AN EXPERIMENTAL INVESTIGATION ON THE ENGINEERING BEHAVIOUR OF CHEMICALLY STABILISED EXPANSIVE SOILS

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by

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CERTIFICATE

This is to certify that the thesis entitled **"An experimental investigation on the engineering behaviour of chemically stabilised expansive soils"** being submitted by Mr**. E. Ramanjaneya Raju** for the award of the degree of **DOCTOR OF PHILOSOPHY** in the Department of Civil Engineering, National Institute Of Technology, Warangal is a record of bonafide research work carried out by him under my supervision and it has not been submitted elsewhere for award of any degree.

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DECLARATION

This is to certify that the work presented in the thesis entitled **"An experimental investigation on the engineering behaviour of chemically stabilised expansive soils"** is a bonafide work done by me under the supervision of **Prof. M. Heeralal** and **Prof. B. R. Phanikumar** was not submitted elsewhere for the award of any degree.

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ABSTRACT

Expansive soils are popularly known in India as black cotton soils. They have an inherent nature to experience changes in volume depending upon changes in water content. Hence, they heave or increase in their volume when they absorb water in rainy seasons and shrink or decrease in their volume when they lose water on evaporation in summer seasons. This propensity of expansive soils to respond to changes in water content makes them highly problematic. Owing to this alternate swelling and shrinkage, lightly loaded infrastructure like residential buildings, canal linings and pavements constructed upon them are damaged. Structural members such as flooring, walls and columns develop severe cracking. Diagonal tension cracks are observed above doors and windows and below window-sills in the case of lightly loaded buildings.

To control these volume expansions and volume reductions of these soils, innovative foundation techniques have been invented. The broad classification of these techniques is as follows: i) mechanical modification (Sand cushion and CNS layer), ii) physical modification, iii) chemical modification and iv) foundations counteracting tension.

In mechanical modification, the top few layers of the expansive clay layer are removed and replaced by layers of non-expansive material such as sand and gravel. Sand cushion is a classic example of mechanical modification. Cohesive non-swelling (CNS) layer technique is another example of mechanical modification. In physical modification, the top layers of the soil stratum are excavated, pulverised and mixed with non-expansive materials like sands and gravels and compacted back in layers. As the soil layers are replaced by non-swelling materials, swelling is reduced.

Foundations counteracting tension are special foundation techniques. The examples are under-reamed piles, large-bottomed piers or belled piers, straight-shafted piers or drilled piers, and granular pile-anchors (GPAs). The swelling pressure of the soil causes the uplift force on these foundations. And skin friction causes the force resisting this uplift. Granular pile-anchors (GPAs) are a recent innovation over conventional granular piles. In a GPA, an anchor is created for the foundation using a mild steel anchor rod and a mild steel anchor plate placed at the bottom of the granular pile. When the expansive clay bed swells by imbibing water, swelling pressure uplifts the foundation. But, the anchorage in the GPA develops resistance to

this uplift which is mobilised over the cylindrical GPA-clay interface. The resistance mobilised acts in the downward direction; and this resistance can be attributed to the weight of the GPA and the GPA-clay interface shear parameters, namely, c' and φ'.
In chemical modification, various chemicals and industrial byproducts such as lime, silica fume (SF), fly ash (*FA*), cement, rice husk ash (*RHA*), pond ash (*PA*), calcium chloride ($CaCl_2$) and *GGBS* are added to expansive clay for controlling their volume changes, reducing their plasticity and improving their engineering behaviour. Chemical stabilisation of expansive clays basically works through flocculation and cementation which are the two important reactions of which one is an immediate reaction and the other is a time-bound reaction. Flocculation reduces plasticity and swelling. And cementation causes cementitious products which harden the blends.

This thesis is a focused research on chemical stabilisation of a remoulded expansive clay. Lime, cement, fly ash and GGBS were used as the additives to the clay. Free swell index (*FSI*), liquid limit (*LL*), plastic limit (*PL*) and plasticity index (*PI*) were the index properties determined at varied amounts of the above additives. Compaction characteristics, hydraulic conductivity, unconfined compressive strength corresponding to varied curing periods and soaked *CBR* were the engineering properties studied. Further, to study the load-settlement characteristics, plate load tests were performed on clay beds stabilised with varied amounts of the above additives. Moreover, in another series of tests, the plain clay bed and the clay beds stabilised with 6% lime, 20% cement, 20% fly ash and 20% *GGBS* were subjected to five swell-shrink cycles. In each of the swell-shrink cycles, swelling was observed for 10 days and shrinkage was observed for 50 days. Thus, each clay bed was continuously monitored for 300 days for swelling and shrinkage.

Furthermore, shrinkage/desiccation cracks developed in the untreated and the treated clay specimens were quantified by image analysis to determine the crack area, the crack density factor and the crack intensity factor.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

SYMBOLS

A	=	Activity
B	=	Diameter of the footing plate
C	=	Clay percentage
C	=	Colloidal content
CH	=	Clay of high plasticity
CDF	=	Crack density factor
CIF	=	Crack intensity factor
CBR	=	California bearing ratio
C_c	=	Compression index
c_v	=	Coefficient of consolidation
c’	=	Effective cohesion of the pile-soil interface
D’	=	Total length of the pier
D_f	=	Diameter of the foundation
D_{gpa}	=	Diameter of GPA
d’	=	Depth of the zone of soil unaffected by wetting
E_s	=	Young’s modulus
E	=	Void ratio
e_0	=	Initial void ratio
e_f	=	Final void ratio
G	=	Specific gravity
H	=	Initial thickness
H_i	=	Initial thickness
H_r	=	Resultant thickness
H_{rn}	=	Resultant thickness for ‘n’ swell-shrink cycles
I_{sw}	=	Swelling index
k	=	Hydraulic conductivity
K_s	=	Lateral swell pressure coefficient
L_{gpa}	=	Length of the GPA
P_u	=	Uplift force

P_R	=	Resisting force
p_s	=	Swelling pressure
q_i	=	Initial surcharge pressure
q_0	=	Stress
S	=	Percentage swell
S%	=	Swell potential
S_r	=	Degree of saturation
S_e	=	Elastic settlement
V_w	=	Volume of soil in water
V_k	=	Volume of soil in kerosene
W	=	Water content
w_i	=	Initial water content
w_{sat}	=	Water content
Δe	=	Change in void ratio
ΔH	=	Heave
ΔH	=	Change in thickness
$+\Delta H$	=	Swelling
$-\Delta H$	=	Shrinkage
ΔH_{sw}	=	Thickness of the clay bed which undergoes swelling
ΔH_{sr}	=	Thickness of the clay bed which undergoes shrinkage
γ_d	=	Dry unit weight
γ_d	=	Unit weight of soil solids
γ_w	=	Unit weight of water
γ_{di}	=	Initial dry density
log p	=	Applied pressure
μ_s	=	Poisson’s ratio of the soil
φ'	=	Effective angle of internal friction of the pile-soil interface

CHAPTER - 1

INTRODUCTION

Expansive soils are popularly known in India as black cotton soils. They exist world-wide in various countries such as Canada, Australia, South Africa, Israel, India and the United States of America. They have an innate tendency to experience volumetric expansion or swelling when they absorb water, and volumetric reduction or shrinkage when they lose water on evaporation. Thus, in a field expansive soil layer, swelling occurs when water infiltrates into it in rainy seasons and shrinkage occurs when water evaporates from it in summers. Residential structures, canal linings and pavements, which are lightly loaded infrastructure, get damaged when they are founded in expansive soils, owing to this alternate swelling and shrinkage. Structural members such as flooring, walls and columns develop severe cracking. Unsightly diagonal tension cracks are observed above doors and windows and below window sills in lightly loaded buildings. As a result, a lot of money needs to be spent on repair of civil engineering infrastructure and also on innovative foundation techniques.

Expansive soils are identified in situ by visual inspection of dark grey or black colour and shrinkage cracks. Further, they are identified in the laboratory by mineralogical studies and determination of swell potential and swelling pressure. Mineral montmorillonite belonging to the smectite group is predominant in expansive clays. As the mineral montmorillonite has expanding lattice structure, expansive clays swell upon absorbing water.

For controlling the volume changes of expansive soils and for ameliorating the distress caused to civil engineering structures founded in them, certain efficacious foundation techniques have been invented. These innovative foundation practices can be broadly classified as i) mechanical modification (Sand cushion and CNS layer), ii) physical modification, iii) chemical modification and iv) foundations counteracting tension.

Mechanical modification suggests that the top portion of the expansive soil stratum is replaced by layers of non-expansive material such as sand and gravel. Sand cushion is a classic example of mechanical modification. Cohesive non-swelling (CNS) layer technique is also an example of mechanical modification. In physical modification, the upper portion of the stratum is excavated, pulverised and mixed with non-expansive materials like sands and gravels and are compacted back in layers.

Foundations counteracting tension are understood to be special foundations. The examples are under-reamed piles, large-bottomed piers or belled piers, straight-shafted piers or drilled piers, and granular pile-anchors (GPAs). In drilled piers, resistance to uplift is mobilised through skin friction. The uplift force is because of swelling pressure of the soil. Under-reamed piles and belled piers are deep foundations with enlarged bases. Resistance to uplift in such foundations is developed over an enlarged perimeter and reduces heave. Under-reamed pile foundation is an innovative technique developed by Central Building Research Institute (CBRI), Roorkee, India.

Granular pile-anchors (GPA) are a recently developed tension-resistant foundation technique. GPA is an innovation over stone columns or granular piles. The philosophy of granular pile-anchors (GPAs) proposes that an anchor comprising of a mild steel anchor rod and a mild steel anchor plate (which is placed at the bottom of the granular pile) be created in the granular pile so that the system can resist tension. Thus, foundations become anchored in the granular pile-anchor technique. When swelling pressure mobilised in the soil lifts the foundation, resistance to the upward force is developed over the cylindrical GPA-clay interface by virtue of the interface frictional parameters, c' and φ'.

In chemical modification, the nature of the clay mineral is altered or modified by adding chemical re-agents to the expansive clay. The various chemicals and industrial by-products used in chemical alteration are lime, cement, calcium chloride ($CaCl_2$), pond ash (*PA*), rice husk ash (*RHA*), fly ash (*FA*), GGBS and silica fume. These reagents reduce swelling and plasticity of clays and improve their properties such as compressibility behaviour, shear strength characteristics, compaction characteristics and load-settlement behaviour. Chemical stabilisation of expansive clays basically works through two reactions of which one is an immediate reaction and the other is a time-bound reaction. The immediate reaction is called flocculation which occurs within a short time of adding chemical reagents in the presence of water and which causes flocs to form in the clay-additive blend. The formation of flocs reduces plasticity and swelling. The time-bound reaction is called cementation in which cementitious products develop in the blends by virtue of which the blends harden.

This thesis is an experimental research on chemical stabilisation of a remoulded expansive clay. Lime, cement, fly ash and GGBS were used as the additives to the expansive clay. Free swell index (*FSI*), liquid limit (*LL*), plastic limit (*PL*) and plasticity index (*PI*) were the important index properties determined at varied amounts of the above additives. Compaction characteristics, unconfined compressive strength (corresponding to varied curing periods), hydraulic conductivity and soaked CBR of the expansive clay stabilised by varied amounts of the above additives were the engineering properties studied. Further, plate load tests were performed on the clay beds stabilised with varied amounts of the above additives for the determination of the load-settlement behaviour. Moreover, the effect of wetting-drying cycles on the volumetric changes, namely, swelling and shrinkage, of the plain expansive clay bed and the expansive clay bed stabilised with the above-mentioned additives. Every clay bed was subjected to five swell-shrink cycles, each of duration 60 days. In every cycle, swelling of the clay beds was observed for 10 days and shrinkage 50 days. Thus, every clay bed was continuously observed for 300 days. Linear or radial shrinkage cracks and polygonal shrinkage cracks were periodically measured. Moreover, quantification of desiccation cracks was also done on thin clay samples stabilised with the above additives using MATLAB.

Chapter-2 presents a detailed review of literature on foundation practices in expansive soils including physical and chemical modification, mechanical modification and foundations counteracting tension. Experimental procedures employed for the determination of various index (physical) and engineering properties of the untreated soil and the soil stabilised by different additives have been explained in detail in Chapter-3.

Chapter-4 presents a detailed discussion on the index (physical) properties and engineering properties of the chemically stabilised expansive clay. The influence of the additives and their amounts on *FSI*, plasticity, *OMC* and *MDD* and hydraulic conductivity of the expansive clay has been thoroughly analysed. Further, Chapter-4 discusses the influence of chemical stabilisation on stress-strain behaviour, unconfined compressive strength (*UCS*), the effect of curing period on *UCS* and soaked *CBR* in detail. Chapter-5 discusses the load-settlement behaviour. Load-settlement characteristics were studied in as-compacted condition of the clay beds and also in their saturated condition. For the load tests on clay beds in saturated condition, the clay beds were compacted and allowed to undergo heave through the process of continuous inundation or wetting. After saturation was indicated by profiles of heave or swelling, load tests were conducted.

Chapter-6 describes swell-shrink studies conducted on the clay beds. Swell-shrink studies were conducted on the plain expansive clay bed and the clay beds stabilised with 6% lime, 20% cement, 20% fly ash and 20% GGBS. These were made to undergo five swell-shrink cycles. In each cycle, swelling or heave was monitored for ten days and volume reduction or shrinkage for 50 days. Hence, swelling and shrinkage of the above clay beds were observed continuously for 300 days. Linear or radial shrinkage and polygonal shrinkage cracks were also monitored.

Chapter-7 presents quantification of the shrinkage/desiccation cracks in the untreated and the treated expansive clay specimens prepared at various water contents, namely, liquid limit, plastic limit, plasticity index and optimum moisture contents and subjected to image analysis technique through MATLAB.

Conclusions drawn from different aspects of the study are presented in Chapter-8.

CHAPTER - 2

REVIEW OF LITERATURE

2.1. Introduction:

Expansive soils, which are rich in mineral montmorillonite that has an expanding lattice structure, undergo swelling or volume expansion upon absorption of water and shrinkage or volume reduction upon evaporation of water. As a result, residential buildings and pavements are subjected to detrimental cracks (Chen, 1988; Rees and Thomas, 1993). Indian expansive soils are called black cotton soils which abound in most of the states in India.

According to Mohan (1977), the entire expansive soil stratum in field does not respond to moisture absorption and evaporation but only the top few layers. These layers are called active zone and the rest is inactive zone (Katti, 1978; Snethen, 1980). In dry summer seasons, polygonal shrinkage cracks develop in these soils. Depending upon the soil properties, these cracks can be quite wide and deep.

2.2. Identification of expansive soils:

Identification of these soils can be done by study of index properties, swelling characteristics and mineralogy. As mentioned above, mineral montmorillonite is abundant in expansive clays though kaolinite and illite may be present in small or negligible proportions. (Shreiner, 1987; Chen, 1988; Omari and Hamodi, 1991; Nelson and Miller, 1992). Different sophisticated techniques such as XRD are used in the study of mineralogy.

Free swell index (FSI), clay content, activity, liquid limit, plasticity index are used for characterising degree of expansion, swelling or volume expansion and shrinkage or volume reduction which are generally the indicators of severity of damage done to structures.

According to Skempton (1953) and later modification by Seed et al. (1962), activity can be expressed as

$$\text{Activity} = \frac{Plasticity\ index}{(\%\ clay\ finer\ than\ 2\mu - 5\%)} \tag{2.1}$$

A soil having higher activity (>1.25) is more expansive; and a soil having less activity (<0.75) is less expansive.

Based on shrinkage limit, the degree of expansion of a soil was characterised by Altmeyer (1955). Linear shrinkage, which is expressed as a percentage, is given by the nearest whole number. Table 2.1 shows the characterisation of degree of expansion (%) based on linear shrinkage (%).

Table 2.1 Characterisation of degree of expansion

Shrinkage limit (%)	Linear shrinkage (%)	Probable swell (%)	Degree of expansion (%)
<10	>8	>1.5	Critical
10-12	5-8	0.5-1.5	Marginal
>12	<5	<0.5	Non — critical

FSI is another important index property of expansive soils. Holtz and Gibbs (1956) and Mohan and Goel (1959) worked on developing a simple test for determining FSI, which is expressed as

$$FSI = (V_w - V_k)/V_k \times 100 \quad (2.2)$$

A soil having a high degree of expansion must have an FSI more than 100%.

A relationship between plasticity index (PI) and swell potential (S.P) was proposed by Seed et al. (1962) as,

$$S.P = 2.16 \times 10^{-3} (PI)^{2.44} \quad (2.3)$$

Further, Seed et al.(1962) was also developed the following expression by considering clay percentage (C) and activity (A).

$$S.P = 3.6 \times 10^{-5} (A^{2.44}) (C^{3.44}) \quad (2.4)$$

While Ranganatham and Satyanarayana (1965) considered shrinkage index (*S.I*), Chen (1988) used plasticity index (*PI*) in their correlations for swell potential. Moreover, Murthy and Raman (1977) used a parameter, "swelling index" (I_{sw}) in their correlation. And Bandyopadhyay (1981) made use of colloid content (*C*) in his correlation for swell potential (*S*).

2.2.1 Swelling characteristics:

The two important swelling characteristics of expansive soils are swell potential (S%) and swelling pressure (p_s). They are dependent on three important placement conditions, namely, initial water content (w_i), initial dry density (γ_{di}) and initial surcharge pressure (q_i). Hence, correlations were developed for S% and p_s in terms of these placement conditions (Vijayvergiya and Ghazzaly, 1973). The best way to identify an expansive soil is to determine swell potential (S%) swelling pressure (p_s) directly in the laboratory by performing a one-dimensional swell-consolidation test.

2.2.2. Soil suction:

Another important property in the case of expansive soils is suction, which is measured in both lab and the field (Snethen, 1979). In the case of unsaturated expansive soils, it is negative pore pressure to which swelling responds sensitively (Rees and Thomas, 1993; Kenneth et al. 1993). While Weisberg et al. (1990) observed that rate of wetting could be governed by suction, Dhowian (1992) said that suction could be governed by depth, time and swelling. McKeen (1988) suggested measurement of suction by filter paper method. And many other researchers tried to determine swelling in terms of water content.

2.3. Remedial measures:

The detrimental cracking developed in the structures because of the alternate swelling and shrinkage causes heavy financial loss (Gourley et al. 1993). To counteract this, remedial measures in the form of innovative foundations need to be devised. The various remedial measures undertaken in the case of expansive soils can be grouped as mechanical modification, physical modification, chemical modification and foundations counteracting tension.

Sand cushion method (Satynarayana, 1966) and cohesive non- swelling (CNS) layer method (Katti, 1978) are illustrations for mechanical modification. Physical modification (Satynarayana, 1966; Phanikumar et al. 2012) advocates mixing gravels and sands with the expansive soil in the top layers for reducing swelling. Tensile forces develop in the active zone when water is absorbed and swell is triggered. To counteract these tensile forces some innovative foundation techniques have been devised, which are drilled piers or straight-shafted piers, belled piers or large-bottomed piers, granular-pile anchors or GPAs and under-reamed piles (Chen, 1988; Phanikumar, 1997; Phanikumar et al. 2004).

The mechanisms, the working philosophies, the uplifting and the resisting forces generated for the equilibrium of these foundations are shown in Figures 2.1 to 2.5. The in-situ behaviour of GPAs (Rao et al. 2007; Rao et al. 2008; Phanikumar et al. 2008) and the group action of GPAs (Muthukumar and Phanikumar, 2014; Phanikumar and Muthukumar, 2015; Phanikumar and Muthukumar, 2019) were also studied.

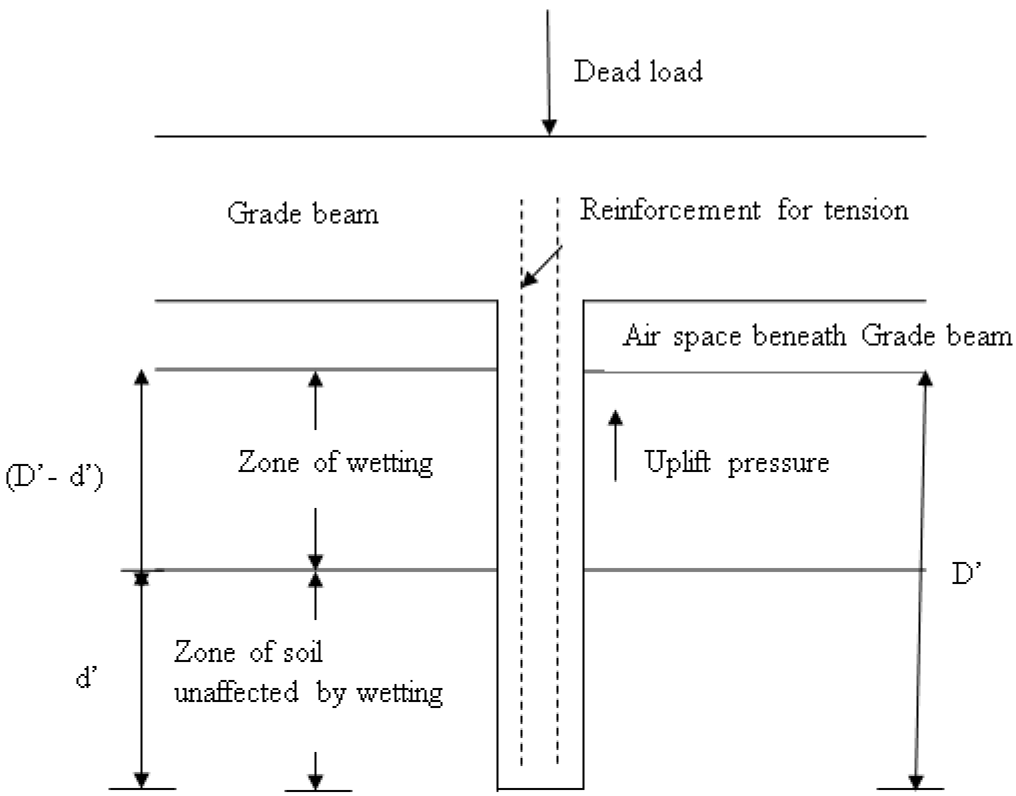


Figure 2.1 Straight-shafted piers or drilled Piers

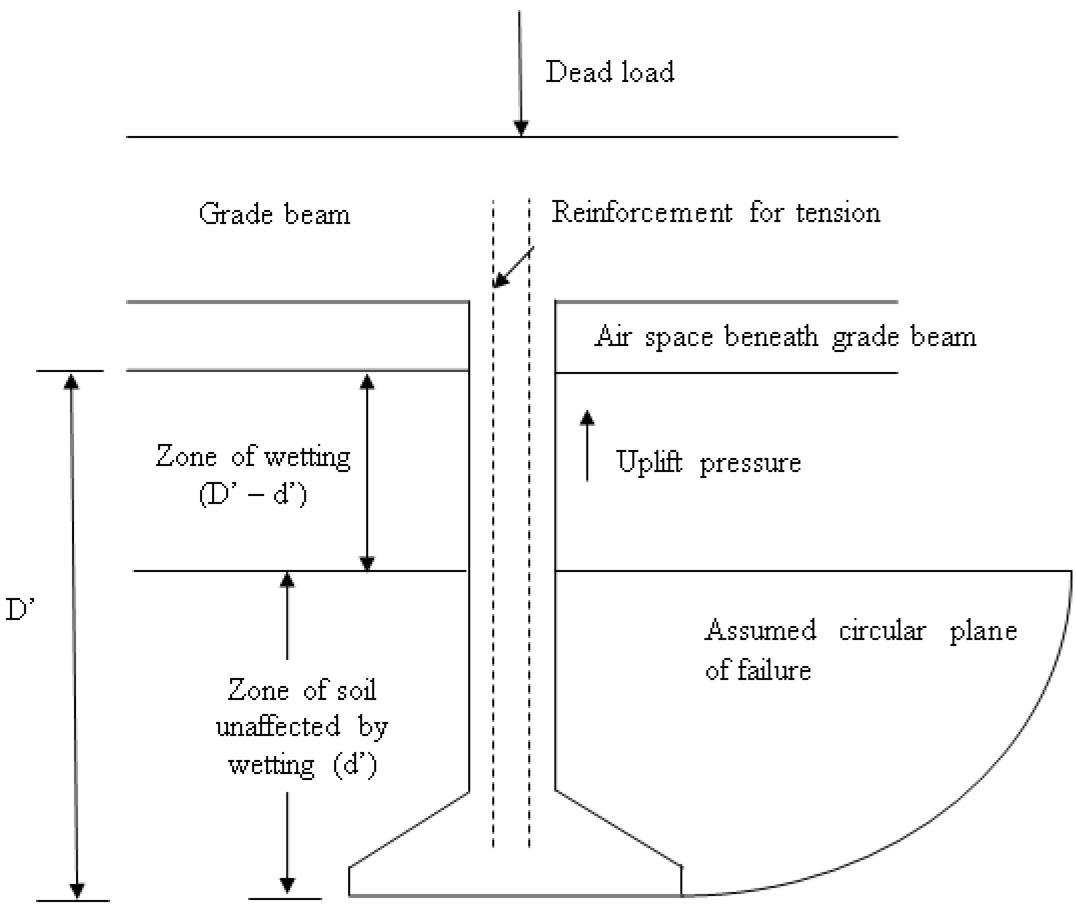


Figure 2.2 Large-bottomed piers or belled Piers

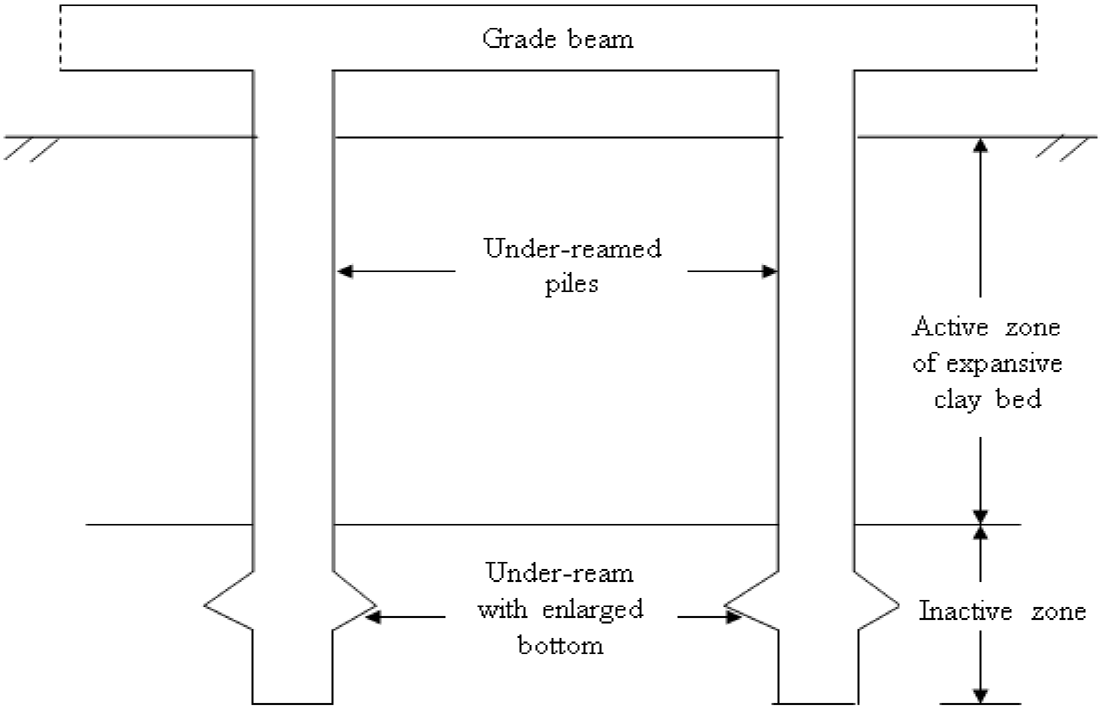


Figure 2.3 Under-reamed pile

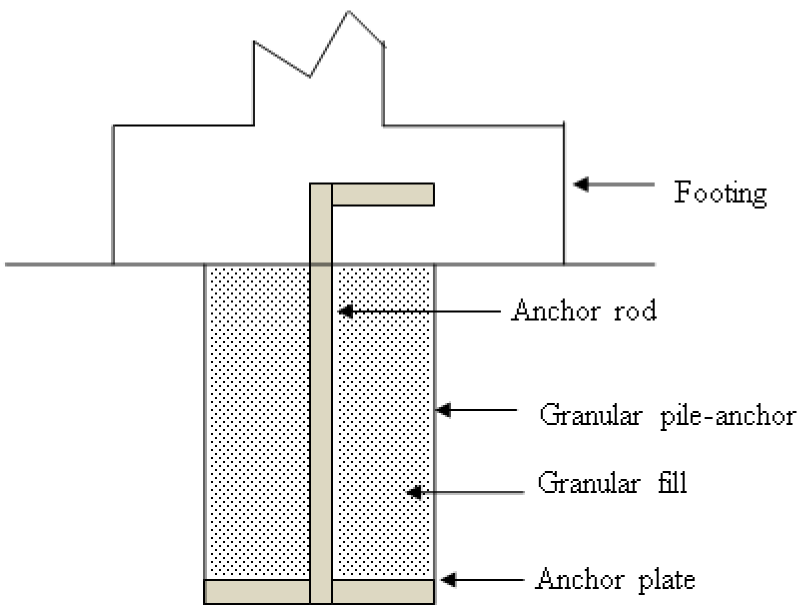


Figure 2.4 Concept of a GPA

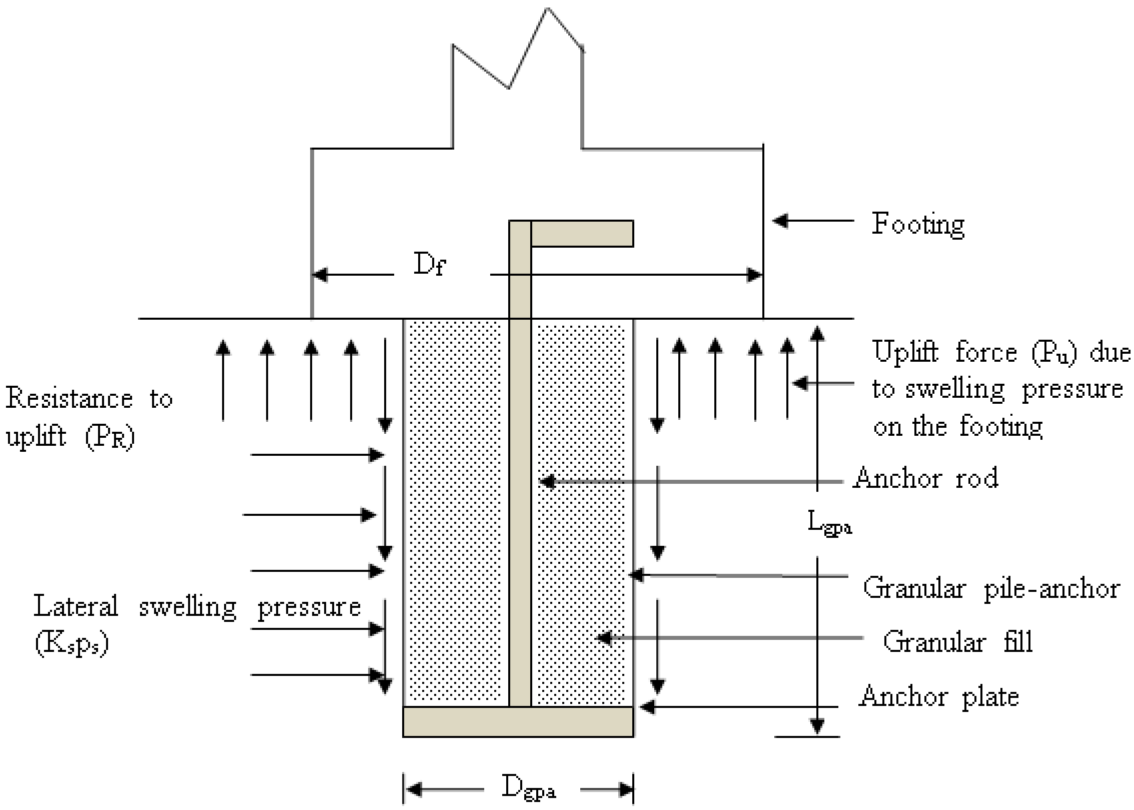


Figure 2.5 Forces acting on a GPA

2.3.1. Chemical alteration:

This involves addition of chemicals and industrial by-products to expansive clays. This is done basically to reduce swelling by changing the propensity of the predominant minerals in the clay mass. Modification of expansive clays using lime, cement, fly ash, GGBS, rice husk ash (RHA) and calcium chloride is efficacious in reducing swelling and improving strength (Shanker and Maruthi, 1989; Sridharan et al. 1997; Phanikumar et al. 2001; Cokca, 2001, Rao et al. 2008, Phanikumar and Sastry, 2001; Puppala, 2001; Sivapullaiah et al. 2004; Ramanamurthy and Harikrishna, 2006, Sharma et al. 2008.).

2.3.1.1. Lime:

Many researchers have investigated the impact of lime on the improvement of properties of soil. They have mentioned that lime stabilisation proceeds through a combination of four mechanisms, namely, cation exchange, flocculation, carbonation and pozzolanic reactions. The first two mechanisms increase soil workability resulting from the changes in the electrical charges of clay minerals (Grim,1962). The second two mechanisms ensure cementitious reactions that produce the increase in the bearing strength.

Bell (1988) stated that the quantity of lime required to modify a clay varies from 1 to 3%. But the amount of lime required to develop cementation varies from 2 to 8%. Expansive clays respond more quickly to lime treatment than kaolinitic clays. Upon addition of lime to clays, calcium ions are adsorbed by clay minerals. This reduces liquid limit and improves workability. Bell (1996) mentioned that addition of lime improves engineering properties of clays. However, the properties of lime-clay blends depend upon the nature of the clay, curing period and construction method.

The effect of lime and fly ash on *FSI*, S%, p_s, compressibility properties such as C_c, c_v, secondary consolidation and strength was compared in an experimental investigation by Phanikumar (2009). The lime content was considered as 2, 4 and 6% and the amount of fly ash was considered as 0, 10 and 20%. There was notable reduction in FSI, S%, p_s, C_c, c_v, secondary consolidation and strength.

Dash and Hussain (2015) investigated shrinkage behaviour of lime-clay blends. Lime increases the shrinkage limit of soils irrespective of their plasticity characteristics; however, it is more in case of high plastic soils than the low plastic ones. Shrinkage decreased up to 5% of lime only. Therefore, in this study, 5% lime was recommended as optimum lime content.

Phanikumar et al. (2015) investigated FSI, swelling and swelling pressure of lime-clay blends. FSI and swelling decreased with increasing lime content. However, upon 3-day inundation, cementitious products developed resulting in high values of p_s. Subhradeep and Hussain (2018) stated that lime causes a strong basic environment in pore fluid. Silica (SiO_2) and Alumina (Al_2O_3) present in the soil are dissolved, which form a cementitious gel (like CSH and CAH), causing a strong bond between them, and increasing the strength.

2.3.1.2. Cement:

The addition of cement to a material, in the presence of moisture, produces hydrated calcium aluminate and silicate gels (C-A-H and C-S-H), which crystallise and bond the material particles together. Addition of cement to the fine-grained soils results in a combination of both mechanical and chemical bonding which involves in a reaction between the cement and the surfaces of soil particles. When clays are treated with cement, the swelling behaviour, plasticity decreases and strength increases (Sherwood, 1995; Croft, 1967).

Sariosseiri and Muhunthan (2008) used cement for stabilising some Washington state soils. The amount of cement was varied as 2.5, 5, 7.5 and 10%. Tests were performed to study *LL*, *PL* and *PI*, compaction behaviour, *UCS* and consolidated-undrained triaxial behaviour. *UCS* and workability improved. Cement treatment leads to significant increase in unconfined compressive strength and modulus of elasticity of the soils. Cement treated soils exhibited much more brittle behaviour compared with non-treated soils. Mohammad and Rasool (2012) stated that UCS, stiffness and C_c increases with increasing the amount of cement.

A research study was initiated by Sireesh et al. (2012) to investigate mechanisms of lime-clay and cement-clay blends. Physical and engineering properties were studied. *UCS* increased up to a curing period of 28 days. However, there was not notable improvement in *UCS* after 28 days. Phanikumar et al. (2015) mentioned that chemical modification of expansive clays could be effective in reducing S% and p_s. Upon addition of lime and cement to an expansive clay, flocculation and cementation occur. Of these, flocculation is an immediate reaction, which causes reduction in plasticity and swell potential. Phanikumar et al. (2015) conducted experiments on lime-clay blends and cement-clay blends. *FSI*, *S%* and p_s were investigated. From the experimental results it was observed that *FSI* and swelling decreased with lime and cement.

Phanikumar and Raju (2020) investigated compaction and strength of an expansive clay blended with lime sludge and cement. *FSI*, *LL*, *PL* and *PI*, compaction behaviour, *UCS* and *CBR* were investigated. As densities did not decrease much upon addition of lime sludge, 10% cement was added to these blends as a secondary additive. Upon addition of 10% cement, *LL*, *PI* and *FSI* further decreased and densities, strengths and CBRs increased.

2.3.1.3. Fly ash:

According to Cokca (2001) and Phanikumar and Sharma (2007), physico-chemical interactions and mechanical changes are two important mechanisms which would occur upon addition of fly ash to an expansive clay. Fly ash particles replace clay particles and cause flocculation by cation exchange. Because of flocculation, *FSI*, S% and p_s decrease. Sharma (1998) observed that reduction in suction is the cause of reduction in swelling pressure. Fly ash increases strength of clays too.

Fly ash alters physical properties of expansive clays according to Sivapullaiah et al. (1996). The amount of variation, however, depends on particle size distribution and pozzolanic reactivity. Nalbantoglu (2004) conducted a series of laboratory experiments to study the effectiveness of Class C fly ash as a clay stabiliser. The test results indicated that fly ash was efficacious in improving the texture and in reducing PI and S%.

Phanikumar and Sharma (2004) found out that S% and p_s decreased nearly by 50% at 20% fly ash. Cokca (1999) stated that p_s decreased with curing period. Further, according to Phanikumar and Sharma (2004) and Phanikumar and Sharma (2007), when fly ash is added to an expansive clay, its FSI, S% and p_s decrease and its strength properties improved.

Kate (2005) studied the strength and volume change behaviour of an expansive clay treated with fly ash. FSI, S% and p_s and UCS were investigated at varied fly ash contents. The clay was also treated with fly ash-lime admixtures. FSI, S% and p_s decreased with increasing fly ash content. They further decreased with the addition of small percentages of lime to fly ash. Zha et al (2008) investigated the efficacy of fly ash and lime-fly ash admixtures in stabilizing an expansive clay. PI, FSI, S%, p_s and activity decreased with fly ash content and lime-fly ash content.

Phanikumar et al. (2009) suggested fly ash-columns (*FACs*) to reduce swelling and improve bearing capacity. Heave tests and load tests were performed on clay beds reinforced by *FACs*. Heave decreased and load response improved. Mir and Sridharan (2019) mentioned that stabilisation of problematic soils such as black cotton soils with fly ash could be effective in improving both the compressibility characteristics and strength parameters. They observed that 20% of class C fly ash could be recommended as the optimum amount for improving the clay properties in preference to 60% of class F fly ash.

2.3.1.4. Ground Granulated Blast Furnace Slag (GGBS):

GGBS, an industrial by-product, which is a product of fusion of limestone flux with coal ash, basically consists of silicates and alumino-silicates of lime (Yadu and Tripathi, 2013; Higgins, 2005). Flocculation is promoted by cation exchange causing flocculated fabric (Cokca, 2001) and reduced swelling (Sharma and Sivapulliah, 2016). GGBS could cause reduction in plasticity and increase in strength (Gupta and Seehra, 1989; Akinmusuru, 1991; Yadu and Tripathi, 2013; Higgins, 2005).

The use of GGBS has a lot of advantages than the regular stabilisation techniques. The plasticity nature, strength and compressibility behaviour of clayey soil were effectively modified when GGBS blended with little amounts of activators (Nidzam and Kinuthia, 2009). When fly ash and GGBS are mixed for a binder, swelling is further reduced and there can be significant improvement in soil properties (Sharma and Sivapulliah 2015; Sharma and Sivapulliah 2016).

Mujtaba et al. (2018) collected two expansive clay samples to study and investigate the effect of GGBS on the properties. In various percentages GGBS was added between 0% and 55% to the clay specimens, and different properties were studied. Mixing GGBS with expansive clay resulted in increase in the engineering properties.

This thesis is an experimental investigation on a highly swelling remoulded expansive clay stabilised with varied quantities of lime, cement, fly ash and ground granulated blast furnace slag (GGBS). Significant index properties of the expansive clay such as Atterberg limits and free swell index (*FSI*), compaction characteristics, stress-strain behaviour, load-settlement behaviour and hydraulic conductivity of the expansive clay were investigated in detail at varied quantities of the above additives. Further, swell-shrink behaviour of the plain expansive clay beds and the clay bed stabilised with the above additives was studied by subjecting the clay beds to five wetting-drying cycles or swell-shrink cycles, each cycle spanning over a period of 60 days. Quantification of desiccation cracks was also performed through image analysis using MATLAB.

Chapter 3 presents the experimental investigations carried out and the analysis of test results in the subsequent chapters.

CHAPTER - 3

EXPERIMENTAL INVESTIGATION

3.1. Introduction:

This chapter describes in detail the objectives of this experimental research and the different experimental procedures that were carried out in order to realise those objectives.

3.2. Objectives of the experimental research:

The chief objectives of this study on chemical stabilisation of a remoulded expansive clay are:

(i) To investigate the influence of lime ($Ca(OH)_2$), cement, fly ash and ground granulated blast furnace slag (GGBS) on important index properties;

(ii) To study the influence of the above chemicals on the compaction characteristics, namely, optimum moisture content (*OMC*) and maximum dry density (*MDD*) of the clay;

(iii) To study the effect of the above chemicals on the stress-strain behaviour and the unconfined compressive strength of the clay;

(iv) To explore the effect of these chemicals on the hydraulic conductivity of the clay;

(v) To investigate their effect on California bearing ratio (*CBR*) of the clay;

(vi) To study their influence on the stress-settlement behaviour of the clay in as-compacted and saturated conditions;

(vii) To study their influence on the rate and amount of heave at *OMC* and *MDD*;

(viii) To investigate their influence on the swell-shrink behaviour at *OMC* and *MDD*, the linear shrinkage and the polygonal shrinkage crack propagation observed in the clay beds and

(ix) To quantify the desiccation cracks by image analysis using MATLAB.

