A STUDY ON GEOTECHNICAL CHARACTERIZATION OF RESIDUAL LATERITIC SOIL AND BRAHMAPUTRA SAND BLENDED WITH FLY ASH AND CEMENT

Submitted in Partial Fulfillment of the Requirements for the Degree of

DOCTOR OF PHILOSOPHY

By

AJANTA KALITA

DEPARTMENT OF CIVIL ENGINEERING

INDIAN INSTITUTE OF TECHNOLOGY GUWAHATI

AUGUST 2011

ACKNOWLEDGEMENTS

I am highly grateful and obliged to my research guide Dr. Baleshwar Singh, Associate Professor, Department of Civil Engineering, Indian Institute of Technology Guwahati, for his sincere guidance, valuable advice, keen supervision and continuous encouragement during all the stages of research work without which, it would have been difficult for me to bring this dissertation at the final stage.

I extend my thanks to the members of my Doctoral Committee, Prof. S. Talukdar, Dr. A. K. Singh and Dr. S. Chakraborty for their time and effort in reviewing my progress.

The most essential aspect of this study was the laboratory testing, and it could not have been accomplished without the assistance of Mr. Hari Ram Upadhyaya, Mr. Mukul Kalita and Mr. Siva Kalita. I am also thankful to the scientific officers of Civil Engineering Department, Mr. A. Borsaikia and Dr. K. Pallav, for extending all possible support.

My sincere thanks to my friends Jeevan, Vinot, Sumesh, Abhishek, Pranjal, Minaxi, Swapnita, Madhuri, Albino, Kamal, Arpana, Rupak, Sandip, Shyam, Ravi, Vikram, Rimil, Sanjay, Lalit, Zela, Dhruba, Sanjeeb and Rupam who provided emotional support and friendship throughout the many stages of this thesis.

It is not possible to express in words the inspiration from my parents, sisters, brothers and Hari Shankar whose invaluable love, endless patience and unstinted support have brought me to this position. This work, in more than one sense, is a tribute to their trust and confidence in me. Lastly, I offer my regards to all of those who supported me in any respect during the completion of the project.

Ajanta Kalita

CERTIFICATE

This is to certify that the thesis entitled **"A Study on Geotechnical Characterization of Residual Lateritic Soil and Brahmaputra Sand Blended with Fly Ash and Cement"** submitted by **Ajanta Kalita** to the Indian Institute of Technology Guwahati, for the award of the degree of Doctor of Philosophy, is a bonafide record of research work carried out by her under my supervision. The contents of this thesis, in parts or in full, have not been submitted to any other Institute or University for the award of any degree or diploma.

Date:

Dr. Baleshwar Singh
Associate Professor
Department of Civil Engineering
Indian Institute of Technology Guwahati
Guwahati – 781 039, INDIA

TH-1088_AKALITA

Dedicated to my parents

TH-1088_AKALITA

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

ABBREVIATIONS

CBR	California Bearing Ratio
LL	Liquid Limit
MDD	Maximum Dry Unit weight
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plastic Limit
UC	Unconfined Compression
UCS	Unconfined Compressive Strength
CU	Consolidated Undrained

TH-1088_AKALITA

NOTATIONS

c	Cohesion intercept
c'	Cohesion intercept based on effective stress
c_{cu}	Cohesion intercept based on total stress
c_d	Cohesion intercept based on drained test
d	Intercept of K_f line with the Y-axis
G	Specific gravity
LL	Liquid limit
p	Stress path parameter $(\sigma_1+\sigma_3)/2$
q	Stress path parameter $(\sigma_1-\sigma_3)/2$
σ_1	Major principal stress
σ_3	Minor principal stress
σ_1'	Major principal stress based on effective stress
σ_3'	Minor principal stress based on effective stress
ϕ	Angle of shearing resistance
ϕ'	Angle of shearing resistance based on effective stress
ϕ_{cu}	Angle of shearing resistance based on total stress
ϕ_d	Angle of shearing resistance based on drained test
α	Inclination angle of K_f line with the X-axis

ABSTRACT

The shear strength of soils is one of the important aspects to be considered in any geotechnical activity. The problems concerned with the bearing capacity of road subgrade, slope stability of embankments and the design of retaining structures are all dependent on the shear strength characteristics of soils. Depending on the application in hand and the requirements, one needs to carry out modification of the soil properties. There is ample scope for the bulk utilization of fly ash in soil mixes for geotechnical applications such as in the construction of roads, embankments and backfilling behind retaining structures.

In this research work, the characteristics of two soils, a fine-grained residual lateritic soil (red soil) and granular riverbank sand (Brahmaputra sand) blended with a low-calcium fly ash and ordinary Portland cement was studied through a systematic series of compaction tests, direct shear tests, CBR tests, unconfined compression tests and triaxial consolidated drained tests. Depending on the type of test, the maximum amount of soil replaced with fly ash by weight ranged from 50% to 90%. The amount of cement added was up to a maximum of 3 % to 5% by weight of the soil-fly ash mixes. This provided a wide range of gradation and texture of the mixes. Compacted specimens of the mixes were cured up to 28 days. Based on the maximum dry unit weight obtained from the compaction tests, the specimens for the remaining tests were prepared. The strength tests were conducted on the as-compacted specimens without saturating them. Direct shear tests were carried out only on specimens of sand and sand-fly ash mixes.

The results from the tests were analysed to examine the compaction and strength characteristics of the modified soils due to the application of fly ash and cement. The

obtained values will also be useful in judging the suitability of those proportions of the soil mixes for their use in diverse geotechnical applications that include the construction of subgrade and subbase of pavements. As the fly ashes produced in the same plant at different times can be different, and as locally available soils can vary widely in their properties, characterization of their mixes is a must for their field usage. It is to be noted that fly ash can be used to stabilize soil, and even soil can be used to stabilize fly ash as well.

TH-1088_AKALITA

CHAPTER - 1
INTRODUCTION

1.1 Introduction

In geotechnical engineering practice, the soils at a given site are often less than ideal and may cause damage to structures. Soils that are encountered by the practicing engineers in the field vary widely in their properties and in their response to any external stimulus. Not all soils respond favourably under all circumstances. When soils with unfavourable characteristics are met with in the field, they have to be either discarded in total or have to be treated for the modification of their unfavourable properties so as to suit the field requirements.

While stabilization has been used in nearly every type of soil engineering problem, its most common application is the strengthening of the soil components of highway and airfield pavements. The topmost soil layer over which the road section is built is known as subgrade. It is the result of earthwork operations, and it may consist of undisturbed existing soil or material excavated elsewhere and placed as fill. The subgrade acts as a foundation for the road and is normally of 30 to 50 cm thickness. The structural design of the road in terms of the thicknesses of the overlying pavement layers is dependent on the bearing capacity of this subgrade layer. Thus, subgrade quality has a striking impact on both the initial cost of the road and the subsequent maintenance costs. Options for dealing with weak subgrades include designing a thick and expensive road section, reinforcing the subgrade with a geosynthetic material, or applying traditional stabilizers such as lime and cement.

The subgrade soil can be amended so as to obtain an improved soil mix by using mechanical or chemical means. It involves adding one or more selected materials in different proportions with or without a binder. The added material can be another soil or available fly ash, bottom ash or pond ash. In mechanical amendment, the soil is made more stable by adjusting the particle size distribution. In chemical amendment, the most widely used binders are lime and Portland cement. The degree of improvement of an in-situ soil may differ between several alternative methods. The selected method should be tested and verified in the laboratory prior to construction and before laying down specifications.

Although the original objective of soil stabilization is to increase the strength or stability of soil, gradually techniques of soil treatment have been developed. Now, stabilization is used to increase or decrease almost every engineering property of soil as follows:

1. To increase strength or reduce the sensitivity of strength to environment changes, especially moisture changes
2. To increase or decrease permeability
3. To reduce compressibility
4. To reduce frost susceptibility

Fly ash is a fine-grained material of mostly silt size particles. The properties that pose problems in majority of the applications using fly ashes include the uniform gradation of particles and lack of cohesion. Even though they have very good frictional resistance, they may not have the cohesion component of shear strength. Uniform/poor gradation of the particles results in lower compacted density and higher permeability that are unwanted engineering properties in the construction of embankments and structural fills. The absence of inherent cohesion in certain fly ashes does not allow their use in applications that do not provide any confinement, in spite of their good performance from strength viewpoint when confined. In addition, lack of cohesion makes them highly dispersive, limiting their application in water front structures as they become highly erodible. Fly ashes are also highly frost susceptible, making them unfavourable for use in regions where temperature goes below freezing point.

Most of these limitations can be overcome, and the performance of fly ashes can be improved by using soil as mechanical admixture and lime or cement as the chemical admixture to achieve the strength required for use as base and subbase courses in pavements. Fly ash for soil stabilization has been a research focus for many years mainly due to the possibility of its bulk utilization. It is not necessary that only cohesive soils need to be stabilised always. Even cohesionless soils also need to be stabilised in specific cases.

This thesis examines the geotechnical characteristics of a residual lateritic soil and Brahmaputra sand blended with fly ash and cement.

1.2 Residual Lateritic Soil

Residual soils are those that form from rock and remain at the place where they were formed. 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. Lateritic soils are residual soils which are found in the humid tropical regions. High temperature and abundant rainfall are necessary for their formation. 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 conditions. This leads to the accumulation of sesquioxides at the top of the soil profile in elevated areas,

TH-1088_AKALITA

giving rise to the characteristic reddish colour.

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

In wet conditions during rainy seasons, there can be loss of strength associated with the development of high plasticity. As a result, handling of the soil becomes difficult during construction. Thus there is a need to amend the soil when used in the subgrade of paved roads or in the surface of earthen roads, so that the bearing capacity is optimum and durable throughout the intended design lifetime.

Deposits of lateritic soil formations are found extensively in many places over India (Fig.1.1). There are numerous occurrences over parts of Maharastra, Madhya Pradesh, Orissa, and West Bengal. Most of these soil formations show 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). These deposits are derived mainly from dolerite and basalt formation as well as from feldspathic rocks containing a moderate amount of alumina and silica, such as gneisses and granites.

Large deposits of lateritic soils are found in the northeastern part of India in the Shillong plateau, Mikir hills and their numerous extensions in the state of Assam. The average normal rainfall in the area is around 2300 mm/year. The hot and humid climate of this region is very much conducive to the formation of lateritic soil. These soils are also extensively found as capping in numerous isolated inselbergs formed by the projection of the above formations scattered along the 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

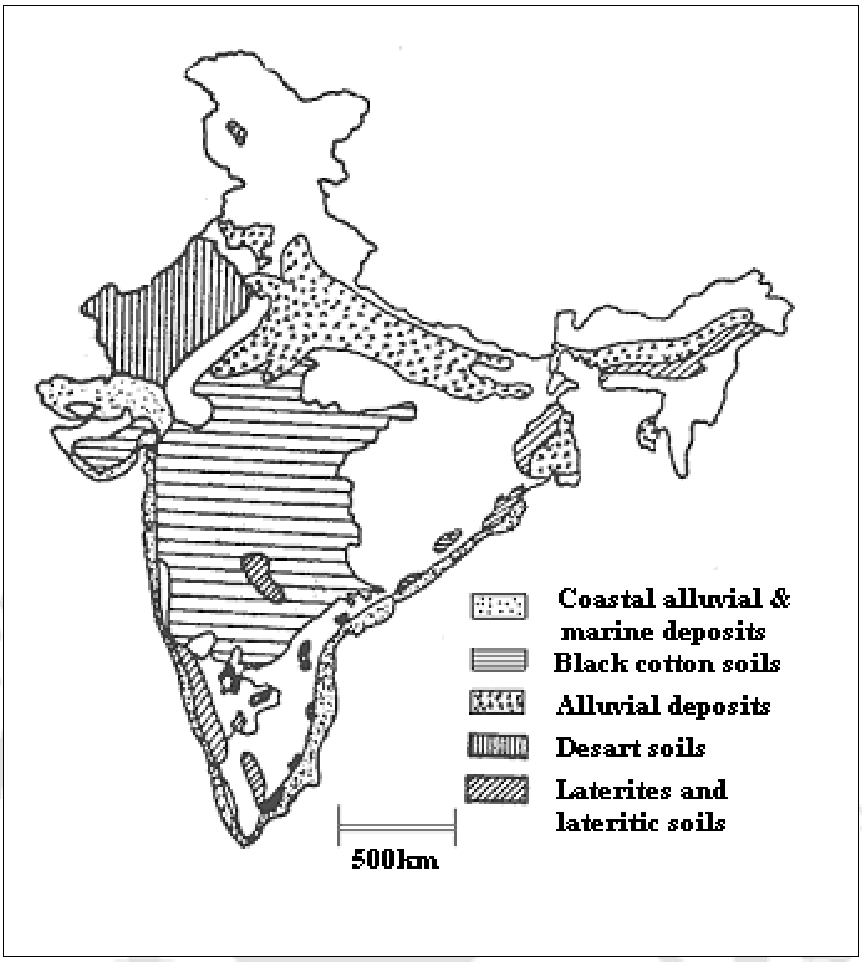


Fig.1.1 Distribution of soils in Indian mainland (after Katti et. al, 1975)

1.3 Brahmaputra Sand

The soils of the Brahmaputra River are classified according to the soil taxonomy and represent widely divergent soil groups. Climate has been the most dominant factor in formation and development of soils of the river. More specially, the rainfall and temperature have exercised the most conspicuous influence.The soils along the river have wide textural variations ranging from sand to clay. The flood plain soils are predominantly silty loam to sand. The upland soils are loam at the surface horizons and clay loam to clay at the lower horizons.

Flood plain soils of the Brahmaputra River contain a higher percentage of sand at lower depths. Sand content of upland soils is less than 28 per cent at the surface and it decreases further at lower depths. The fine and very fine fractions of sand together constitute the major portion of the total sand. Soils near the Brahmaputra River are coarser in texture and those away from the river are finer in texture.

1.4 Fly Ash

Fly ash is the mineral matter extracted from the flue gases of furnaces fired with coal in thermal power plants. Fly ash is classified into two classes based on the chemical composition according to ASTM C 618-99 (1999). Class F fly ash is produced from burning anthracite and bituminous coals, and contains a small amount of lime (CaO). This fly ash has siliceous and aluminous oxides, which give its pozzolanic property. This material itself possesses little or no cementitious value, but chemically reacts with added lime in the presence of moisture at ordinary temperature to form cementitious compounds (Chu and Kao, 1993). Class C fly ash is produced from burning of lignite and sub-bituminous coals, and usually contains a significant amount of calcium oxide (CaO). In addition to having pozzolanic properties, Class C fly ash also is cementitious. 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.

In 2005, the production of fly ash in India was about 100 million tons per annum and this is likely to touch 155 million tonnes per annum by 2020. In 1994, the Government of India commissioned Fly Ash Mission with the broad objective of building confidence among the producer and consumer agencies in the safe disposal and utilization of fly ash, through technology demonstration projects. Thus, fly ash utilization increased from 3% in 1994 to 13% in 2002.

To reduce the cost of disposal as well as to avoid environmental pollution, more and more innovative methods are being explored now days for fly ash utilization in many fields. Soil amendment is required in many places because soils vary from place to place and the engineering properties are also equally variable. Because of its pozzolanic character, fly ash can contribute 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 reactions 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. The reaction products are normally expressed as hydrated calcium silicate gel or calcium aluminate gel and they can bind inert materials together. 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 soils to some extent, react in a similar manner when lime is added to soil for stabilization purpose.

1.5 Objective and Scope of the Present Study

It is evident that fly ash disposal problem in India is of great concern. The use of fly ash directly or in combination with abundantly available local soils, would facilitate its mass utilization. Keeping view of the above, the main objective of the present study is to understand the effect of a low-calcium fly ash on the engineering characteristics of two local soils, namely, a fine-grained residual lateritic soil and granular riverbank sand. Ordinary Portland cement is to be used as an additional admixture in small quantities. This will bring out the suitability of the various mixes of compacted soil-fly ash and soil-fly ash-cement for various applications.

The scope of the present work can be summarized as follows:

1. To investigate the compaction characteristics of various mixes consisting of the soils and their mixes with varying contents of fly ash and cement.
2. To study the strength characteristics of the soil mixes by means of direct shear tests, California Bearing Ratio tests, and unconfined compression tests, particularly to identify:
    a) the individual and combined effects of the addition of different proportions of fly ash with and without cement, and
    b) the short-term and long-term changes with curing.
3. To investigate the shear strength behaviour of the mixes through triaxial compression tests to understand:
    a) the effects of fly ash and cement contents, and
    b) the influence of curing.
4. To examine the geotechnical performance of the fine-grained residual lateritic soil and the granular sand, due to the application of fly ash and cement, by focussing on the strength characteristics of the modified soils.

1.6 Organisation of the Thesis

The topics dealt under this dissertation work are divided in nine chapters. Chapter 1 presents introduction of the work, abundantly available local soils, need for bulk utilization of fly ash, and the objectives of the present study. A literature review relevant to the present study is presented in Chapter 2, summarizing the works carried out by various researchers based on the materials used and experimental methods adopted. Chapter 3 describes the physical and chemical properties of the soils and fly ash, types of laboratory tests, and specimen preparation procedure. Chapter 4 presents and discusses the details of results obtained from compaction tests. Chapters 5, 6, 7 and 8 deal with the testing programme, experimental details and results of direct shear tests, California Bearing Ratio tests, unconfined compression tests and triaxial compression tests, respectively. Chapter 9 summarizes the results obtained and conclusions drawn from the present study.

TH-1088_AKALITA

CHAPTER - 2
OVERVIEW OF THE LITERATURE

2.1 Introduction

A brief review of the literature is presented in this chapter. The available literature is itemised in seven groups, namely, studies on lateritic soils, studies on fly ash, studies on soil-fly ash mixes, studies on fly ash-cement mixes, studies on soil-cement mixes, studies on soil-fly ash-cement mixes and studies on soil-cement-fibre mixes.

2.2 Studies on Residual Lateritic Soils

Residual lateritic soils have a wide range of index and engineering properties depending on their parent rock forming minerals, intensity of weathering, amount of rainfall and temperature. These factors are in turn govered by the geographical location and prevailing weather conditions. Engineering properties not only vary with spatial locations but also with depth.

Most residual lateritic soils are affected by drying. Some moisture exists as water of crystallization within the structure of the minerals present in the solid particles of the soils (Blight 1997). This water may be removed by drying at a temperature of 110°C ±5°C. Index properties may change drastically even by partial drying. It could be due to alteration of the clay minerals on partial dehydration or due to aggregation of fine particles to form larger particles which remain bonded together even on re-wetting.

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

TH-1088_AKALITA

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.

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 range from 1.281 – 1.682 g/cm^3. Table 2.1 presents examples of Proctor dry unit weight and optimum moisture content (OMC) for selected lateritic soils.

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 unit weight (MDD) was also considerably changed .

Table 2.1: Examples of Proctor dry unit weight and optimum moisture contents for lateritic soils (after Townsend, 1985)

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

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.

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 2.2 presents the examples of strength parameters for lateritic soils.

Table 2.2: Typical strength parameters for lateritic soils (after Townsend, 1985)

Soil type and location	ϕ' (degrees)	c' (kPa)
Granitic lateritic soil, Venezuela	21.5	20
Lateritic clays, Africa	22.5	0-100
Lateritic gravels,Africa	37.5	0- 40
Lateritic soil, Panama	38.0	0
Granitic laterite, Brazil	31.0	0
Lateritic soil, India	19.0-27.8	25-50

In India, a few attempts have been made to characterize laterites and lateritic soils (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 geotechnical of some Indian lateritic soils is presented in Table 2.3.

Table 2.3: Geotechnical properties of Indian laterite deposits (after Anirudhan, 1998)

Region	Grain size	Specific gravity	Natural water content (%)	Index properties				Compaction MDD (g/ cm^3)	Shear strength properties (in kg/cm^2)
				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
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
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
Calicut, Rajamundry	Clay 18% Clay 39%		25 to 30 15 to 20	72 62	38.8 24.0	33.5 38.0			
Quilon, Kerala		2.68	22	50 to 54	33 to 34	17 to 20			ϕ' = 29° to 31° c' = 1.0 to 1.25
Changanachery, Kerala		2.75 to 2.8	22	50 to 54	33 to 34	17 to 20	26		ϕ' = 30° c' = 1.0
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	
Ranchi		2.76		43.0	26.5	17.0	21.2	OMC = 17.5% MDD = 1.85	ϕ' = 23° (at OMC) c' = 0.53
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
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			
Calicut	Gravel 80 to 90%	2.68 to 2.78		48	29	18		OMC = 28% MDD = 1.95	

2.3 Studies on Fly Ashes

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 and generally fall into the grain size category of silt. 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. It is a usual and standard practice to report the chemical composition of fly ash in terms of oxides. Typical ranges of the properties and composition reviewed from different sources are tabulated in Table 2.4. It can be seen 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. The presence and availability of silicon dioxide and aluminium oxide in fly ash is of great importance for pozzolanic reactivity, which is responsible for producing cementitious compounds on hydration reaction with lime.

Leonard and Bailey (1982) studied the geotechnical properties of pulverised coal ash to determine its suitability as a structural fill. Laboratory tests conducted include gradation, chemical analysis, particle shape and composition, compaction, shear strength (unconfined compression and triaxial shear tests), and consolidation tests. Field tests conducted include field compaction tests, plate load tests, standard penetration tests and Dutch cone tests. It was observed that the strength values for fine ash were higher than those determined for the coarser ash specimens. They noted that untreated pulverized coal ash, with no cementing qualities could be used successfully as a material for structural fill.

Table 2.4: 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 (Percentage retained on #325 sieve)	9.8-44.8 (Percentage retained on 45μm sieve)	16.8-92.5 (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 Lohtia (1997)
India – Pandian et al. (1998a), Sridharan (2002)

Sridharan et al. (2001), from a comparison of various compaction curves of a number of Indian fly ashes reported that these compaction curves resemble cohesionless sands or sandy soils. They also observed that water content does not have an appreciable effect on dry unit weight, and that the compaction curves are 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 reported by Sridharan et al. (1998), Srinivas et al. (1999) and Sridharan (2002) are summarised in Table 2.5.

TH-1088_AKALITA

Table 2.5 Typical ranges of shear strength parameters of Indian fly ashes obtained from different methods and test conditions

Method Test condition	Angle of internal friction (Degrees)	Internal cohesion (kPa)
Direct shear test (Drained condition)		
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)	---
Triaxial test (Undrained conditon)		
Compacted (at 95% proctor maximum unit weight) and saturated	20 to 41 (ϕ_{cu})	0
	26 to 39 (ϕ')	16 to 96 (c')

Cokca (1997) studied frost susceptibility properties of a Class C fly ash when used in embankments. Changes in strength with curing time and freeze-thaw cycles were measured using unconfined compression tests, and increases in strength with curing time and the number of freeze-thaw cycles were observed. The fly ash showed pozzolanic or age hardening behaviour, and the low degree of saturation of the compacted fly ash specimens had a positive effect on its freezing and thawing resistance.

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 give a lower solid volume occupation, higher range of water volume content 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 give 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.

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.

Kaniraj and Gayathri (2004) conducted detailed testing for geotechnical utilization of a fly ash, which included classification, compaction, consolidation, permeability and triaxial shear tests. The influence of the head loss across the specimen, the effective stress and the void ratio on the permeability was studied. They reported that the coefficient of permeability and consolidation of the compacted fly ash were comparable to those of non-plastic silts.

Pandian (2004) emphasized the necessary characterization of fly ashes with reference to geotechnical applications, and presented a review of physical and chemical properties as well as compaction, strength, California bearing ratio, consolidation, permeability, dispersive and leaching behaviour of fly ashes. He reported that fly ashes have low specific gravity, freely draining nature, ease of compaction, insensitiveness to changes in moisture content, good frictional properties, and therefore can be used in the construction of embankments, roads, reclamation of low-lying areas, and fill behind retaining structures.

Das and Yudhbir (2005) conducted grain size, specific gravity, compaction and unconfined compression strength tests on both low and high calcium fly ashes to evaluate their suitability as embankment materials and reclamation fills. The reported that the specific gravity of high calcium fly ash is greater than that of low calcium fly ash, and is comparable to that of soil. It was also noticed that the residual carbon content seems to be the controlling factor for compaction characteristics of fly ash.

Kim et al. (2005) evaluated the suitability of Class F fly ash-bottom ash mixtures as construction materials for highway embankments. Fly ash/bottom ash mixtures with fly ash contents of 50%, 75% and 100% were used. Laboratory tests conducted include standard compaction, hydraulic conductivity, one-dimensional compression and drained triaxial tests.

They found that the hydraulic conductivity of compacted ash mixtures was found to decrease slightly with increasing fly ash content. Ash mixtures at 95% relative compaction typically exhibited behaviour similar to that of sandy soils in dense states (i.e., dilatant behaviour) whereas those at 90% relative compaction resembled sand in loose states. Increasing bottom ash content also tended to decrease dilatancy, primarily due to crushing of bottom ash particles during shearing.

Bera et al. (2007) studied the effects of different compaction controlling parameters, such as moulding moisture content, compaction energy, mould area, tank size and specific gravity on the compaction characteristics of pond ash. Three different types of pond ash were used. With increase in mould area, keeping moisture content and compaction energy constant, dry unit weight was found to increase. The dry unit weight was more at the same compaction energy for tanks having higher area than that of the Proctor mould. Based on the experimental results, empiricals models were developed to estimate dry unit weight and also MDD and OMC, which may be helpful in field compaction control.

2.4 Studies on Soil-Fly Ash Mixes

Kaniraj and Havanagi (1999a) conducted an experimental program to investigate the geotechnical characteristics of soil-fly ash mixtures. Two fly ashes and two soils were used. It was concluded that the degree of compaction of the fly ash-soil mixtures was not sensitive to water content. The unconfined compressive strength of the fly ash-soil mixtures increased with increase in the fly ash content. In direct shear tests, the angle of shearing resistance increased with increase in fly ash content, but there was no consistent trend in the variation of the cohesion intercept. In unconsolidated undrained triaxial shear tests, addition of soil to fly ash decreased the maximum deviator stress. This tended to reduce ϕ_{uu} but the change in c_{uu} was not consistent.

Pandian et al. (2001) conducted CBR tests to evaluate the effectiveness of different percentages of fly ash in the behaviour of black cotton soil-fly ash mixtures. It was concluded that only certain proportions of black cotton soil and fly ash improved the CBR values. There were two optimum fly ash content values (20% fly ash and 70% fly ash) at which black cotton soil-fly ash mixture exhibit higher CBR value.

Pandian and Krishna (2003) conducted CBR tests to study the engineering properties of black cotton soil-fly ash mixes for use as sub-base material in pavement construction. The pozzolanic effect of fly ash on the CBR was investigated. It was observed that with increase in fly ash content, the CBR increases for all the curing periods (immediate, 7, 14 and 28 days). The CBR increases rapidly during the first 7 days of curing. Beyond this period, the rate of increase of CBR is very low. This means that the cementation due to pozzolanic reaction was practically over within the initial 7 days of curing.

Leelavathamma and Pandian (2005) conducted an experimental work to study the CBR behaviour of black cotton soil and a class C fly ash placed in layers with the fly ash on top, and compared the same with the CBR of mixes of the same soil and fly ash. The CBR values of the mixes showed better performance than layers. But since proper mixing may not be possible in the field, layers can be used in the field without any additives. Black cotton soil-fly ash layers (fly ash on top) meet the CBR requirement for subgrades, and can also be used directly as a subbase material.

Sezer et al. (2006) presented an investigation into the stabilization of a soft clay subgrade with a very high lime fly ash. Unconfined compressive strength and direct shear tests were carried out. Fly ash addition increased the unconfined compressive strength of the soil. Beyond 28 days, there was no appreciable increase in unconfined compressive strength. Internal friction angle increased considerably with increasing fly ash inclusion level. There was a considerable increase in cohesion intercept at later ages in sample containing high percents of fly ash.

2.5 Studies on Fly Ash-Cement Mixes

Kaniraj and Gayathri (2003) conducted a laboratory experimental study to investigate the effect of cement content, curing period, controlled and uncontrolled ambient conditions of curing, unit weight, and water content on the development of the strength of cement stabilized Class F fly ashes with reference to their use as pavement base courses. Two types of Class F fly ashes were mixed with Portland cement. They concluded that by appropriate selection of dry unit weight and degree of saturation of the mixes, it might be possible to achieve the required strength by providing adequate water for hydration of cement. For any cement content, the rate of change in strength decreased as the curing period increased. Immersion of the specimens in water before the unconfined compression test increased their water contents and decreased their strength. The increase in strength upon curing was more during summer than the monsoon season because of the higher temperature during these seasons.

Lav et al. (2006) conducted a laboratory study of undrained triaxial tests on a Class F fly ash stabilized with cement for use as base material in road pavements. Only aggregate free stabilized mistures were used to utilize high volumes of fly ash. Cement content was varied between 2% and 10% by weight. They concluded that the cement content and layer thickness should not be less than 8% and 300 mm, respectively. The mixes having cement content less than 8% may be used as subbase materials instead of being used in pavement base.

2.6 Studies on Soil-Cement Mixes

Baghdadi and Shihata (1999) presented a study with soil-cement to investigate alternative pavement systems. The possibility of simplifying the durability test and use of soil of the site to construct soil-cement courses were also investigated. They concluded that the weight loss during durability test decreases with increasing cement content and compressive strength increases with increasing cement content. A similar trend was also obtained when the residual strength of unbrushed or brushed durability samples was correlated with the weight loss. Comparison of unbrushed and brushed compressive strengths of durability samples showed little affect on strength due to brushing.

Shihata and Baghdadi (2001) conducted freeze-thaw durability test to explore the possibility of using unconfined compressive strength (UCS) of soil-cement as a credible indicator of durability resistance. The UCS was determined for specimens cured for 7 days without subjecting it to the alternating cycles. The residual UCS was determined for specimens subjected to cycles of the standard freeze-thaw durability test but not brushed. They established a strong correlation between percent mass loss from the standard freeze-thaw durability test and the UCS tests.

Schnaid et al. (2001) studied the stress-strain-strength behaviour of an artificially cemented sandy soil produced through the addition of Portland cement. Soil used was classified as non-plastic silty sand. Laboratory tests conducted include unconfined compression tests and drained triaxial tests. They found that the unconfined compressive strength seemed to be a direct measure of the degree of cementation in triaxial compression. The shear strength of the cemented soil measured in conventional triaxial tests could be determined as a function of the unconfined compressive strength and the uncemented friction angle.

Tremblay et al. (2001) summarized the results of a research project conducted to define the general mechanical behaviour of high water content clayey soils treated with lime or cement, in terms of compressibility. Four inorganic soils and three organic soils were studied. One-dimensional compression tests were conducted. The general compressibility behaviour was obtained through relationships between initial void ratio, additive content, and vertical yield stress for a given inorganic or organic soil.

Consoli et al. (2007) investigated the influence of the amount of cement, the porosity and the moisture content on the strength of a sandy soil artificially cemented. Unconfined compression tests, triaxial compression tests and tests for measurements of matric suction were carried out. The use of a water/cement ratio and a void/cement ratio to assess its unconfined compression strength were also evaluated. It was concluded that the unconfined compression strength increased approximately linearly with an increase in the cement content, and exponentially with the reduction in porosity of the compacted mixture. An increase in strength was observed with the moisture content increase until a maximum value, after which the strength reduced again. It was found that there is no relationship between the unconfined compression strength and the water/cement ratio for the material studied.

Stavridakis (2006) presented review studies on the stabilization of problematic soils using cement and lime. It was concluded that a suitable ground improvement technique is needed for surface or deep excavations in swelling soils to increase strength, reduce deformability, provide volume stability, reduce permeability, and increase durability. Clayey soils with a liquid limit less than 40% were stabilized successfully by using economical amounts of cement, while clayey soils with large liquid limits (60%) usually contain expansive clays and react with large amounts of cement. He concluded that the suitability of a soil for stabilization, the most appropriate stabilizing agent, and the quantity of this agent required are dependent on the mineralogical composition (qualitative and quantitative) and texture (particle size distribution) of the soil.

2.7 Studies on Soil-Fly Ash-Cement Mixes

Kaniraj and Havanagi (1999b) conducted a laboratory study to determine the unconfined compressive strength of locally available soils mixed with fly ash and ordinary Portland cement in different proportions. Two Class F fly ashes were used along with two types of soils. It was concluded that the gain in unconfined compressive strength and secant modulus of the mixtures with time was dependent on the fly ash and cement contents. The gain in strength and modulus decreased as fly ash content increased, but increased as cement content increased. The cement content had a significantly higher influence than the fly ash content. The water content of a mixture determined after the strength test depended on the curing time and cement content. The influence of cement content was more pronounced than that of the curing time

Kaniraj and Havanagi (2001) presented an experimental work to study the individual and combined effects of randomly oriented fiber inclusions and cement stabilization on the geotechnical characteristics of soil-fly ash mixtures. A silty soil and a sandy soil were mixed with a Class F fly ash. Laboratory tests conducted include light compaction, direct shear, unconfined compression, unconsolidated undrained and drained triaxial shear, and consolidation tests. It was concluded that in direct shear tests, the randomly oriented fiber inclusions increased the failure displacement and the vertical displacement of the fly ash-soil specimens compacted at the MDD-OMC state.The fly ash-soil specimens compacted at the MDD-OMC state exhibit brittle behaviour in unconfined compression tests.The brittle behaviour was more marked in cement-stabilized specimens than in unstabilized specimens. The fiber inclusions changed the behaviour in both instances to ductile behaviour. The UCS of soil-fly ash mixture increased due to addition of cement and fibers.

Cokca (2001) presented the effect of lime, cement and fly ash addition in reducing the swelling potential of an expansive soil. High-calcium and low-calcium Class C types were used. Lime and cement were also added to the expansive soil to establish base line values. Test specimens were subjected to chemical composition, grain size distribution, consistency limits and free swell tests. It was concluded that plasticity index, activity, and swelling potential of the samples decreased with increasing percent of stabilizer and curing time. Addition of 20% fly ash decreased the swelling potential to nearby the swelling potential obtained with 8% lime addition. There was only a slight decrease in swelling potential from 20 to 25% fly ash addition. Therefore optimum fly ash content was near 20%. Swelling potential values of each sample were highest for those without curing and lowest for 28 day curing. They concluded that the expansive soil could be successfully stabilized by fly ashes.

Pandian and Krishna (2002) studied the effect of the addition of ordinary Portland cement on the CBR of black cotton soil-fly ash mixes. It was concluded that the variation of CBR with the addition of fly ash showed two optimum levels, one at 20% fly ash and the other at 70% fly ash addition. The variation of unconsolidated undrained shear strength parameters has resulted in the friction angle (ϕ_{uu}) following the trend of two optimum levels similar to CBR variation. It turned out that a small percentage of cement added brought about significant changes in the strength in terms of the CBR. Scanning electron microscope studies on the cement-stabilized samples revealed that at the first optimum level, the mix was observed to be massive and the particles were aggregated compared to other mix proportions. The second optimum level was aggregated due to the bridging effect of needle-like structures (ettringites).

Lo and Wardani (2002) conducted an experimental work to study the strength and dilatancy of a silt stabilized with the addition of a cement and fly ash mixture in a slurry form. Unconfined compressive strength and triaxial tests were carried out. The stress–strain–strength test results implied that the cementing agent contributed to both stiffness and strength via two mechanisms, namely bonding between grains and additional dilation. The bonding between grains was measured directly by special zero effective confinement tests.

The UC strength for a slow loading rate was consistently higher than that for a fast loading rate. The cementing agent always led to an increase in peak strength via an increase in dilatancy at failure.

Leelavathamma et al. (2005) conducted CBR tests to investigate the CBR behaviour of black cotton soil and a class F fly ash placed in layers (fly ash on top), and compared the same with the CBR of mixes of the same soil and fly ash. In the field, thorough mixing of fly ash with soil may not be possible, and placement in layers may be more convenient. The results showed that the CBR values of soil-fly ash mixes were better than layers.To improve the strength of the fly ash layer, cement was used as an additive. It was concluded that the addition of small percentage of cement with fly ash gave significant change in CBR.

Kolias et al. (2005) conducted a laboratory study to evaluate the effectiveness of using high calcium fly ash and cement in stabilising fine-grained clayey soils. Strength tests in uniaxial compression, in indirect (splitting) tension and flexure were carried out on samples to which various percentages of fly ash and cement had been added. Modulus of elasticity was determined at 90 days with different types of load application and 90-day soaked CBR values are also reported. The mechanical and strength properties of the mixes were found to be considerably enhanced.

2.8 Studies on Soil-Cement-Fibre Mixes

Consoli et al. (1998) conducted an experimental work to evaluate the overall behaviour of fiber-reinforced cemented and uncemented soils under static compression loading. Drained triaxial compression tests were carried out. Materials used were a non-plastic silty sand, chopped fiberglass and Portland pozzolanic cement. They concluded that the addition of cement to soil increased stiffness and peak strength. Fiber reinforcement increased both the peak and residual triaxial strengths, decreased stiffness, and changed the cemented soil's brittle behaviour to a more ductile one. The peak friction angle increased from a minimum of 35° for uncemented non-reinforced soil to 41° or 46°, respectively, when cement or fiber was added to the soil.The friction angle value was also dependent on the confining stress.The cohesion intercept was practically unaffected by fiber inclusion, but affected only by cement addition.

Consoli et al. (2002) conducted a laboratory investigation to evaluate the benefit of utilizing randomly distributed polyethylene terephthalate fiber obtained from recycling waste plastic bottles, alone or combined with rapid hardening Portland cement, to improve the engineering behaviour of uniform fine sand. Unconfined compression tests, splitting tensile tests and saturated drained triaxial compression tests with local strain measurement were carried out. The results showed that the fiber reinforcement improved the peak and ultimate strength of both cemented and uncemented soil and somewhat reduced the brittleness of the cemented sand. In addition, the initial stiffness was not significantly changed by the inclusion of fibers.

2.9 Remarks

The properties of fly ash such as compaction, strength, permeability, effects of curing time, effects of delay in compaction, influence of cement, lime and other additives, pozzolanic activity, leaching characteristics etc. have been studied in detail by a large number of researchers. Substantial literature is also available on stabilization of various types of soils using fly ash and many other additives. Large deposits of residual lateritic soils and riverbank sand are available in the northeastern part of India. There is a need to understand the characteristics of mixtures made with these locally available soils and fly ash along with cement. This is the motivation behind the present study.

CHAPTER - 3
MATERIALS AND METHODS

3.1 Introduction

This chapter summarizes the materials and procedures used in performing the investigation. The different sections are for the materials used and their properties, for the laboratory test procedures, and for the preparation of test specimens of various mixes with different proportions.

3.2 Materials Used for Testing

Two types of local soils were used in the present study: a fine-grained residual lateritic soil and granular riverbank sand. They are found in abundance within the city of Guwahati in Assam state. The lateritic soil was reddish in colour and was procured from nearby hills as shown in Figure 3.1. This red soil was excavated with a shovel from a depth of 0.4 m below the surface to avoid humus and roots. The sand used in this investigation was collected from the nearby bank location of Brahmaputra River, as shown in Figure 3.2. The sand was also excavated with a shovel from a depth of 0.4 m below the surface. In the laboratory, the soils were air-dried. Finally, aggregates of the red soil were carefully broken down using a rubber mallet. The soils were characterized using laboratory tests as per Indian Standards. The fly ash was collected from the electrostatic precipitators of Farakka thermal power plant located in the adjoining state of West Bengal. The fly ash was of Class F type. Ordinary Portland cement of 53 Grade was used.

Fig. 3.1 Red soil sampling location from top of nearby hills

Fig. 3.2 Sand sampling location from nearby bank of Brahmaputra River

3.2.1 Physical and chemical properties of red soil

Grain size analysis of the soil was done by wet sieving method, using un-dried soil in natural state. The grain size distribution and consistency limits of the red soil are listed in Table 3.1. The red soil can be classified as MI (silt of intermediate plasticity) according to the Indian Soil Classification System. Table 3.2 presents chemical properties of the red soil. The ratio of SiO_2 to Al_2O_3 for the soil is 2.37, which indicates an initial stage of laterization. The soil has a substantial proportion of aluminium and iron oxides (30.20%).

Table 3.1: Physical properties of red soil

Property	Value
Specific gravity	2.51
Grain size analysis	
Sand size fraction (%)	4.2
Silt size fraction (%)	76.0
Clay size fraction (%)	19.8
Consistency limits	
Liquid limit (%)	43
Plastic limit (%)	29
Plasticity index (%)	14

Table 3.2: Chemical composition of red soil

Major Oxides	Value (%)
Silica (SiO_2)	47.52
Alumina (Al_2O_3)	20.01
Iron oxide (Fe_2O_3)	10.19
Lime (CaO)	0.23
Magnesia (MgO)	0.04
Soda (Na_2O)	0.11
Potash (K_2O)	1.03

3.2.2 Physical and chemical properties of sand

The specific gravity of the sand is 2.71, and its grain size distribution curve is shown in Fig. 3.3. The sand can be classified as SP (poorly graded fine sand). Table 3.3 presents chemical composition of the sand.

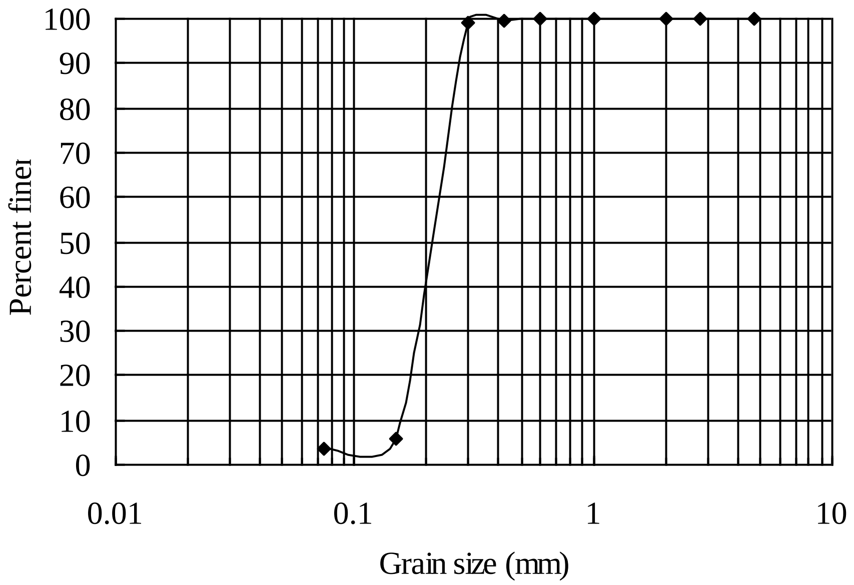


Fig. 3.3 Grain size distribution curve of sand

Table 3.3: Chemical composition of sand

Major Oxides	Value (%)
Silica (SiO_2)	70.08
Alumina (Al_2O_3)	14.46
Iron oxide (Fe_2O_3)	4.40
Lime (CaO)	1.22
Magnesia (MgO)	0.58
Soda (Na_2O)	3.06
Potash (K_2O)	3.37
Manganese oxide(MnO)	0.15
Titanium oxide (TiO_2)	0.42

3.2.3 Physical properties of fly ash

The specific gravity of the fly ash is 2.08, which is substantially low in comparison to that of the two soils. The low specific gravity is because of the hollow nature of the fly ash particles, or the cenospheres. The specific gravity of Indian coal ashes varies from 1.46 to 2.66 (Pandian et al., 1998). Figure 3.4 presents the grain size distribution of the fly ash. The fly ash consists predominantly of fine sand-size fraction (95.13%) with some silt and clay-size fraction (4.85%).

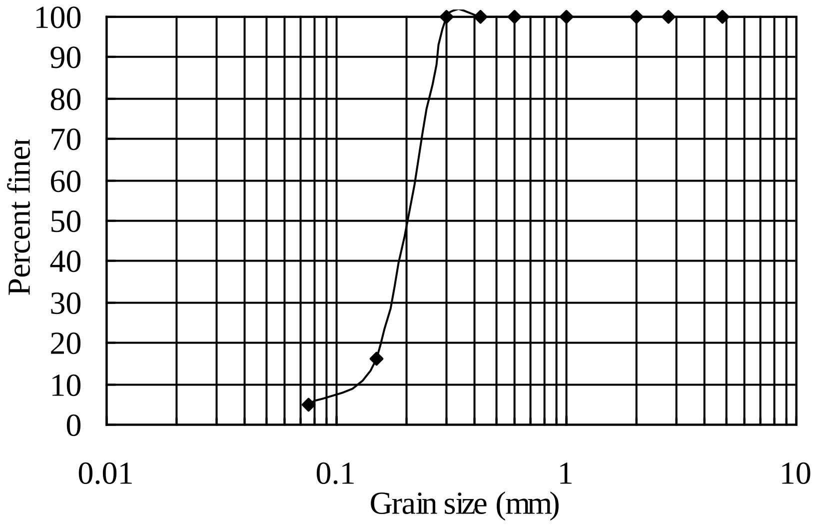


Fig. 3.4 Grain size distribution curve of fly ash

3.2.4 Physical and chemical properties of cement

The specific gravity of the cement is 3.08, which is more in comparison to that of the two soils. Table 3.4 presents the chemical composition of the cement used.

Table 3.4: Chemical composition of the cement

Major Oxides	Value (%)
Silica (SiO_2)	23.09
Alumina (Al_2O_3)	3.80
Iron oxide (Fe_2O_3)	2.31
Lime (CaO)	59.90
Magnesia (MgO)	1.32
Soda (Na_2O)	0.21
Potash (K_2O)	0.37
Manganese oxide (MnO)	0.05
Titanium oxide (TiO_2)	0.23
Phosphorus oxide (P_2O_5)	0.05

3.3 Laboratory Test Procedures

3.3.1 Compaction tests (IS: 2720-Part 7, 1992)

Light Proctor compaction tests were performed to determine the maximum dry unit weight (MDD) and optimum moisture content (OMC) for the soil, soil-fly ash, soil-cement and soil-fly ash- cement mixes. The MDD and OMC values were then used to prepare specimens for other tests such as unconfined compression tests, California bearing ratio tests and triaxial tests to determine the strength properties.

3.3.2 Direct shear tests (IS: 2720-Part 13, 1992)

In direct shear tests, a small box consisting of two halves of size 60x60x20 mm each was used to evaluate the shear strength for only sand and sand-fly ash mixes. The fine-grained soil was not used. The tests were carried out using five normal stress values of 50, 75, 100, 125 and 150 kPa. The shearing rate was kept at 0.2 mm/min. The tests were continued until the shear stress became essentially constant or until a maximum shear deformation of 8 mm was reached.

3.3.3 California bearing ratio tests (IS: 2720-Part 16, 1992)

CBR tests were conducted on different untreated soil, soil-fly ash and cement treated soil-fly ash specimens for unsoaked and soaked conditions. The specimens were prepared in a mould of 150 mm diameter and 175 mm height. Soaking period of 4 days was used for soaked CBR tests.

3.3.4 Unconfined compression tests (IS: 2720-Part 10, 1995)

Unconfined compression (UC) tests were performed on all specimens using a strain rate of 1.25 mm/min. Each specimen was loaded until peak stress was obtained. The test type is undrained as it is a quick test.

3.3.5 Triaxial compression tests (IS: 2720-Part 12, 1992)

Consolidated drained tests (CD) were performed on the compacted specimens without saturating them. The tests were performed at confining pressures of 100, 200, 300 and 400 kPa. The specimens for the tests were prepared in the similar procedure as for UC tests.

3.4 Preparation of Test Spcimens

For preparing test specimens for direct shear, CBR, UC and CD tests, first the required amounts of soil, fly ash and cement were mixed together in a dry state, and then the required amount of water was added equal to the corresponding optimum moisture content of the mix. All mixing was done by hand and proper care was taken to prepare a homogeneous mixe at each stage of mixing.

Specimens for UC and CD tests were prepared to achieve their respective maximum dry densities using a simple miniature static compaction tool, which was designed and fabricated in the laboratory (Fig. 3.5). The tool comprises of a tubular mould, two end-collars, and two end-plungers each connected to a handle through a threaded rod. The mould is of 38 mm internal diameter, 225 mm long, and approximately 6 mm thick with its both open ends threaded externally. The end-collars are threaded internally to fit onto the mould. Each of the end-collars has a threaded central hole through which the threaded rod of the end-plunger can travel as the handle is rotated. The lengths of travel of the plungers, which in turn determines the length of the compacted specimen, can be controlled with the help of two check nuts mounted on each of the threaded rods. Two dummy plates of 37.9 mm diameter are also used, one at each end, so as to isolate the ends of the specimen from the rotary movement of the plunger plate, which may otherwise cause specimen disturbance.

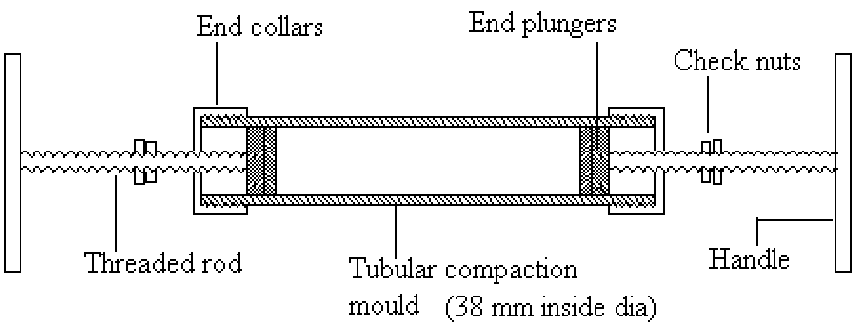


Fig. 3.5 Fabricated miniature static compaction tool

After fixing an end-collar to one end of the mould, the required quantity of soil mix is introduced from the other end. Then, the other end collar is screwed on. Thereafter, both the end-plungers are advanced simultaneously from either mould end so as to compact the mix to the required density. The compacted specimens were of 76 mm in length. The specimen was extruded from the mould immediately and cured according for different durations. Curing periods of 0, 3, 7, 14 and 28 days were used in this investigation. All specimens were cured at room temperature inside desiccators at constant relative humidity of 100%.

3.4.1 Designations of test specimens

Table 3.5 shows the details of different soil-fly ash mixes used in the present investigation and their designations. The designations used are: RS for red soil (residual lateritic soil), BS for sand (Brahmaputra sand), FA for fly ash and RS+FA for the red soil-fly ash mix, BS+FA for the sand-fly ash mix, RS+FA+C for red soil-fly ash-cement mix and BS+FA+C for sand-fly ash-cement mix, respectively. In the mix designation, the amount of soil replaced with fly ash by weight is indicated by the numeral prefixed before the symbol FA. For example, 20FA indicates that 20% by weight is fly ash and the remaining is soil. The amount of cement added as a percentage of the weight of soil-fly ash mix is indicated by the numeral preceding the symbol C. For example, 1C indicates that 1% by weight has been added to the soil-fly ash mix. Thus, RS+20FA+1C designation is for a mix of red soil with 20% fly ash content and 1% cement content. Similarly, the designation for other mixes will differ according to the soil type, fly ash content and cement content.

Table 3.5: Designations of mixes used in the present investigation

Designation	Type of mix
RS	Red soil only
BS	Sand only
FA	Fly ash only
RS+FA	Red soil-fly ash mix
BS+FA	Sand-fly ash mix
RS+FA+C	Red soil-fly ash-cement mix
BS+FA+C	Sand-fly ash-cement mix
RS+20FA	80% red soil-20% fly ash
RS+20FA+1C	80% red soil-20% fly ash-1% cement

TH-1088_AKALITA

CHAPTER - 4

COMPACTION TESTS ON SOIL-FLY ASH-CEMENT MIXES

4.1 Introduction

Compaction is a process of densification of a soil material by the application of mechanical energy. This process involves packing the particles closer together with the reduction in the volume of air voids. The dry unit weight of the compacted material is a measure of the degree of compaction achieved. This is a function of the amount and method of energy application, water content adopted during compaction and material characteristics such as specific gravity, grain size distribution, gradation, particle shape and plasticity. The densification of the soil material controls its other engineering properties such as compressibility, shear strength and permeability.

Compaction generally leads to an increase in shear strength and helps to improve the bearing capacity and stability of a soil. The compaction characteristics of fly ashes are complex compared with soils by virtue of more number of variables affecting the behaviour of coal ashes. The additional factors affecting the compaction behaviour of fly ashes include free lime content and cenospheres content. Due to its hollow shape, fly ash particles mixed with the soil are prone to physical breakdown during compaction and the process is likely to affect the compaction behaviour of the mixes.

4.2 Test Programme

Table 4.1 summarizes the composition of the various mixes used. Three different series of tests were conducted. In the first series, the compaction properties of the soils and fly ash were investigated separately. In the second series, tests were conducted to study the effect of blending of the soils with fly ash. In the third series, tests were carried out after the addition of cement to the soil-fly ash mixes.

Table 4.1: Composition of various mixes for compaction tests

Soil	Fly ash	Soil-fly ash mixes	Soil-cement mixes	Soil-fly ash-cement mixes
RS	FA	RS+20FA	RS+1C	RS+20FA+1C
		RS+35FA	RS+2C	RS+20FA+2C
		RS+50FA	RS+3C	RS+20FA+3C
		RS+65FA	RS+4C	RS+20FA+4C
		RS+80FA		
				RS+35FA+1C
				RS+35FA+2C
				RS+35FA+3C
				RS+35FA+4C
				RS+50FA+1C
				RS+50FA+2C
				RS+50FA+3C
				RS+50FA+4C
				RS+65FA+1C
				RS+65FA+2C
				RS+80FA+1C
				RS+80FA+2C
BS		BS+10FA	BS+1C	BS+20FA+1C
		BS+20FA	BS+2C	BS+20FA+2C
		BS+35FA	BS+3C	BS+20FA+3C
		BS+50FA	BS+4C	BS+20FA+4C
		BS+65FA	BS+5C	BS+20FA+5C
		BS+80FA		
		BS+90FA		BS+35FA+1C
				BS+35FA+2C
				BS+35FA+3C
				BS+35FA+4C
				BS+35FA+5C
				BS+50FA+1C
				BS+50FA+2C
				BS+50FA+3C
				BS+50FA+4C
				BS+50FA+5C
				BS+65FA+1C
				BS+65FA+2C
				BS+80FA+1C
				BS+80FA+2C

4.3 Results and Discussion

4.3.1 Compaction of red soil-fly ash mixes

The compaction curves of the red soil-fly ash mixes are shown in Fig. 4.1. Fig. 4.2 depicts the variation of compaction characteristics of the mixes. As the fly ash content increases, the maximum dry unit weight (MDD) is noted to decrease gradually. This is similar to the findings of Kaniraj and Havanagi (1999a). In the compaction behavior, the dry unit weight of fly ash is found to be less sensitive to variation in moisture content than the red soil. It is because of the higher air voids content of fly ash. This higher air void content tends to limit the build up of pore pressure during compaction, thus allowing the fly ash to be compacted over a large range of water content. Similar observation was reported by Sridharan et al. (2001).

The fly ash has MDD of 13.68%, which is lower than that of the red soil (16.48 kN/m^3). The specific gravity values of the fly ash and red soil are is 2.08 and 2.51, respectively. Thus, the MDD is the highest for the RS+20FA mix and the least for the RS+90FA mix. The decrease in MDD is attributed to the lower specific gravity of the ash particles, increased resistance offered by relatively coarser particles present in the fly ash and poor gradation of fly ash.

