reaction was not available in the soil, addition of fly ash supplemented the silica and contributed to early strength generation. The rate of strength gain during the induction period was found to depend on the lime fixation values of the mixes.

It was indicated that the loss of strength commonly observed in residual lateritic soils on soaking or flooding can be significantly reduced by the combined addition of fly ash and lime and subsequent curing after compaction. The reason for the improvement can be attributed to the increased bonding developed due to the hydration reactions between lime and fly ash.

The effects of time-dependent solidification reactions on the compaction and strength behaviour of the mixes were studied by delaying the compaction process up to 3 days. For soil-fly ash mixes without lime, delay in compaction caused very little change in the MDD and OMC while marginally reducing the UC strength. The reduction in strength was found to decrease with increase in fly ash content. On the addition of lime, the MDD progressively decreased with delay. Comparable values of MDD were obtained after a compaction delay period of 3 days for both 2% and 4% lime contents suggesting that equivalent short-term modification is achievable with prolonged contact between lime, fly ash and soil particles. The OMC increased noticeably within 24 hours of delay, after which the increase was observed to be gradual. Delay between wet mixing and compaction caused considerable reduction in UCS of the lime-treated soil-fly ash mixes and the effect was found to increase notably with the increase in lime content. It was observed that mixes with higher short-term strength gain on curing showed greater reduction in UCS upon delay in compaction.

Consolidated undrained triaxial tests showed that treatment of soil with fly ash alone slightly lowered the peak friction angle with cohesion intercept remaining unchanged at zero value. Addition of a small amount of lime (2%) to the soil-fly ash mix significantly increased the peak friction angle with the appearance of a cohesion intercept and the effect resulted in considerable increase in peak deviator stress, brittleness, accompanied by reduction in failure strain. With further addition of lime, higher pozzolanic activity and cementation caused significant increase in both peak friction angle and cohesion intercept.

It was indicated that for maximum strength, the stabilised soil must be compacted at optimum moisture content. Compaction at moisture content other than the optimum is likely to cause reduction in peak strength with wet side showing greater reduction than the dry side. Consolidated undrained test conducted at variable compaction conditions indicated that structure imparted by compaction considerably influenced the peak strength and brittleness but showed limited influence on the stiffness of the stabilised material. Compaction on dry side of the optimum resulted marginal reduction of the peak friction angle, while the reduction was found to be substantial for wet side of optimum. On dry side, the material behaved in a more brittle manner than the wet side. The structure imparted by compaction condition had limited influence on the cohesion intercept indicating limited influence of material structure on the cementation process. The behaviour is mainly attributed to the relatively high initial degree of saturation that caused very little variation in the water-to-binder ratio of the stabilised material even after the slight change of compaction condition

Analysis of flow data obtained from column leaching tests suggested that addition of small amount (up to 20%) of fly ash to the soil is likely to reduce the hydraulic

conductivity of residual lateritic soil. However, it would increase rapidly with the increase in fly ash content as well as addition of lime. Increase in pozzolanic activity due to increased lime addition would marginally reduce the hydraulic conductivity of the mixes. The effect is also likely to persist during continuous permeation and may result in marginal reduction of hydraulic conductivity of lime and fly ash stabilised mixes, particularly at higher lime content. In case of soil, fly ash and their various mixes without lime, no significant effects of permeation on hydraulic conductivity were noticed.

The environmental performance of the stabilised soil with respect to potential and realistic estimates of leaching for metals and their variations due to change in fly ash and lime content and the effects of continuous permeation were studied. The concentration of none of the metals exceeded the threshold limits for any combination of soil, fly ash and lime. Except for Cd and Cr, concentrations of all other metals remained within the allowable limits for all the mix proportions. Compared to the soil, the fly ash appeared to have played more active role in influencing the metal concentrations in the leachate derived from soil-fly ash mixes. Irrespective of their original concentrations in fly ash and soil, the leaching potential of almost all the metals, except Hg and Ni, in general, reduced with the addition of a low volume (20%) of fly ash to the soil. Addition of lime increased the leaching potential for Fe, Ni, Mg, Mn, and Al; while leaching potential decreased in cases of Hg and Zn; and remained somewhat similar in case of Cd, Cr, Na and K along with Cu and Pb. The quantum of change and its direction varied from species to species depending on the amount of soil replaced with fly ash. The initial effluent concentration of various metals obtained from the column leaching tests on the mixes, in general, showed non-linear variation with the change in fly ash content. The non-linearity occurred due to the variation in pH of the mixes with fly ash and lime content, which

affected the solubility and adsorption of the metals. Study of release pattern of metal showed that except for Cr and Hg, release patterns of most of the metals resembled the first flush type. The release pattern of Cr and Hg showed a release pattern similar to the lagged flush type of response.

