PERFORMANCE ENHANCEMENT OF EXPANSIVE SOIL BY APPLICATION OF FLY ASH AND LIME

A THESIS

submitted by

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of

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Dedicated to:

Baba, my saviour

and

All my 'Guru's

CERTIFICATE

This is to certify that the thesis entitled **"Performance Enhancement of Expansive Soil by Application of Fly Ash and Lime"** submitted by Mr. Shailen Deka to the Indian Institute of Technology Guwahati, for the award of the degree of Doctor of Philosophy in Civil Engineering is a record of bonafide research work carried out by him under our supervision and guidance. The thesis work, in our opinion, has reached the requisite standard fulfilling the requirement for the degree of Doctor of Philosophy.

The results contained in this thesis have not been submitted in part or full to any other University or Institute for award of any degree or diploma.

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STATEMENT

I do hereby declare that the matter embodied in this thesis is the result of investigations carried out by me in the Department of Civil Engineering, Indian Institute of Technology Guwahati, Guwahati, Assam, India.

In keeping with the general practice of reporting scientific observations, due acknowledgements have been made wherever the work described is based on the findings of other investigators.

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ABSTRACT

Expansive soils can imbibe large amount of water and undergo high volume changes, causing seasonal heave and shrinkage according to the availability of water. This often causes distress and failure of structures founded on such soils. Methods such as preloading, moisture control, replacement of affected soil, and additives have been used to deal with the swelling problems associated with these soils. Literature reveals that studies were conducted to modify the behavior of expansive soils using additives such as lime, cement, fly ash and other industrial wastes. However, not many studies have been performed on the combined application of lime and fly ash, particularly the non-self cementing fly ash, for treatment of expansive soils. In this research work, attempt has been made to study the geotechnical characteristics of expansive soil mixed with different composition of lime and fly ash. The main focus of this study is to find meaningful application of waste fly ash that does not qualify in construction industry and to use lime as an optimal modifier of properties wherever required.

The geotechnical characteristics dealt in this study include plasticity, compaction, consolidation, swelling and strength. The variation of these properties due to combined addition of fly ash and lime has been systematically investigated. Here plasticity characteristics depict the modification reaction occurring in the soil mix where as the variation in strength characteristics indicate modification and solidification reaction. Therefore, effect of aging on these two parameters has been extensively studied. The undesirable effect of higher percentage of lime, over and above reported in the literature has been studied. It is noted that there is considerable influence of combined addition of fly ash and lime and aging on geotechnical properties of expansive soil. Some of the properties like plasticity and strength are influenced much by higher percentage of lime.

The performance enhancement of expansive soil due to combined addition of lime and fly ash is clearly discussed and quantified in this study. Based on the results, suggestions have been made to maximize the utility of fly ash wherever possible. However, there are some chemical characteristics that need to be extensively investigated further for better explanation of certain trends observed in this study.

Keywords: expansive soil, fly ash, lime treatment of soil, soil stabilization, soil modification, plasticity, compaction, swell, consolidation, strength.

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ABBREVIATIONS

BET	Brunauer, Emmett and Teller theory of surface adsorption of gas
CBR	California bearing ratio
CEC	Cation exchange capacity
CH	Clay of high plasticity (as per Unified Soil Classification System)
CL	Clay of low plasticity (as per Unified Soil Classification System)
CAH	Calcium aluminate hydrate (as per cement chemists' notations)
CSH	Calcium silicate hydrate (as per cement chemists' notations)
CSH1	Calcium silicate hydrate type I (as per cement chemists' notations)
DDL	Diffuse double layer
EDX	Energy dispersive X-ray
ES	test expansive soil
FA	test fly ash
FSI	Free swell index
F0	Test ES-FA mix with 100% ES and 0% fly ash
F20	Test ES-FA mix with 80% ES and 20% fly ash
F40	Test ES-FA mix with 60% ES and 40% fly ash
F60	Test ES-FA mix with 40% ES and 60% fly ash
F80	Test ES-FA mix with 20% ES and 80% fly ash
F100	Test ES-FA mix with 0% ES and 100% fly ash
ICL	Initial consumption of lime
kPa	kilo Pascal
LL	Liquid limit
MH	Elastic silt (as per Unified Soil Classification System)
ML	Silt (as per Unified Soil Classification System)
MDD	Maximum dry density
OMC	Optimum moisture content