The OMC of the red soil-fly ash mixes decreases initially up to 35% fly ash content, and increases gradually up to 65% fly ash content after which it remains more or less the same. The fly ash has OMC of 19.68%, which is marginally lower than that of the red soil (20.34%). The initial decrease in OMC is mainly 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 50% fly ash can be attributed to the higher water holding capacity of the broken and porous fly ash particles.

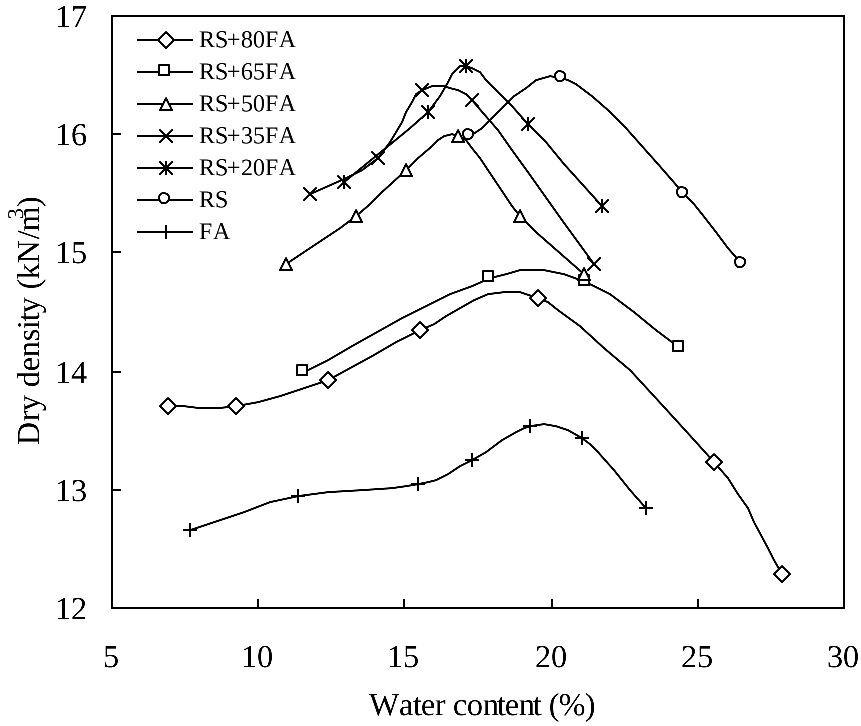


Fig. 4.1 Compaction curves of RS-fly ash mixes

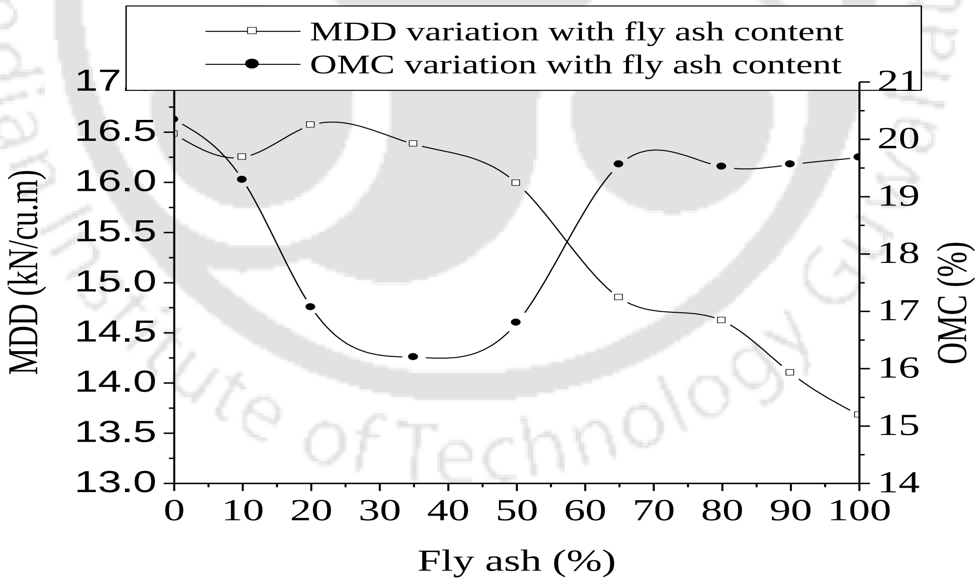


Fig. 4.2 Variation of compaction characteristics of RS-fly ash mixes

4.3.2 Compaction of red soil-cement mixes

The compaction curves of the different red soil-cement mixes are shown in Fig. 4.3. Fig. 4.4 depicts variation of compaction characteristics of the mixes. It can be noted that there is little variation in the magnitude of MDD with cement content. However, the OMC is found to increase with cement content. This is because as cement content increases, more water is required for the hydration process.

4.3.3 Compaction of red soil-fly ash-cement mixes

Figures 4.5, 4.7, 4.9, 4.11 and 4.13 show the compaction curves of red soil-20FA-cement, red soil-35FA-cement, red soil-50FA-cement, red soil-65FA-cement, and red soil-80FA-cement mixes, respectively. Figures 4.6, 4.8, 4.10, 4.12 and 4.14 show the variation of compaction characteristics of each of the above mixes, respectively.

The values of MDD and OMC of all the red soil mixes are listed in Table 4.2, whereas Figs. 4.15(a) and 4.15(b) show the variations of MDD and OMC only of red soil-fly ash-cement mixes, respectively. From Fig. 4.15(a), it is observed that red soil-20FA-cement mixes show the highest MDD values whereas the red soil-80FA-cement mixes show the least values. As the fly ash content increases, the MDD decreases gradually. From Fig. 4.15(b), it is noted that the OMC of the mixes up to 50% fly ash content shows an increasing trend with cement content, whereas the OMC of mixes above 50% fly ash content shows a decreasing trend.

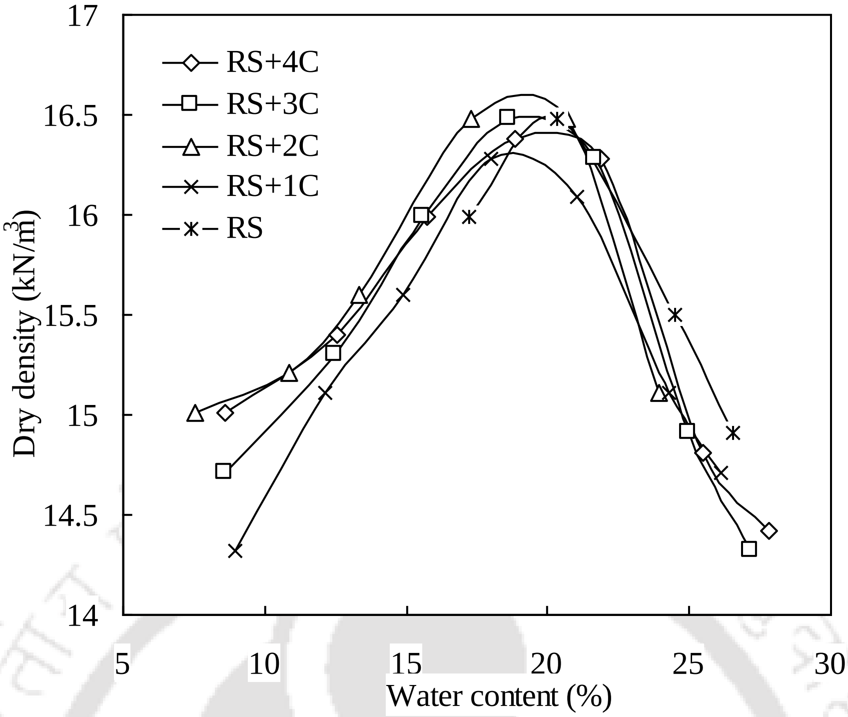


Fig. 4.3 Compaction curves of RS-cement mixes

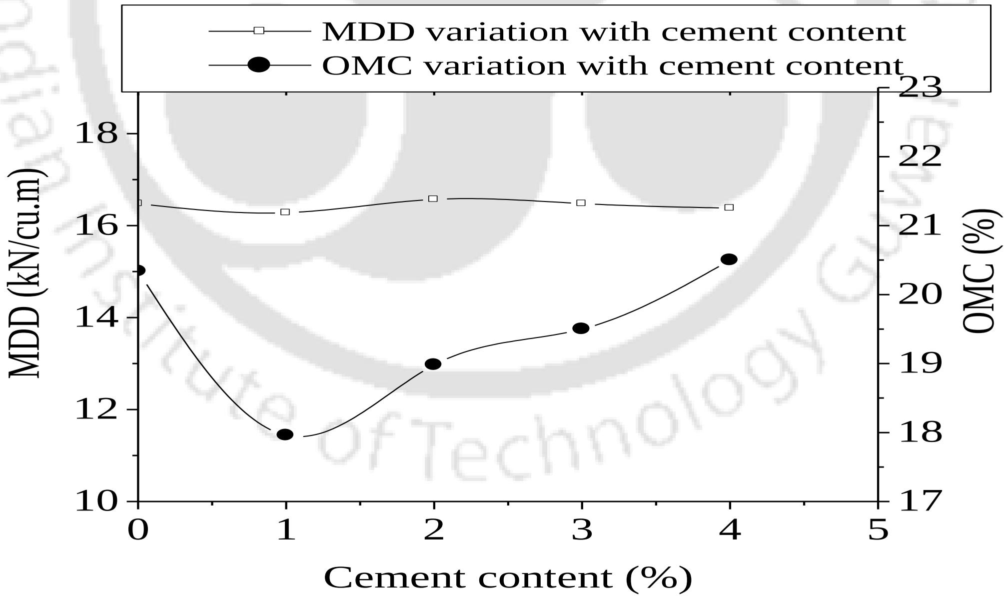


Fig. 4.4 Variation of compaction characteristics of RS-cement mixes

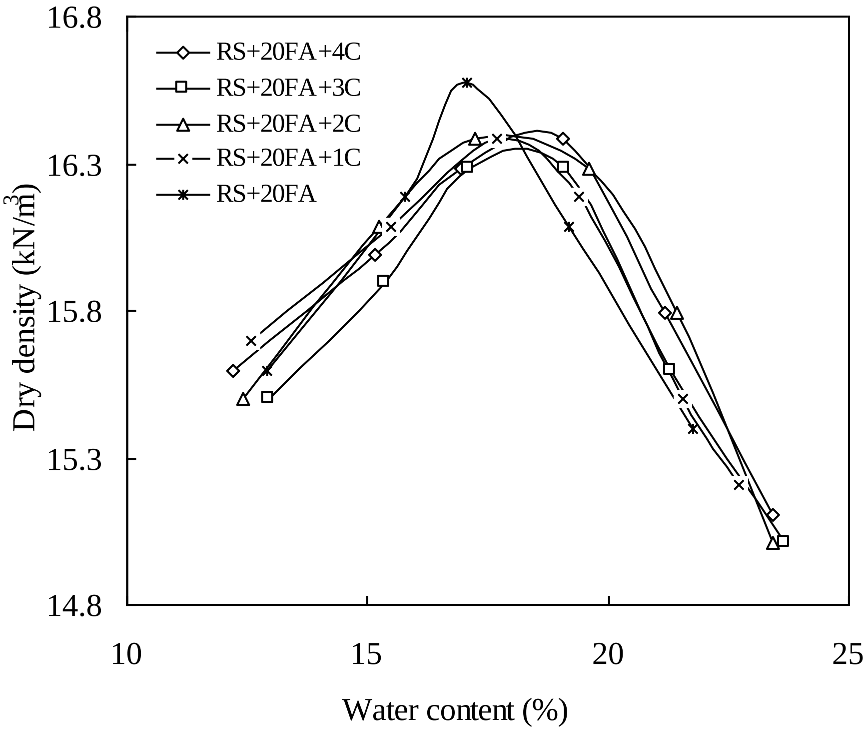


Fig. 4.5 Compaction curves of RS-20FA-cement mixes

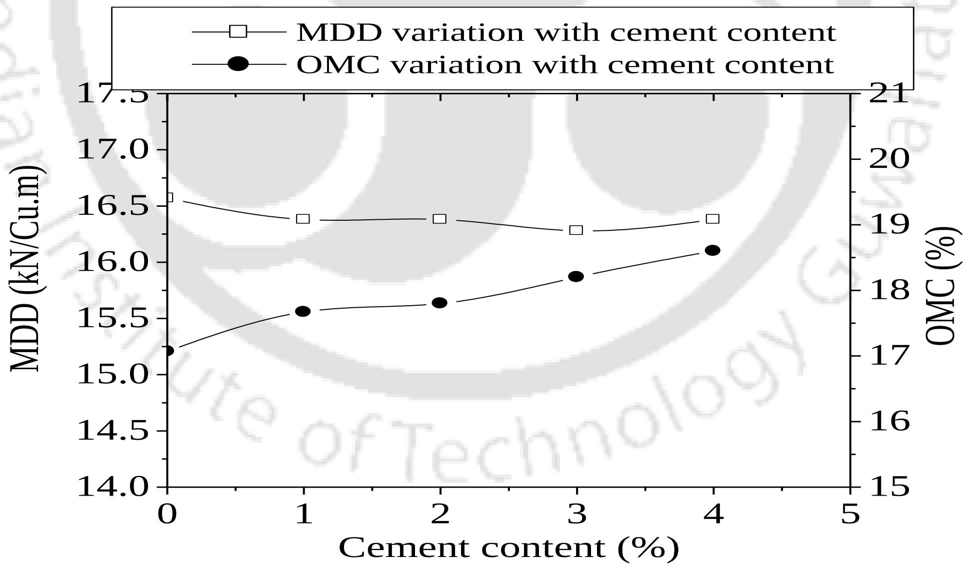


Fig. 4.6 Variation of compaction characteristics of RS-20FA-cement mixes

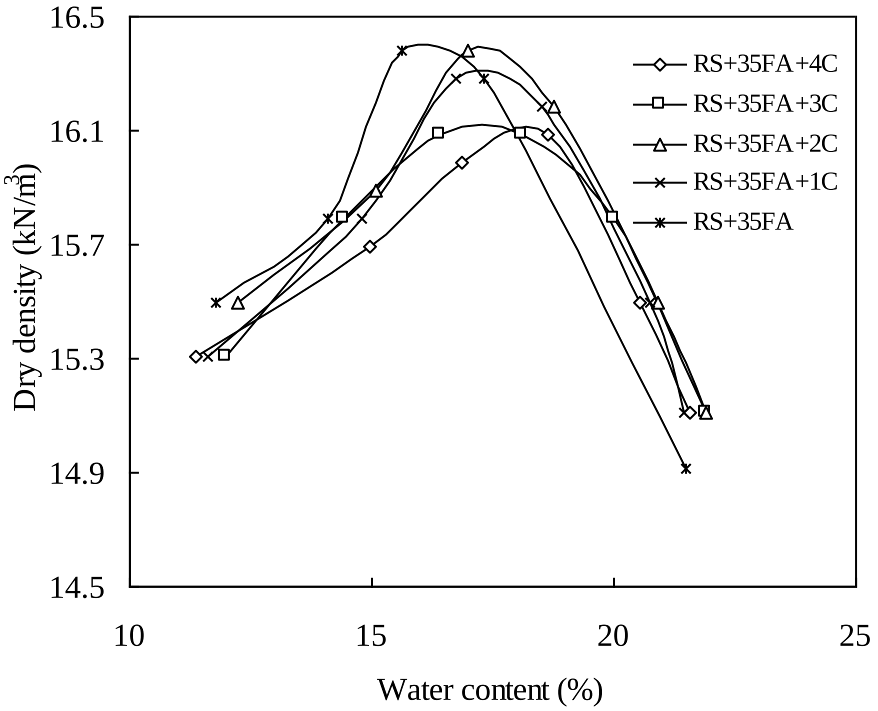


Fig. 4.7 Compaction curves of RS-35FA-cement mixes

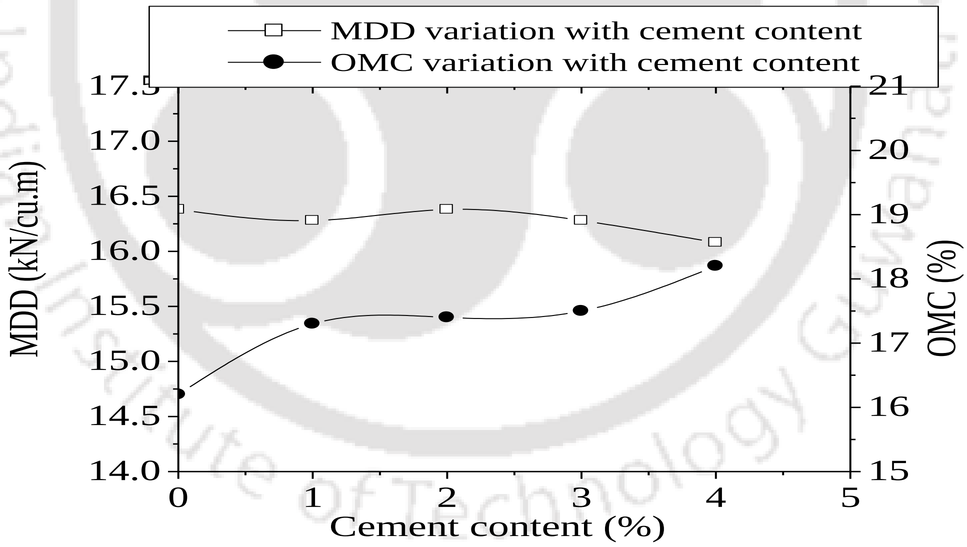


Fig. 4.8 Variation of compaction characteristics of RS-35FA-cement mixes

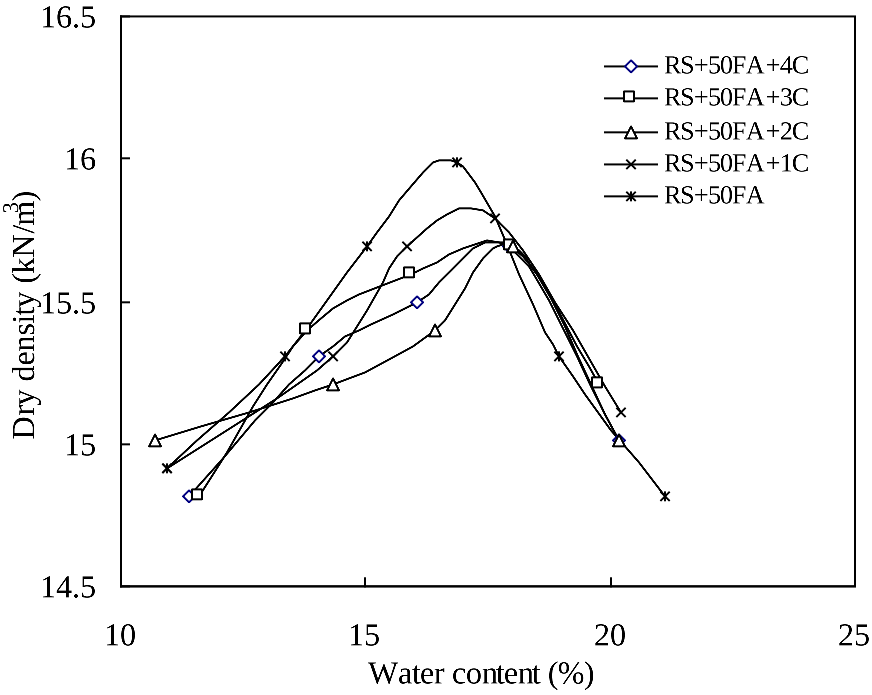


Fig. 4.9 Compaction curves of RS-50FA-cement mixes

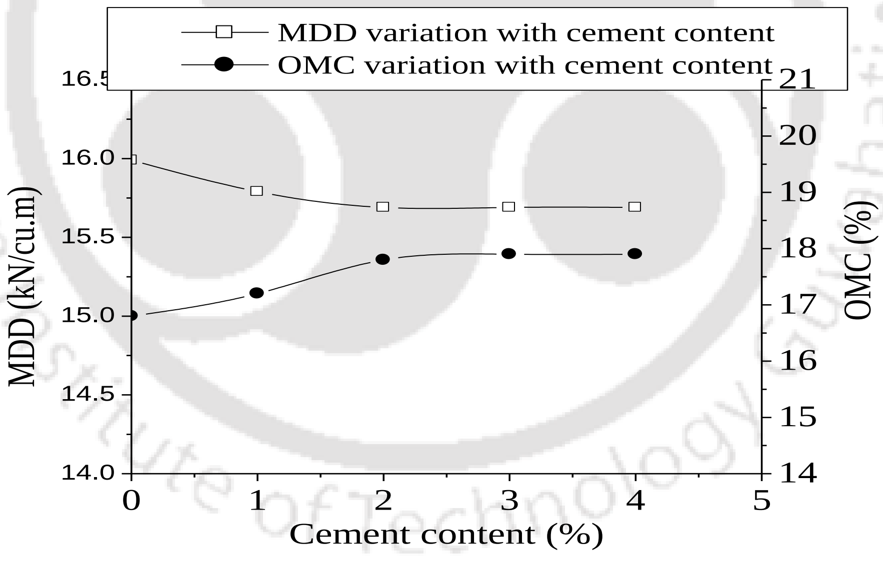


Fig. 4.10 Variation of compaction characteristics of RS-50FA-cement mixes

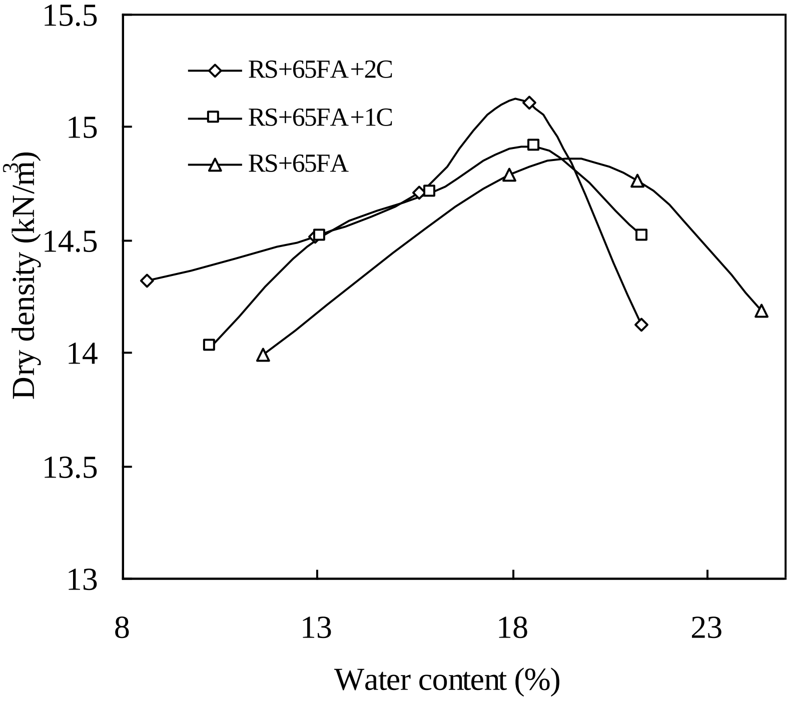


Fig. 4.11 Compaction curves of RS-65FA-cement mixes

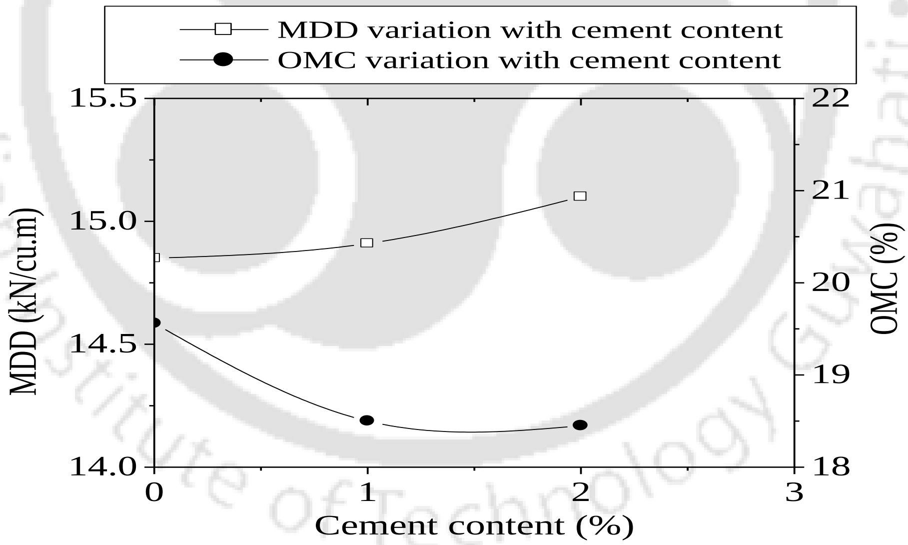


Fig. 4.12 Variation of compaction characteristics of RS-65FA-cement mixes

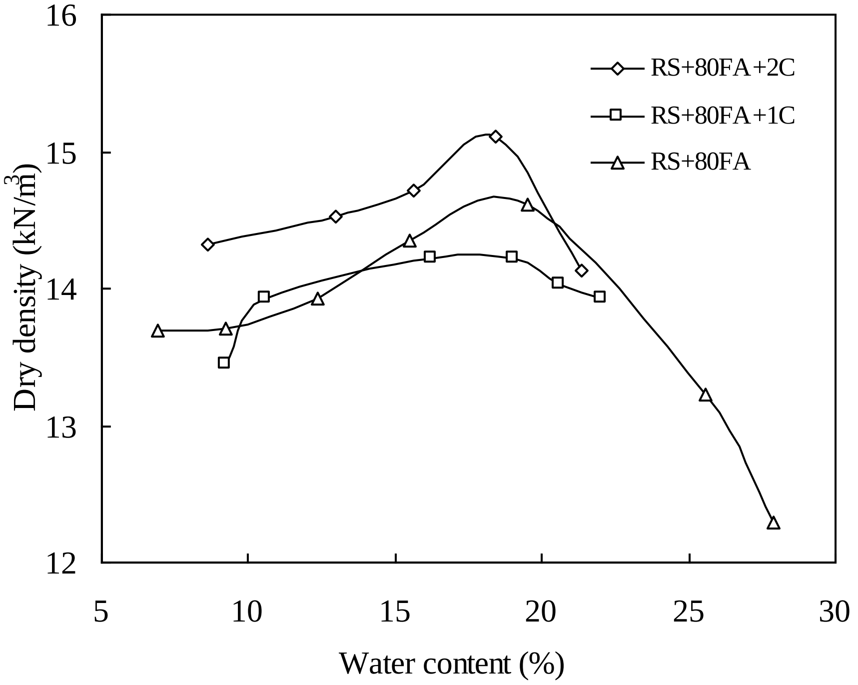


Fig. 4.13 Compaction curves of RS-80FA-cement mixes

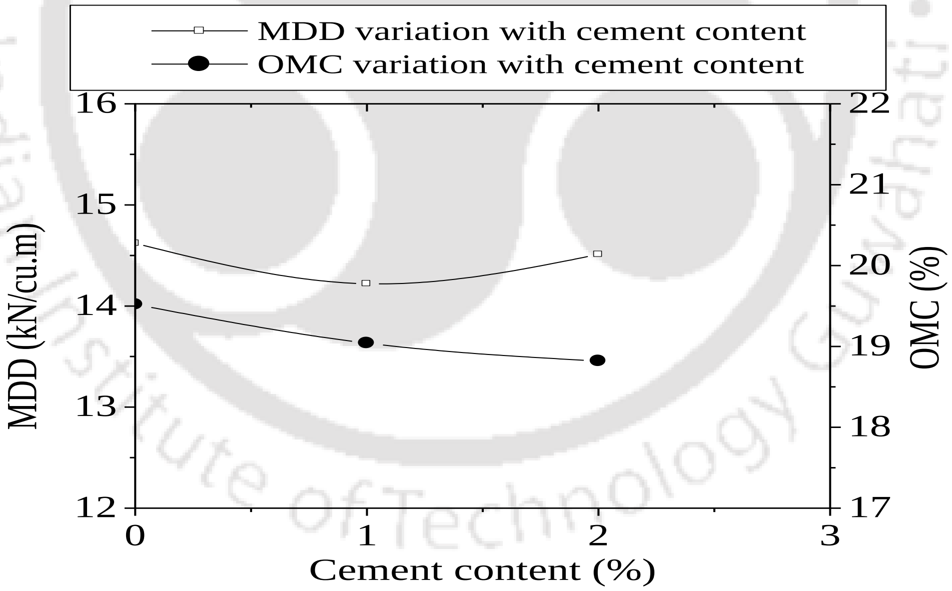


Fig. 4.14 Variation of compaction characteristics of RS-80FA-cement mixes

Indian Institute of Technology Guwahati

Table 4.2: MDD and OMC values of all red soil mixes

S.No.	Mix	MDD (kN/m^3)	OMC (%)
1	RS	16.48	20.34
2	FA	13.68	19.68
3	RS+20FA	16.57	17.07
4	RS+35FA	16.38	16.2
5	RS+50FA	15.99	16.8
6	RS+65FA	14.85	19.56
7	RS+80FA	14.62	19.52
8	RS+1C	16.28	17.96
9	RS+2C	16.57	18.98
10	RS+3C	16.48	19.5
11	RS+4C	16.38	20.5
12	RS+20FA+1C	16.38	17.67
13	RS+20FA+2C	16.38	17.8
14	RS+20FA+3C	16.28	18.2
15	RS+20FA+4C	16.38	18.6
16	RS+35FA+1C	16.28	17.3
17	RS+35FA+2C	16.38	17.4
18	RS+35FA+3C	16.28	17.5
19	RS+35FA+4C	16.08	18.2
20	RS+50FA+1C	15.79	17.2
21	RS+50FA+2C	15.69	17.8
22	RS+50FA+3C	15.69	17.9
23	RS+50FA+4C	15.69	17.9
24	RS+65FA+1C	14.91	18.50
25	RS+65FA+2C	15.10	18.45
26	RS+80FA+1C	14.22	19.04
27	RS+80FA+2C	14.51	18.82

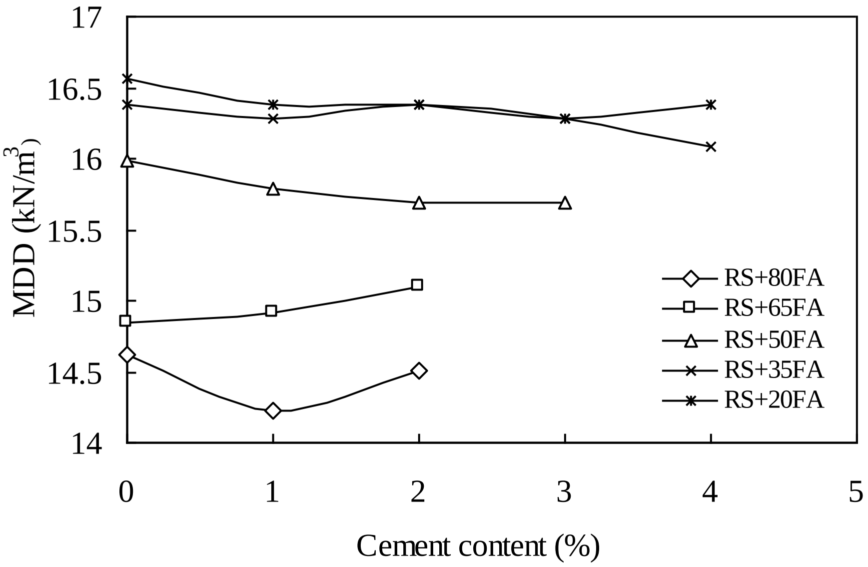


Fig. 4.15(a) Variation of MDD of red soil-fly ash-cement mixes

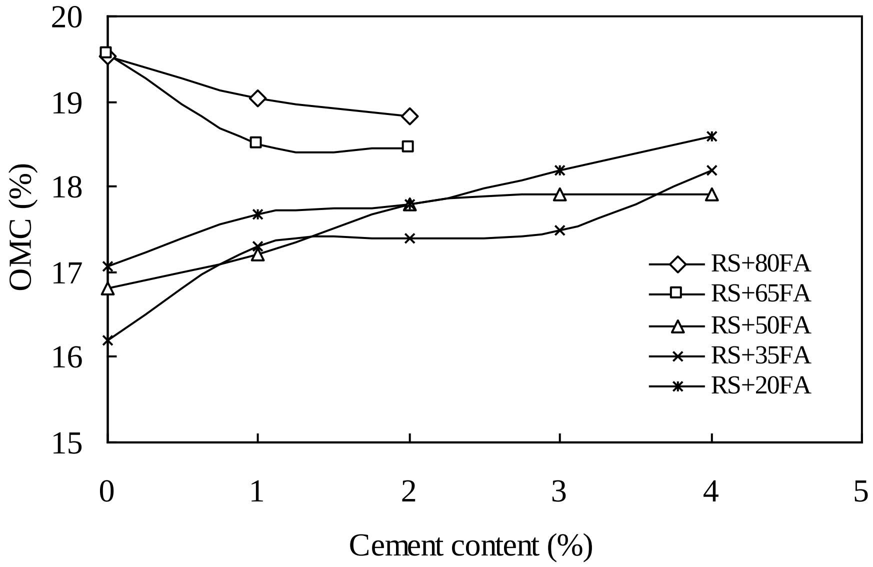


Fig. 4.15(b) Variation of OMC of red soil-fly ash-cement mixes

4.3.4 Compaction of sand-fly ash mixes

The compaction curves of the different sand-fly ash mixes are depicted in Fig. 4.16. It is observed that the compaction behaviour of most of the various mixes resemble more that of cohesionless soils. The variation of dry unit weight with moisture content of sand-fly ash mixes is less compared to those of red soil-fly ash mixes. Fig. 4.17 shows variation of compaction characteristics of the mixes. The specific gravity of the sand is 2.71 whereas that of fly ash is 2.08. As the fly ash content increases, the MDD is observed to decrease gradually. The fly ash has MDD of 13.68%, which is lower than that of the sand (15.69 kN/m^3). Thus, the MDD is the maximum for the BS+20FA mix and the minimum for the BS+90FA mix. The decrease in MDD is attributed to the lower specific gravity of the ash particles.

The OMC of the sand-fly ash mixes without cement decreases initially up to 20% fly ash content, remains at the same level up to 65% fly ash content, and then increases with fly ash content. Although the fly ash has OMC of 19.68%, which is higher than that of the sand (16.38%), 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 65% fly ash can be attributed to the higher water holding capacity of the porous and broken fly ash particles.

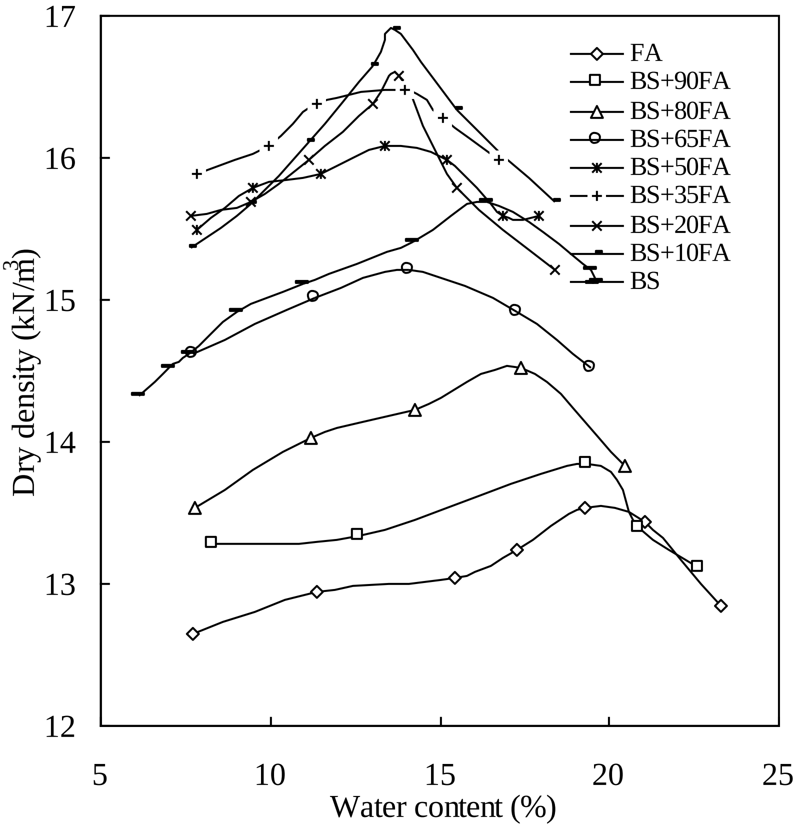


Fig. 4.16 Compaction curves of BS-fly ash mixes

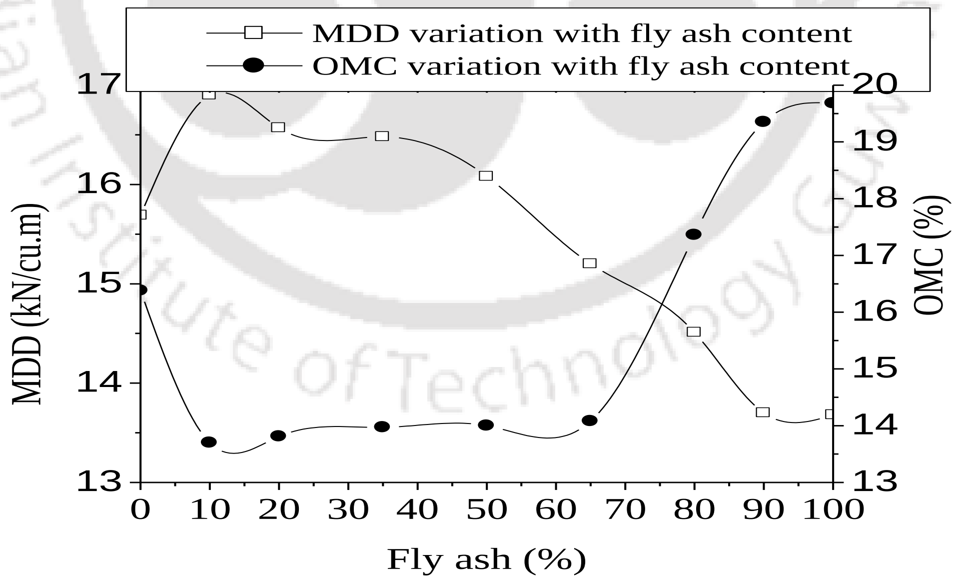


Fig. 4.17 Variation of compaction characteristics of BS-fly ash mixes

4.3.5 Compaction of sand-cement mixes

The compaction curves of the different sand-cement mixes are shown in Fig. 4.18. Fig. 4.19 depicts variation of compaction characteristics of the mixes. It can be observed that there is little variation in the magnitude of MDD with cement content. However, the OMC is found to decrease initially and then increase with cement content.

The cement-mixed soil should be compacted immediately after mixing, because cement begins to hydrate as soon as it comes in contact with water. If the compaction is delayed, some of the cemented bonds that have been formed will be broken down and lost. The presence of sufficient water in the fresh mix is important both for the compaction and for the hydration reactions to proceed. In this study, the compaction test was completed within 2 hours after addition of water.

4.3.6 Compaction of sand-fly ash-cement mixes

Figures 4.20, 4.22, 4.24, 4.26 and 4.28 show the compaction curves of sand-20FA-cement, sand-35FA-cement, sand-50FA-cement, sand-65FA-cement, and sand-80FA-cement mixes, respectively. Figures 4.21, 4.23, 4.25, 4.27 and 4.29 show the variation of compaction characteristics of the above mixes, respectively. The addition of more cement to the sand-fly ash mixes generally causes an increase in the MDD and a decrease in the OMC.

The values of MDD and OMC of all the sand mixes are presented in Table 4.3, whereas Figs .4.30(a) and 4.30(b) show the variations of MDD and OMC only of sand-fly ash-cement mixes, respectively. It is observed that sand-20FA-cement mixes show higher MDD than the other mixes. Sand-80FA-cement mixes show lower MDD and higher OMC than the other mixes.