Based on the above findings it is possible to provide broad guidelines for ascertaining appropriate site workability and potential for utilisation of both residual soil and bulk fly ash. Results obtained from the present investigation indicates that addition of fly ash having grain size finer than the soil and low hydration potential is likely to cause significant reduction in site workability of residual lateritic soil, which is evident from the continuous reduction in plastic limit of the soil with the increase in fly ash proportion. However, the apparent loss of workability of the soil-fly ash mixes can be successfully recovered by addition of adequate quantity of lime. Conditioning further improves the workability of the soil-fly ash-lime mixes, mainly due to significant increase in the plastic limit and concomitant reduction of plasticity. The improvement of the plasticity properties will allow the construction operations to continue under inclement environmental conditions typical of tropical and subtropical regions. The improved soil-fly ash-lime mixes show low dry unit weight, which will be beneficial for its use over soft formations as settlement of underlying soil will be considerably reduced. In view of predominant wet condition prevailing in most of these regions, increase in optimum moisture content due to addition of lime and fly ash would indirectly help in reducing the extent of drying requirement.

The present research shows that geotechnical application of fly ash has a good potential for facilitating bulk utilisation of fly ash. Replacement of the soil with 20% fly ash will cause very little change in the density and strength of the material, except slightly

lowering the workability of the soil. Increase in the replacement level up to 50% will require a minimum supplement of 2% lime to improve the workability and strength of the stabilised material to be equivalent of the soil. Any additional lime added to the soil-fly ash mix will cause significant further improvement, thereby remarkably enhancing its durability with time. In terms of compressive strength, the replacement of the soil by 35% fly ash appears to give the best long-term results after lime treatment. Another advantage of using fly ash and lime for the improvement the lateritic soil will be the significant reduction of the collapse potential, often exhibited by this type of soil. Further, since mixing before compaction is often recommended as such to ensure quality control in view of high spatial variability of properties of residual soil, use of fly ash and lime will involve no additional cost on account of mixing.

Unlike many other soils, the use of fly ash and lime for stabilisation of residual lateritic soil can be of great advantage to the practical utilisation due to the early strength generation shown by the treated material and thereby saving precious construction time. As the modification process resulting from the addition of fly ash and lime to the soil continues for a long time, the amount of lime required to achieve desired degree of improvement can be suitably adjusted depending on the time available and amount of fly ash to be added to the soil. The use of such admixtures will not have any significant impact on the environment, as most of the toxic metals usually present in fly ash will not leach out beyond their threshold limits. Application such as for liners of hazardous waste-fills however, may require specific studies to ascertain risk.

SCOPE FOR FURTHER RESEARCH

Given the immense scope for improvement of properties of residual lateritic soil with fly ash and lime as well as potential for bulk utilisation of fly ash, there remain many avenues for further research. Some of these possible areas are:

- Change in behaviour of the stabilized material when soils from different weathered zones are used and when combinations of soils from different horizons of the soil profile are attempted.
- In view of high strength generated due to the lime-fly ash stabilization of residual lateritic soil, further study of its performance for specific uses (e.g. for road construction).
- Evaluation of field performance of the stabilized material by conducting pilot field study followed by full-scale field trial.
- Effects of change in gradation and structure induced by excessive compaction (similar to successive re-compaction carried out) on the stress-strain-strength characteristics of the stabilized material.
- What changes take place during the solidification process of the lateritic soil-fly ash-lime mixes with respect to mineralogy and microstructure development and the role of sesquioxides in such process?
- Behaviour of the stabilized material under repetitive and dynamic loading.
- Retention behaviour of the stabilized material for various toxic metals and its performance as a liner material.
- Effects of long-term permeation on the performance of the stabilized material and potential impact on the environment.

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