PL	Plastic limit
PI	Plasticity Index
SEM	Scanning electron microscopy
Sp.Gr.	Specific gravity
SSA	Specific surface area
UCS	Unconfined compressive strength
XRD	X-Ray Diffraction
ZAV	Zero air void

NOTATIONS

c_c	Compression index
D_{10}	Effective size, 10% finer size
D_{50}	50% finer size
D_{90}	90% finer size
e	Void ratio
m_v	Coefficient of volume compressibility
p	Consolidation pressure
p_s	Swell pressure
γ_d	Dry density
γ_{dmax}	Maximum dry density
γ_{dPL}	Dry density at plastic limit
δ	Settlement
ε	Swell percentage
ε_{max}	Maximum swell percentage

CHAPTER 1

INTRODUCTION

The uncertainties related to the civil engineering structures founded on expansive soils have motivated several researchers to study this topic in detail (Wild et al. 1998; Boardman et al. 2001). The important challenges are the excessive volume change during wetting and drying cycles and low strength of expansive soils. Different measures have been proposed and methodologies adopted for overcoming the problems associated with expansive soil. One of the methods is to strengthen foundation to minimize the effect of expansiveness on the former, which include belled piers, granular pile anchors, sand cushion technique etc. (Phanikumar 2009).

Another effective method for expansive soil stabilization is the use of additives that helps to minimize the volume change due to swelling. The different additives used for stabilizing expansive soil include lime, cement, cohesionless material like sand and fly ash. Among these, lime treatment has gained a lot of popularity due to its capability to reduce swelling. However, a few studies advocate against the use of lime in specific type of expansive soil comprising of sulfate due to the undesirable reaction enhancing heave (Hunter 1988). Fly ash, which is a waste product from thermal power plants have an excellent potential for overcoming the issues related to swelling by replacing fraction of expansive soil. The minimal amount of calcium present in fly ash can also induce desirable stabilization reaction in expansive soil with time. In addition, the stabilization can be improved by addition of optimal lime percentage to fly ash modified expansive soil. It is noted that there are not many studies that deals with the physical and geotechnical characterization of expansive soil modified with both fly ash and lime. The present research work aims to systematically quantify the performance enhancement of expansive soil due to the combined addition of fly ash and lime.

Burning of coal for thermal power generation produces fly ash as the byproduct, safe disposal of which involves large cost and land space. With ever increasing demand of power, worldwide, huge quantities of fly ash are being piled up. Researchers worldwide are in constant search for sustainable solutions for bulk utilization of this otherwise waste product. The basic objective of the present study is to explore the possibility of maximizing the utility of fly ash, in the context of expansive soils. In order to achieve this objective a comprehensive experimental investigation has been carried out. The parameters studied are; plasticity characteristics, compaction behaviour, consolidation and swelling responses, strength development. The various aspects of this study are presented in different chapters. The broad contents of which are as follows.

Review of the current literatures pertaining to the present research work is presented in Chapter 2. Chapter 3 presents the details of experiments carried out, materials used, test setup and the test procedure adopted. The obtained results are presented and discussed in Chapters 4-8. Chapter 4 examines the influence of lime and fly ash over the plasticity characteristics of the expansive soil. Chapter 5 deals with the compaction behaviour of the fly ash-lime amended expansive soil. Chapter 6 and Chapter 7 have examined the swelling and consolidation characteristics of the same. Results of unconfined compression strength tests are presented in Chapter 8. Finally, conclusions drawn from the present study are summarized in Chapter 9.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

Expansive soils are residually derived soil that is abundantly found over the vast stretch of semi-arid regions across the world. Due to the presence of the montmorillonite mineral, these soils exhibit high affinity for water and therefore experience large moisture related volume changes (i.e. swell-shrink) during wet and dry seasons. Stabilization of such soils through chemical modifications is being extensively used worldwide. Such a treatment apart from minimizing the swell-shrink potential of the soil substantially improves its strength and deformation characteristics. Among the various chemicals being used for stabilization of expansive soils, lime [$Ca(OH)_2$] is found to be the most effective one (Bell, 1993).