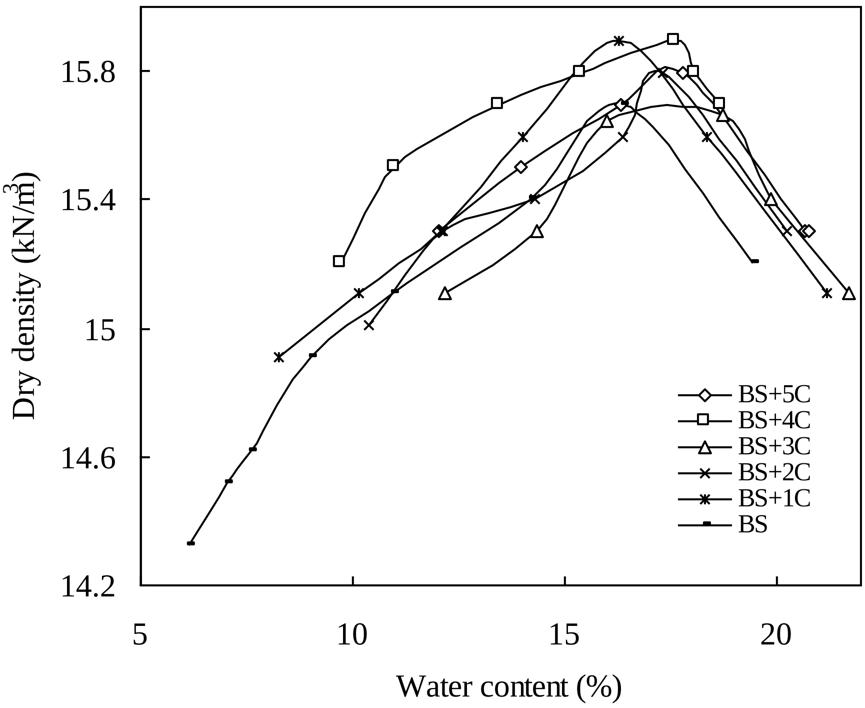


Fig. 4.18 Compaction curves of BS-cement mixes

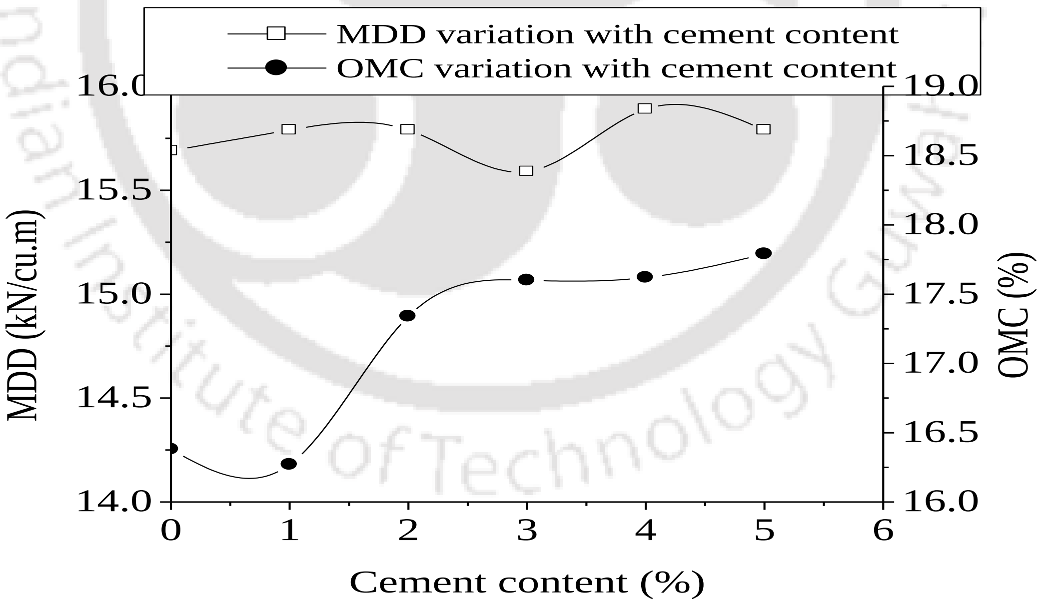


Fig. 4.19 Variation of compaction characteristics of BS-cement mixes

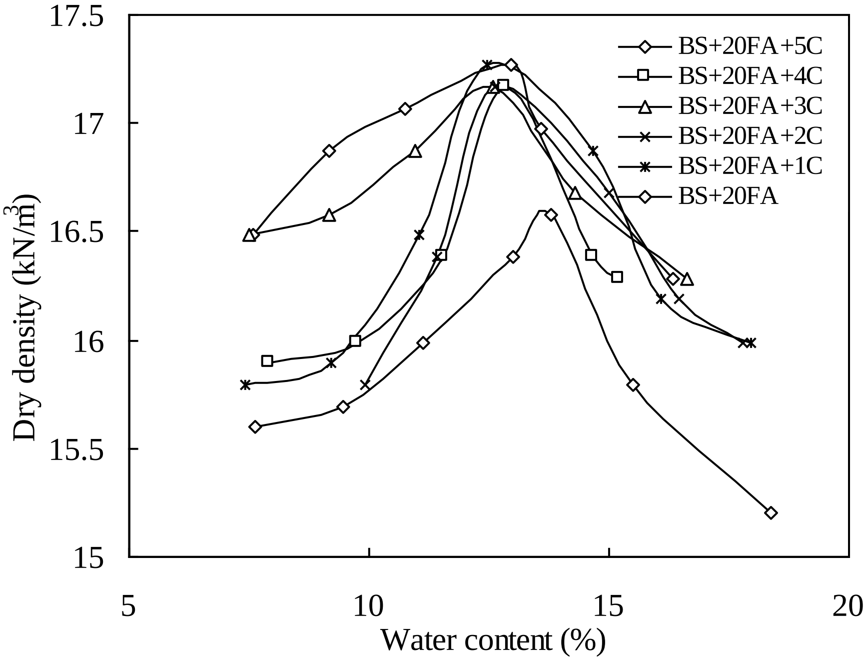


Fig. 4.20 Compaction curves of BS-20FA-cement mixes

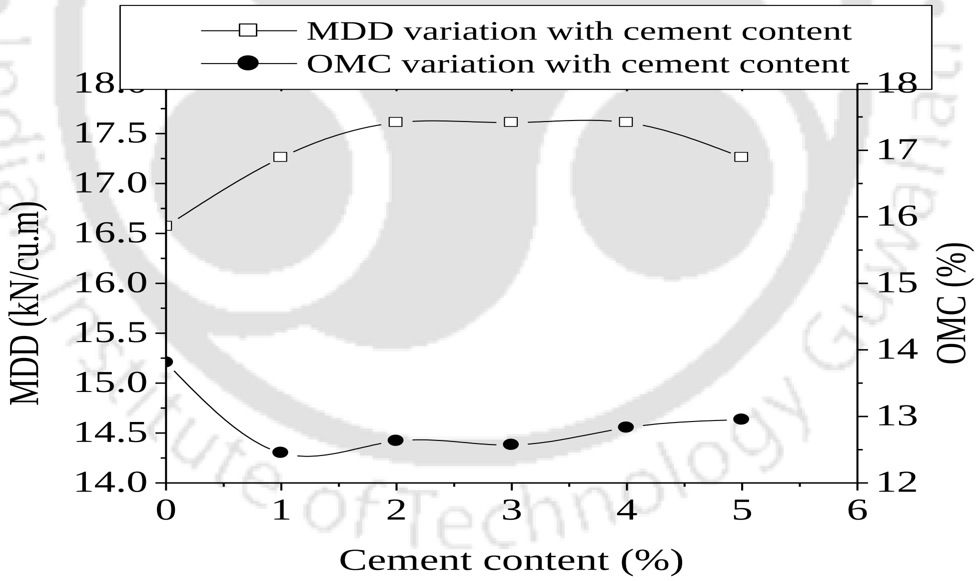


Fig. 4.21 Variation of compaction characteristics of BS-20FA-cement mixes

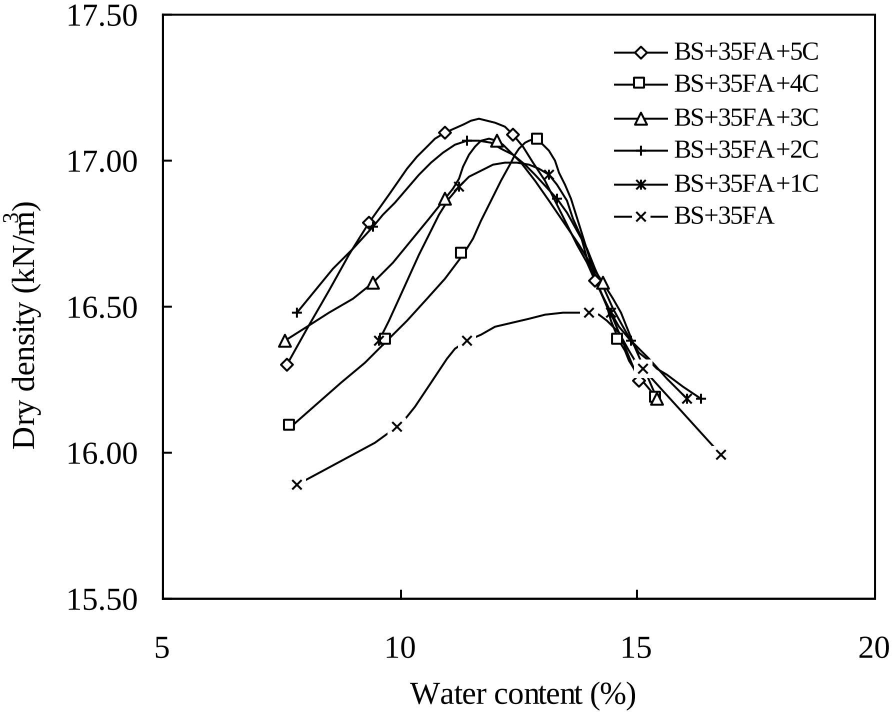


Fig. 4.22 Compaction curves of BS-35FA-cement mixes

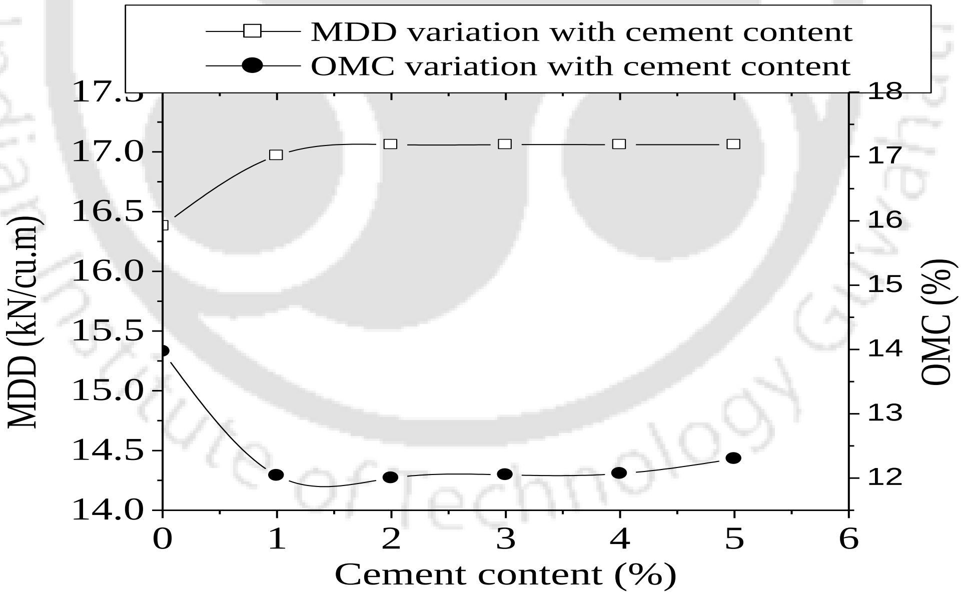


Fig. 4.23 Variation of compaction characteristics of BS-35FA-cement mixes

TH-1088_AKALITA

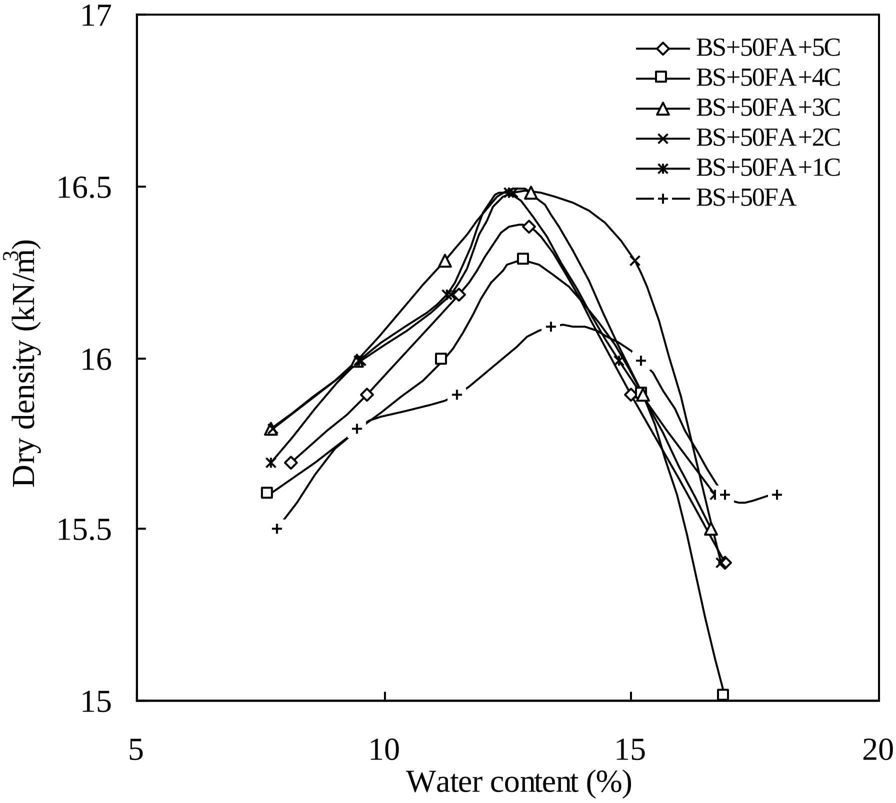


Fig. 4.24 Compaction curves of BS-50FA-cement mixes

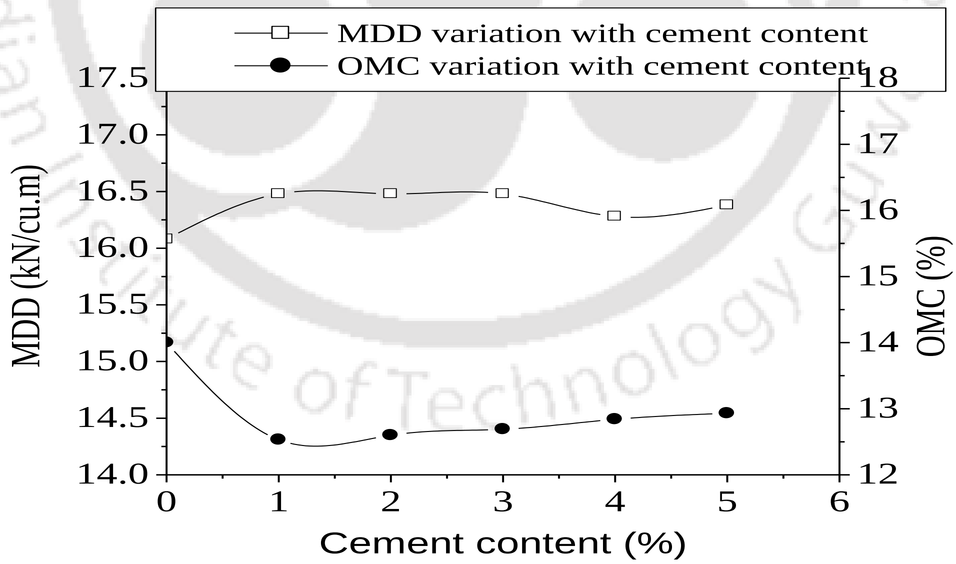


Fig. 4.25 Variation of compaction characteristics of BS-50FA-cement mixes

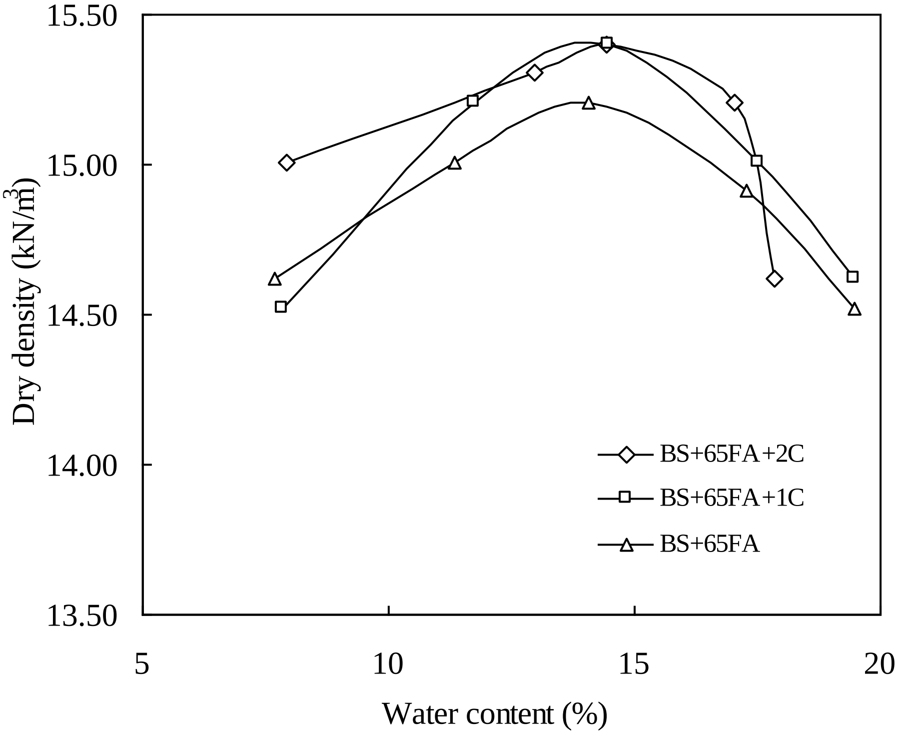


Fig. 4.26 Compaction curves of BS-65FA-cement mixes

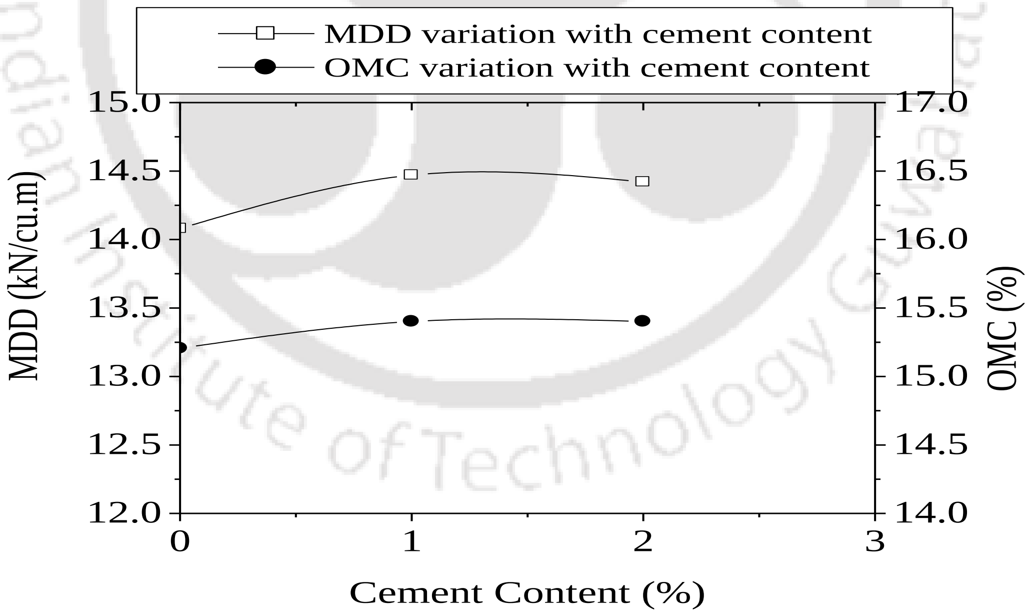


Fig. 4.27 Variation of compaction characteristics of BS-65FA-cement mixes

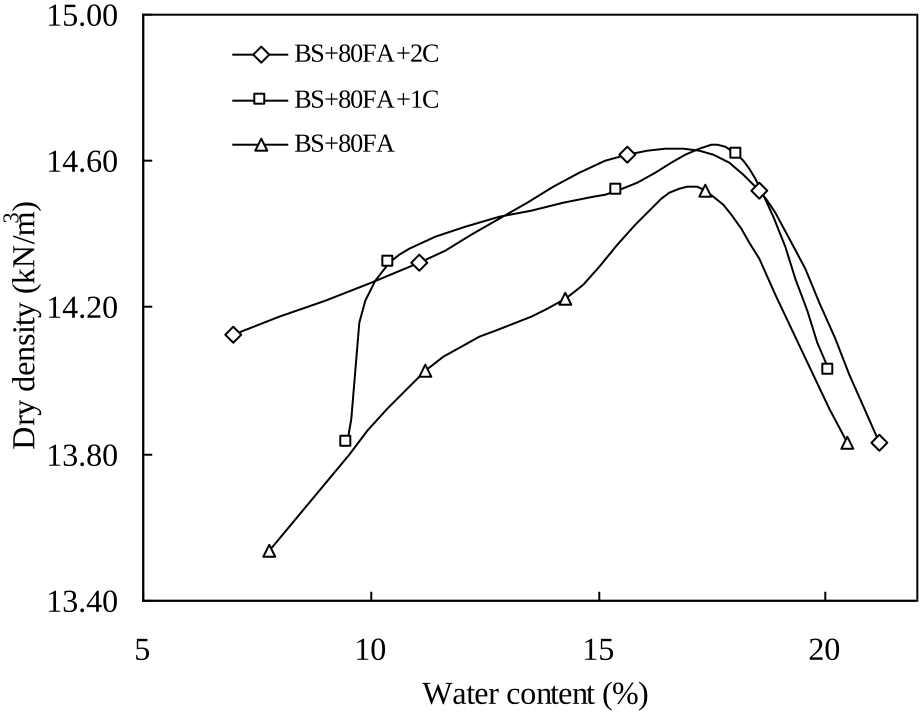


Fig. 4.28 Compaction curves of BS-80FA-cement mixes

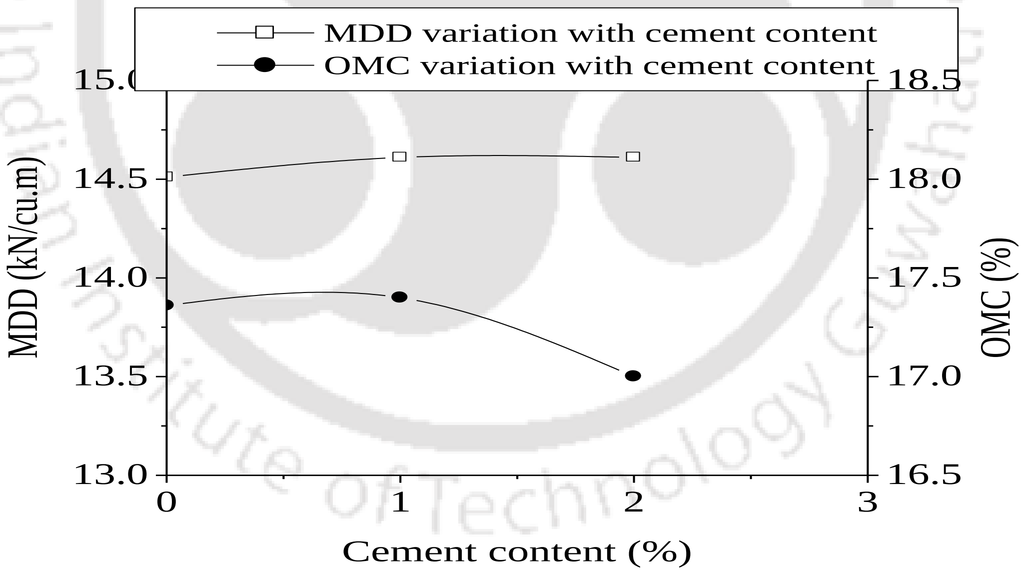


Fig. 4.29 Variation of compaction characteristics of BS-80FA-cement mixes

Table 4.3: MDD and OMC values of all sand mixes

S.No.	Mix	MDD (kN/m^3)	OMC (%)
1	BS	15.69	16.38
2	BS+10FA	16.90	13.70
3	BS+20FA	16.57	13.81
4	BS+35FA	16.48	13.97
5	BS+50FA	16.08	14.00
6	BS+65FA	15.20	14.08
7	BS+80FA	14.51	17.36
8	BS+90FA	13.70	19.35
10	BS+1C	15.79	16.27
11	BS+2C	15.79	17.34
12	BS+3C	15.59	17.6
13	BS+4C	15.89	17.62
14	BS+5C	15.79	17.79
15	BS+20FA+1C	17.26	12.45
16	BS+20FA+2C	17.61	12.63
17	BS+20FA+3C	17.61	12.57
18	BS+20FA+4C	17.61	12.83
19	BS+20FA+5C	17.26	12.95
20	BS+35FA+1C	16.97	12.04
21	BS+35FA+2C	17.06	12.0
22	BS+35FA+3C	17.06	12.05
23	BS+35FA+4C	17.06	12.07
24	BS+35FA+5C	17.06	12.3
25	BS+50FA+1C	16.48	12.53
26	BS+50FA+2C	16.48	12.6
27	BS+50FA+3C	16.48	12.69
28	BS+50FA+4C	16.28	12.84
29	BS+50FA+5C	16.38	12.93
30	BS+65FA+1C	15.40	14.47
31	BS+65FA+2C	15.59	14.10
32	BS+80FA+1C	14.61	17.4
33	BS+80FA+2C	14.61	17.0

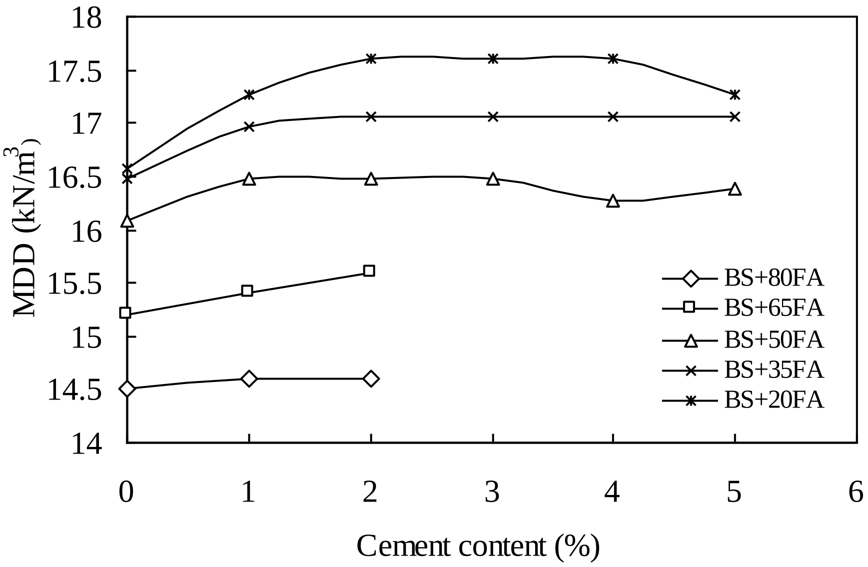


Fig. 4.30(a) Variation of MDD of sand-fly ash-cement mixes

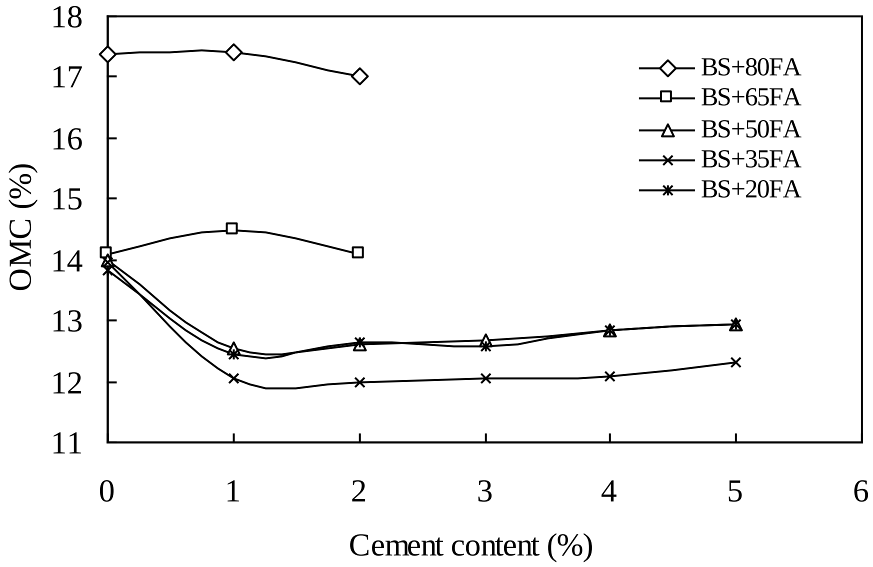


Fig. 4.30(b) Variation of OMC of sand-fly ash-cement mixes

4.4 Conclusions

When fly ash is added to either soil type, the maximum dry unit weight decreases significantly with increasing fly ash content. The MDD values of red soil-fly ash mixes are lower than that of sand-fly ash mixes at the same fly ash content, whereas the corresponding OMC values of red soil-fly ash mixes are higher. The variation of dry unit weight with moisture content of sand-fly ash mixes is less compared to those of red soil-fly ash mixes. When cement is added to the above mixes, the same trends are observed while comparing MDD and OMC values of mixes of the two soils.

The fine-grained red soil has a relatively higher unit weight than the granular sand under a given compactive effort. Hence, they exhibit a lower MDD and a higher OMC. The red soil particles cannot be packed as compactly as the sand particles by virtue of either double layer repulsion or flocculent nature of the soil fabric. With the result, their water holding capacity is also relatively more.

For selection of the appropriate mix proportions for practical applications, it is necessary to investigate the development and variation of strength of the specimens compacted at the respective MDD and OMC values, in uncured and cured states as well as under both unconfined and confined conditions through various strength tests.

CHAPTER - 5
DIRECT SHEAR TESTS ON SAND-FLY ASH MIXES

5.1 Introduction

Shear strength of sand and sand-fly ash mixes can be measured in a direct shear box test. The shear strength of the soil mix is a measure of its ability to support imposed stresses. It is a function of inter-granular frictional resistance and cohesion. In this test, the specimen is subjected to an applied shear stress under a normal stress, which mobilizes the shear strength across a predefined horizontal failure plane. If the test is conducted at a higher rate of shearing, then that test is known as quick test, which results in undrained shear strength parameters. On the other hand, if the shearing is done with a lower rate of strain, the test is known as the drained test, which results in drained strength parameters. The shear stress is determined at failure for each normal stress. The data from direct shear box test are plotted in the form of failure envelope as shear stress vs. normal stress plot with the same scale for both the ordinate and abscissa on the chart. The shear strength parameters that are measured from this graph are the angle of shearing resistance, which is the angle of inclination of the failure envelope with the normal stress axis and the cohesion intercept, which is the intercept made by the failure envelope.

5.2 Test Programme

Table 5.1 summarizes the composition of various mixes used in the direct shear test programme. Two series of tests were conducted. In the first series, only sand specimens were tested at different normal stresses. In the second series, tests were carried out on specimens of various sand-fly ash mixes.

TH-1088_AKALITA

Table 5.1: Composition of various mixes for direct shear tests

Soil	Soil-fly ash mixes
BS	BS+20FA
	BS+35FA
	BS+50FA
	BS+65FA
	BS+80FA

5.3 Results and Discussion

Fig. 5.1 shows variation of shear stress at failure for all sand-fly mixes at various normal stresses used, and the values are listed in Table 5.2. From Fig.5.1, it is observed that the shear stress at failure of all sand-fly mixes is lower than that of sand alone. Among the mixes, the BS+35FA mix generally shows higher shear stress at failure than the remaining mixes at any normal stress. As the fly ash content increases, the range of shear stresses at failure becomes narrower between the lowest and the highest normal stresses.

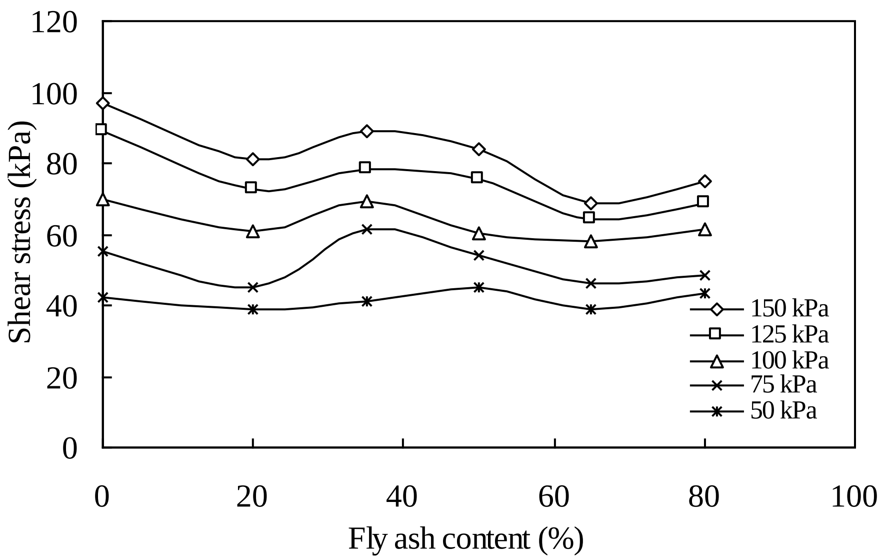


Fig. 5.1 Variation of shear stress at failure with fly ash content and normal stresses

TH-1088_AKALITA

Table 5.2 Shear stress at failure for all mixes at various normal stresses

	Shear stress (kPa)					
Normal stress (kPa)	BS	BS+20FA	BS+35FA	BS+50FA	BS+65FA	BS+80FA
50	42.5	38.69	41.23	45.16	39.00	43.14
75	55.18	45.04	61.43	53.87	46.00	48.21
100	69.78	60.90	69.41	60.26	58.00	61.53
125	88.81	72.47	78.32	75.49	64.07	69.00
150	97.00	81.25	89.00	84.00	69	75.00

Figure 5.2 depicts failure envelopes of all the sand-fly ash mixes. The angle of shearing resistance (ϕ) and cohesion intercept (c) of the mixes obtained from linear regression analyses are presented in Table 5.3. It is found that the correlation coefficients are almost equal to unity in the analyses ranging from 0.97 to 0.99.

Figure 5.3 shows the variation of friction angle and cohesion with fly ash content. The friction angle is noted to decrease with fly ash content. In contrast, the cohesion intercept increases with fly ash content, and the amount of increase is quite less pronounced at low fly ash content of 20% and is more prominent at higher fly ash contents. When fly ash is added to the sand, the frictional component of shear strength decreases whereas the cohesion component increases. The overall shear strength decreases as noted from the shear stresses at failure in Fig. 5.1.

The increase in the cohesion of sand-fly ash matrix is due to the improved gradation of the mix. Under moist conditions, the presence of more fly ash in the mix is expected to lead to significant interlocking effect, resulting in still higher cohesion intercept. It should be noted that the MDD of moist sand-fly ash mixes decreases with increasing fly ash content as observed from Fig. 4.17.

TH-1088_AKALITA

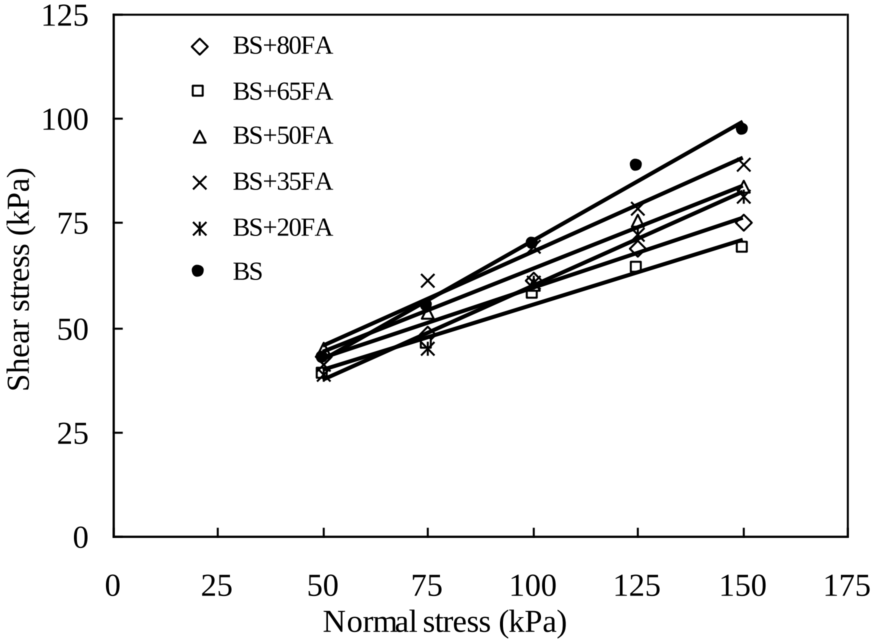


Fig. 5.2 Failure envelopes of sand-fly ash mixes

Table 5.3 Strength parameters of sand-fly ash mixes

MIX	c_d (kPa)	ϕ_d (Degrees)	Correlation coefficient
BS	13.60	29.7	0.99
BS+20FA	14.65	24.22	0.99
BS+35FA	22.91	22.66	0.97
BS+50FA	24.03	21.80	0.98
BS+65FA	23.98	17.35	0.98
BS+80FA	25.56	18.67	0.98

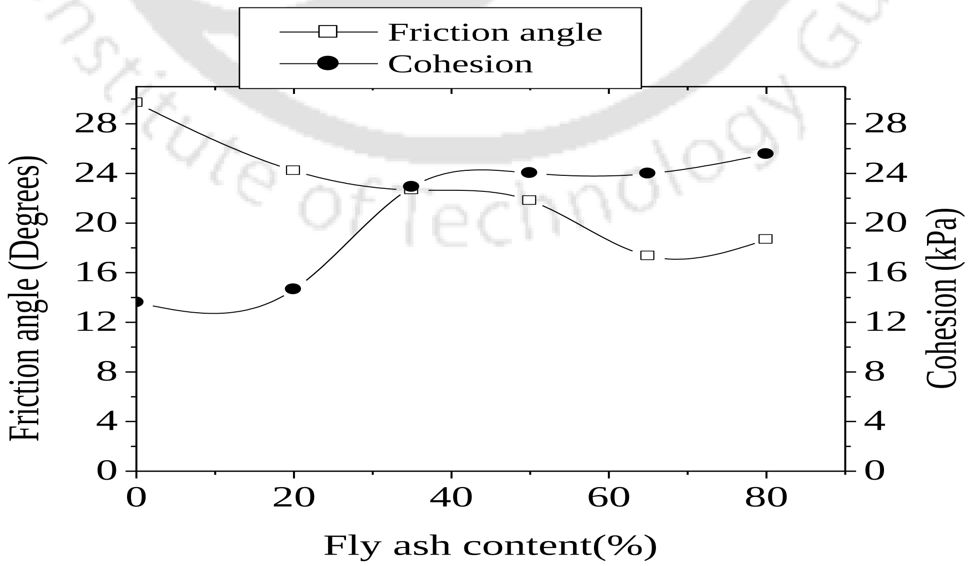


Fig. 5.3 Variation of friction angle and cohesion of various sand-fly ash mixes

TH-1088_AKALITA

Figures 5.4 to 5.9 depict vertical displacement-shear displacement plots of sand, sand-20FA, sand-35FA, sand-50FA, sand-65FA and sand-80FA mixes, respectively. The sand and the mixes exhibit similar trends of variation in vertical vs. shear displacements indicating an initial contraction followed by dilation. The sand alone dilates after a small contraction. But sand-fly ash mixes are first condensed and then dilate as shear progresses. At any fly ash content, the expansion in volume decreases generally with increasing normal stress. As the fly ash content of the mix increases, the dilatancy effect becomes progressively lesser than that of sand alone.

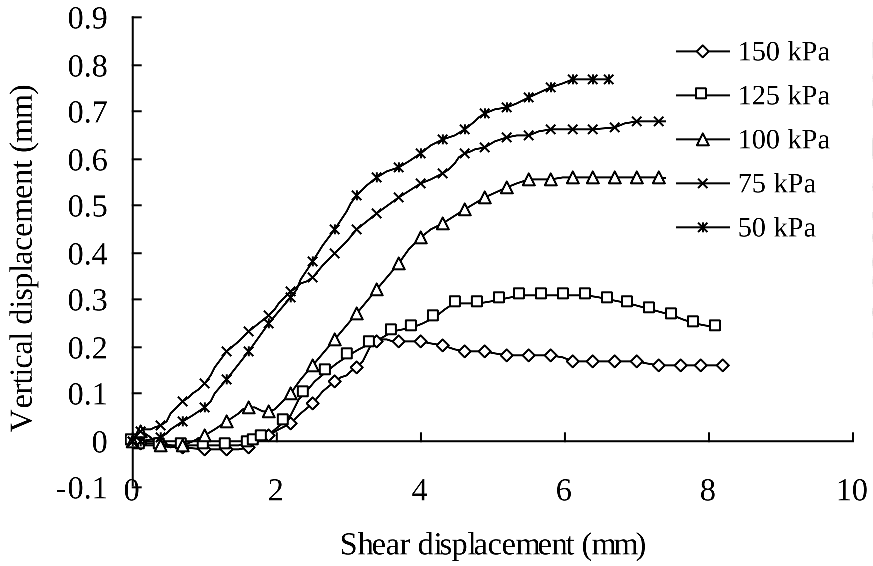


Fig. 5.4 Vertical displacement vs. shear displacement plots of sand

TH-1088_AKALITA

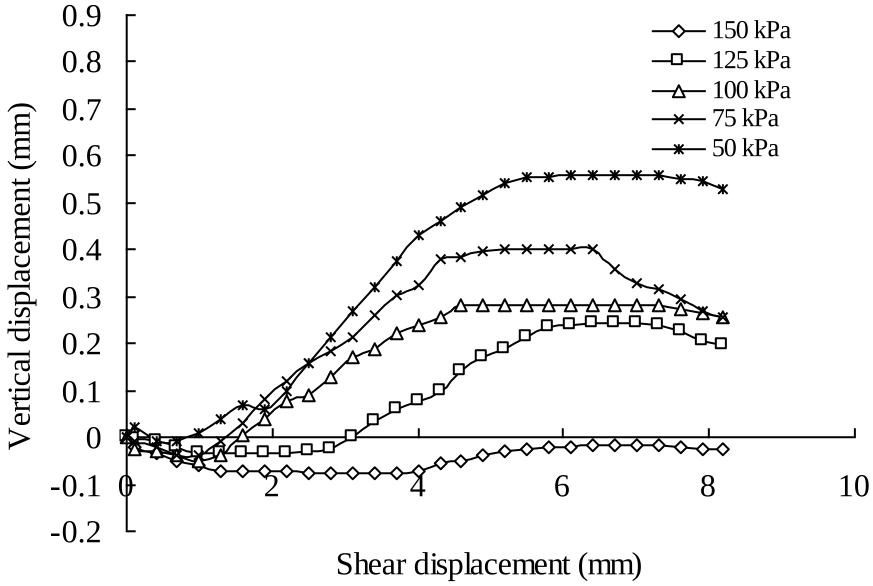


Fig. 5.5 Vertical displacement vs. shear displacement plots of sand-20FA mix

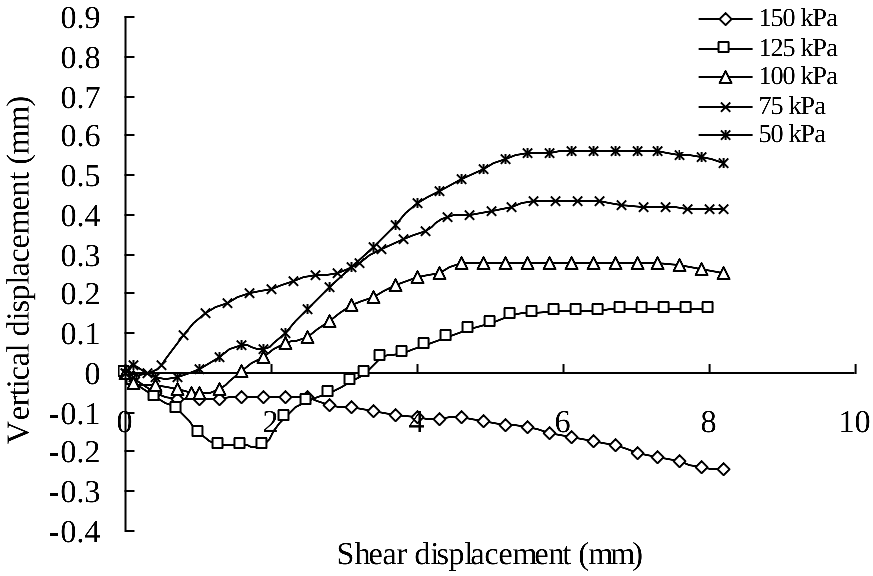


Fig. 5.6 Vertical displacement vs. shear displacement plots of sand-35FA mix

TH-1088_AKALITA

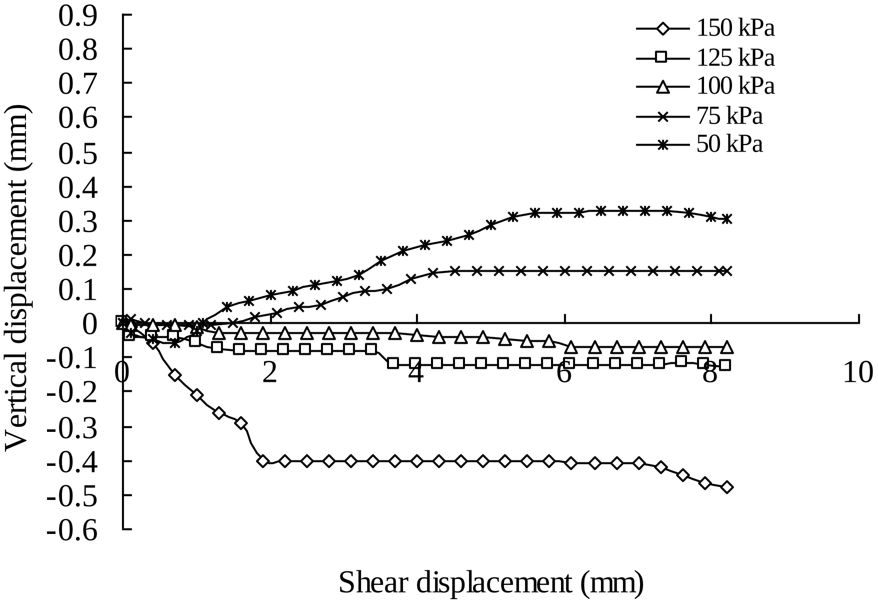


Fig. 5.7 Vertical displacement vs. shear displacement plots of sand-50FA mix

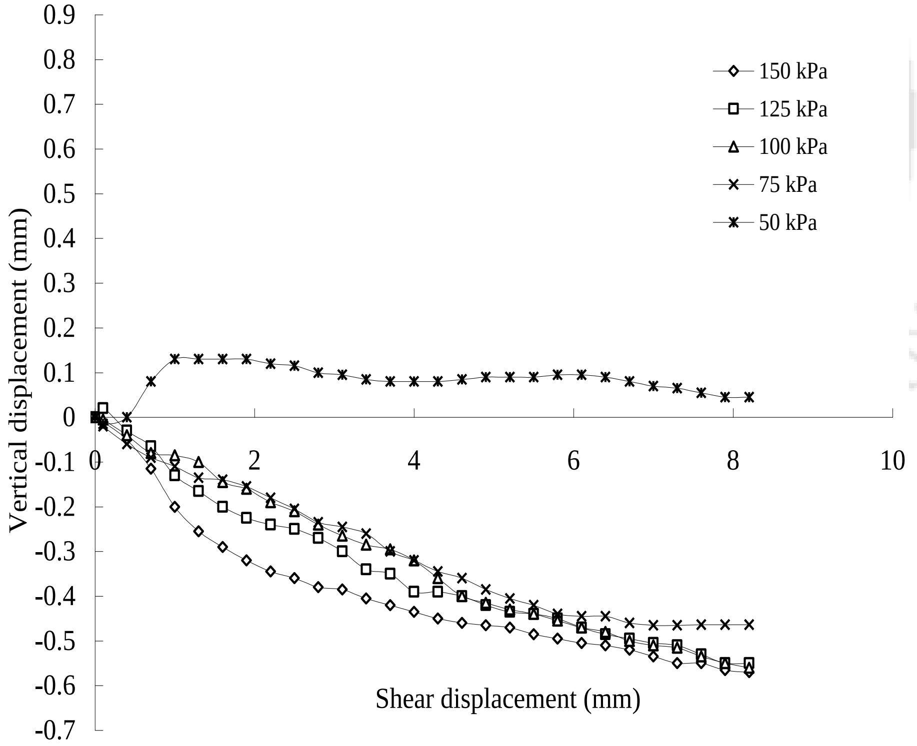


Fig. 5.8 Vertical displacement vs. shear displacement plots of sand-65FA mix

TH-1088_AKALITA

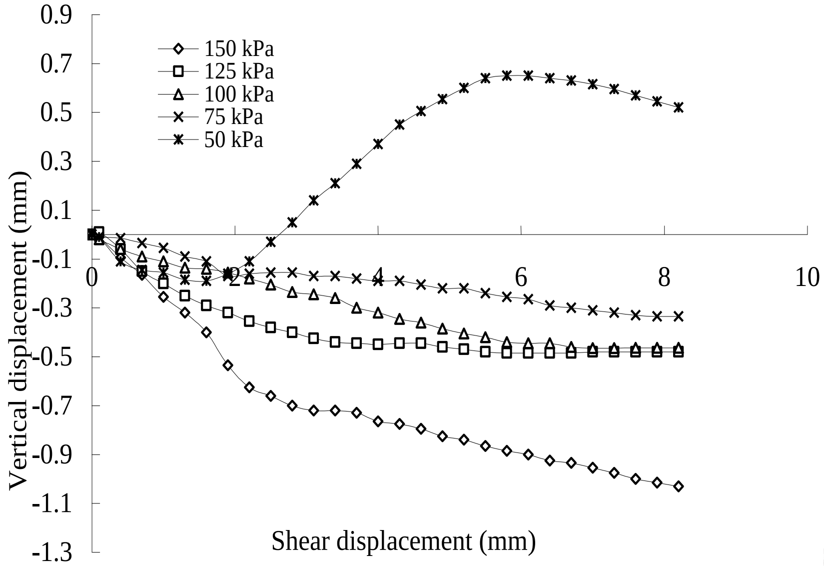


Fig. 5.9 Vertical displacement vs. shear displacement plots of sand-80FA mix

5.4 Conclusions

When fly ash is added to the sand, the cohesion component of shear strength increases whereas the frictional component decreases. The increase in cohesion is more prominent at higher fly ash contents. However the overall shear strength is the maximum for BS+35FA mix followed by BS+50FA and BS+20FA mixes. The mixes exhibit similar trends of variation in vertical vs. shear displacement indicating an initial contraction followed by dilation. As the fly ash content of the mix increases, the dilatancy effect becomes progressively lesser than that of sand alone. At any fly ash content, the expansion in volume decreases generally with increasing normal stress.

In order to understand the contribution of development of cementation between grains on the shear strength of soil-fly ash mixes treated with cement, it is necessary to conduct CBR, unconfined compression and triaxial tests on compacted specimens after various stages of curing.

TH-1088_AKALITA

CHAPTER-6

CALIFORNIA BEARING RATIO TESTS ON SOIL-FLY ASH-CEMENT MIXES

6.1 Introduction

The basis for the design of low-volume flexible roads in India is the strength of subgrade determined in terms of soaked CBR value. Higher the CBR value of subgrade, lower will be the overall pavement thickness that is required. The literature review indicates that several studies have been carried out which are related to the use of fly ash and cement for improving CBR values of subgrade soils. Pandian et al. (2001) investigated the effect of different percentages of fly ash addition to black cotton soil and found that only proper proportions of black cotton soil and fly ash improved the CBR values. The addition of fly ash reduced the volume change characteristics and improved the subgrade strength. Pandian and Krishna (2002) studied the effect of the addition of ordinary Portland cement on the CBR of black cotton soil-fly ash mixes.

6.2 Test Programme

Table 6.1 summarizes the composition of the various mixes used in the CBR test programme. These tests were done to investigate the strength under confined conditions, which are different from those of unconfined compression (UC) tests. Three different series of tests were conducted. In the first series, the CBR characteristics of the soils and fly ash were investigated separately. In the second series, tests were conducted to study the effect of blending of the soils with fly ash. In the third series, tests were carried out after the addition of cement to the soil-fly ash mixes. The CBR tests were carried out on all specimens of the mixes under both unsoaked and soaked conditions.

TH-1088_AKALITA

Table 6.1: Composition of various mixes for CBR tests

Soil	Fly ash	Soil-fly ash mixes	Soil-cement mixes	Soil-fly ash-cement mixes
RS	FA	RS+10FA	RS+1C	RS+10FA+1C
		RS+20FA	RS+2C	RS+10FA+2C
		RS+35FA	RS+3C	
		RS+50FA		RS+20FA+1C
		RS+65FA		RS+20FA+2C
		RS+80FA		RS+20FA+3C
		RS+90FA		
				RS+35FA+1C
				RS+35FA+2C
				RS+35FA+3C
				RS+50FA+1C
				RS+50FA+2C
				RS+65FA+1C
				RS+65FA+2C
				RS+80FA+1C
				RS+80FA+2C
				RS+90FA+1C
				RS+90FA+2C
BS		BS+10FA	BS+1C	BS+10FA+1C
		BS+20FA	BS+2C	BS+10FA+2C
		BS+35FA	BS+3C	
		BS+50FA		BS+20FA+1C
		BS+65FA		BS+20FA+2C
		BS+80FA		BS+20FA+3C
		BS+90FA		
				BS+35FA+1C
				BS+35FA+2C
				BS+35FA+3C
				BS+50FA+1C
				BS+50FA+2C
				BS+65FA+1C
				BS+65FA+2C
				BS+80FA+1C
				BS+80FA+2C
				BS+90FA+1C
				BS+90FA+2C

TH-1088_AKALITA

6.3 Results and Discussion

6.3.1 CBR of both soil-fly ash mixes

Figures 6.1 and 6.2 show variations of CBR values of red soil-fly ash and sand-fly ash mixes, respectively. In each figure, the plots for soil and fly ash alone are also included for comparison. It is observed that the soaked CBR values are generally less than the unsoaked values since the surface tension forces, which were offering additional resistance to penetration under the unsoaked condition get destroyed under soaked condition.

The results under unsoaked conditions show higher CBR values for both the soil-fly ash mixes. Similar observation was reported by Pandian (2004). This is mainly because fly ash, when placed at 95% Proctor maximum dry unit weight and corresponding water content, exhibits capillary forces, in addition to friction for resisting the penetration of the plunger, and thus high values of CBR are obtained. When the same fly ash specimens are soaked for 4 days, they exhibit very low values of CBR due to the destruction of capillary forces under soaked conditions.

The CBR values of red soil, sand and fly ash were found to 6.6%, 19.1% and 53.1% for the unsoaked condition, and 4.7%, 11.0% and 1.86% for the soaked condition, respectively. The unsoaked CBR value of red soil increases from 6.6% with no fly ash to a value of 24.6% at 90% fly ash content, whereas the corresponding soaked CBR values are 4.7% and 4.5%. In case of sand, the unsoaked CBR value increases from 19.1% with no fly ash to a value of 58.8% at 90% fly ash content, whereas the corresponding soaked CBR values are 11.1% and 2.9%.

From Figure 6.1, in the unsoaked condition, it is observed that there is no well-defined peak for red soil-fly ash mixes. The unsoaked CBR value increases up to 80% fly ash content and then decreases. The unsoaked CBR value of fly ash alone (53.1%) is

TH-1088_AKALITA

greater than that of other mixes.

From Fig. 6.1, in the soaked condition, it is further observed that there are two peaks for red soil-fly ash mixes. From the gradation curve of fly ash (Fig.3.4), it is observed that the major portion is fine sand fraction with a negligible silt and clay size fraction. The fly ash is a cohesionless material and therefore non-plastic in nature, whereas the red soil is cohesive in nature. The first peak occurring at 20% addition of fly ash (80% red soil) is due to soil stabilization through fly ash as the admixture, whereas the second peak occurring at 50% fly ash addition (50% red soil) represents the fly ash stabilization with the soil as the admixture.

At the first peak, the deficiency of coarse particles in the red soil is taken care of by the coarser fly ash particles, resulting in a higher dry unit weight and greater frictional strength of the soil. This appears to have contributed to higher CBR. At the second peak, the deficiency of fine particles in the fly ash is made up by adding the finer soil particles, resulting in cohesion imparted by the addition of the red soil. This must have contributed to a CBR higher than that of the red soil or fly ash alone. Therefore the double peaks in the CBR values are due to the improvement in the gradation of mixes as well as the cohesion contribution. This observation is similar to that of Pandian et al. (2001).

From Fig. 6.2, in the unsoaked condition, it can be seen that as fly ash is added, the CBR value of the sand-fly ash mixes is lower than that of the sand only at 20% and 35% fly ash contents. This may be due to poor gradation of the resulting two mixes.

From Fig. 6.2, in the soaked condition, it is observed that there are two peaks observed in sand-fly ash mixes exhibiting higher CBR value under soaked condition. At the first peak, the fly ash content is 10% at which CBR value is 20.72%, and the second peak fly ash content is 35% which has CBR value of 17.02%. The CBR values at these two fly ash contents are significantly higher than that of sand alone (11.09%). The first

TH-1088_AKALITA

peak observed at 10% fly ash content is due to the better interlocking between the ash and sand particles. The second peak observed at 35% fly ash content is mainly due to cohesion imparted by the fly ash to the sandy soil by acting as a binder. This observation is similar to that of Pandian et al. (2001) and Pandian and Krishna (2003).

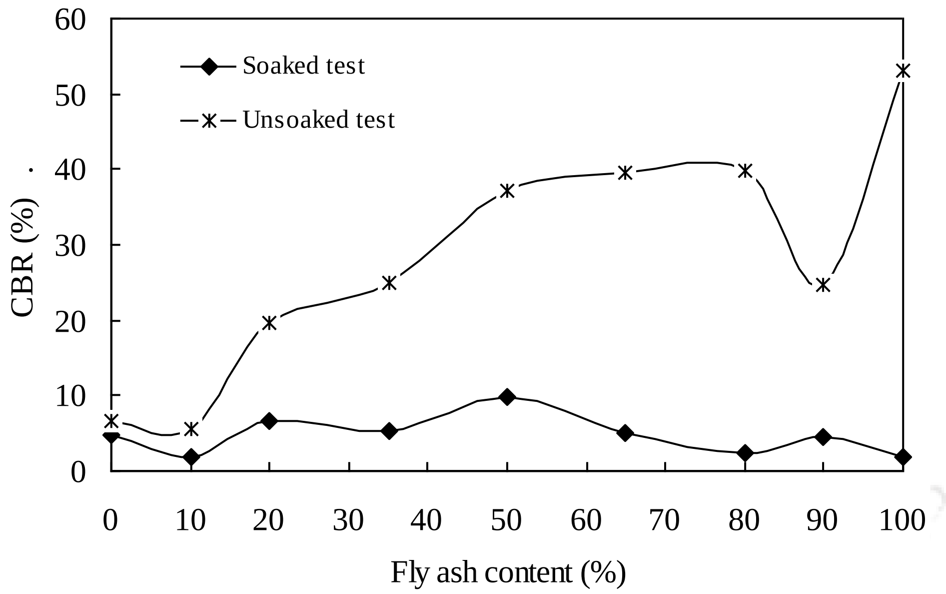


Fig. 6.1 Comparison between unsoaked and soaked CBR values of RS-fly ash mixes

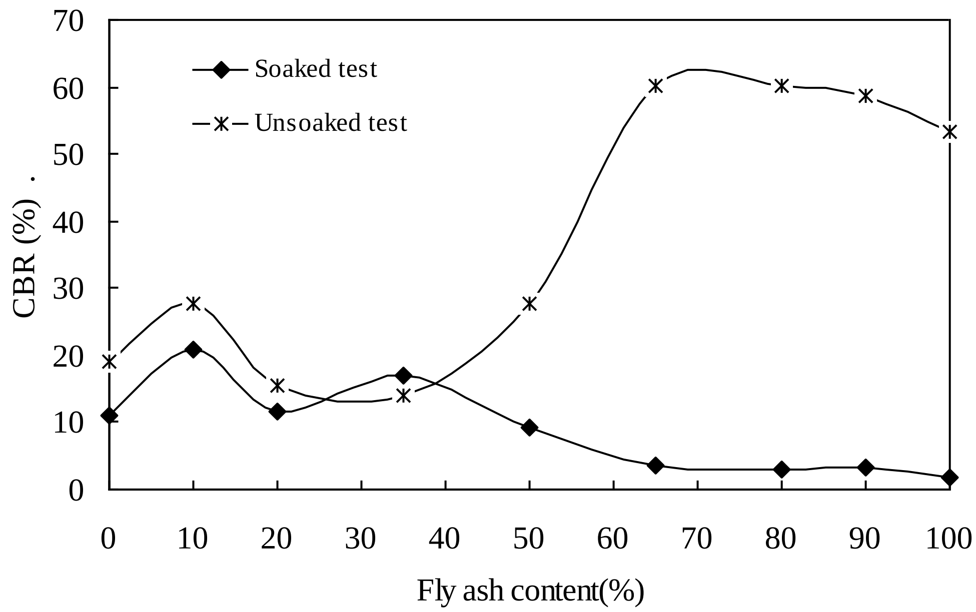


Fig. 6.2 Comparison between unsoaked and soaked CBR values of BS-fly ash mixes

Figure 6.3 shows comparison between soaked CBR values of RS-fly ash and BS-

TH-1088_AKALITA

fly ash mixes. It is observed that CBR values of sand-fly ash mixes are greater than that of red soil-fly ash mixes at lower fly ash contents up to 50%, and they are comparable at higher fly ash contents.

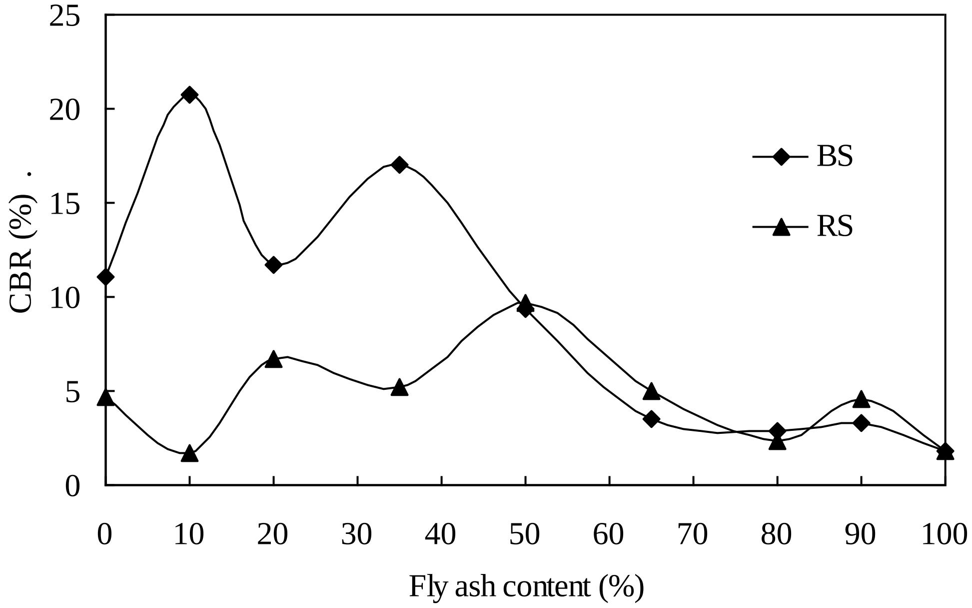


Fig. 6.3 Comparison between soaked CBR values of RS-fly ash and BS-fly ash mixes

6.3.2 CBR of both soil-cement mixes

Figures 6.4 and 6.5 present the variations of CBR values under both unsoaked and soaked conditions for red soil-cement and sand-cement mixes, respectively. The addition of cement has increased the CBR of the mixes for both the soils under soaked conditions. This is because hydration of cement forms calcium silicate hydrate gel, and with the addition of increasing amounts of cement, more of this gel will be formed. As such, there will be a continuous increase in the CBR.

From Figure 6.4, it is observed that the unsoaked CBR value of red soil-cement mixes varies from 18.4% with 1% cement to a value of 17.1% at 3% cement content. Under unsoaked condition, as cement is added, CBR value decreases due to incomplete hydration. In soaked condition, the CBR values are 15.0% and 28.6% at 1% and 3% cement contents, respectively. It is noted that soaked CBR value of red soil increases by

TH-1088_AKALITA

2.19 times even at 1% addition of cement and by 5.08 times after addition of 3% cement content.

From Figure 6.5, it is observed that unsoaked CBR value of sand-cement mixes decreases from 17.6% with 1% cement to a value of 11.8% at 3% cement content. The CBR value decreases due to incomplete hydration of cement under unsoaked condition. In soaked condition, the CBR values are 23.2% and 34.3% at 1% and 3% cement contents. It can be noted that the soaked CBR value of sand increases by 1.10 times at 1% addition of cement and by 2.0 times upon addition of 3% cement content.

The percent gain in soaked CBR values of red soil-cement mixes is found to be generally greater than that of sand-cement mixes. It should be noted that only soaked CBR values are used in the design of pavements. Figure 6.6 shows comparison between soaked CBR values of RS-cement and BS-cement mixes. It is observed that CBR values of sand-cement mixes are higher than that of red soil-cement mixes.

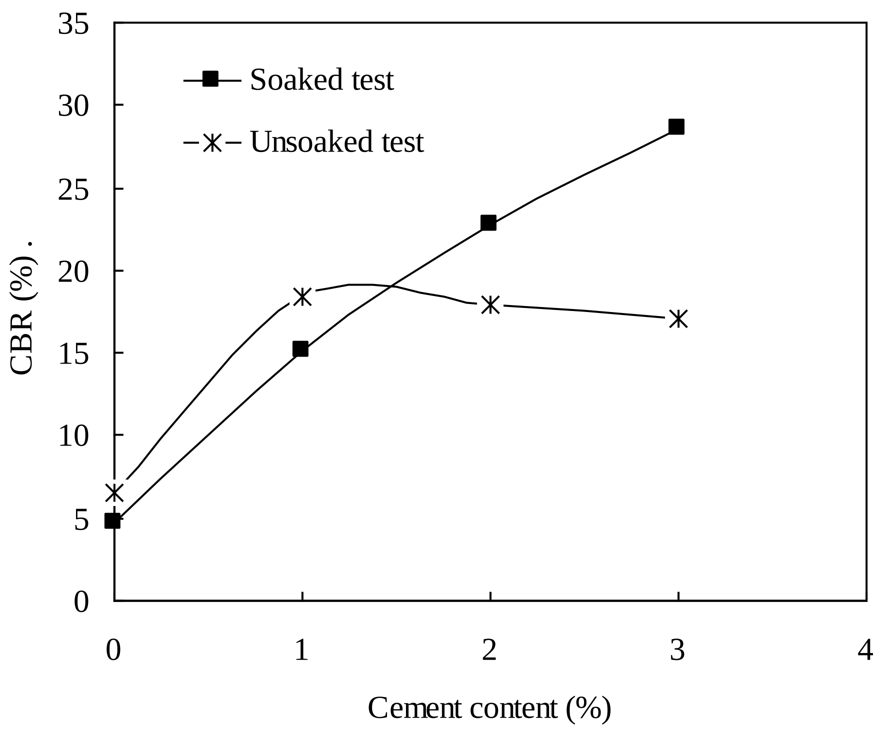


Fig.6.4 Comparison between unsoaked and soaked CBR values of RS-cement mixes

TH-1088_AKALITA

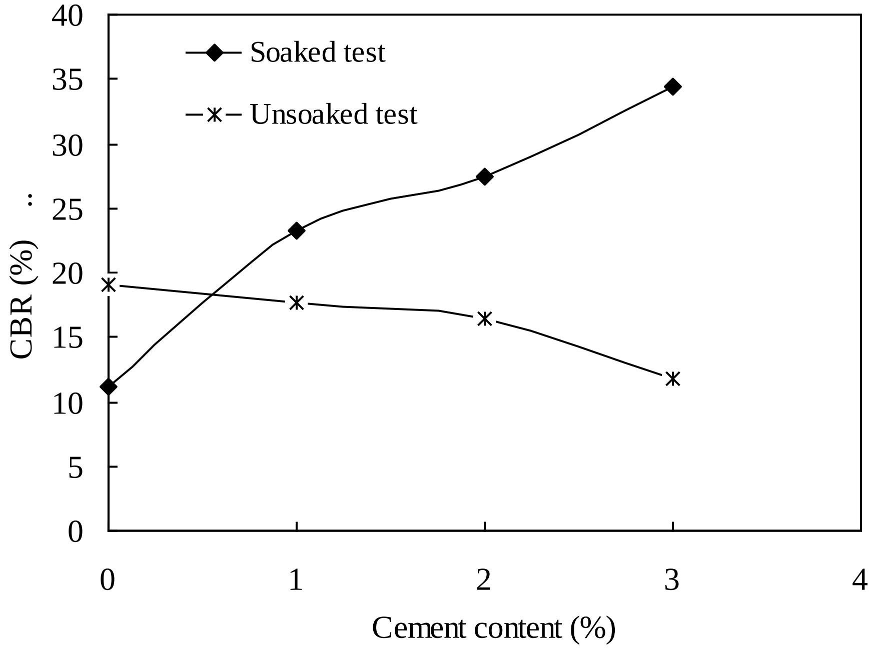


Fig. 6.5 Comparison between unsoaked and soaked CBR values of BS-cement mixes

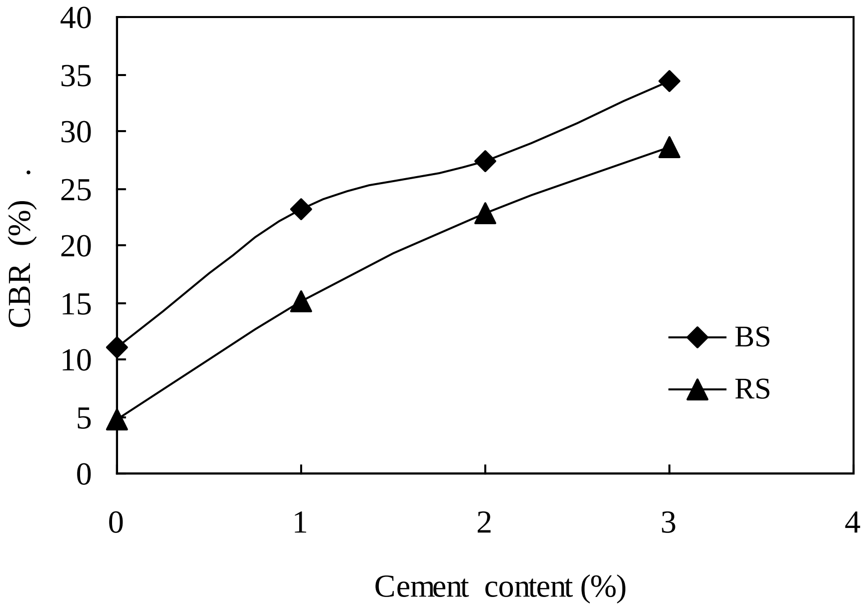


Fig. 6.6 Comparison between soaked CBR values of RS-cement and BS-cement mixes

6.3.3 CBR of red soil-fly ash-cement mixes

Figures 6.7 to 6.13 show comparisons in the variations of CBR of various red soil-fly ash-cement mixes for both unsoaked and soaked conditions. The soaked CBR value is observed to have increased with cement content, whereas the trend for unsoaked

TH-1088_AKALITA

CBR value is not always regular for all the mixes. The addition of a small percentage of cement gives significant improvement in the soaked CBR value. The hydration products contribute to the development of strength (Pandian and Krishna, 2002).