3.3. Test materials:

A highly swelling clay, obtained from Bhimavaram, A. P., India, having a *FSI* of 134% was used in this research. Its liquid limit and plasticity index suggested that it could be categorised as *CH*. Table 3.1 depicts the properties of the clay along with the *ASTM* codes of practice according to which the index properties were determined. Table 3.2 depicts the chemical composition of the expansive clay.

Table 3.1. Physical (Index) properties

	Result	ASTM code
Specific gravity, *G*	2.69	D 854-14
Gravel (%)	0	D 6913M-17
Sand (%)	2.50	D 6913M-17
Silt (%)	34.00	D 6913M-17
Clay (%)	63.50	D 6913M-17
Liquid limit, *LL* (%)	87.00	D 4318-00
Plastic limit, *PL* (%)	28.00	D 4318-00
Plasticity index, *PI* (%)	59.00	D 4318-00
Free swell index, *FSI* (%)	134.00	D 5890-02
Classification by USCS	*CH*	-

Table 3.2. Chemical composition of the clay

Component	clay
SiO_2	63.17
Al_2O_3	19.36
Fe_2O_3	4.32
CaO	0.67
MgO	1.79
Na_2O	8.73
K_2O	1.73
SO_3	-
$Ca(OH)_2)$	-
$CaCO_3$	-
$CaSO_4$	-
H_2O	-
Loss on ignition	0.23

The chemicals used for modifying the expansive clay were lime ($Ca(OH)_2$), cement, fly ash and *GGBS*. Hydrated lime which was in powder form was used in the study and it was obtained from Venspra Labs Limited, Vijayawada, A. P., India. The cement used was ordinary Portland cement. Class-C fly ash, which was also in powder form, was obtained from Vijayawada Thermal Power Station (VTPS), Vijayawada, A. P., India. GGBS, also in powder form, was obtained from Venspra Labs Limited, Vijayawada, A. P., India. Table 3.3 depicts the chemical composition of the above additives.

The choice of these additives was made considering the difference in their nature and the way they react with the expansive clays and bring about chemical reactions like cation exchange and cementation. Of the above additives, lime is the quickest pozzolanic material to cause quick flocculation in the blends while cement is the strongest cementitious material. Fly ash and GGBS are both moderate in these two reactions.

Table 3.3 Chemical composition of the additives

Ingradient/ Parameter	Lime	Cement	Fly ash	GGBS
SiO_2	1.34	20.10	33.80	40.00
Al_2O_3	0.14	4.50	22.90	14.50
Fe_2O_3	0.19	2.00	6.10	1.80
CaO	-	65.10	28.60	39.90
MgO	0.73	4.30	4.60	3.60
Na_2O	-	-	-	-
K_2O	-	-	-	-
SO_3	-	2.70	2.80	0.20
$Ca(OH)_2$	95.42	-	-	-
$CaCO_3$	1.84	-	-	-
$CaSO_4$	0.09	-	-	-
H_2O	0.25	-	-	-
Loss on ignition	-	1.30	1.20	0.0

3.4. Test variables:

In all the tests performed in this research, the amount of lime was varied as 0, 1, 2, 4 and 6% by dry weight of the expansive clay. Similarly, the amounts of cement, fly ash and GGBS were varied as 0, 5, 10, 15 and 20% by dry weight of the clay. The influence of these additives on *FSI*, *LL*, *PL* and *OMC* and *MDD* was studied at the above dosages.

The hydraulic conductivity tests, the unconfined compressive strength tests, the CBR tests, the plate load tests (for studying the load-settlement characteristics) and the swell-shrink tests were conducted on clay beds prepared at the respective *OMC* and *MDD* of the additive-clay blends which varied depending on the dosages of the additives.

3.5. Tests conducted:

As already mentioned, the tests listed below were conducted on additive-clay blends.

i) *LL* and *PL* tests;
ii) *FSI* tests;
iii) Proctor compaction tests;
iv) Unconfined compressive strength tests;
v) California bearing ratio (*CBR*) tests;
vi) Hydraulic conductivity tests;
vii) Plate load tests in as-compacted and saturated conditions;
viii) Heave tests;
ix) Swell-shrink tests over 5 wetting-drying cycles and
x) Image analysis

3.6. Sample preparation and test procedure:

3.6.1. LL and PL tests:

LL and *PL* tests were performed as per ASTM D 4318-00 on clay passing 425μm sieve. In tests on the expansive clay modified by the above additives, the clay was replaced by the respective additive by its dry weight based on the additive content. The additive-clay mixes were thoroughly mixed and the tests performed.

3.6.2. Free swell index (FSI) tests:

FSI tests were carried out in accordance with the ASTM D 5890-02 on oven-dried clay passing 425μm sieve. 10gms of this powder were poured into a jar with 100ml capacity containing de-ionised water and a jar with 100ml capacity containing kerosene. The jars were kept to stand for one day (24 hours). After 24 hours, the final volumes of the soil in the two jars were noted. *FSI* is expressed as

$$FSI = (V_w - V_k)/V_k \times 100 \qquad (3.1)$$

In the *FSI* tests on the clay-additives blends, the clay was replaced by the respective additive by its dry weight based on the additive content.

3.6.3. Proctor compaction tests:

These tests were conducted as per ASTM 2000, D698a on the plain clay and the clay blended with varied dosages of the above additives. For these tests, the clay passing 4.75mm sieve was used. In the case of the compaction tests on additive-clay blends, the expansive clay was replaced by the respective additive by its dry weight based on the additive content. The clay-additive mixes were fully mixed and the tests performed.

3.6.4. Unconfined compressive strength (UCS) tests:

UCS tests were conducted on the plain expansive clay and the clay blended with different dosages of the above additives. Air-dried soil passing 4.75mm sieve was used in the unconfined compressive strength tests. The tests were carried out in accordance with the ASTM 2000 D2166. In the tests on blend samples, the clay was replaced by the respective additive by its dry weight based on the additive content. The influence of curing period was also studied at 0, 7, 14 and 28 days. Total samples were prepared at the respective *OMC* and *MDD* of the additive-clay blends. *OMC* and *MDD* of the additive-clay blends are presented in Table 4.2 and discussed in Section 4.3.4 of Chapter 4.

3.6.5. California bearing ratio (CBR) tests:

These tests were performed in soaked condition on the unblended expansive clay and the clay blended with different dosages of the above additives according to ASTM D 1833-00. For the

CBR tests also, air-dried soil passing 4.75mm sieve was used. These samples were also compacted at the respective *OMC* and *MDD* of the additive-clay blends.

3.6.6. Hydraulic conductivity (k) tests:

These tests were performed on the unblended clay and on the clay-additive blends at *OMC* and *MDD*. The hydraulic conductivity (*k*, cm/sec) was determined by variable head permeameter tests which were performed as per ASTM D2434. De-ionised water (DIW) was used as the permeating fluid in the tests. The clay-additive blends for these tests were prepared varying lime content as 1, 2, 4 and 6%, cement content as 5, 10, 15 and 20%, fly ash content as 5, 10, 15 and 20% and GGBS content as 5, 10, 15 and 20% by dry weight of the expansive clay. The blends were prepared by mixing their materials the weights of which depended on their *OMC*s and *MDD*s. They were mixed and compacted, ensuring uniform density. The samples were soaked for three days (72 hours) after the compaction, and then the tests were conducted on them with a 10mm standpipe. 3 trials were performed for all the specimens and the average of the 3 values was considered as the hydraulic conductivity (*k*, cm/sec) of the specimens. The same procedure was adopted for all the above-mentioned dosages of all the additives.

3.6.7. Plate load tests:

To determine the load response, laboratory scale plate load tests were performed on the unblended clay bed and the clay bed stabilised with the above-mentioned dosages of additives in two series: (a) in as-compacted state and (b) in saturated state. In both the series, the plate load tests were conducted using reaction type of loading with a loading jack. The test data were monitored using a proving ring and a dial gauge. Figure 3.1 depicts the arrangement for conducting the plate load tests. A footing plate of 60mm diameter and 10mm thickness was used for loading the clay beds. The weight of the footing plate was 300g. The plain clay and the additive-clay blends were all compacted at their *OMC* and *MDD* in moulds of dia 300mm and thickness 400mm. The clay bed thickness was constant at 200mm. The clay beds were compacted in 4 layers the thickness of which was monitored to ensure the required density.

3.6.7.1. Plate load tests in as-compacted condition:

In this series, the plate load tests on the plain clay bed and that stabilised chemically were performed in the as-compacted state according to ASTM D 1196-93 (ASTM, 1997). In all the tests, the individual stress increment was fixed as 20kPa based on the undrained strength of the material. The settlement of the clay beds under loads was monitored with a dial gauge at 1, 2.25, 4, 6.25, 9, 16 and 25 minutes and later at 1 hour intervals. The rate of settlement was observed under each stress increment, and when the rate of settlement was less than 0.02mm/minute, the next stress increment was applied. The tests were conducted up to a stress of 120kPa.

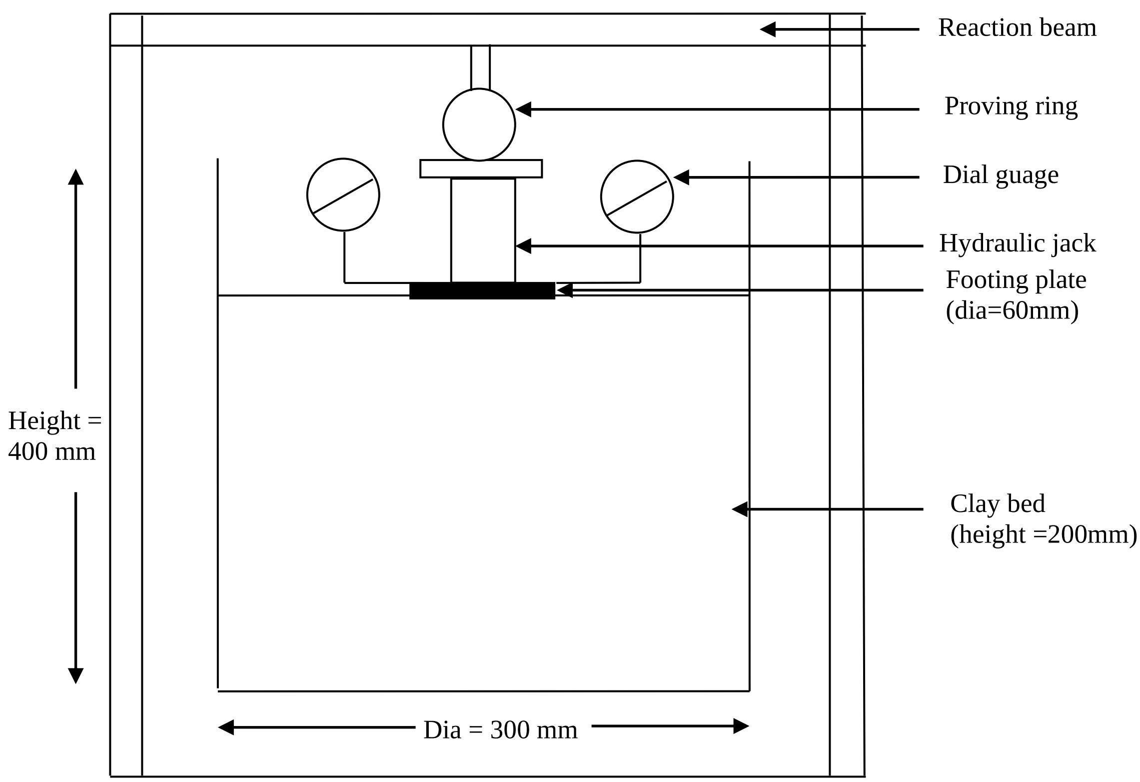


Figure 3.1. Experimental set-up for plate load tests

3.6.7.2. Heave tests:

As an introductory step leading to the performance of plate load tests in saturated condition, heave tests were performed on all the clay beds in which they were continuously wetted up to equilibrium heave which corresponded to saturation. For the load tests in saturated state, the plain clay and the additive-clay blends were compacted in layers in the test moulds at their respective *OMC* and *MDD*. After the compaction was over, the clay beds were wetted continuously adding water from the top. Heave of the clay beds was monitored with a dial gauge whose spindle rested on a weightless wooden surface plate of diameter 100mm kept on the top of the beds. Figure 3.2 depicts the arrangement for heave tests. Water was added from the top for 10days, a time period in which pilot tests showed near saturation. Heave of the clay beds was monitored corresponding to suitable time intervals.

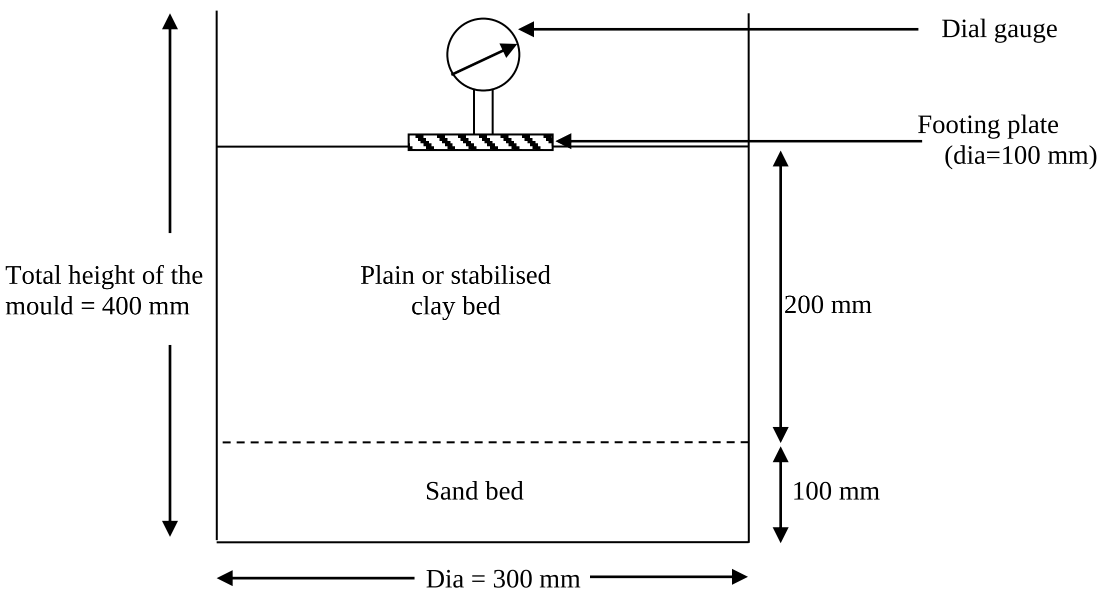


Figure 3.2. Experimental set-up for heave tests

3.6.7.3. Plate load tests in saturated condition:

When the saturation of the clay beds was ensured from their equilibrium heave (obtained in 10 days), tests were conducted on the saturated clay beds according to ASTM D 1196-93 (ASTM, 1997). In the tests in saturated condition, the highest stress increment was 40kPa in all the tests. After the plate load tests were performed, clay samples were collected at various depths for finding out water contents and degrees of saturation to verify whether all the clay beds were wetted to near saturation.

3.6.8. Swell-shrink tests:

As mentioned above, swell-shrink tests were conducted by subjecting the plain clay bed and the clay bed stabilised by 6% lime, 20% cement, 20% fly ash and 20% *GGBS* to five alternate swell-shrink cycles or wetting-drying cycles. Heave or swelling (mm) and shrinkage (mm) were monitored in these alternate swell-shrink cycles. As *FSI* decreased to the lowest at the above additive contents, these tests were performed only on the clay bed stabilised independently at these additive contents.

The test moulds fabricated for performing these tests were 300 mm dia and 400 mm high cylindrical moulds. Figure 3.3 shows the arrangement for swell-shrink tests. The thickness of all the clay beds compacted in these moulds was 200 mm. The plain clay bed and that stabilised by 6% lime, 20% cement, 20% fly ash and 20% GGBS were compacted at their respective *OMC* and *MDD*. All the clay beds were compacted in four layers the thickness of which was carefully monitored to ensure the required density.

After the compaction was over, the clay beds were wetted continuously by adding water. Swelling (mm) was monitored for 10 days with a dial gauge whose spindle rested on a weightless wooden surface plate, 100 mm dia, kept centrally on the top of the clay beds. Water required (litres) for saturation was determined and added to them from the top over a period of 10 days, a time period in which pilot tests showed near saturation. Thus, the swelling phase of each swell-shrink cycle spanned over 10 days. Swelling (mm) of the clay beds was monitored at suitable time intervals.

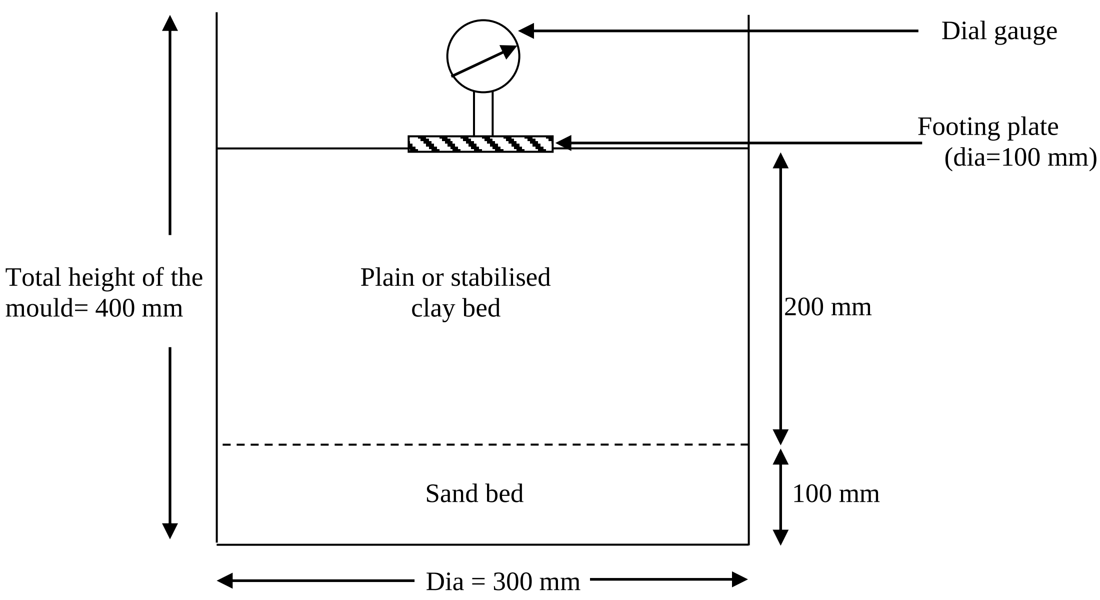


Figure 3.3. Experimental set-up for swell-shrink tests

After the clay beds attained equilibrium swelling, they were permitted to shrink. The wooden piece was removed during volume reduction or shrinkage, which was observed continuously for 50 days. At the end of the 1st cycle, the footing plate was kept on the clay bed for monitoring heave or swelling for the 2nd cycle. After the 2nd cycle, the process was repeated for the remaining cycles. Hence, in every cycle, swelling or heave for ten days and volume reduction or shrinkage for 50 days were monitored. Therefore, all the clay beds were monitored for 300 days in five cycles.

As the clay beds underwent shrinkage or volume reduction, cracks developed in them in two patterns, namely, polygonal shrinkage pattern and linear shrinkage pattern. The largest width (mm) of these two different types of cracks was determined every ten days of the period of 50 days shrinkage period in each cycle.

3.6.9. Quantification of desiccation cracks by image analysis:

A series of experiments was performed in which the plain expansive clay samples and the chemically stabilised expansive clay samples were prepared in shrinkage cups and allowed to dry up so that the pattern of shrinkage cracks developing in the samples could be quantified. Clay powder finer than 425μ m sieve was used for preparing the samples. About 50g of the air-dry clay powder was taken for preparing each sample – plain or treated chemically. Tests were performed on the plain clay and on the additive-clay blends. The air-dry clay and the amounts of the additives based on their dosages were thoroughly mixed for preparing the dry blend specimens. The moulding water contents for preparing the wet blend specimens were chosen as their respective *LL*, *PL*, *PI* and *OMC*. The thoroughly blended wet specimens were carefully transferred to the shrinkage cups which were patted gently in order to make the top surface level. Then, the specimens in the cups were left for air-drying for 48 hrs and then were oven-dried for 24 hrs at 105^0C temperature. After that, the samples were observed to have reduced in their volumes and diameters and to have developed polygonal shrinkage cracks or desiccation cracks. The samples also underwent linear shrinkage.

3.6.9.1. Image analysis:

Then, using a camera of 24 Mega pixels, pictures of all the shrunken samples were taken from a particular height (in this case 400mm) for ensuring clarity of the images. For quantifying the shrinkage or desiccation cracks by image analysis, these pictures or images of the samples were transferred to MATLAB and modified as grey scale images. Using the final binary images, the shrinkage area was evaluated.

3.6.10. FESEM analysis:

The microstructure of the chemically treated clay samples and the parent clay sample (untreated sample) was studied through FESEM analysis. Among the clay-additive blends, samples treated with 4% and 6% lime and 10% and 20% cement, fly ash and GGBS were considered. Or, clay samples treated with large amounts of additives were subjected to this analysis.

CHAPTER - 4

Influence of chemical stabilisation on index and engineering properties

4.1. Introduction:

This chapter discusses the influence of lime, cement, fly ash and GGBS on *FSI*, *LL*, *PI*, compaction, soaked *CBR*, hydraulic conductivity (*k*) and strength. As mentioned in Chapter 3, the amount of lime was varied as 0, 1, 2, 4 and 6%, and the other additive contents were varied as 0, 5, 10, 15 and 20% by dry weight of the soil for performing the above tests.

4.2. Experimental investigation:

The following sections describe the test materials and the different tests performed on the remoulded expansive clay in unmodified and chemically modified conditions.

4.2.1. Test materials:

4.2.1.1. Expansive clay: The clay used here was collected at 1.2m from Bhimavaram, A.P., India. It was dark in appearance and had a water content of 12%. It had an *FSI* of 134%. The clay was categorised as *CH* as per its LL (87%) and PI (58%) according to the *USC* System.

4.2.1.2. Chemical additives:

(a) *Lime:* Lime was used as $Ca(OH)_2$ in powder form.

(b) *Cement:* Ordinary Portland cement in powder form was used as a blend material.

(c) Fly ash: Class-C fly ash (also in powder form) was obtained from VTPS, Vijayawada, A.P., India.

(d) GGBS: It was obtained from Venspra Labs Limited, Vijayawada, A.P., India. It was also in powder form.

4.2.2. Tests conducted, variables studied and test procedures:

LL, *PL* and *FSI* of the plain clay and the clay blended with lime, cement, fly ash and GGBS were determined. For these tests, mixes of clay powder and additive were prepared varying lime content as 1, 2, 4 and 6%, cement as 5, 10, 15 and 20%, fly ash as 5, 10, 15 and 20% and GGBS content as 5, 10, 15 and 20% by dry weight of the clay.

Standard Proctor compaction tests, hydraulic conductivity tests, unconfined compressive strength (*UCS*) tests, California bearing ratio (*CBR*) tests were also conducted on the plain clay and the clay mixed with the above-mentioned dosages of the additives. For hydraulic conductivity tests, the samples were saturated for 3 days before the performance of the tests. In the *UCS* tests, the cylindrical specimens were cured for 0, 7, 14 and 28 days. *CBR* was determined in soaked condition.

Plate load tests were also performed on the additive-clay blends in two series: (a) in as-compacted condition and (b) in saturated condition. A series of swell-shrink tests was also conducted on the plain clay bed and the clay bed stabilised independently by 6% lime, 20% cement, 20% fly ash and 20% GGBS subjecting them to five alternate swell-shrink cycles or wetting-drying cycles. All these tests have been described in detail in Section 3.6.

A lone 1-*D* swell-consolidation test was performed on the unblended expansive clay specimen to assess its swell potential and swelling pressure. It is described in detail in the following section:

4.2.2.1. One–dimensional (1-D) swell-consolidation test on the unblended expansive soil:

A 1-*D* swell-consolidation test was done in order to determine the swell potential (*S%*) and swelling pressure (p_s) of the unblended expansive soil as an understanding of these swelling characteristics of the expansive soil is required. In the 1-*D* swell-consolidation test on the expansive soil, the initial water content (w_i) and the dry unit weight (γ_d), which were the placement conditions at which the test was performed, were kept constant at 12kN/m^3 and 0% respectively. The dry unit weight of 12kN/m^3 was arbitrarily chosen for convenience of compaction; and the water content of 0% was chosen in order to obtain the highest amount of heave or swell potential at that dry unit weight. Oven-dried expansive soil passing 4.75mm sieve was used for the test.

Oven-dry clay, the weight of which was determined as per the γ_d and the oedometer ring volume, was compacted in the ring so that uniform density was maintained. Using filter papers and porous stones for double drainage and placing the loading pad centrally, the entire assembly was placed on the loading frame for axial application of compressive loads. The sample was inundated for free swell under a surcharge pressure of 5kPa. Swelling was monitored continuously until equilibrium.

Swell potential (*S%*) can be written as

$$S\,(\%) = \frac{\Delta H}{H}\;X\;100 \qquad (4.1)$$

where ΔH is heave or increase in thickness and H is the initial thickness.

Under increased stresses, the specimen was subjected to consolidation until it attained its initial void ratio (e_0). Making a graph between void ratio (e) and log of applied pressure (log p), swelling pressure (p_s) was determined.

4.3. Discussion of test results:

4.3.1. Influence of additives on LL, PL and PI:

Figures 4.1, 4.2 and 4.3 respectively depict the effect of additives on *LL*, *PL* and *PI*. As can be seen in Figure 4.1, *LL* decreased significantly with increasing amounts of additives. As chemical reagents are added to an expansive clay, flocculation occurs as a result of pozzolanic reaction which leads to increase in size of particle. Hence, *LL* decreases as it depends on size of the particle. The smaller the particle size of the clay, the higher would be the value of *LL* because *LL* is the amount of water at which a clay material behaves like a liquid. In the case of clays having finer particles, the higher surface area of the particles would require more amount of water at which the material would behave like a liquid. Therefore, when flocculation occurs in the expansive clay because of the addition of chemical reagents such as lime, *LL* decreases as the amount of water required by the blend of increased particle size to behave like a liquid decreases. *LL* decreased from 87% to 72.7% when the lime content increased from 0% to 4% and 6%, showing that 4% lime is the optimum lime content. The % reduction in *LL* was 16.44% in the case of lime. In the cases of cement, fly ash and GGBS, *LL* decreased continuously up to the additive content of 20%. For example, *LL* decreased by 21%, 16% and 19.5% respectively in the cases of cement, fly ash and GGBS when the dosage was 20%.

PL increased notably with increasing amounts of lime, cement, fly ash and GGBS as can be seen from Figure 4.2. Increase in *PL* also can be attributed to flocculation occurring in the additive-clay blends. *PL* increased from 28% to 32% when the lime increased from 0% to 4% and 6%, indicating an increase of 14%. In the cases of cement, fly ash and GGBS, *PL* respectively increased by 16%, 18% and 19% when the dosage was 20%. *PI*, decreased with increasing amounts of lime, cement, fly ash and GGBS. The variation of *PI* with additive content for the above reagents is shown in Figure 4.3. *PI* decreased from 59% to 41% when the lime amount increased from 0% to 4%, reflecting a reduction of 31%. *PI* did not decrease further at the lime content of 6% as *LL* and *PL* also did not show any change when the lime increased from 4% to 6%. So, with the addition of higher lime contents, the lime-clay blends were rendered low plastic or non-plastic. In the cases of cement, fly ash and GGBS, *PI* respectively decreased by 39%, 32% and 38% when the dosage was 20%. These data also indicated notable reduction in *PI*. Table 4.1 summarises the data on *LL*, *PL* and *PI*.

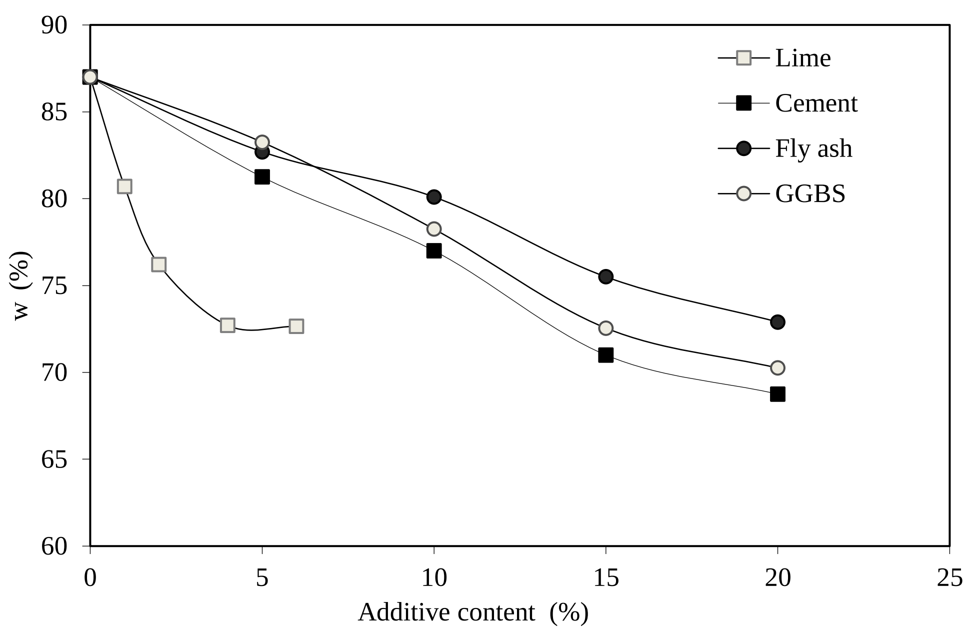


Figure 4.1. Influence of chemical additives on LL

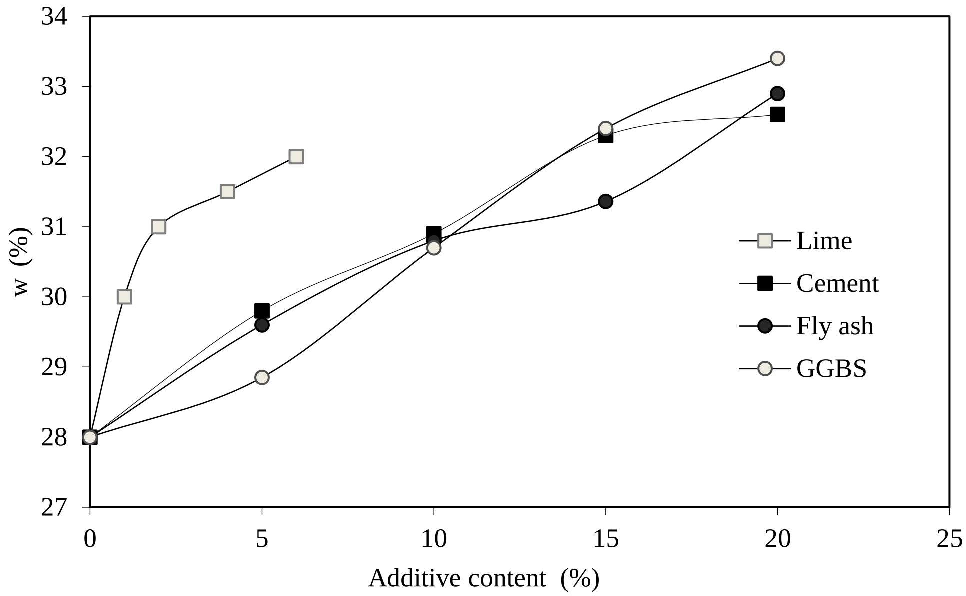


Figure 4.2. Influence of chemical additives on plastic limit (PL)

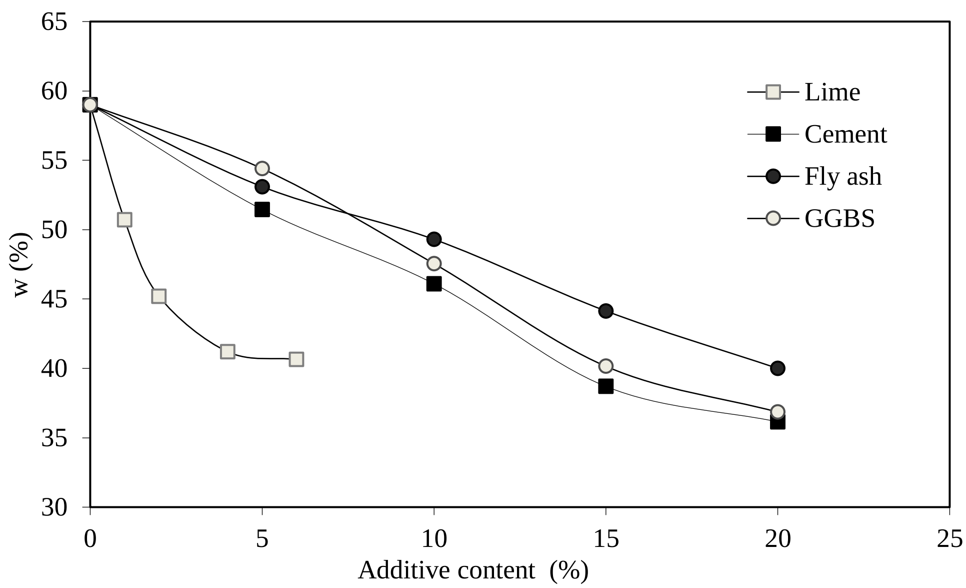


Figure 4.3. Effect of chemical additives on plasticity index (PI)

4.3.2. Influence of chemical additives on free swell index:

Free swell index (*FSI*) indicates the inherent potential they have for swelling. *FSI* indicates whether the soil is highly expansive or moderately expansive or non-expansive. Chemical treatment of expansive soils is well known to reduce *FSI* and decrease their potential for swelling. As chemical reagents are added to an expansive clay, cation exchange and flocculation occur as a result of pozzolanic reaction. This leads to increase in particle size, which decreases

FSI. *FSI* values are higher for clays of finer particles. Further, when expansive clay particles are replaced by non-expansive chemical additives, it contributes to reduction in *FSI*. Figure 4.4 shows the variation of *FSI* with additive content for lime, cement, fly ash and GGBS. *FSI* decreased from 134% to 86.36% when the lime content increased from 0% to 6%, showing a reduction of 35.55%. *FSI* respectively decreased to 83.33%, 83.33% and 72.73% in the cases of cement, fly ash and *GGBS* when the dosage was 20%, indicating a reduction of 38% in the case of cement and fly ash and 46% in the case of GGBS. The data on *FSI* show that chemical treatment with lime, cement, fly ash and GGBS considerably decreased the potential expansiveness of the soil. The *FSI* data correspond to the previous research on chemically modified expansive soils (Phanikumar and Sharma, 2004; Phanikumar, 2009; Phanikumar et al. 2015). Table 4.1 shows the effect of amounts of various additives on *FSI*.

Table 4.1. Effect of chemical additives on index properties of the expansive clay

	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	FSI (%)
Expansive clay	87.00	28.00	59.00	134.00
1% lime	80.70	30.00	50.70	113.04
2% lime	76.20	31.00	45.20	107.14
4% lime	72.70	31.50	41.20	95.00
6% lime	72.65	32.00	40.65	86.36
5% cement	81.25	29.80	51.45	127.27
10% cement	77.00	30.90	46.10	102.27
15% cement	71.00	32.30	38.70	95.45
20% cement	68.75	32.60	36.15	83.33
5% fly ash	82.70	29.60	53.10	121.74
10% fly ash	80.10	30.80	49.30	104.17
15% fly ash	75.50	31.36	44.14	93.48
20% fly ash	72.90	32.90	40.00	83.33
5% GGBS	83.25	28.85	54.40	114.58
10% GGBS	78.25	30.70	47.55	104.35
15% GGBS	72.55	32.40	40.15	75.00
20% GGBS	70.25	33.40	36.85	72.73

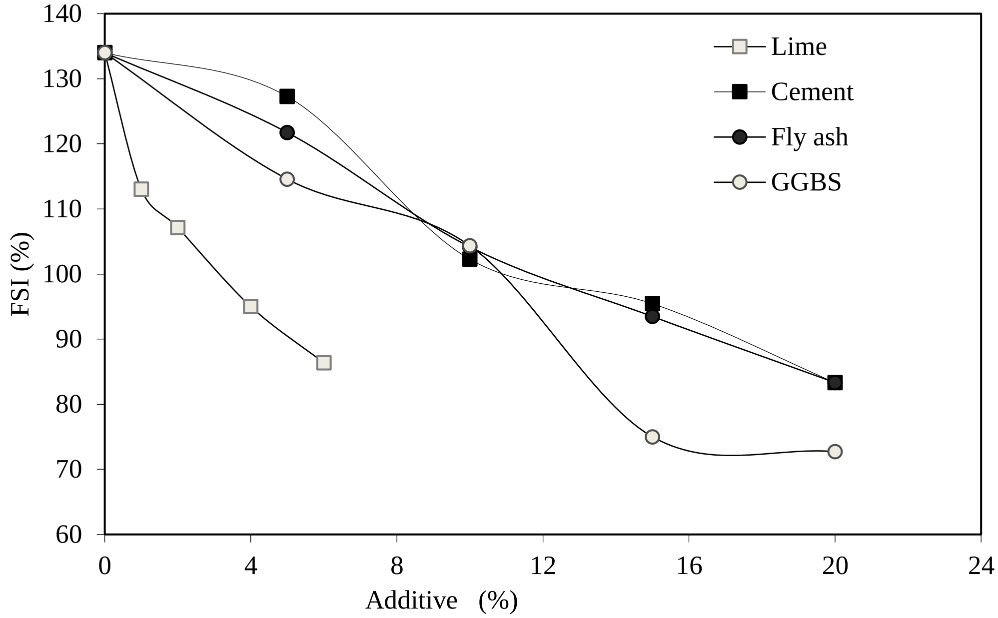


Figure 4.4. Influence of chemical additives on FSI

4.3.3. Rate and amount of heave, swell potential and swelling pressure of the unblended expansive clay:

Figure 4.5 depicts the rate of heave of the plain clay in the form of heave (mm) – log time (minutes) plot. The data show that the equilibrium heave or swelling was attained by the specimen in 3 days. The equilibrium heave was confirmed by the curve becoming asymptotic with the X-axis. The plain clay resulted in a heave or swelling of 1.70 mm.

Swell potential (*S%*) determined from equation (4.1) was, therefore, equal to 8.5%. Figure 4.6 depicts the e-log p curve of the plain clay. The curve shows that the initial void ratio (e_0) of the specimen was 1.242 which increased to 1.433 upon swelling.

This new void ratio of 1.433 was the equilibrium void ratio. The swelling pressure (p_s) of the unblended specimen was found from the e-log p curve to be 220kPa. This pressure corresponded to the initial void ratio (e_0) of 1.242 as the figure shows.

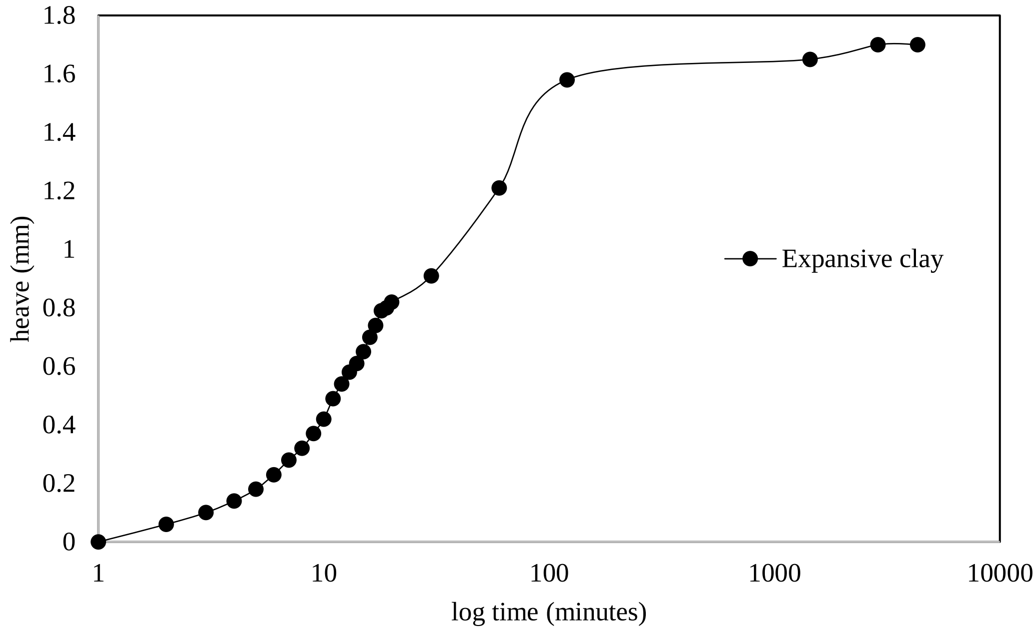


Figure 4.5. Heave profile

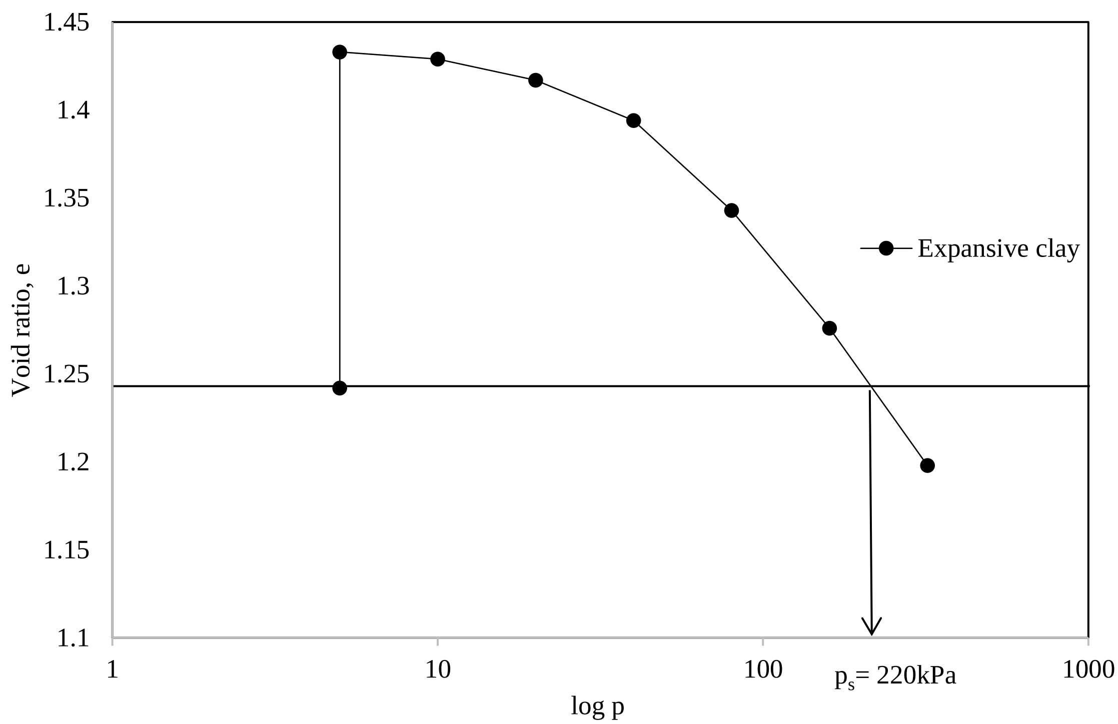


Figure 4.6. e - log p curve

4.3.4. Influence of lime, cement, fly ash and GGBS on compaction characteristics:

Figures 4.7 to 4.10 show the compaction characteristics of lime-clay, cement-clay, fly ash-clay and GGBS-clay blends for varied dosages of lime, cement, fly ash and GGBS. The data shown in the figures indicate that the compaction curves got shifted upwards and towards the left indicating that the clay-additive blends were stabilised and resulted in higher γ_d values for given water contents compared to the unblended clay. This behaviour led to increased values of maximum dry density (*MDD*) and reduced values of optimum moisture contents (*OMC*). When additives such as lime, cement, fly ash and GGBS are added to a clay, flocculation occurs leading to higher densities at given water contents. This indicates that the clay got stabilised upon addition of chemical reagents. The increase in dry densities depends on the degree of flocculation which, in turn, depends on the type of the additive used.