The amount of strength increase in a soil, due to lime treatment, is highly dependent on the pozzolanic characteristics of the soil. When the pozzolanic content in the soil is less, very little improvement in the strength is obtained by adding the lime. To overcome this problem, pozzolanic materials are to be added to the soil. Fly ash is one such material that is by far the most widely used pozzolan. This is partly due to the high percentage of silica present in it apart from being abundantly produced from the thermal power plants as a waste material. On an average each thermal power plant produces about 500000 to 1000000 tonnes of fly ash every year, the safe disposal of which demands more expenditure and land area. Therefore researchers are in constant search for solutions which permit fly ash utility in bulk. Geotechnical engineering is one such avenue where large quantities of fly ash can be utilized for construction of structures such as embankments, foundation beds, highway and railway subgrade, etc.

The following sections deal with the review of different works related to the property enhancement of soil by addition of lime and fly ash. The first part of this chapter deals with the discussion on lime treatment of soil and its implications. The second part deals with the application of fly ash for soil stabilization.

2.2 LIME TREATMENT OF EXPANSIVE SOIL

2.2.1 Mechanism

Use of lime for soil improvement in the modern age started in the beginning of 20^{th} century (Johnson, 1948; Bell, 1996). The studies gained momentum in the 1950s and 1960s with the works of Goldberg and Klein (1952), Clare and Cruchley (1957), Davidson et al. (1960), Eades and Grim (1960), Hilt and Davidson (1960), Herin and Mitchell (1961), Dumbleton (1962), Lambe (1962), Croft (1964), Thompson (1966), etc. and further studies are still continuing till date.

The actions of lime that lead to improvement of soils are, cation exchange, flocculation and agglomeration, carbonation and pozzolanic reaction (Nelson and Miller, 1992; Arabi and Wild, 1989; Khattab et al., 2001). These reactions contribute to physical, mineralogical and microstructural changes in the treated soils (Khattab et al., 2007) leading to reduction in plasticity and swell-shrink potential, improvement in strength and stability (Little, 1999).

The above mechanisms of lime-soil reactions can be classified into two distinct groups – modification and solidification. Low percentage of lime addition results in only modification of expansive soil through cation exchange, and flocculation. A higher percentage of lime results in both modification and solidification (Boardman et al. 2001). Modification results only in flocculation and cation exchange reactions (Salehi and Sivakugan, 2009) whereas solidification results in pozzolanic reaction. Modification may be a reversible process but solidification results in irreversible change in clay characteristics.

Most of the clay minerals react with lime. While lime has a quick and significant effect in clay soils containing montmorillonite, it has less effect on kaolinitic clay soils (Bell, 1996). This is attributed to the high cation exchange capability of the former compared to the latter. Petry and Little (2002) reported that the initial lime-soil reactions occur within about 1 hour of mixing. When lime is added to a clay soil, it must first satisfy the affinity of the soil for lime. This affinity results from the adsorption of Ca^{2+} ions by clay minerals. Lime is not available for pozzolanic reactions until this affinity is satisfied. This limit of lime adsorption is referred to as lime fixation (Hilt and Davidson, 1960; Bell, 1996). This is the optimum quantity of lime needed for maximum modification of the soil and is normally between 1 to 3% by weight. Beyond this, lime is available for structural modification of soil leading to strength gain.