Both the unsoaked and soaked CBR values of all the red soil mixes are presented in Table 6.2. The soaked CBR value of red soil-10FA mix increases from 1.7% with no cement to a value of 23.9% at 2% cement content. Similarly, the corresponding soaked CBR values for red soil-20FA mix are 6.7% and 31.8%, respectively.

Figs. 6.14 and 6.15 present soaked CBR values of all red soil-fly ash-cement mixes. The addition of 2% cement to red soil (RS+2C mix) has resulted in increase of soaked CBR from 4.72% to 22.81% as noted from Fig. 6.6. From Fig 6.15, it is seen that all the red soil-fly ash mixes with a lower quantity of cement at 1% have higher soaked CBR than that of the RS+2C mix. It is not always economical to add more amount of cement to increase the CBR value of soils. Reducing the cement content by way of utilizing more amount of fly ash addition to the red soil leads to significant enhancement in soaked CBR values, and it is also efficient from cost point of view.

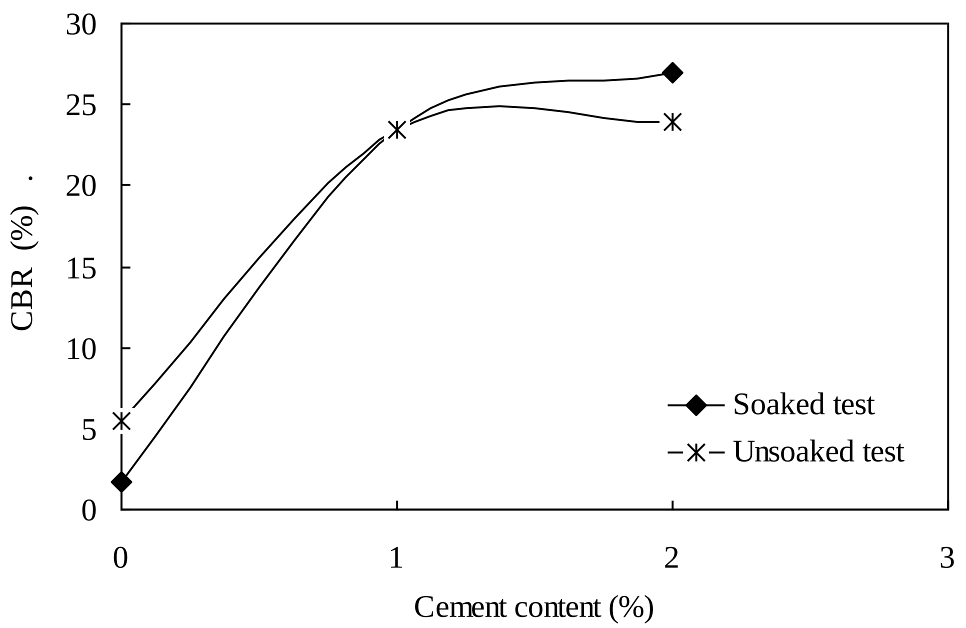


Fig. 6.7 Comparison between unsoaked and soaked CBR values of RS-10FA-cement mixes

TH-1088_AKALITA

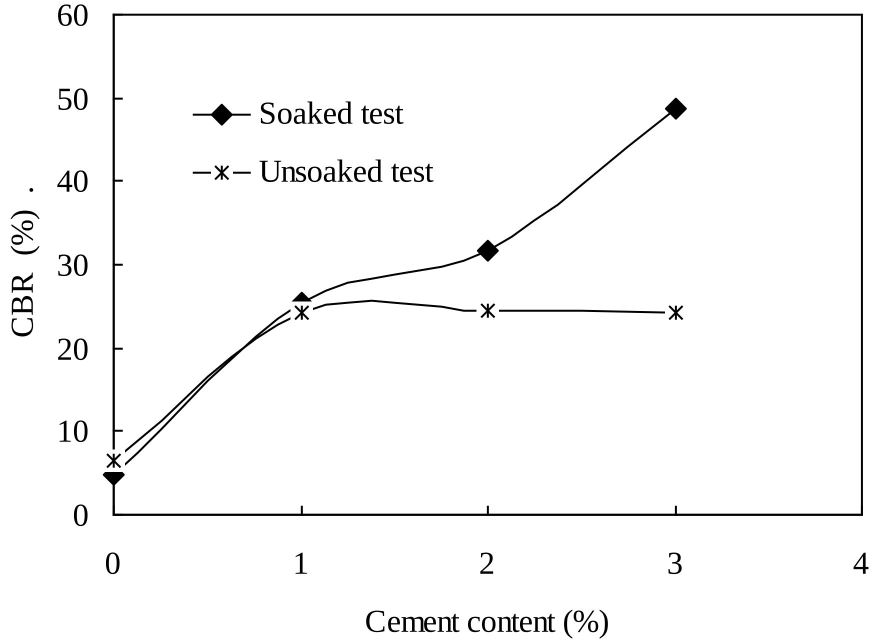


Fig. 6.8 Comparison between unsoaked and soaked CBR values of RS-20FA-cement mixes

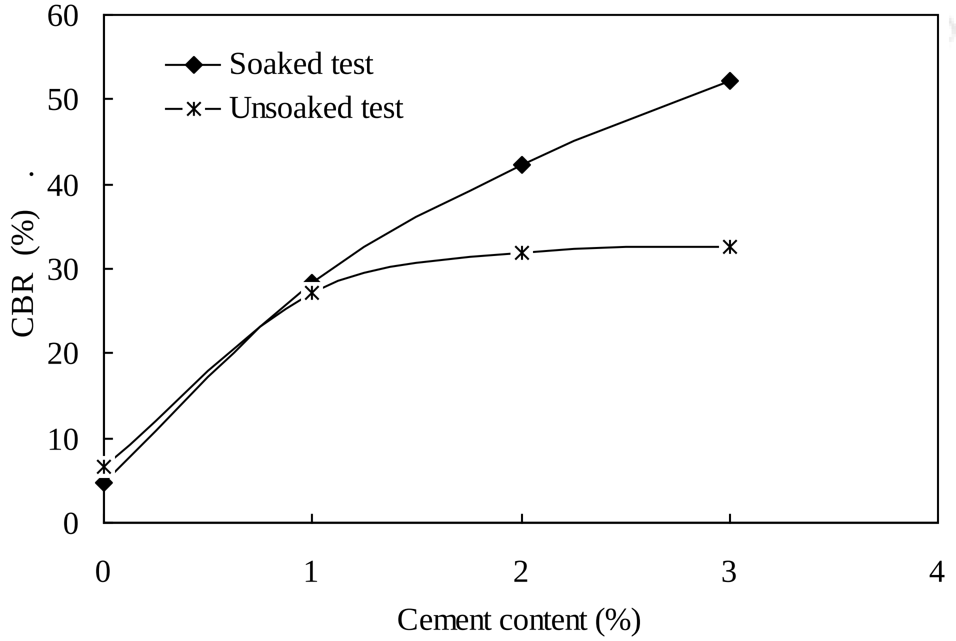


Fig.6.9 Comparison between unsoaked and soaked CBR values of RS-35FA-cement mixes

TH-1088_AKALITA

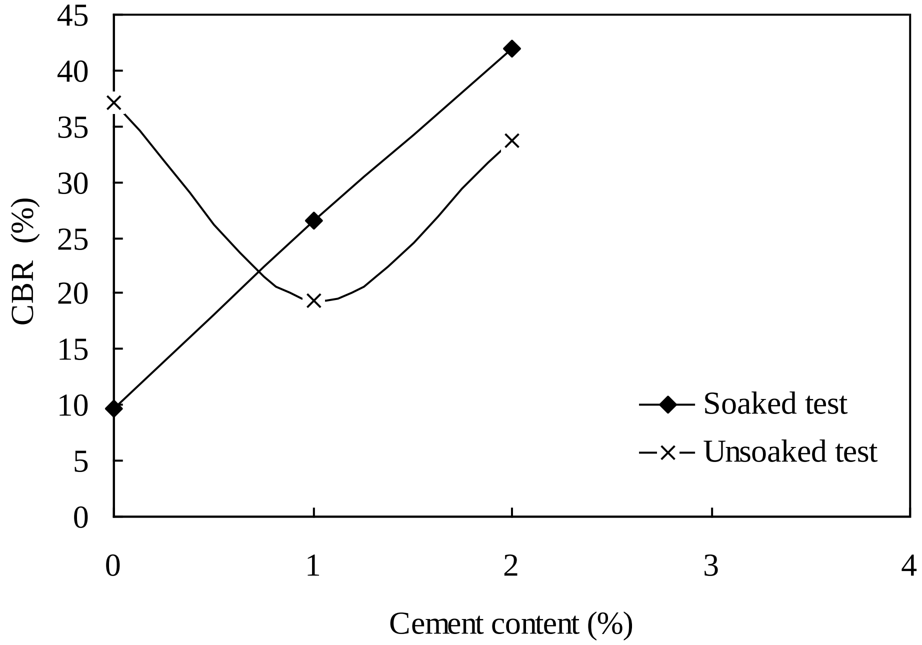


Fig. 6.10 Comparison between unsoaked and soaked CBR values of RS-50FA-cement mixes

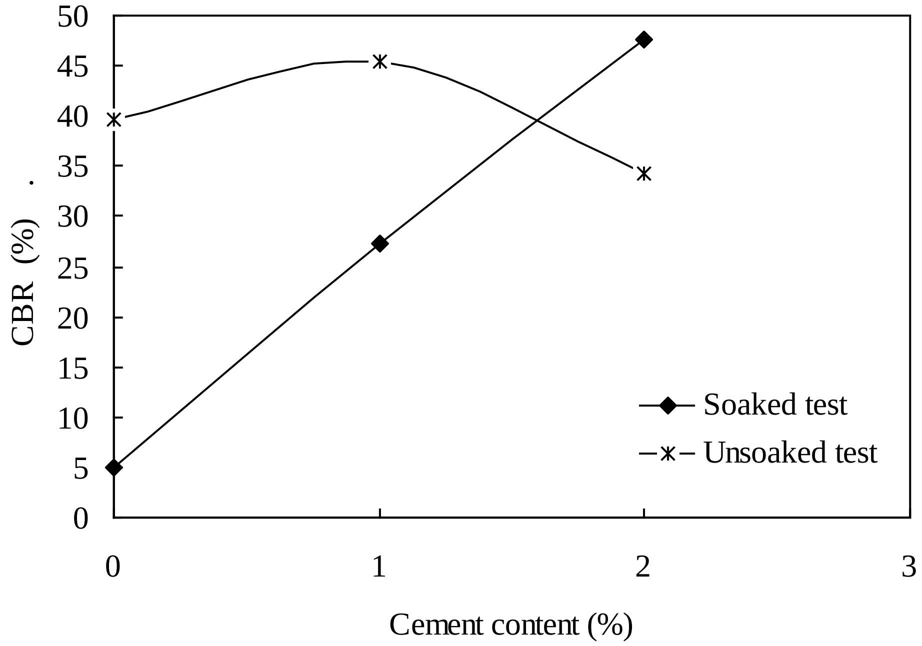


Fig.6.11 Comparison between unsoaked and soaked CBR values of RS-65FA-cement mixes

TH-1088_AKALITA

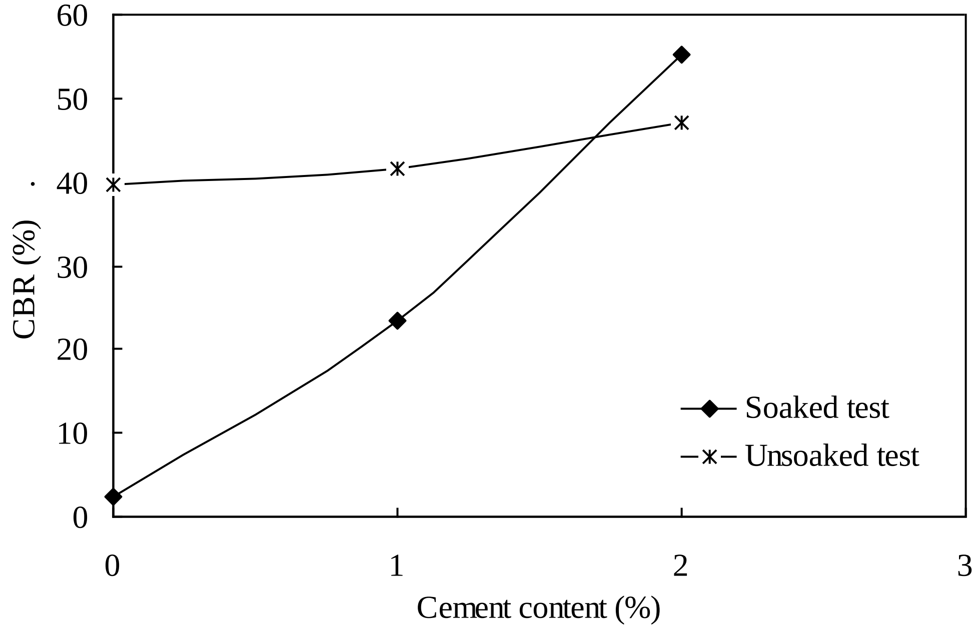


Fig. 6.12 Comparison between unsoaked and soaked CBR values of RS-80FA-cement mixes

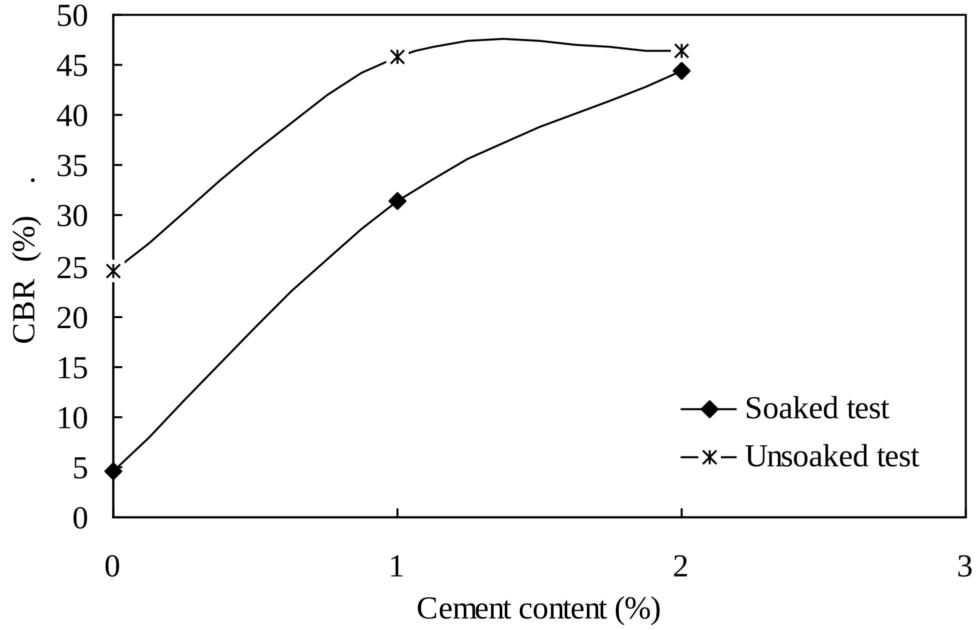


Fig. 6.13 Comparison between unsoaked and soaked CBR values of RS-90FA-cement mixes

TH-1088_AKALITA

Table 6.2: CBR values of all red soil mixes

Mix	Unsoaked CBR (%)	Soaked CBR (%)
RS	6.6	4.7
FA	53.1	1.86
RS+1C	18.1	15.1
RS+2C	17.9	22.8
RS+3C	17.1	28.6
RS+10FA	1.7	5.5
RS+20FA	19.6	6.7
RS+35FA	25	5.2
RS+50FA	37.1	9.7
RS+65FA	39.6	5.0
RS+80FA	39.8	2.3
RS+90FA	24.6	4.5
RS+10FA+1C	23.5	23.4
RS+10FA+2C	23.9	26.9
RS+20FA+1C	24.3	25.4
RS+20FA+2C	24.5	31.8
RS+20FA+3C	24.2	48.6
RS+35FA+1C	27.2	28.4
RS+35FA+2C	31.9	42.2
RS+35FA+3C	32.7	52.1
RS+50FA+1C	19.4	26.6
RS+50FA+2C	33.7	41.9
RS+65FA+1C	45.5	27.2
RS+65FA+2C	34.3	47.6
RS+80FA+1C	41.6	23.4
RS+80FA+2C	47.2	55.3
RS+90FA+1C	45.9	31.5
RS+90FA+2C	46.4	44.3

TH-1088_AKALITA

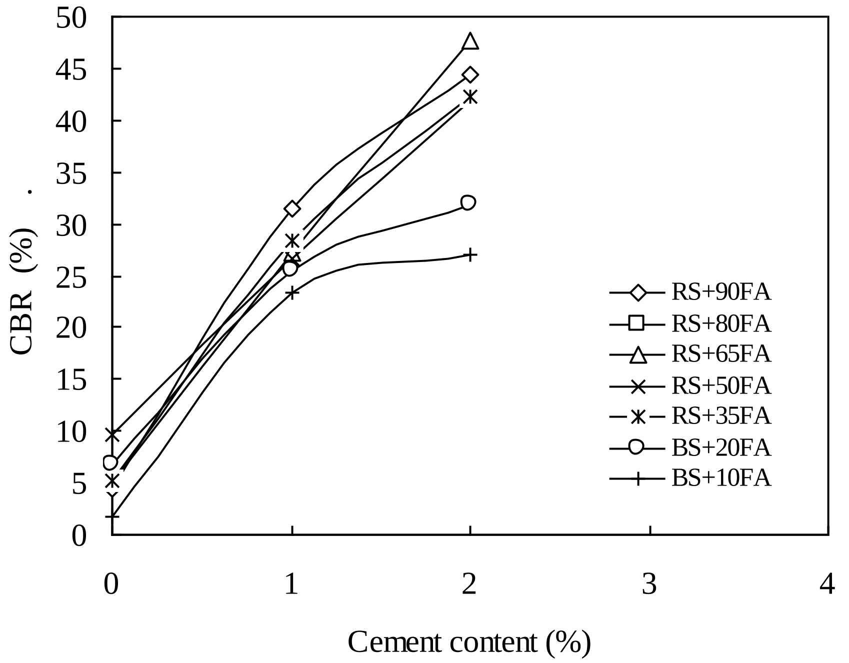


Fig.6.14 Soaked CBR values of all RS-fly ash-cement mixes

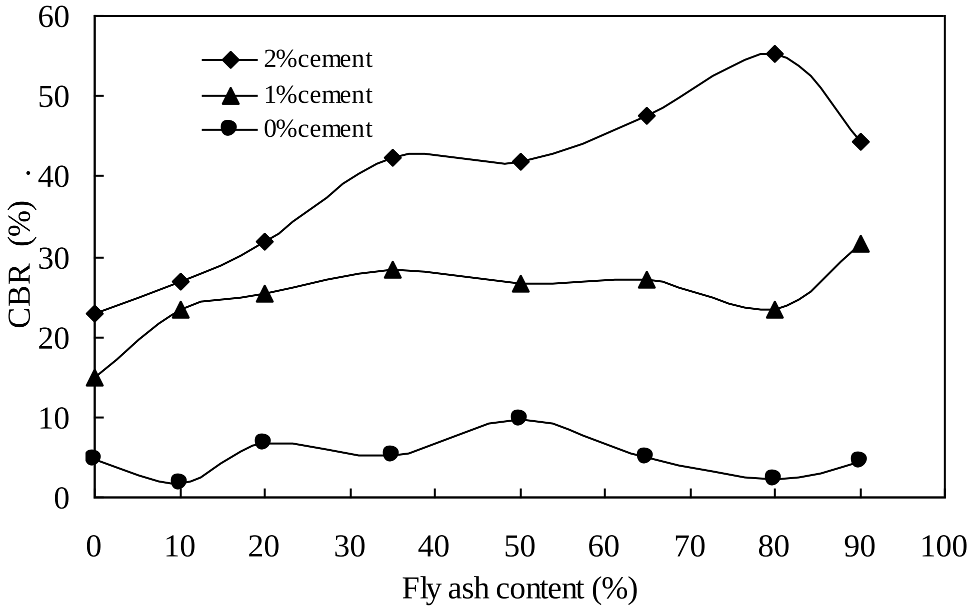


Fig. 6.15 Soaked CBR values of all RS-fly ash mixes with varying cement contents

6.3.4 CBR of sand-fly ash-cement mixes

Figures 6.16 to 6.22 show variations of CBR values after the addition of cement to various sand-fly ash mixes for both unsoaked and soaked conditions. It is observed that the addition of cement to the sand leads to a significant improvement in soaked

TH-1088_AKALITA

CBR, but the trend for unsoaked CBR is irregular. The CBR values of all the sand mixes are presented in Table 6.3.

The addition of 2% cement to sand has resulted in CBR of 27.40% under soaked condition, whereas the CBR for sand alone is 11.09%. The CBR values of all the sand-fly ash mixes with the addition of 1% cement content are greater than that of sand-2% cement mix. It shows that the cement content in the mixes can be reduced by adding more amount of fly ash while still achieving significant improvement in CBR values. This can lead to an effective cost reduction.

Figs. 6.23 and 6.24 depict soaked CBR values of all sand-fly ash-cement mixes. It is observed that BS+50FA and BS+65FA mixes show lower CBR values than the other mixes with lower fly ash content (BS+20FA, BS+35FA and BS+20FA mixees), but after the addition of cement, these two mixes (BS+50FA and BS+65FA) show higher CBR values. It is also noted that for all mixes, the increase in CBR is marginal when cement content is raised from 1% to 2%. By adopting lower cement content and by adding more amount of fly ash to the sand, there is significant enhancement in soaked CBR values, which will be economical and will also lead to greater utilization of fly ash. Hence, proper proportions of these sand-fly ash-cement mixes improve the CBR values considerably.

TH-1088_AKALITA

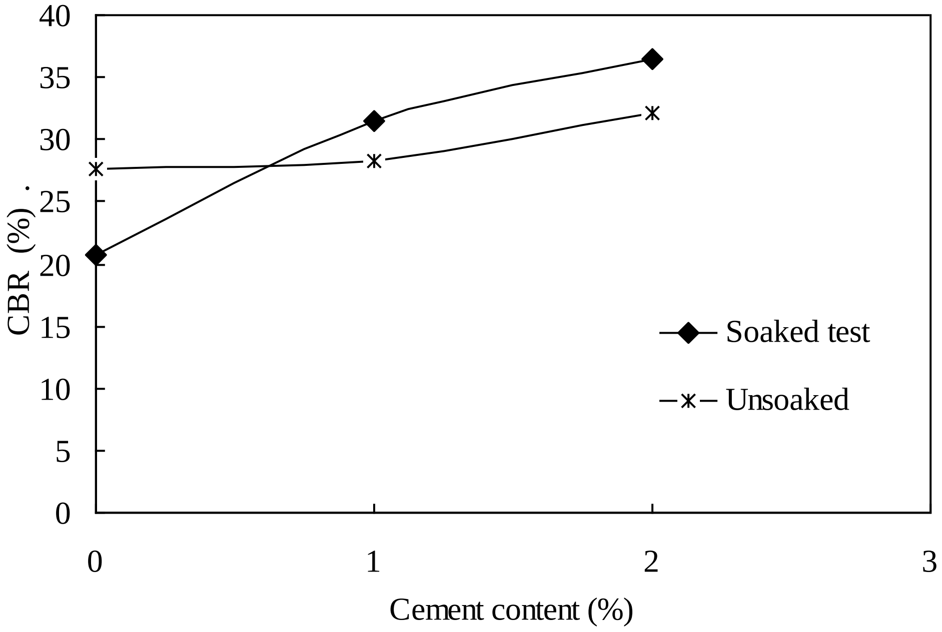


Fig. 6.16 Comparison between unsoaked and soaked CBR values of BS-10FA-cement mixes

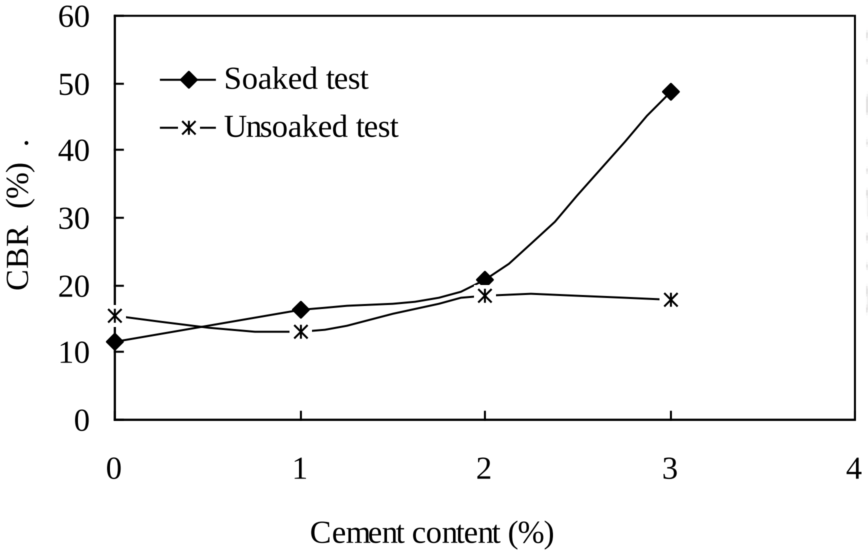


Fig. 6.17 Comparison between unsoaked and soaked CBR values of BS-20FA-cement mixes

TH-1088_AKALITA

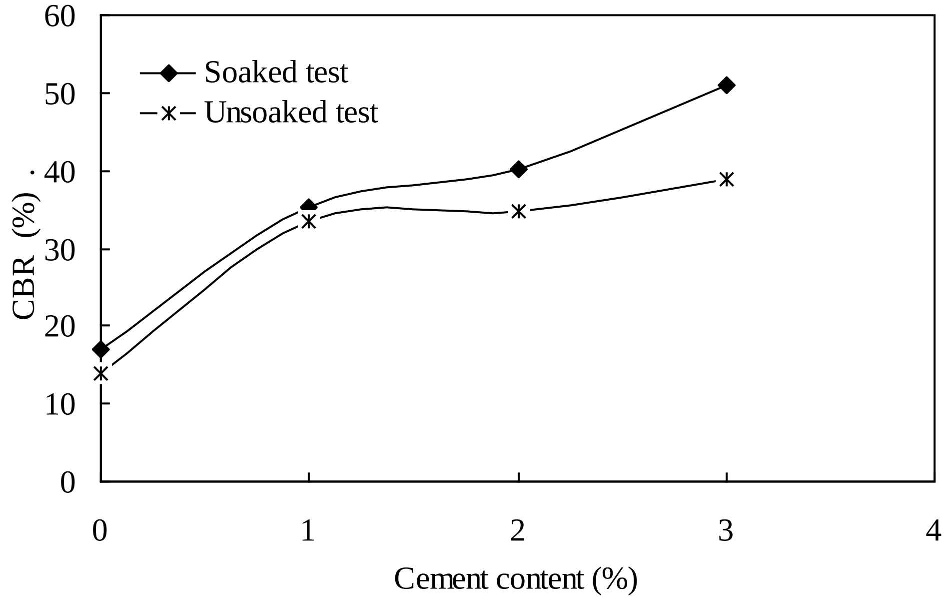


Fig. 6.18 Comparison between unsoaked and soaked CBR values of BS-35FA-cement mixes

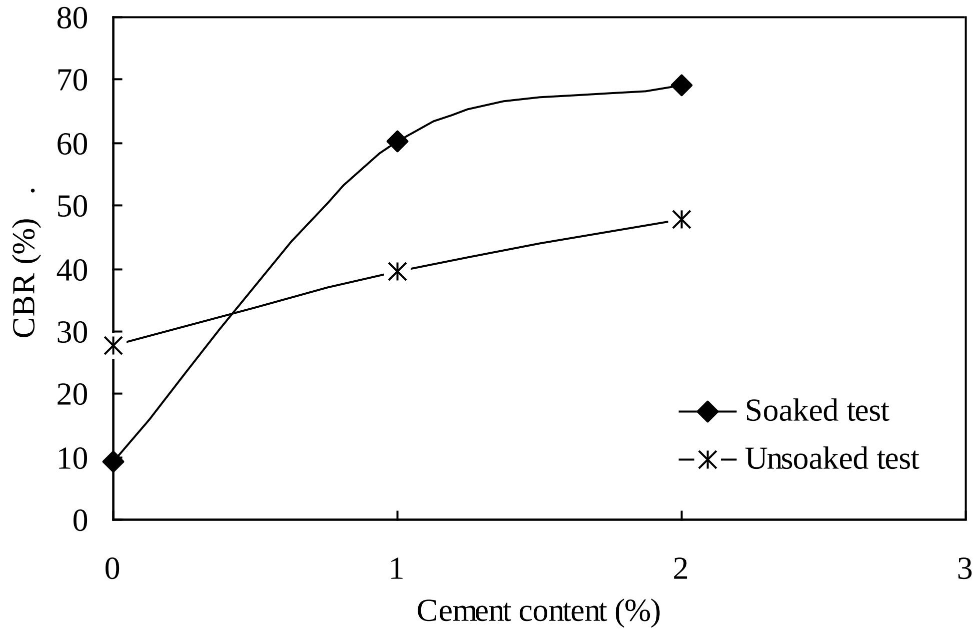


Fig. 6.19 Comparison between unsoaked and soaked CBR values of BS-50FA-cement mixes

TH-1088_AKALITA

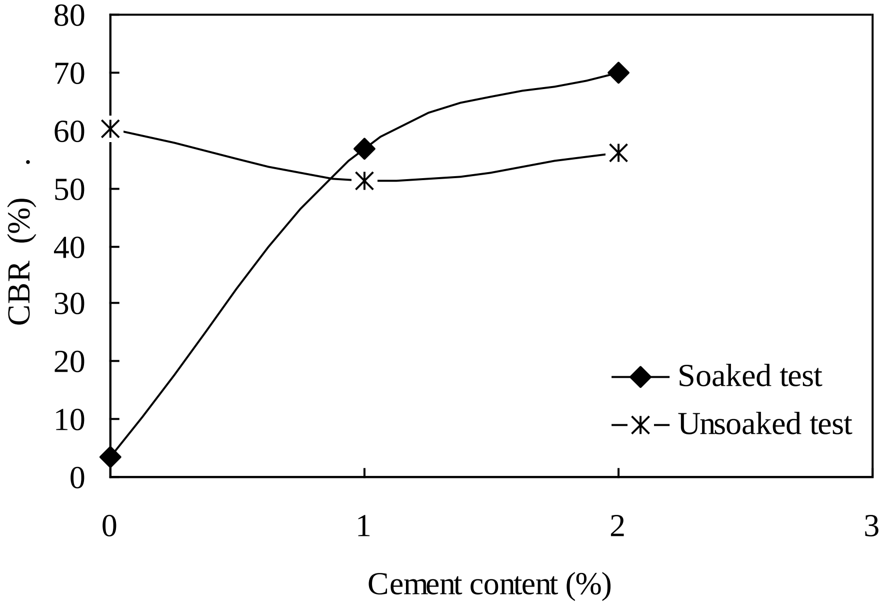


Fig. 6.20 Comparison between unsoaked and soaked CBR values of BS-65FA-cement mixes

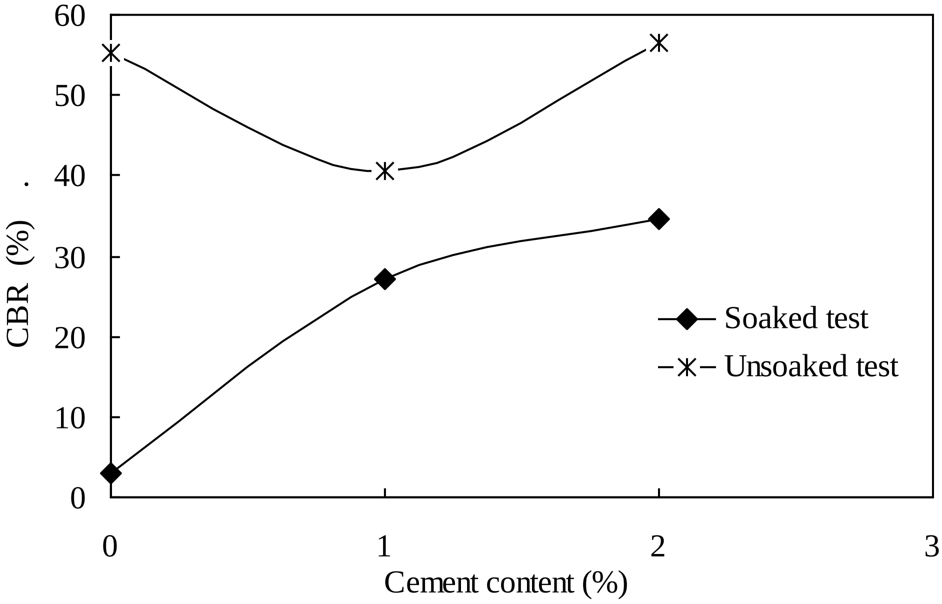


Fig. 6.21 Comparison between unsoaked and soaked CBR values of BS-80FA-cement mixes

TH-1088_AKALITA

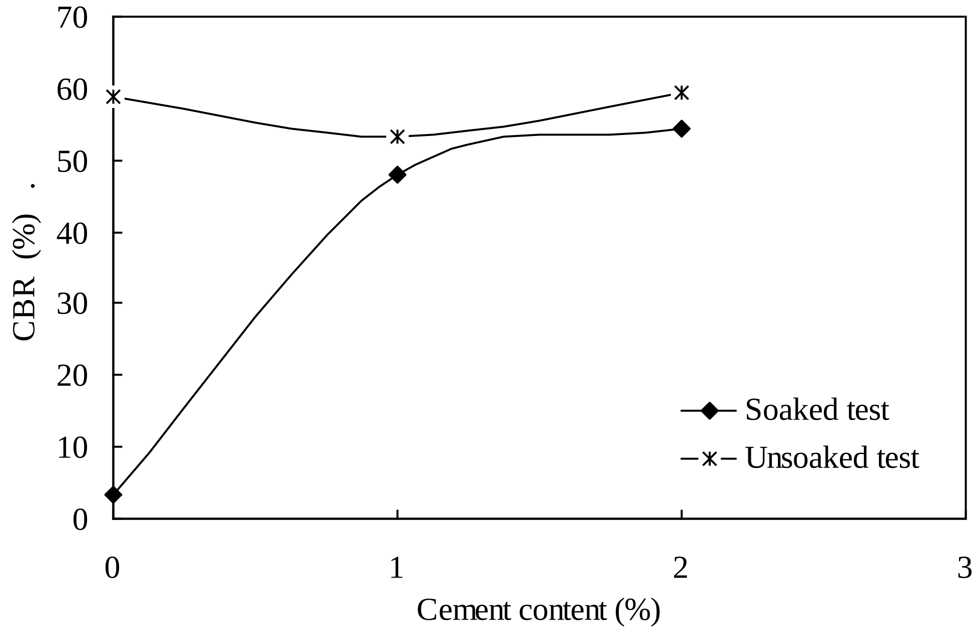


Fig. 6.22 Comparison between unsoaked and soaked CBR values of BS-90FA-cement mixes

TH-1088_AKALITA

Table 6.3: CBR values of all sand mixes

Mix	Unsoaked CBR (%)	Soaked CBR (%)
BS	19.1	11.0
BS+1C	17.6	23.2
BS+2C	16.4	27.4
BS+3C	11.8	34.3
BS+10FA	27.7	20.7
BS+20FA	15.4	11.6
BS+35FA	13.8	17.0
BS+50FA	27.6	9.3
BS+65FA	60.1	2.3
BS+80FA	55.1	2.8
BS+90FA	58.8	3.3
BS+10FA+1C	28.2	31.4
BS+10FA+2C	32.1	36.5
BS+20FA+1C	13.2	16.4
BS+20FA+2C	17.7	20.6
BS+20FA+3C	19.8	48.8
BS+35FA+1C	31.4	35.2
BS+35FA+2C	34.6	40.1
BS+35FA+3C	50.8	50.8
BS+50FA+1C	39.6	60.3
BS+50FA+2C	47.7	69.2
BS+65FA+1C	51.3	56.9
BS+65FA+2C	56.2	69.7
BS+80FA+1C	40.4	27.2
BS+80FA+2C	56.4	34.7
BS+90FA+1C	53.3	48.0
BS+90FA+2C	59.3	54.2

TH-1088_AKALITA

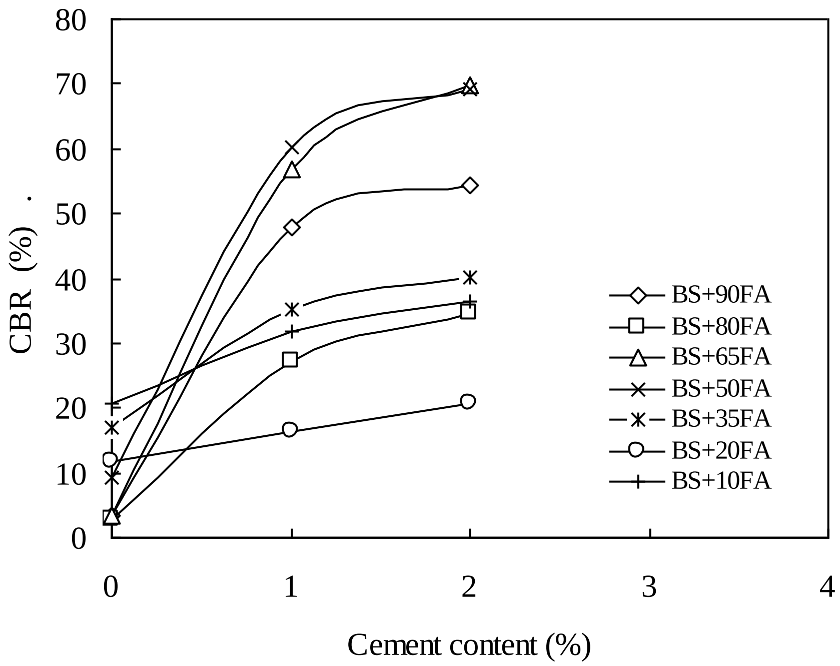


Fig. 6.23 Soaked CBR values of all BS-fly ash-cement mixes

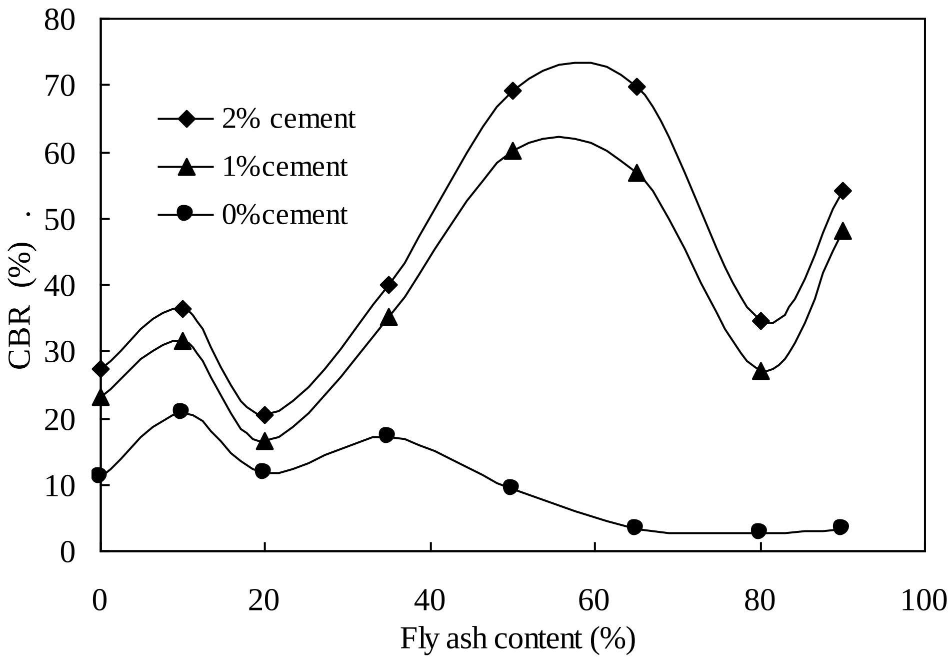


Fig. 6.24 Soaked CBR values of all BS-fly ash mixes with varying cement contents

TH-1088_AKALITA

6.3.5 Comparison between CBR of both soil-fly ash-cement mixes

Figs. 6.25 to 6.27 show comparisons of soaked CBR values of RS-fly ash and BS-fly ash mixes with 0%, 1% and 2% cement contents, respectively. From Fig.6.25, it is observed that when no cement is added, sand-fly ash mixes show higher soaked CBR value than red soil-fly ash mixes up to 50% fly ash content, after which both the soil mixes have comparable CBR values.

From Fig. 6.26, it is seen that BS-fly ash-1% cement mixes show higher soaked CBR than RS-fly ash-1% cement mixes for all the mixes except at 20% fly ash content. Similarly from Fig. 6.27, it is noted that BS-fly ash-2% cement mixes show higher soaked CBR than RS-fly ash-2% cement mixes for all the mixes, except at 20% and 80% fly ash contents.

When cement is added, sand-fly ash mixes show much higher soaked CBR values than red soil-fly ash mixes at fly ash contents of 50% and 65%.

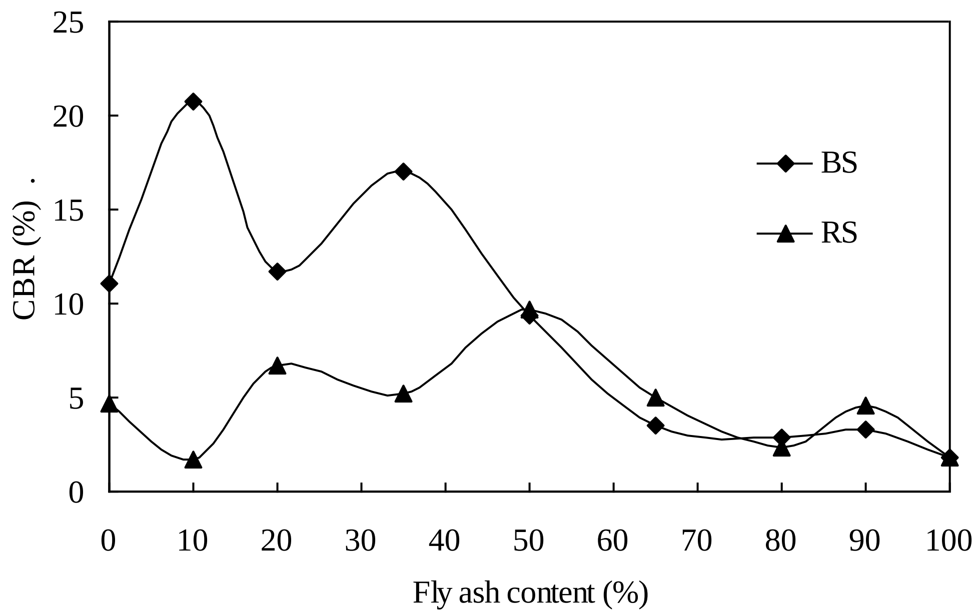


Fig. 6.25 Comparison of soaked CBR values of BS-fly ash and RS-fly ash mixes with 0% cement content

TH-1088_AKALITA

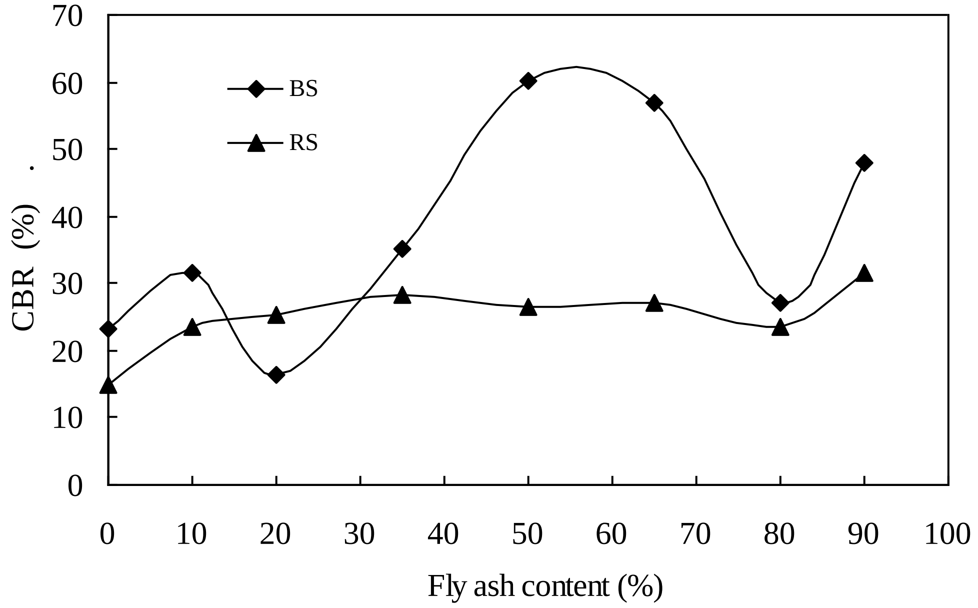


Fig. 6.26 Comparison of soaked CBR values of BS-fly ash and RS-fly ash mixes with 1% cement content

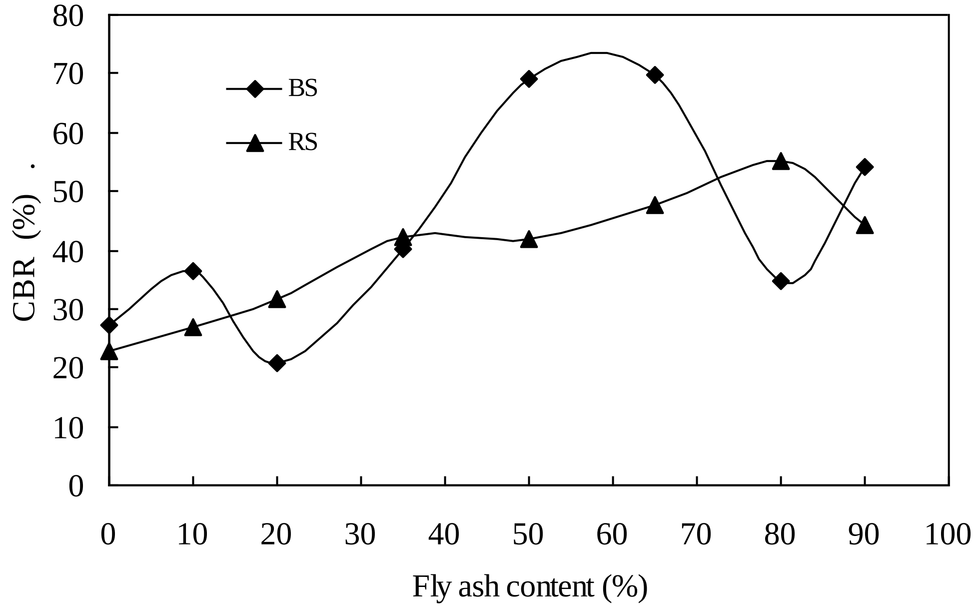


Fig. 6.27 Comparison of soaked CBR values of BS-fly ash and RS-fly ash mixes with 2% cement content

6.4 Conclusions

Only specific proportions of fly ash added to either soil can enhance the CBR values considerably. There are two optimum fly ash contents (20% and 50%) at which red soil-fly ash mixes exhibit higher CBR values under soaked conditions. Similarly,

TH-1088_AKALITA

there are two optimum fly ash contents (10% and 35%) at which sand-fly ash mixes show higher soaked CBR values. When the fly ash addition is less than 50%, soaked CBR values of all red soil-fly ash mixes are lower than those of sand-fly ash mixes.

The soaked CBR of the soil-fly ash mixes improve on further addition of a small amount of cement. Sand-fly ash-cement mixes show much higher soaked CBR values than red soil-fly ash-cement mixes at fly ash contents of 50% and 65%.

Minimum soaked CBR values of 6% and 20% are necessary for use in the subgrade and subbase layers of low-volume flexible pavements. With no cement added, red soil-fly ash mixes with 20% or 50% fly ash content and sand-fly ash mixes up to 50% fly ash content may be used for subgrade layer. With 1% cement added, both the soil-fly ash mixes with 50% fly ash content can be considered for subbase layer.

Further, the strength characteristics of the compacted specimens, cured for longer periods beyond 4 days, need to be determined through unconfined compression and triaxial tests.

TH-1088_AKALITA

CHAPTER – 7
UNCONFINED COMPRESSION TESTS ON SOIL-FLY ASH-CEMENT MIXES

7.1 Introduction

This test is one of the most common tests used to study the strength characteristics of soil mixes, and to evaluate the degree of improvement of treated soils. Although the test conditions do not simulate actual field conditions, the test results can be used for cases of low confinement. Kaniraj and Havanagi (1999b) added cement to stabilize soil-fly ash mixes and established correlations for unconfined compressive strength and secant modulus as functions of curing period, fly ash content, and cement content. Schnaid et al. (2001) studied the stress-strain-strength behaviour of an artificially cemented sandy soil produced through the addition of cement. Lo and Wardani (2002) conducted an experimental work to study the strength and dilatancy of a silt stabilized with the addition of cement-fly ash mixes in a slurry form. Kaniraj and Gayathri (2003) conducted a laboratory experimental study to investigate the effect of cement content, curing period, and curing conditions on the development of the strength of stabilized Class F fly ashes with reference to their use as pavement base courses.

The literature review indicates that the rate of strength development of amended soils and the level of such strength depend on several factors including: type of clay minerals present in the soil, type of fly ash, amount of replacement of soil with fly ash, nature and amount of cement, ambient temperature, and curing environment. An extensive unconfined compression testing programme was carried out on the soil, fly ash and their various mix proportions with and without cement in order to examine the effect of different fractions on the material behaviour with curing.

TH-1088_AKALITA

7.2 Test Programme

Table 7.1 summarizes the various mixes used in the unconfined compression test programme. To study the effect of pozzolanic reactions on the compressive strength of the red soil and sand when blended with different fly ash and cement contents, the specimens were cured up to 0, 3, 7, 14 and 28 days. Two specimens were tested for each set of variables. Tests were performed on all specimens using a strain rate of 1.25 mm/min. Saturation type is moist and test type is undrained. The load and deformation readings were taken up to the failure of the specimens. The average of peak stresses of the two specimens is taken as the unconfined compressive strength (UCS). The failure strain reported is also the average of the two specimens.

7.3 Results and Discussion

7.3.1 Compressive strength of red soil-fly ash mixes

Table 7.2 lists the UCS values and failure strains of the red soil, sand and fly ash specimens compacted at respective OMC values. The UCS of the specimens is found to increase with curing period. It is observed that the sand exhibits a much lower UCS compared to the red soil or fly ash. The UCS of the sand specimens increases from 2.99 to 3.99 kPa over a curing period of 28 days. For the same curing period, the red soil and fly ash specimens show increase of UCS from 69.0 to 158.37 kPa and from 39.52 to 59.55 kPa, respectively. Dry compacted fly ash does not have any UCS as it has no cohesion. Partly saturated compacted fly ash exhibits some UCS when tested, on account of apparent cohesion induced by capillary stresses. For a given dry unit weight, the apparent cohesion increases with the degree of saturation and reaches a maximum. Additional saturation reduces the apparent cohesion. The UCS of the fly ash specimens is attributed to the combined effect of capillary stresses and self-hardening pozzolanic action.

Table 7.1: Composition of various mixes for unconfined compression tests

Soil	Fly ash	Soil-fly ash mixes	Soil-cement mixes	Soil-fly ash-cement mixes
RS	FA	RS+10FA	RS+1C	RS+20FA+1C
		RS+20FA	RS+2C	RS+20FA+2C
		RS+35FA	RS+3C	RS+20FA+3C
		RS+50FA	RS+4C	RS+20FA+4C
		RS+65FA		
		RS+80FA		RS+35FA+1C
		RS+90FA		RS+35FA+2C
				RS+35FA+3C
				RS+35FA+4C
				RS+50FA+1C
				RS+50FA+2C
				RS+50FA+3C
				RS+50FA+4C
				RS+65FA+1C
				RS+65FA+2C
				RS+80FA+1C
				RS+80FA+2C
BS		BS+10FA	BS+1C	BS+20FA+1C
		BS+20FA	BS+2C	BS+20FA+2C
		BS+35FA	BS+3C	BS+20FA+3C
		BS+50FA	BS+4C	BS+20FA+4C
		BS+65FA		
		BS+80FA		BS+35FA+1C
		BS+90FA		BS+35FA+2C
				BS+35FA+3C
				BS+35FA+4C
				BS+50FA+1C
				BS+50FA+2C
				BS+50FA+3C
				BS+50FA+4C
				BS+65FA+1C
				BS+65FA+2C
				BS+80FA+1C
				BS+80FA+2C

The strength of the unblended red soil also increases with curing period, and this is attributed to the aggregation of the soil particles due to its higher content of iron and aluminium oxides.