For example, the influence of lime on compaction characteristics as shown in Figure 4.7 indicates that *MDD* increased to its highest value of 15kN/m^3 corresponding to an *OMC* of 28.4% when the lime content increased to 4% which is the optimum lime content. When the lime content increased to 6%, however, *MDD* slightly decreased to 14.77kN/m^3 corresponding to a marginally increased *OMC* of 29.5%. The other lime contents of 1% and 2% also shifted the compaction curves upward and towards the left.

Figure 4.8 shows that the compaction characteristics were significantly influenced by the addition of cement also. The compaction curves were shifted upwards and towards the left when the expansive clay was modified by cement, indicating that the clay soil got stabilised. Addition of cement ensured that the dry density increased for all water contents, which can be attributed to flocculation and pozzolanic reaction. *MDD* increased and *OMC* decreased continuously with increasing cement content. *MDD* increased from 13.42kN/m^3 to 15.00kN/m^3 and *OMC* decreased from 31.5% to 28% when the cement content increased from 0% to 20%.

Figures 4.9 and 4.10 respectively show the influence of fly ash and GGBS on the compaction characteristics of the expansive soil. With the addition of fly ash and GGBS also the dry density increased for all the water contents, indicating that the soil was stabilised. However, the influence was the highest at 10% fly ash and 10% GGBS. *MDD* increased from 13.42kN/m^3 to

13.73kN/m^3 and 13.6kN/m^3 respectively when the fly ash content and GGBS content increased from 0% to 10% and thereafter the *MDD* marginally decreased. This establishes that a dosage of 10% could be the optimum in the case fly ash and GGBS. While *OMC* remained the same at 31.5% (the *OMC* of the unblended clay) even when the fly ash content increased to 10%, the *OMC* in the case of 10% of GGBS slightly increased to 33%. It can be seen that the influence of fly ash and GGBS was not as much as that of lime and cement. Table 4.2 summarises the compaction data.

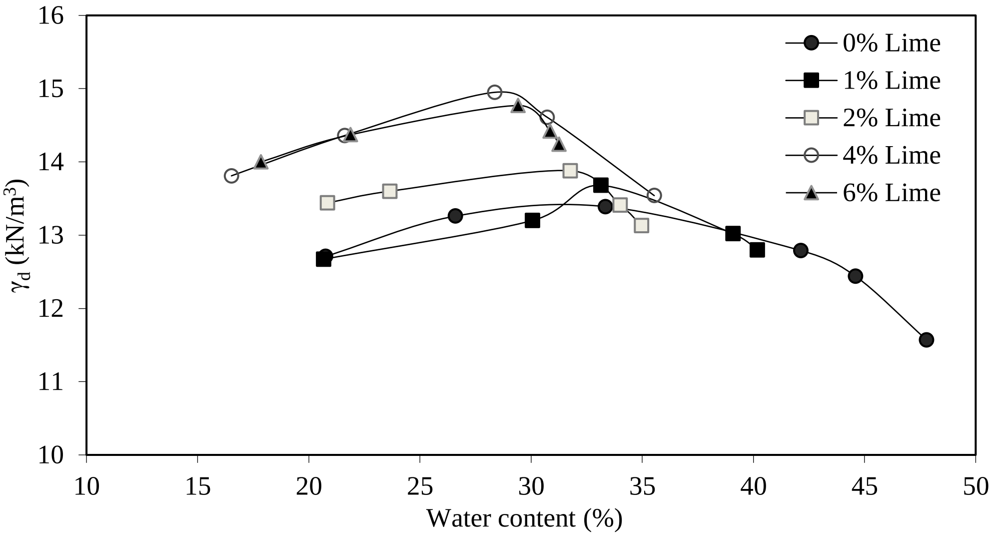


Figure 4.7. Influence of lime on compaction characteristics

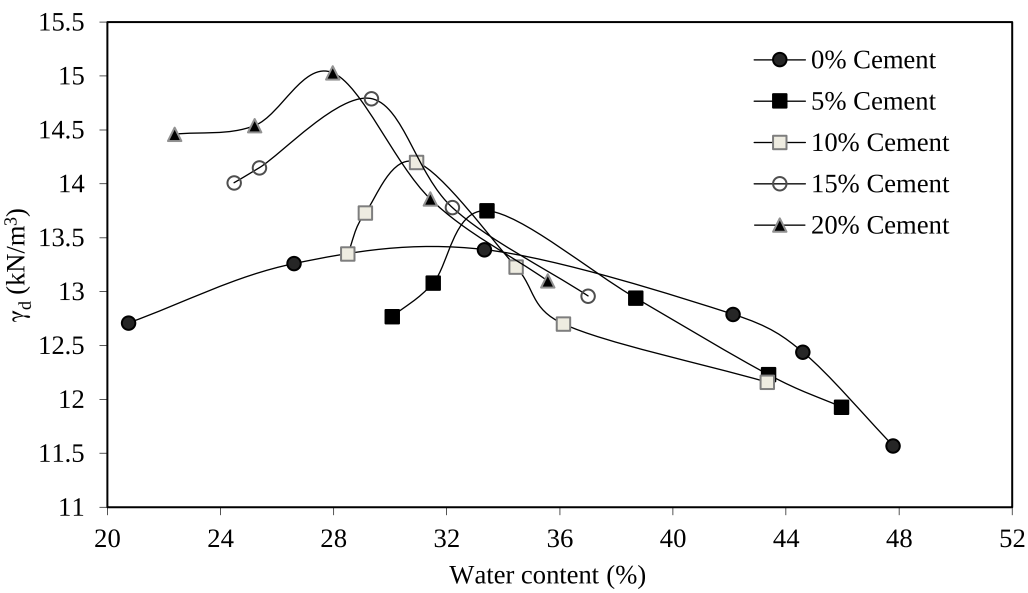


Figure 4.8. Effect of cement on compaction characteristics

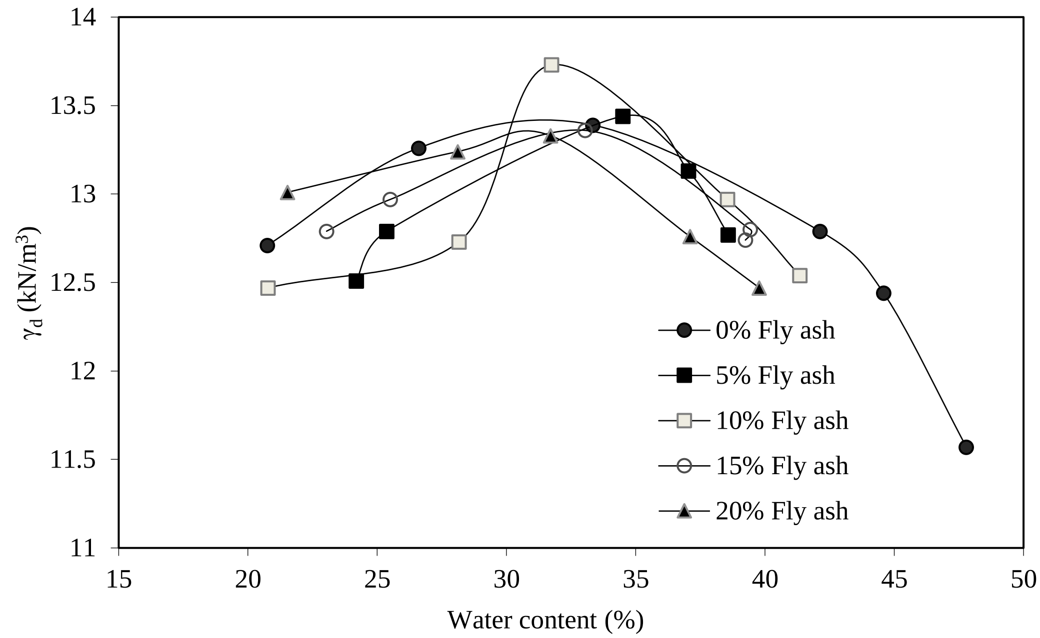


Figure 4.9. Influence of fly ash on compaction

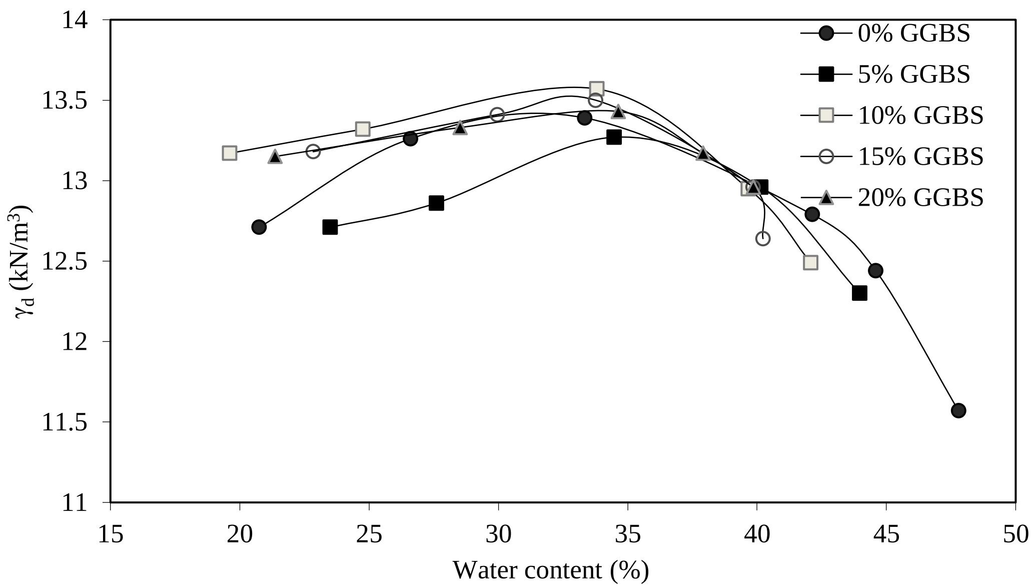


Figure 4.10. Influence of GGBS content on compaction

4.3.5. Influence of lime, cement, fly ash and GGBS on hydraulic conductivity:

Hydraulic conductivity (k, cm/sec) was determined by conducting variable head permeameter tests on the plain clay (0% additive) and the clay blended with various dosages of lime, cement, fly ash and GGBS. All the samples were compacted at their respective *OMC* and *MDD*. Figure 4.11 shows the variation of hydraulic conductivity with the amounts of lime, cement, fly ash and *GGBS*. Hydraulic conductivity (*k*, cm/sec) decreased significantly with increasing amounts of cement, fly ash and *GGBS*. The reduction in hydraulic conductivity can be attributed to increase in the maximum dry density of the blend samples with increasing amounts of cement, fly ash and *GGBS* (see Table 4.2). As the maximum dry density of the blend samples at which their hydraulic conductivity was determined increases, the void ratio and the porosity of the blends decrease, and hence, flow of water through the blend samples is reduced, resulting in reduced hydraulic conductivity.

For example, the hydraulic conductivity (*k*) of the cement-clay blends was 2.66 x 10^{-7}cm/sec, 2.00 x 10^{-7}cm/sec, 1.31 x 10^{-7}cm/sec and 6.57 x 10^{-8}cm/sec respectively for the cement contents of 5%, 10%, 15% and 20%. Similar pattern of reduction in hydraulic conductivity was observed in the cases of fly ash-clay blends and *GGBS*-clay blends also (see Table 4.3). But, the reduction in the hydraulic conductivity of the cement-clay blends was the highest because the increase in the *MDD* with cement content was the highest (see Table 4.2). Previous research on hydraulic conductivity of fly ash-stabilised expansive clay liner (Phanikumar and Umashankar, 2016) also presented similar data at higher fly ash contents.

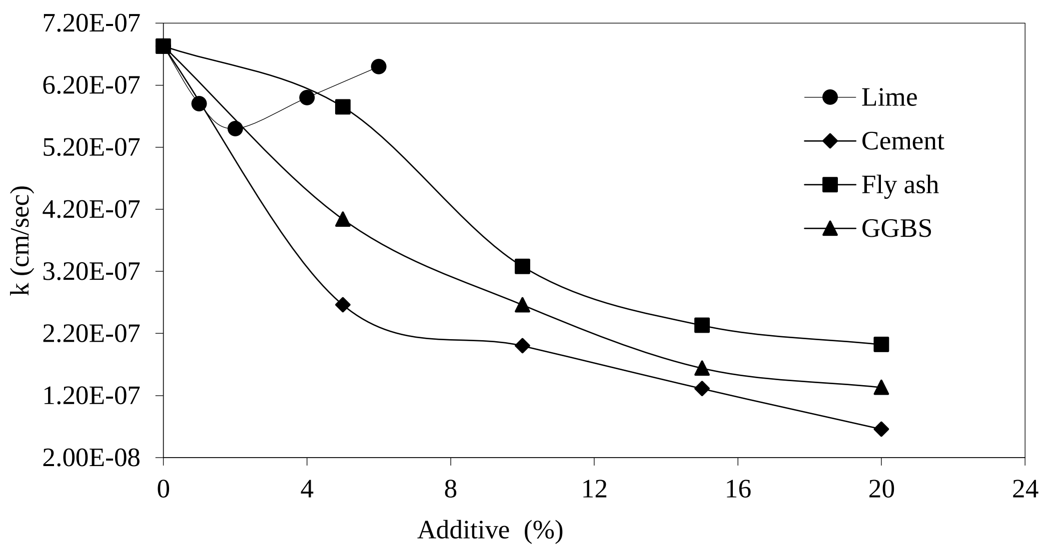


Figure 4.11. Variation of hydraulic conductivity

Regarding the hydraulic conductivity of lime-clay blends, it can be observed from Figure 4.11 that the hydraulic conductivity decreased up to 2% lime and then it increased at higher lime contents. Up to 2% lime, increase in *MDD* and the corresponding reduction in void ratio and porosity were influential in reducing the hydraulic conductivity. However, at higher lime contents, heat release was possible leading to more absorption of water, resulting in heave. This caused increased void space facilitating increase in hydraulic conductivity. Similar observations were made by Umashankar and Phanikumar (2016) and Phanikumar and Umashankar (2016) in their study on *FA*-stabilised clay liners. The hydraulic conductivity was found to be 6.83 x 10^{-7}cm/sec, 5.90 x 10^{-7}cm/sec, 5.50 x 10^{-7}cm/sec, 6.00 x 10^{-7}cm/sec and 6.50 x 10^{-7}cm/sec respectively for the lime contents of 0, 1, 2, 4 and 6%.

Table 4.2. Influence of chemical additives on compaction characteristics

	OMC (%)	MDD (kN/m^3)
Expansive clay	31.50	13.42
1% lime	33.00	13.68
2% lime	31.70	13.88
4% lime	28.40	14.95
6% lime	29.50	14.77
5% cement	33.50	13.75
10% cement	31.00	14.2
15% cement	29.50	14.79
20% cement	28.00	15.03
5% fly ash	35.00	13.45
10% fly ash	31.75	13.73
15% fly ash	33.00	13.37
20% fly ash	31.00	13.34
5% GGBS	34.50	13.27
10% GGBS	33.00	13.58
15% GGBS	33.00	13.52
20% GGBS	34.00	13.44

Table 4.3. Influence of chemical additives on hydraulic conductivity

	Hydraulic conductivity (cm/sec)
Expansive clay	6.83×10^{-7}
1% lime	5.90×10^{-7}
2% lime	5.50×10^{-7}
4% lime	6.00×10^{-7}
6% lime	6.50×10^{-7}
5% cement	2.66×10^{-7}
10% cement	2.00×10^{-7}
15% cement	1.31×10^{-7}
20% cement	6.57×10^{-8}
5% fly ash	5.85×10^{-7}
10% fly ash	3.28×10^{-7}
15% fly ash	2.33×10^{-7}
20% fly ash	2.02×10^{-7}
5% GGBS	4.04×10^{-7}
10% GGBS	2.66×10^{-7}
15% GGBS	1.64×10^{-7}
20% GGBS	1.33×10^{-7}

4.3.6. Effect of chemical additives on stress-strain characteristics:

Figures 4.12 to 4.15 respectively show the stress-strain behaviour of the clay blended with lime (0%, 1%, 2%, 4% and 6%) and cement, fly ash and *GGBS* (0%, 5%, 10%, 15% and 20%) for 0-days curing. When the clay was blended with lime, the axial stress required to be applied on the specimen for a given % strain increased with increasing lime content up to 4% which was the optimum lime content and thereafter it decreased when the lime content increased to 6% (see Figure 4.12). Addition of lime causes flocculation in the specimen which helps resist the applied axial stress to an improved degree, thus resulting and increased failure stress or peak stress. However, at 6% of lime, peak stress decreased as the flocculation was less effective than at 4% which was the reason why the *MDD* decreased at 6% lime compared to 4% lime. The peak stress was respectively 207kPa, 214kPa, 233kPa, 297kPa and 282kPa at the lime contents of 0%, 1%, 2%, 4% and 6%. Similar data were observed for other curing periods also. Figure 4.13 shows the stress-strain behaviour of cement-clay blends for 0-days curing. The data indicate that the axial stress required to be applied on the specimens for a given % strain continuously increased with increasing cement content. Cement imparts strength to the clay in a notable fashion by virtue of its intrinsic properties of flocculation and binding. The peak stress was respectively 207kPa, 227kPa, 250kPa, 271kPa and 302kPa at the cement contents of 0%, 5%, 10%, 15% and 20%. Similar data were observed for other curing periods also.

Figures 4.14 and 4.15 respectively show the stress-strain characteristics for fly ash-clay blends and GGBS-clay blends for 0-days curing. The data shown in the figures show that the axial stress required to be applied on the blend specimens in both the cases increased up to 10% and decreased at the additive contents of 15% and 20%. The peak stress increased to 244kPa and 214kPa respectively for 10% fly ash and 10% GGBS. The stress-strain characteristics of the blend specimens show that lime and cement helped resist the applied stresses better.

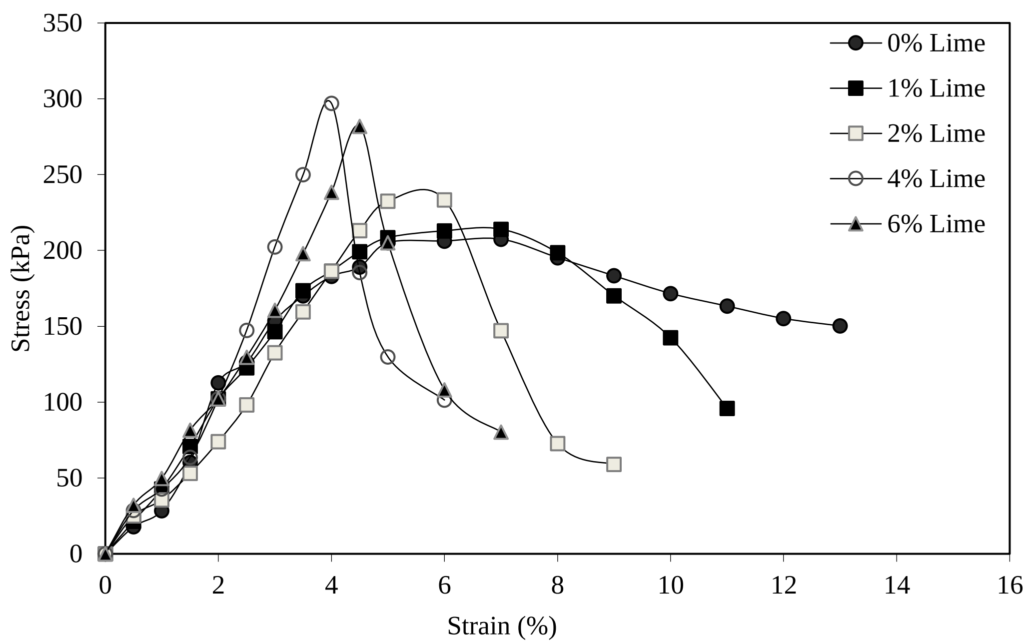


Figure 4.12. Stress-strain behaviour (0 days curing)

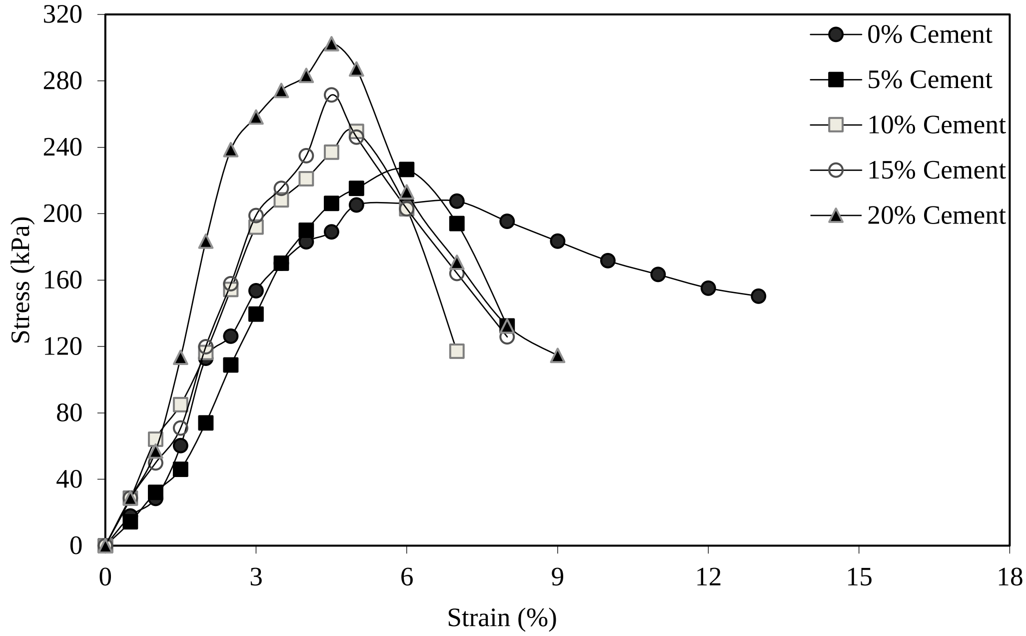


Figure 4.13. Stress-strain behaviour (0 days curing)

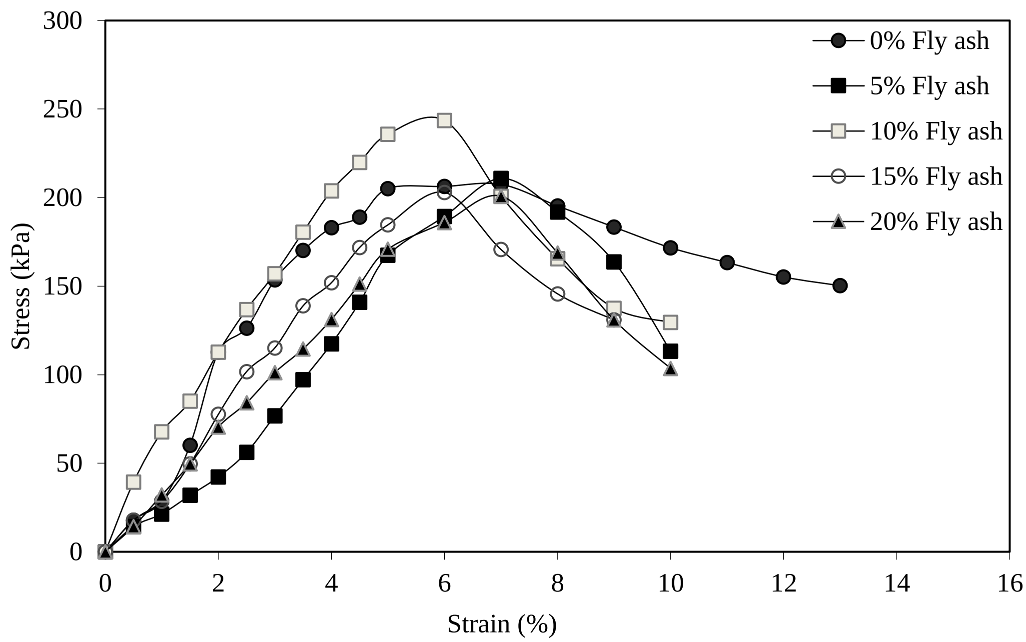


Figure 4.14. Stress-strain behaviour (0 days curing)

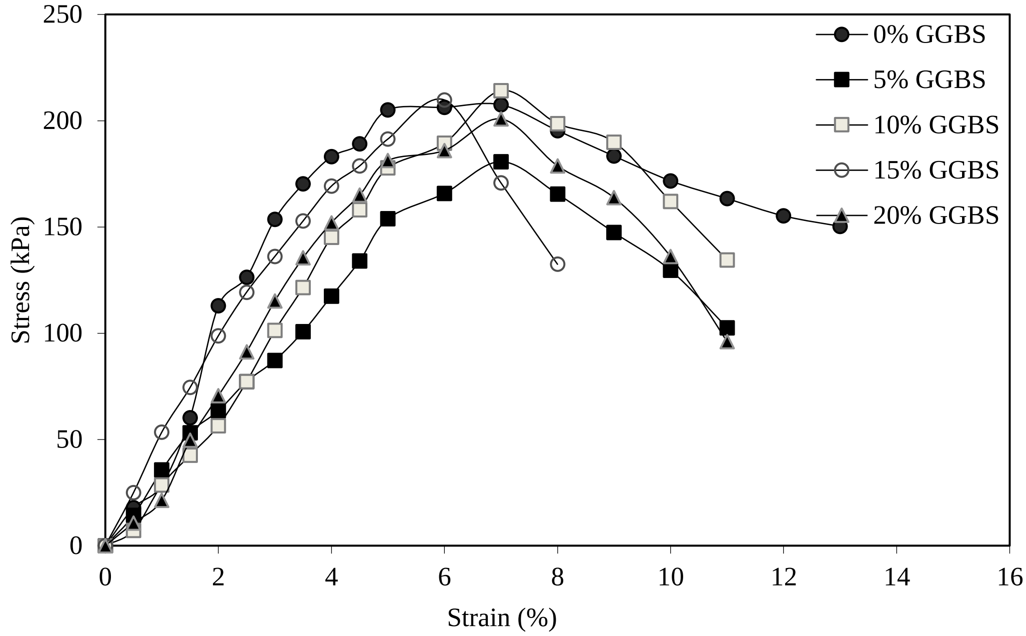


Figure 4.15. Stress-strain behaviour (0 days curing)

Figures 4.16 to 4.19 respectively show the effect of curing period on the stress-strain behaviour of the clay blended with 6% lime and 20% of cement, fly ash and GGBS. The data only on the highest amounts of additives are considered though similar data were observed at other additive contents also. With increasing curing period, cementitious products developed in the additive-clay blends as a result of the second reaction called cementation, which is time-bound in nature. These cementitious products harden the blend samples and impart a high strength to them so that they stiffly resist the applied axial compressive stresses. The resistance offered by the blend samples to the applied stresses depends upon the type of additive used and the degree to which the cementitious products develop.

The stress-strain data on the 6% lime as shown in Figure 4.16 indicate that, for a given % strain, the axial compressive stress required to be applied on the specimen continuously increased with increased curing periods. As already discussed, this is attributable to the development of cementitious products in the blend as it is kept for curing. The peak stress at failure or the UCS was respectively 282kPa, 422kPa, 426kPa and 478kPa for the curing periods of 0, 7, 14 and 28-days. Figure 4.17 also shows a similar stress-strain data for the 20% cement specimen. However, the peak stresses or the failure stresses were much higher than those shown in Figure 4.16 for the 6% lime. Cementitious products developed in the 20% cement specimen with increasing curing period are the strongest because cement is known for its ability to lend strength. Hence, the resistance offered by the 20% cement specimen to the applied axial compressive stress would be the stiffest. The peak stress at failure was respectively 302kPa, 910kPa, 1234kPa and 1805kPa for the curing periods of 0, 7, 14 and 28-days.

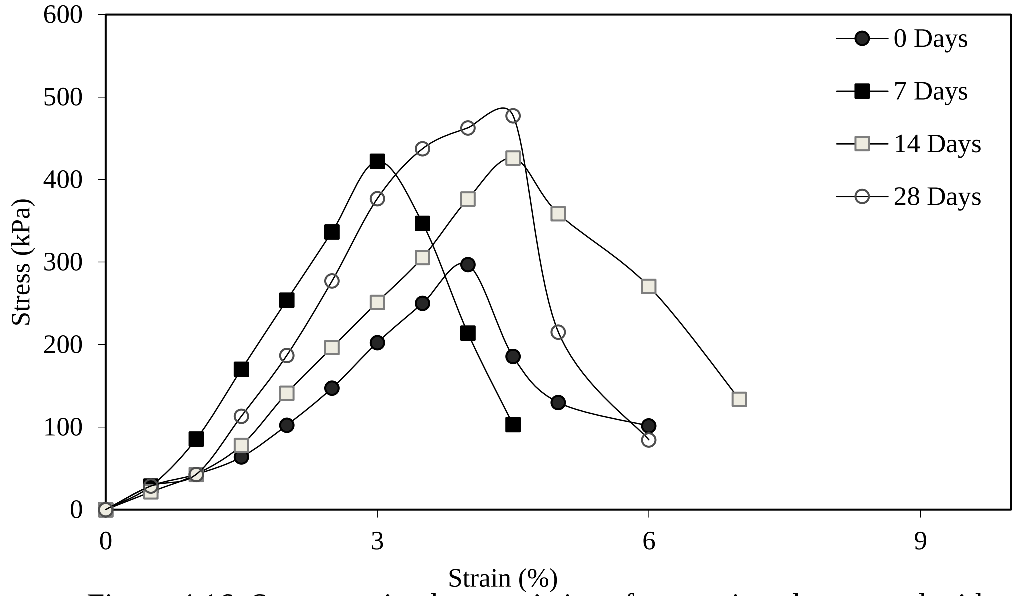


Figure 4.16. Stress-strain characteristics of expansive clay treated with 6% lime content

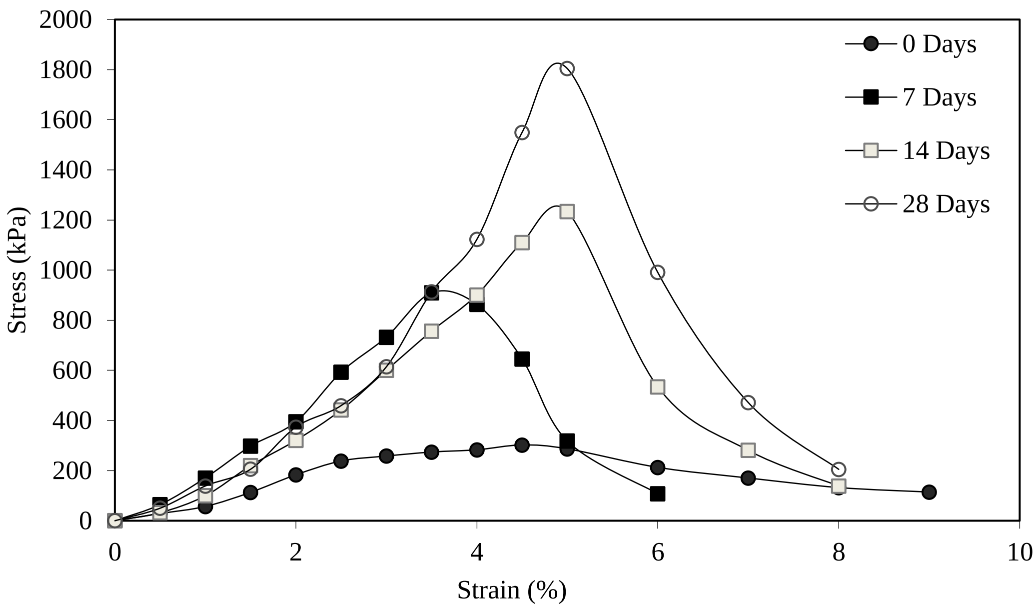


Figure 4.17. Stress-strain characteristics of expansive clay treated with 20% cement content

Similar stress-strain data were observed in the cases of 20% fly ash specimen and 20% GGBS specimen also for different curing periods (Figures 4.18 and 4.19). The behaviour is attributed to the development of cementitious products. However, the peak stresses were much lower than those observed in the 20% cement specimen. At 28 days the peak stress at failure was 318kPa for the 20% fly ash specimen and 729kPa for the 20% GGBS specimen.

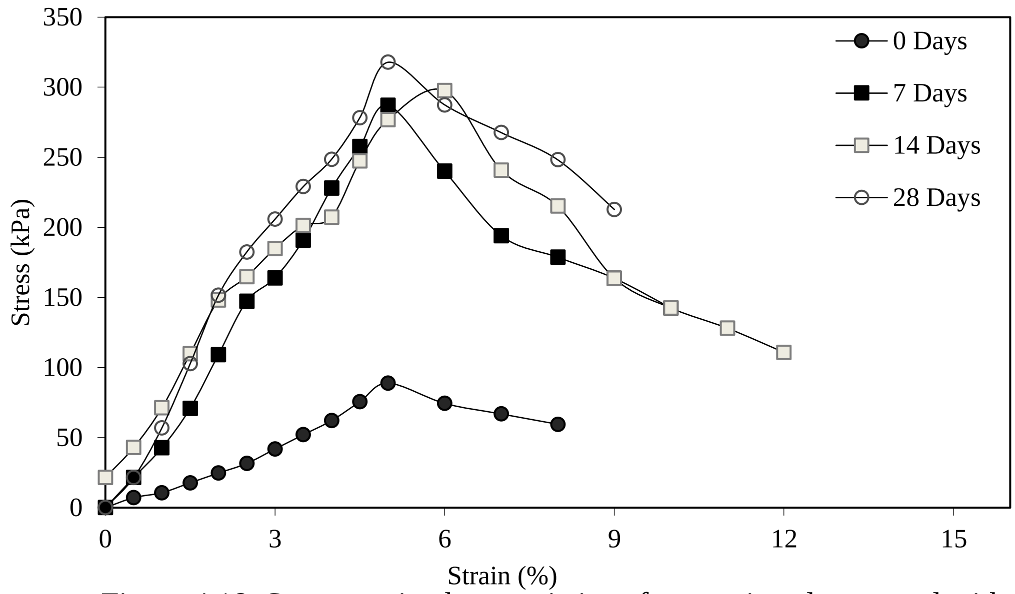


Figure 4.18. Stress-strain characteristics of expansive clay treated with 20% fly ash content

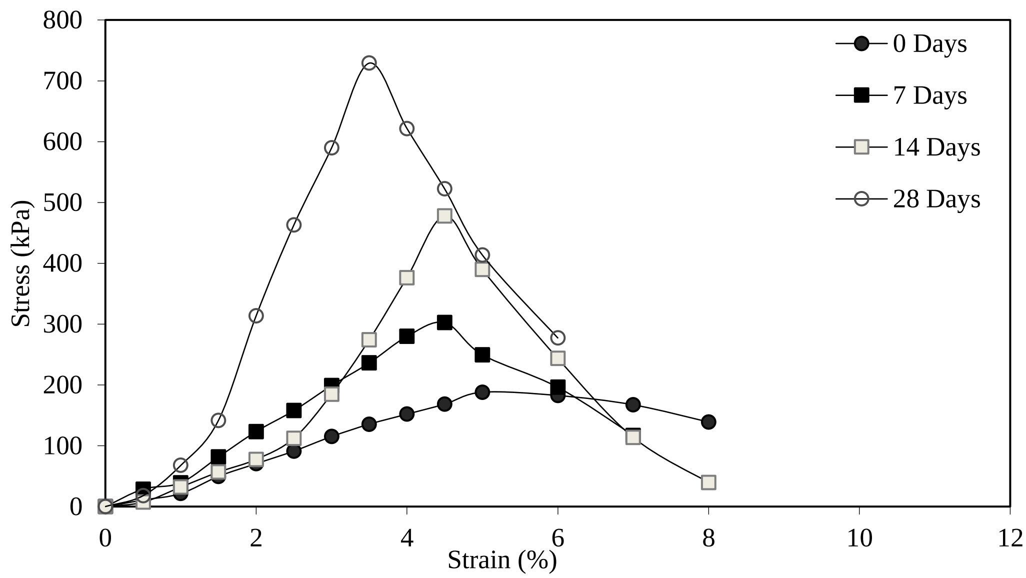


Figure 4.19. Stress-strain characteristics of expansive clay treated with 20% GGBS content

Figures 4.20 to 4.23 summarise the data on peak stress corresponding to varied additive content and curing period for lime, cement, fly ash and GGBS. The data indicate that peak stress increased significantly with increasing curing period. This was true for all additive contents for all types of additives. Similarly at curing periods other than 0-days, peak stress increased with additive content for all types of additives.