The highly alkaline environment ($pH \geq 12.4$) produced by the addition of lime promotes slow dissolution of alumino-silicates, which are then precipitated as hydrated cementitious reaction products. Pozzolans are finely divided silicaceous and aluminous material which, in the presence of water and calcium hydroxide, will form cemented products such as calcium-silicate-hydrates or calcium-aluminate-silicate-hydrates. Thus clay, which is a source of silica and alumina, is by this definition, a pozzolan. During the pozzolanic reactions, calcium silicate hydrates and calcium aluminate hydrates are formed as the calcium from the lime reacts with the aluminates and silicates dissolved from the clay mineral surface. These reaction products bond adjacent soil particles together and as curing occurs they strengthen the soil (Eades and Grim, 1960; Esrig 1999; Petry and Little, 2002; Khattab et al., 2007). This reaction can begin quickly, however, the full term pozzolanic reaction can continue for a very long period of time spanning even years, as long as enough lime is present and the pH remains above 10 (Littlle, 1999).

For maximum reactivity, the pH value of the pore fluids in the voids should remain at around 12.4 (Eades and Grim, 1960; Bell, 1996). The solubility of silicon and aluminium ions is very high at this value. Calcium silicates are formed as long as the highly alkaline conditions persist. In addition, small amounts of calcium aluminate hydrate phases (e.g., C_4AH_{13} and C_3AH_6, where C is CaO, A is Al_20_3 and H is H_2O, as per Cement Chemists' notation) and calcium silicate aluminate hydrate phases (C_2ASH_8, where S is SiO_2) may develop, especially when kaolinitic clays are treated with lime (Bell, 1996). On treating montmorillonite with lime, Bell (1996) found the formation of reaction products such as, calcium aluminate hydrates CAH, C_4AH_{13} or CAH_{10} along with calcium silicate hydrates (CSH). While treating quartz with lime, it has been observed that calcium silicate hydrates $C_3S_2H_3$ are formed. Al-Mukhtar et al. (2010) quoting various sources reported that different new products may develop from the lime-clay pozzolanic reactions, e.g. calcium silicate hydrates (CSH), calcium aluminate hydrates (CAH) and calcium alumino-silicate hydrates (CASH), tobermorite (CSHI), CAH_{10}, CAH_{11}, C_3AH_6, and C_4AH_{13}.

Wild et al. (1986) observed that the cementitious gel formed during reaction of lime with clay soil is derived from the breakdown of the original clay particles as a result of gradual substitution of Ca for both the inter and intra-layer cations within the clay.

López-Lara et al. (2005) examined the effectiveness of lime treated expansive soils, over a long span of time. During the lime-treated stabilization process for ten weeks, there was calcite formation which increased exponentially, reaching its maximum value at 6 weeks; from there on, it remained constant. It was found that the lime-stabilized soil aged for 6 years corresponds approximately to 2 weeks in the stabilization kinetic process at laboratory level. Therefore, the effectiveness of this treatment will last for more years. The study also indicates that montmorillonitic clay soils stabilized with lime does not recover its initial plastic properties. It is recommended to treat a minimum thickness of soil in the field. For example, with soil-lime at 7%, the study reported functional effective thickness to be approximately, 60 cm, because it was observed that this thickness is subjected to the maximum water content variations, depending on weather conditions, beyond which it was marginal. Further, it was found that this thickness of stabilized soil works as an impermeable barrier for the expansive soil underneath, without affecting the expansive clay soil below.

Extended curing period and elevated temperature are conducive for pozzolanic reactions (Arabi and Wild, 1986; Bell, 1996; Rao and Shivananda, 2005; Hafez et al., 2008). It is found that pozzolanic activity commences after 1 day of curing at 25°C in comparison to 7 days needed at 11.5°C (Rao and Shivananda, 2005). Bell (1996) reported that pozzolanic reactions may remain dormant during periods of low temperatures (i.e. <4° C), and regain the reaction potential when temperature increases.

In addition to the pozzolanic reactions, the other reaction that contributes to the cementation process is carbonation, which is the reaction between lime and atmospheric carbon dioxide (Hafez et al., 2008). The resulting product, calcium carbonate, acts as a weak cementing agent (Goldberg and Klein, 1952; Eades and Grim, 1960; Arman and

Munkfah, 1970). Lime in excess of the requirement of initial reactions may convert back to calcium carbonate on exposure with the atmospheric CO_2. It contributes to a very small strength increase because of solidification or setting of lime, but it reduces lime availability for pozzolanic reactions, reducing long term strength gain (Arman and Munfakh, 1970).