The influences of fly ash content and curing period on the unconfined compressive strength of red soil-fly ash mixes are shown in Figs. 7.1(a-b). The UCS values and the corresponding failure strains are tabulated in Table 7.3.

It is noted that there is gain in UCS only for mixes with fly ash content up to 50%. For these mixes only, one observes that the UCS steadily increases with curing period. At any curing period, the highest UCS values are observed at 35% fly ash content followed by 20% fly ash content (i.e. RS+35FA and RS+20FA mixes). There is no benefit of adding higher fly ash contents above 50%, and the UCS values of these mixes remain less than that of the red soil alone. As already shown in Table 7.2, the UCS of compacted red soil is greater than that of fly ash alone.

As the fly ash content in mix increases, the non-cohesive fraction in the mix decreases which reduces the cohesion component of the strength. It can also be noted that the loss of strength beyond 50% fly ash addition is nearly proportional to the amount of fly ash added to the soil (Fig. 7.1(b)), and this can be attributed to the change in gradation/texture of the soil.

The failure strains of different mixes (10FA, 20FA, 35FA, 50FA, 65FA, 80FA, and 90FA contents) for all curing periods remain within a narrow range varying between 1.53 to 2.4%, 2.14 to 4.74%, 2.51 to 4.42%, 2.5 to 5.42%, 2.28 to 3.8%, 1.81 to 2.54% and 1.55 to 2.25%, respectively (Table 7.3).

Figure 7.1(c) shows failure patterns of UC test specimens for red soil with different fly ash contents. Clear crack patterns at failure are observed in the three specimens. When the specimens are subjected to vertical axial compression loading, bulging takes place showing a plastic or barrelled failure with an increase of the cross-sectional area. The red soil is a clayey silt soil having intermediate plasticity.

Table 7.2: Average unconfined compressive strength and failure strain of sand, red soil and fly ash specimens compacted at MDD

Curing period (Days)	Red soil (RS)		Sand (BS)		Fly ash (FA)	
	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)
0	69.0	3.11	2.99	0.65	39.52	1.18
3	105.56	4.33	3.94	1.84	42.90	2.36
7	126.68	3.14	3.95	1.68	48.19	1.55
14	131.88	6.09	3.97	0.78	49.30	1.71
28	158.37	2.86	3.99	0.52	59.55	1.46

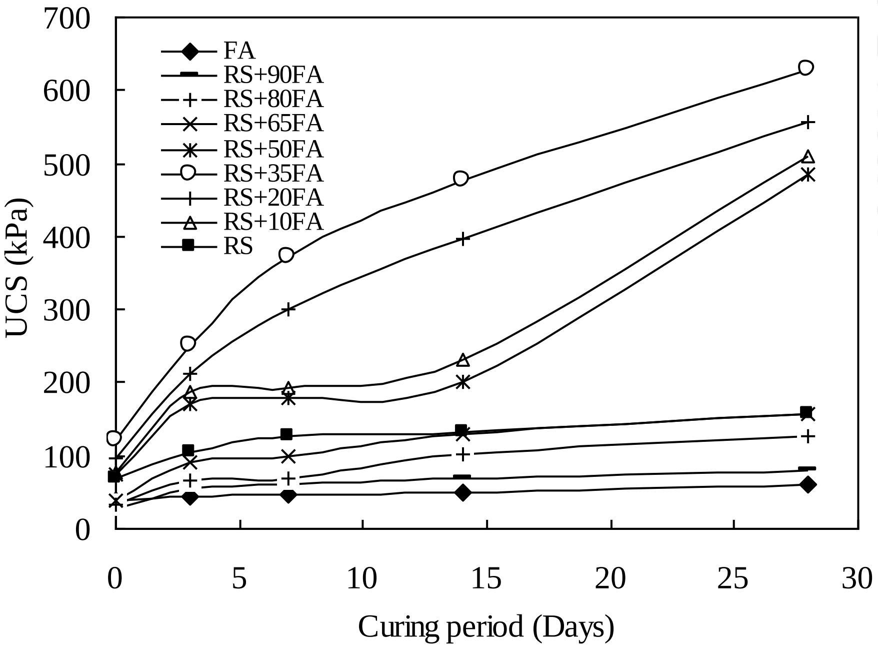


Fig. 7.1(a) Variation of average UCS of red soil-fly ash mixes with curing period

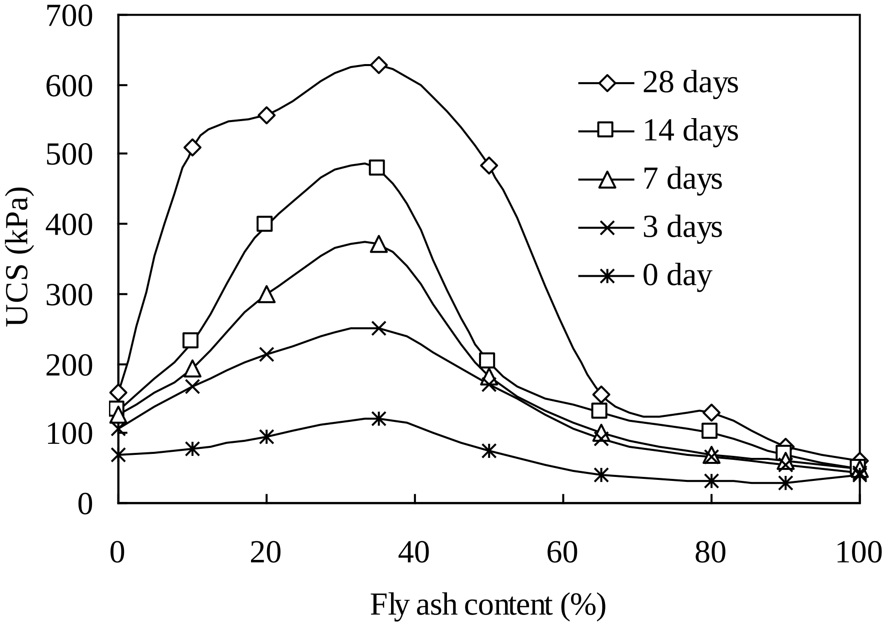


Fig. 7.1(b) Variation of average UCS of red soil-fly ash mixes at different curing periods

RS+20FA RS+50FA RS+90FA

Fig. 7.1(c) Failure patterns of red soil-fly ash specimens

Table 7.3: Average unconfined compressive strength and failure strain of red soil-fly ash mixes with no cement addition

Curing period (Days)	RS		RS+10FA		RS+20FA		RS+35FA		RS+50FA		RS+65FA		RS+80FA		RS+90FA		FA	
	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS kPa)	Strain (%)
0	69.0	3.11	77.91	1.82	95.37	3.28	121.68	3.15	74.89	2.5	32.97	3.8	32.42	1.81	28.73	1.55	39.52	1.18
3	105.56	4.33	186.71	1.92	212.12	4.74	250.48	3.04	169.92	2.85	65.75	2.28	65.75	1.82	55.10	1.84	42.90	2.36
7	126.68	3.14	194.11	2.0	300.06	2.25	373.01	3.07	180.37	5.42	69.95	3.72	69.90	2.54	60.97	2.25	48.19	1.55
14	131.88	6.09	230.43	2.4	396.34	3.06	477.16	4.42	200.35	2.76	128.56	2.49	101.61	2.44	70.00	1.86	49.30	1.71
28	158.37	2.86	510.44	1.53	556.35	2.14	628.82	2.51	485.08	2.88	156.91	3.14	128.82	2.46	79.70	1.58	59.55	1.46

7.3.2 Compressive strength of red soil-cement mixes

Figure 7.2 depicts variation of unconfined compressive strength with curing period for RS-cement mixes. Table 7.4 summarizes the UCS values and their corresponding strains.

The results show that the addition of cement significantly improves the strength and stiffness properties of red soil. At all cement contents, the UCS increases with curing period. Strength development is due to the formation of hydration and pozzolanic reaction products, which bind the soil particles.

From a comparison of Tables 7.3 and 7.4, it is noted that minimum 2% cement is to be added to the red soil (i.e. RS+2C mix) to obtain UCS values higher than the RS+35FA mix at all curing periods. At 28 days curing, the respective UCS values are 631.06 and 628.82 kPa.

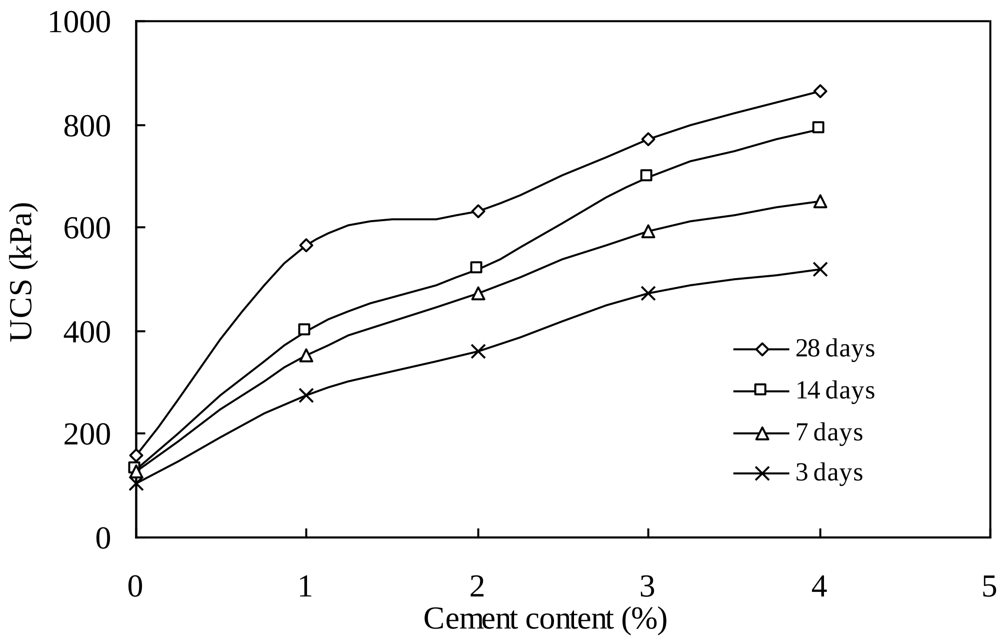


Fig. 7.2 Variation of average UCS of red soil-cement mixes at different curing periods

TH-1088_AKALITA

Table 7.4: Average unconfined compressive strength and failure strain of red soil-cement mixes

Curing period (Days)	RS		RS+1C		RS+2C		RS+3C		RS+4C	
	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)
3	105.56	4.33	274.35	2.50	362.27	2.08	473.35	2.30	519.37	2.15
7	126.68	3.14	352.57	2.32	471.88	2.13	592.15	2.14	652.76	2.47
14	131.88	6.09	397.69	3.67	518.69	4.13	696.50	3.56	791.37	3.51
28	158.37	2.86	567.40	2.50	631.06	1.90	773.13	2.09	866.12	2.55

7.3.3 Compressive strength of red soil-fly ash-cement mixes

Figures 7.3(a-d) depict the variations of unconfined compressive strength of various red soil-fly ash-cement mixes at different curing periods. The UCS values are summarized in Tables 7.5 to 7.8, respectively.

From the figures, one observes that RS-35FA-cement mixes show maximum UCS values followed by RS-20FA-cement mixes. Minimum UCS values are shown by RS-80FA-cement mixes. The strength and stiffness of all mixes increase significantly compared to the red soil alone or the red soil-fly ash mixes.

From Tables 7.5 and 7.8, it is noted that the addition of only 1% cement to red soil-fly ash mixes increases the strength substantially when curing period is increased from 3 to 28 days. The UCS of the RS+35FA+1C mix shows increase from 448.26 kPa to 710.4 kPa. Over the same curing period, the RS+20FA+1C mix shows increase from 348.8 to 637.57 kPa, whereas the UCS of RS+80FA+1C mix increases from 134.55 to 348.8 kPa.

Figures 7.4(a-d) depict the variations of UCS with curing period for red soil-fly ash mixes with increasing cement contents from 1% to 4%, respectively. The gain in strength is dependent on the fly ash and cement contents. Similar finding was reported by Kaniraj

TH-1088_AKALITA

and Havanagi (1999b). One notes that that strength is still the highest at 35% fly ash content at all cement contents.

Sufficient strength can be achieved with the addition of just a small percentage of cement at 1%. At 28 days, the UCS value of 710.4 kPa for the RS+35FA+1C mix is greater than that of the RS+2C mix (631.06 kPa). Therefore, reducing the quantity of cement by adding more fly ash can satisfy strength requirements while being cost-effective also. Addition of 2 to 4% cement to red soil-fly ash mixes may not be economical.

Figure 7.5 shows the failure patterns of UC test specimens for RS+50FA+2C and RS+50FA+3C mixes. The specimens exhibit shear planes indicating a tendency towards brittle failure. The addition of cement to the red soil-fly ash mixes changes the failure mode from plastic to brittle.

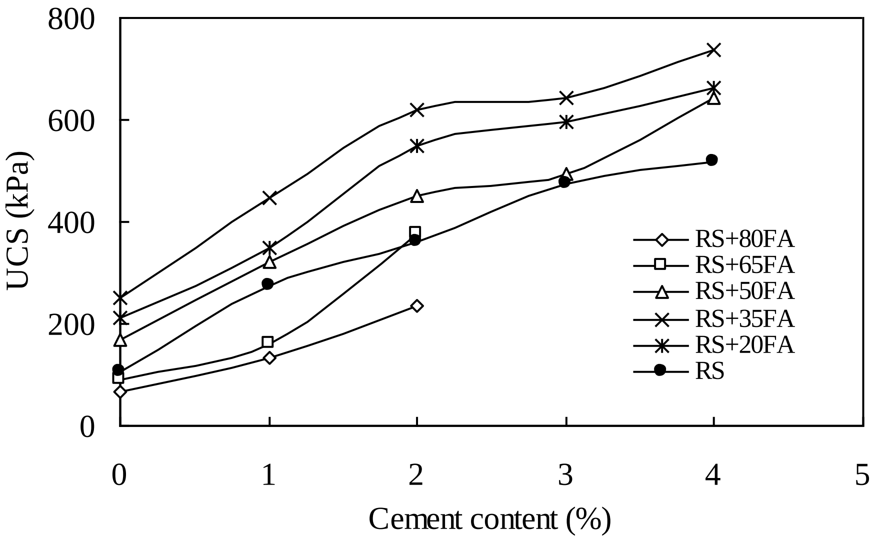


Fig. 7.3(a) Variation of average UCS of red soil-fly ash mixes with cement content (3 days curing)

TH-1088_AKALITA

Table 7.5: Average UCS of red soil-fly ash-cement mixes (3 days curing)

Cement content (%)	Unconfined compressive strength (kPa)					
	RS	RS+20FA	RS+35FA	RS+50FA	RS+65FA	RS+80FA
0	105.56	212.79	250.48	169.92	90.91	65.75
1	274.35	348.8	448.26	320.41	161.47	134.55
2	362.27	550.82	619.08	450.93	377.55	236.97
3	473.35	595.88	644.12	493.91		
4	519.37	664.63	737.97	645.02		

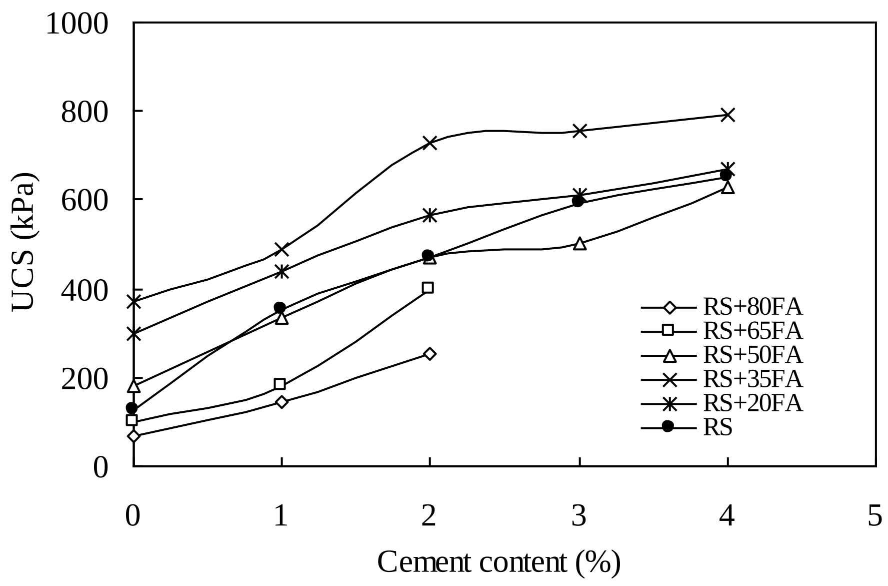


Fig. 7.3(b) Variation of average UCS of red soil-fly ash mixes with cement content (7 days curing)

Table 7.6: Average UCS of red soil-fly ash-cement mixes (7 days curing)

Cement content (%)	Unconfined compressive strength (kPa)					
	RS	RS+20FA	RS+35FA	RS+50FA	RS+65FA	RS+80FA
0	126.68	300.06	373.01	180.37	99.95	69.9
1	352.57	439.97	490.78	335.12	182	145.66
2	471.88	563.56	728.27	472.11	396.3	253.28
3	592.15	612.13	757	500.74		
4	652.76	670.84	789.82	630.73		

TH-1088_AKALITA

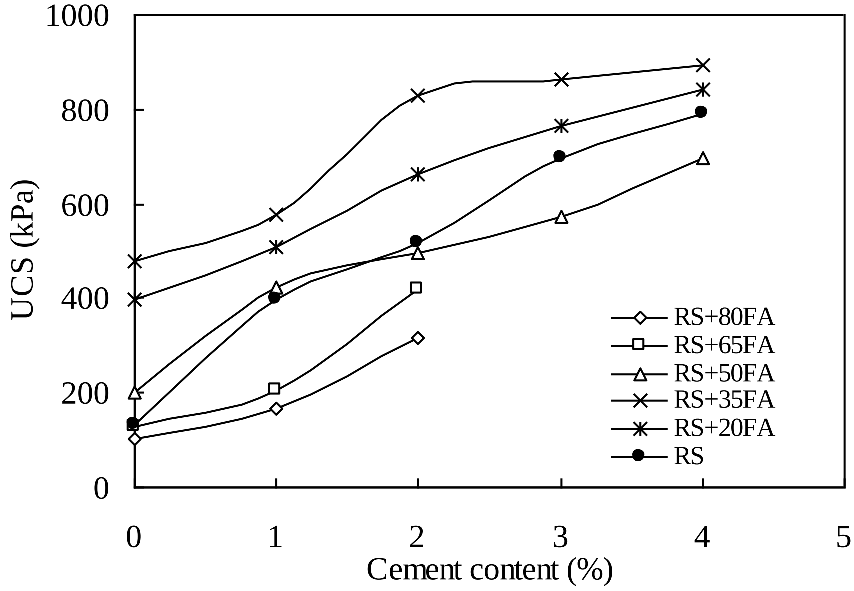


Fig. 7.3(c) Variation of average UCS of red soil-fly ash mixes with cement content (14 days curing)

Table 7.7: Average UCS of red soil-fly ash-cement mixes (14 days curing)

Cement content (%)	Unconfined compressive strength (kPa)					
	RS	RS+20FA	RS+35FA	RS+50FA	RS+65FA	RS+80FA
0	131.88	396.34	477.16	200.35	128.56	101.61
1	397.69	510.15	578	423.46	203.71	167.32
2	518.69	663.95	828.64	494.65	420.76	315.55
3	696.5	763.03	861.26	571.58		
4	791.37	840.62	891.06	697.71		

TH-1088_AKALITA

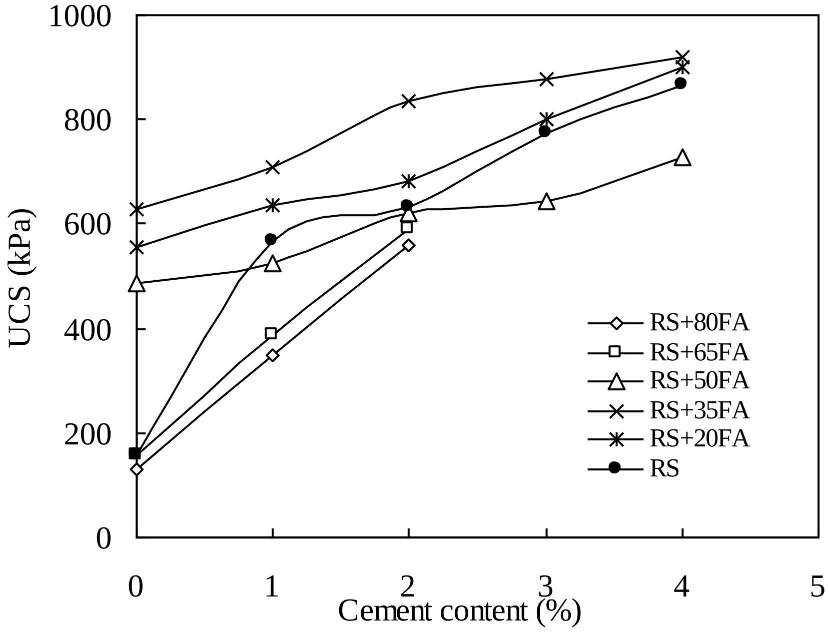


Fig. 7.3(d) Variation of average UCS of red soil-fly ash mixes with cement content (28 days curing)

Table 7.8: Average UCS of red soil-fly ash-cement mixes (28 days curing)

Cement content (%)	Unconfined compressive strength (kPa)					
	RS	RS+20FA	RS+35FA	RS+50FA	RS+65FA	RS+80FA
0	158.37	556.35	628.14	485.35	156.91	128.82
1	567.4	637.57	710.4	523.57	385.48	348.8
2	631.06	682.34	835.53	620.63	590.34	561.04
3	773.13	802.54	878.56	642.3		
4	866.12	900.98	918.53	728.91		

TH-1088_AKALITA

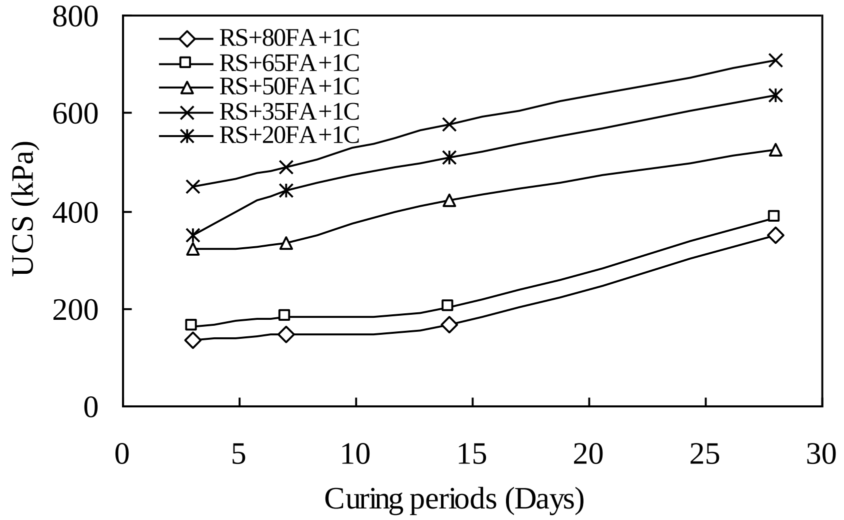


Fig. 7.4(a) Variation of average UCS of red soil-fly ash-1% cement mixes with curing period

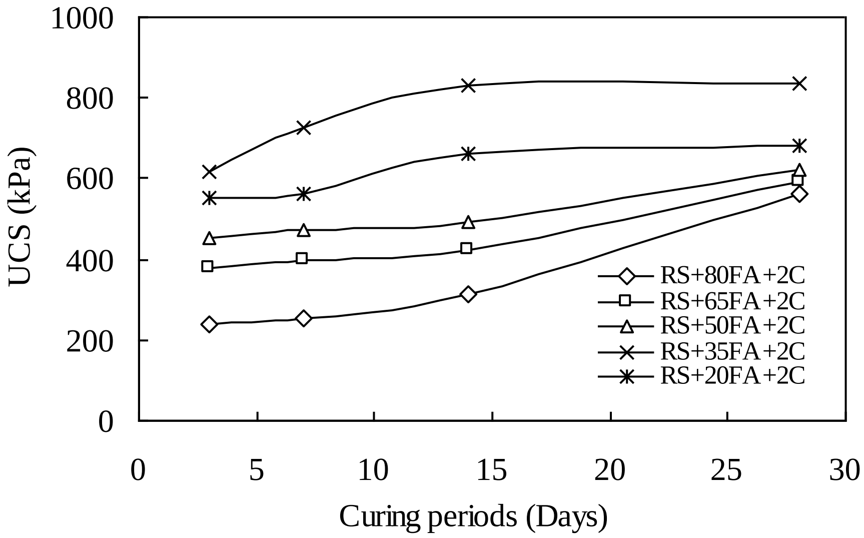


Fig. 7.4(b) Variation of average UCS of red soil-fly ash-2% cement mixes with curing period

TH-1088_AKALITA

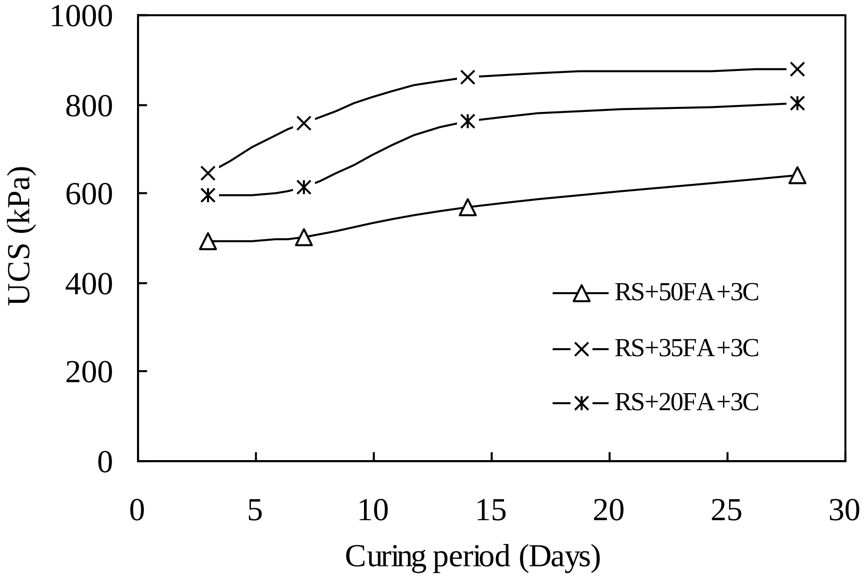


Fig. 7.4(c) Variation of average UCS of red soil-fly ash-3% cement mixes with curing period

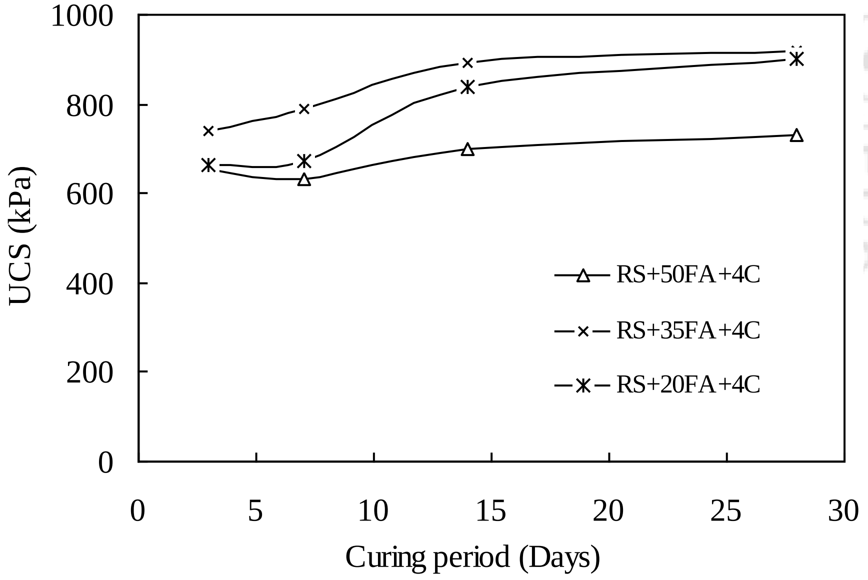


Fig. 7.4(d) Variation of average UCS of red soil-fly ash-4% cement mixes with curing period

TH-1088_AKALITA

RS+50FA+2C

RS+50FA+3C

Fig.7.5 Failure patterns of red soil-fly ash-cement specimens

7.3.4 Compressive strength of sand-fly ash mixes

The influences of fly ash content and curing period on the unconfined compressive strength of sand-fly ash mixes are depicted in Figs. 7.6(a-b). The UCS values along with failure strains are summarized in Table 7.9. One notes that the compressive strength of the mixes increases with fly ash content and also with the increase in curing period. This is due to the occurrence of time-dependent pozzolanic reactions. As already shown in Table 7.2, the UCS of compacted fly ash is greater than that of sand alone.

The pozzolanic reaction accelerates at a later stage of curing, and hence the gain in strength is significant after a relatively longer curing period, and similar behaviour was reported by Consoli et al. (2001). The unconfined compressive strength of the mixes steadily increases with the increase in the fly ash content as well as curing period.

From Table 7.9, it is observed that the BS+90FA and BS+10FA mixes show maximum and minimum UCS values, respectively. The UCS of BS+10FA mix increases from 3.96 to 5.87 kPa over a curing period of 28 days. For the same curing period, the BS+90FA mix shows increase from 49.1 to 167.84 kPa. The failure strains of different mixes (10FA, 20FA, 35FA, 50FA, 65FA, 80FA, and 90FA mixes) for different curing

TH-1088_AKALITA

periods remain within a narrow range varying between 1.05 to 2.85%, 0.91 to 1.44%, 1.17 to 1.97%, 2.07 to 2.5%, 1.27 to 2.26%, 1.43 to 2.2% and 1.64 to 2.56%, respectively.

Figure 7.6(c) shows the failure patterns of UC test specimens for sand with different fly ash contents. The sand alone has no stiffness, and it fails by bulging while subjected to compressive loads. There is no resistance developed against deformation in sand specimens and will undergo large deformation. From the figure, it can be seen that the addition of 20% fly ash to sand increases the stiffness marginally, and small cracks with more bulging effect are observed. The presence of higher fly ash content can bind the sand grains together as a result of mild pozzolanic reactions. Thus, BS+50FA mix sand-fly ash mix mobilizes resistance against deformation, and shows an inclined rupture plane with less deformation compared to BS+20FA mix. In the BS+90FA mix, the rupture plane is more clearly visible due to the higher amount of fly ash addition. This also indicates a tendency towards brittle behaviour.

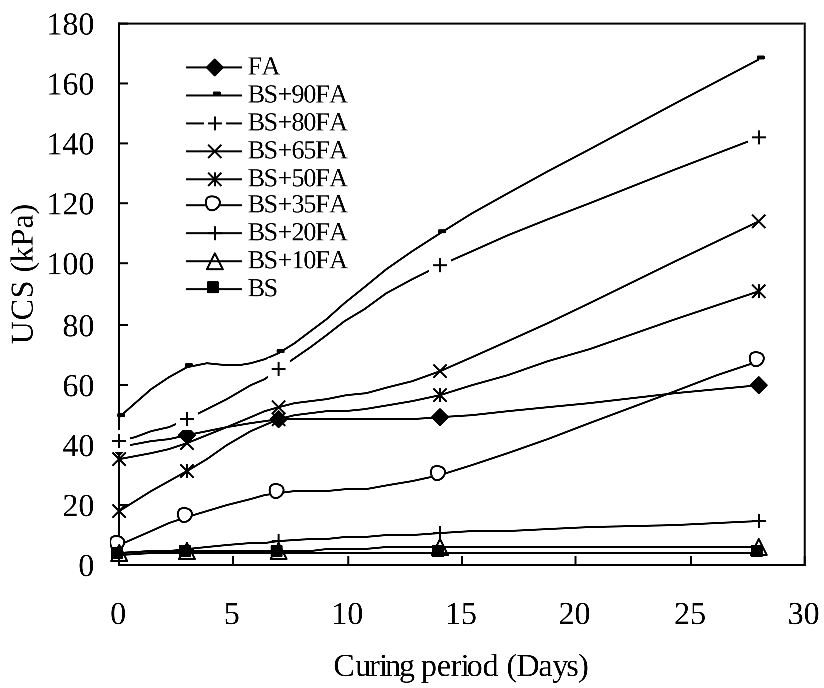


Fig. 7.6(a) Variation of average UCS of sand-fly ash mixes with curing period

TH-1088_AKALITA

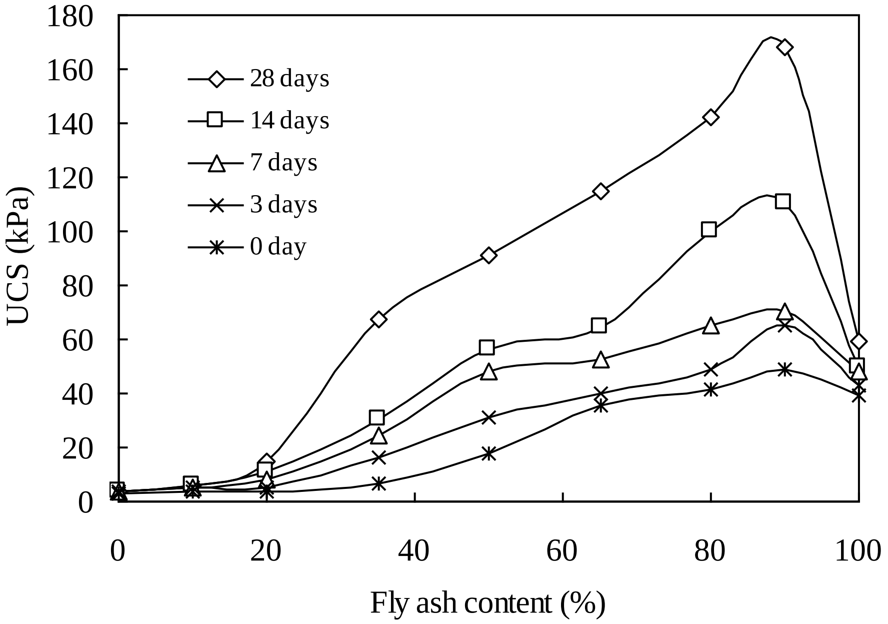


Fig. 7.6(b) Variation of average UCS of sand-fly ash mixes at different curing periods

.6(c) Failure pa fly ash specin

TH-1088_AKALITA

Table 7.9: Average unconfined compressive strength and failure strain of sand-fly ash mixes with no cement addition

Curing period (Days)	BS		BS+10FA		BS+20FA		BS+35FA		BS+50FA		BS+65FA		BS+80FA		BS+90FA		FA	
	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)
0	2.99	0.65	3.96	1.42	3.97	1.05	6.90	1.17	17.79	2.5	35.53	2.10	41.29	2.20	49.10	2.56	39.52	1.18
3	3.94	1.84	4.93	2.85	5.05	1.44	16.05	1.23	31.24	2.15	40.24	2.26	48.53	1.43	65.50	1.26	42.90	2.36
7	3.95	1.68	4.94	1.05	7.97	0.91	24.15	1.30	48.23	2.07	52.39	1.68	64.92	2.15	70.64	2.06	48.19	1.55
14	3.97	0.78	5.83	1.06	10.87	1.42	30.09	1.50	56.23	2.46	64.20	1.74	99.75	1.75	110.05	1.77	49.30	1.71
28	3.99	0.52	5.87	1.57	14.92	1.16	67.44	1.97	91.03	2.07	114.54	1.27	142.31	1.61	167.84	1.64	59.55	1.46

TH-1088_AKALITA

7.3.5 Compressive strength of sand-cement mixes

Figure 7.7 depicts variation of unconfined compressive strength with curing period for BS-cement mixes. The UCS values and corresponding strains are tabulated in Table 7.10. At all cement contents, the UCS increases with curing period.

From a comparison of Tables 7.9 and 7.10, it is noted that minimum 4% cement is to be added to the sand (i.e. BS+4C mix) to obtain UCS values higher than the BS+90FA mix. At 28 days curing, the respective UCS values are 208.61 and 167.84 kPa. This shows that the effect of adding only cement to the soil is less pronounced than that of adding only fly ash.

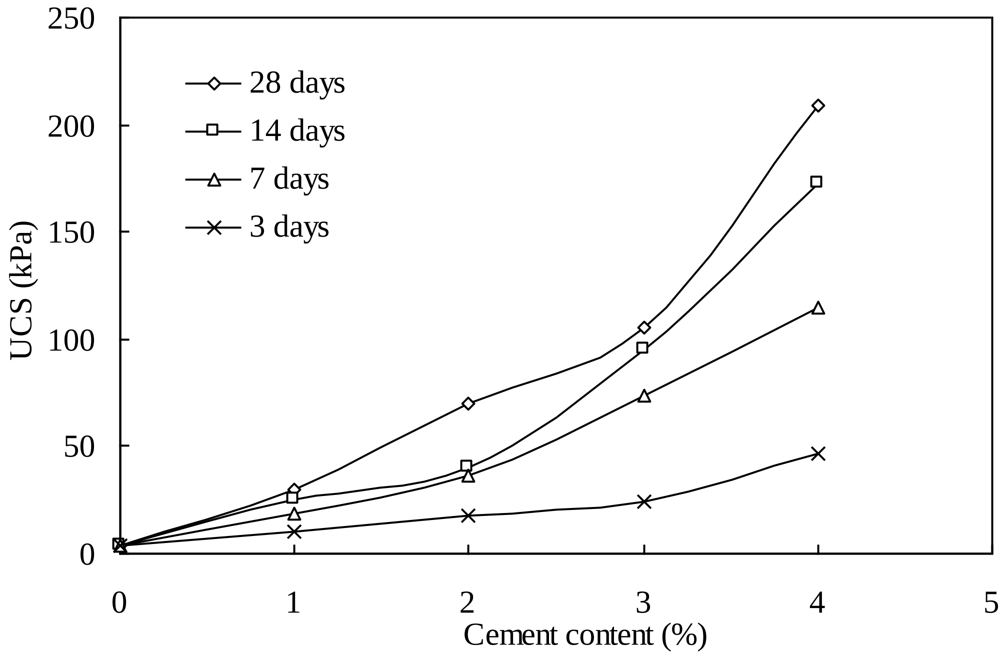


Fig. 7.7 Variation of average UCS of sand-cement mixes at different curing periods

TH-1088_AKALITA

Table 7.10: Average unconfined compressive strength and failure strain of sand-cement mixes

Curing period (Days)	BS		BS+1C		BS+2C		BS+3C		BS+4C	
	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)
3	3.94	1.84	9.92	1.28	17.87	1.23	23.93	0.78	46.76	1.10
7	3.95	1.68	18.98	0.58	35.99	0.48	73.66	0.98	114.39	1.05
14	3.97	0.78	24.96	0.65	39.86	0.84	95.25	1.09	172.86	1.18
28	3.99	0.52	29.88	0.90	69.74	0.90	105.40	1.29	208.61	1.17

7.3.6 Compressive strength of sand-fly ash-cement mixes

Figures 7.8(a-d) show the variations of unconfined compressive strength of various sand-fly ash-cement mixes at different curing periods. Tables 7.11 to 7.14 respectively summarize the UCS values. From the figures, one notes that when cement is added to any mix of sand-fly ash, the UCS is increased still further. The UCS is greater for higher fly ash content

From Tables 7.11 to 7.14, one observes that the addition of a small amount of cement (1% cement) to sand-fly ash mixes increases the strength considerably after curing. When the curing period is increased from 3 to 28 days, the UCS of BS+80FA+1C mix shows increase from 179.25 to 493.48 kPa. The corresponding UCS values for BS+20FA+1C are 61.12 and 101.79 kPa. The addition of cement in the sand-fly ash mixes increases the brittleness. It enhances the stiffness and peak strength.

Figures 7.9(a-d) depicts the variations of UCS with curing period for sand-fly ash mixes with increasing cement contents from 1% to 4%, respectively. As the fly ash content increases, the UCS also increases in all the curing periods.

Sufficient strength can be achieved with the addition of just a small percentage of cement at 1%. After 28 days curing, the UCS values of BS+65FA+1C and BS+80FA+1C mixes are 324.57 and 493.48 kPa, respectively. These values are greater than that of the

TH-1088_AKALITA

BS+4C mix (208.61 kPa). Therefore, reducing the quantity of cement by adding more fly ash can satisfy strength requirements. Addition of 2 to 4% cement to sand-fly ash mixes may not be economical. The aim is to utilize as much fly ash as possible, and at the same time the mix should also satisfy the strength criteria.

Figure 7.10 shows the failure pattern of BS+35FA+1C specimen. The failure plane is clearly visible indicating brittle failure. At higher fly ash and cement contents, tendency towards increased brittleness was observed.

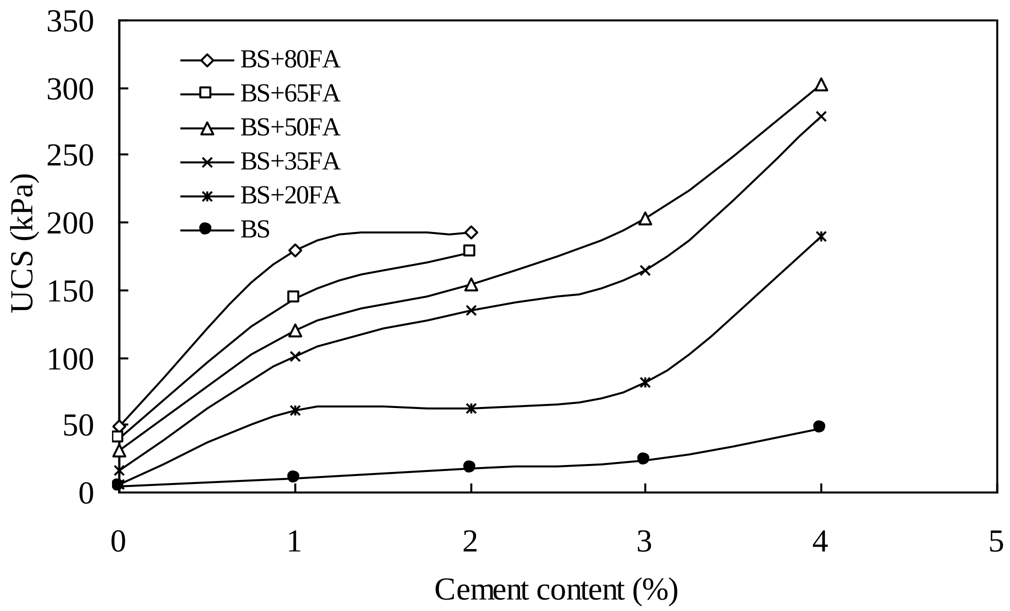


Fig. 7.8(a) Variation of average UCS of sand-fly ash mixes with cement content (3 days curing)

Table 7.11: Average UCS of sand-fly ash-cement mixes (3 days curing)

Cement content (%)	Unconfined compressive strength (kPa)					
	BS	BS+20FA	BS+35FA	BS+50FA	BS+65FA	BS+80FA
0	3.94	5.05	16.05	31.24	40.24	48.53
1	9.92	61.12	100.62	119.68	144.3	179.25
2	17.87	62.25	135.09	154.83	177.87	192.31
3	23.93	81.95	165.31	203.3		
4	46.76	190.1	279.37	303.26		

TH-1088_AKALITA

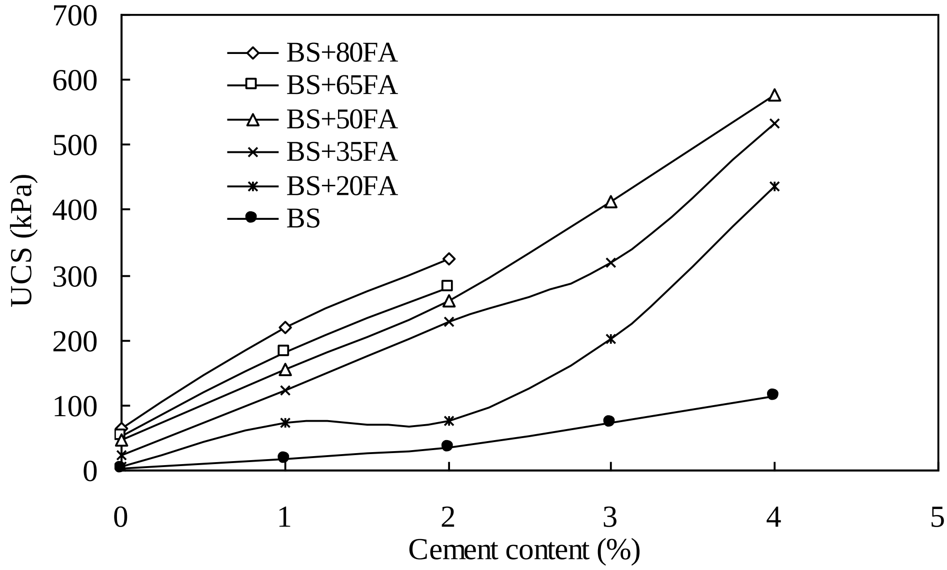


Fig. 7.8(b) Variation of average UCS of sand-fly ash mixes with cement content (7 days curing)

Table 7.12: Average UCS of sand-fly ash-cement mixes (7 days curing)

Cement content (%)	Unconfined compressive strength (kPa)					
	BS	BS+20FA	BS+35FA	BS+50FA	BS+65FA	BS+80FA
0	3.95	5.97	24.15	48.23	52.39	64.92
1	18.98	73.31	123.8	154.24	182.65	219.82
2	35.99	77.01	229	260.04	282.54	323.9
3	73.66	202.37	318.07	412.02		
4	114.39	435.8	532.57	577.26		

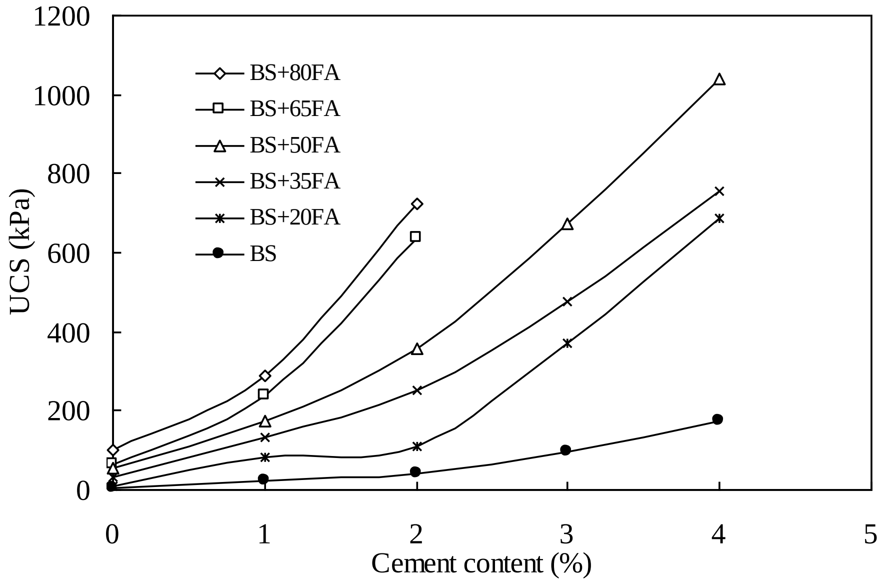


Fig. 7.8(c) Variation of average UCS of sand-fly ash mixes with cement content (14 days curing)

TH-1088_AKALITA

Table 7.13: Average UCS of sand-fly ash-cement mixes (14 days curing)

Cement Content (%)	Unconfined compressive strength (kPa)					
	BS	BS+20FA	BS+35FA	BS+50FA	BS+65FA	BS+80FA
0	3.97	10.87	30.09	56.23	64.2	99.75
1	24.96	81.1	134.19	173.31	238.42	287.4
2	39.86	112.1	253.79	358.67	635.92	723.71
3	95.25	369.11	474.44	673.89		
4	172.86	685.93	755.83	1041.9		

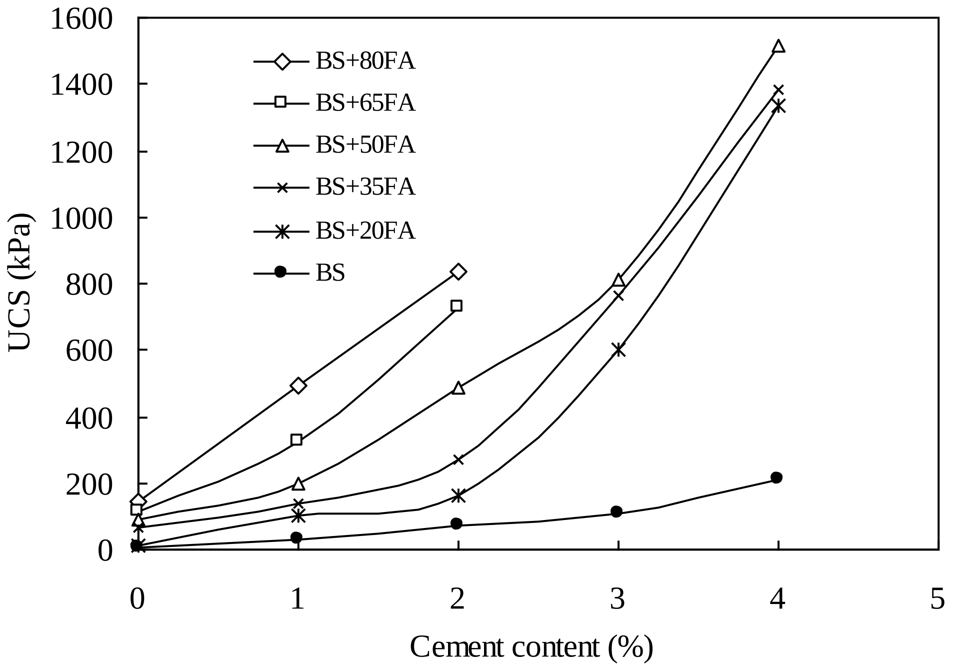


Fig. 7.8(d) Variation of average UCS of sand-fly ash mixes with cement content (28 days curing)

Table 7.14: Average UCS of sand-fly ash-cement mixes (28 days curing)

Cement Content (%)	Unconfined compressive strength (kPa)					
	BS	BS+20FA	BS+35FA	BS+50FA	BS+65FA	BS+80FA
0	3.99	11.92	67.44	91.03	114.54	142.31
1	29.88	101.79	139.39	196.05	324.57	493.48
2	69.74	165	270.72	487.25	727.25	837.89
3	105.40	600.48	762.5	814.89		
4	208.61	1337.27	1384.06	1518.54		

TH-1088_AKALITA

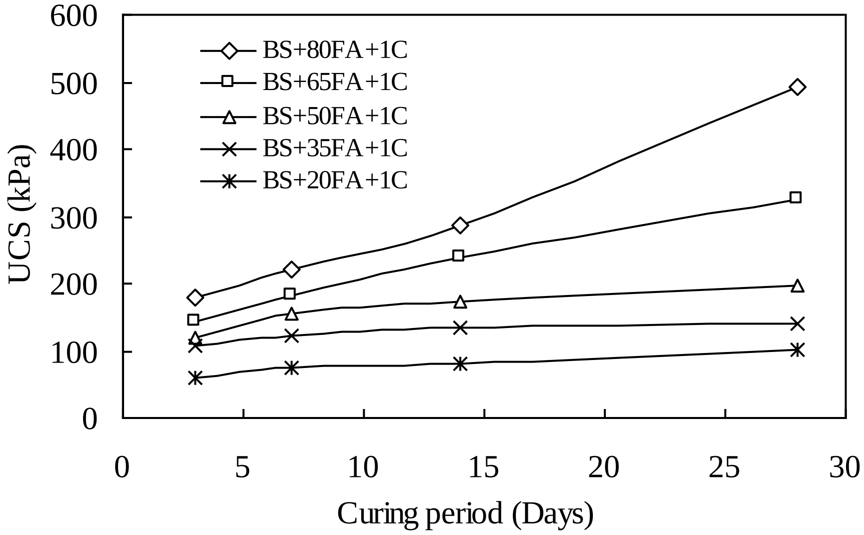


Fig. 7.9(a) Variation of average UCS of sand-fly ash-1% cement mixes with curing period

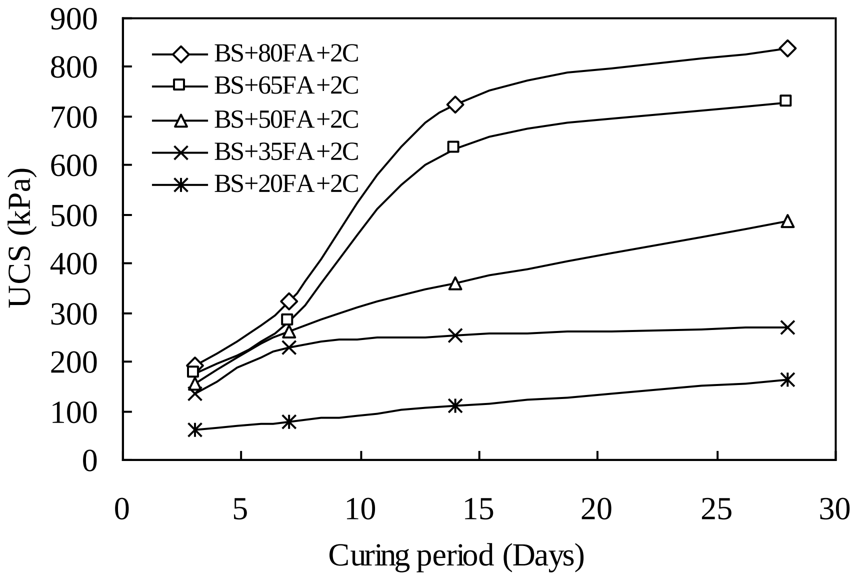


Fig. 7.9(b) Variation of average UCS of sand-fly ash-2% cement mixes with curing period

TH-1088_AKALITA

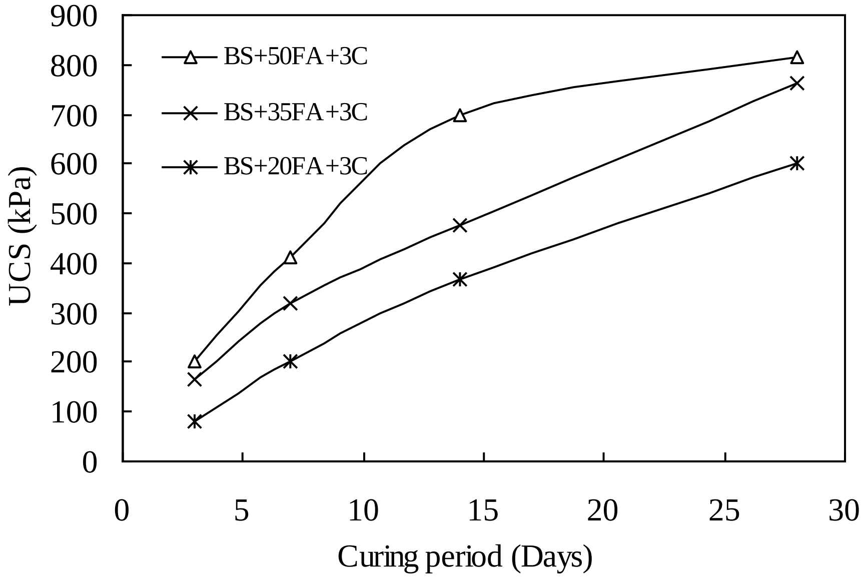


Fig. 7.9(c) Variation of average UCS of sand-fly ash-3% cement mixes with curing period

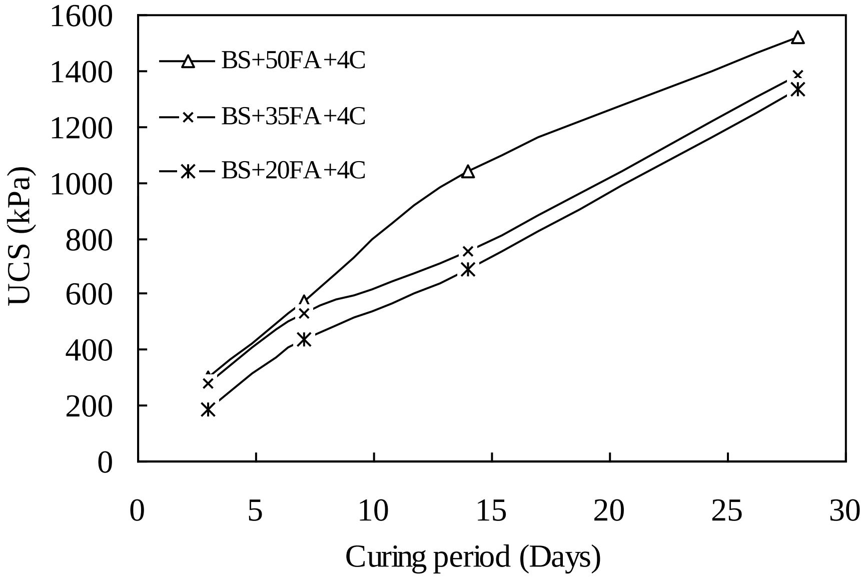


Fig. 7.9(d) Variation of average UCS of sand-fly ash-4% cement mixes with curing period

BS+35FA+1C

Fig. 7.10 Failure pattern of a typical sand-fly ash-cement specimen

TH-1088_AKALITA

7.4 Conclusions

When fly ash is added to the red soil, 28 days UCS values are observed to be the highest for mixes with 35% fly ash content followed by 20% fly ash content. There is no benefit of fly ash addition above 50%, and the UCS values of these mixes remain less than that of the red soil alone. Addition of cement to the red soil significantly improves the UCS. The 28 days UCS of RS+35FA and RS+2C mixes are comparable, with the respective values being 628.82 kPa and 631.06 kPa. At 28 days, the UCS value of 710.4 kPa for the RS+35FA+1C mix is greater than the above values.