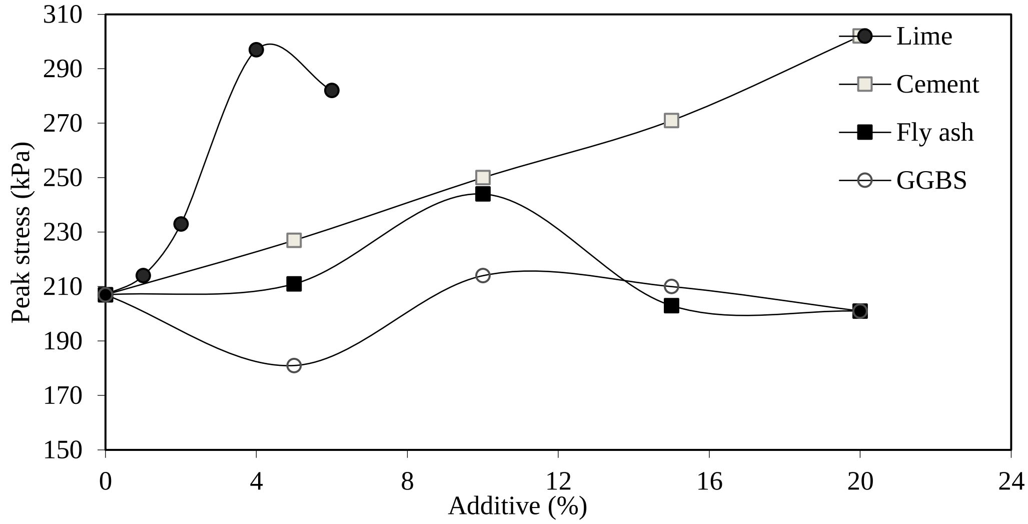


Figure 4.20. Variation of peak stress (0 days curing)

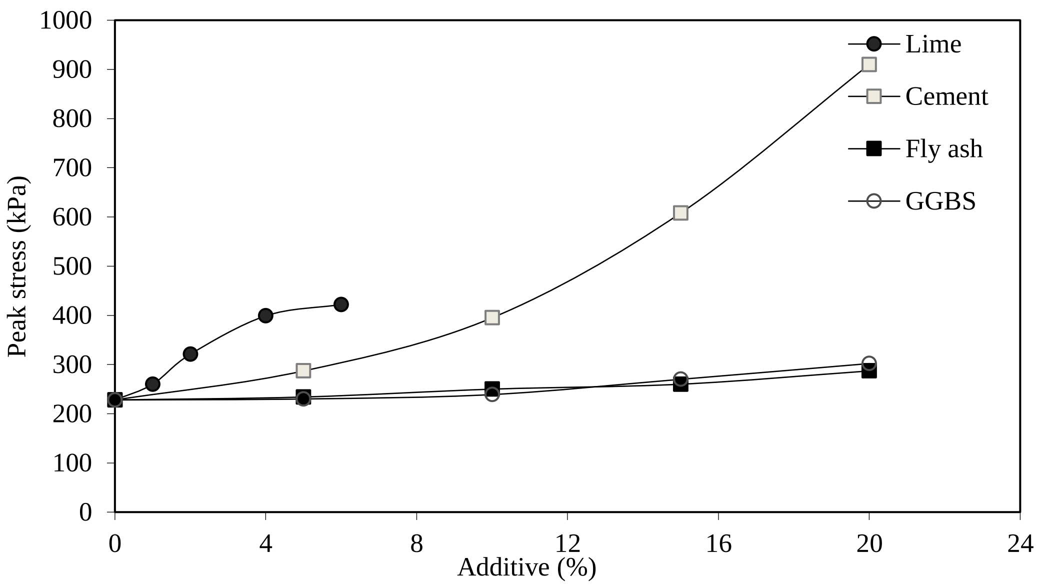


Figure 4.21. Variation of peak stress (7 days curing)

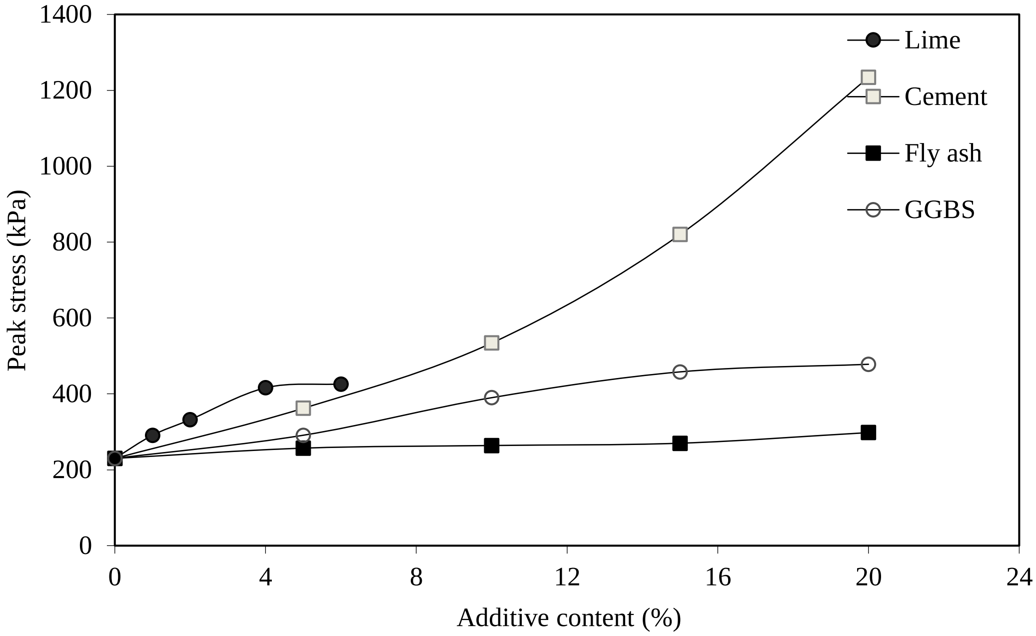


Figure 4.22. Variation of peak stress (14 days curing)

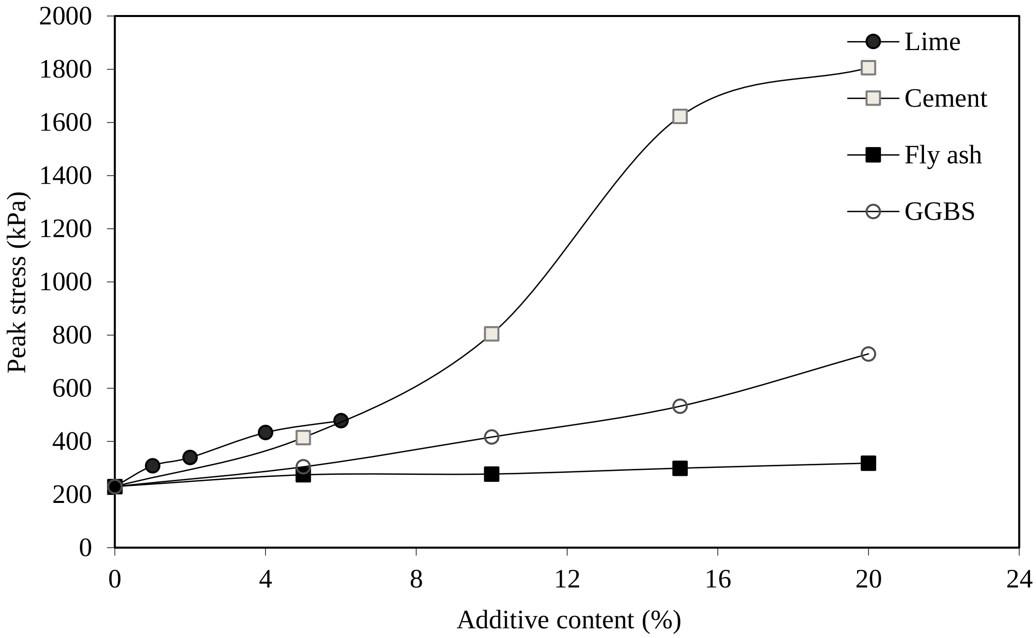


Figure 4.23. Variation of peak stress (28 days curing)

Figures 4.24 to 4.27 summarise the data on failure strain (%). The failure strain (%) decreased with increasing curing period, a phenomenon which was found to be true for all additive contents for all types of additives. Similarly, failure strain (%) decreased with additive contents for all types of additives, a fact which was true for all curing periods. Table 4.4 summarises the data on *UCS* and percent peak strain.

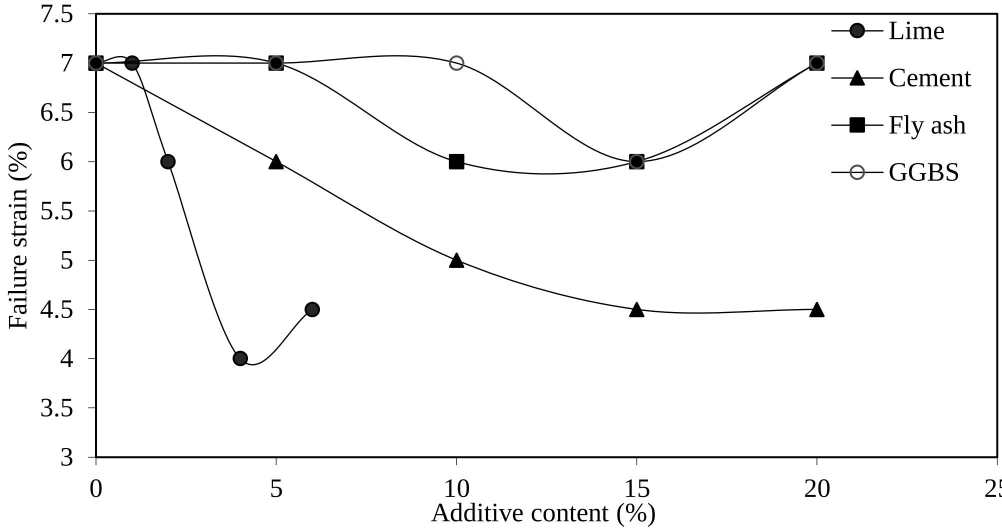


Figure 4.24. Variation of failure strain (0 days curing)

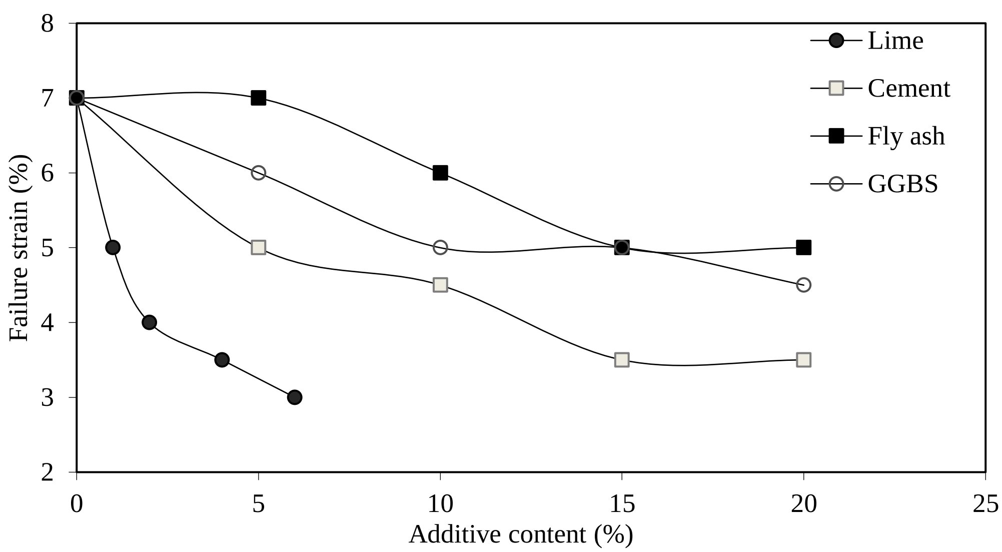


Figure 4.25. Variation of failure strain (7 days curing)

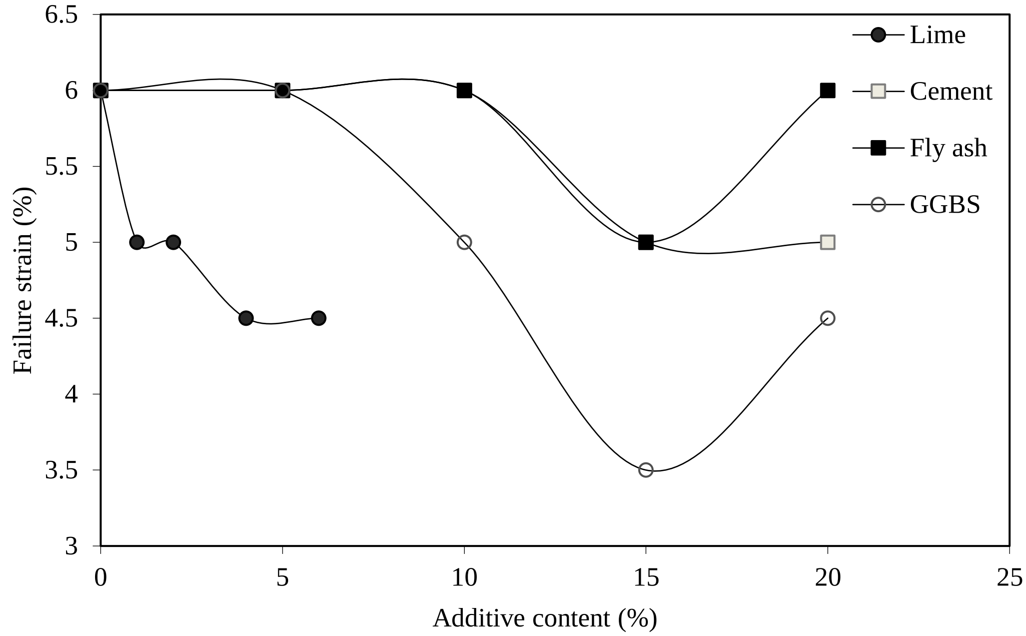


Figure 4.26. Variation of failure strain (14 days curing)

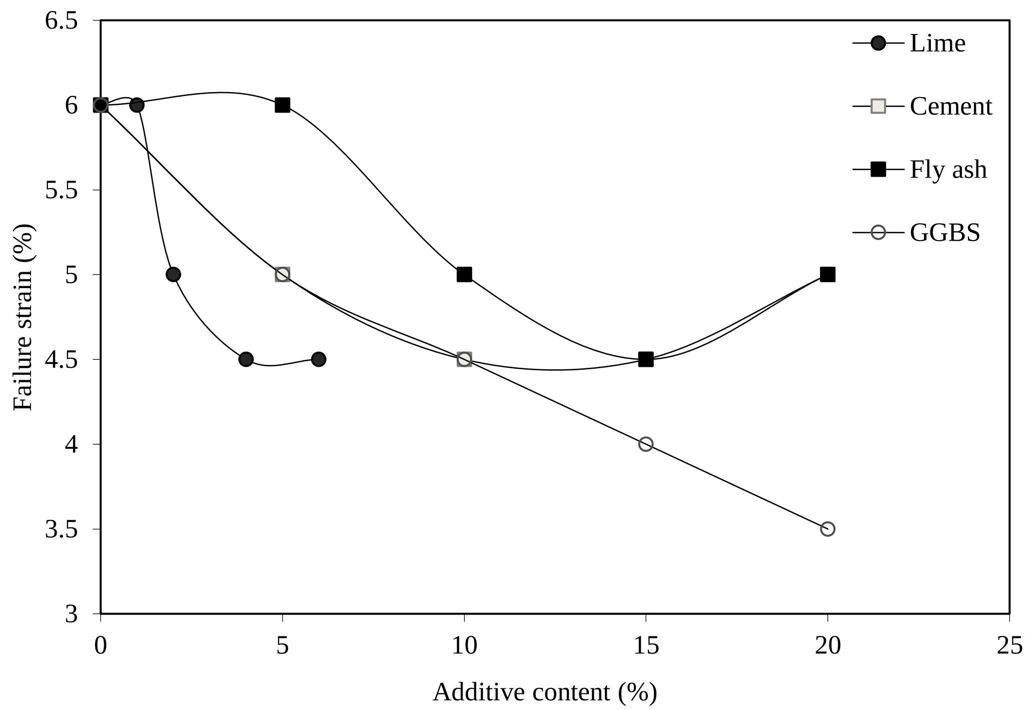


Figure 4.27. Variation of failure strain (28 days curing)

Table 4.4. Effect of chemical additives on UCS and peak strain (%)

	0-days		7-days		14-days		28-days	
	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)	Strain (%)	UCS (kPa)
Expansive clay	7	207	7	228	6	230	6	230
1% lime	7	214	5	260	5	291	6	308
2% lime	6	233	4	321	5	332	5	339
4% lime	4	297	3.5	399	4.5	416	4.5	433
6% lime	4.5	282	3	422	4.5	426	4.5	478
5% cement	6	227	5	287	6	362	5	414
10% cement	5	250	4.5	395	6	534	4.5	804
15% cement	4.5	271	3.5	608	5	820	4.5	1622
20% cement	4.5	302	3.5	910	5	1234	5	1805
5% fly ash	7	211	7	234	6	257	6	274
10% fly ash	6	244	6	250	6	264	5	277
15% fly ash	6	203	5	260	5	270	4.5	299
20% fly ash	7	201	5	287	6	298	5	318
5% GGBS	7	181	6	230	6	291	5	304
10% GGBS	7	214	5	239	5	390	4.5	416
15% GGBS	6	210	5	270	3.5	458	4	532
20% GGBS	5	201	4.5	302	4.5	478	3.5	729

4.3.7. Influence of additives on CBR:

Figure 4.28 depicts the variation of soaked *CBR* (%) with additive content. The data shown pertain to lime, cement, fly ash and GGBS. In general, CBR increased with increasing additive content irrespective of the type of the additive. *CBR* increased from 1.60% to 39.51% and 45.55% respectively when the lime and cement content increased to 6% and 20%. Strong cementitious products developed in the 6% lime and 20% cement specimens were the cause for this significant increase of *CBR*. And *CBR* increased from 1.606% to 4.38% and 14.23% respectively when the fly ash and GGBS content increased to 20%. Though the increase in *CBR* is attributable to the development of cementitious products in these blends, the degree of improvement in *CBR* suggests that these products are weaker than those that developed in the case of lime and cement. Table 4.5 shows the *CBR* data.

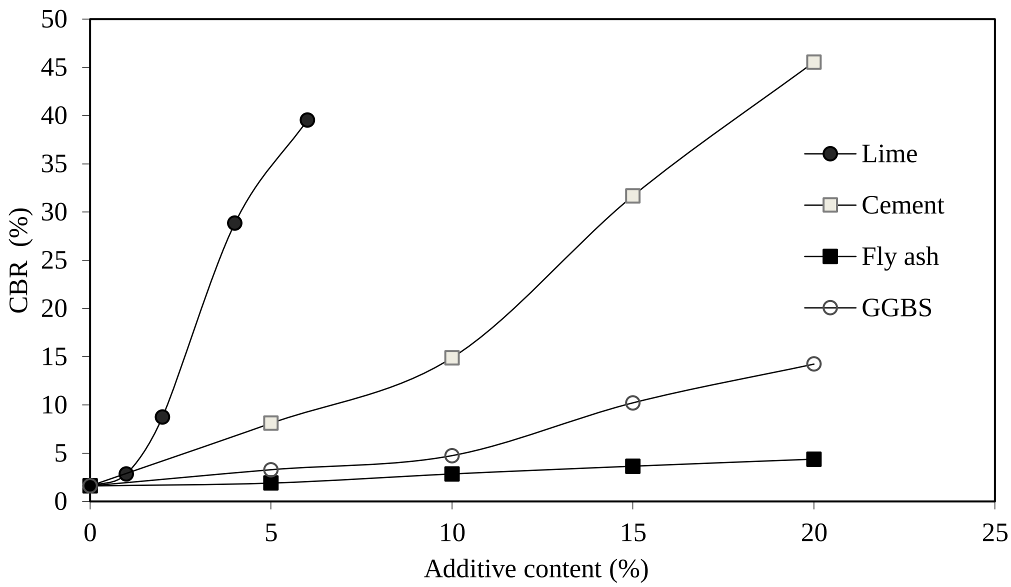


Figure 4.28. Influence of additives on CBR

Table 4.5. Effect of the additives on CBR

	CBR (%)
Expansive clay	1.61
1% lime	2.85
2% lime	8.76
4% lime	28.83
6% lime	39.51
5% cement	8.10
10% cement	14.89
15% cement	31.68
20% cement	45.55
5% fly ash	1.89
10% fly ash	2.85
15% fly ash	3.65
20% fly ash	4.38
5% GGBS	3.28
10% GGBS	4.74
15% GGBS	10.22
20% GGBS	14.23

4.3.8. FESEM analysis:

FESEM analysis of the chemically treated samples and the unblended sample (parent clay) is shown in Figure 4.29 to 4.33. The analysis was done for studying the microstructural behaviour of the parent clay sample and the clay samples treated with 4% and 6% of lime, and 10% and 20% of cement, fly ash and GGBS. The FESEM analysis of the chemically treated samples indicates ettringite formation in all the additive-clay blends. The formation of ettringite was more pronounced in the lime-clay and cement-clay blends and less pronounced in the fly ash-clay and GGBS-clay blends. In the lime-clay and cement-clay blends rich in montmorillonite mineral, the formation of ettringite starts in the early stages of curing itself. The amount and the type of sulphates present is also a factor influencing the quantity and crystal morphology of the ettringite formed. Further, the pore sizes in the FESEM images vary with an increase in the percentages of lime, cement, fly ash and GGBS and an increase in curing time. These observations are in agreement with the previous research (Mitchell and Darmatas, 1992; Ural, 2015; Consoli et al. 2016). These changes in the blends cause respective variations in their behaviour with reference to porosity and strength and volumetric changes such as compressibility, swelling and shrinkage.

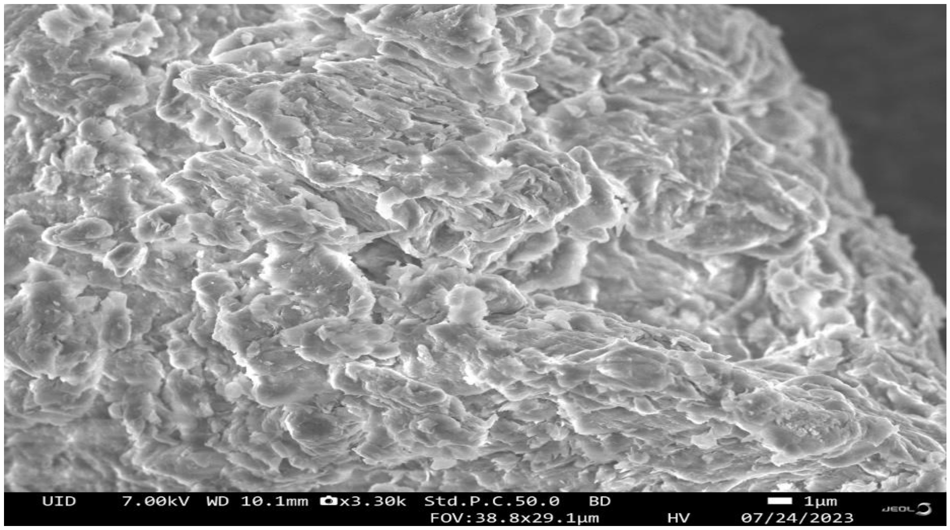
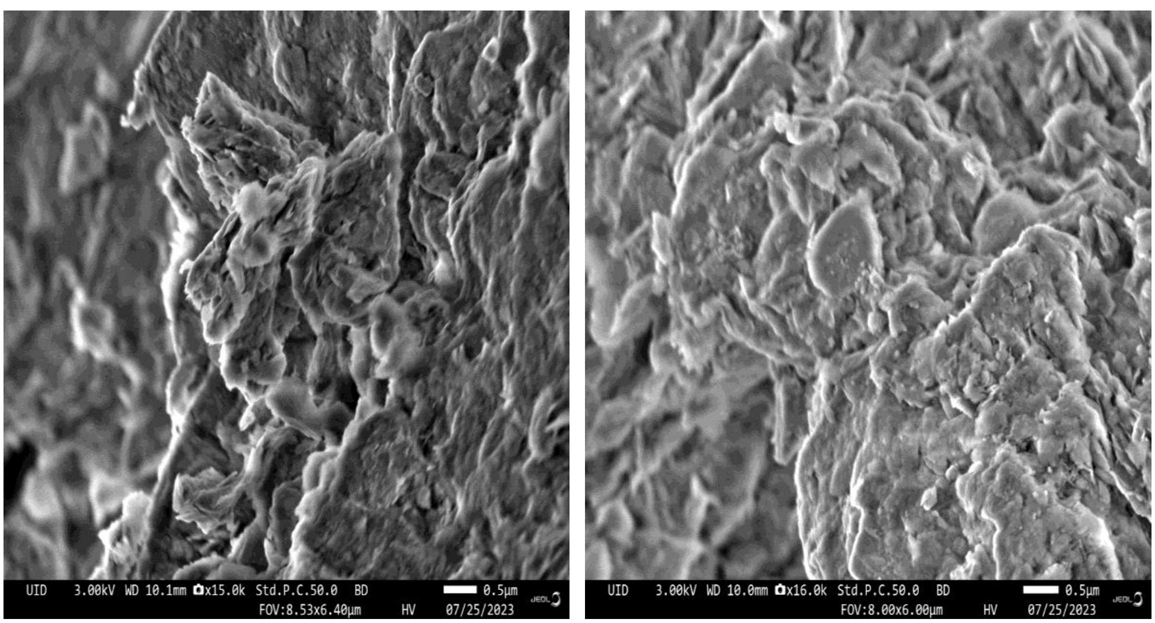
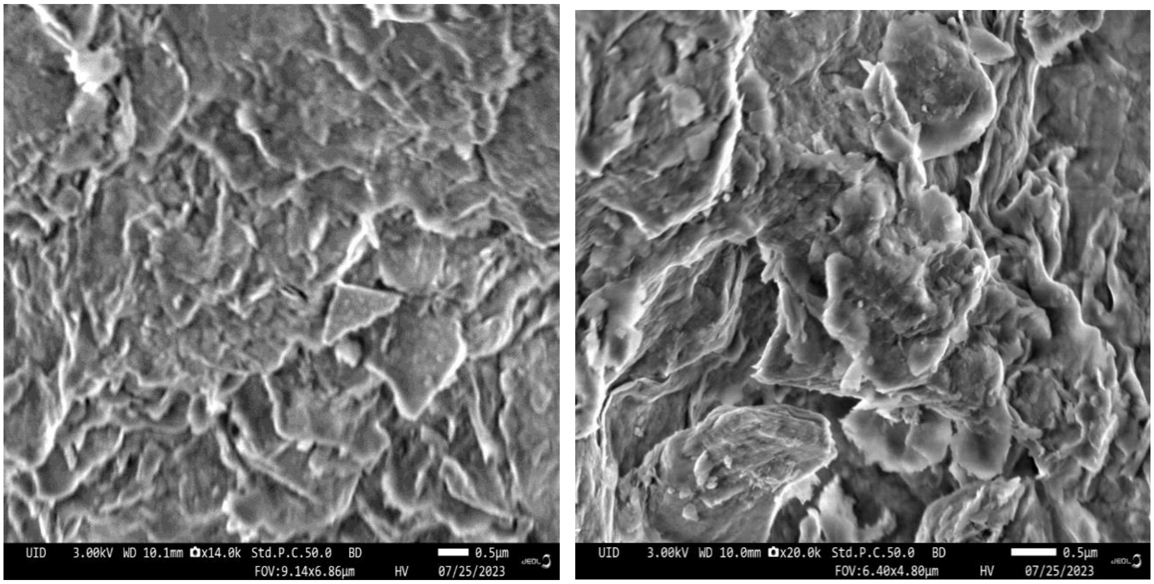


Figure 4.29. FESEM picture of parent clay



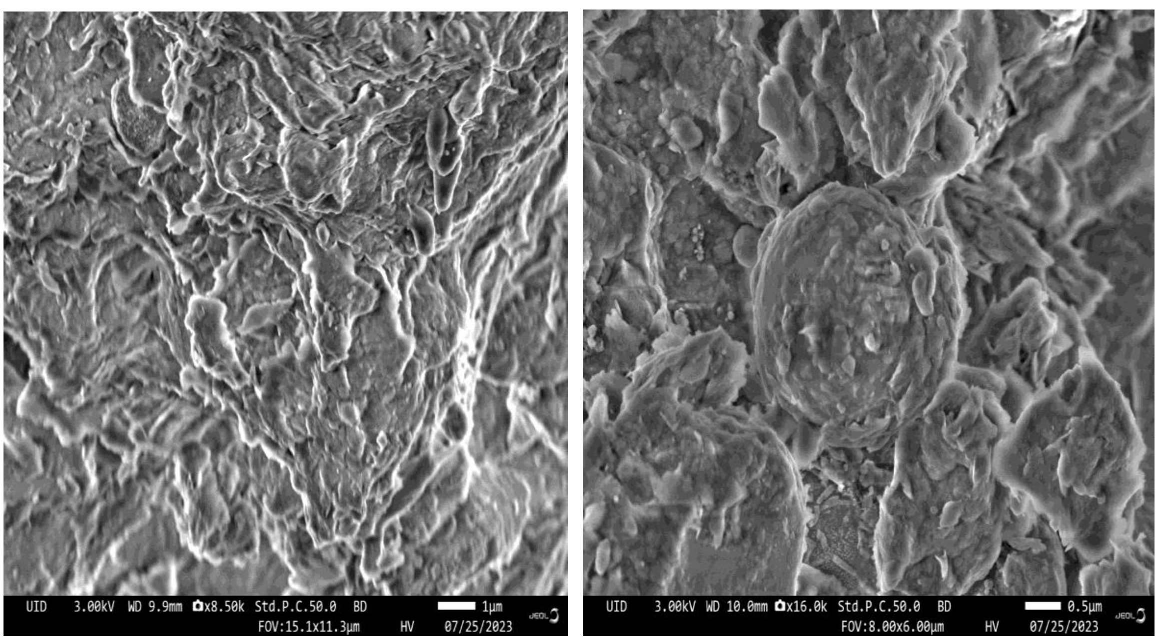
(a) (b)

Figure 4.30. FESEM pictures of (a) 4% lime treated clay (b) 6% lime treated clay



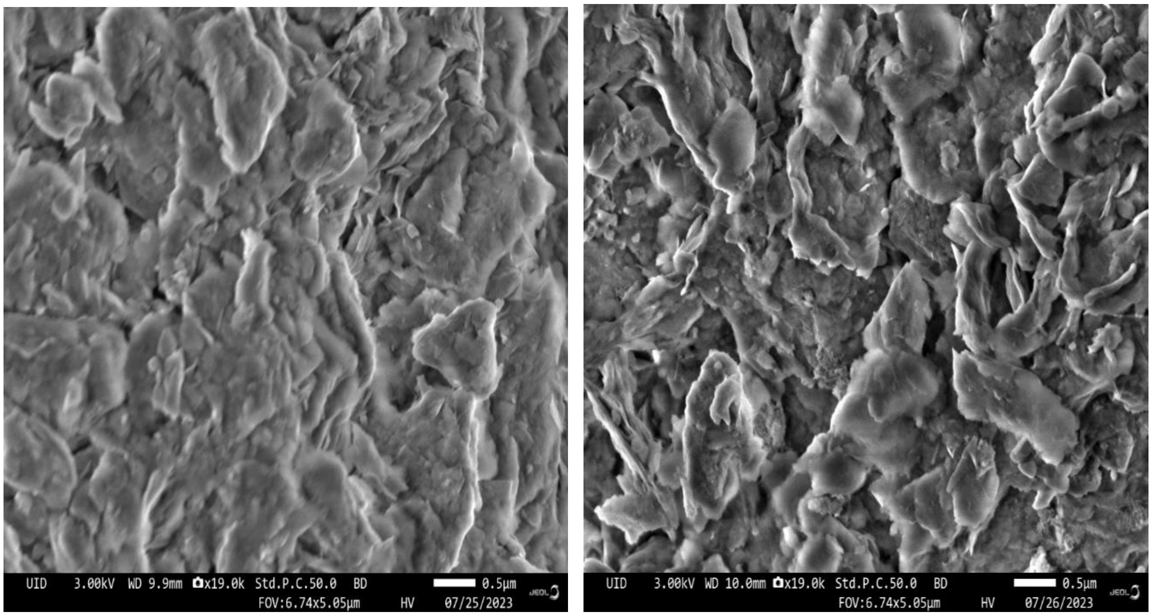
(a) (b)

Figure 4.31. FESEM images of (a) 10% cement treated clay (b) 20% cement treated clay



(a) (b)

Figure 4.32. FESEM images of (a) 10% fly ash treated clay (b) 20% fly ash treated clay



(a) (b)

Figure 4.33. FESEM images of (a) 10% GGBS treated clay (b) 20% GGBS treated clay

4.4. Conclusions:

The following conclusions can be drawn from the fore-going experimental investigation:

1. Addition of chemical additives such as lime, cement, fly ash and GGBS reduced *LL* and *PI* of the expansive clay, thus reducing the plasticity characteristics of the clay. At 20% additive content, cement and GGBS resulted in the highest reduction in *LL* and *PI* among all the additives used in this investigation. But, lime was observed to be the most effective pozzolanic chemical additive considering the fact that an appreciable reduction in *LL* and *PI* occurred at 4% lime itself. Hence, these chemical additives can be advantageously used for stabilising expansive clays, for reducing their plasticity and for increasing their workability.
2. *FSI* of the clay also decreased significantly upon the addition of lime, cement, fly ash and GGBS. Excepting in the case of GGBS, *FSI* decreased, more or less, by the same amount at the highest additive content irrespective of the type of the additive, which again indicated that lime can be thought of as the most effective pozzolanic chemical additive.

3. The swell potential and the swelling pressure of the unblended expansive clay, determined at a water content of 0% and at a dry unit weight of $12kN/m^3$, were respectively 8.5% and 220kPa.
4. The *MDD* of the clay-additive blends increased and the OMC of the blends decreased with increasing additive content, indicating that the chemical additives were instrumental in stabilising the expansive clay.
5. The hydraulic conductivity (*k*, cm/sec) of the expansive clay-additive blends, determined at their respective *OMC* and *MDD* by performing the variable head permeameter tests, continuously decreased with increasing additive content excepting in the case of lime. The hydraulic conductivity decreased with increasing additive content as the *MDD* of the clay-additive blends increased with increasing additive content. However, in the case of lime-clay blends, the hydraulic conductivity decreased up to 2% lime and thereafter, it increased. This could be attributed to the possible heat release at higher lime contents and a consequent absorption of more water.
6. For a given curing period, peak stress or failure stress increased and peak strain or failure strain decreased with increasing additive content. Similarly, at a given additive content, peak stress or failure stress increased and peak strain or failure strain decreased with increasing curing period.
7. The soaked *CBR* of the clay-additive blends, determined at their respective *OMC* and *MDD*, increased significantly with increasing additive content. The highest *CBR* values were observed in the cases of lime-clay blends and cement-clay blends though the percentage increase in the *CBR* of the fly ash-clay blends and the GGBS-clay blends was also quite high. This suggested that the chemical additives studied in this investigation proved quite effective in stabilising the expansive clay. Therefore, they can be used in strengthening the expansive clay subgrades.

CHAPTER - 5

Heave and compressive load response

5.1. Introduction:

This chapter discusses swelling or heave behaviour and compressive load response and the effect of the additives on these aspects. The compressive load response was assessed through performance of plate load tests on chemically stabilised clay beds. As already mentioned, the load response was determined at as-compacted condition and at saturated condition of the clay beds. During saturation, swelling and rate of swelling were studied.

5.2. Experimental Investigation:

5.2.1. Test Materials:

The test materials, namely, a highly swelling expansive soil, and the previously mentioned additives have been already discussed in detail in Sections 3.3 and 4.2.1.

5.2.2. Tests conducted:

Plate load tests were conducted in two different series as described in detail in Section 3.6.7.

5.3. Discussion of test results:

5.3.1. Influence of lime, cement, fly ash and GGBS on compaction characteristics:

The effect of the above additives on compaction behaviour of the clay was discussed in detail in Section 4.3.4 of Chapter-4. The influence of these additives on compaction was depicted in Figures 4.7 to 4.10. Upon the addition of these chemicals, the expansive clay got stabilised and showed higher γ_d values for given water contents compared to the plain clay. Hence, *MDD* increased and *OMC* decreased in general.

The compaction data are summarised in Table 4.2.

5.3.2. Compressive load response in as-compacted condition:

Figure 5.1 shows the load response through stress-settlement curves for the plain clay bed and for the clay bed blended with various amounts of lime. The data show that the stress-settlement behaviour of the clay bed improved upon addition of lime. This meant that the stress to be applied for a given settlement increased with increasing lime content. This is because the lime-treated clay beds offered increased resistance to the applied compressive loads, which can be attributed to the increased dry density of the clay bed at increased lime contents. However, this was noted up to the optimum lime content of 4%. Thereafter, a decrease was observed as the dry density decreased at a lime content of 6%. For example, for a settlement of 0.5mm of the clay bed, the stress required to be applied was respectively 28kPa, 30kPa, 32kPa, 80kPa and 48kPa respectively for the plain clay bed and the clay bed stabilised with 1% lime, 2% lime, 4% lime and 6% lime. The increase in the stress for a settlement of 0.5mm was 186% when the lime content increased from 0% to 4%. This shows that, when lime increased, the stiffness of the clay bed increased.

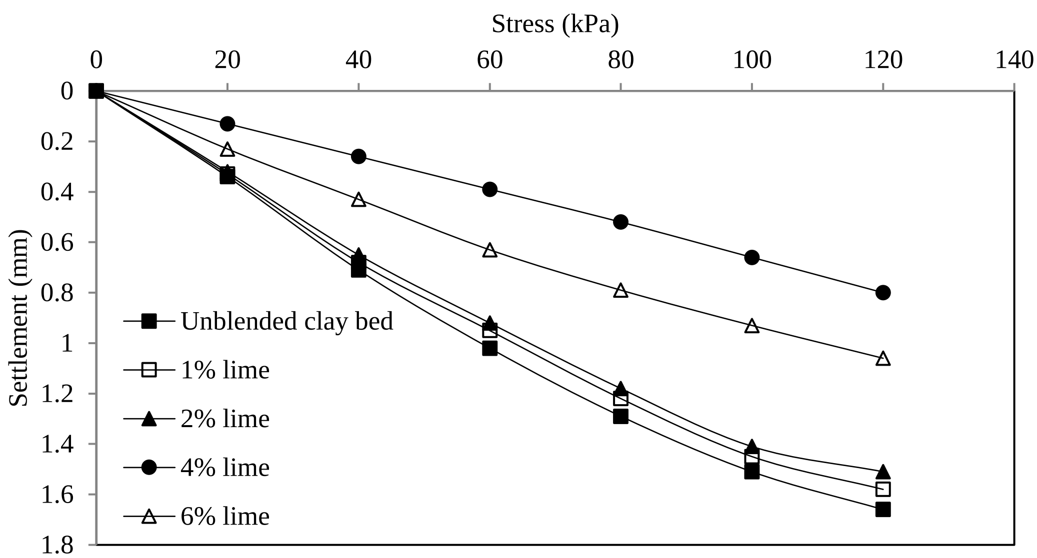


Figure 5.1. Stress-settlement behaviour of plain and lime-treated clay in as-compacted condition

Similar data were obtained from the plate load tests conducted on the clay beds blended with varied amounts of cement (5%, 10%, 15% and 20%). Fig 5.2 shows the load response for the plain clay and for the clay blended with increased amounts of cement. The load response improved when the clay was blended with cement. The stress required to be applied for a given settlement increased with the addition of cement. For example, for 0.5mm settlement, the stress to be applied was respectively 28kPa, 34kPa, 40kPa, 72kPa and 84kPa for the plain clay bed and the clay bed stabilised with 5% cement, 10% cement, 15% cement and 20% cement. The cement-blended clay beds offered higher resistance to the applied compressive stresses. The stress required to be applied for a settlement of 0.5mm increased by 200% when the cement content increased from 0% to 20%, indicating increased stiffness.

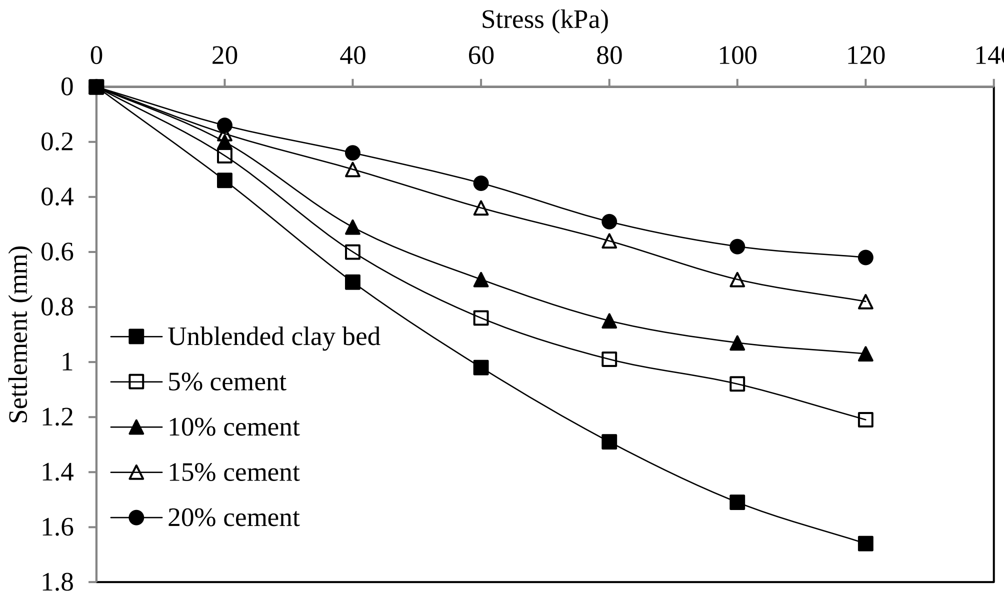


Figure 5.2. Stress-settlement behaviour of untreated and cement-treated expansive clay in as-compacted condition

Figures 5.3 and 5.4 respectively show the load response for the unblended clay bed and the clay bed blended with varied amounts of fly ash and GGBS. The fly ash-blended and the GGBS-blended clay beds also offered higher resistance to the compressive loads. The stress for a given settlement increased with the addition of fly ash and GGBS. This behaviour was observed only up to 10% fly ash and 10% GGBS. At higher additive contents, the stress for a given settlement decreased. For instance, the stress for 0.5mm settlement increased from 28kPa to 60kPa and 38kPa respectively when fly ash and GGBS increased to 10%, indicating an increase of 114% and 36%.

Figure 5.5 summarises the data on the stresses for a settlement of 0.5mm for the clay beds stabilised with lime, cement, fly ash and GGBS. The stress increased continuously with increasing cement content. However, in the cases of lime, fly ash and GGBS, the stress increased respectively up to 4%, 10% and 10%, and thereafter, it decreased. The best compressive load response was observed at 4% lime and 20% cement. In the cases of fly ash and GGBS, 10% was found to be the optimum content with reference to stress-settlement response also. Table 5.1 shows the stress for a settlement of 0.5mm for different cases in as-compacted condition.