Comparing the effect of Portland cement and quicklime, Esrig (1999) reported that lime releases more heat and consumes more water in the hydration process in comparison to Portland cement, and hence, quicklime is more efficient in reducing moisture content of clay soil. This is equivalent to increase in consolidation pressure. For soils having natural water content of about 35-40%, are prone to large increase in undrained shear strength. However, for soils having very high water content e.g. expansive soils, the percentage loss of moisture being too small, more lime is required for adequate strength gain of the otherwise weak soil.

Khattab et al., (2007) examined the long-term stability characteristics of a bentonite soil using 4% lime treatment on the basis of wetting–drying cycles and leaching tests. The tests were carried out on specimens compacted at optimum moisture content and maximum dry density conditions. It was reported that lime treatment induces changes in the pore size distribution leading to increase in the coefficient of permeability of the clayey soil. Leaching did not reduce the efficiency of the treatment as the quantity of lime displaced by the water flow under the applied conditions (hydraulic gradient of 10) was very small.

Kawamura and Diamond (1975) have studied the effect of lime stabilization of clay soils against erosion loss using hydrated lime or cement. The reaction products formed due to soil-lime reactions include calcium silicate hydrate (CSH) gel in a reticulated network (well-knit framework) which binds the individual clay particles together to form aggregations. Locat *et al.* (1990) reported the formation of platy calcium aluminate silicate hydrate (CASH) and reticular CSH cementitious compounds in the lime-treated soil system.

2.2.2 Effect on Plasticity Characteristics

Several researchers observed that the liquid limit of clayey soils decrease with addition of lime (Thompson, 1966; Holtz, 1969; Bell, 1996; Boardman et al., 2001; Galvao et al. 2004; Kavak and Akyarli, 2007; Khattab et al., 2007). This was attributed to the reduction in thickness of the diffuse double layer, which takes place due to cation exchange and flocculation-agglomeration reactions (Lambe, 1962; Thompson, 1966). However, the liquid limit of some clayey soils was found to increase with addition of lime (Ingles and Metcalf, 1972; Prakash et al., 1989; Bell, 1996; Galvao et al. 2004). The plastic limit of clayey soils usually increases on addition of lime. This change is more prominent with more clay content in the soil. Montmorillonitic soils show the maximum increase in plastic limit (Hilt and Davidsson, 1960). As a result, the plasticity index is usually reduced with the addition of lime (Herrin and Mitchell, 1961; Dumbleton, 1962).

Bell (1996) noted that the effect of lime on plasticity of clay is almost instantaneous. Such an effect is mostly attributed to modification reaction. Clay particles aggregated and behaved like silt on treatment with lime. Plastic limit of montmorillonite increased up to 4% lime content, beyond which PL decreased slightly. Liquid limit of montmorillonite decreased with lime content. In contrast, liquid limit of kaolinite increased on addition of lime and plastic limit decreased beyond 2% addition of lime. Thus, it may be noted that the mechanisms leading to change in liquid limit due to addition of lime for different types of clays are different. Very small quantities of lime are required to bring about these changes in plasticity. Generally the amount needed varies from 1 to 3% depending on the amount and type of clay minerals present in the soil.

It was also observed that liquid and plastic limit values are affected by the curing time of soil with lime (Prakash et al., 1989; Sivapullaiah et al., 2000). Afes and Didier (2000) observed, in the case of expansive soil of Mila (Algeria), that liquid limit reduced for soils treated with 3% and 6% hydrated lime, and cured for 7 and 28 days. For 90 days curing, liquid limit again increased to equal or even did surpass that of the untreated soils.

Sridharan and Rao (1975), Sridharan et al. (1988) reported that on addition of lime, both increases and decreases in liquid limits can occur depending upon the soil type and associated exchangeable cations. The liquid limit of clays is primarily controlled by (a) shearing resistance at the particle level and (b) thickness of the diffuse double layer. An increase in attractive forces or a decrease in repulsive forces results in the effective stress increase and hence the shearing resistance increases at the particle level (Sridharan and Rao 1975). The liquid limit of montmorillonite is a function of diffuse double layer thickness, which in turn depends on surface area, cation exchange capacity, size and valence of cation and pore fluid (Sivapullaiah et al., 2000; Sridharan et al., 1986).