When fly ash is added to the sand, the UCS progressively increases with fly ash content and also with curing duration. As cement is added to the sand, the unconfined strength is also increased. The 28 days UCS of BS+80FA mix is 167.84 kPa, which is higher than the UCS value of 105.4 kPa for the BS+3C mix. At 28 days, the UCS of BS+80FA+1C mix is considerably greater at 493.48 kPa.

Addition of more amount of cement to soil is not an economical option. The presence of fly ash is fundamental to improve the strength of the soil mix by facilitating time-dependent pozzolanic reactions. To allow this, maximum replacement of the red soil and sand by 35% and 80% respectively can be made so as to satisfy strength requirements. At the respective optimum mix proportions of fly ash, the unconfined strength of the red soil mixes is found to be higher than the sand mixes. Upon addition of cement, the additional increase in unconfined strength is generally higher for the sand-fly mixes. To assure that the soil mix has the potential to perform satisfactorily in the field, it should satisfy strength criteria for the type of application based on the 7 days UCS.

However, as there are limitations in using only the unconfined test to assess the shear strength of the compacted and cured specimens, the more comprehensive method of triaxial testing need to be performed for better evaluation of strength characteristics of the mixes.

TH-1088_AKALITA

CHAPTER - 8

TRIAXIAL COMPRESSION TESTS ON SOIL-FLY ASH-CEMENT MIXES

8.1 Introduction

Unconfined compression tests showed that the combined addition of fly ash and cement resulted in significant improvement of the strength of both the fine-grained and granular soils. Consolidated drained triaxial tests were carried out on the as-compacted specimens of selected mixes of soil-fly ash and soil-fly ash-cement to understand the influence of fly ash and cement on the shear strength.

Several triaxial test studies on such mixes are reported in literature. Consoli et al. (1998) conducted drained triaxial tests to evaluate the effect of randomly distributed fibre reinforcement and cement inclusion on the response of a sandy soil, and found that the addition of cement increased stiffness, brittleness and peak strength. The triaxial peak strength increase due to fibre inclusion was found to be more effective for uncemented soil. Kaniraj and Havanagi (2001) presented an experimental work to study the individual and combined effects of randomly oriented fiber inclusions and cement stabilization on the geotechnical characteristics of soil-fly ash mixtures. A silty soil and a sandy soil were mixed with a Class F fly ash. Brittle behaviour was more marked in cement-stabilized specimens than in unstabilized specimens. The fiber inclusions changed the behaviour in both instances to ductile behaviour.

Lo and Wardani (2002) conducted both drained and undrained triaxial tests to investigate the mechanical behaviour of a silt stabilized by cement and fly ash slurry. It was found that the cementing agent contributed to both stiffness and strength via two mechanisms, namely bonding between grains and additional dilation. Lav et al. (2006)

TH-1088_AKALITA

conducted a laboratory study of undrained triaxial tests on fly ash stabilized with cement for use as base material in road pavements. Consoli et al. (2009) conducted drained triaxial tests on an artificially cemented soil and investigated the influence of the voids/cement ratio on the initial shear modulus and effective strength parameters.

8.2 Test Programme

Table 8.1 summarizes the composition of the selected mixes used in the triaxial test programme. Fly ash addition was up to 50% only, and the maximum amount of cement added was 3%. The tests were performed at confining pressures of 100, 200, 300 and 400 kPa. The rate of shear loading was 0.24 mm/min. Many of these tests were repeated to check the reproducibility of the results. The triaxial set-up used is shown in Fig. 8.1.

Table 8.1: Composition of various mixes for triaxial compression tests

Soil-fly ash mixes	Soil-cement mixes	Soil-fly ash-cement mixes
RS+20FA	RS+1C	RS+20FA+1C
RS+35FA	RS+2C	RS+20FA+2C
RS+50FA	RS+3C	RS+20FA+3C
		RS+35FA+1C
		RS+35FA+2C
		RS+35FA+3C
		RS+50FA+1C
		RS+50FA+2C
		RS+50FA+3C
BS+20FA	BS+1C	BS+20FA+1C
BS+35FA	BS+2C	BS+20FA+2C
BS+50FA	BS+3C	BS+20FA+3C
		BS+35FA+1C
		BS+35FA+2C
		BS+35FA+3C
		BS+50FA+1C
		BS+50FA+2C
		BS+50FA+3C

TH-1088_AKALITA

The set-up uses a pneumatic system for application of confining pressures. Axial load and axial deformation were measured with electronic load cell and linear variable displacement transducer (LVDT) respectively.

(a) A close up view of the triaxial cell

(b) A view of a test in progress

Fig. 8.1 Experimental set-up for triaxial tests

TH-1088_AKALITA

8.3 Results and Discussion

The criterion used to define failure of any specimen in the drained triaxial test is the maximum value of deviatoric stress, which is taken as either the peak stress observed within 10% axial strain or the stress corresponding to 10% axial strain if no peak is noticed. This maximum stress is being termed as deviatoric strength. Prolonged shearing beyond the failure state will result in residual shear strength.

8.3.1 Deviatoric strength of red soil-fly ash mixes

The deviatoric strength values of all the red soil mixes tested are tabulated in Table 8.2. The influences of fly ash content and curing period on the deviatoric strength of red soil-fly ash mixes are shown in Figures 8.2(a-d) for the four confining pressures, respectively. It is observed that the deviatoric strength of the mixes gradually increases up to 28 days, and the magnitude is greater for higher fly ash contents.

Figure 8.2(e) shows failure patterns of red soil-fly ash specimens tested after 28 days curing at 100 kPa confining pressure. All the three mixes show a clear failure plane indicating brittle behaviour under triaxial test conditions. Similar pattern was also observed at higher confining pressures. This is in contrast to the bulging failure observed in unconfined compression tests on red soil-fly ash specimens (See Fig. 7.1(c)).

In the undrained unconfined compression tests, it was observed that the 28 days UCS value is the highest for the RS+35FA mix followed by that of the RS+20FA mix (See Fig. 7.1(a) and Table 7.3). The 28 days UCS of this RS+35FA mix is 628 kPa. In the drained triaxial tests, the deviatoric strength values of the RS+35FA mix after the 28 days curing period are 483, 711, 800 and 1010 kPa under the confining pressures of 100, 200, 300 and 400 kPa, respectively (See Table 8.2). Thus in comparison, it is noted the unconfined compressive strength is greater than the deviatoric strength only at 100 kPa

TH-1088_AKALITA

confining pressure, but is lesser at higher confining pressures. It should be noted that the UCS is obtained under a quick loading condition with no drainage allowed.

8.3.2 Deviatoric strength of red soil-cement mixes

The effects of cement content and curing period on the strength of red soil-cement mixes are depicted in Figures 8.2(f-i). The results show that the addition of cement improves the deviatoric strength of the red soil, which increases further with curing period. The strength also increases with cement content. Figure 8.2(j) shows typical failure patterns of the red soil-cement specimens after 28 days curing. All the specimens have sheared along well-defined failure planes.

In the unconfined compression tests, it was observed that the 28 days UCS of RS+35FA and RS+2C mixes are comparable, with the value of RS+2C being 631 kPa (See Fig. 7.2 and Table 7.4). In the triaxial tests, the deviatoric strength values of the RS+2C mix after 28 days are 696, 763, 900, 1003 kPa under the confining pressures of 100, 200, 300, 400 kPa, respectively (See Table 8.2). As expected, the UCS value is found to be lower than the deviatoric strength at all the confining pressures.

8.3.3 Deviatoric strength of red soil-fly ash-cement mixes

8.3.3.1 Red soil-20FA-cement mixes

The variations of deviatoric strength with curing period for red soil-20FA-cement mixes are shown in Figs. 8.3(a-d). The strength is noted to increase as curing proceeds and with increasing cement content. Figure 8.3(e) shows the failure patterns of red soil-20FA-cement mixes after 28 days curing when tested at 100 kPa confining pressure. The specimens show brittle behaviour with well-defined failure planes.

TH-1088_AKALITA

8.3.3.2 Red soil-35FA-cement mixes

The variations of deviatoric strength of red soil-35FA-cement mixes with curing period are presented in Figures 8.4(a-d) at different confining pressures. At all cement contents, the deviatoric strength increases with curing time. Figure 8.4(e) shows the failure patterns of red soil-35FA-cement specimens cured for 28 days and subjected to 100 kPa confining stress. Shear failure planes are observed to be well-defined.

In the undrained UC tests, it was noted that the 28 days UCS value of 710 kPa for the RS+35FA+1C mix (See Fig. 7.3(d) and Table 7.8) is greater than the corresponding values of both the RS+35FA and RS+2C mixes. In the triaxial drained tests, the deviatoric strength values of the RS+35FA+1C mix after 28 days curing are 656, 736, 834 and 1091 kPa under the respective confining pressures of 100, 200, 300 and 400 kPa (See Table 8.2). Thus, it is observed that the UCS is greater than deviatoric strength at only 100 kPa confining pressure, but is lesser at higher confining pressures.

8.3.3.3 Red soil-50FA-cement mixes

Figures 8.5(a-d) show the variations of deviatoric strength with curing period for red soil-50FA-cement mixes at different confining pressures. The failure patterns of the specimens when sheared completely are shown in Fig. 8.5(e). Under the influence of confining stress, the failure planes of the specimens are found to be well-defined and prominent in most of the specimens.

8.3.4 Shear strength of all red soil mixes

Stress path in a *p*-*q* plot has been utilized to interpret the drained triaxial test results and obtain the shear strength parameters of the unsaturated as-compacted specimens. For any mix, the deviatoric strength values at the four confining pressures are used. The variables, *p* and *q*, are defined as follows:

TH-1088_AKALITA

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

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

where, σ_1 = major principal stress,

σ_3 = minor principal stress = confining pressure, and

$(\sigma_1 - \sigma_3)$ = deviatoric strength

In the *p*-*q* plot, the K_f line is obtained by joining the maximum stress points of the failure circles for the four tests. The intercept of the K_f line with the Y-axis is *d*, and the angle that the K_f line makes with the horizontal is α .

The equation of the K_f line and the equation of the shear strength envelope are related as follows:

$$\tan\alpha = \sin\phi \tag{8.1}$$

$$c = \frac{d}{\cos\phi} \tag{8.2}$$

where, ϕ = angle of shearing resistance, and

c = cohesion intercept of the shear strength envelope

For all tested specimens, the drained shear strength parameters, namely cohesion intercept, c_d, and angle of shearing resistance, ϕ_d, determined by using Eqns. 8.1 and 8.2 are listed in Table 8.3.

Figs. 8.6(a) and 8.6(b) show the c_d and ϕ_d values of red soil-fly ash mixes at various curing periods. The cohesion of the mix increases with both curing period and also with fly ash content. The friction angle also increases with curing but only marginally beyond 35% fly ash content.

Figures 8.6(c) and 8.6(d) depict the cohesion and friction angle values of red soil-cement mixes for various curing periods. Addition of more cement to red soil tends to augment cohesion but does not improve the friction angle.

TH-1088_AKALITA

Figures 8.6(e) to 8.6(j) present the variations of c_d and ϕ_d values of all red soil-fly ash-cement mixes with curing period. Addition of cement to red soil-fly ash mixes tends to increase both cohesion and friction angle for all curing times.

The cohesion intercept of the as-compacted specimens depends primarily on the degree of saturation. All the specimens have been compacted at their respective maximum dry unit weight and optimum moisture content obtained from the compaction tests. For soils, the degree of saturation at standard Proctor's optimum conditions varies in the range of 90 to 95%. While it is relatively easier to de-air soil specimens, it is almost impossible to remove air from the hollow cenospheres of fly ash. As a result, the air content in soil-fly ash and soil-fly ash-cement mixes is relatively more, and hence a lower degree of saturation results. Another factor that affects the shear strength parameters is the curing period.

Addition of fly ash to the red soil increases both the shear strength parameters, namely the cohesion intercept and the angle of shearing resistance. The increase in cohesion intercept reflects the increase in cementation. After 28 days curing, the values of c_d and ϕ_d at 20% and 50% fly ash contents are 89.39 kPa, 22.7° and 137.77 kPa, 27.8°, respectively.

Addition of cement to the red soil increases the cohesion intercept substantially, but the friction angle is reduced at higher cement contents. After a curing time of 28 days, the values of c_d and ϕ_d at 1% and 3% cement contents are 165.53 kPa, 20.9° and 256.99 kPa, 19.5°, respectively.

Addition of both fly ash and cement to the red soil generates an increase in cohesion intercept without a trend, but the angle of shearing resistance increases with a definite trend. The values of c_d and ϕ_d of RS+20FA+3C and RS+50FA+3C mixes are 281.58 kPa, 21.7° and 207.9 kPa, 33.1°, respectively after 28 days curing.

TH-1088_AKALITA

8.3.5 Deviatoric strength of sand-fly ash mixes

The deviatoric strength values of all sand mixes tested are summarized in Table 8.4. The influences of fly ash content and curing period on the deviatoric strength of sand-fly ash mixes are shown in Figures 8.7(a-c). The strength is noted to increase with confining pressure and with curing period. Among the mixes tested, the strength of the BS+50FA mix with more fly ash content is found to be the maximum.

In the unconfined compression tests, it was also observed that the UCS of sand-fly mixes progressively increases with fly ash content up to 90% and also with curing duration (See Figs. 7.6(a) and 7.6(b)). It was also noted that the 28 days UCS for the BS+50FA mix is 91 kPa (See Table 7.9). In the triaxial compression tests, the deviatoric strength values of the BS+50FA mix after the 28 days curing period are 453, 662, 818 and 987 kPa under the confining pressures of 100, 200, 300 and 400 kPa, respectively (See Table 8.4). These values are less than the corresponding strength values of 606, 875, 946 and 1155 kPa for the RS+50FA mix (See Table 8.2).

8.3.6 Deviatoric strength of sand-cement mixes

The variations of deviatoric strength with curing period for sand-cement mixes are shown in Figs. 8.8(a-d) at different confining pressures. The results show that the addition of cement significantly improves the strength of the sand.

From UC tests, it was found that the 28 days UCS value of 105 kPa for BS+3C mix (See Fig. 7.7 and Table 7.10) is more than that of BS+50FA mix (91 kPa). From triaxial tests, the deviatoric strength values of the BS+3C mix at 28 days curing are 563, 826, 981 and 1146 kPa under the confining pressures of 100, 200, 300 and 400 kPa, respectively (See Table 8.4). These values are lower than the corresponding deviatoric strength values of 783, 1003, 1049 and 1079 kPa for the RS+3C mix except at 400 kPa confining pressure (See Table 8.2).

TH-1088_AKALITA

8.3.7 Deviatoric strength of sand-fly ash-cement mixes

8.3.7.1 Sand-20FA-cement mixes

The variations of deviatoric strength at different confining pressures and curing periods of sand-20FA-cement mixes are presented in Figs. 8.9(a-d). It is observed that the addition of cement to the sand-fly ash mixes changes the failure mode from ductile to brittle. The stiffness and peak strength of the mixes is enhanced resulting in an improved response to loading.

Figure 8.9(e) shows the failure patterns of BS-20FA-cement mixes after 28 days curing. A combination of bulging failure and brittle failure is observed at 1% cement content, whereas brittle failure is prominent at higher cement contents. The brittle failure is noticed at an axial strain ranging from below 5%. This finding is similar to that of Consoli et al. (2001) from drained triaxial tests on sand-fly ash-lime mixtures.

8.3.7.2 Sand-35FA-cement mixes

The variations of deviatoric strength of sand-35FA-cement mixes with confining pressure and curing period are shown in Figs. 8.10(a-d). Figure 8.10(e) shows the failure patterns of specimens of the mixes tested at 100 kPa confining pressure after 28 days curing time. For all mixes, the rupture plane is clearly visible.

8.3.7.3 Sand-50FA-cement mixes

The variations of deviatoric strength of sand-50FA-cement mixes with curing period and confining pressure are shown in Figs. 8.11(a-d). Figure 8.11(e) shows the failure patterns of sand-50FA-cement specimens after curing for 28 days and tested at 100 kPa confining pressure. Clear failure planes are observed in all of them.

From UC tests, the 28 days UCS of BS+50FA+3C mix was found to be 814 kPa (See Fig. 7.8(d) and Table 7.14). From the drained triaxial tests, the deviatoric strength

TH-1088_AKALITA

values of the BS+50FA+3C mix are 1253, 1542, 1588 and 2100 kPa at the confining pressures of 100, 200, 300 and 400 kPa, respectively after 28 days curing (See Table 8.4). These values are higher than the respective deviatoric strength values of 1062, 1177, 1504 and 1745 kPa for the RS+50FA+3C mix (See Table 8.2). The deviatoric strength of sand-fly ash-cement mixes is higher than that of sand-fly ash mixes.

8.3.8 Shear strength of all sand mixes

For every mix, cohesion and friction angle values have been calculated by drawing the K_f line using the deviatoric strength values at the four confining pressures. Table 8.5 summarizes the cohesion (c_d) and friction angle (ϕ_d) values of various mixes of sand tested.

Figures 8.12(a) and 8.12(b) show the variations of cohesion and friction angle vs. curing period, respectively for sand-fly ash mixes. The values of c_d and ϕ_d at 20% and 50% fly ash contents after 28 days curing are 91.9 kPa, 24.5° and 96.59 kPa, 26.5°, respectively. With no curing, the corresponding values are 50.6 kPa, 19.8° and 71.47 kPa, 23.9°, respectively (See Table 8.5). Addition of fly ash to the sand tends to increase both the cohesion and friction angle. With curing, improved strength parameters are obtained for all the sand-fly ash mixes. In direct shear tests, it was observed that the drained shear strength parameters of BS+20FA and BS+50FA mixes with no curing are 14.65 kPa, 24.22° and 24.03 kPa, 21.8°, respectively (See Table 5.3). These values are lower than those obtained from the triaxial tests.

Figures 8.12(c) and 8.12(d) depict the variations of cohesion and friction angle vs. curing period of sand-cement mixes. Addition of cement to the sand tends to increase both the cohesion and friction angle. After a curing duration of 28 days, the values of c_d and ϕ_d at 1% and 3% cement contents are 94.13 kPa, 25.9° and 116.76 kPa, 29.3°,

TH-1088_AKALITA

respectively. Both the shear strength parameters values of BS+3C mix are higher than those of BS+50FA mix.

Figures 8.12(e) to 8.12(j) show the variations of c_d and ϕ_d values of sand-fly ash-cement mixes with curing times. Addition of cement to sand-fly ash mixes tends to increase both the cohesion and friction angle at all curing periods. After 28 days curing, the values of c_d and ϕ_d of BS+20FA+3C and BS+50FA+3C mixes are 306.93 kPa, 33.7° and 187.39 kPa, 35.3°, respectively. These friction angle values are higher than the corresponding values for RS+20FA+3C and RS+50FA+3C mixes.

TH-1088_AKALITA

Table 8.2: Deviatoric strength (in kPa) of red soil mixes at different curing periods and confining pressures

MIXES	0 days Confining pressures (kPa)				3 days Confining pressures (kPa)				7 days Confining pressures (kPa)				14 days Confining pressures (kPa)				28 days Confining pressures (kPa)			
	100	200	300	400	100	200	300	400	100	200	300	400	100	200	300	400	100	200	300	400
RS+20FA	157	310	373	426	190	358	458	478	206	375	493	508	286	431	510	640	467	512	519	832
RS+35FA	265	373	514	611	288	437	619	672	307	509	702	713	375	593	749	849	483	711	800	1010
RS+50FA	344	390	559	646	419	521	772	859	458	598	793	933	505	718	859	1005	606	875	946	1155
RS+1C	341	360	400	503	410	394	506	593	450	499	593	689	517	538	672	776	630	681	768	961
RS+2C	374	433	480	526	426	480	551	619	538	552	650	726	596	617	734	812	696	763	900	1003
RS+3C	422	489	507	558	500	533	580	648	645	596	673	789	722	756	793	951	783	1003	1049	1079
RS+20FA+1C	400	459	495	620	428	505	578	700	471	500	622	746	476	590	722	800	595	634	797	952
RS+20FA+2C	426	540	641	678	542	600	700	820	554	638	757	859	646	709	807	972	855	911	935	1032
RS+20FA+3C	486	588	681	746	592	724	658	838	624	724	824	925	765	814	878	1103	969	1086	1111	1338
RS+35FA+1C	333	469	539	619	441	614	640	792	500	625	734	859	589	675	784	983	656	736	834	1091
RS+35FA+2C	369	486	595	650	486	671	700	854	575	748	819	951	706	756	848	1086	739	809	959	1130
RS+35FA+3C	484	533	651	721	595	713	824	925	650	802	848	1006	755	854	878	1155	803	918	997	1256
RS+50FA+1C	362	463	589	670	462	562	619	795	543	618	735	905	663	745	826	1093	762	890	1089	1201
RS+50FA+2C	463	600	700	865	545	695	773	972	649	802	858	1105	742	901	981	1259	883	1116	1325	1500
RS+50FA+3C	506	646	753	919	577	767	877	1057	689	868	964	1205	842	1015	1175	1432	1062	1177	1504	1745

TH-1088_AKALITA

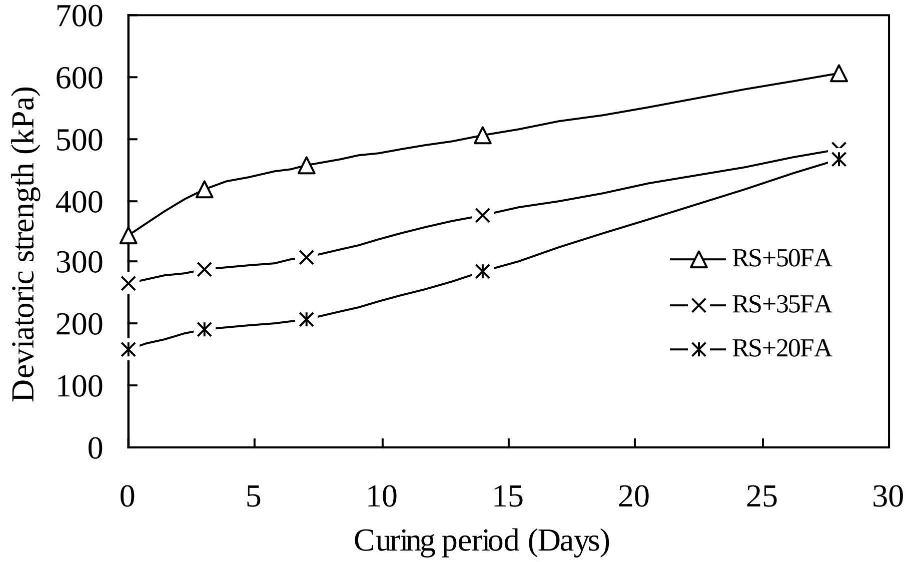


Fig. 8.2(a) Variation of deviatoric strength of red soil-fly ash mixes with curing period (100 kPa confining pressure)

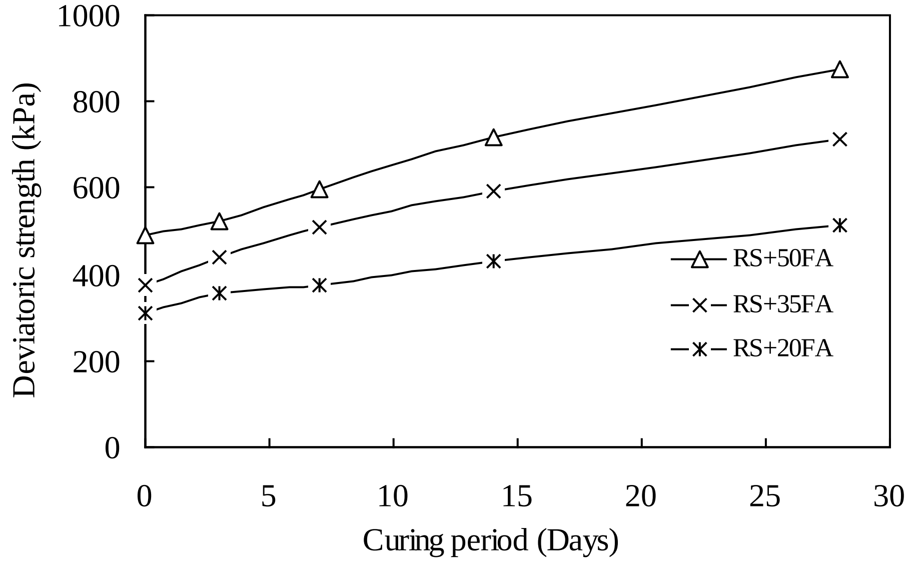


Fig. 8.2(b) Variation of deviatoric strength of red soil-fly ash mixes with curing period (200 kPa confining pressure)

TH-1088_AKALITA

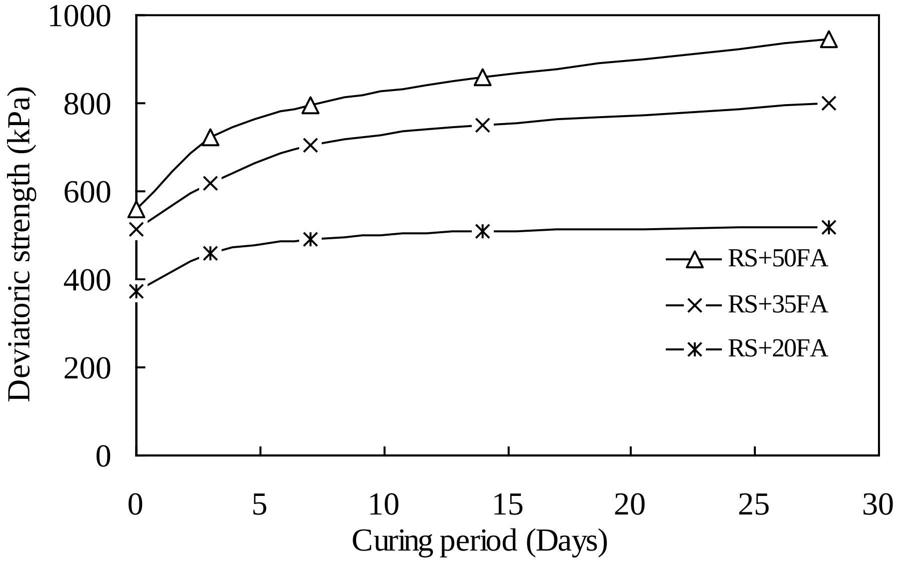


Fig. 8.2(c) Variation of deviatoric strength of red soil-fly ash mixes with curing period (300 kPa confining pressure)

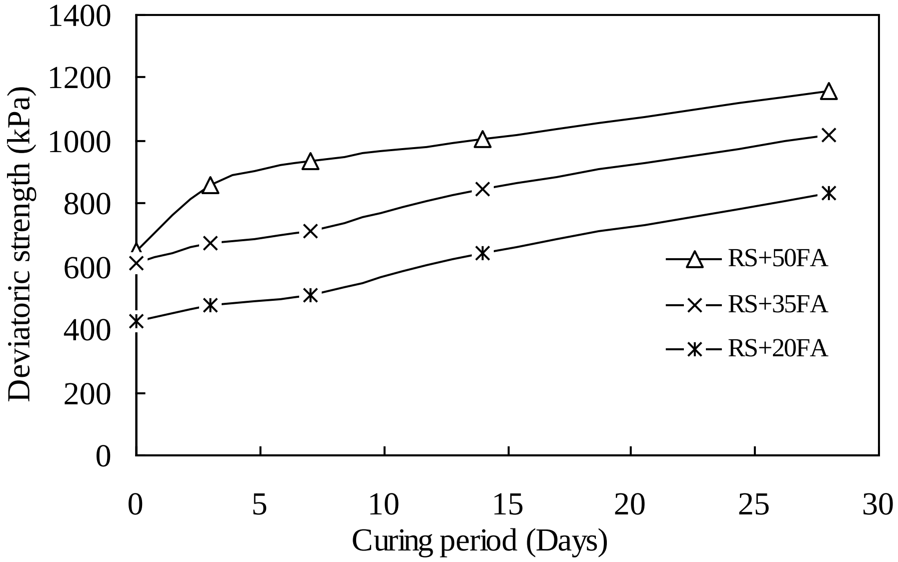


Fig. 8.2(d) Variation of deviatoric strength of red soil-fly ash mixes with curing period (400 kPa confining pressure)

TH-1088_AKALITA

RS+20FA RS+35FA RS+50FA

Fig. 8.2(e) Failure patterns of red soil-fly ash specimens a [illegible] ring
(100 kPa confining pressure)

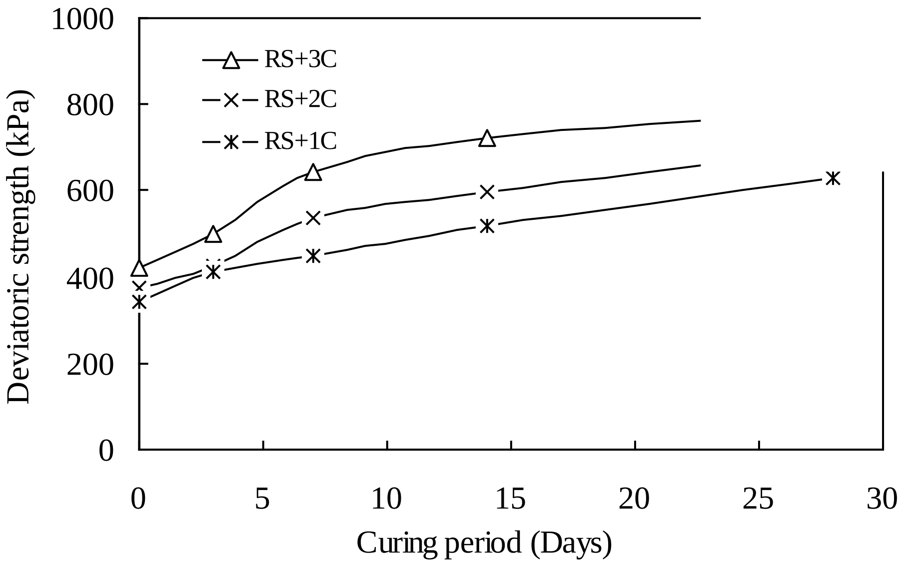


Fig. 8.2(f) Variation of deviatoric strength of red soil-cement mixes with curing period
(100 kPa confining pressure)

TH-1088_AKALITA

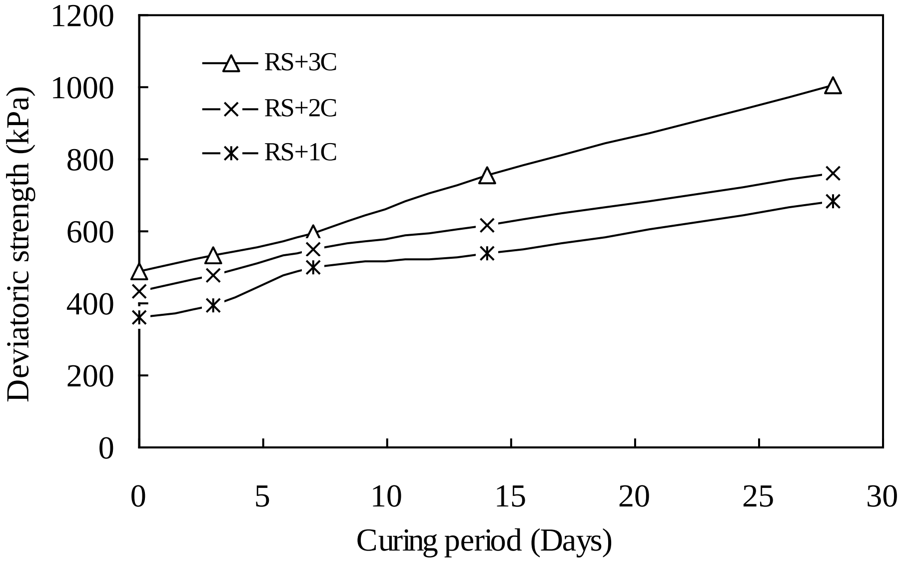


Fig. 8.2(g) Variation of deviatoric strength of red soil-cement mixes with curing period (200 kPa confining pressure)

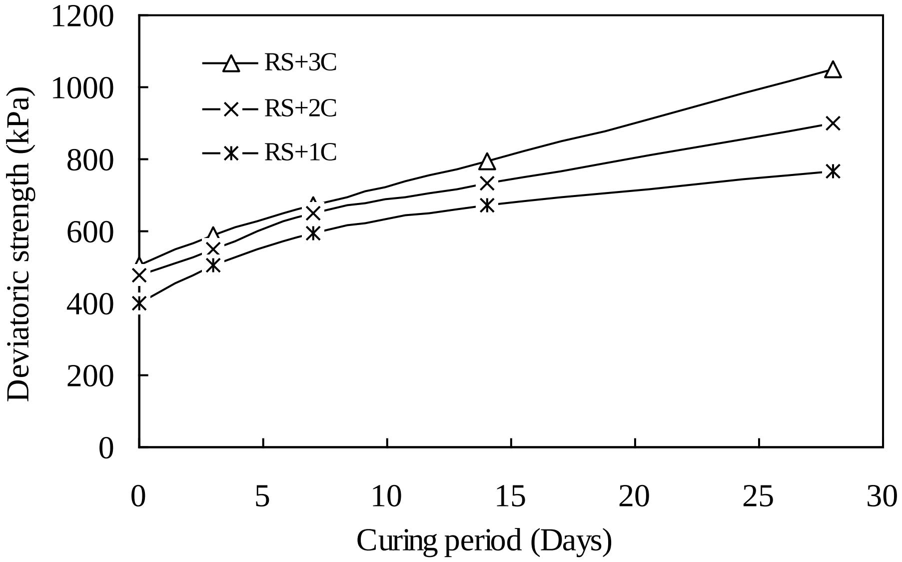


Fig. 8.2(h) Variation of deviatoric strength of red soil-cement mixes with curing period (300 kPa confining pressure)

TH-1088_AKALITA

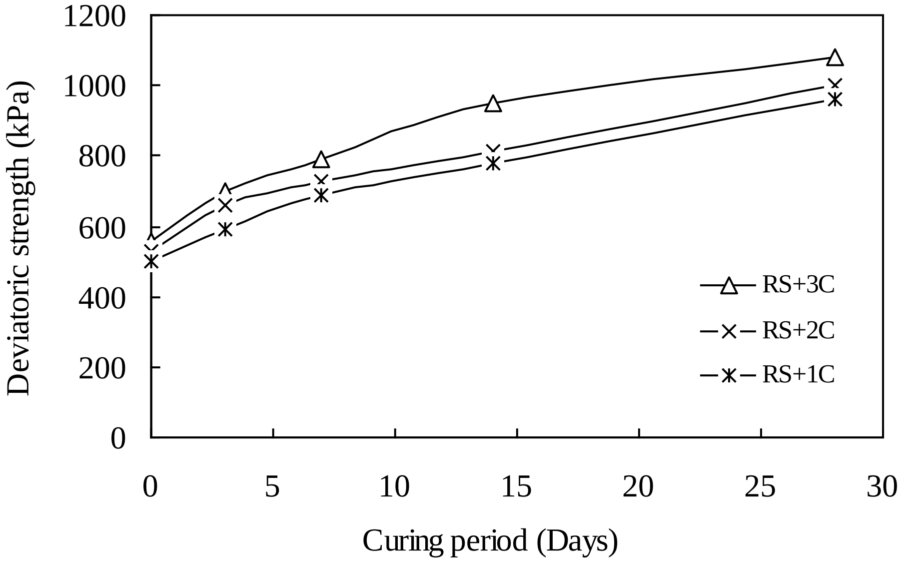


Fig. 8.2(i) Variation of deviatoric strength of red soil-cement mixes with curing period (400 kPa confining pressure)

RS+1C RS+2C RS+3C

Fig. 8 … atterns of re … specimens a … uring (100 kP … essure)

TH-1088_AKALITA

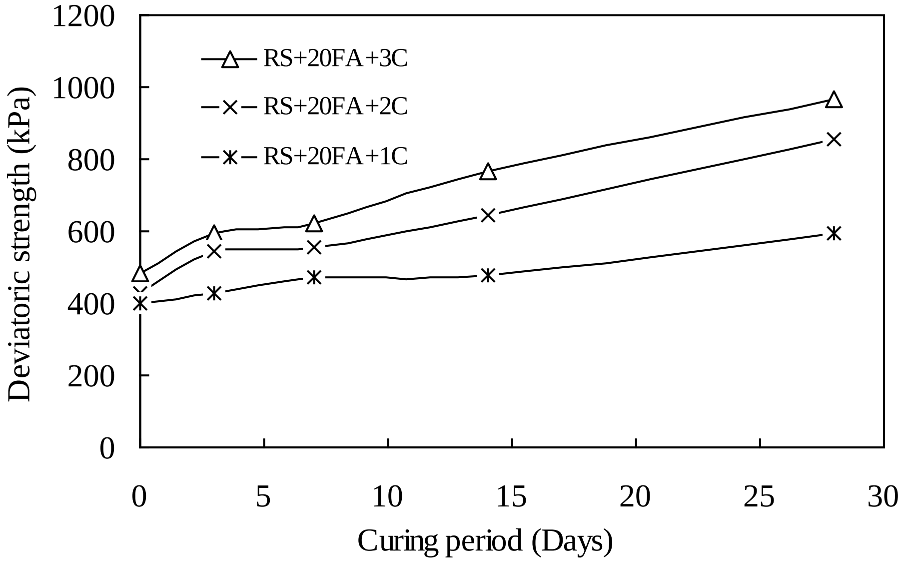


Fig. 8.3(a) Variation of deviatoric strength of red soil-20FA-cement mixes with curing period (100 kPa confining pressure)

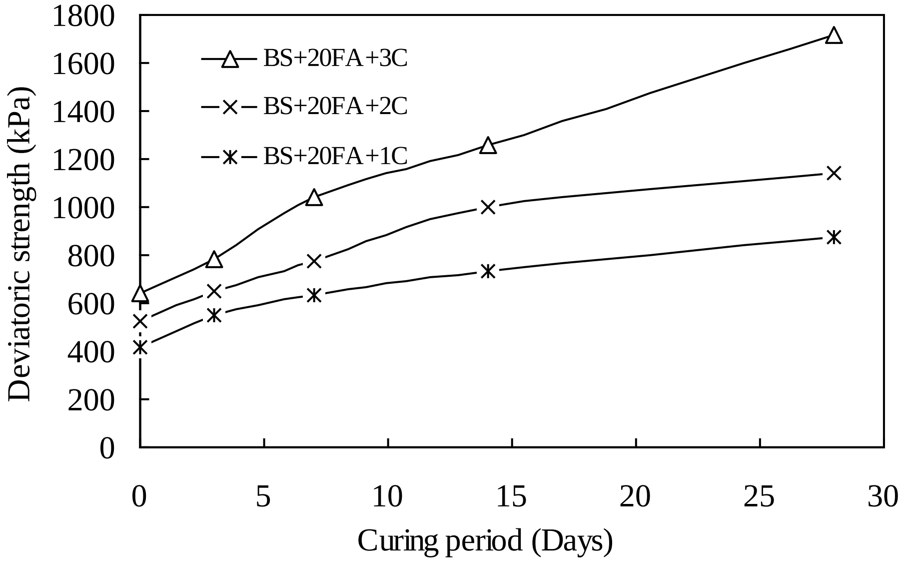


Fig. 8.3(b) Variation of deviatoric strength of red soil-20FA-cement mixes with curing period (200 kPa confining pressure)

TH-1088_AKALITA

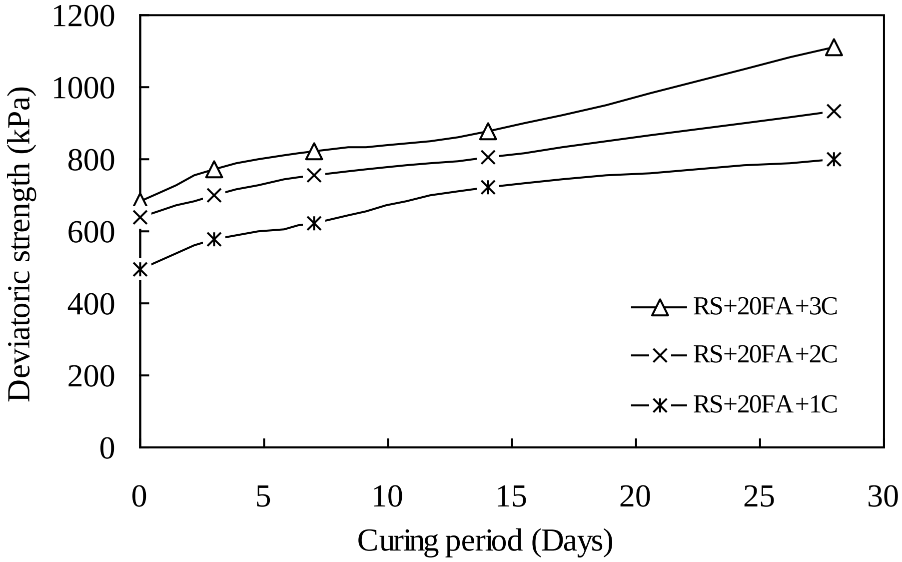


Fig. 8.3(c) Variation of deviatoric strength of red soil-20FA-cement mixes with curing period (300 kPa confining pressure)

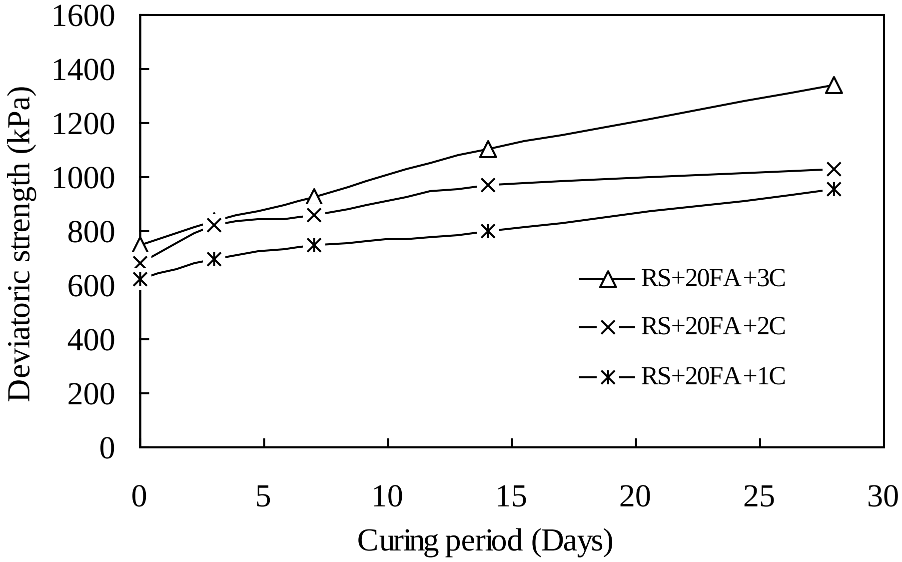


Fig. 8.3(d) Variation of deviatoric strength of red soil-20FA-cement mixes with curing period (400 kPa confining pressure)

TH-1088_AKALITA

RS+20FA+1C RS+20FA+2C RS+20FA+3C

patterns of ... ent specim ... uring
(1 ... essure)

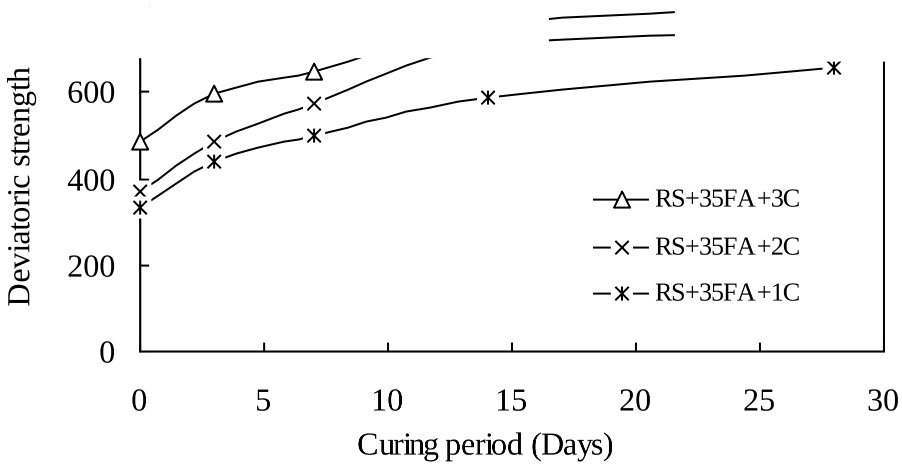


Fig. 8.4(a) Variation of deviatoric strength of red soil-35FA-cement mixes with curing period (100 kPa confining pressure)

TH-1088_AKALITA

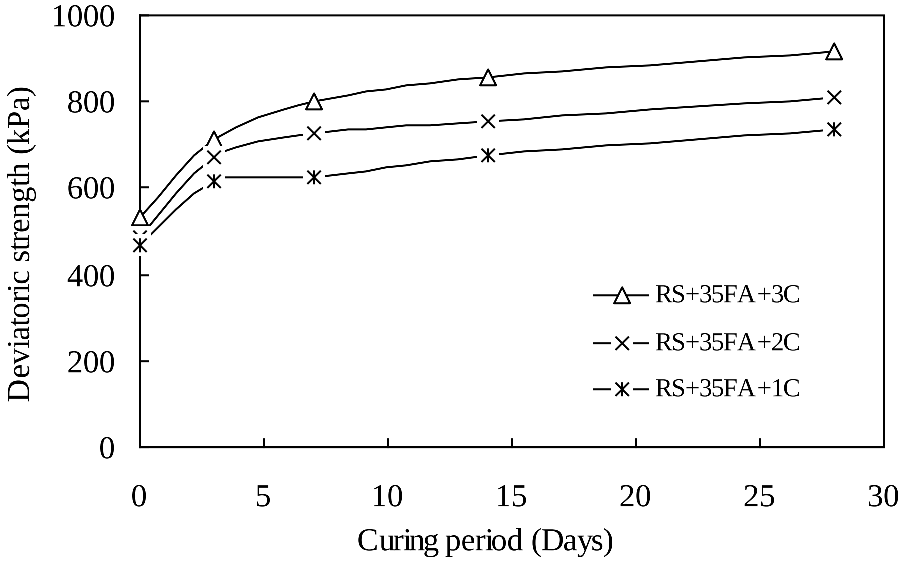


Fig. 8.4(b) Variation of deviatoric strength of red soil-35FA-cement mixes with curing period (200 kPa confining pressure)

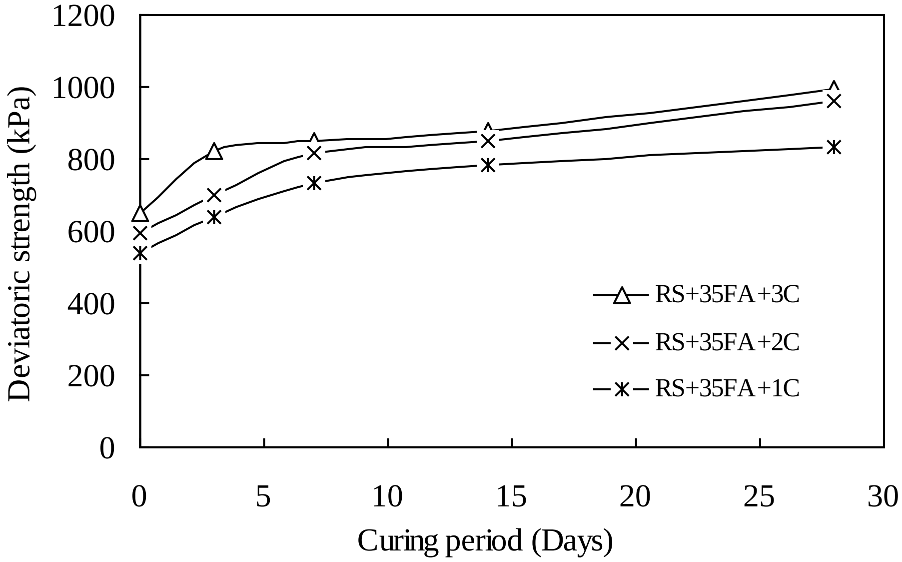


Fig. 8.4(c) Variation of deviatoric strength of red soil-35FA-cement mixes with curing period (300 kPa confining pressure)

TH-1088_AKALITA

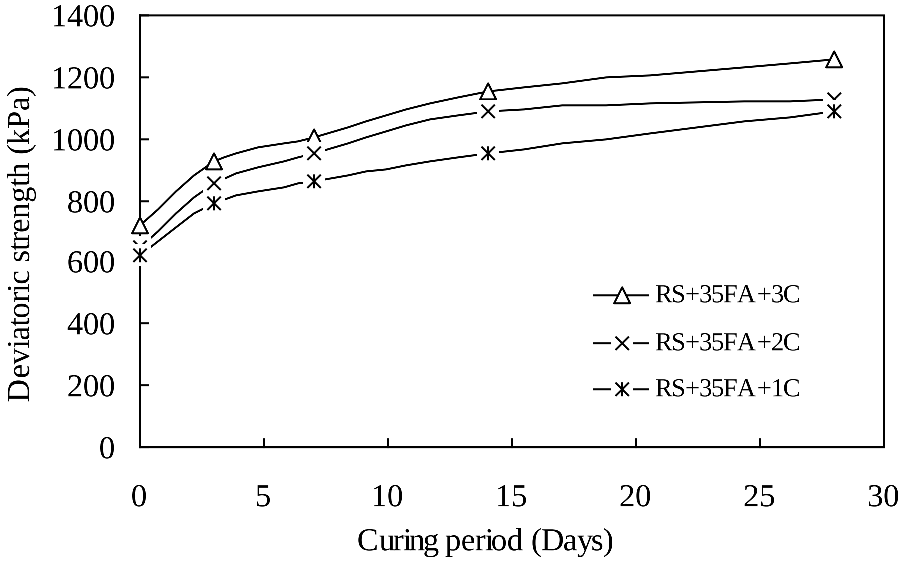


Fig. 8.4(d) Variation of deviatoric strength of red soil-35FA-cement mixes with curing period (400 kPa confining pressure)

RS+35FA+1C RS+35FA+2C RS+35FA+3C

F atterns o ient spe ig
(ressure)

TH-1088_AKALITA

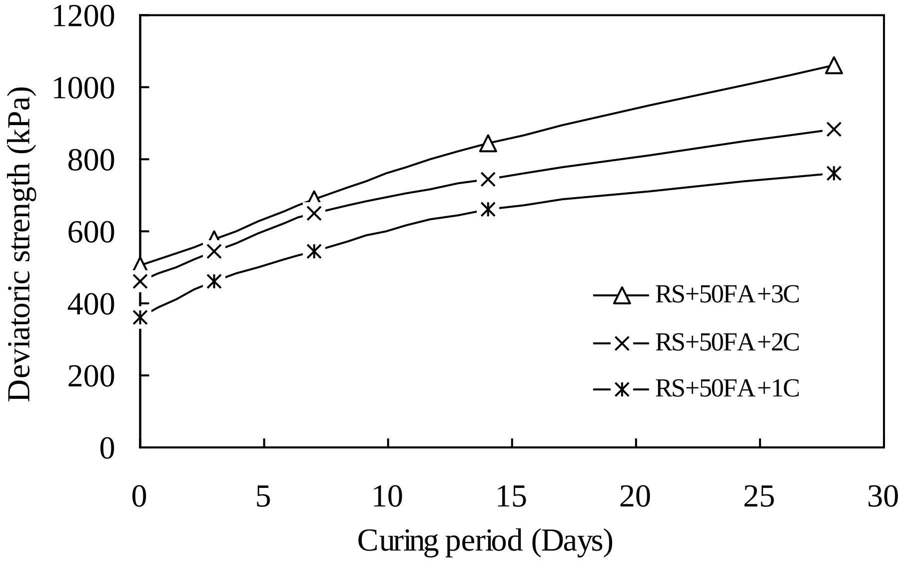


Fig. 8.5(a) Variation of deviatoric strength of red soil-50FA-cement mixes with curing period (100 kPa confining pressure)