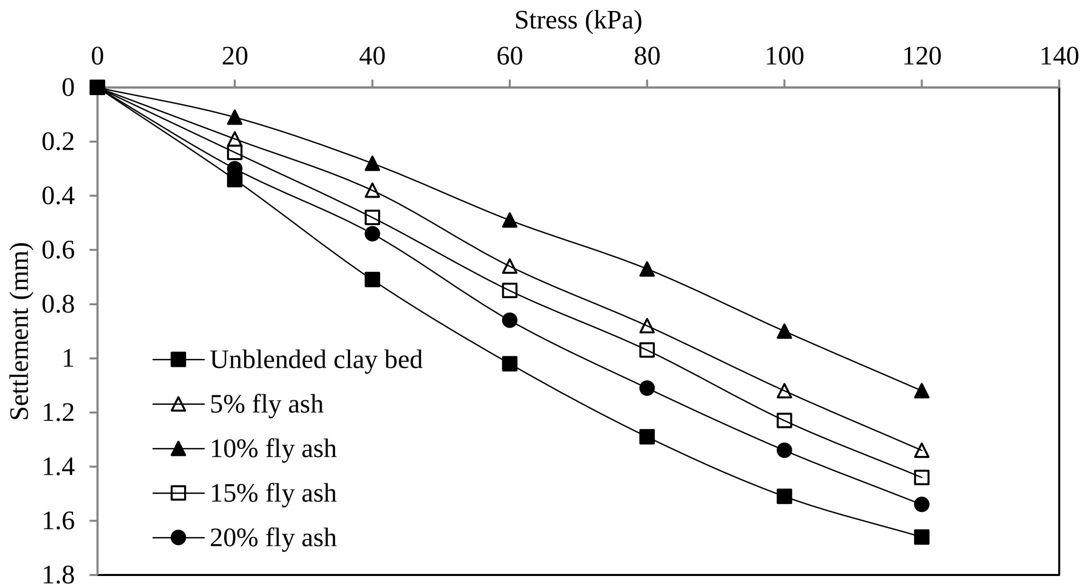


Figure 5.3. Stress-settlement behaviour of plain and fly ash-treated clay in as-compacted condition

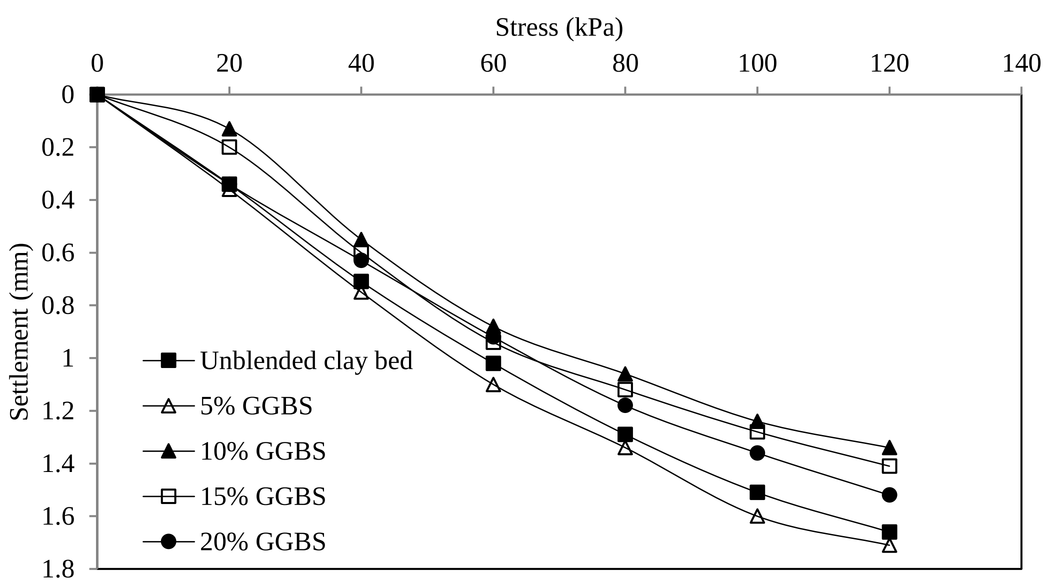


Figure 5.4. Stress-settlement behaviour of untreated and GGBS-treated expansive clay in as-compacted condition

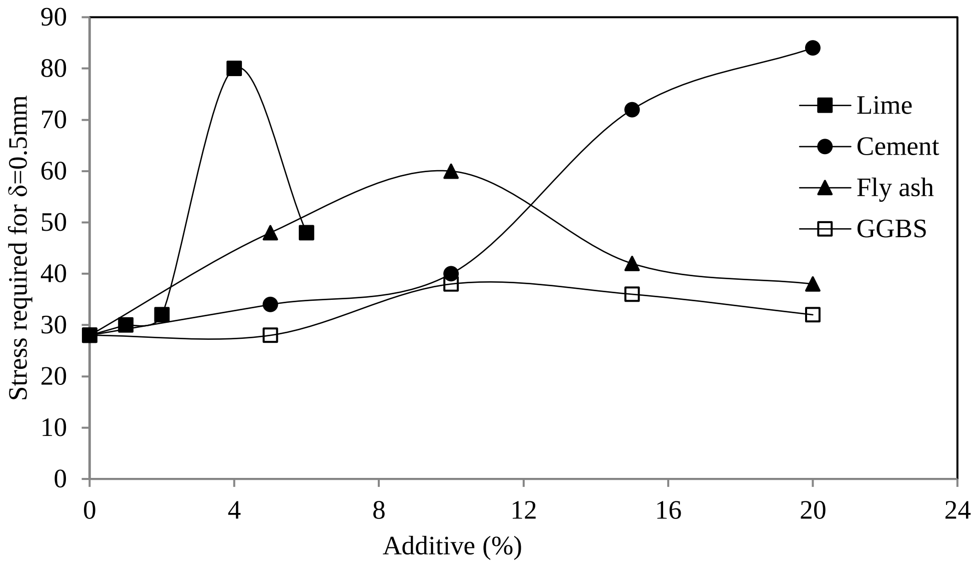


Figure 5.5. Influence of additive content on stress corresponding to 0.5 mm settlement in as-compacted condition

Table 5.1. Stress values for 0.5mm settlement in as-compacted and saturated conditions

	Stress (kPa) in as-compacted condition	Stress (kPa) in saturated condition
Expansive clay	28	6
1% lime	30	10
2% lime	32	12
4% lime	80	25
6% lime	48	-
5% cement	34	30
10% cement	40	-
15% cement	72	-
20% cement	84	-
5% fly ash	48	10
10% fly ash	60	12
15% fly ash	42	14
20% fly ash	38	16
5% GGBS	28	18
10% GGBS	38	20
15% GGBS	36	26
20% GGBS	32	28

5.3.3. Rate and amount of heave:

As already mentioned, the plain expansive clay bed and the clay beds stabilised with varied amounts of lime, cement, fly ash and GGBS were subjected to continuous wetting for 10 days and allowed to undergo free swell or heave. Heave was monitored corresponding to different time periods till the equilibrium heave. This section discusses the rates and amounts of the heave of the clay beds.

Figure 5.6 shows, by comparison, the rate of heave of the unblended expansive clay bed and the clay bed blended with varied amounts of lime. The heave profile clearly indicates three stages in the development of heave. These three stages are initial heave, primary heave and secondary heave. The initial stage occurs at the macrostructural level and is normally associated with relatively minor swelling strains or heave strains. The stage of primary heave takes place at the microstructural level and progresses along with larger swelling strains indicated by the steep-sloped linear portion of the heave profile. And the stage of secondary heave or secondary swelling which also occurs at the microstructural level is again associated with small strains. The asymptotic portion of the heave profile confirms equilibrium heave. All these stages of development of heave can be observed in Figures 5.6 to 5.9 showing the heave profiles of different clay beds.

The unblended expansive clay bed showed significant increase in heave with increasing wetting period. By its asymptoticity with the X-axis, the heave profile shows that the clay bed attained equilibrium heave by the end of 10 days. Equilibrium heave attained by the unblended clay bed was 36mm. However, upon getting blended with lime, the rate and the amount of heave significantly decreased. The reduction in the rate and the amount of heave became more prominent as the lime content in the clay bed increased. This could be due to lime stabilisation. The amount of equilibrium heave decreased to 8.05mm, 6.38mm, 3.28mm and 0.6mm respectively when lime increased to 1, 2, 4 and 6%. Heave decreased by 98% at 6% lime.

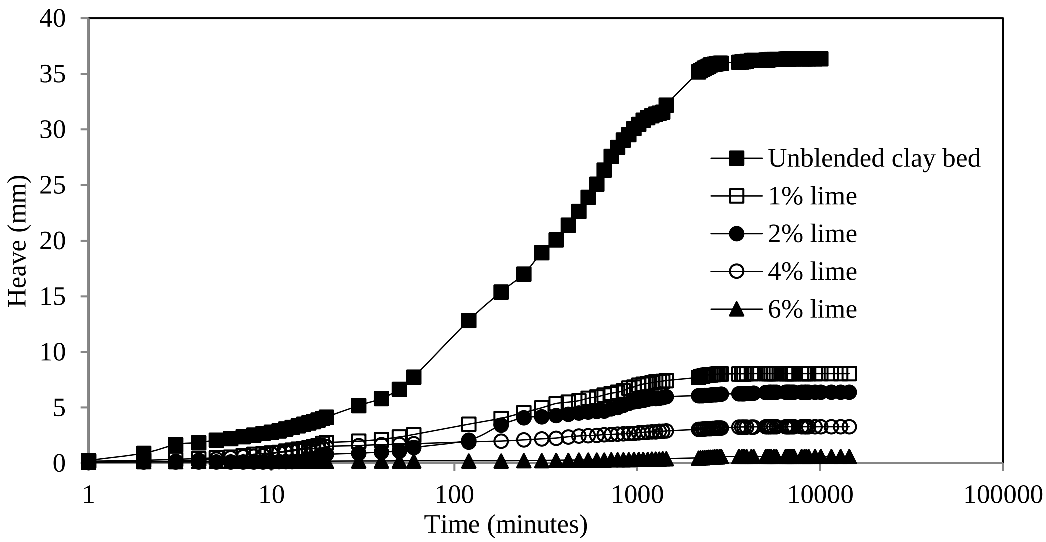


Figure 5.6. Heave profiles

Figure 5.7 shows, by comparison, the profiles of rate of heave of the unblended clay bed and the clay bed stabilised with cement contents of 5%, 10%, 15% and 20%. Cement stabilisation also significantly reduced the rate of heave and the amount of heave. This was more pronounced at higher cement contents. The equilibrium heave decreased from 36mm to 4.15mm, 2.3mm, 1.75mm and 0.25mm when the cement content increased to 0, 5, 10, 15 and 20% respectively. Heave decreased by 99% at 20% cement content.

Figures 5.8 and 5.9 respectively show the heave profiles of the fly ash-blended and the GGBS-blended expansive clay beds in comparison with that of the unblended clay bed. The rate and the amount of heave decreased because of stabilisation with fly ash and GGBS also. The equilibrium heave decreased by 53% and 74% respectively at 20% fly ash content and 20% GGBS content. Figure 5.10 summarises the data on heave of the chemically stabilised clay beds.

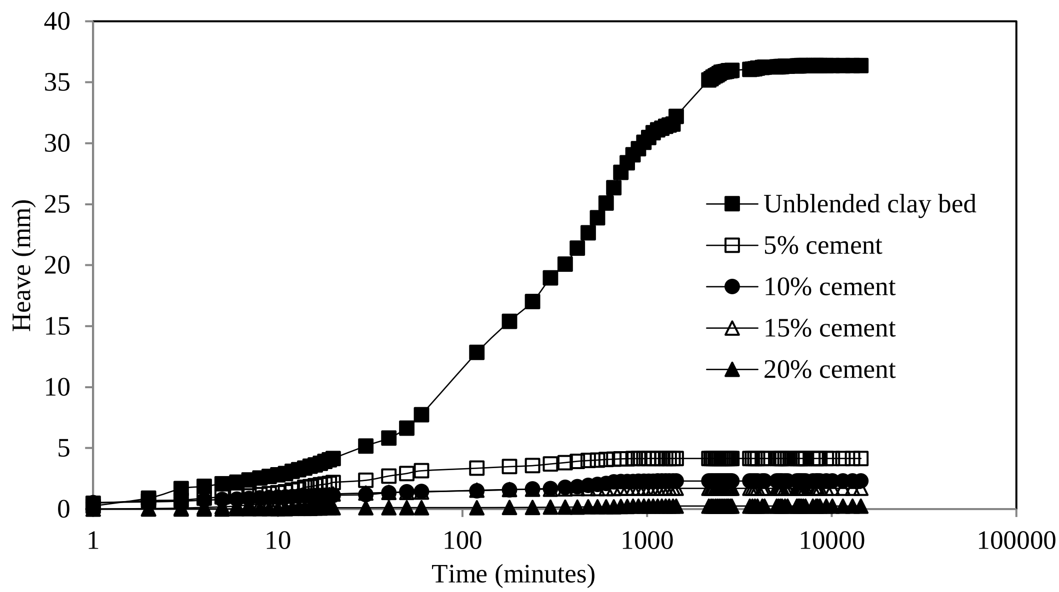


Figure 5.7. Heave profiles

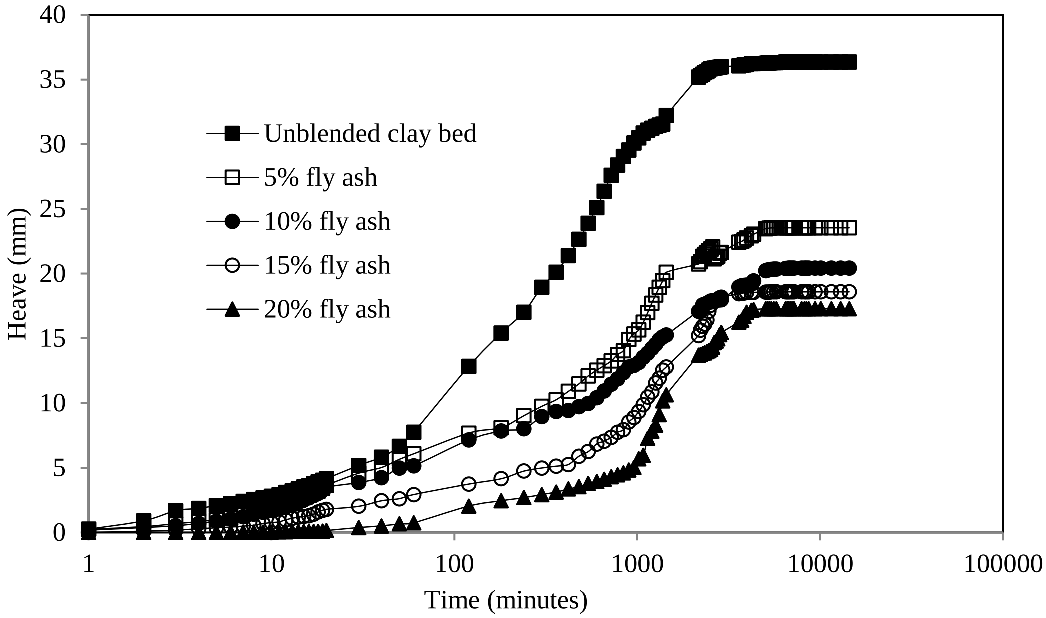


Figure 5.8. Heave profiles

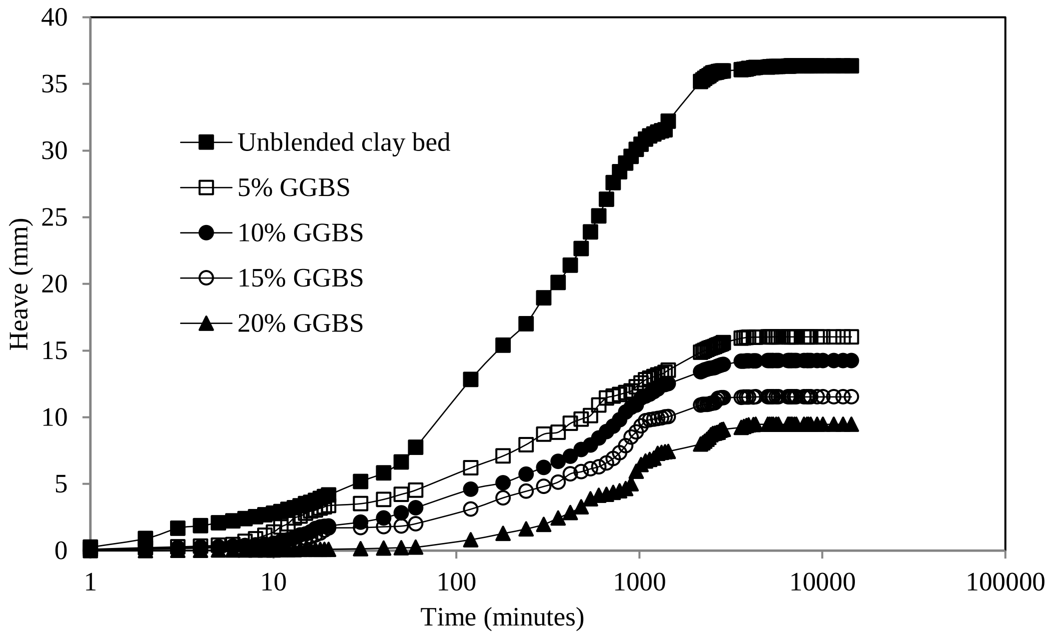


Figure 5.9. Heave profiles

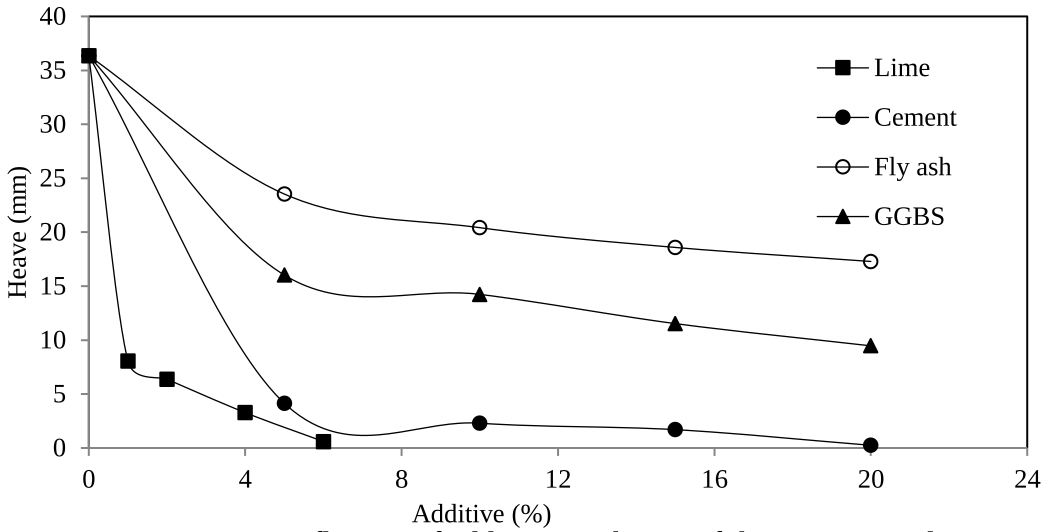


Figure 5.10. Influence of additives on heave of the expansive clay

5.3.4. Load response in saturated condition:

As mentioned in the previous sections, plate load tests were conducted on the clay beds after the attainment of the equilibrium heave in a saturated or near saturated condition. Figure 5.11 depicts the load response for the plain clay bed and the clay bed stabilised with varied amounts of lime. The load response of the clay beds in the saturated condition depends upon their heave response or the amounts of heave they undergo. The clay bed which undergoes the least amount of heave offers the highest resistance to the applied compressive loads and vice versa. Hence, the compressive load response in the saturated condition improved with increasing lime content as heave decreased. In other words, the stress required to be applied on the clay bed for a given settlement increased with increasing lime content. For example, the stress for a settlement of 0.5mm was respectively 6kPa, 10kPa, 12kPa and 25kPa for the lime contents of 0% (unblended clay bed), 1%, 2% and 4%. The clay bed stabilised with 6% lime underwent the highest settlement only of 0.25mm at the applied stress of 40kPa. As the clay beds were saturated, the compressive stress response became much poorer than that in the as-compacted condition. Or, the stiffness of the clay beds decreased drastically compared to the as-compacted condition. As the clay beds swell, they lose their intactness.

A similar compressive stress response in the saturated condition was evinced by the clay beds as shown in Figures 5.12, 5.13 and 5.14. While the clay beds stabilised by fly ash and GGBS settled more, those stabilised by cement settled less. This was because the cement-stabilised clay beds attained lesser heave. Hence, irrespective of the type of the chemical additive used for stabilising the clay beds, compressive load response in saturated condition was much poorer than that in the as-compacted condition.

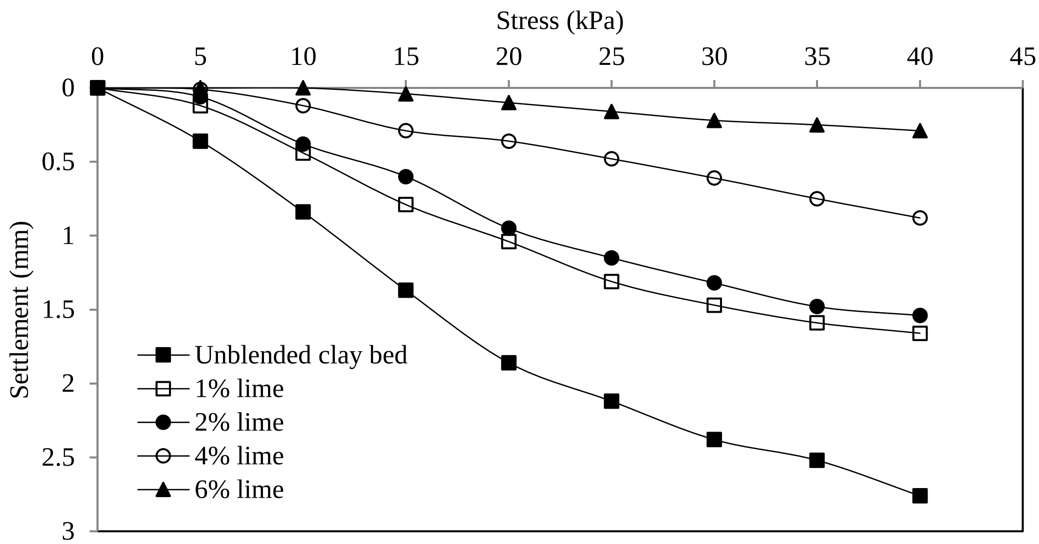


Figure 5.11. Stress-settlement behaviour of plain and lime-blended clay in saturated condition

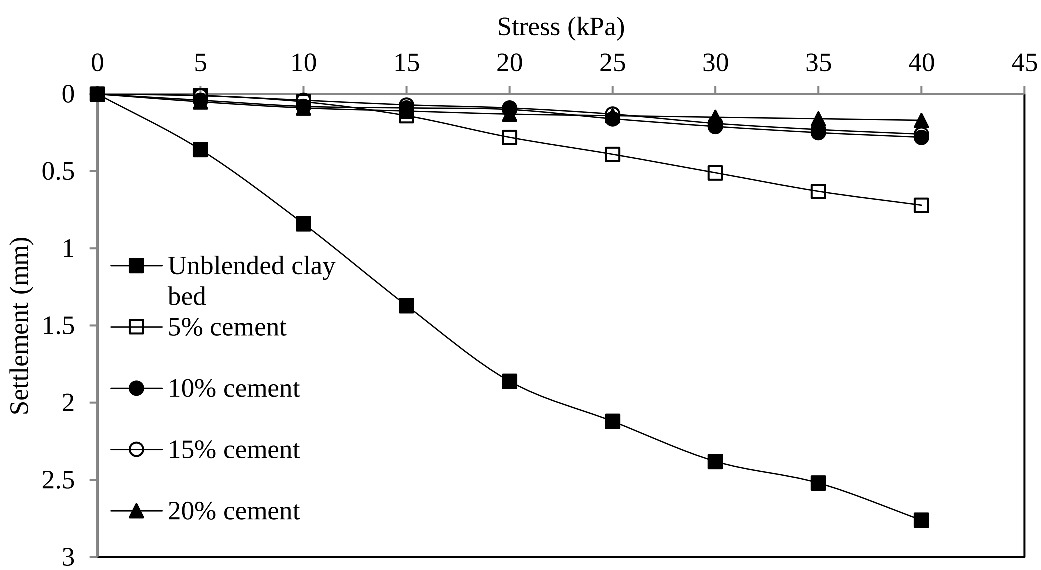


Figure 5.12. Stress-settlement behaviour of plain and cement-blended clay in saturated condition

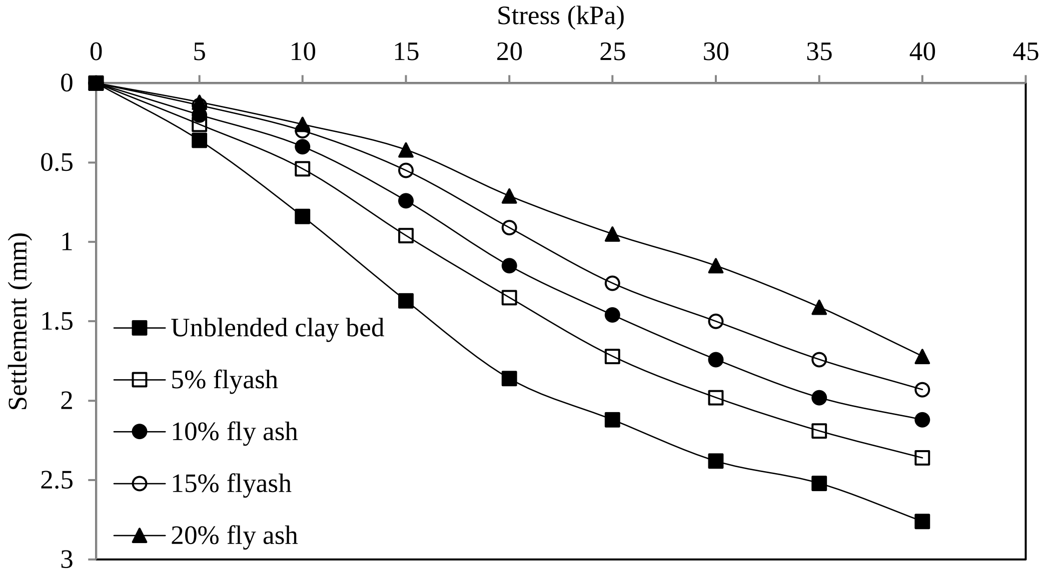


Figure 5.13. Stress-settlement behaviour of plain and fly ash-blended clay in saturated condition

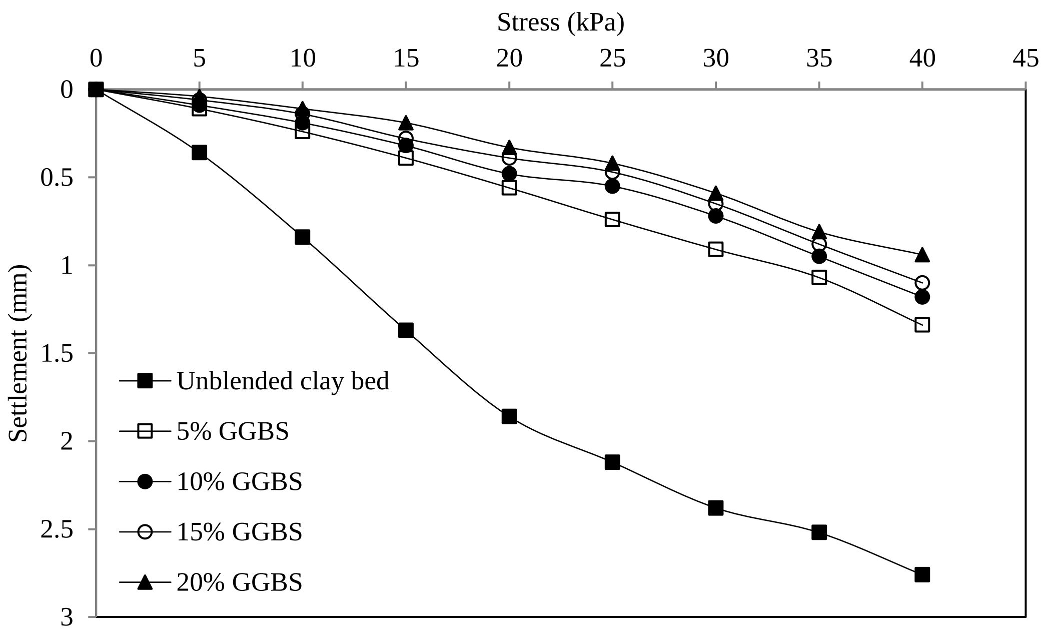


Figure 5.14. Stress-settlement behaviour of plain and GGBS-blended clay in saturated condition

Figure 5.15 summarises the stress for the settlement of 0.5mm in saturated condition. Table 5.1 shows the stress for a settlement of 0.5mm for different cases in saturated condition.

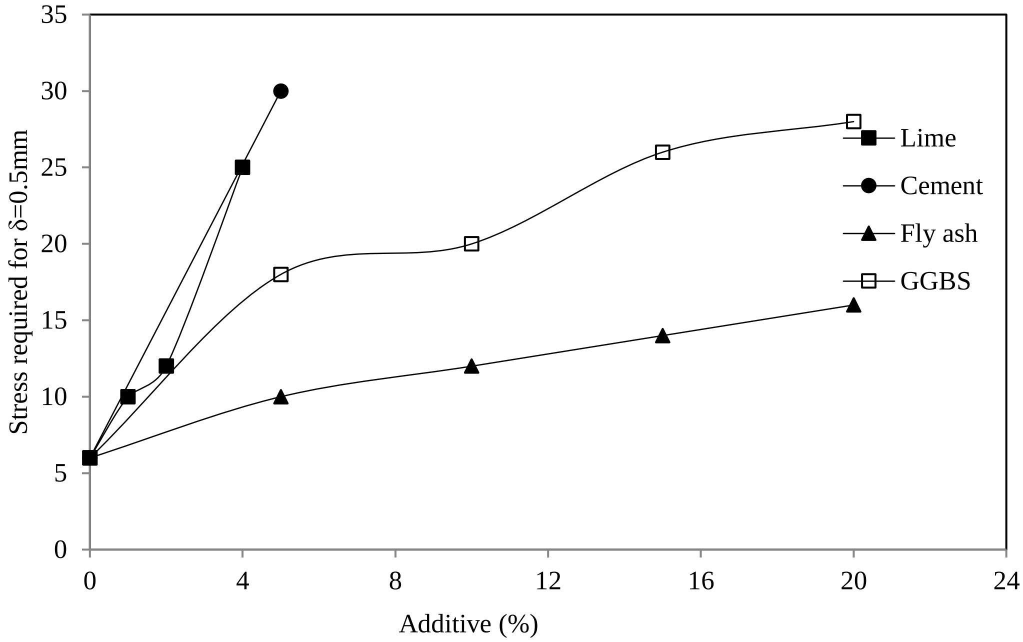


Figure 5.15. Effect of additives on stress corresponding to 0.5mm settlement in saturated condition

5.3.5. Degrees of saturation of the clay beds:

As already mentioned in the section on experimental investigation, soil samples for the determination of water content (w_{sat}) were collected from various depths (0mm, 100mm and 200mm) of the clay beds at the end of inundation or upon the attainment of equilibrium heave. From the saturated water contents measured, the degrees of saturation (S_r) of the clay beds stabilised by various amounts of lime, cement, fly ash and GGBS were determined from the final void ratio (e_f) using the following equation:

$$e_f S_r = w_{sat} G \tag{5.1}$$

where e_f is the final void ratio of the clay bed upon its attaining the equilibrium heave at the end of the inundation period, w_{sat} is the water content of the soil samples at the end of the inundation period and G is the specific gravity of the soil solids.

In one-dimensional swell-consolidation, the change in void ratio (Δe) is related to the change in the thickness of the clay samples or the clay bed. It is expressed as

$\Delta e/(1+e_0) = \Delta H/H$ (5.2)

where e_0 is the initial void ratio of the clay bed, Δe is the change in void ratio (upon swelling or consolidation), H is the initial thickness and ΔH is the change in the thickness (upon swelling or consolidation).

ΔH, or increase in the thickness upon swelling, is assessed through the dial gauge reading at the end of the inundation period, and e_o is determined from the expression

$e_0 = G\gamma_w/\gamma_d - 1$ (5.3)

Hence Δe can be calculated from the equation 5.2. The final void ratio (e_f) of the clay bed is then equal to

$e_f = (e_0 + \Delta e)$ (5.4)

From e_f, S_r can be calculated using the equation (5.1).

The degrees of saturation (S_r) at different depths of the clay beds (shown in Table 5.2) show that all the beds were nearly saturated. Figure 5.16 depicts the water content profiles of the clay beds. Only a few cases were presented in Figure 5.16 to avoid cluster of the data. Water contents and degrees of saturation of the clay beds at different depths are shown in Table 5.2. They indicate a near saturation of the clay beds through proper penetration of water content to various depths of the clay beds.

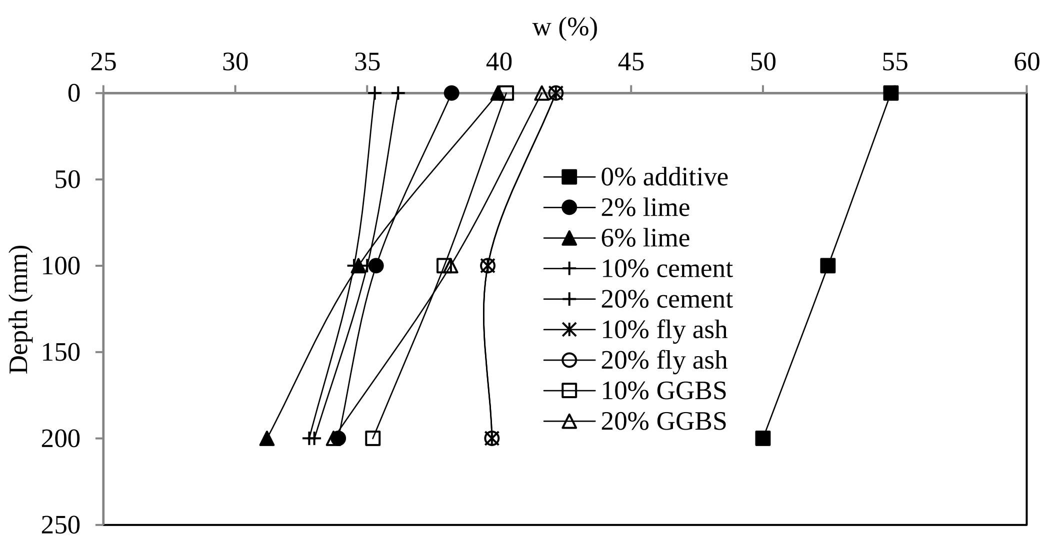


Figure 5.16. Variation of water content with depth for untreated and chemically treated expansive clay

Table 5.2. Degrees of saturation (S_r) and water contents and at different heights

	Expansive clay	2% lime	6% lime	10% cement	20% cement	10% fly ash	20% fly ash	10% GGBS	20% GGBS
Water content (0mm)	54.85	38.2	39.96	36.17	35.29	42.15	41.5	40.27	41.62
Water content (100mm)	52.47	35.34	34.67	35	34.5	39.57	37.45	37.92	38.16
Water content (200mm)	50	33.9	31.2	33	32.8	39.73	39.25	35.21	33.72
S_r (0mm)	100	100	100	100	100	100	97.16	100	100
S_r (100mm)	100	98.82	100	100	100	95.29	87.68	94.27	97.11
S_r (200mm)	100	94.79	100	100	100	95.68	91.89	87.54	85.82

5.3.6. Young's modulus (E_s) of the clay beds:

To assess the stiffness of the clay beds stabilised by varied amounts of lime, cement, fly ash and GGBS, their Young's modulus (E_s) was determined considering the plate load test data. The following equation for the elastic settlement (S_e) was used for computing (E_s):

$$S_e = Bq_0 (1 - \mu_s^2)/E_s \tag{5.5}$$

A settlement value (S_e) of 0.5mm was chosen for the calculation of E_s values. The stress value (q_0) corresponding to S_e of 0.5mm was read from the stress-settlement curves and tabulated in Table 5.3. The Poisson's ratio of the soil (μ_s) was assumed to be 0.35 for the unsaturated, as-compacted clay beds and 0.5 for the saturated clay beds. The diameter of the footing plate (B) was 60mm or 0.06m as already mentioned. Using these data, E_s of all the clay beds was calculated and shown in Table 5.3.

Figure 5.17 depicts the variation of the Young's modulus (E_s) with the additive content for lime, cement, fly ash and GGBS. The data shown pertain to the load tests performed in as-compacted condition. While E_s increased significantly with increasing cement content, it increased only up to 4% lime (which is the optimum lime content), and thereafter, it decreased at 6% lime. E_s of the fly ash-stabilised clay beds and the GGBS-stabilised clay beds increased only up to 10% additive content, and thereafter, it decreased at higher fly ash contents and GGBS contents. For example, E_s value was found to be 2950kPa, 8424kPa, 8845kPa, 6318kPa and 4001kPa respectively for the unblended clay bed (0% additive) and the clay beds stabilised by 4% lime, 20% cement, 10% fly ash and 10% GGBS. The increase in the E_s indicated that chemical stabilisation of the clay beds increased their stiffness significantly.

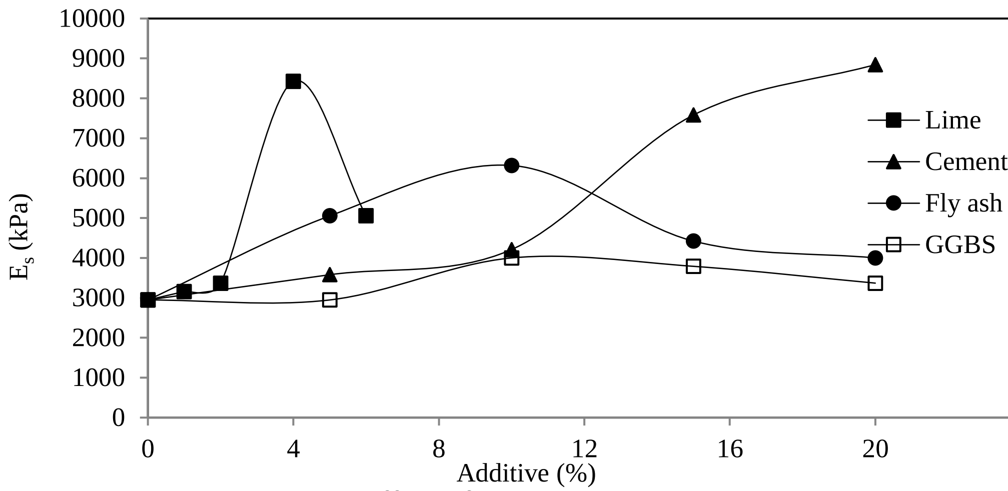


Figure 5.17. Effect of additives on Young's modulus in as-compacted condition

Figure 5.18 depicts the variation of E_s with additive content for lime, cement, fly ash and GGBS. The data pertain to the load tests performed in saturated condition. Though E_s decreased in the saturated condition compared to that in the as-compacted condition, there was considerable increase in the E_s with increasing additive content. E_s was found to be 540kPa, 2250kPa, 2700kPa, 1440kPa and 2520kPa respectively for the unblended clay bed (0% additive) and the clay beds stabilised by 4% lime, 5% cement, 20% fly ash and 20% GGBS.

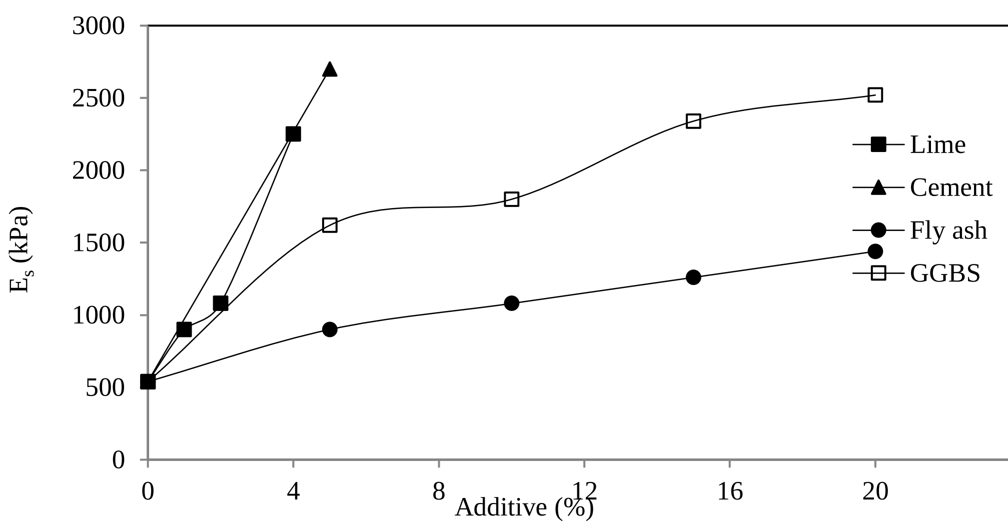


Figure 5.18. Effect of additives on Young's modulus in saturated condition

The settlement of 0.5mm could not be realised at cement contents greater than 5% in saturated condition. Table 5.3 summarises the E_s values in both as-compacted and saturated conditions. The test data indicate that E_s of the clay bed in both the as-compacted and the saturated states increased with the addition of additives though the percentages of the additives at which E_s was the highest varied.

Table 5.3. Young's modulus values for the stresses corresponding to 0.5mm settlement in as-compacted and saturated conditions

	Stress (kPa) corresponding to 0.5mm settlement in as-compacted condition	E_s (kPa) values in as-compacted condition	Stress (kPa) corresponding to 0.5mm settlement in saturated condition	E_s (kPa) values in saturated condition
Expansive clay	28	2950	6	540
1% lime	30	3159	10	900
2% lime	32	3370	12	1080
4% lime	80	8424	25	2250
6% lime	48	5054	-	-
5% cement	34	3580	30	2700
10% cement	40	4212	-	-
15% cement	72	7582	-	-
20% cement	84	8845	-	-
5% fly ash	48	5054	10	900
10% fly ash	60	6318	12	1080
15% fly ash	42	4422	14	1260
20% fly ash	38	4001	16	1440
5% GGBS	28	2950	18	1620
10% GGBS	38	4001	20	1800
15% GGBS	36	3790	26	2340
20% GGBS	32	3370	28	2520

5.4. Conclusions:

The following conclusions can be drawn from the foregoing experimental investigation:

1. Addition of lime, cement, fly ash and GGBS to the expansive clay was observed to improve its compaction characteristics. The MDD of the expansive clay-additive blends increased and their OMC decreased with increasing additive content.
2. The stress-settlement characteristics of the clay beds improved with increasing additive content in the as-compacted condition. The stress for a settlement of 0.5mm increased with the additive content. While in the case of cement this stress continuously increased with cement content, in the cases of lime, fly ash and GGBS, it increased respectively up to 4%, 10% and 10%, and decreased thereafter.
3. The stress for a settlement of 0.5mm in the saturated condition increased with additive content in the cases of fly ash and GGBS. In the case of the lime-clay blends, this stress increased up to 4% lime. At 6% lime, the test bed could not be compressed up to the settlement of 0.5mm. The test beds of the cement-clay blends also could not be compressed up to 0.5mm at cement contents higher than 5%.
4. The Young's modulus (E_s) of the chemically stabilised expansive clay beds, determined in the as-compacted condition, increased with increasing cement content. However, E_s increased only up to 4% lime, 10% fly ash and 10% GGBS respectively in the cases of lime-clay, fly ash-clay and GGBS-clay blends, and decreased thereafter.
5. The Young's modulus (E_s) of the chemically stabilised expansive clay beds, determined in the saturated condition, increased with the additive content for fly ash and GGBS. In the case of the lime-clay blends, E_s increased up to 4% lime. At 6% lime, E_s could not be determined in the saturated condition. E_s of the cement-clay blends also could not be determined at cement contents higher than 5% in the saturated condition.
6. The equilibrium heave of the clay beds decreased with increasing additive contents. Of all the additives, lime and cement had the most prominent effect on heave. Heave decreased respectively by 98% and 99% at 6% lime and 20% cement; and it decreased respectively by 53% and 74% at 20% fly ash and 20% GGBS. The degrees of saturation at different depths of the clay beds obtained from the water content profiles of the beds indicated that all the clay beds were nearly saturated.

CHAPTER - 6

Swell-shrink behaviour

6.1. Introduction:

Expansive soils undergo swelling and shrinkage alternately upon absorption and evaporation of water. Residential buildings and pavements founded in them, therefore, undergo alternate upward and downward movements corresponding to swelling and shrinkage, which leads to detrimental cracking in the buildings. There is a need to study the swell-shrink behaviour of expansive soils consequent upon moisture absorption and moisture evaporation in wet and dry seasons. This chapter presents experimental data on alternate swelling and shrinkage of a chemically stabilised clay bed. Swell-shrink data of the expansive clay bed independently stabilised by lime, cement, fly ash and GGBS were monitored in five alternate wetting and drying cycles. Swelling was monitored by continuously wetting the clay bed for 10 days, and shrinkage was monitored by drying the clay bed (or allowing the water to evaporate from the clay bed) continuously for 50 days. Thus, each cycle of wetting and drying spanned over 60 days. The clay beds were subjected to five such swell-shrink cycles or wetting-drying cycles for monitoring the swell-shrink data. Both swelling (mm) and shrinkage (mm) decreased with increasing number of cycles which was true for all the additives. Polygonal shrinkage as well as linear shrinkage increased with duration of shrinkage. However, they both decreased with increasing number of swell-shrink cycles.