Addition of lime may have the following effects (Sivapullaiah et al. 2000, Thompson, 1966; Uehara and Gillman, 1981; Sridharan and Jayadeva, 1982; Sridharan and Rao, 1975):

- An increase in the electrolyte concentration reduces the double layer, aiding to form clay clusters, which lead to a decrease in the liquid limit.

- If the exchangeable cations present in soil are monovalent, addition of lime decreases the thickness of the double layer due to the higher valence of calcium, which in turn brings down the liquid limit.
- If the cation present is divalent, cation exchange will have very little effect. However, replacement with calcium ions generally induces flocculation, which in turn increases the liquid limit.
- If the exchangeable cations are more than divalent, partial replacement of higher-valence ions with calcium, increases the liquid limit.
- Addition of lime increases pH, which increases the cation exchange capacity and hence leads to an increase in liquid limit.
- Addition of lime causes flocculation of clay particles. When the fabric becomes more flocculent, the water-holding capacity increases and hence liquid limit increases.

Thus, addition of lime to soil can bring about an increase or decrease in liquid limit, depending upon which of the above factors dominates (Sivapullaiah et al., 2000).

2.2.3 Effect on Compaction

As water is added to the expansive soil, the thickness of diffuse double layer increases. Due to this, the dry density tends to decrease. According to Murthy et al. (1985), when a small quantity of water is added to fine grained soils, due to inadequacy of available water, clay particles share the available water resulting in the formation of clusters as shown in Fig.2.1. These clusters are in equilibrium under the influence of physico-chemical forces and pore water tension. The rigidity of such clusters depends on the physico-chemical properties of soils (expressed in the liquid limit) and the mixing moisture content. The density achieveable for a compactive effort is inversely proportional to the rigidity of the clusters. Since addition of lime changes liquid limit of soil, so it changes the rigidity of the clusters, ultimately changing the compaction behaviour of soil.

The addition of lime to clayey soils increases the optimum moisture content and reduces the maximum dry density for the same compactive effort (Prakash et al., 1989; Bell, 1996; Holt and Freer-Hewish, 1996; Sivapullaiah et al., 1998). This is due to the formation of cementitious products, which reduce compactness (Bell, 1996).

However, this was not to be considered as a disadvantage, as the loss in dry density is more than compensated by the gain in strength. Croft (1964) showed that kaolinite was more amenable to compaction compared to expansive soils.

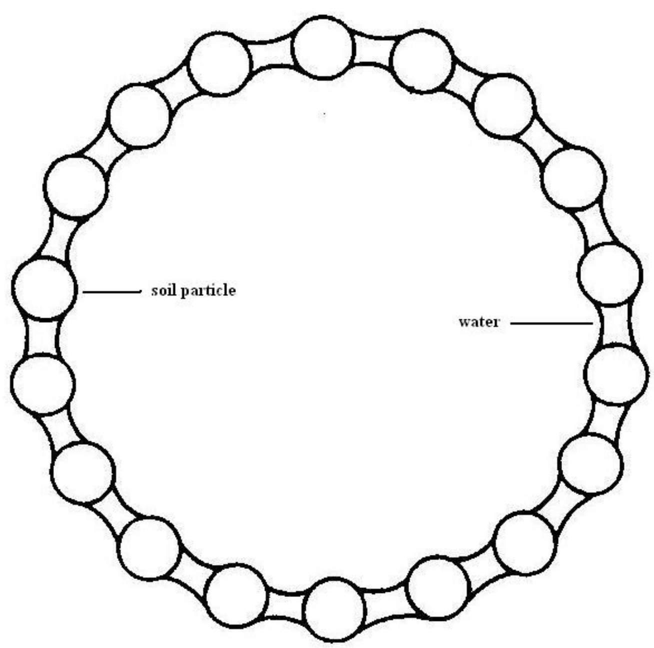


Fig. 2.1 Formation of soil cluster on the dry side of MDD

(Murthy et al., 1985)

Sivapullaiah et al. (1998) observed that the maximum dry density of expansive soil decreased steeply when the lime content is below the lime fixation point. No further decrease in MDD was seen beyond lime fixation point. The `lime fixation point' is defined as the optimum lime content for maximum increase of the plastic limit of the soil (Mateos, 1964; Sivapullaiah et al., 2000). This is primarily dependent on the cation exchange capacity and type of exchangeable ions present in the soil.