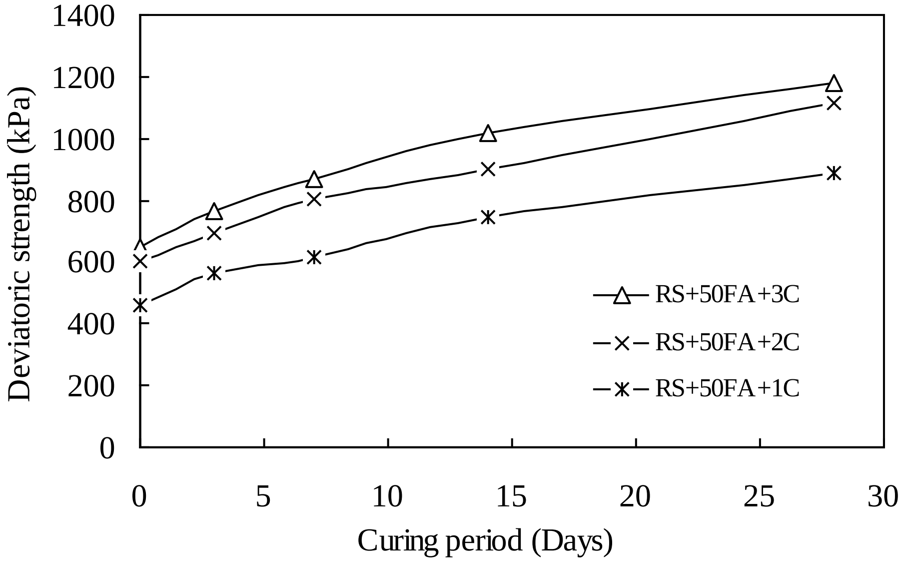


Fig. 8.5(b) Variation of deviatoric strength of red soil-50FA-cement mixes with curing period (200 kPa confining pressure)

TH-1088_AKALITA

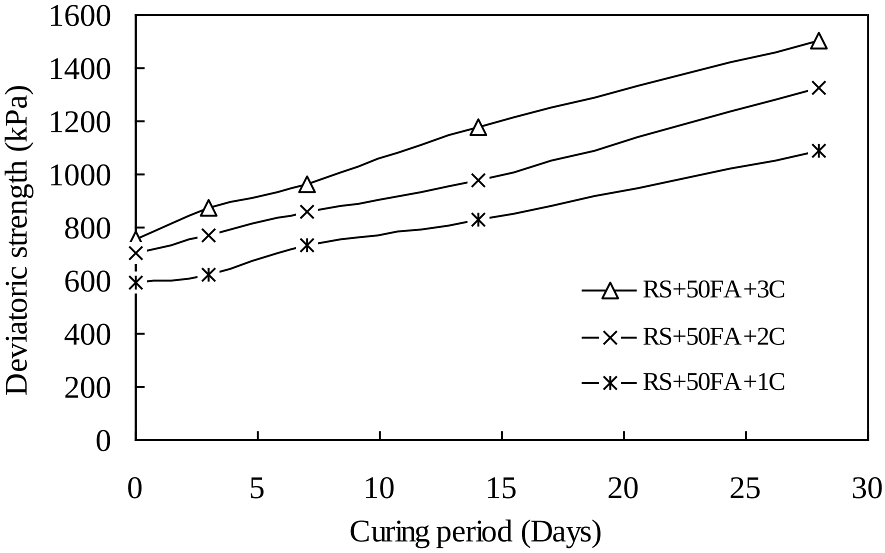


Fig. 8.5(c) Variation of deviatoric strength of red soil-50FA-cement mixes with curing period (300 kPa confining pressure)

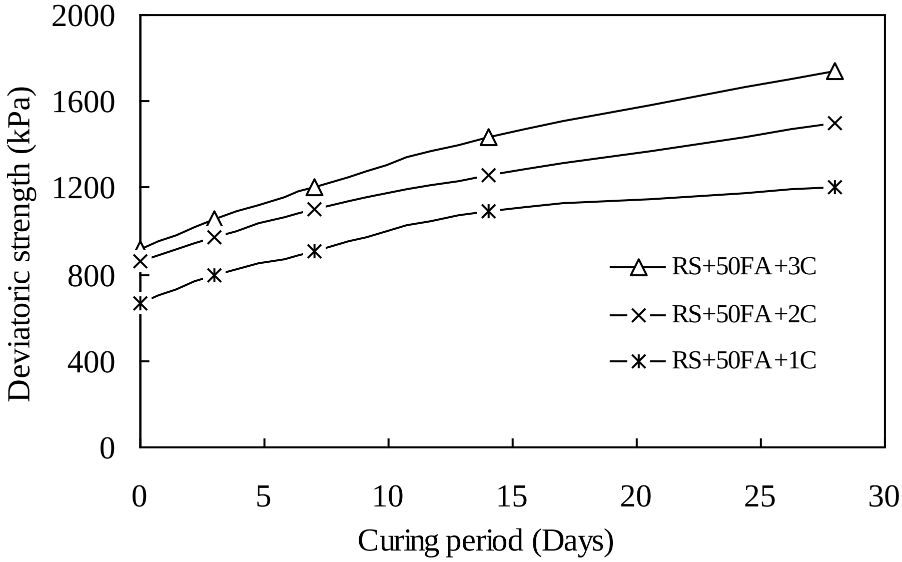


Fig. 8.5(d) Variation of deviatoric strength of red soil-50FA-cement mixes with curing period (400 kPa confining pressure)

RS+50FA+1C RS+50FA+2C RS+50FA+3C

Fig rns of cime
(1

TH-1088_AKALITA

Table 8.3: Drained shear strength parameters of various mixes of red soil

MIXES	0 days		3 days		7 days		14 days		28 days	
	c_d (kPa)	ϕ_d (°)	c_d (kPa)	ϕ_d (°)	c_d (kPa)	ϕ_d (°)	c_d (kPa)	ϕ_d (°)	c_d (kPa)	ϕ_d (°)
RS+20FA	34.2	17.9	42.67	19.4	45.05	20.3	61.5	21.3	89.39	22.7
RS+35FA	49.2	21.7	53.7	23.8	60.22	25	75.1	26.3	115.16	27.5
RS+50FA	73.26	20.6	75.38	26.3	89.41	26.6	110.46	26.8	137.77	27.8
RS+1C	107.08	12.28	116.67	14.85	131.31	16.8	141.54	18.5	165.53	20.9
RS+2C	113.80	15.09	164.35	11.7	173.21	14.5	185.78	16.2	199.99	20.3
RS+3C	126.30	16.27	209.62	8.7	213.41	12.7	232.5	15.9	256.99	19.5
RS+20FA+1C	120.99	15.1	119.68	17.9	122.5	18.93	127.44	20.8	142.7	22.7
RS+20FA+2C	129.55	17.6	154.4	18.6	155.30	19.9	176.31	20.6	220.07	22.9
RS+20FA+3C	148.36	17.7	173.03	17.7	184.95	19.5	207.42	21.2	281.58	21.7
RS+35FA+1C	103.58	15.1	119.19	20.8	129.48	21.8	139.87	23.1	148.56	24.9
RS+35FA+2C	113.01	17.0	131.94	21.5	158.3	22.1	173.96	23	187.77	23.6
RS+35FA+3C	143.44	17.1	168.6	20.8	186.45	21.1	192.32	23.1	197.75	25.1
RS+50FA+1C	90.01	20.1	118.39	20.4	132.96	22.2	151.87	24.6	190.31	25.6
RS+50FA+2C	108.35	23.3	130.74	24	155.6	24.9	169.11	27	197.08	30.5
RS+50FA+3C	120.24	23.7	134.57	25.9	157.96	27	183.79	29.5	207.9	33.1

TH-1088_AKALITA

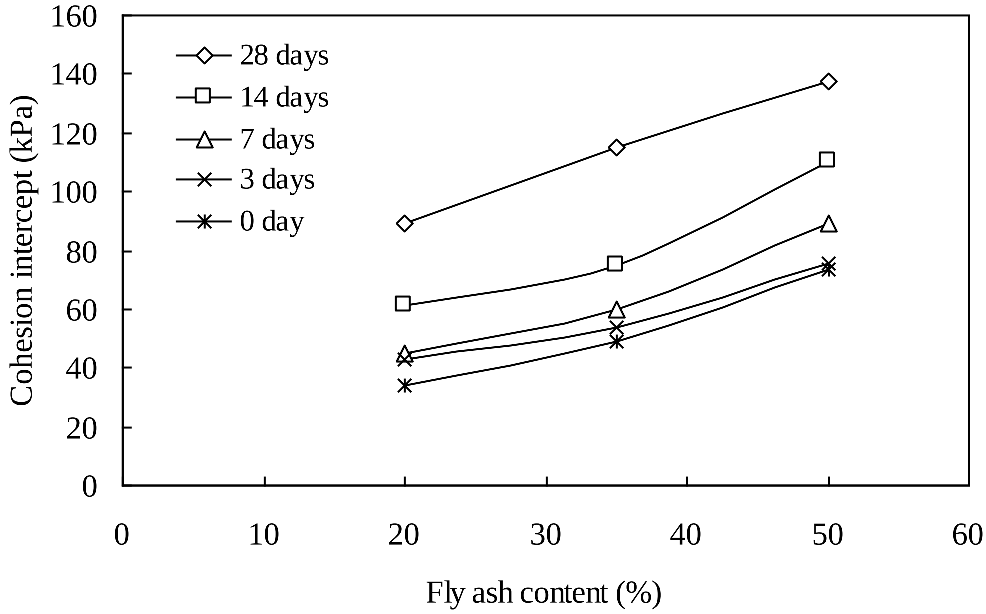


Fig. 8.6(a) Variation of cohesion of red soil-fly ash mixes at different curing periods

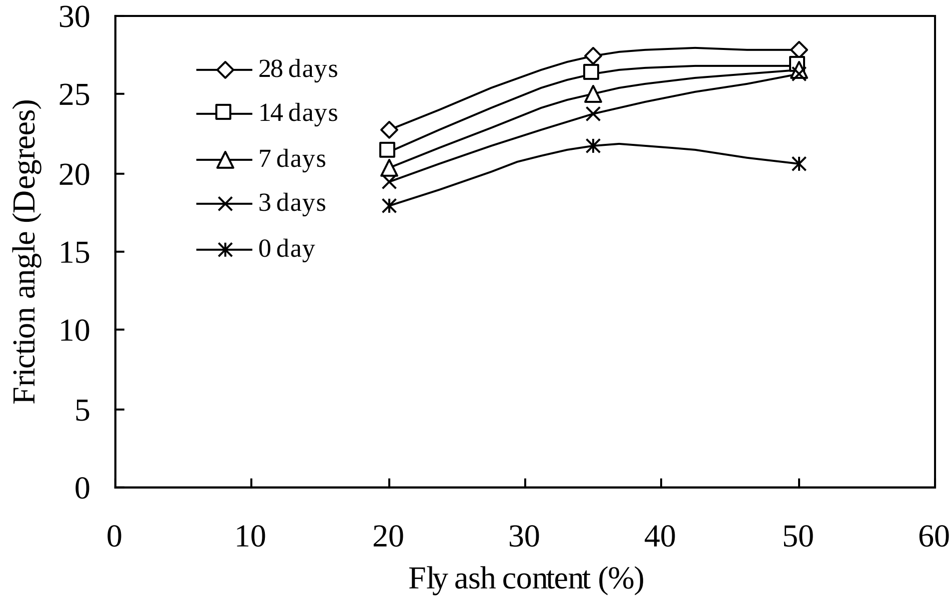


Fig. 8.6(b) Variation of friction angle of red soil-fly ash mixes at different curing periods

TH-1088_AKALITA

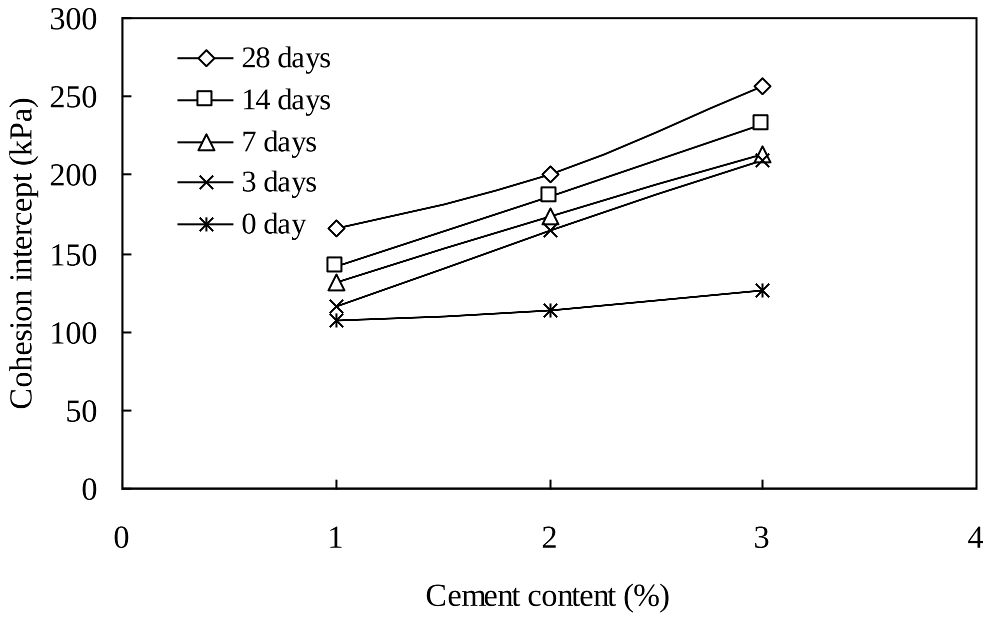


Fig. 8.6(c) Variation of cohesion of red soil-cement mixes at different curing periods

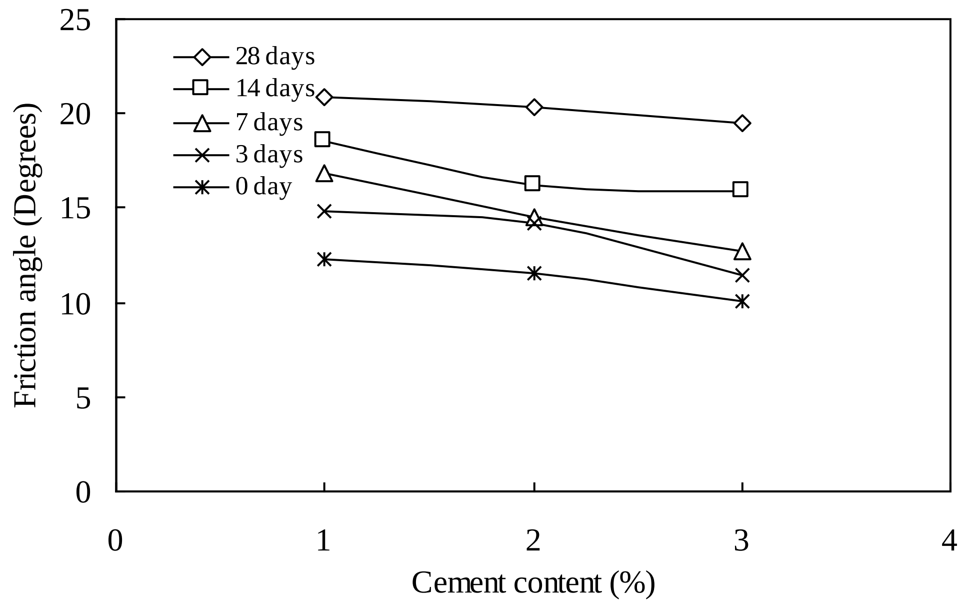


Fig. 8.6(d) Variation of friction angle of red soil-cement mixes at different curing periods

TH-1088_AKALITA

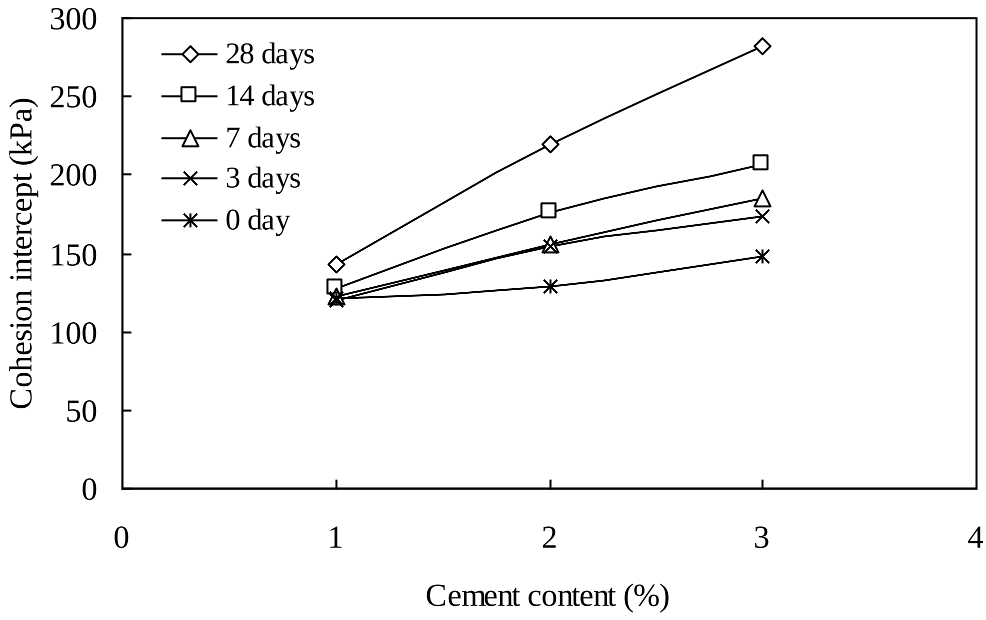


Fig. 8.6(e) Variation of cohesion of red soil-20FA-cement mixes at different curing periods

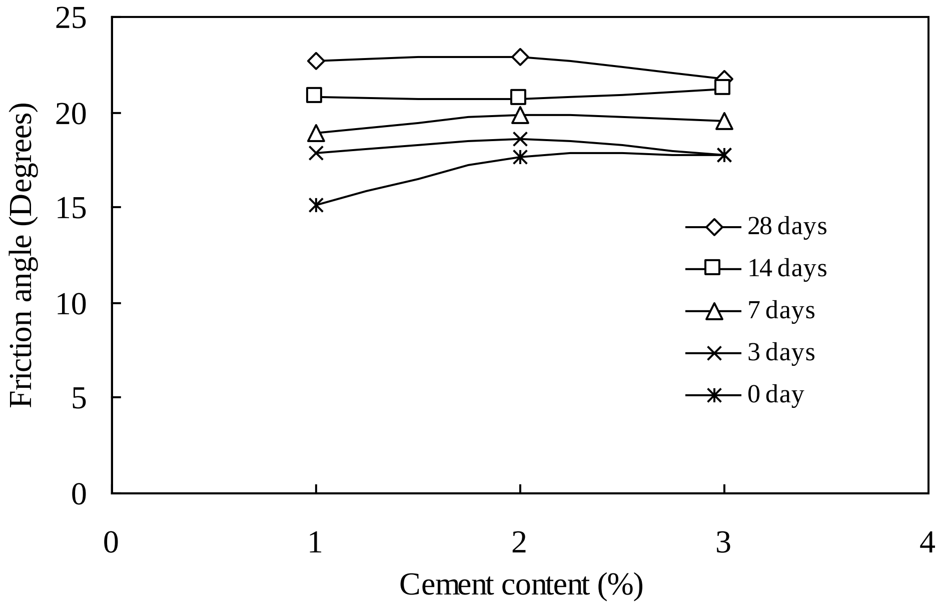


Fig. 8.6(f) Variation of friction angle of red soil-20FA-cement mixes at different curing periods

TH-1088_AKALITA

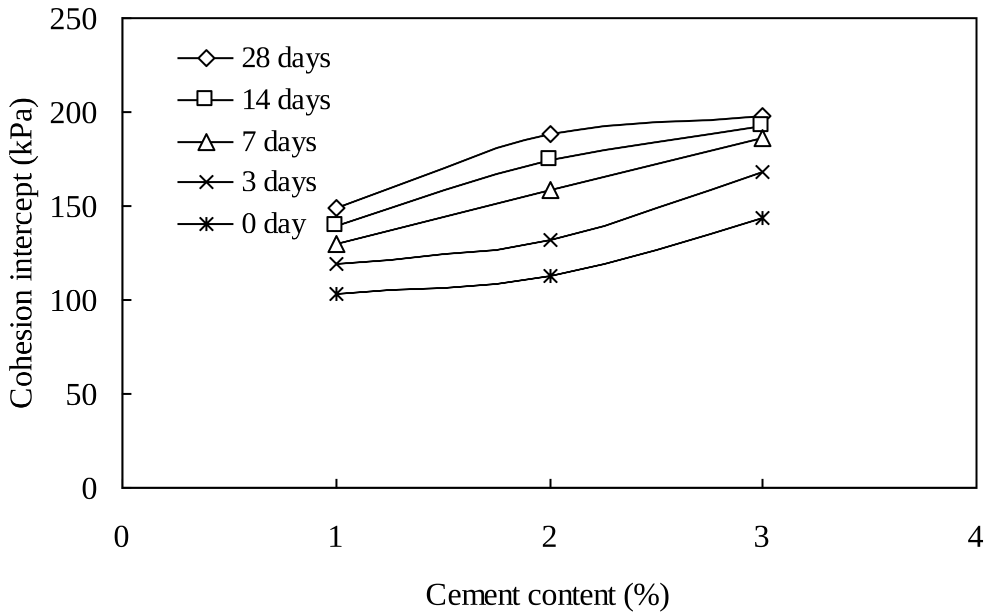


Fig. 8.6(g) Variation of cohesion of red soil-35FA-cement mixes at different curing periods

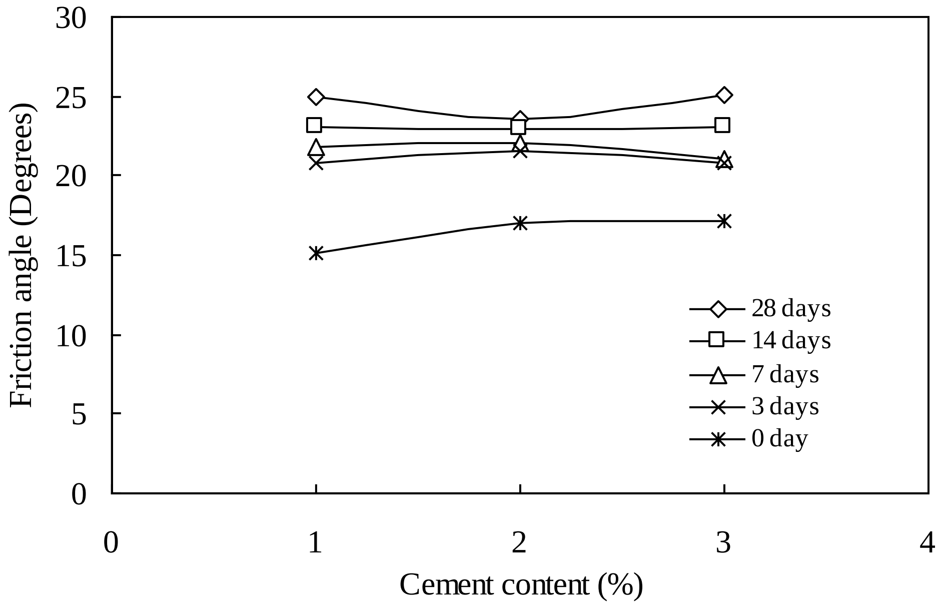


Fig. 8.6(h) Variation of friction angle of red soil-35FA-cement mixes at different curing periods

TH-1088_AKALITA

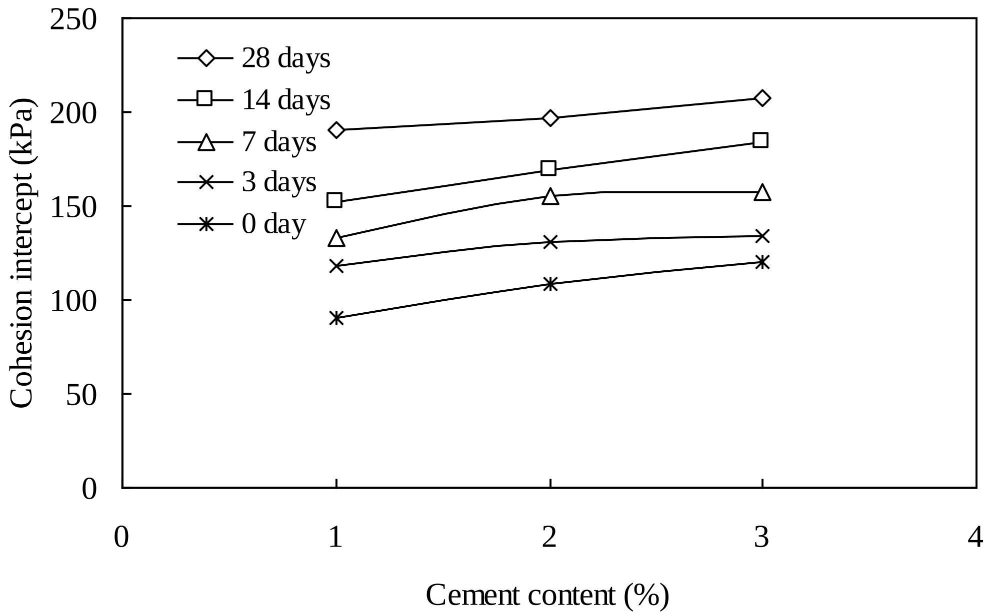


Fig. 8.6(i) Variation of cohesion of red soil-50FA-cement mixes at different curing periods

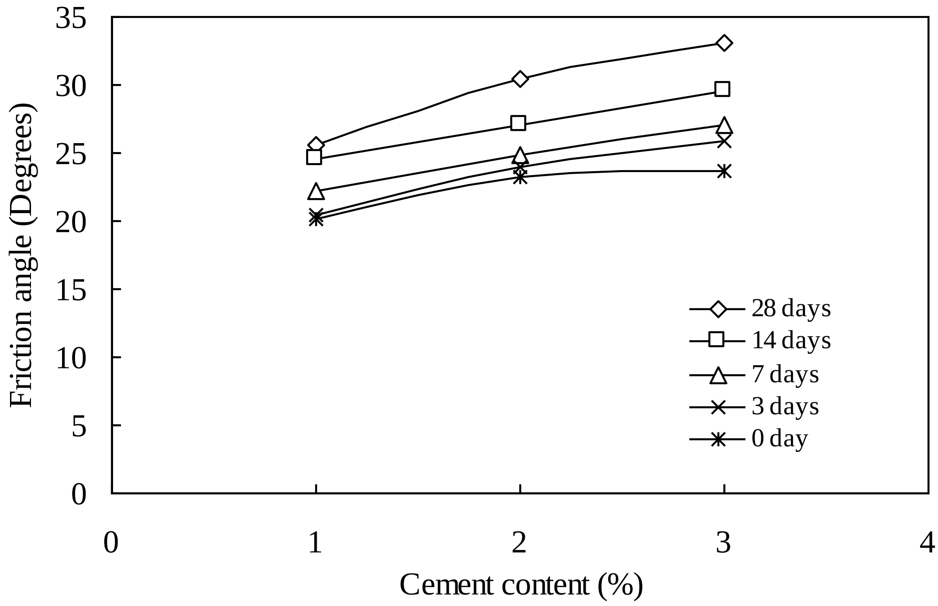


Fig. 8.6(j) Variation of friction angle of red soil-50FA-cement mixes at different curing periods

TH-1088_AKALITA

Table 8.4: Deviatoric strength (in kPa) of sand mixes at different curing periods and confining pressures

MIXES	0 days				3 days				7 days				14 days				28 days			
	Confining pressures (kPa)				Confining pressures (kPa)				Confining pressures (kPa)				Confining pressures (kPa)				Confining pressures (kPa)			
	100	200	300	400	100	200	300	400	100	200	300	400	100	200	300	400	100	200	300	400
BS+20FA	280	307	455	568	313	451	588	703	354	500	637	770	400	557	688	830	425	623	694	858
BS+35FA	308	402	632	609	356	500	668	748	385	541	700	824	419	576	725	853	440	646	748	887
BS+50FA	357	472	679	735	400	525	725	827	426	571	761	900	436	600	778	963	453	662	818	987
BS+1C	302	436	471	675	329	501	558	755	375	522	634	824	436	553	663	874	467	601	759	932
BS+2C	364	495	601	720	422	557	659	832	477	574	753	900	528	633	773	1003	532	725	911	1050
BS+3C	400	576	723	800	448	623	800	889	491	643	859	943	540	677	873	1040	563	826	981	1146
BS+20FA+1C	359	418	589	706	400	547	838	813	466	635	811	1006	537	729	913	1147	556	1020	1094	1184
BS+20FA+2C	454	523	704	852	456	651	866	938	555	773	979	1109	779	1003	1208	1359	919	1142	1332	1522
BS+20FA+3C	536	643	824	959	582	786	929	1075	655	1045	1118	1323	956	1261	1445	1633	1421	1713	1762	2200
BS+35FA+1C	327	457	577	780	425	525	734	889	488	572	781	961	548	708	826	1100	563	815	871	1169
BS+35FA+2C	381	500	698	892	511	647	934	1076	557	728	1084	1099	652	845	1187	1323	816	1052	1305	1580
BS+35FA+3C	443	567	773	941	626	818	1029	1231	754	994	1214	1464	843	1064	1393	1559	944	1289	1420	1820
BS+50FA+1C	321	487	575	737	413	564	668	896	462	623	716	965	471	651	802	980	523	791	1039	1076
BS+50FA+2C	405	570	703	808	501	667	780	951	538	750	857	1034	679	826	995	1230	736	999	1153	1370
BS+50FA+3C	474	696	777	944	548	780	965	1002	678	962	1150	1187	805	1226	1311	1438	1253	1542	1588	2100

TH-1088_AKALITA

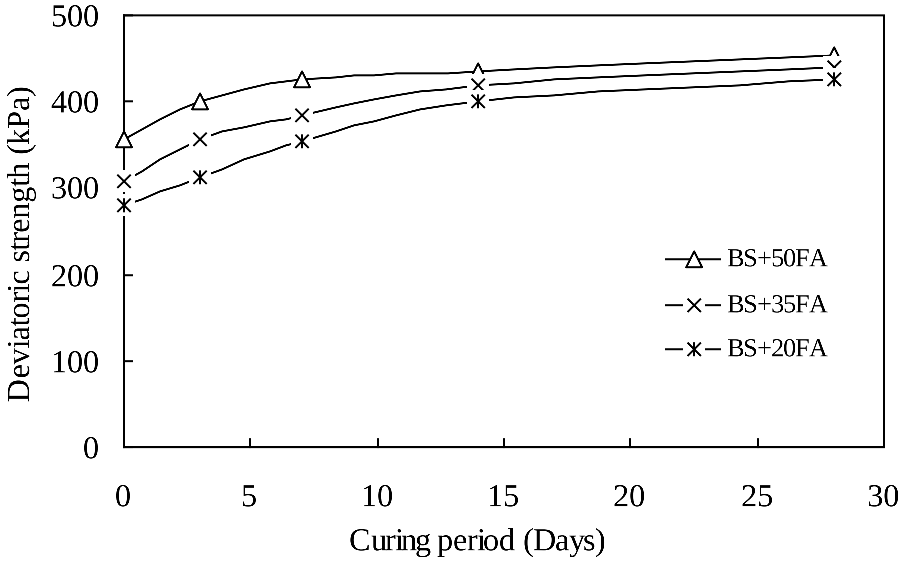


Fig. 8.7(a) Variation of deviatoric strength of sand-fly ash mixes with curing period (100 kPa confining pressure)

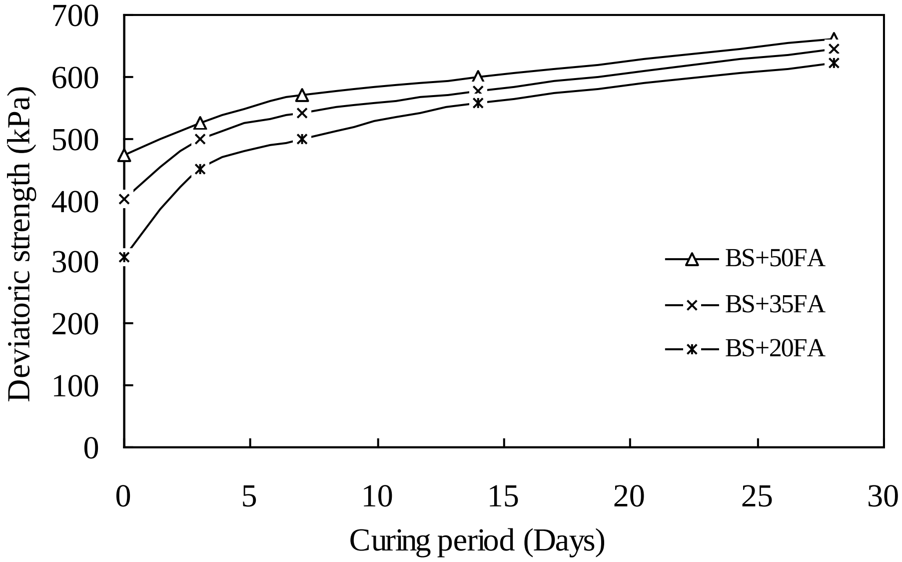


Fig. 8.7(b) Variation of deviatoric strength of sand-fly ash mixes with curing period (200 kPa confining pressure)

TH-1088_AKALITA

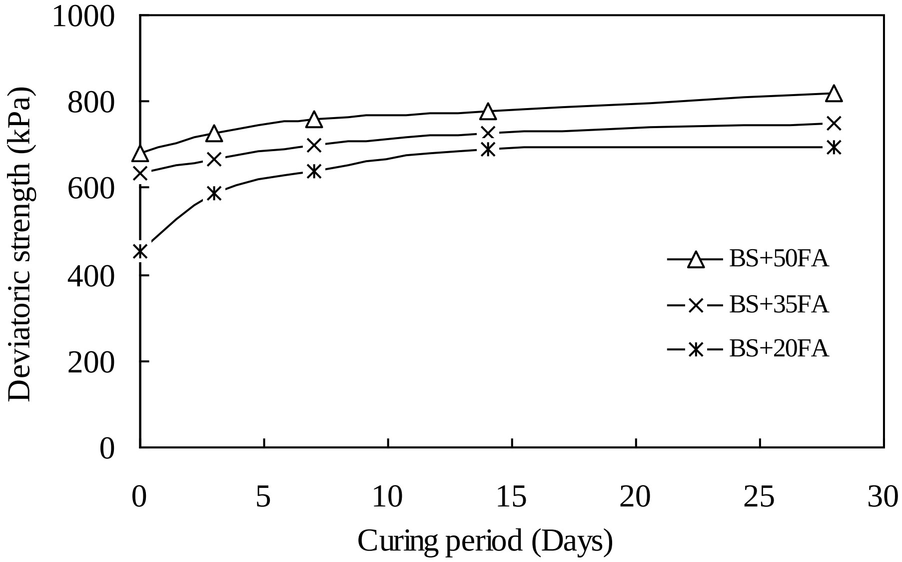


Fig. 8.7(c) Variation of deviatoric strength of sand-fly ash mixes with curing period (300 kPa confining pressure)

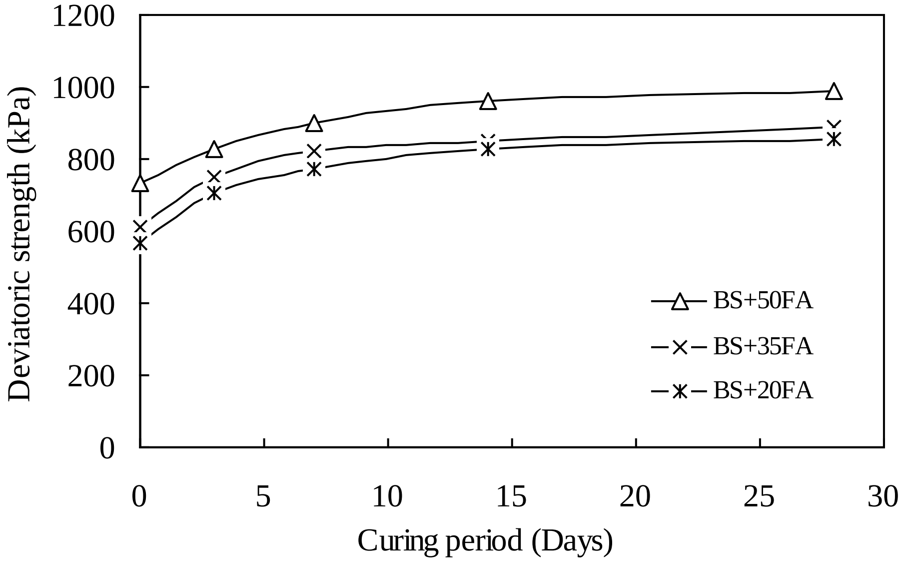


Fig. 8.7(d) Variation of deviatoric strength of sand-fly ash mixes with curing period (400 kPa confining pressure)

TH-1088_AKALITA

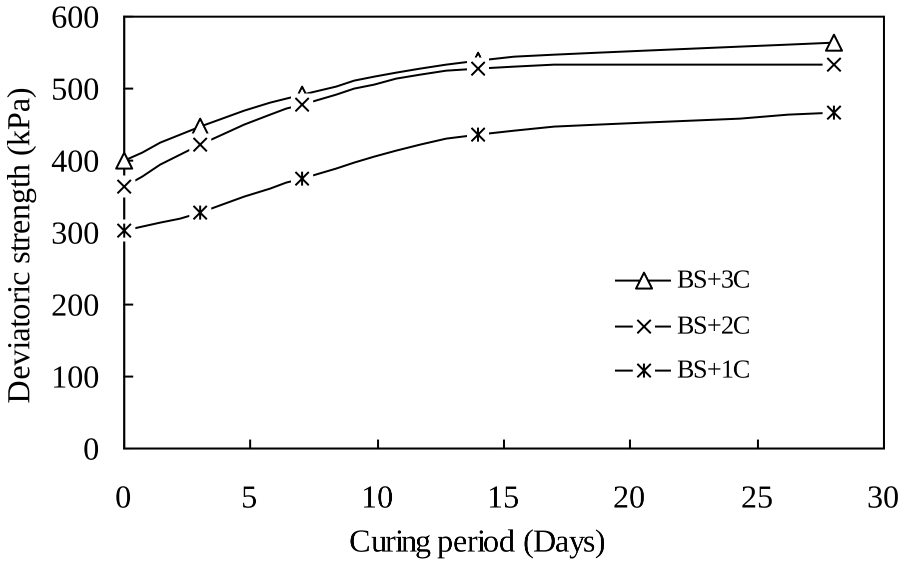


Fig. 8.8(a) Variation of deviatoric strength of sand-cement mixes with curing period (100 kPa confining pressure)

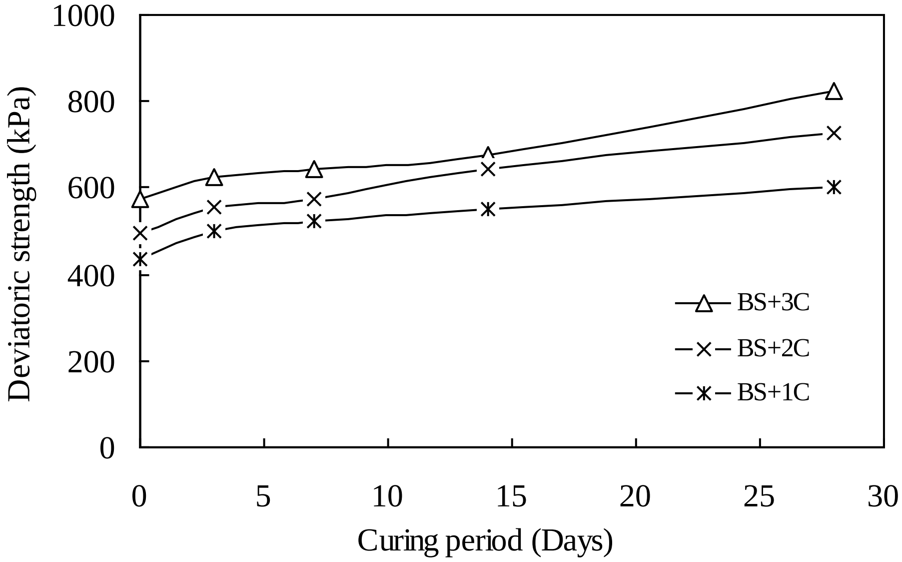


Fig. 8.8(b) Variation of deviatoric strength of sand-cement mixes with curing period (200 kPa confining pressure)

TH-1088_AKALITA

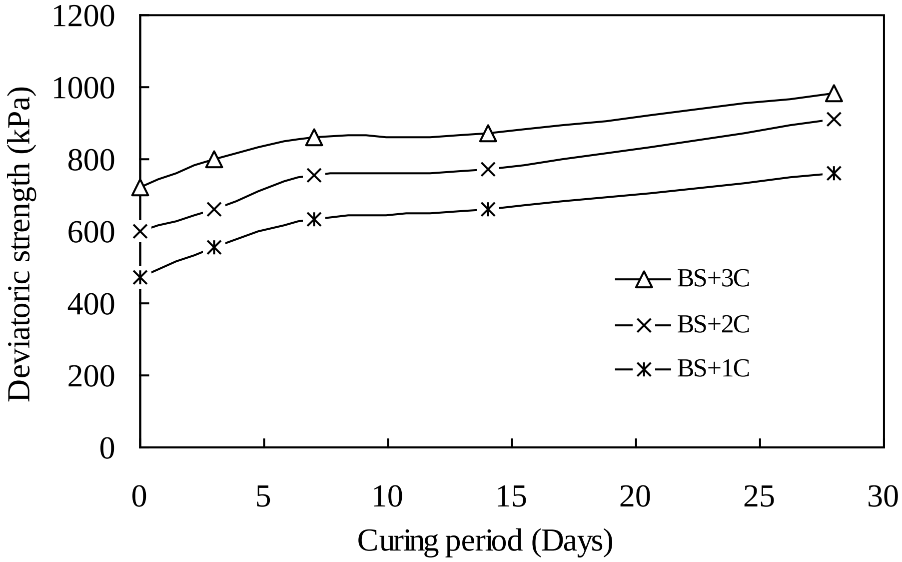


Fig. 8.8(c) Variation of deviatoric strength of sand-cement mixes with curing period (300 kPa confining pressure)

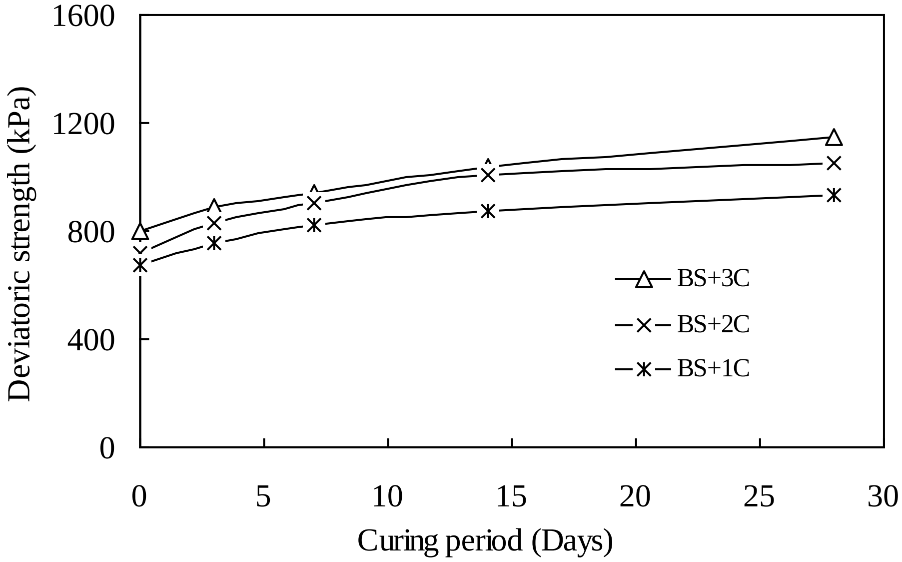


Fig. 8.8(d) Variation of deviatoric strength of sand-cement mixes with curing period (400 kPa confining pressure)

TH-1088_AKALITA

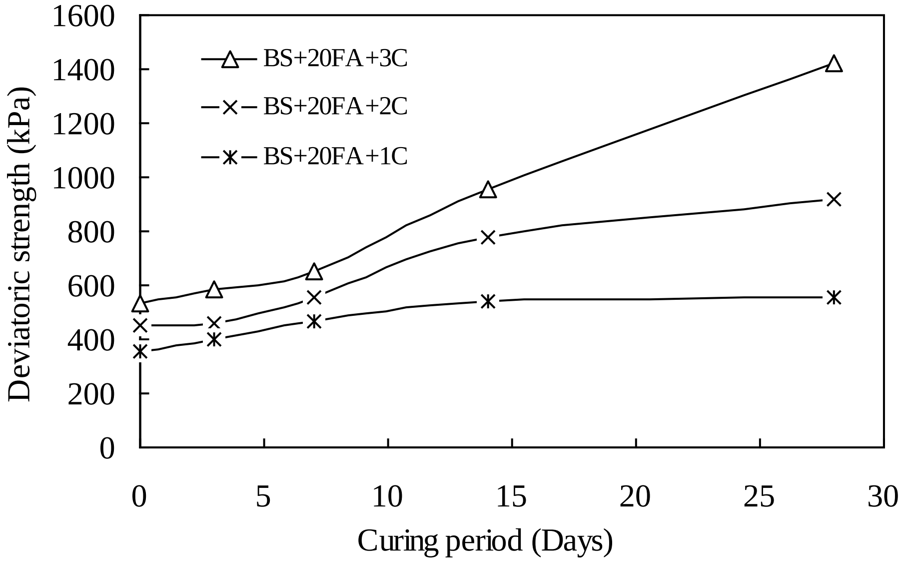


Fig. 8.9(a) Variation of deviatoric strength of sand-20FA-cement mixes with curing period (100 kPa confining pressure)

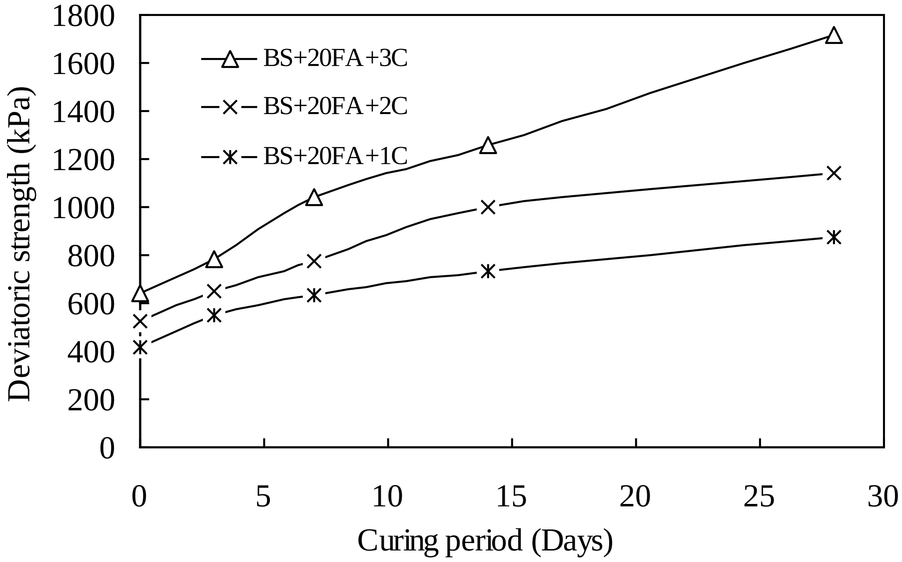


Fig. 8.9(b) Variation of deviatoric strength of sand-20FA-cement mixes with curing period (200 kPa confining pressure)

TH-1088_AKALITA

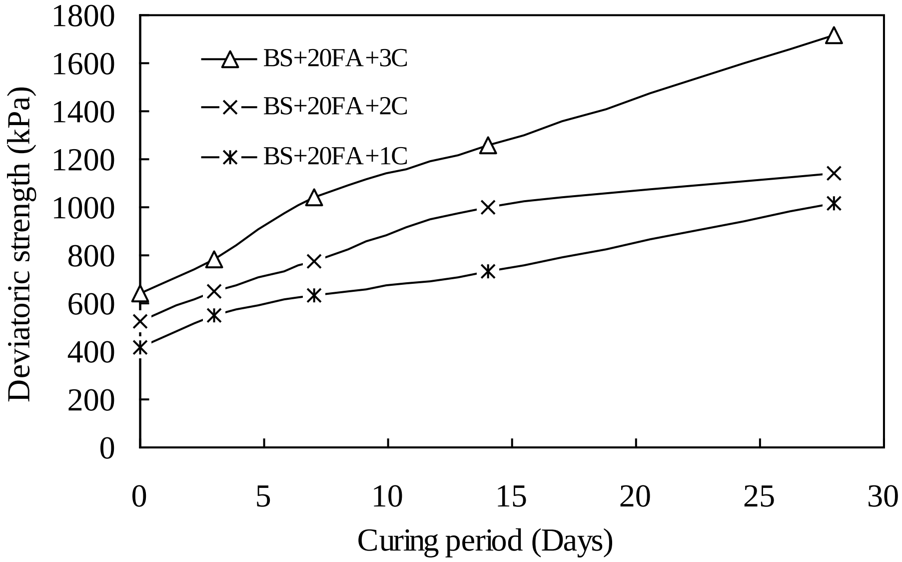


Fig. 8.9(c) Variation of deviatoric strength of sand-20FA-cement mixes with curing period (300 kPa confining pressure)

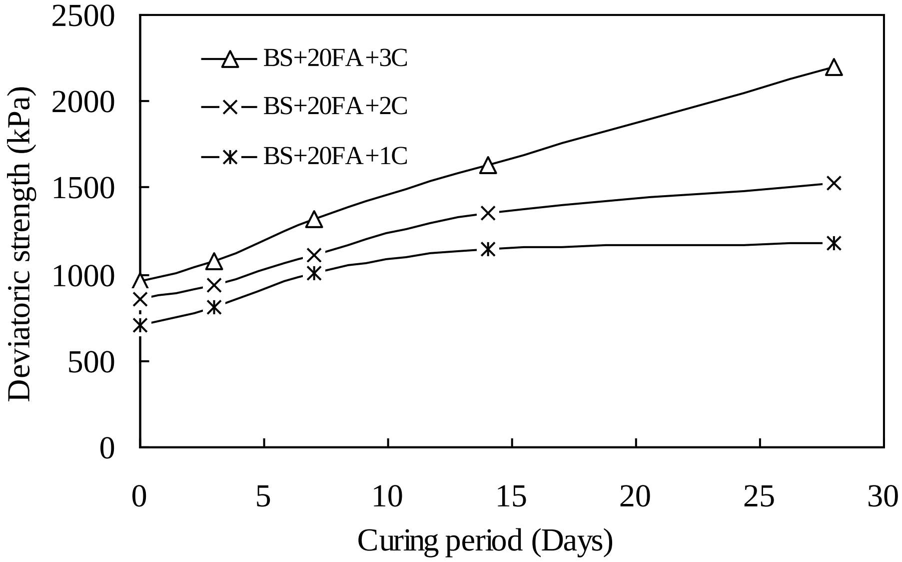


Fig. 8.9(d) Variation of deviatoric strength of sand-20FA-cement mixes with curing period (400 kPa confining pressure)

TH-1088_AKALITA

BS+20FA+1C BS+20FA+2C BS+20FA+3C

Fig. 8.9(f BS ime uring
0 kP e)

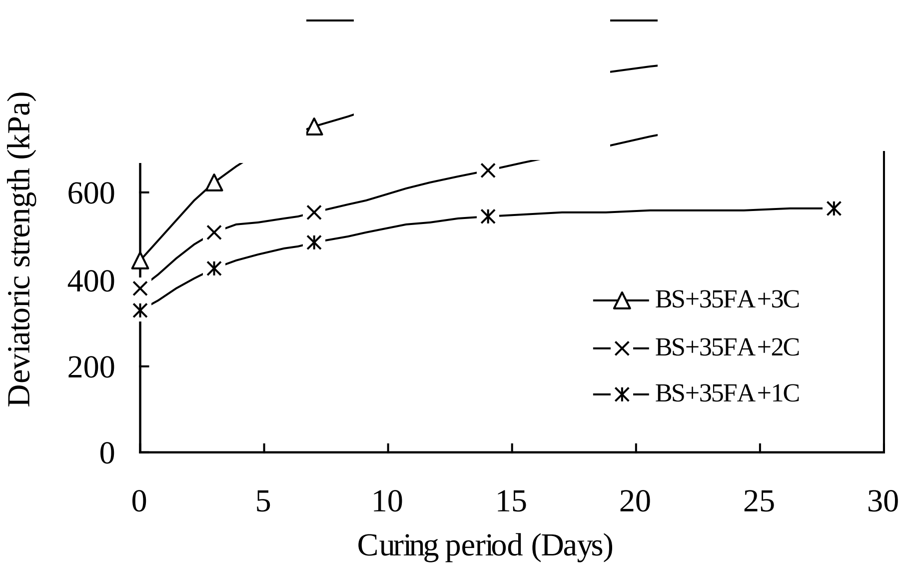


Fig. 8.10(a) Variation of deviatoric strength of sand-35FA-cement mixes with curing period (100 kPa confining pressure)

TH-1088_AKALITA

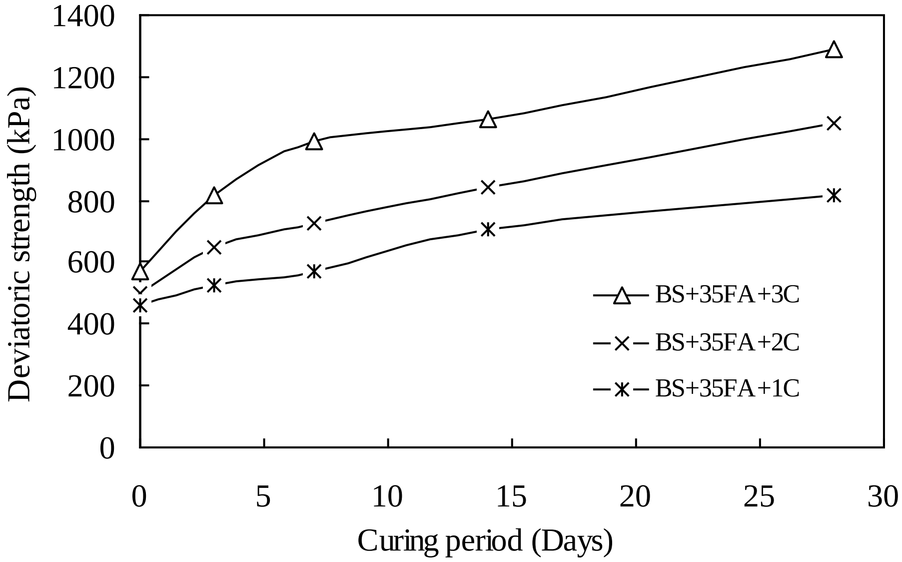
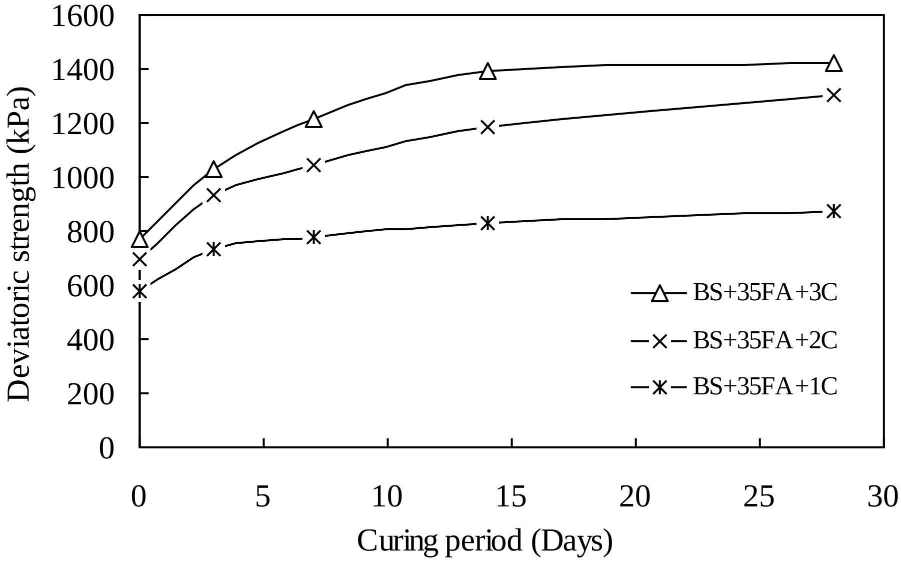