The following sections present the experimental results and discuss them in detail.

6.2. Experimental investigation:

6.2.1. Test materials and tests conducted:

Test materials have been described in detail in Sections 3.3 and 4.2.1

As mentioned above, swell-shrink tests were conducted on the plain clay bed and the clay bed stabilised by 6% lime, 20% cement, 20% fly ash and 20% GGBS and subjected to five alternate swell-shrink cycles. Swelling (mm) and shrinkage (mm) of the clay beds were monitored in these alternate swell-shrink cycles. As *FSI* decreased to the lowest at the above additive contents, these tests were conducted only on the clay bed stabilised independently by these additive contents. The following section describes the swell-shrink tests in detail.

6.2.1.1. Swell-shrink tests:

The plain clay bed and the clay bed stabilised by 6% lime, 20% cement, 20% fly ash and 20% GGBS were subjected to five alternate swell-shrink cycles. Swelling (mm) and shrinkage (mm) of the clay beds were monitored in these alternate swell-shrink cycles.

The expansive soil used in this series of tests was also air-dried and passed through 4.75 mm sieve. The test moulds fabricated for performing these tests were cylindrical moulds having a diameter of 300 mm and a total height of 400 mm. Figure 3.3 depicts the experimental set-up. The height of all the clay beds compacted in these moulds was 200 mm. The plain clay bed and the clay beds stabilised by 6% lime, 20% cement, 20% fly ash and 20% GGBS were compacted at their respective *OMC* and *MDD*. All the clay beds were statically compacted in four layers the thickness of which was carefully monitored to ensure the required density.

After the compaction was over, the clay beds were wetted continuously by adding water from the top. Swelling (mm) of the clay beds was monitored for 10 days with a dial gauge whose spindle was made to rest on a weightless wooden surface plate of dia 100 mm kept centrally on the top of the clay beds. The amount of water required to be added for saturating the clay beds was calculated and added to them from the top over a period of 10days, a time period in which pilot tests showed near saturation. Thus, the swelling phase of each swell-shrink cycle spanned over 10 days. Swelling (mm) of the clay beds was monitored corresponding to suitable time intervals.

After the clay beds attained equilibrium swelling, they were allowed to shrink freely. During shrinkage, the surface wooden piece was removed. Shrinkage was monitored continuously for 50 days. At the end of the 1st cycle, the surface footing plate was placed on the clay bed again for monitoring heave or swelling for the 2nd cycle. And at the end of the 2nd cycle, the procedure was repeated for the remaining cycles. Thus, swelling was observed for ten days and shrinkage for 50 days in every cycle, and all the clay beds were subjected to five swell-shrink cycles.

As the clay beds underwent shrinkage for 50 days in each cycle, shrinkage cracks developed in them in two patterns, namely, polygonal shrinkage pattern and linear shrinkage pattern. The largest width (mm) of these two different types of cracks was measured every ten days of 50 days shrinkage period in each cycle. The following section discusses the results in detail.

6.3. Discussion of test results:

6.3.1. Effect of chemical stabilisation:

Figure 6.1 compares the rate of swelling and shrinkage of the plain clay bed and the clay bed blended with lime, cement, fly ash and GGBS. As already mentioned in the section on experimental investigation, the clay bed was independently stabilised with 6% lime, 20% cement, 20% fly ash and 20% GGBS. Swelling (ΔH, mm) and shrinkage ($-\Delta H$, mm) are plotted on the Y-axis against log time (days) plotted on X-axis. Swelling was monitored for ten days and shrinkage for 50 days as mentioned in the experimental investigation and as shown in the Figure 3.3. As both swelling and shrinkage were measured only at the top of the clay beds, the data shown in the figure pertain to z=0mm. Further, the data refer to the first cycle of swelling and shrinkage. As can be observed, both swelling and shrinkage of the clay beds increased with increase in time period. The figure shows the plain clay bed underwent maximum swelling and shrinkage which were respectively 35mm and 11.25 mm at the end of the 1st swell-shrink cycle. Or, the plain or the unblended expansive clay bed swelled to the maximum in 10 days and shrank to the minimum in 50 days. However, when the clay bed was stabilised with chemical additives both swelling and shrinkage decreased. In other words, chemical stabilisation of expansive clays had a significant influence on the rate and the amount of swelling and shrinkage. Chemical stabilisation of expansive clays results in flocculation of particles which is chiefly instrumental in reducing swelling and shrinkage. The reduction in the rate and the amount of swelling and shrinkage, however, varied with the type of the additive. For example, the swelling of 35mm of the clay bed in the un-blended condition was reduced respectively to 2.84mm, 0.23mm, 15.9mm and 10.7mm when the clay bed was stabilised with 6% lime, 20% cement, 20% fly ash and 20% GGBS. Hence, at the end of the 1st cycle the reduction in swelling due to chemical stabilisation was 91.88%, 99.34%, 54.57% and 69.4% respectively for 6% lime, 20% cement, 20% fly ash and 20% GGBS. When water evaporated, they underwent a shrinkage of 2.52mm, 0.04mm, 11.14mm and 6.65mm in comparison to a shrinkage of 11.25mm when the clay bed was unblended or unstabilised. Hence, at the end of the 1st cycle, the reduction in shrinkage due to chemical stabilisation was 77.60%, 99.65%, 0.97% and 40.88% respectively for 6% lime, 20% cement, 20% fly ash and 20% GGBS. The unblended clay bed which swelled most also shrank most.

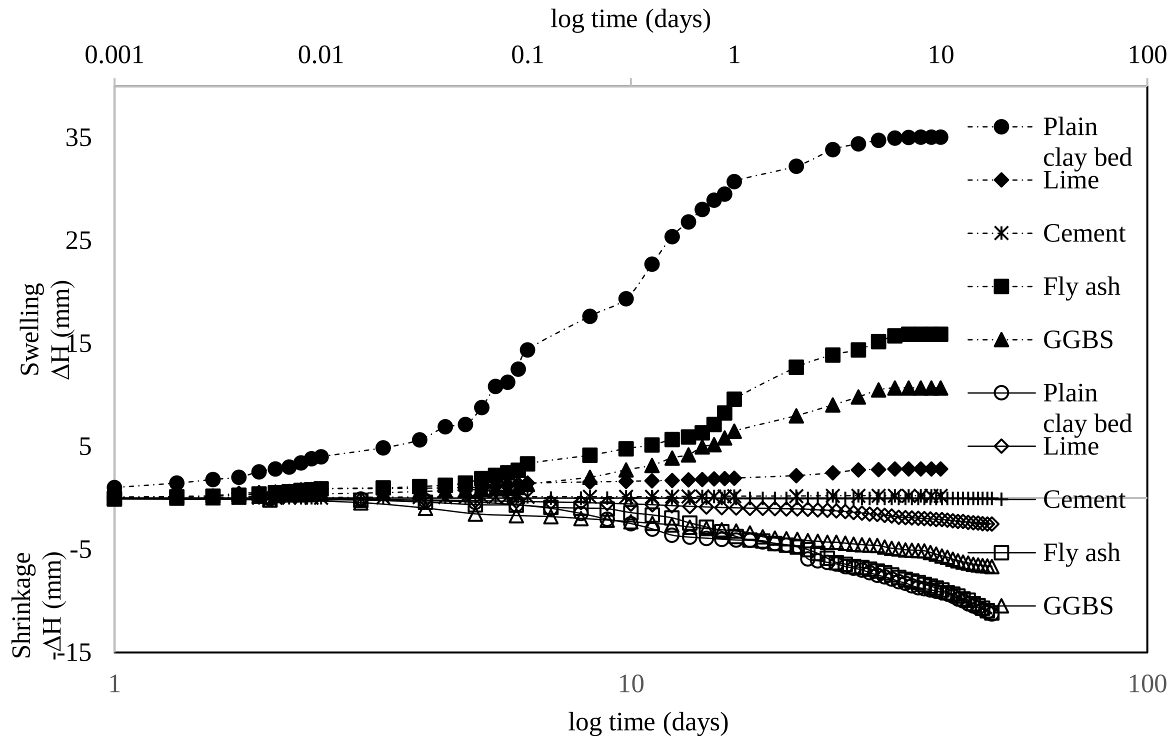


Figure 6.1. Swell-shrink profiles (cycle-1)

Similar data were recorded during the 2nd, 3rd, 4th and the 5th swell-shrink cycles also for the unblended clay bed and the clay beds stabilised with 6% lime, 20% cement, 20% fly ash and 20% GGBS. Figures 6.2 to 6.5 respectively show the swell-shrink data of the clay beds for the 2nd, 3rd, 4th and the 5th swell-shrink cycles. The data observed in the 2nd, 3rd, 4th and 5th cycles indicate that both swelling and shrinkage decreased with number of swell-shrink cycles.

The swell-shrink data shown in Figures 6.1 to 6.5 indicate that the clay bed stabilised by 6% lime and 20% cement resulted in the lowest values of swelling and shrinkage. This was true for all the swell-shrink cycles. 6% lime resulted in a reduction of 91.88% in swelling and 77.60% in shrinkage in the 1st cycle itself. Lime is known to be the most effective pozzolanic material resulting in flocculation, highly efficacious in controlling volumetric changes of expansive soils (Chen, 1988; Phanikumar, 2009). Reasonably good amounts of lime effectively control swelling and shrinkage which are detrimental volumetric changes in expansive soils. Cement-stabilised expansive clay beds resulted in the least amount of swelling and shrinkage. 20% of cement resulted in a staggering reduction of 99.34% in swelling and 99.65% in shrinkage in the 1st cycle itself. It can be said that the clay bed was rendered nearly free of volumetric strains by 20% cement. It was observed during the experimentation that the 20% cement-stabilised clay bed became very hard and practically did not allow any penetration of water during swelling. Hence, the least values of swelling and shrinkage. Fly ash and GGBS, being moderately pozzolanic materials though effective, control swelling and shrinkage reasonably well. The test data corroborate this. Of these two again, 20% GGBS proved to be more effective.

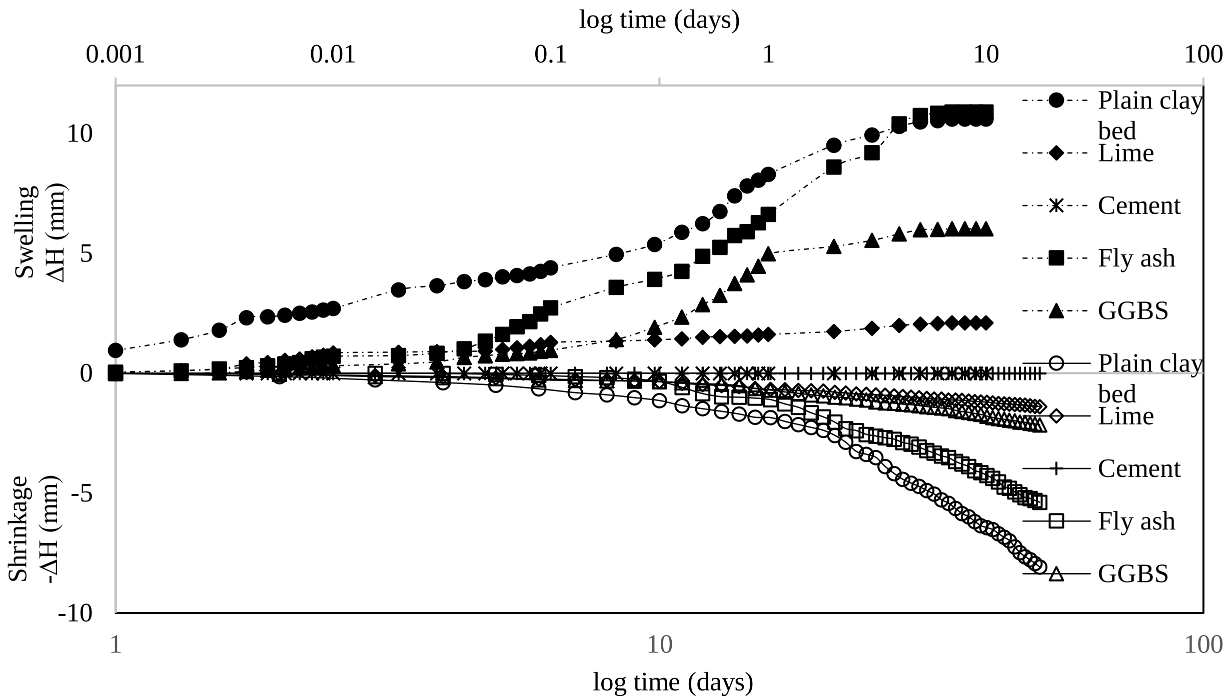


Figure 6.2. Swell-shrink profiles (cycle-2)

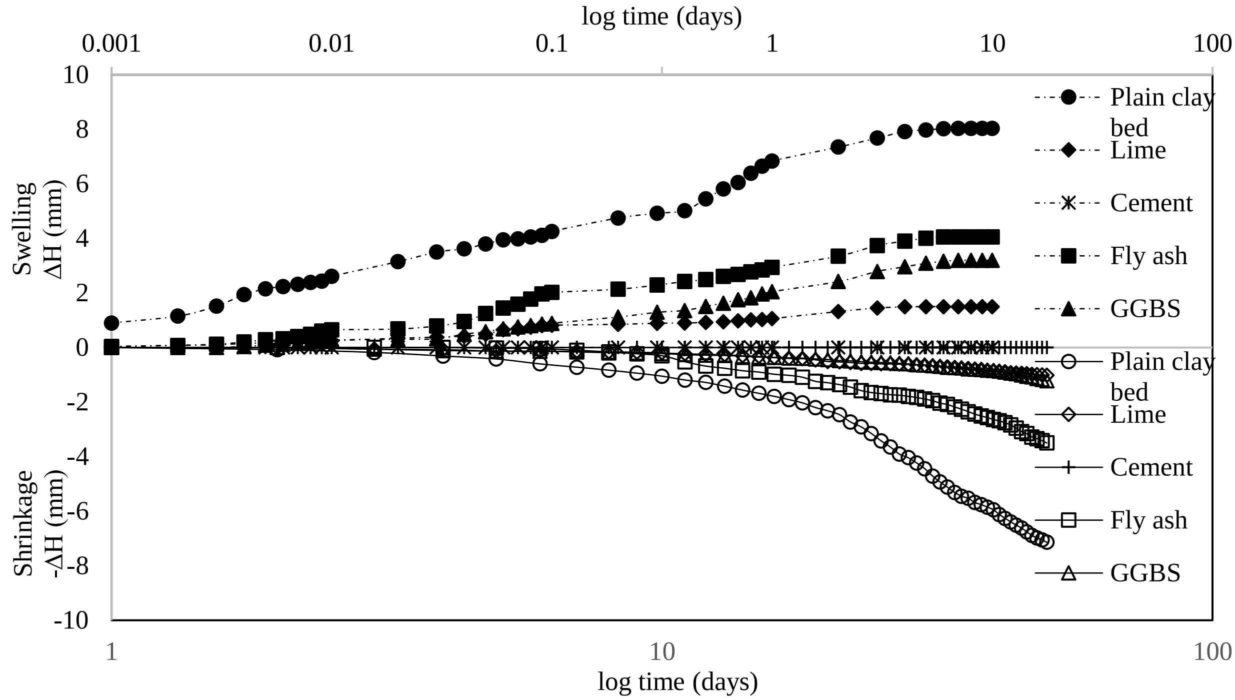


Figure 6.3. Swell-shrink profiles (cycle-3)

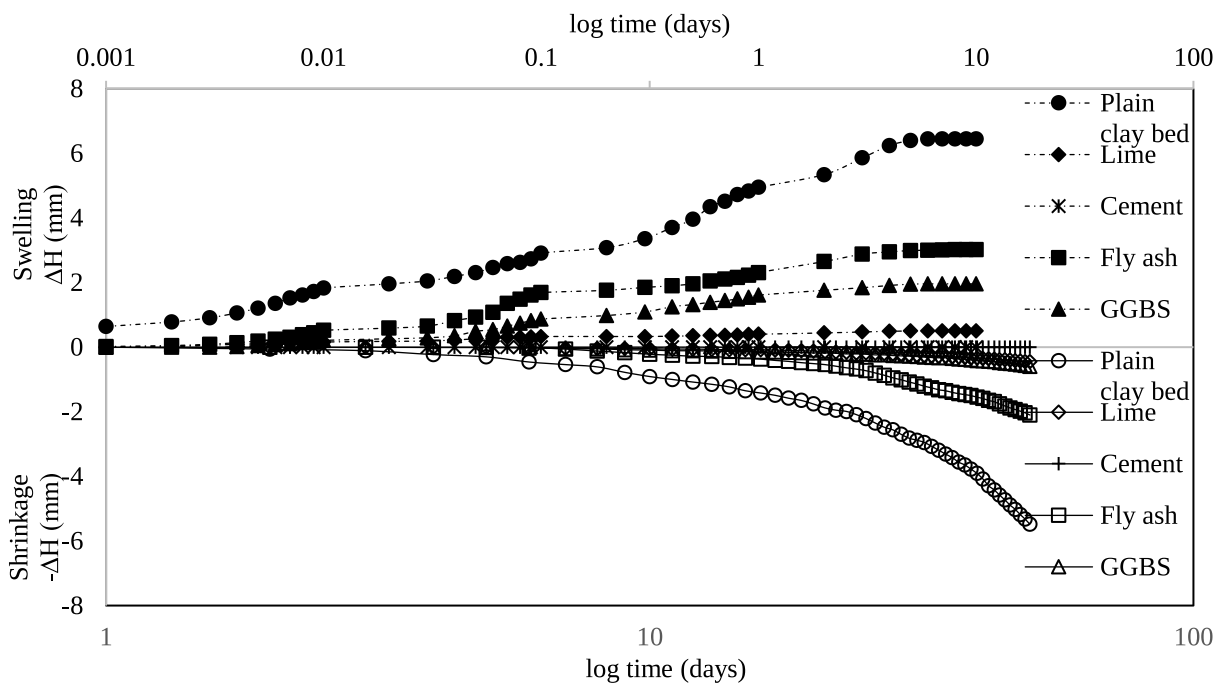


Figure 6.4. Swell-shrink profiles (cycle-4)

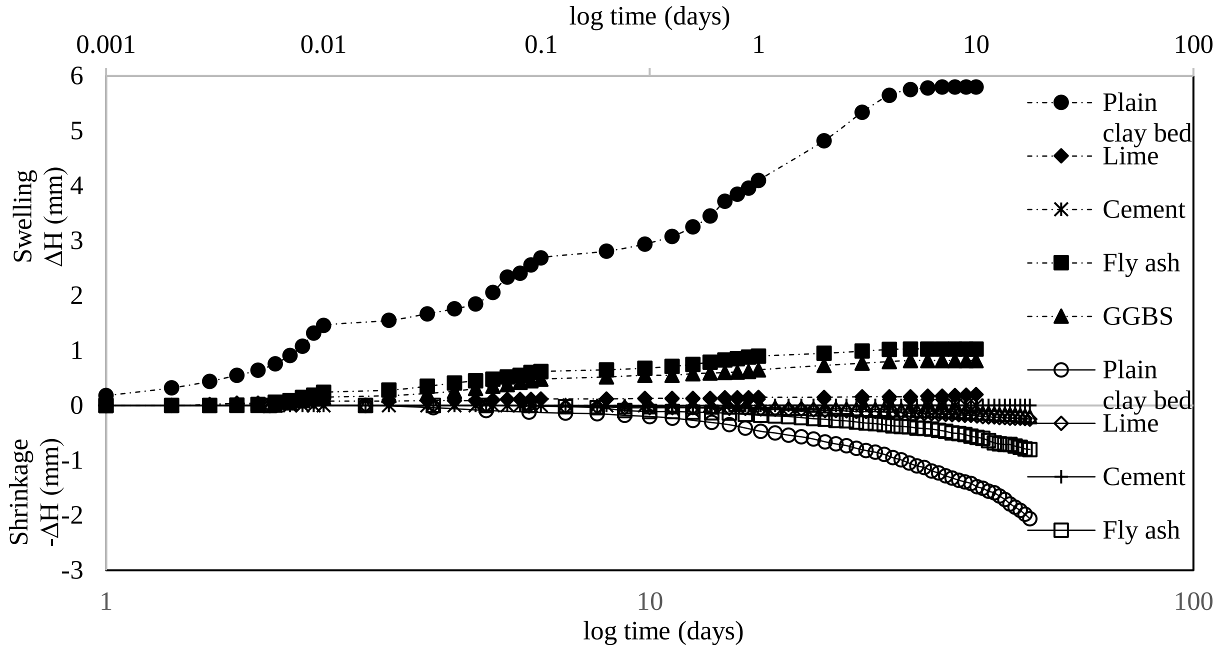


Figure 6.5. Swell-shrink profiles (cycle-5)

Based on the data shown in Figures 6.1 to 6.5, the resultant thickness (H_r) of the given clay bed at the end of the given cycle can be determined for the computation of % swelling and % shrinkage of the clay bed.

6.3.2. Influence of swell-shrink cycles:

Figure 6.6 compares the rate of swelling and shrinkage of the unblended clay bed which underwent five alternate swell-shrink cycles. The data refer to swelling and shrinkage measured at the top (z=0). The data show the effect of swell-shrink cycles on the rate and amount of swelling and shrinkage of the clay bed. At the end of 1st cycle, swelling was 35mm which decreased considerably during the 2nd, 3rd, 4th and the 5th cycles. When a clay bed is subjected to alternate swell-shrink cycles, it experiences fatigue and hence, results in reduced swelling. Swelling recorded for the 2nd, 3rd, 4th and the 5th cycles was respectively 10.6 mm, 8.04 mm, 6.45 mm and 5.8 mm, showing a reduction of 69.71%, 77.02%, 81.57% and 83.42% respectively. The data also reflect that the rate of swelling decreased with number of swell-shrink cycles. Similarly, shrinkage (mm) of the unblended expansive clay bed also decreased with increasing number of swell-shrink cycles. This is also because of fatigue of the clay bed over a series of swell-shrink cycles. The amount of shrinkage at the end of the1st cycle was 11.25mm, which decreased to 8.08mm, 7.14mm, 5.48mm and 2.06mm respectively at the end of the 2nd, 3rd, 4th

and the 5th cycles resulting in a reduction of 28.17%, 36.53%, 51.28% and 81.68% respectively. Therefore, swelling decreased by 83.42% and shrinkage by 81.68% when the number of swell-shrink cycles increased from 1 to 5.

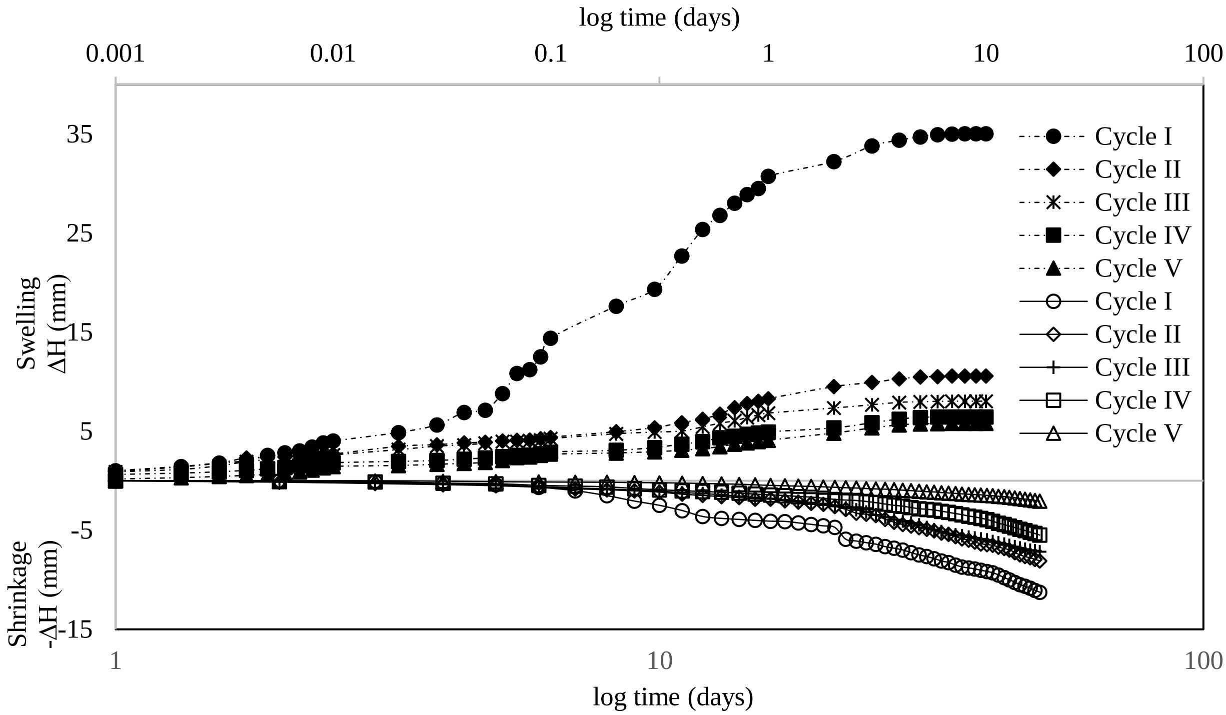


Figure 6.6. Swell-shrink profile of plain clay bed in different cycles

Figures 6.7 to 6.10 respectively compare the rate of swelling and shrinkage of the clay beds stabilised by 6% lime, 20% cement, 20% fly ash and 20% GGBS which underwent alternate swelling and shrinkage through 1st to 5th wetting-drying cycles. Rate and amount of swelling and shrinkage decreased with number of swell-shrink cycles. This was true for all the additives. This could be attributed to fatigue. The data refer to the top of the clay beds (z=0). Swelling and shrinkage of the chemically stabilised clay beds (6% lime, 20% cement, 20% fly ash and 20% GGBS) observed in 2nd, 3rd, 4th and 5th cycles were much less than that for the 1st cycle. Further, swelling and shrinkage decreased considerably with number of swell-shrink cycles. For example, in the case of 6% lime-stabilised expansive clay bed, swelling was 2.84mm for the 1st cycle which decreased to 0.20mm for the 5th and the last cycle, showing a reduction of 93%. Similar swelling data were obtained for the clay beds stabilised by 20% cement, 20% fly ash and 20% GGBS also. Similarly, in the case of 6% lime-stabilised expansive clay bed, shrinkage was 2.52 mm, which decreased to 0.24mm for the 5th and the last cycle showing a reduction of 90%.

Similar shrinkage data were obtained for the clay beds stabilised by 20% cement, 20% fly ash and 20% GGBS also.

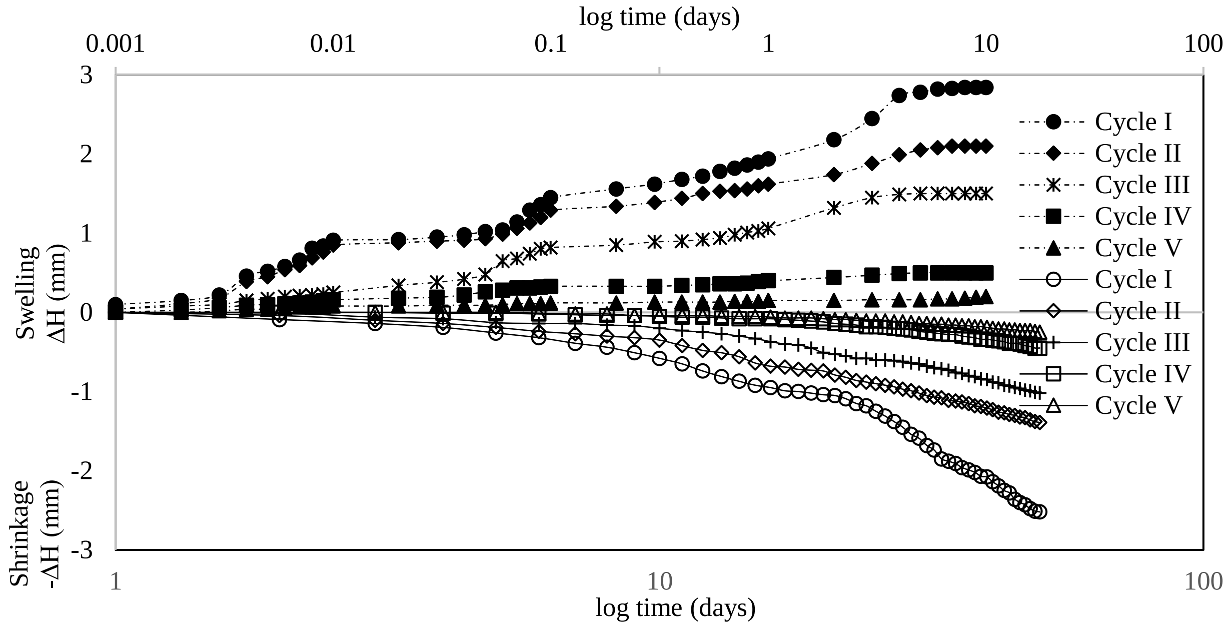


Figure 6.7. Swell-shrink profiles of lime-stabilised clay bed

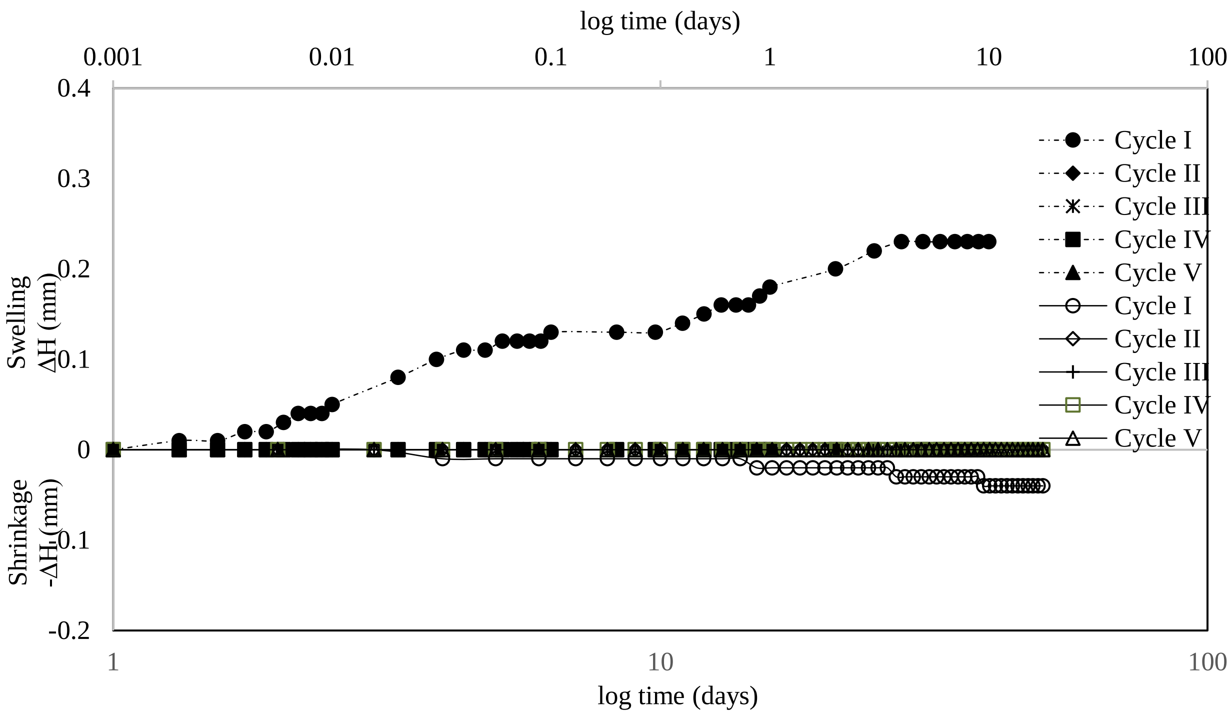


Figure 6.8. Swell-shrink profiles of cement-stabilised clay bed

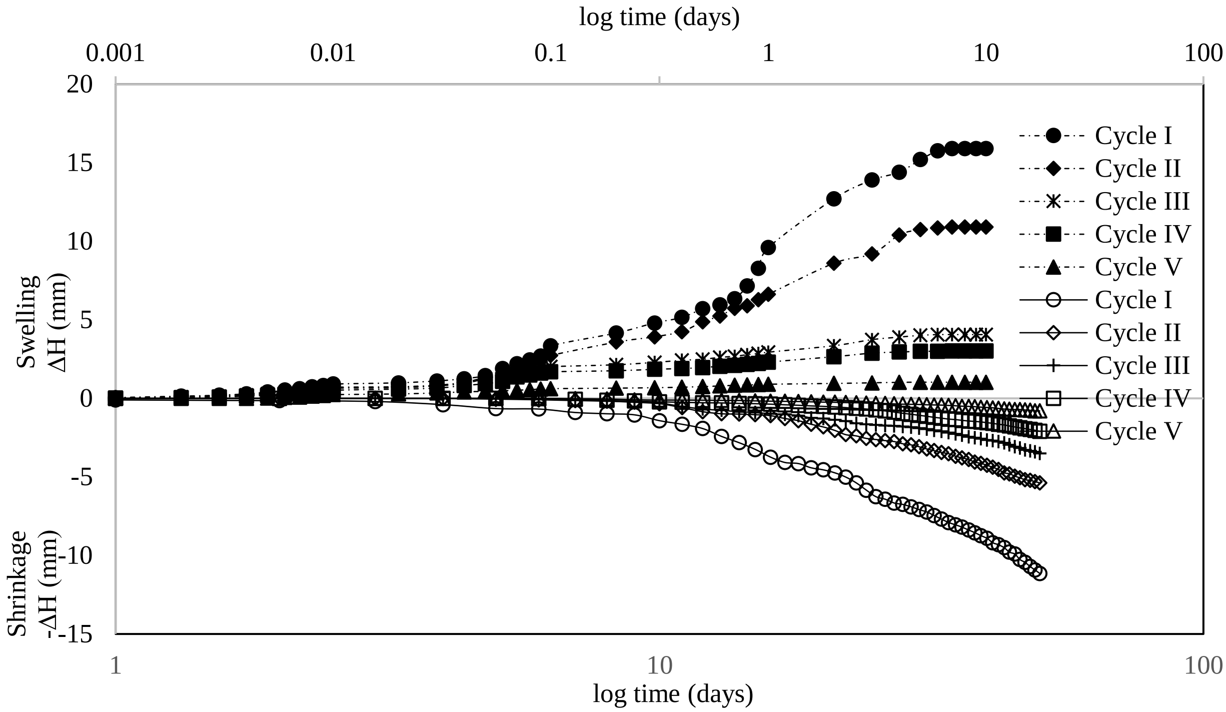


Figure 6.9. Swell-shrink profiles of fly ash-stabilised clay bed

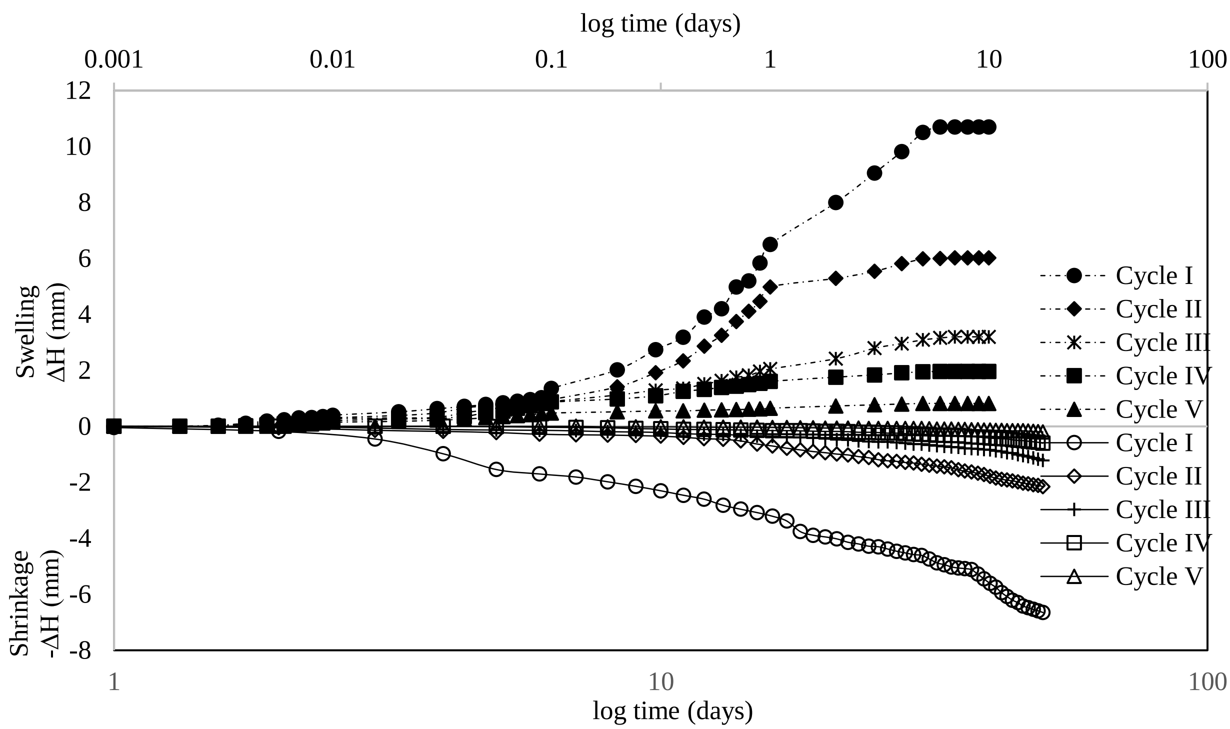


Figure 6.10. Swell-shrink profiles of GGBS-stabilised clay bed

From the data of Figures 6.6 to 6.10, the significant reduction in swelling and shrinkage with increasing number of swell-shrink cycles was unmistakable though the % reduction might have varied depending upon the type of additive. For example, the effect of the increase in the number of swell-shrink or wetting-drying cycles on swelling and shrinkage was tremendous in the cases of the expansive clay beds stabilised by 6% lime and 20% cement; lime and cement are highly pozzolanic as discussed above. And the effect of the increase in the number of swell-shrink cycles was slightly less in the cases of the clay beds stabilised by 20% fly ash and 20% GGBS. Hence, the increased number of cycles and the high pozzolanic activity of lime and cement significantly reduced swelling and shrinkage of the clay beds. A comparison of Figures 6.7 and 6.8 with Figures 6.9 and 6.10 reveals this.

Table 6.1 summarises the swell-shrink data of different expansive clay beds undergoing swelling and shrinkage in different wetting-drying cycles.

Table 6.1. Swell-shrink data

	Cycle-1		Cycle-2		Cycle-3		Cycle-4		Cycle-5	
	Swelling (mm)	Shrinkage (mm)	Swelling (mm)	Shrinkage (mm)	Swelling (mm)	Shrinkage (mm)	Swelling (mm)	Shrinkage (mm)	Swelling (mm)	Shrinkage (mm)
Expansive clay	35.00	11.25	10.60	8.08	8.04	7.14	6.45	5.48	5.80	2.06
Expansive clay + 6% lime	2.84	2.52	2.10	1.39	1.50	1.02	0.50	0.45	0.20	0.25
Expansive clay + 20% cement	0.23	0.04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Expansive clay + 20% fly ash	15.90	11.14	10.90	5.38	4.06	3.50	3.02	2.10	1.03	0.80
Expansive clay + 20% GGBS	10.70	6.65	6.02	2.15	3.20	1.22	1.96	0.60	0.82	0.20

Figure 6.11 shows the variation of swelling (mm) with number of swell-shrink cycles. The values of swelling plotted are those of equilibrium swelling recorded at the end of the swelling phase. They pertain to the plain clay bed and to the clay beds stabilised by 6% lime, 20% cement, 20% fly ash and 20% GGBS. Swelling decreased significantly with number of swell-shrink cycles. This was true for all the above clay beds. The number of swell-shrink cycles

would have a huge influence on volumetric changes such as swelling. As the clay bed is subjected to various swell-shrink cycles, it experiences fatigue and can not exhibit the same potential to swell or shrink as it may have exhibited in the previous cycle(s). The effect of fatigue on swelling owing to increased number of swell-shrink cycles was found to be significant in all the above clay beds. In the cases of chemically stabilised clay beds, this influence of fatigue was particularly large. This could be attributed to flocculation and pozzolanic reaction. For example, the swelling (mm) of the plain expansive clay bed decreased from 35.00 mm in the 1stcycle to 5.80 mm in the 5thcycle, indicating a reduction of 83.42%. And the clay beds stabilised by 6% lime, 20% cement, 20% fly ash and 20% GGBS exhibited a reduction of 92.95%, 100%, 93.52% and 92.33% respectively when they passed from the 1st cycle to 5th cycle.

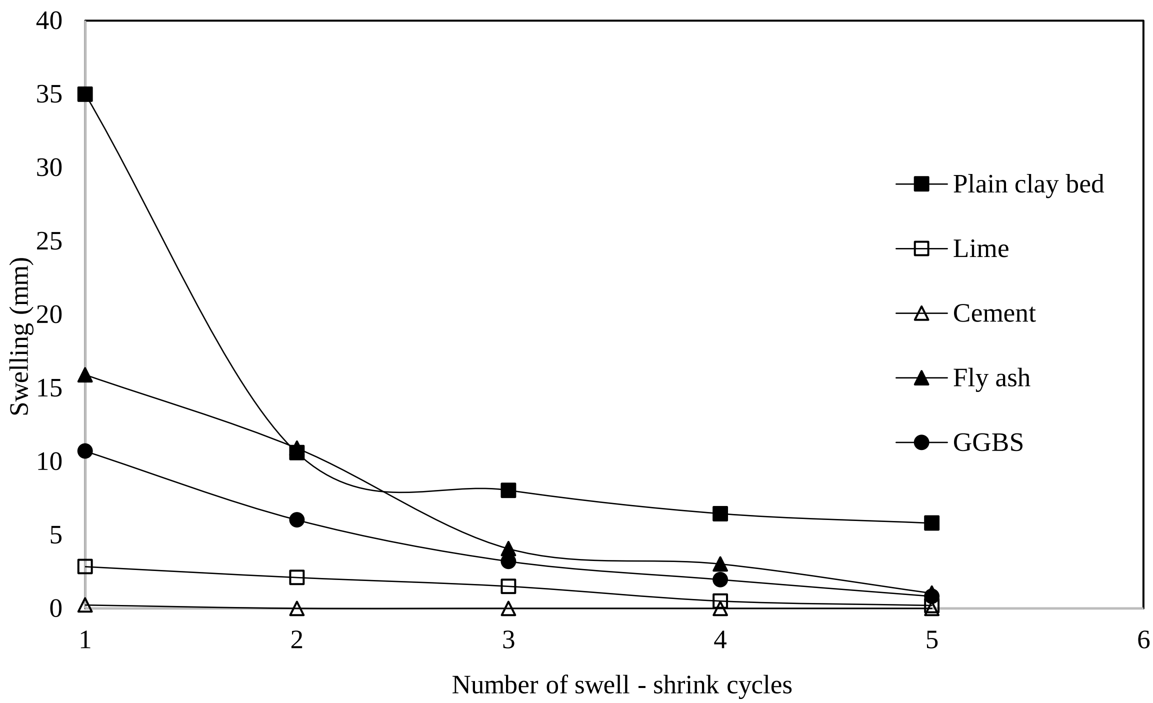


Figure 6.11. Effect of swell-shrink cycles on swelling

Figure 6.12 depicts the variation of shrinkage (mm) with number of swell-shrink cycles. The values of shrinkage plotted are those of equilibrium shrinkage recorded at the end of the shrinkage phase. They pertain to the plain or the unblended expansive clay bed and to the clay beds stabilised by 6% lime, 20% cement, 20% fly ash and 20% GGBS. Shrinkage decreased significantly with number of swell-shrink cycles. This was true for all the above-mentioned clay beds. Upon being subjected to a series of swell-shrink cycles, the clay bed experiences fatigue and can not possess the same potential for volumetric shrinkage as it may have undergone in the previous cycle(s). The effect of fatigue on shrinkage owing to increased number of swell-shrink cycles was found to be significant in all the clay beds. In the cases of chemically stabilised clay beds, this influence of fatigue on shrinkage was particularly large. This could be attributed to flocculation and pozzolanic reaction. For example, the shrinkage (mm) of the plain expansive clay bed decreased from 11.25mm in the 1stcycle to 2.06mm in the 5thcycle, indicating a reduction of 81.60%. And the clay beds stabilised by 6% lime, 20% cement, 20% fly ash and 20% GGBS exhibited a reduction of 90.07%, 100%, 92.80% and 96.99% respectively when they passed from the 1stcycle to 5thcycle.

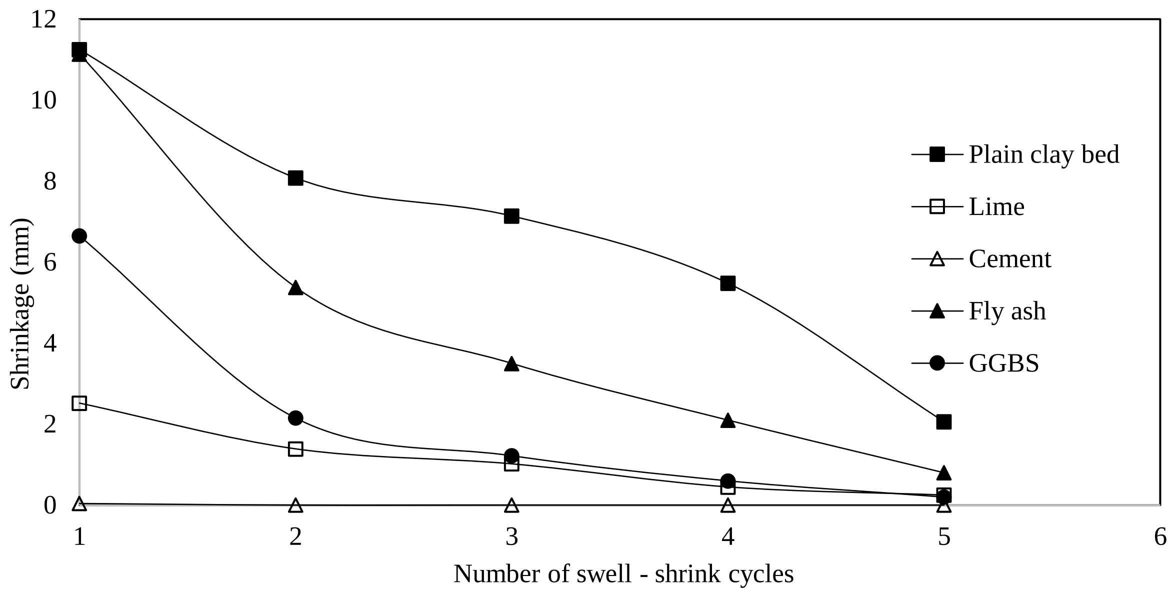


Figure 6.12. Influence of swell-shrink cycles on shrinkage

6.3.3. Resultant thickness (H_r):

As the clay beds undergo alternate swelling and shrinkage upon their absorbing water and losing water to the atmosphere respectively, their thickness at the end of the equilibrium of either phase (swelling phase or shrinkage phase) keeps changing. It is the datum thickness from which the clay bed either swells or shrinks depending upon which phase it is in. It depends on the amount of swelling or shrinkage the clay bed may have undergone in the previous phase of the cycle under consideration. The subsequent % swelling or % shrinkage depends on this thickness of the clay bed. This can be termed resultant thickness (H_r). For example, if H_i is the initial thickness of the clay bed which undergoes a swelling (mm) of $(\Delta H_{sw})_1$ and a shrinkage (mm) of $(\Delta H_{shr})_1$

during the 1st swell-shrink cycle, then its resultant thickness at the end of the 1st cycle $(H_r)_1$ or the beginning of the 2nd cycle can be written as

$$H_{r1} = H_i + (\Delta H_{sw})_1 - (\Delta H_{shr})_1 \quad (6.1)$$

The same principle applies in the computation of H_r in the subsequent cycles also. Hence, if an clay bed is subjected to n number of swell-shrink cycles and undergoes a swelling of $(\Delta H_{sw})_1$, $(\Delta H_{sw})_2$ etc. in the 1st cycle, 2nd cycle etc., and undergoes a shrinkage of $(\Delta H_{shr})_1$, $(\Delta H_{shr})_2$ etc. in the 1st cycle, 2nd cycle etc., then at the end of all the n swell-shrink cycles, the resultant thickness can be computed from the following equation:

$$H_{rn} = H_i + [(\Delta H_{sw})_1+(\Delta H_{sw})_2----+(\Delta H_{sw})_n]-[(\Delta H_{shr})_1+(\Delta H_{shr})_2----+(\Delta H_{shr})_n] \quad (6.2)$$

Resultant thickness (H_r) is most useful in computing percent swell $[(\Delta H_{sw})/H \text{ x } 100]$ or percent shrinkage $[(\Delta H_{shr})/(H+\Delta H_{sw}) \text{ x}100]$ of the expansive clay beds in different cycles.

Figure 6.13 depicts the variation of resultant thickness (H_r) with the number of swell-shrink cycles for the plain clay bed and the chemically stabilised expansive clay beds. The data shown pertain to H_r for swelling. The data indicate that H_r for swelling showed considerable variation with the number of cycles excepting in the cases of lime-stabilised and cement-stabilised clay beds. When the expansive clay beds were stabilised by 6% lime and 20% cement, H_r for swelling was nearly constant indicating reduced volumetric changes. Figure 6.14 depicts the variation of the resultant thickness (H_r) with the number of swell-shrink cycles for all the above clay beds. The data shown pertain to H_r for shrinkage. The data indicate that H_r for shrinkage was nearly constant in all the cycles for all the clay beds.

Table 6.2 summarises the values of resultant thickness for all the expansive clay beds in different cycles.

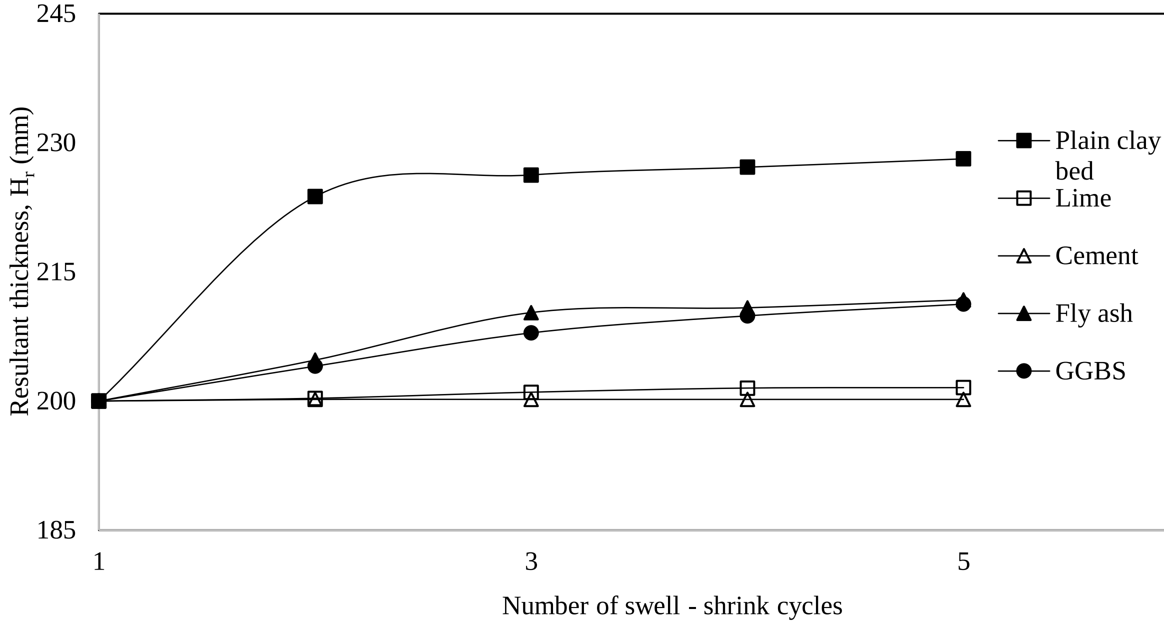


Figure 6.13. Variation of H_r (for swelling)

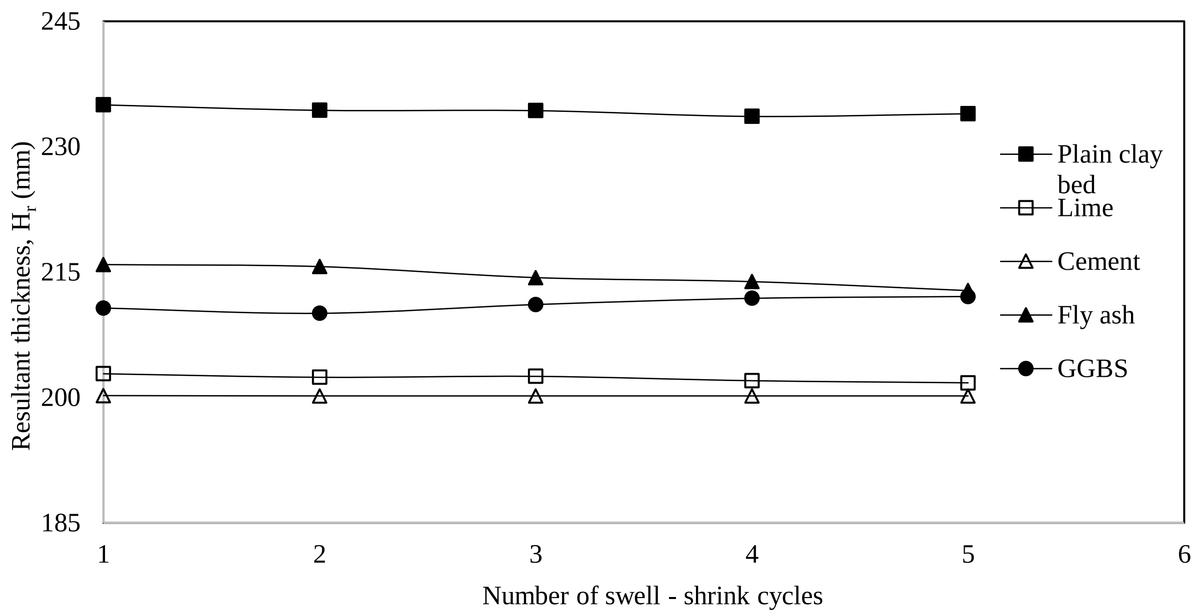


Figure 6.14. Variation of H_r (for shrinkage)

Figure 6.15 summarises the swell-shrink data divided into five swell-shrink cycles. The figure depicts the data recorded during the five swell-shrink cycles. The figure shows how swelling (mm) and shrinkage (mm) of the clay beds decreased with number of swell-shrink cycles. Further, data for resultant thickness (H_r), based on which % swelling and % shrinkage can be computed in a particular cycle, are also provided in the figure.

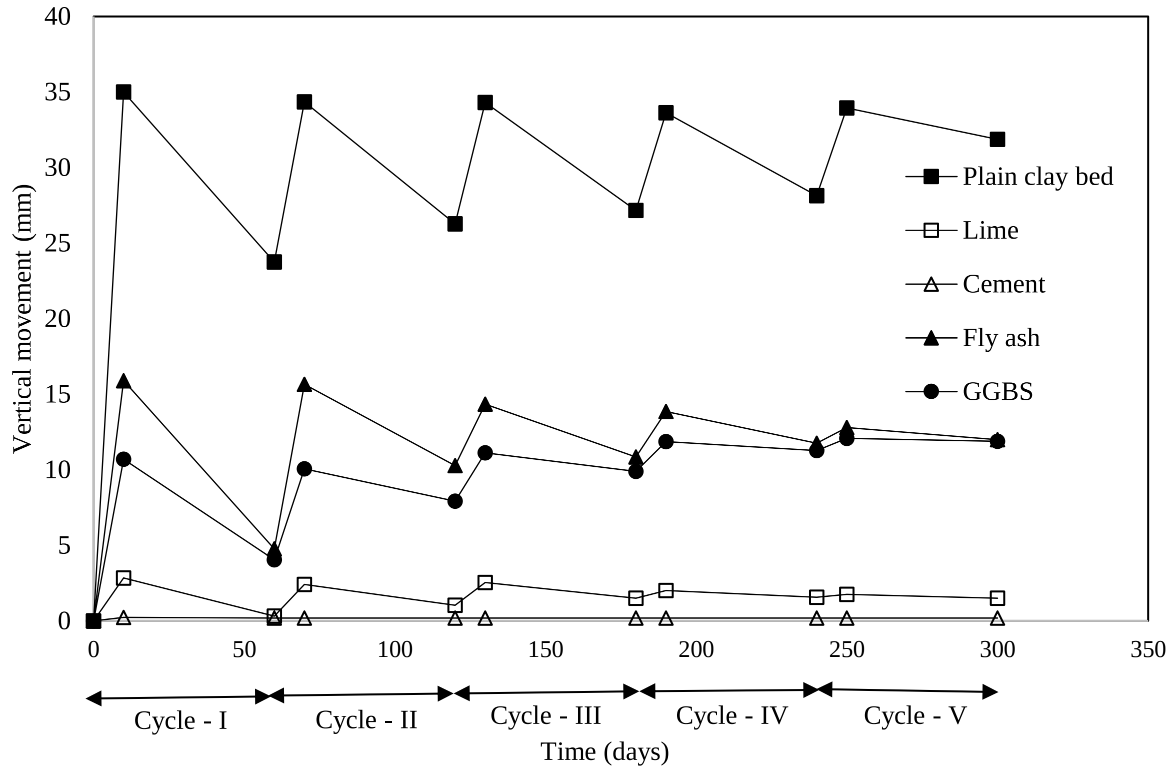


Figure 6.15. Combined swelling and shrinkage (mm) versus time (days)

Table 6.2. Resultant thickness (mm)

	Expansive clay	Expansive clay + 6% lime	Expansive clay + 20% cement	Expansive clay + 20% fly ash	Expansive clay + 20% GGBS
1st cycle swelling	200.00	200.00	200.00	200.00	200.00
1st cycle shrinkage	235.00	202.84	200.23	215.90	210.70
2nd cycle swelling	223.75	200.32	200.19	204.76	204.05
2nd cycle shrinkage	234.35	202.42	200.19	215.66	210.07
3rd cycle swelling	226.27	201.03	200.19	210.28	207.92
3rd cycle shrinkage	234.31	202.53	200.19	214.34	211.12
4th cycle swelling	227.17	201.51	200.19	210.84	209.90
4th cycle shrinkage	233.62	202.01	200.19	213.86	211.86
5th cycle swelling	228.14	201.56	200.19	211.76	211.26
5th cycle shrinkage	233.94	201.76	200.19	212.79	212.08

6.3.4. Shrinkage cracks:

As the clay beds underwent volumetric reduction or shrinkage, cracks developed in them in two patterns, namely, polygonal shrinkage pattern and linear shrinkage pattern. The largest width (mm) of these two different types of cracks was determined every ten days of the period of 50 days in each cycle.

Figures 6.16 and 6.17 respectively show the rate of propagation of polygonal shrinkage cracks in all the clay beds in the 1^{st} cycle and the 5^{th} cycle. The crack width (mm) was plotted in arithmetic scale on the *Y*-axis against time in days shown in log scale on the *X*-axis. The crack width data observed in the other cycles are not being shown for want of space. Crack width increased with increase in shrinkage period. This was found to be true in all the swell-shrink cycles. As shrinkage increased with the shrinkage period, shrinkage crack width also increased with increase in time period.

The propagation of polygonal shrinkage crack width was most rapid in the plain clay bed. However, it decreased when the clay beds were chemically stabilised. It was the least when the clay beds were stabilised by 6% lime and 20% cement indicating that 6% lime and 20% cement reduced the volumetric changes of the expansive clay bed quite effectively. This pattern was found in all the swell-shrink cycles though with increase in number of cycles the crack width decreased significantly. For the plain or the unblended expansive clay bed the largest polygonal crack width was 18mm in the 1^{st} cycle and 10mm in the 5^{th} cycle; and it was 9mm, 2mm, 14mm and 5mm in the 1^{st} cycle and 5mm, 0mm, 8mm and 0mm in the 5^{th} cycle respectively for the clay beds stabilised by 6% lime, 20% cement, 20% fly ash and 20% GGBS. The width of the polygonal shrinkage cracks lay in between these values for the other cycles.

Figure 6.18 summarises the variation of the polygonal shrinkage crack width of all the expansive clay beds with the number of swell-shrink cycles. The polygonal shrinkage crack width decreased with increase in number of swell-shrink cycles. Table 6.3 summarises the width (mm) of the polygonal shrinkage crack developed in different cycles in all the clay beds.

Table 6.3. Polygonal shrinkage crack width (mm)

	Cycle-1	Cycle-2	Cycle-3	Cycle-4	Cycle-5
Expansive clay	18.00	16.00	12.00	11.00	10.00
Expansive clay + 6% lime	9.00	7.00	6.00	5.50	5.00
Expansive clay + 20% cement	2.00	1.50	0.50	0.50	0.00
Expansive clay + 20% fly ash	14.00	11.00	10.00	9.50	8.00
Expansive clay + 20% GGBS	5.00	3.50	2.50	1.00	0.00

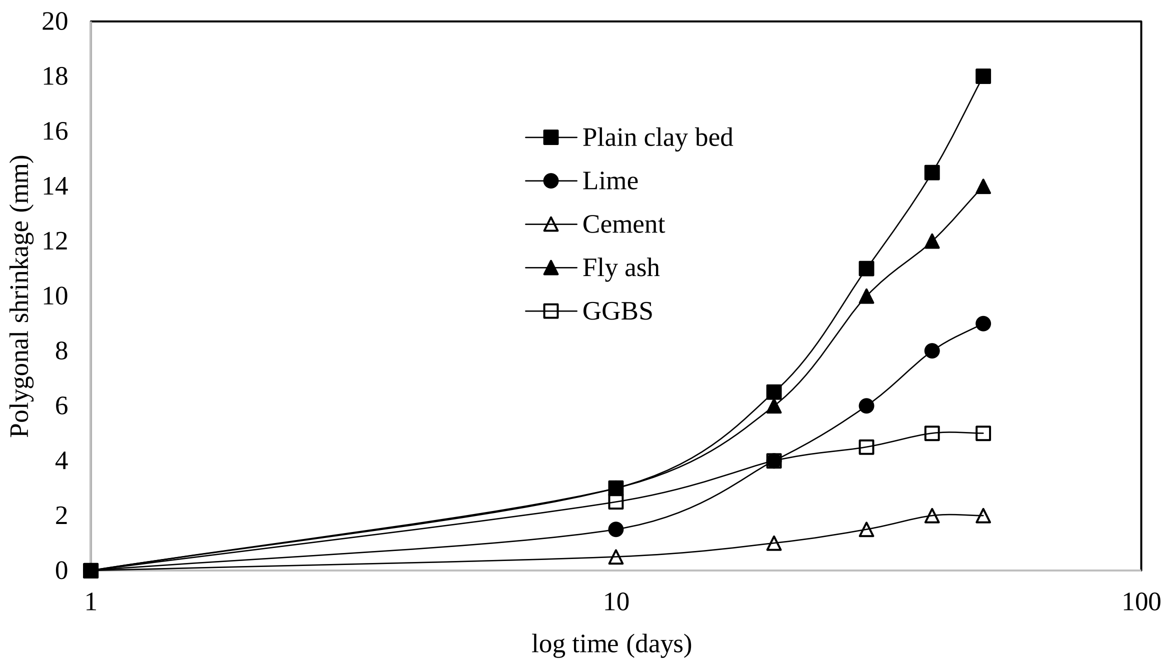


Figure 6.16. Variation of polygonal shrinkage crack width with time (cycle-1)

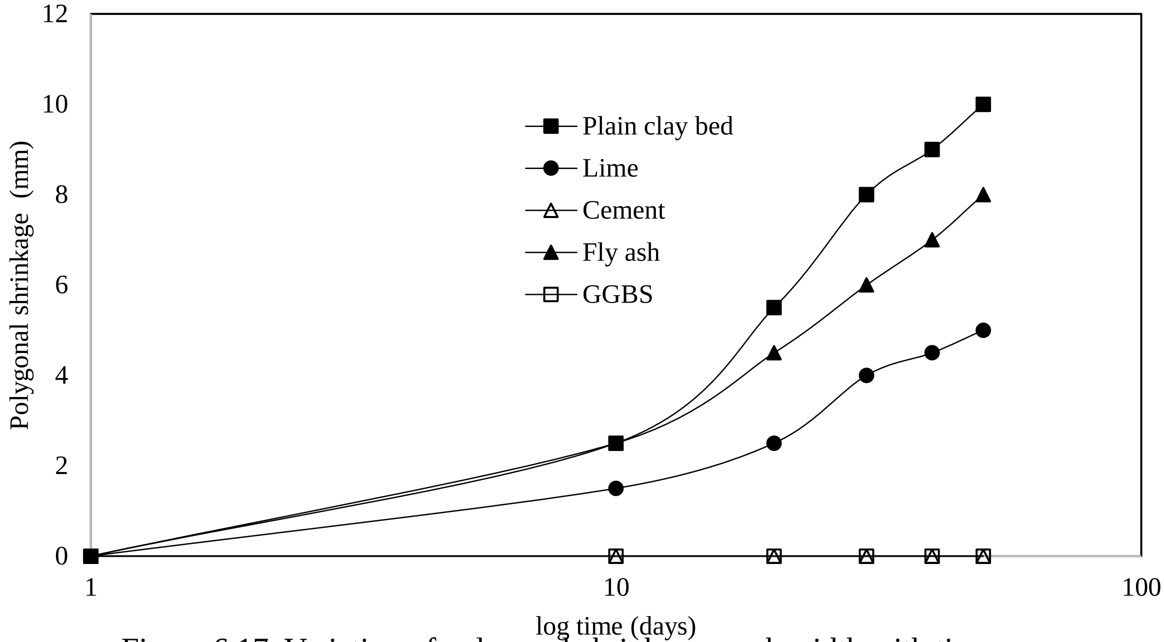


Figure 6.17. Variation of polygonal shrinkage crack width with time (cycle-5)

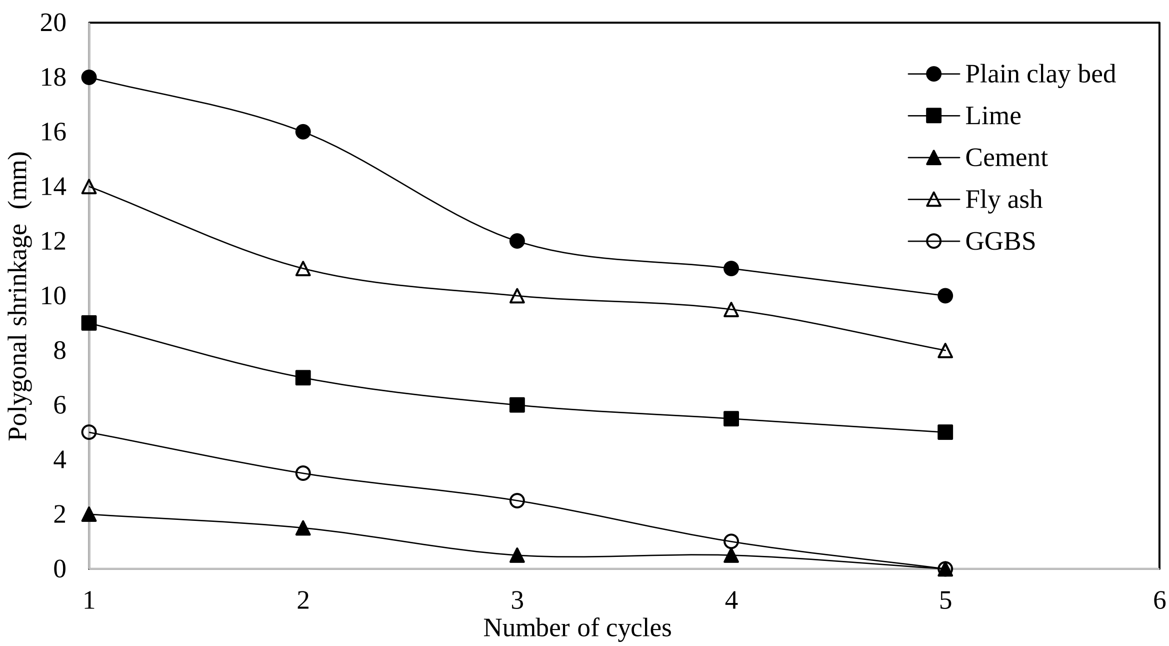


Figure 6.18. Variation of polygonal shrinkage

Linear shrinkage crack width was also measured in all the clay beds every ten days during the period of 50 days. It was also measured in all the swell-shrink cycles. Figures 6.19 and 6.20 respectively show the rate of propagation of linear shrinkage cracks in all the clay beds in the 1st cycle and the 5th cycle. The crack width (mm) was plotted in arithmetic scale on *Y*-axis against time in days shown in log scale on the *X*-axis. The crack width data observed in the other cycles are not being shown for want of space. Linear shrinkage crack width also increased with increase in shrinkage period. This was found to be true in all the swell-shrink cycles.

The propagation of the linear shrinkage was most rapid in the plain clay bed. However, it decreased in the chemically stabilised clay beds. Linear shrinkage was also found to be the least in the clay beds stabilised by 6% lime and 20% cement indicating that 6% lime and 20% cement effectively reduced the volumetric changes of the clay bed. This pattern was found in all the swell-shrink cycles though linear shrinkage also decreased with increasing number of cycles. For the plain or the unblended expansive clay bed the largest linear shrinkage crack width was 20mm in the 1st cycle and 14.5mm in 5th cycle; and it was 8mm, 2mm, 16mm and 6mm in the 1st cycle and 4mm, 0mm, 8.5mm and 0mm in the 5th cycle respectively for the clay beds stabilised by 6% lime, 20% cement, 20% fly ash and 20% GGBS. The width of the linear shrinkage cracks lay in between these values for the other cycles.

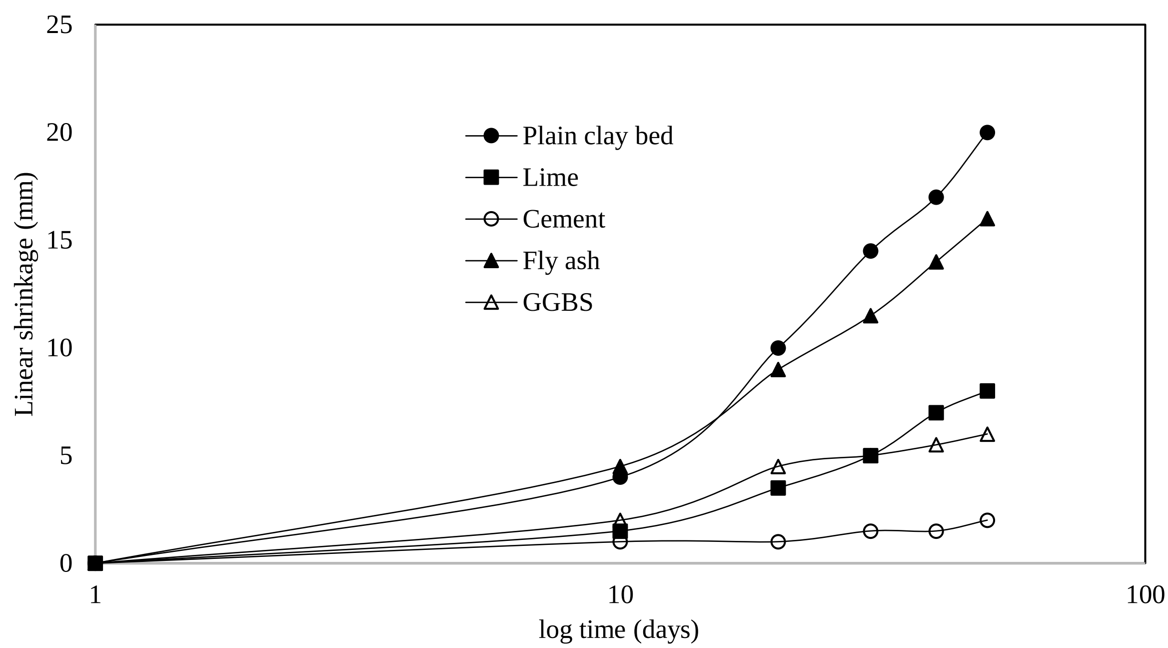


Figure 6.19. Variation of linear shrinkage crack width with time (cycle-1)

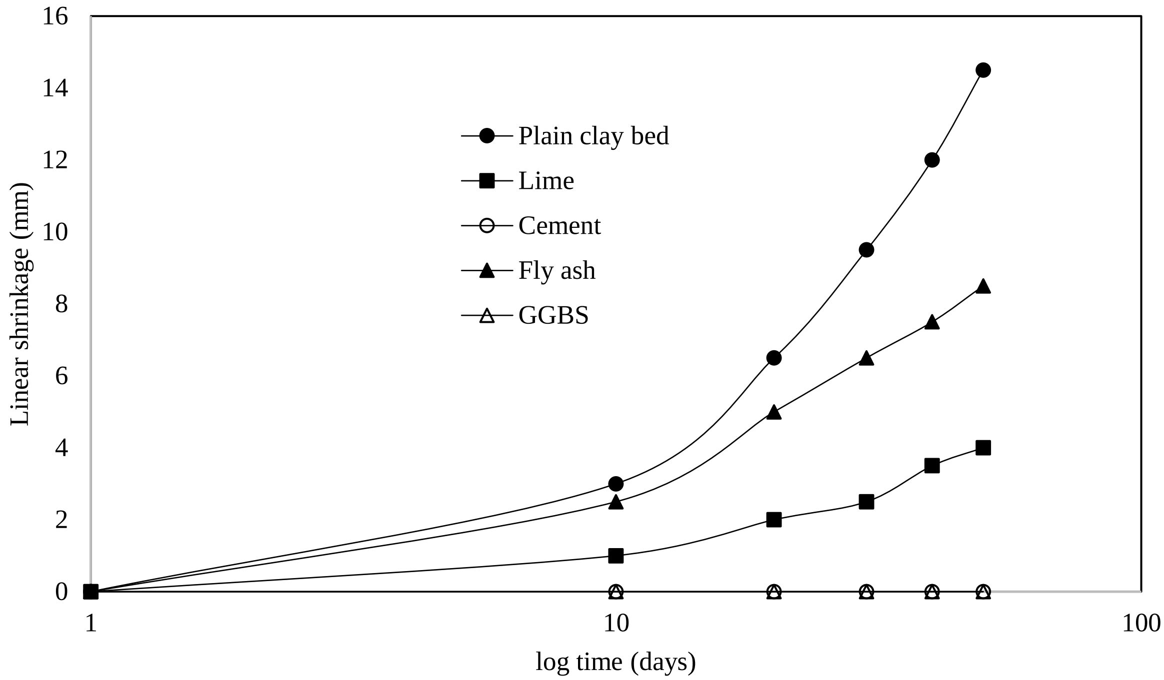


Figure 6.20. Variation of linear shrinkage crack width with time (cycle-5)

Figure 6.21 summarises the variation of the linear shrinkage crack width of all the expansive clay beds with the number of swell-shrink cycles. The linear shrinkage crack width also decreased with increasing number of swell-shrink cycles. Table 6.4 summarises the width (mm) of the linear shrinkage crack developed in different cycles in all the clay beds.

Table 6.4. Linear shrinkage crack width (mm)

	Cycle-1	Cycle-2	Cycle-3	Cycle-4	Cycle-5
Expansive clay	20.00	19.00	17.00	16.00	14.50
Expansive clay + 6% lime	8.00	7.00	6.50	5.50	4.00
Expansive clay + 20% cement	2.00	1.00	0.50	0.00	0.00
Expansive clay + 20% fly ash	16.00	14.00	11.50	10.00	8.50
Expansive clay + 20% GGBS	6.00	4.00	2.50	0.50	0.00

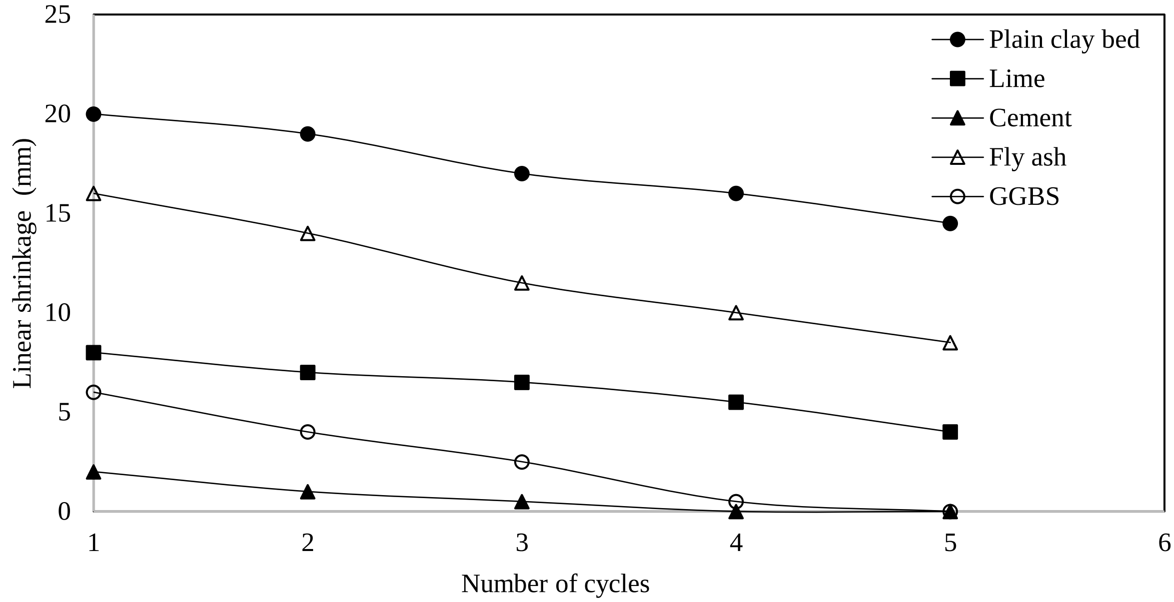


Figure 6.21. Variation of linear shrinkage

6.4. Conclusions:

A remoulded expansive clay bed stabilised by 6% lime, 20% cement, 20% fly ash and 20% GGBS was made to undergo five swell-shrink cycles. The upward and downward movements during the alternate swell-shrink cycles was recorded and compared with that of the plain or the unblended clay bed. The propagation of polygonal and linear shrinkage cracks was also studied. The following conclusions can be drawn from the study:

1. In all the swell-shrink cycles, the chemically stabilised clay beds showed reduced swelling and shrinkage compared to the plain or the unblended expansive clay bed. This could be attributed to flocculation and pozzolanic reaction which resulted in reduced volumetric changes, be it swelling or shrinkage. The expansive clay beds stabilised by 6% lime and 20% cement exhibited the lowest range of swelling and shrinkage. This was true in all the cycles. Of these two chemicals again, 20% cement resulted in the lowest amount of swelling and shrinkage.

2. As lime and cement are the most effective pozzolanic additives, swelling and shrinkage of the expansive clay bed were effectively reduced when stabilised by them. It may be

mentioned that, when stabilised by 20% cement, the expansive clay bed swelled and shrank by negligible amounts of 0.23mm and 0.04mm respectively in the 1st cycle itself. In the subsequent cycles, the swelling and shrinkage of the clay bed stabilised by 20% cement was 0mm. In controlling swelling and shrinkage to negligible amounts, 6% lime was second only to 20% cement.

3. As the swell-shrink cycles progressed, the clay beds experienced fatigue. Hence, swelling and shrinkage decreased due to fatigue. The reduction in swelling and shrinkage with number of cycles was more predominant in the chemically stabilised expansive clay beds than in the plain expansive clay bed. 20% cement caused 100% reduction in swelling and shrinkage as the clay bed passed from Cycle - 1 to Cycle - 5; and 6% lime caused a reduction of 93% in swelling and 90% in shrinkage as the clay bed passed from Cycle - 1 to Cycle – 5.

4. The process of shrinkage caused polygonal and linear shrinkage cracks in the clay beds. The width of the largest polygonal shrinkage crack was found to be the greatest in the plain clay bed. However, it decreased as the clay bed was stabilised by various chemicals. This was found to be true in all the cycles though the polygonal shrinkage crack width in a given clay bed decreased with increasing number of swell-shrink cycles. Thus, the combined effect of chemical stabilisation and the progression in swell-shrink cycles was to considerably reduce the polygonal shrinkage cracks.

5. Linear shrinkage was also found to be the maximum in the plain clay bed, and it decreased as the clay bed was stabilised by various chemicals. This was found to be true in all the cycles. For a given clay bed, plain or chemically stabilised, linear shrinkage decreased with increasing number of swell-shrink cycles. Linear shrinkage was also considerably reduced by the combined effect of chemical stabilisation and advancement of swell-shrink cycles.

CHAPTER - 7

Quantification of shrinkage cracks using image analysis technique

7.1. Introduction:

Swelling clays or expansive clays belonging to smectite group abound in mineral montmorillonite which has an expanding lattice structure. These clays swell upon water absorption and shrink upon evaporation of water. The amount of swelling and shrinkage depends upon the amount of mineral montmorillonite present in them. Further, temperature variations and moisture absorption or evaporation also play an important role on the amount of swelling and shrinkage.

During shrinkage, which occurs because of evaporation of water upon drying, volumetric reduction takes place in the expansive clay specimens leading to development of shrinkage cracks or desiccation cracks. These cracks develop generally in a polygonal pattern. The width and the depth of the cracks and the volume of the cracks depend upon the amount of shrinkage undergone by the clay. Just as an understanding of the rate and amount of swelling helps in predicting the expansive clay behaviour, even so is an understanding of the development and propagation of the shrinkage cracks helpful in assessing the expansive clay phenomena. Hence, it is important to quantify both the types of volumetric changes occurring in expansive clays, namely, swelling and shrinkage.

This chapter, which studies the quantification of shrinkage cracks developing in relatively much smaller expansive clay specimens (prepared in shrinkage cups), is an appendix to a more detailed study of swelling and shrinkage of chemically stabilised expansive clay beds compacted in large test moulds of dia 300mm and thickness 400mm presented in Chapter-6.

7.2. Experimental investigation:

A series of experiments was performed in which the plain expansive clay samples and the chemically stabilised expansive clay samples were prepared in shrinkage cups and allowed to dry up so that the pattern of shrinkage cracks developing in the samples could be quantified. An air-dry expansive clay sample finer than 425µm sieve was used for preparing the samples. About 50g of the air-dried expansive clay powder was taken for preparing each sample – plain or treated chemically. The chemical additives and their dosages were as mentioned in Chapter-3 on Experimental investigation. The air-dried expansive clay and the amounts of the additives based on their dosages were thoroughly mixed for preparing the dry blend specimens. The moulding water contents for preparing the wet blend specimens were chosen as their respective *LL*, *PL*, *PI* and *OMC*. The thoroughly blended wet specimens were carefully transferred to the shrinkage cups which were patted gently in order to make the top surface level. Then, the specimens in the cups were left for air-drying for 48 hrs and then were oven-dried for 24 hrs at 105^0C temperature. After oven-drying, the samples were observed to have reduced in their volumes and diameters and to have developed polygonal shrinkage cracks or desiccation cracks. The samples also underwent linear shrinkage.

7.3. Image analysis:

Then, using a camera of 24 Mega pixels, pictures of all the shrunken samples were taken from a particular height (in this case 400mm) for ensuring clarity of the images. For quantifying the shrinkage or desiccation cracks by image analysis, these pictures or images of the samples were transferred to MATLAB and modified as grey scale images. Using the final binary images, the shrinkage area was evaluated. Based on the amount of the shrinkage undergone by the specimens as reflected in the crack widths and the shrinkage area, in the second stage of the image analysis, the binary images were processed again to obtain the crack patterns, crack widths, crack lengths, and crack areas (Al-Jeznawi et al. 2020; Singh et al. 2017; Lakshmikantha et al. 2009; Puppala et al. 2004). Two more important parameters, namely, crack density factor (*CDF*) and crack intensity factor (*CIF*) that reflect the crack area were also obtained from the above factors. They are given as percentages and are respectively written as below:

$$CDF\ (\%) = \frac{(Total\ crack\ area + shrinkage\ area)}{Initial\ specimen\ area} \times 100 \qquad (7.1)$$

and

$$CIF\ (\%) = \frac{Total\ crack\ area}{Reduced\ specimen\ area} \times 100 \qquad (7.2)$$

7.4. Results and discussion

7.4.1. Influence of additive contents and moulding water contents on CDF:

Figure 7.1 depicts the variation of *CDF* (%) with lime content. The data pertain to the moulding water contents used in the test programme, namely, *LL*, *PL*, *PI* and *OMC* of the lime-clay blends. As *PL* and *PI* could not be determined beyond 1% lime content (see Chapter-4), *CDF* data are not available for *PL* and *PI* beyond 1% lime content. The graphical data give an impression that, with increasing lime content, *CDF* (%) decreased. This was true for all the moulding water contents – *LL*, *PL*, *PI* and *OMC*. This is because of (a) increasing lime content in the blends and (b) decreasing *LL* with increasing lime content. As lime content increases, the clay-lime blends undergo less swelling and consequently reduced shrinkage. This leads to reduced crack pattern, crack width, crack length and crack volume. As non-expansive lime content increases in the blends, both swelling and shrinkage of the blends decrease. Hence, *CDF* (%) decreased with increasing lime content. This trend was observed at all the moulding water contents.