Fig. 8.10(b) Variation of deviatoric strength of sand-35FA-cement mixes with curing period (200 kPa confining pressure)



=

Fig. 8.10(c) Variation of deviatoric strength of sand-35FA-cement mixes with curing period (300 kPa confining pressure)

TH-1088_AKALITA

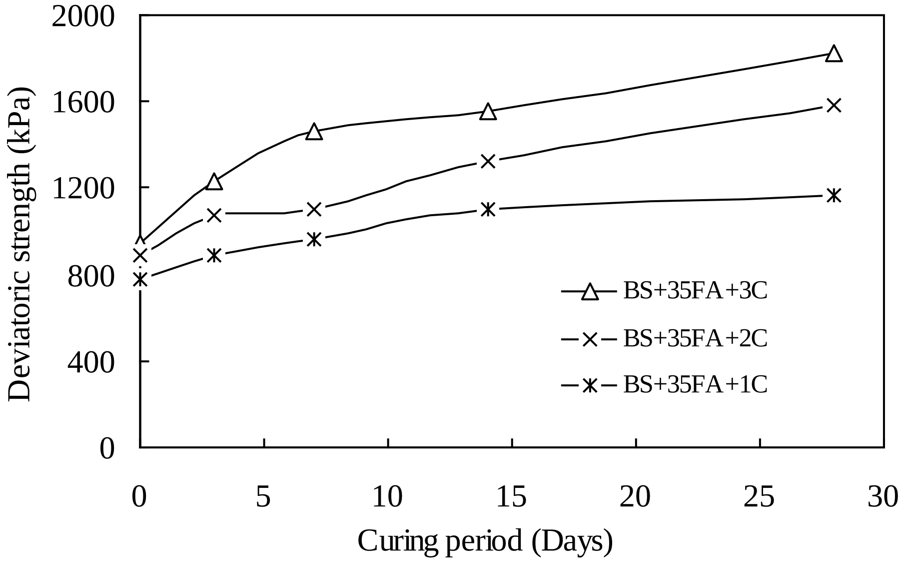


Fig. 8.10(d) Variation of deviatoric strength of sand-35FA-cement mixes with curing period (400 kPa confining pressure)

BS+35FA+1C BS+35FA+2C BS+35FA+3C

Fig. tterns of s specimer ring
(100 k sure)

TH-1088_AKALITA

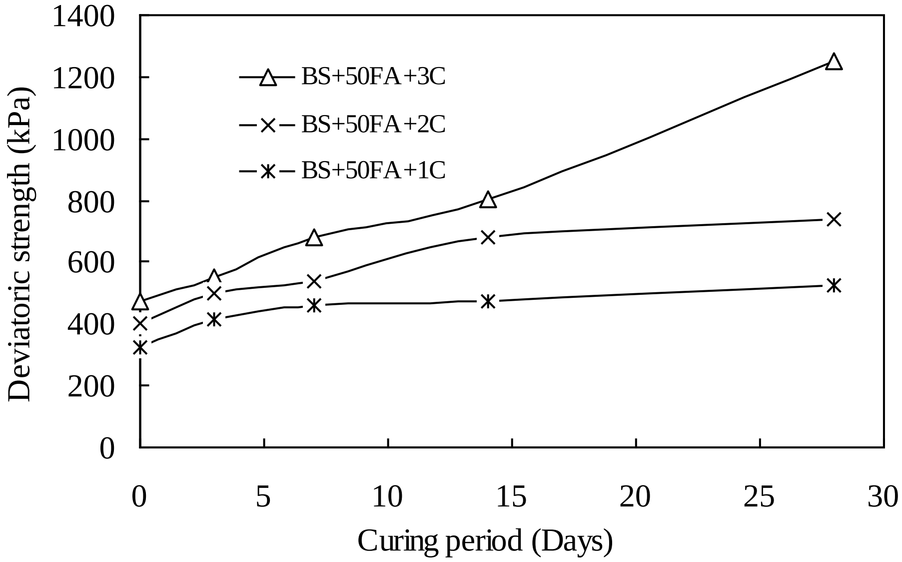


Fig. 8.11(a) Variation of deviatoric strength of sand-50FA-cement mixes with curing period (100 kPa confining pressure)

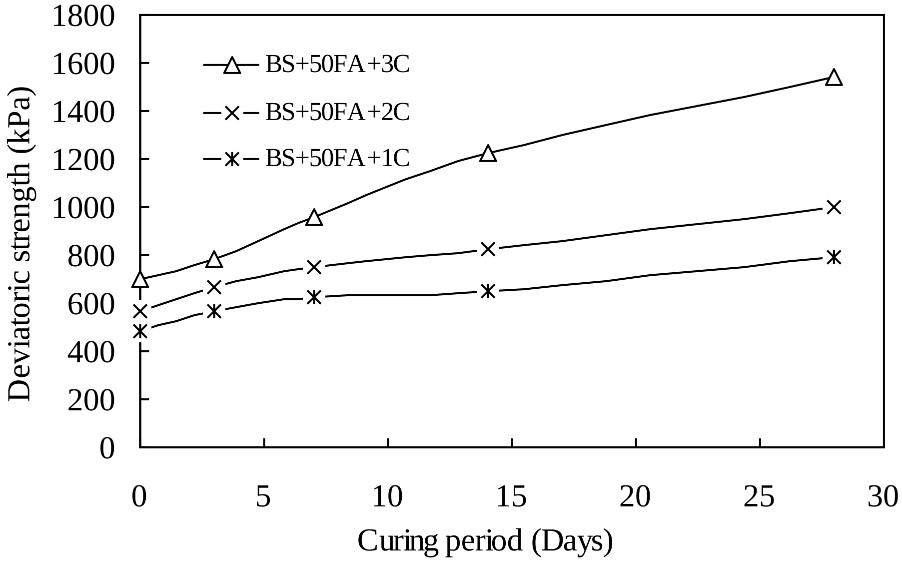


Fig. 8.11(b) Variation of deviatoric strength of sand-50FA-cement mixes with curing period (200 kPa confining pressure)

TH-1088_AKALITA

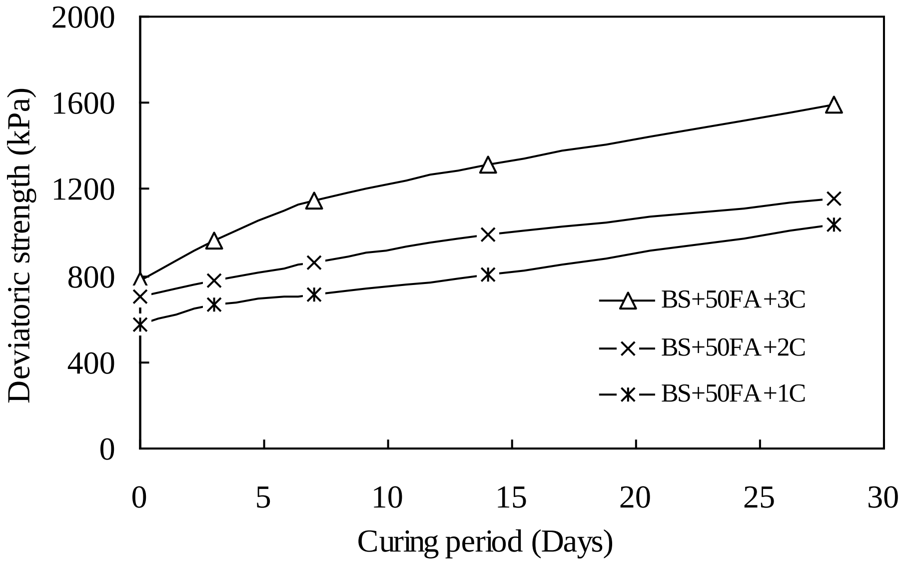


Fig. 8.11(c) Variation of deviatoric strength of sand-50FA-cement mixes with curing period (300 kPa confining pressure)

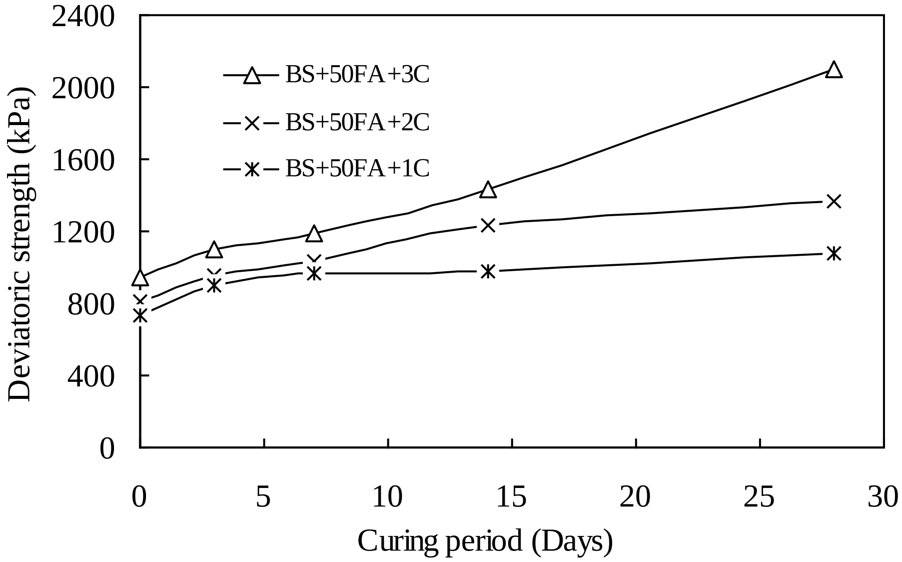


Fig. 8.11(d) Variation of deviatoric strength of sand-50FA-cement mixes with curing period (400 kPa confining pressure)

BS+50FA+1C BS+50FA+2C BS+50FA+3C

Fig. 8.1 of ɔecimɛ ıring
00 ıre)

TH-1088_AKALITA

Table 8.5: Drained shear strength parameters of various mixes of sand

MIXES	0 days		3 days		7 days		14 days		28 days	
	c_d (kPa)	ϕ_d (°)	c_d (kPa)	ϕ_d (°)	c_d (kPa)	ϕ_d (°)	c_d (kPa)	ϕ_d (°)	c_d (kPa)	ϕ_d (°)
BS+20FA	50.6	19.8	61.46	23.2	70.8	24.1	76.95	24.5	91.9	24.5
BS+35FA	62.9	22	74.8	23.8	77.19	25.1	89.46	24.8	100.97	24.9
BS+50FA	71.47	23.9	78.15	26.5	80.66	27.4	86.31	26.4	96.59	26.5
BS+1C	59.15	21.8	64.14	23.6	77.41	24	86.87	24.7	94.13	25.9
BS+2C	85.21	21.7	92.59	23.6	99.6	24.9	104.8	26.2	111.21	27.7
BS+3C	92.46	23.8	98.34	25.5	104.62	26.2	108.84	27.3	116.76	29.3
BS+20FA+1C	70.89	22.3	74.44	26.6	83.79	28.2	94.1	30.1	117.66	31.4
BS+20FA+2C	92.37	24.2	93.15	27.2	113.23	28.9	174.38	29.6	193.29	31.6
BS+20FA+3C	120.19	24.9	134.5	26.6	138.51	31.3	212.66	31.8	306.93	33.7
BS+35FA+1C	51.64	25.2	74.27	26.5	88.71	26.8	103.03	28.2	105.77	29.3
BS+35FA+2C	55.16	27.7	83.94	30	100.41	32.3	109.97	32.9	146.43	34.0
BS+35FA+3C	77.3	27.4	120.55	30.2	141.41	32.3	157.9	33.7	170.14	35.7
BS+50FA+1C	104.5	23.6	63.45	24	75.75	26.6	87.87	27.1	93.56	29.8
BS+50FA+2C	156.49	23.7	92.89	25	114	26.4	121.87	28.7	140.91	30.5
BS+50FA+3C	240	25.4	108.95	26.4	131.04	28.2	162.84	31	187.39	35.3

TH-1088_AKALITA

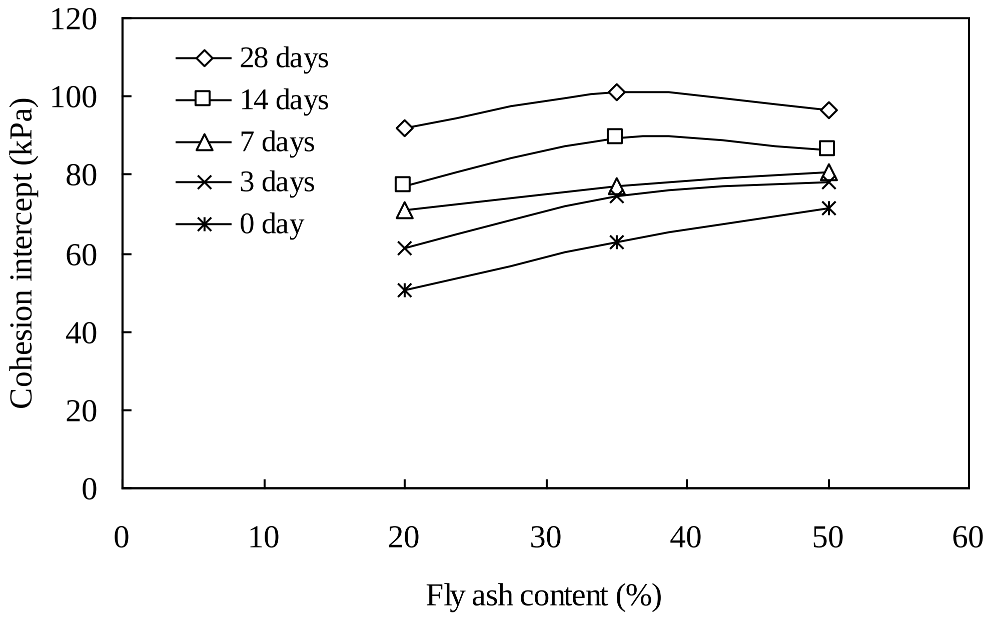


Fig. 8.12(a) Variation of cohesion of sand-fly ash mixes at different curing periods

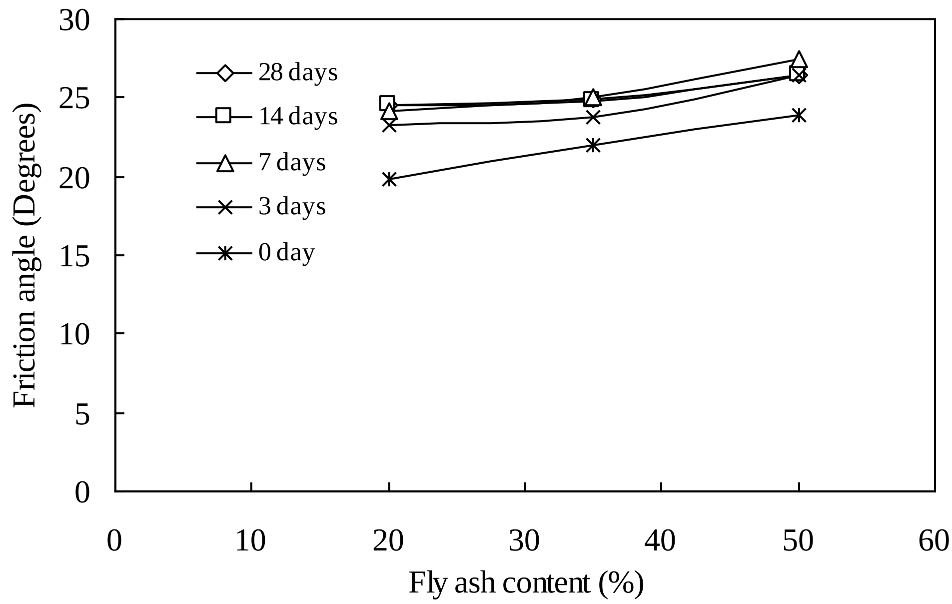


Fig. 8.12(b) Variation of friction angle of sand-fly ash mixes at different curing periods

TH-1088_AKALITA

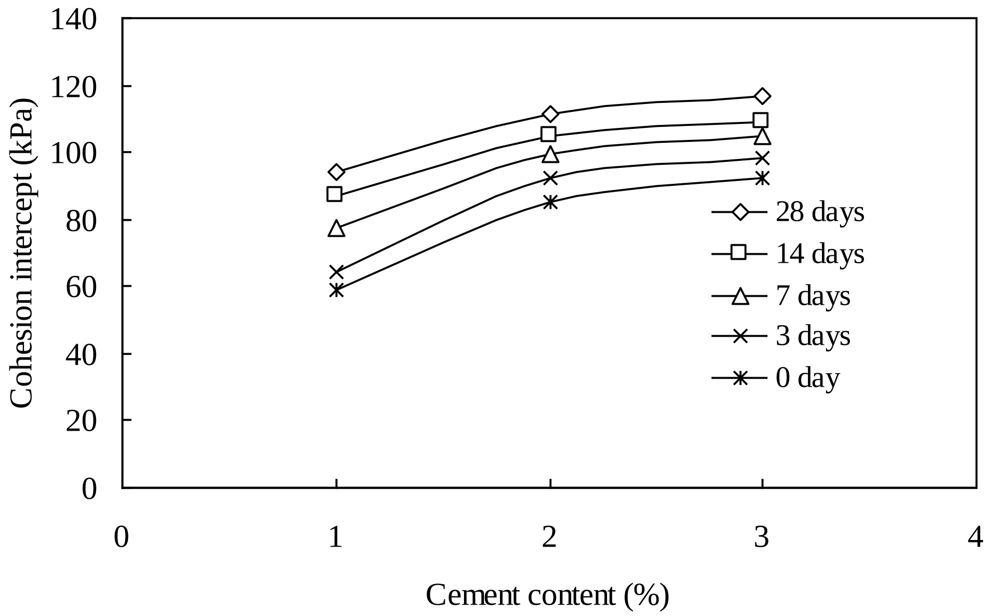


Fig. 8.12(c) Variation of cohesion of sand-cement mixes at different curing periods

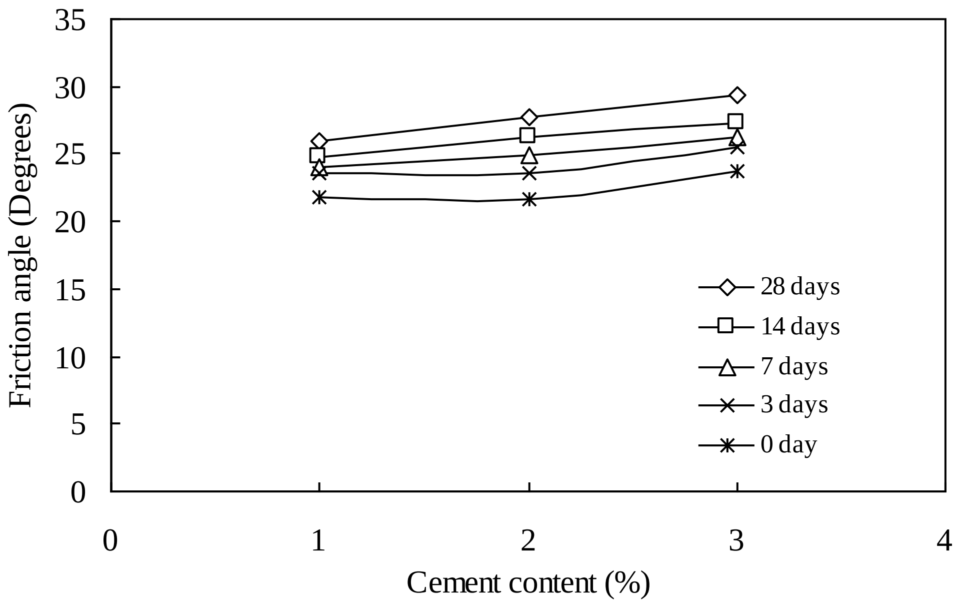


Fig. 8.12(d) Variation of friction angle of sand-cement mixes at different curing periods

TH-1088_AKALITA

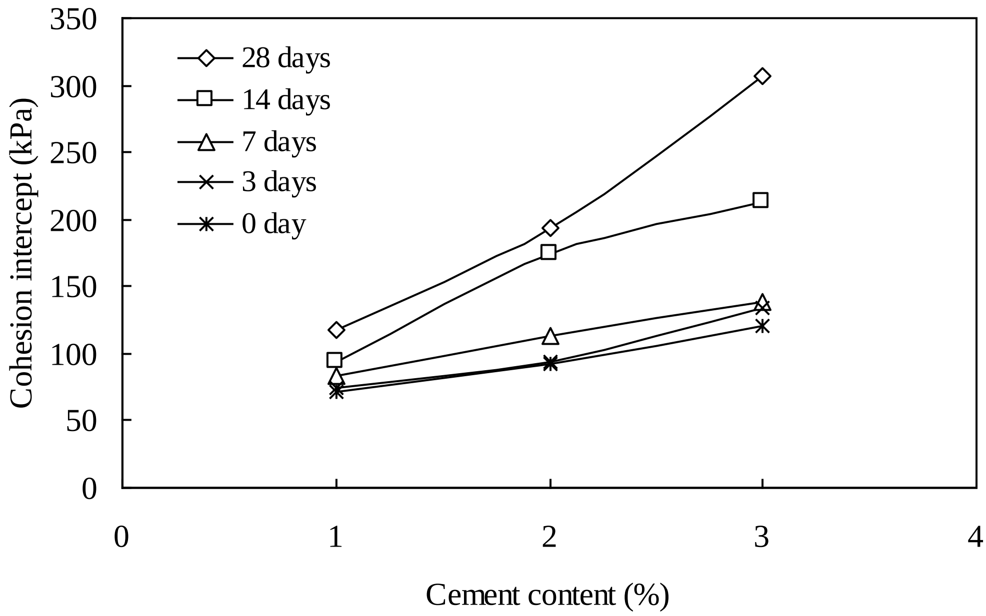


Fig. 8.12(e) Variation of cohesion of sand-20FA-cement mixes at different curing periods

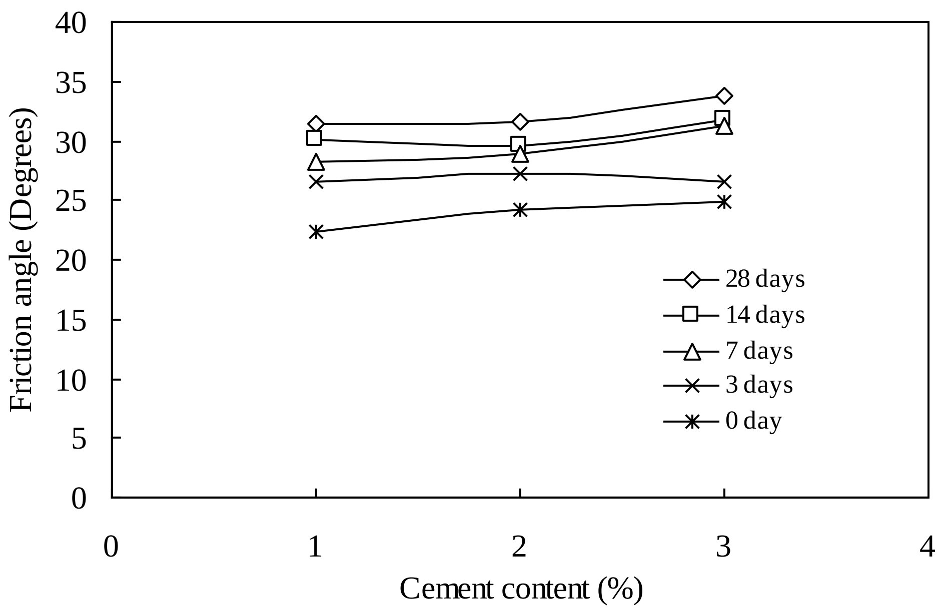


Fig. 8.12(f) Variation of friction angle of sand-20FA-cement mixes at different curing periods

TH-1088_AKALITA

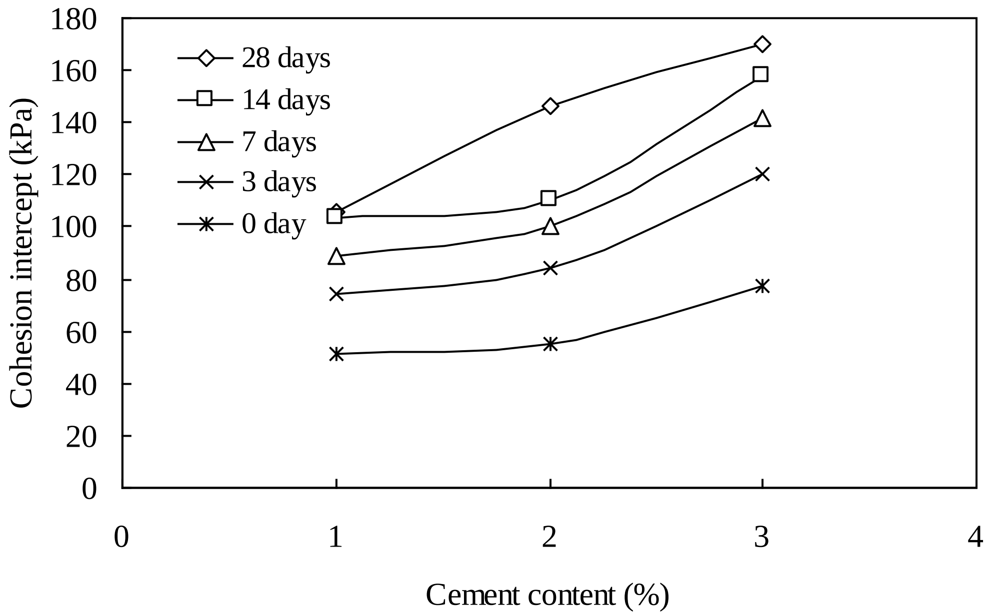


Fig. 8.12(g) Variation of cohesion of sand-35FA-cement mixes at different curing periods

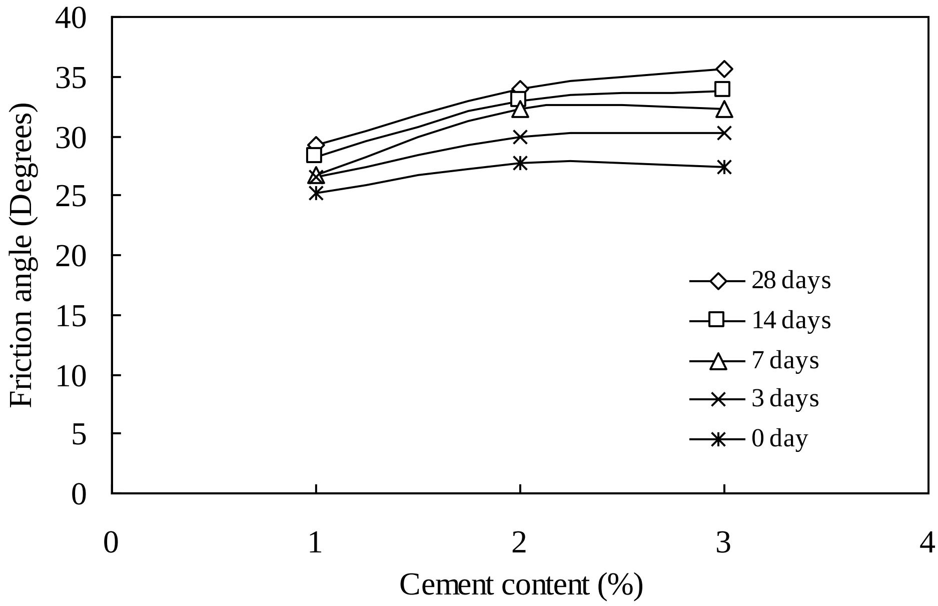


Fig. 8.12(h) Variation of friction angle of sand-35FA-cement mixes at different curing periods

TH-1088_AKALITA

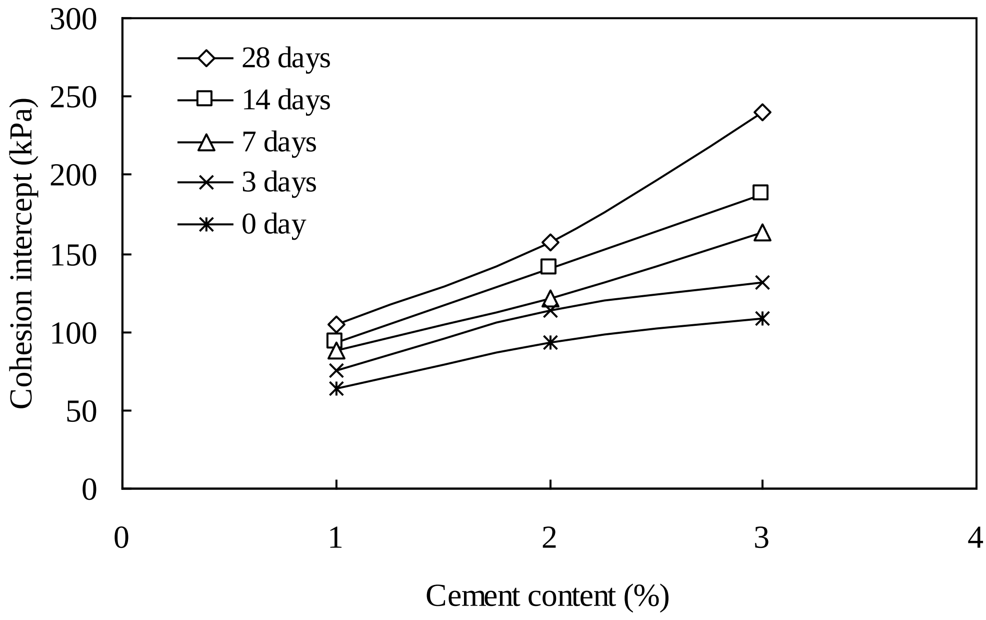


Fig. 8.12(i) Variation of cohesion of sand-50FA-cement mixes at different curing periods

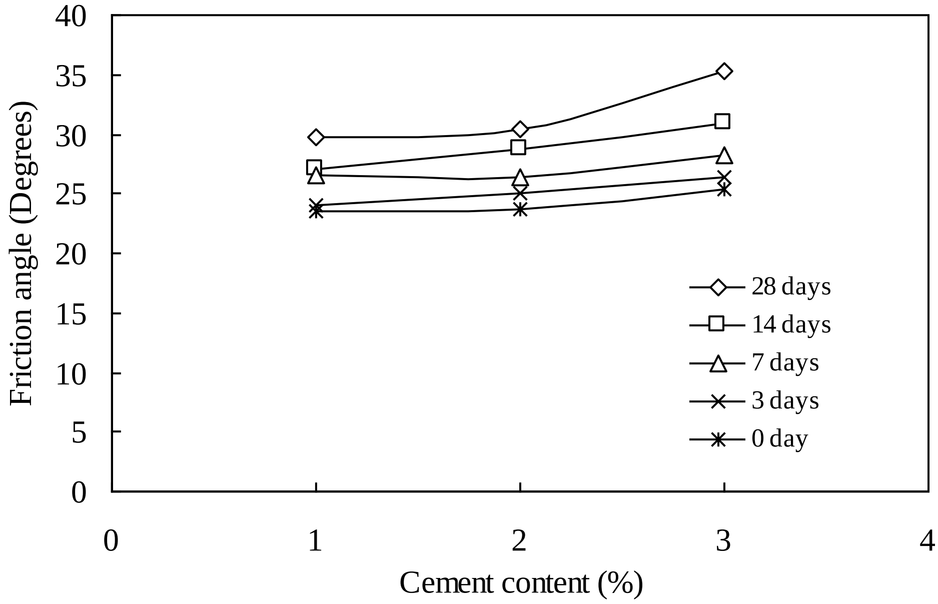


Fig. 8.12(j) Variation of friction angle of sand-50FA-cement mixes at different curing periods

8.4 Conclusions

Addition of fly ash or cement to either the fine-grained soil or the granular soil increases the shear strength parameters. Curing is noted to enhance them further. The increase in cohesion intercept reflects the increase in cementation through bonding

TH-1088_AKALITA

between the soil grains, whereas the increase in angle of shearing resistance is attributed to the change in texture and gradation of the mix. Upon saturation, any apparent cohesion will be reduced and hence the strength parameter of major importance is the friction angle which will remain unchanged. As the level of cementation increases, a higher deviatoric stress is required to cause failure. The increase in non-cohesive fraction in the mix, which decreases unconfined strength, has led to strength increase when tested under confined condition.

When only fly ash is added and then cured, the increase in cohesion of the fine-grained red soil is higher than that of the granular sand. After 28 days curing, the c_d and ϕ_d values of RS+50FA and BS+50FA mixes are 137.77 kPa, 27.8° and 96.59 kPa, 26.5°, respectively. For the BS+50FA mix with no curing, the c_d and ϕ_d values are 71.47 kPa and 23.9°. In the direct shear test which is conducted on uncured specimens, the corresponding cohesion intercept and friction angle values of the BS+50FA mix are only 24.03 kPa and 21.8°. This shows that the confinement has substantially increased the cohesion intercept.

When only cement is added and after 28 days curing, the c_d and ϕ_d values of RS+3C and BS+3C mixes are 256.99 kPa, 19.5° and 116.76 kPa, 29.3°, respectively. The shows that the cement has increased the cohesion more in the fine-grained red soil, and the friction angle by a larger margin in the granular sand. Similar trends are observed under the combined influence of fly ash and cement. After curing for 28 days, the c_d and ϕ_d values of RS+50FA+3C and BS+50FA+3C mixes are 207.9 kPa, 33.1° and 187.39 kPa, 35.3°, respectively.

The triaxial tests results indicate that the compacted specimens of the mixes have strength gain within short-term curing also when confined and that the strength continues to increase with curing time.

TH-1088_AKALITA

CHAPTER-9
SUMMARY AND CONCLUSIONS

9.1 Summary

In this research work, the characteristics of two soils, a fine-grained residual lateritic soil (red soil) and granular riverbank sand (Brahmaputra sand) blended with a low-calcium fly ash and ordinary Portland cement was studied through a systematic series of compaction tests, direct shear tests, CBR tests, unconfined compression tests and triaxial consolidated drained tests. Depending on the type of test, the maximum amount of soil replaced with fly ash by weight ranged from 50% to 90%. The amount of cement added was up to a maximum of 3 % to 5% by weight of the soil-fly ash mixes. This provided a wide range of gradation and texture of the mixes. Compacted specimens of the mixes were cured up to 28 days. Based on the maximum dry unit weight obtained from the compaction tests, the specimens for the remaining tests were prepared. The strength tests were conducted on the as-compacted specimens without saturating them. Direct shear tests were carried out only on specimens of sand and sand-fly ash mixes.

The results from the tests were analysed to examine the compaction and strength characteristics of the modified soils due to the application of fly ash and cement. The obtained values will also be useful in judging the suitability of those proportions of the soil mixes for their use in diverse geotechnical applications that include the construction of subgrade and subbase of pavements. As the fly ashes produced in the same plant at different times can be different, and as locally available soils can vary widely in their properties, characterization of their mixes is a must for their field usage.

TH-1088_AKALITA

9.2 Conclusions

Based on the laboratory test results and the analyses carried out, the following conclusions have been drawn:

Compaction characteristics

1. The lower specific gravity and poorly graded particles of the fly ash along with the presence of cenospheres are dominating factors controlling the compaction behaviour of the soil mixes.
2. As fly ash is added to either soil type, the maximum dry unit weight decreases significantly with increasing fly ash content. The MDD values of red soil-fly ash mixes are lower than that of sand-fly ash mixes at the same fly ash content, whereas the corresponding OMC values of red soil-fly ash mixes are higher. The variation of dry unit weight with moisture content of sand-fly ash mixes is less compared to that of red soil-fly ash mixes.
3. When cement is added to the above mixes, the same trends are observed while comparing MDD and OMC values of mixes of the two soils.
4. The cohesive red soil has a relatively higher unit weight than the cohesionless sand under a given compactive effort. The red soil particles cannot be packed as compactly as the sand particles by virtue of either double layer repulsion or flocculent nature of the soil fabric. With the result, their water holding capacity is also relatively more. Hence, red soil mixes exhibit a lower MDD and a higher OMC.

For selection of the appropriate mix proportions for practical applications, the development and variation of strength of the specimens, compacted at the respective MDD and OMC values, have been investigated in uncured and cured states as well as under both unconfined and confined conditions through various strength tests.

Direct shear strength characteristics

1. When fly ash is added to the sand, the cohesion component of shear strength increases whereas the frictional component decreases. The increase in cohesion is more prominent at higher fly ash contents. However, the overall shear strength of sand-fly ash mixes decreases with increasing fly ash content.
2. The mixes exhibit similar trends of variation in vertical vs. shear displacement indicating an initial contraction followed by dilation. As the fly ash content of the mix increases, the dilatancy effect becomes progressively lesser than that of sand alone. At any fly ash content, the expansion in volume decreases generally with increasing normal stress.

The existence of cohesion intercept is due to the presence of capillary stresses in the compacted specimens whose gradation and texture have changed. In order to understand the contribution of development of cementation between grains on the shear strength of soil-fly ash mixes treated with cement, CBR, unconfined compression and triaxial tests have been conducted on compacted specimens after various stages of curing.

CBR characteristics

1. Only specific proportions of fly ash added to either soil can enhance the CBR values considerably, due to improved gradation of the mix and better grain size packing. There are two optimum fly ash contents (20% and 50%) at which red soil-fly ash mixes exhibit higher CBR values under soaked conditions. Similarly, there are two optimum fly ash contents (10% and 35%) at which sand-fly ash mixes show higher soaked CBR values. When the fly ash addition is less than 50%, soaked CBR values of all red soil-fly ash mixes are lower than those of sand-fly ash mixes.
2. The soaked CBR values of the soil-fly ash mixes improve on further addition of a small amount of cement. Sand-fly ash-cement mixes show considerably higher soaked CBR

values than red soil-fly ash-cement mixes at fly ash contents of 50% and 65%.

Minimum soaked CBR values of 6% and 20% are necessary for use in the subgrade and subbase layers of low-volume flexible pavements. With no cement added, red soil-fly ash mixes with 20% or 50% fly ash content and sand-fly ash mixes up to 50% fly ash content may be used for subgrade layer. With 1% cement added, both the soil-fly ash mixes with 50% fly ash content can be considered for subbase layer.

Further, the strength characteristics of the compacted specimens, cured for longer periods beyond 4 days, have been determined through unconfined compression and triaxial tests.

Unconfined strength characteristics

1. In unconfined compression tests, the strength value shown is by virtue of the cohesive component of the shear strength of the mix, and the frictional strength is not depicted. Compacted sand exhibits some strength on account of the apparent cohesion.
2. Addition of fly ash to either soil can enhance the unconfined strength significantly. For the red soil, 28 days UCS values are observed to be the highest for mixes with 35% fly ash content followed by 20% fly ash content. There is no benefit of fly ash addition above 50%, and the UCS values of these mixes remain less than that of the red soil alone. Addition of cement to the red soil significantly improves the UCS. The 28 days UCS of RS+35FA and RS+2C mixes are comparable, whereas the UCS of the RS+35FA+1C mix is greater than either of the two mixes.
3. For the sand, the UCS progressively increases with fly ash content and also with curing duration. As cement is added to the sand, the unconfined strength is also increased. The 28 days UCS of BS+80FA mix is higher than the UCS of the BS+3C mix, but is considerably lower than that of BS+80FA+1C mix.

4. Addition of more amount of cement to soil is not an economical option. The presence of fly ash is fundamental to improve the strength of the soil mix by facilitating time-dependent pozzolanic reactions. To allow this, maximum replacement of the red soil and sand by 35% and 80% respectively can be made so as to satisfy strength requirements.
5. At the respective optimum mix proportions of fly ash, the unconfined strength of the red soil mixes is found to be higher than the sand mixes. Upon addition of cement, the additional increase in unconfined strength is generally higher for the sand-fly mixes.

To assure that the soil mix has the potential to perform satisfactorily in the field, it should satisfy strength criteria for the type of application based on the 7 days UCS.

However, as there are limitations in using only the unconfined test to assess the shear strength of the compacted and cured specimens, the more comprehensive method of triaxial testing has been performed for better evaluation of strength characteristics of the mixes.

Shear strength characteristics

1. Addition of fly ash or cement to either the fine-grained soil or the granular soil increases the shear strength parameters. Curing is noted to enhance them further. The increase in cohesion intercept reflects the increase in cementation through bonding between the soil grains, whereas the increase in angle of shearing resistance is attributed to the change in texture and gradation of the mix. Upon saturation, any apparent cohesion will be reduced and hence the strength parameter of major importance is the friction angle which will remain unchanged.
2. When only fly ash is added and then cured, the increase in cohesion of the fine-grained red soil is higher than that of the granular sand. After 28 days curing, the c_d and ϕ_d values of RS+50FA and BS+50FA mixes are 137.77 kPa, 27.8° and 96.59 kPa, 26.5°, respectively. For the BS+50FA mix with no curing, the c_d and ϕ_d values are 71.47 kPa

and 23.9°.

3. From direct shear test on uncured specimens of the BS+50FA mix, the corresponding cohesion intercept and friction angle values are only 24.03 kPa and 21.8°. This shows that the confinement has substantially increased the cohesion intercept.
4. The addition of only cement and curing have increased the cohesion more in the fine-grained red soil, and the friction angle by a larger margin in the granular sand. After 28 days curing, the c_d and ϕ_d values of RS+3C and BS+3C mixes are 256.99 kPa, 19.5° and 116.76 kPa, 29.3°, respectively.
5. Similar trends are observed under the combined influence of fly ash and cement. After curing for 28 days, the c_d and ϕ_d values of RS+50FA+3C and BS+50FA+3C mixes are 207.9 kPa, 33.1° and 187.39 kPa, 35.3°, respectively.

The triaxial tests results indicate that the compacted specimens of the mixes have strength gain within short-term curing also when confined and that the strength continues to increase with curing time.

9.3 Scope for Further Research

The following suggestions are made for any further work in this area:

- Consolidated undrained tests on similar mixes with measurement of pore water pressure to obtain effective shear strength parameters.
- Resilient modulus tests on similar mixes to evaluate the subgrade modulus that is important for supporting long-term vehicular traffic loads.

REFERENCES

1. Anirudhan, I. V. (1998). “Lateritic soils in engineering practice.” *Published by Indian Geotechnical Society*, Madras Chapter, Chennai, India.

2. ASTM C 618-99 (1999). “Standard specification for coal fly ash and raw or calcined natural pozzolana for use as a mineral admixture in Portland cement concrete.” *Annual Book of ASTM Standards*, ASTM, Philadelphia.

3. Baghdadi, Z. A. and Shihata, S. A. (1999). “On the durability and strength of soil-cement.” *Ground Improvement*, 3, 1-6.

4. Bera, A. K., Ghosh, A. and Ghosh, A. (2007). “Compaction characteristics of pond ash.” *Journal of Materials in Civil Engineering*, ASCE, 19(4), 349-357.

5. Blight, G. E. (1997). “Mechanics of residual soils.” Balkema, Rotterdam, The Netherlands.

6. Chu, S. C., and Kao, H. S. (1993). “A study of engineering properties of a clay modified by fly ash and slag.” *Fly Ash For Soil Improvement*, ASCE Geotechnical Special Publication No. 36, 89–99.

7. Cokca, E. (1997). “Frost susceptibility properties of soma-B fly ash.” *Journal of Energy Engineering*, ASCE, 123(1), 1-10.

8. Cokca, E. (2001). “Use of class C fly ashes for the stabilization of an expansive soil.” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 127(7), 568-573.

9. Consoli, N. C., Prietto, P. D. M. and Ulbrich, L. A. (1998). “Influence of fiber and cement addition on behavior of sandy soil.” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 124(12), 1211-1214.

10. Consoli, N. C., Prietto, P. D. M., Carraro, J. A. H. and Heineck, S. (2001). “Behavior of compacted soil-fly ash-carbide lime mixtures.” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 127(9), 774-782.

TH-1088_AKALITA

11. Consoli, N. C., Montardo, J. P., Prietto, P. D. M., and Pasa, G. S. (2002). "Engineering behavior of a sand reinforced with plastic waste." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 128(6), 462-472.

12. Consoli, N. C., Foppa, D. Festugato, L. and Heineck, K. S. (2007). "Key parameters for strength control of artificially cemented soils." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 133 (2), 197-205.

13. Consoli, N.C., Fondeca, A.V. Cruz, R.C. and Heineck, K.S. (2009). "Fundamental parameters for the stiffness and strength control of artificially cemented sand." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 135 (9), pp.1347-1353.

14. Das, S. K. and Yudhbir. (2005). "Geotechnical characterization of some Indian fly ashes." *Journal of Materials in Civil Engineering*, ASCE, 17(5), 544-553.

15. Electric Power Research Institute (1987). "Classification of fly ash for use in cement concrete." CS-5116, Project 2422-10. Orlando, Florida.

16. Gidigasu, M. D. (1974). "Degree of weathering in the identification of laterite material for engineering purpose – a review." *Engineering Geology*, 8(3), 213-266.

17. Gidigasu, M. D. and Dogbey, J. L. K. (1980). "The importance of strength criterion in selecting some residual gravels for pavement construction." *Proceedings of Seventh Regional Conference on Soil Mechanics and Foundation Engineering*, Accra, Ghana, 300-317.

18. IS: 2720-Part 7 (1992). "Method of test for soils: Determination of water content-dry density relation using light compaction." Bureau of Indian Standards, New Delhi.

19. IS: 2720-Part 13 (1992). "Method of test for soils: Direct shear test." Bureau of Indian Standards, New Delhi.

20. IS: 2720-Part 12 (1992). "Method of test for soils: Determination of shear strength parameters of soils from consolidated undrained test with the measurement of pore water pressure." Bureau of Indian Standards, New Delhi.

21. IS: 2720-Part 16 (1992). "Method of test for soils: Laboratory determination of

TH-1088_AKALITA

CBR." Bureau of Indian Standards, New Delhi.

22. IS: 2720-Part 10 (1995). "Method of test for soils: Determination of unconfined compressive strength." Bureau of Indian Standards, New Delhi.

23. Joshi, R.C. and Marsh, B.K. (1987). "Some physical, chemical and mineralogical properties of some Canadian fly ashes." *Proc. Materials Research Society*, Vol. 86, MRS. Pittsburgh, USA.

24. Joshi, R.C. and Lohtia, R.P.(1997). "Fly ash in concrete: production, properties, and uses." *Advances in Concrete*, Gordon and Breach Science Publishers, Ottawa, Canada, Vol. 2.

25. Kaniraj, S. R. and Havanagi, V. G. (1999a). "Geotechnical characteristics of fly ash-soil mixtures." *Geotechnical Engineering Journal*, 30(2)129-146.

26. Kaniraj, S. R. and Havanagi, V. G. (1999b). "Compressive strength of cement stabilized fly ash-soil mixtures." *Cement and Concrete Research*, 29, 673-677.

27. Kaniraj, S. R. and Havanagi, V. G. (2001). "Behavior of cement–stabilized fiber reinforced fly ash-soil mixtures." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 127(7), 574-584.

28. Kaniraj, S. R. and Gayathri, V. (2003). "Factors influencing the strength of cement fly ash base courses." *Journal of Transportation Engineering*, ASCE, 129(5), 538-548.

29. Kaniraj, S. R. and Gayathri, V. (2004). "Permeability and consolidation characteristics of compacted fly ash." *Journal of Energy Engineering*, ASCE, 130(1), 18-43.

30. Katti, R. K., Kulkerni, K. R. Chandrasekharan, V. S., Venkatachalam, G., and Dewaikar, D. M. (1975). "Regional deposits in India–state of art report." *Proceeding of Fifth Asian Regional Conference on Soil Mechanics and Foundation Engineering*, Bangalore, pp. 17.

31. Kim, B. Prezzi. M. and Salgado, R. (2005). "Geotechnical properties of fly and bottom ash mixtures for use in highway embankment." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 131(7), 914-924.

32 Kolias, S., Kasselouri-Rigopoulou, V. and Karahalios, A. (2005). "Stabilisation of

TH-1088_AKALITA

clayey soils with high calcium fly ash and cement." *Cement & Concrete Composition*, 27, 301-313.

33. Krishnan, M. S. (1982). "Geology of India and Burma." 6th Edition, CBS Publishers and Distributors, India, 517-525.

34. Lav, A. H., Lav, M. A. and Goktepe, A. B. (2006). "Analysis and design of a stabilized fly ash as pavement base material." *Fuel*, 85, 2359-2370.

35. Leelavathamma, B. and Pandian, N. S. (2005). "Effect of class C fly ash on the California Bearing Ratio behavior of soil-fly ash mixes and layered system." *Journal of Testing and Evaluation*, ASTM, 33 (2), 1-6.

36. Loughman, F. C. (1969). "Chemical weathering of the silicate minerals." *American Elsevior*, New York.

37. Leonards, G. A., F. and Bailey, B. M. (1982). "Pulverized coal ash as structural fill." *Journal of the Geotechnical Engineering Division*, ASCE, 108(GT4), 517-531.

38. Lo, S. R. and Wardani, S. P. R. (2002). "Strength and dilatancy of a silt stabilized by a cement and fly ash mixture." *Can. Geotech. Journal*, 39: 77-89.

39. Pandian, N. S., Rajasekhar, C., and Sridharan, A. (1998). "Studies on the specific gravity of some Indian fly ashes." *Journal of Testing and Evaluation*, ASTM, 26(3), 177 – 186.

40. Pandian, N. S., Sridharan, A., and Chittibabu, G. (1999). "Effect of curing on the strength behaviour of fly ashes." *Proceedings of Indian Geotechnical Conference*, Calcutta, 235–238.

41. Pandian, N. S. Krishna, K. C. and Sridharan, A. (2001). "California Bearing Ratio behavior of soil/fly ash mixtures." *Journal of Testing and Evaluation*, ASTM, 29 (2), 220-226.

42. Pandian, N. S. and Krishna, K. C. (2002). "California Bearing Ratio of cement-stabilized fly ash soil-mixes." *Journals of Testing and Evaluation*, ASTM, 36 (6), 1-5.

43. Pandian, N. S. and Krishna, K. C. (2003). "The pozzolanic effect of fly ash on the California Bearing Ratio behavior of black cotton soil." *Journal of Testing and*

TH-1088_AKALITA

Evaluation, ASTM, 31 (6), 1-7.

44. Pandian, N. S. (2004). "Fly ash characterization with reference to geotechnical applications." *J. Indian Inst. Sci*, 84, 189–216.

45. Prashanth, J. P., Sivapullaiah, P. V. and Sridharan, A. (1999). "Compaction behaviour of fly ash." *Ground improvement*, 3, 77-88.

46. Schnaid, F., Prietto, P. D.M. and Consoli, N. C. (2001). "Characterization of cemented sand in triaxial compression." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 127(10), 857-868.

47. Sezer, A., Inan, G., Yilmaz, H. R. and Ramyar, K. (2006). "Utilization of a very high lime fly ash for improvement of Izmir clay." *Building and Environment*, 41,150-155.

48. Sharma, R. C. and Krishnamoorthy, S. (1992). "Instrumental techniques to classify fly ashes in terms of their reactivity." *Cemento* 3,149-156.

49. Shihata, S. A. and Baghdadi, Z. A. (2001). "Simplified method to assess freeze-thaw durability of soil cement," *Journal of Materials in Civil Engineering, ASCE,* 13 (4), 243-247.

50. Simmons, J. V., and Blight, G. E. (1997). "Compaction." in G. E. Blight (Ed.). *Mechanics of Residual Soils*. Balkema, Rotterdam, The Netherlands, 57-63.

51. Sridharan, A., Rao, S. M., and Ramnath, K. P. and Nagaraj, H. B. (1991). "The role of iron hydroxide in the engineering behaviour of tropical soils." *Proc., 10th Reg. Conf. for Africa on Soil Mechanics and Fdn. Engg.*, Maseru, 1, 7-14.

52. Sridharan, A., Pandian, N. S. and Srinivasa Rao, P. (1998). "Shear strength characteristics of some indian fly ashes." *Ground Improvement*, Institution of Civil Engineers, Thomas Telford, London, No. 2, pp. 141-146.

53. Sridharan, A., Pandian, N. S., and Srinivas, S. (2001). "Compaction behaviour of indian coal ashes," *Ground Improvement*, Institution of Civil Engineers, Thomas Telford, London, No. 5, pp. 13-22.

54. Sridharan, A. (2002). "Characterisation of Indian coal ashes for geotechnical applications." *Proceedings of Indian Geotechnical Conference*, Allahabad, 2, 700-

TH-1088_AKALITA

712.

55. Stavridakis, E. I. (2006). “Stabilization of problematic soils using cement and lime.” *Expansive Soils – Recent Advances in Characterization and Treatment,* Taylor & Francis, London, 385-397.

56. Townsend, F.C. (1985). “Geotechnical characteristics of residual soils.” *Journal of Geotechnical Engineering Division,* ASCE, 111(1), 77-94.

57. Tremblay, H., Leroueil, S. and Locat, J. (2001). “Mechanical improvement and vertical yield stress prediction of clayey soils from eastern Canada treated with lime or cement.” *Can. Geotech. Journal*, 38, 567–579.

TH-1088_AKALITA

LIST OF PUBLICATIONS

1. Kalita, A. and Singh, B. (2009). "Investigation on soils blended with fly ash and other additives." *Student Symposium on Research in Civil Engineering*, SSRCE09, Chennai, Paper No. GTE 34.
2. Kalita, A. and Singh, B. (2009). "Experimental study on strength behaviour of cement blended soil-fly ash mixtures." *Indian Geotechnical Conference*, IGC-2009, Guntur, 344-347.
3. Kalita, A. and Singh, B. (2010). "Strength characteristics of a local red soil blended with class F fly ash and cement." *Ground Improvement and Geosynthetics*, ASCE Geotechnical Special Publication No. 207, pp. 143-148.
4. Sumesh, M., Kalita, A. and Singh, B. (2010). "An experimental investigation on strength properties of fly ash blended soils treated with cement." *Journal of Environmental Research and Development*, 5(2), 322-329.

TH-1088_AKALITA