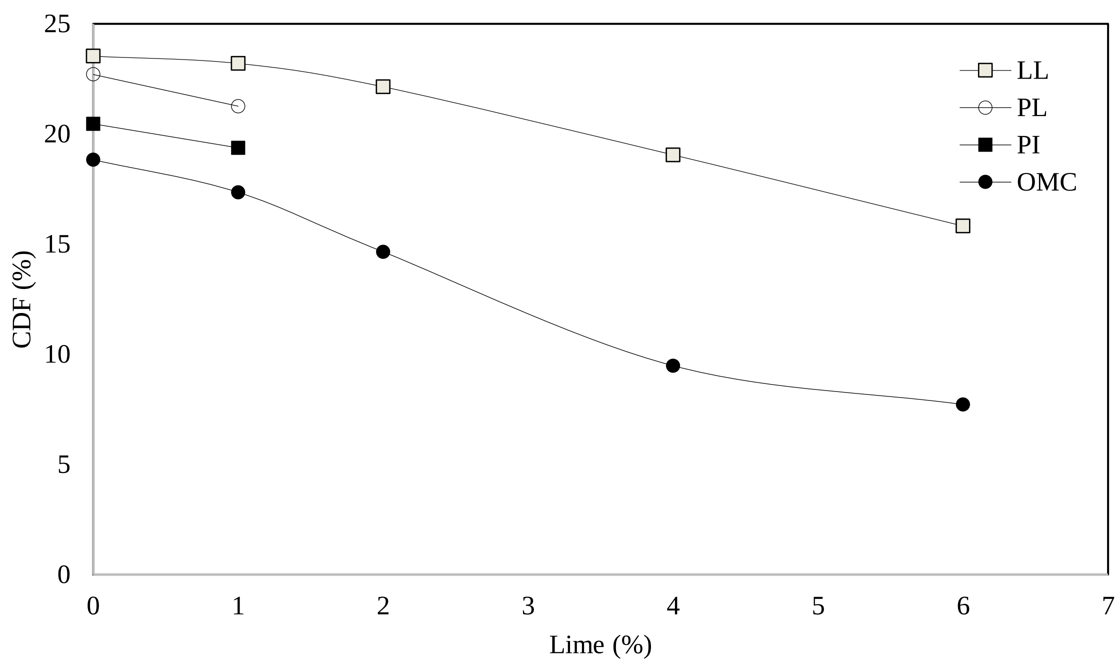


Figure 7.1. Effect of lime content on CDF

Table 7.1 summarises the values of *CDF* (%).

For example, *CDF* (%) at *LL* decreased from 23.83% to 15.82% when the lime content increased from 0% to 6%, exhibiting a reduction of 33.61%. At *OMC*, *CDF* decreased from 18.83% to 7.72% when the lime content increased from 0% to 6%, exhibiting a reduction of 59.00%. These are considerable reductions in *CDF* (%) establishing lime as an effective stabilising agent. A near 60% reduction in *CDF* (%) at *OMC* indicates that, if the expansive soil is stabilised with 6% lime at a water content corresponding to *OMC*, a considerably reduced crack pattern and crack widths can be expected in dry seasons. This increases the strength and intactness of the compacted blends.

Figure 7.2 depicts the variation of *CDF* (%) with cement content. The data pertain to the moulding water contents of *LL*, *PL*, *PI* and *OMC*. *CDF* (%) decreased significantly with cement content. This was true for all the moulding water contents. As cement is also an effective pozzolanic material, its addition to an expansive soil reduces both swelling and shrinkage. As non-swelling cement content increases in the clay-cement blends, the volumetric changes decrease reducing swelling and shrinkage. Hence, *CDF* (%) decreased reflecting reduced crack width and crack volume.

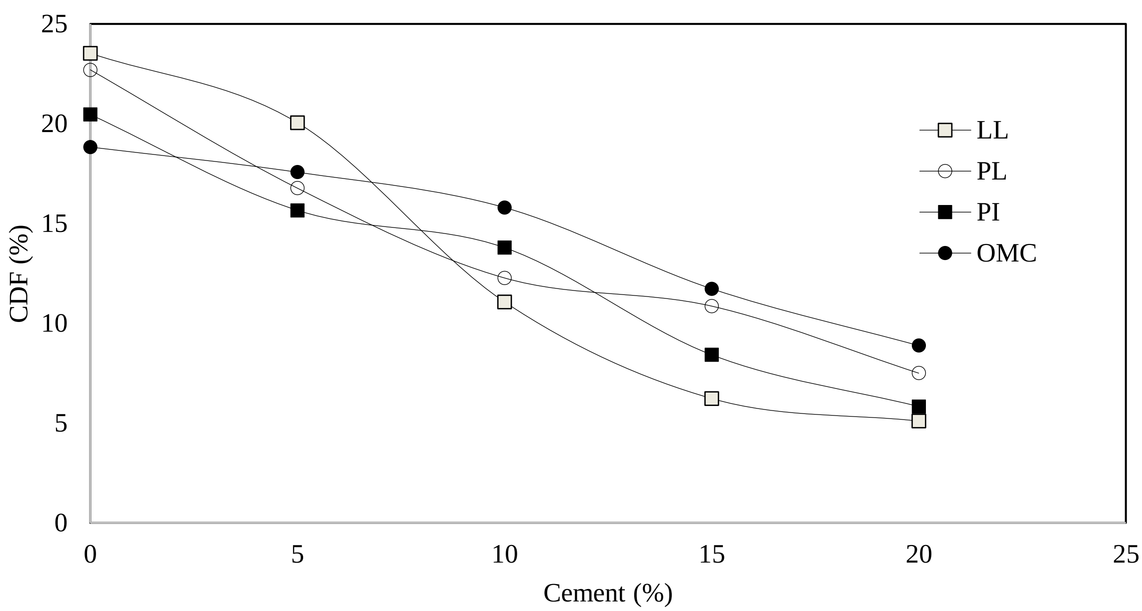


Figure 7.2. Influence of cement on CDF

For example, *CDF* (%) at *LL* decreased from 23.53% to 5.1% when the cement content increased from 0% to 20%, indicating a reduction of 78.32%. A similar reduction in *CDF* (%) was observed at *PL*, *PI* and *OMC* also. The reduction in *CDF* (%) at 20% cement was respectively 67%, 71.5% and 53% at *PL*, *PI* and *OMC*. These are significant reductions, higher than those observed in the case of lime, which establish the efficacy of cement as an excellent stabilising agent of expansive clays. The data show that, if the expansive clay is stabilised with a cement content ranging from 10% to 20% at any water content corresponding to either *LL* or *PL* or *OMC*, much reduced crack patterns, crack widths and crack volumes would result in dry seasons.

Table 7.1. Summary of *CDF* (%) Values

S. No.	Description	CDF (%) at LL	CDF (%) at PL	CDF (%) at PI	CDF (%) at OMC
1	Expansive clay	23.53	22.7	20.46	18.83
2	1% lime	23.20	21.26	19.37	17.35
3	2% lime	22.14	-	-	14.65
4	4% lime	19.05	-	-	9.48
5	6% lime	15.82	-	-	7.72
6	5% cement	20.05	16.77	15.66	17.57
7	10% cement	11.07	12.27	13.79	15.8
8	15% cement	6.23	10.86	8.42	11.73
9	20% cement	5.10	7.5	5.83	8.88
10	5% fly ash	22.73	17.17	16.87	17.04
11	10% fly ash	21.72	15.73	15.59	16.39
12	15% fly ash	20.20	14.46	14.70	14.65
13	20% fly ash	18.50	13.26	12.46	13.42
14	5% GGBS	22.50	16.25	16.59	16.16
15	10% GGBS	19.40	15.88	15.24	15.26
16	15% GGBS	17.50	15.25	14.04	14.14
17	20% GGBS	15.20	14.75	13.29	12.74

Figures 7.3 and 7.4 show the variation of *CDF* (%) respectively with fly ash content and GGBS content. As in the cases of lime and cement, *CDF* (%) decreased continuously with fly ash content and GGBS content also, indicating that fly ash and GGBS also were influential in reducing crack widths and crack volumes. However, these were not as effective as lime and cement. Table 7.1 shows that, at 20% fly ash and at 20% GGBS, the reduction in *CDF* (%) at a given moulding water content was close to each other. Another interesting feature which Figures 7.3 and 7.4 indicated was a close cluster of lines of variation at moulding water contents lower than *LL*, namely, *PL*, *PI* and *OMC*.

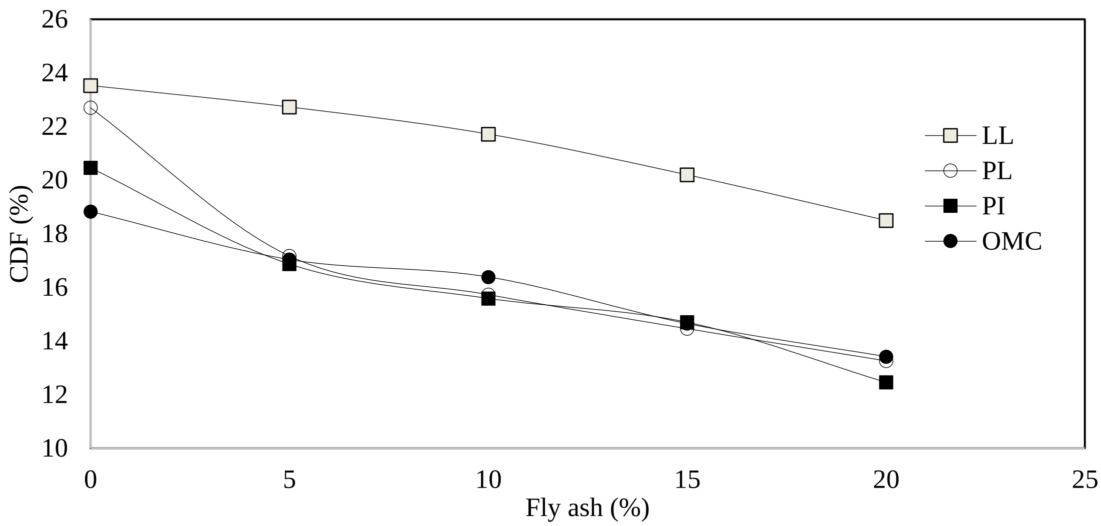


Figure 7.3. Influence of fly ash on CDF

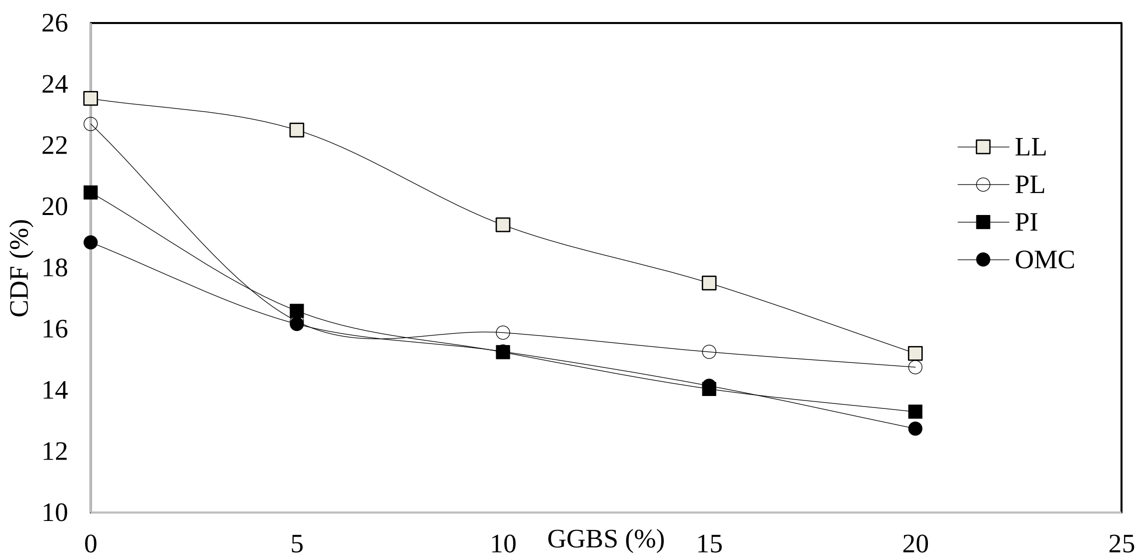


Figure 7.4. Influence of GGBS on CDF

Figure 7.5 compares the influence of different additives - lime, cement, fly ash and GGBS – on *CDF* (%) at *LL*. Other moulding water contents also gave a similar picture. The figure depicts the variation of *CDF* (%) with additive content. The data pertain only to *LL*. The figure shows that *CDF* (%) decreased with increasing lime content and cement content. The reduction in *CDF* (%) was not that significant in the cases of fly ash and GGBS. At 6% lime and 20% cement, *CDF* (%) decreased respectively by 33.6% and 78.3%, whereas *CDF* (%) decreased by 21.37% and 35.40% respectively at 20 % fly ash and 20% GGBS. These data established that lime and cement are the most effective stabilising agents of expansive clays.

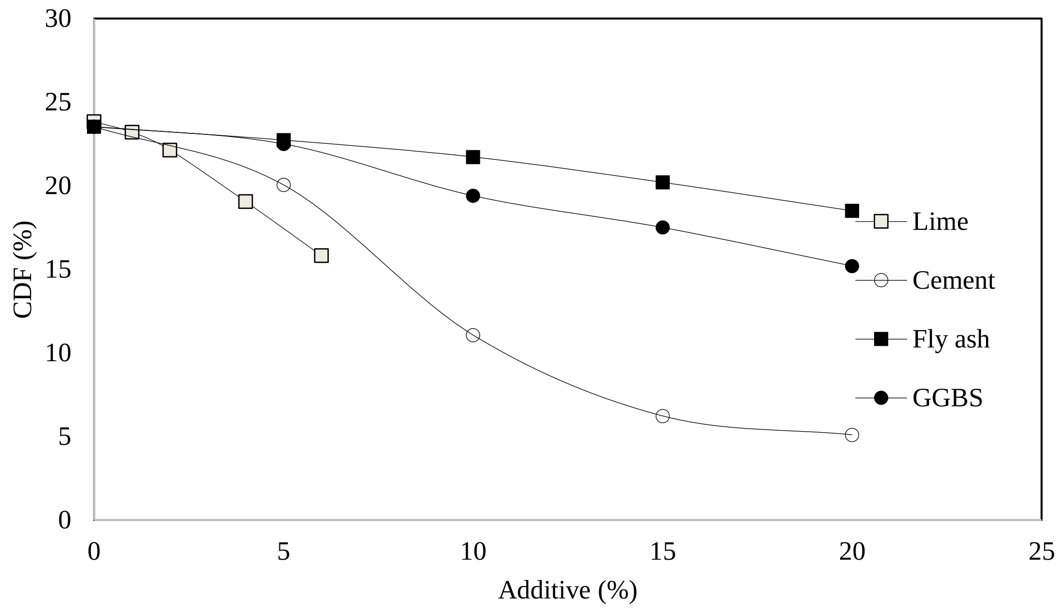


Figure 7.5. Effect of different additives on CDF at liquid limit

7.4.2. Influence of additive contents and moulding water contents on CIF:

Figure 7.6 shows the variation of *CIF* (%) with lime content for the moulding water contents of *LL*, *PL*, *PI* and *OMC*. *CIF* (%) decreased significantly with lime content. This was true for all the moulding water contents. As mentioned before, beyond 1% lime, *PL* and *PI* could not be determined. As lime content increases, the volumetric changes in the lime-clay blends decrease. Hence, swelling and shrinkage decrease with increasing lime content. As shrinkage decreases, the crack patterns, the crack widths and the crack volumes decrease thereby reducing *CIF* (%) significantly. This could be attributed to (a) increase in the non-expansive lime content and (b) the pozzolanic reactions bringing down the volumetric changes. Table 7.2 summarises the values of *CIF* (%).

CIF (%) at *LL* decreased from 10.07% to 1.74% when the amount of lime increased from 0% to 6%, reflecting a reduction of 82.7%. At *OMC*, *CIF* (%) decreased from 4.65% to 0.95% when the lime content increased from 0% to 6%, showing a reduction of 80.2%. These considerable reductions in *CIF* (%) establish lime as an effective additive. A reduction of 80% in *CIF* (%) at *OMC* indicates that, if the expansive soil is stabilised with 6% lime at a water content corresponding to *OMC*, a considerably reduced crack pattern and crack widths can be expected in dry seasons.

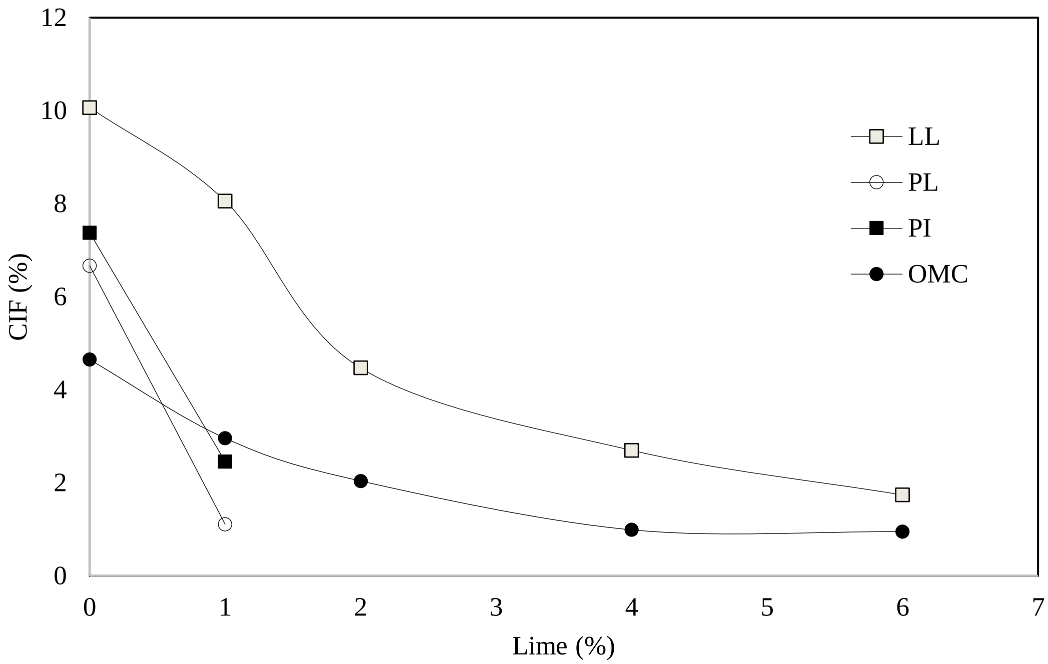


Figure 7.6. Influence of lime on CIF

Figure 7.7 shows the variation of *CIF* (%) with cement content. The data pertain to the moulding water contents of *LL*, *PL*, *PI* and *OMC*. *CIF* (%) decreased significantly with cement content. This was true for all the moulding water contents (see Figure 7.7). As cement is also an effective pozzolanic material, its addition to an expansive soil reduces both swelling and shrinkage. As non-swelling cement content increases in the clay-cement blends, volumetric changes decrease reducing swelling and shrinkage. Hence, *CIF* (%) decreased reflecting reduced crack width and crack volume.

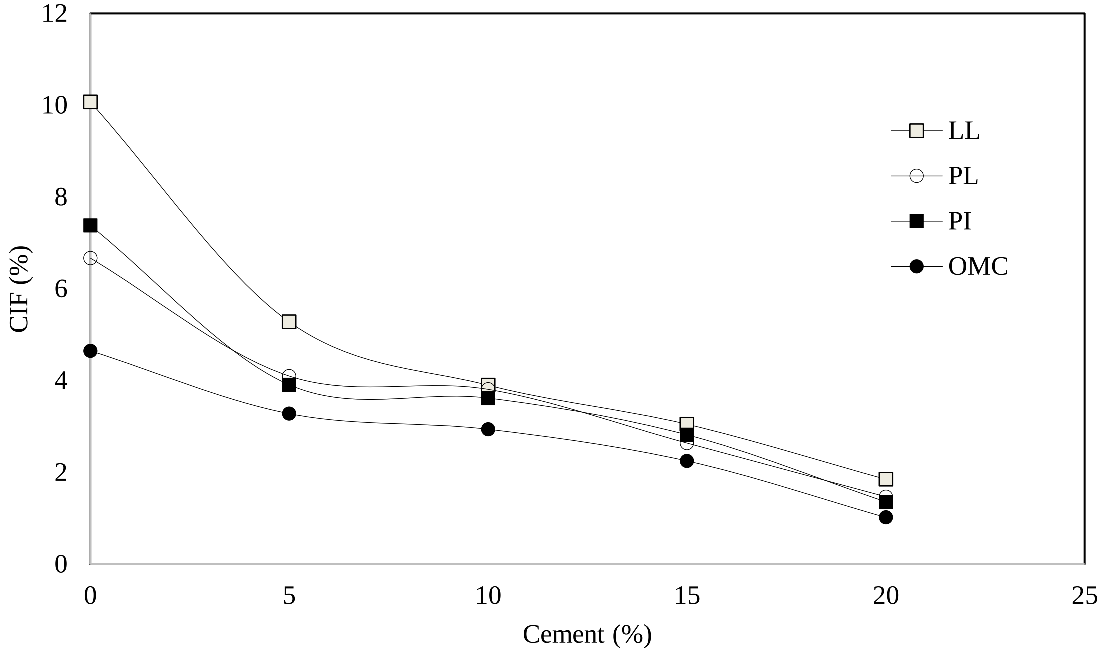


Figure 7.7. Influence of cement on CIF

For example, *CIF* (%) at *LL* decreased from 10.07% to 1.85% when the cement content increased from 0% to 20%, indicating a reduction of 81.6%. A similar reduction in *CIF* (%) was observed at *PL*, *PI* and *OMC* also (see Table 7.2). The reduction in *CIF* (%) at 20% cement was respectively 77.9%, 81.5% and 78% at *PL*, *PI* and *OMC*. These are notable reductions in *CIF* (%), which establish the efficacy of cement as an excellent stabilising agent of expansive clays that can reduce their shrinkage significantly. The data show that, if the expansive clay is stabilised with a cement content ranging from 10% to 20% at any water content corresponding to either *LL* or *PL* or *OMC*, much reduced crack patterns, crack widths and crack volumes would result in dry seasons.

When fly ash and GGBS were used as stabilising agents, then also *CIF* (%) decreased with the additive contents in a discernible manner. Figure 7.8 shows the variation of *CIF* (%) with fly ash content for all the moulding water contents. *CIF* (%) decreased continuously with increasing fly ash content. This was true for all the moulding water contents though the reduction was quite high at *LL*. As non-expansive fly ash particles were added to the expansive clay, shrinkage decreased, resulting in reduced crack widths and crack volumes. CIF (%) at *LL* decreased from 10.07% to 3.05% when the fly ash content increased from 0% to 20%, reflecting a reduction of

69.7%. Similarly, at *PL*, *PI* and *OMC*, the reduction in *CIF* (%) was respectively 57.7%, 67.4% and 54.8% when the amount of fly ash increased from 0% to 20%.

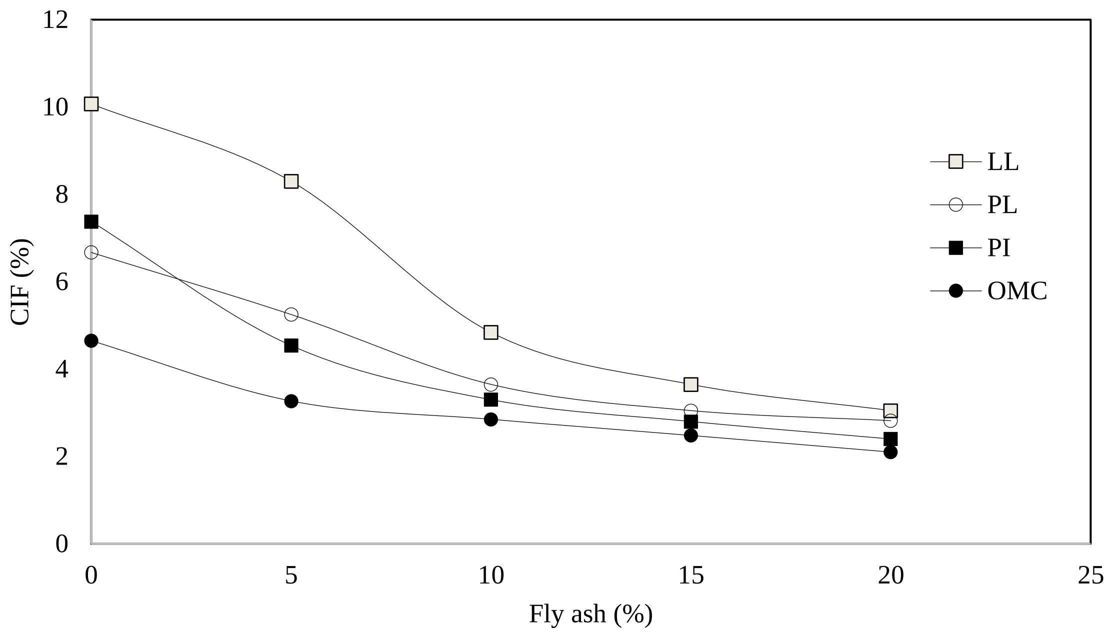


Figure 7.8. Influence of fly ash on CIF

Figure 7.9 depicts the variation of *CIF* (%) with GGBS content. The data pertain to the moulding water contents of *LL*, *PL*, *PI* and *OMC*. *CIF* (%) decreased continuously with increasing GGBS content in a notable manner. This was true for all the moulding water contents. Addition of non-expansive GGBS particles to the expansive clay resulted in the reduction of shrinkage leading to reduced crack widths. For example, *CIF* (%) at *LL* decreased from 10.07% to 2.85% when the GGBS amount increased from 0% to 20%, reflecting a reduction of 71.9%. Similarly, at *PL*, *PI* and *OMC*, the reduction in *CIF* (%) was respectively 62.5%, 67.4% and 54.8% when the GGBS amount increased from 0% to 20%.

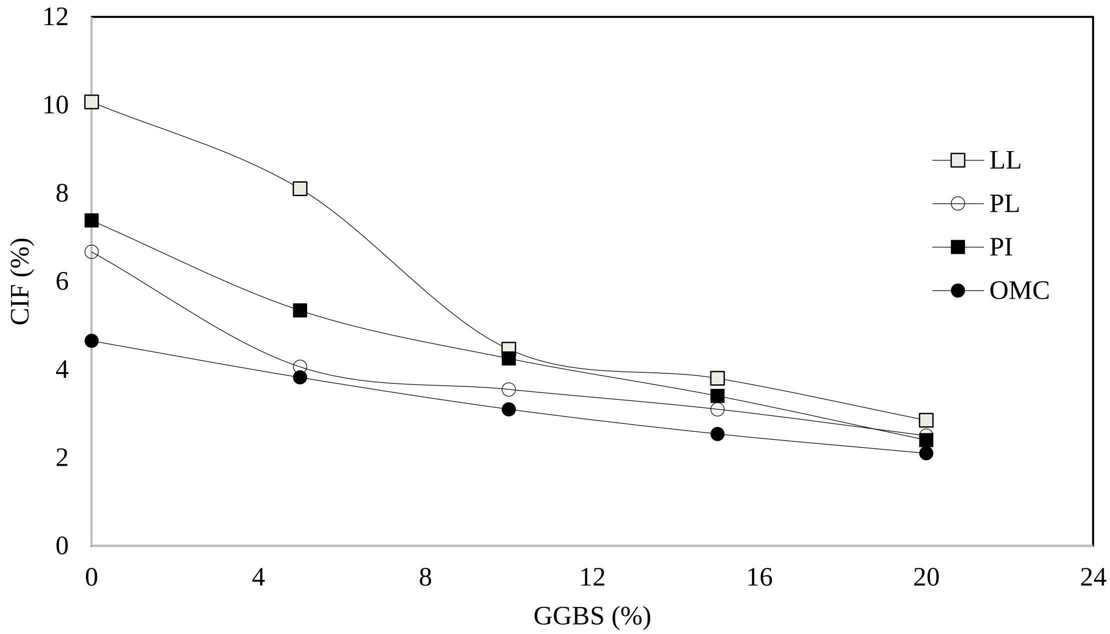


Figure 7.9. Influence of GGBS on CIF

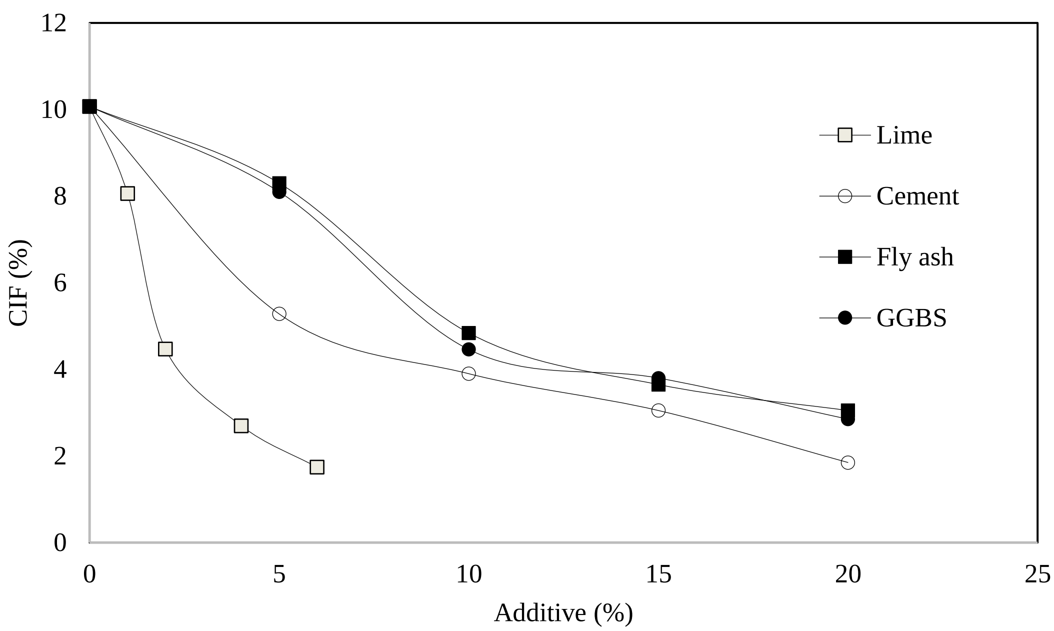


Figure 7.10. Effect of different additives on CIF at liquid limit

Figure 7.10 compares the influence of lime, cement, fly ash and GGBS on *CIF* (%). The data shown in the figure pertain to liquid limit as the moulding water content. Other moulding water contents also resulted in similar variations of *CIF* (%) with additive content. The figure shows that, *CIF* (%) decreased significantly with additive content. This was true for all the additives used in the test programme. However, the data established that, when lime was the additive, there was a drastic reduction in *CIF* (%). Even at small amounts of lime content, *CIF* (%) decreased enormously showing that shrinkage was effectively controlled by lime content. The data show that cement was also an additive quite efficacious in reducing shrinkage and crack widths, followed by fly ash and GGBS.

Table 7.2. Summary of *CIF* (%) Values

S. No.	Description	CIF (%) at LL	CIF (%) at PL	CIF (%) at PI	CIF (%) at OMC
1	Expansive clay	10.07	6.67	7.38	4.65
2	1% lime	8.06	1.11	2.46	2.96
3	2% lime	4.47	-	-	2.04
4	4% lime	2.70	-	-	0.99
5	6% lime	1.74	-	-	0.95
6	5% cement	5.28	4.10	3.91	3.28
7	10% cement	3.90	3.81	3.62	2.94
8	15% cement	3.05	2.64	2.82	2.25
9	20% cement	1.85	1.47	1.36	1.02
10	5% fly ash	8.30	5.25	4.54	3.27
11	10% fly ash	4.84	3.65	3.30	2.85
12	15% fly ash	3.65	3.05	2.80	2.48
13	20% fly ash	3.05	2.82	2.40	2.10
14	5% GGBS	8.10	4.06	5.34	3.82
15	10% GGBS	4.46	3.55	4.25	3.10
16	15% GGBS	3.80	3.10	3.40	2.54
17	20% GGBS	2.85	2.50	2.40	2.10

7.5. CONCLUSIONS:

The following conclusions can be arrived at from the investigation:

1. The amount of shrinkage, shrinkage crack pattern and shrinkage crack widths decreased considerably with increasing amounts of the additives used in the experimental programme, namely, lime, cement, fly ash and GGBS. This was found to be true for all the moulding water contents at which the test samples were prepared, namely, *LL*, *PL*, *PI* and *OMC*.

2. *CDF* and *CIF* of the samples also indicated reduced amounts of shrinkage and reduced crack widths.

3. Both *CDF* and *CIF* significantly decreased with increasing additive content (for all the additives) at all the moulding water contents (*LL*, *PL*, *PI* and *OMC*).

4. At a given moulding water content, lime and cement were found to have the highest impact on *CDF* and *CIF*; and the impact of fly ash and GGBS on *CDF* and *CIF* at a given moulding water content was found to be less.

5. The experimental data indicate that, if the expansive clay is stabilised with the optimum contents of these additives at any of the moulding water contents studied in this investigation, much reduced crack patterns and crack widths could result in dry seasons.

CHAPTER - 8

SUMMARY OF CONCLUSIONS

A detailed experimentation was conducted on chemical stabilisation of an expansive clay. Lime, cement, fly ash and GGBS were used as the additives. *FSI*, *LL* and *PI*, compaction behaviour, strength and hydraulic conductivity, *CBR* and load-settlement behaviour, rate of heave and amount of heave, swell-shrink behaviour and propagation of shrinkage cracks were studied to assess the influence of the above additives.

The following conclusions can be drawn from the fore-going experimental investigation:

1. Addition of lime, cement, fly ash and GGBS reduced *LL* and *PI* of the expansive clay, thus reducing the plasticity characteristics. At 20% additive content, cement and GGBS resulted in the highest reduction in *LL* and *PI* among all the additives used in this investigation. But, lime was observed to be the most effective pozzolanic chemical additive as an appreciable reduction in *LL* and *PI* occurred at 4% lime itself. Hence, these chemical additives can be advantageously used for stabilising expansive clays, for decreasing their plasticity and for improving their workability.

2. Free swell index (*FSI*) of the expansive clay also decreased significantly upon the addition of lime, cement, fly ash and *GGBS*. Excepting in the case of *GGBS*, *FSI* decreased, more or less, by the same amount at the highest additive content irrespective of the type of the additive, which again indicated that lime can be thought of as the most effective pozzolanic chemical additive.

3. *MDD* and *OMC* of the clay-additive mixes decreased with increasing additive content, showing that the chemical additives were instrumental in stabilising the expansive clay.

4. The hydraulic conductivity (*k*, cm/sec) of the expansive clay-additive blends, found out at their respective *OMC* and *MDD* by performing the variable head permeameter tests, continuously decreased with increasing additive content excepting in the case of lime. The hydraulic conductivity decreased with increasing additive content as the *MDD* of the clay-additive blends increased. However, in the case of lime-clay blends, the hydraulic

conductivity decreased up to 2% lime and thereafter, it increased. This could be attributed to the possible heat release at higher lime contents and a consequent absorption of more water.

5. For a given curing period, peak stress or failure stress increased and peak strain or failure strain decreased with increasing additive content. Similarly, at a given additive content, peak stress or failure stress increased and peak strain or failure strain decreased with increasing curing period.

6. The soaked *CBR* of the clay-additive blends, determined at their respective *OMC* and *MDD*, increased significantly with increasing additive content. The highest *CBR* values were observed in the cases of lime-clay blends and cement-clay blends though the percentage increase in the *CBR* of the fly ash-clay blends and the GGBS-clay blends was also quite high. This suggested that the chemical additives studied in this investigation proved quite effective in stabilising the expansive clay. Therefore, they can be used in strengthening the expansive clay subgrades.

7. The compressive load response of the clay beds showed betterment with increasing additive content in the as-compacted condition. The stress for a settlement of 0.5mm increased with the additive content. While in the case of cement this stress continuously increased with cement content, in the cases of lime, fly ash and GGBS, it increased respectively up to 4%, 10% and 10%, and decreased thereafter.

8. The stress for a settlement of 0.5mm in the saturated condition also increased with additive content in the cases of fly ash and GGBS. In the case of the lime-clay blends, this stress increased up to 4% lime. At 6% lime, the test bed could not be compressed up to the settlement of 0.5mm. The test beds of the cement-clay blends also could not be compressed up to 0.5mm at cement contents higher than 5%.

9. The Young's modulus (E_s) of the chemically stabilised expansive clay beds, determined in the as-compacted condition, increased continuously with increasing cement content. However, E_s increased only up to 4% lime, 10% fly ash and 10% GGBS respectively in the cases of lime-clay, fly ash-clay and GGBS-clay blends, and decreased thereafter.

10. The Young's modulus (E_s) of the chemically stabilised expansive clay beds, determined in the saturated condition, increased with the additive content for fly ash and GGBS. In the case of the lime-clay blends, E_s increased up to 4% lime. At 6% lime, E_s could not be determined in the saturated condition. E_s of the cement-clay blends also could not be determined at cement contents higher than 5% in the saturated condition.

11. The equilibrium heave of the clay beds decreased with increasing additive contents. Of all the additives, lime and cement had the most prominent effect on heave. Heave decreased respectively by 98% and 99% at 6% lime and 20% cement; and it decreased respectively by 53% and 74% at 20% fly ash and 20% GGBS.

12. In all the swell-shrink cycles, the chemically stabilised expansive clay beds showed reduced swelling and shrinkage compared to the plain or the unblended expansive clay bed. The expansive clay beds stabilised by 6% lime and 20% cement exhibited the lowest range of swelling and shrinkage. This was found to be true in all the swell-shrink cycles. Of these two chemicals again, 20% cement resulted in the lowest amount of swelling and shrinkage.

13. As lime and cement are the most effective pozzolanic additives, swelling and shrinkage of the expansive clay bed were effectively reduced when stabilised by them. When stabilised by 20% cement, the expansive clay bed swelled and shrank by negligible amounts of 0.23 mm and 0.04 mm respectively in the 1st cycle itself. In the subsequent cycles, the swelling and shrinkage of the clay bed stabilised by 20% cement was 0 mm. 6% lime was also quite effective in controlling swelling and shrinkage to negligible amounts.

14. As the swell-shrink cycles progressed, all the clay beds experienced fatigue and exhibited lower amounts of swelling and shrinkage. Swelling and shrinkage decreased due to fatigue caused by number of swell-shrink cycles. The reduction in swelling and shrinkage with increasing number of swell-shrink cycles was more predominant in the chemically stabilised expansive clay beds than in the plain expansive clay bed. 20% cement caused 100% reduction in swelling and shrinkage as the clay bed passed from Cycle - 1 to Cycle

- 5; and 6% lime caused a reduction of 93% in swelling and 90% in shrinkage as the clay bed passed from Cycle - 1 to Cycle – 5.

15. The process of shrinkage caused polygonal and linear shrinkage cracks in the clay beds. The width of the largest polygonal shrinkage crack was found to be the greatest in the plain clay bed. However, it decreased as the clay bed was stabilised by various chemicals. This was found to be true in all the cycles. The polygonal shrinkage crack width in a given clay bed decreased with increasing number of swell-shrink cycles. Thus, the combined effect of chemical stabilisation and the progression in swell-shrink cycles was to considerably reduce the polygonal shrinkage cracks.

16. Linear shrinkage was also found to be the maximum in the plain clay bed, and it decreased as the clay bed was stabilised by various chemicals. This was found to be true in all the cycles. For a given clay bed, plain or chemically stabilised, linear shrinkage decreased with increasing number of swell-shrink cycles. Linear shrinkage was also considerably reduced by the combined effect of chemical stabilisation and advancement of swell-shrink cycles.

17. The amount of shrinkage, shrinkage crack pattern and shrinkage crack widths decreased considerably with increasing amounts of the additives used in the experimental programme, namely, lime, cement, fly ash and GGBS. This was found to be true for all the moulding water contents at which the test samples were prepared, namely, *LL*, *PL*, *PI* and *OMC*.

18. *CDF* and *CIF* of the samples also indicated reduced amounts of shrinkage and reduced crack widths.

19. Both CDF and CIF significantly decreased with increasing additive content (for all the additives) at all the moulding water contents (*LL, PL, PI* and *OMC*).

20. At a given moulding water content, lime and cement were found to have the highest impact on *CDF* and *CIF*; and the impact of fly ash and GGBS on *CDF* and *CIF* at a given moulding water content was found to be less.

21. The experimental data indicate that, if the expansive clay is stabilised with the optimum contents of these additives at any of the moulding water contents studied in this investigation, much reduced crack patterns and crack widths could result in dry seasons.

SCOPE FOR FURTHER WORK

- This research is a study on swell-shrinkage behaviour of chemically stabilised expansive clay beds of laboratory scale considering various dosages of lime, cement, fly ash and GGBS. Further, various other additives can also be tried either individually or as combination.

- One-dimensional swell-consolidation tests can be another important aspect of further work to study swell potential and swelling pressure and other important compressibility data.

- In this work, swell-shrink data were generated without any load or surcharge pressure on the clay beds. In future research, the effect of surcharge pressure on swell-shrink behavior could be studied.

- In this work, quantification of desiccation cracks of chemically stabilised expansive clays was analysed using MATLAB. The same work can be carried out for different soils having different clay minerology in order to predict the soil behaviour by Artificial Intelligence.

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