Different authors reported correlation of MDD and OMC with LL, PL and PI. However, the wide variety of soil properties makes it difficult to arrive at a generalised correlation. The Design Manual of U.S. Navy (1962) gives the following empirical relationships for estimation of OMC and MDD under Standard Proctor compaction:

$$OMC = 6.77 + 0.43LL - 0.21PI, \quad (2.1)$$

$$MDD = 20.48 - 0.13LL + 0.05PI \quad (2.2)$$

Hammond (1980) found the following relationship for black cotton soils:

$$OMC = 0.96PL - 7.7 \quad (2.3)$$

Yemington (1958) developed a chart (Fig. 5.4) which could be used to estimate OMC and MDD from the Atterberg limits of soils within the range of LL up to 90% and PL upto 50%.

Osinubi and Nwaiwu (2006) studied the effect of delay of compaction after lime and water mixing with red soil. They found significant reductions in MDD and OMC values associated with compaction delays. The reduction of UCS due to compaction delays was also significant.

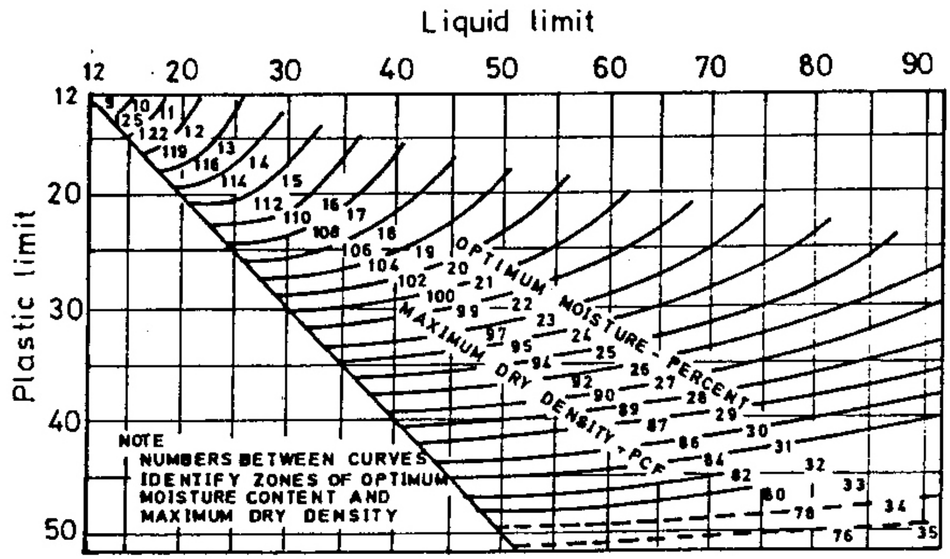


Fig. 2.2 Estimation of MDD and OMC from Atterberg limits of soils (Yemington, 1958)

2.2.4 Effect on Strength

Clay soils gain in strength significantly when stabilized with lime (Bell, 1996, Kavak and Akyarli, 2007). There is an optimum lime content for strength gain which is about 4% for montmorillonite (Bell, 1996). Expansive clays respond more quickly to strength increase. In comparison, strength gain in kaolinite was more dependent on time. On the other hand, Kavak and Akyarli (2007) have found that the unconfined compressive strengths increased by 6 fold for bentonite and 12 fold for kaolinite in a period of about one month. The long term cured samples have shown to experience further strength increase. The nature of failure was found to have changed to brittle from ductile.

Long-term strength improvement in lime stabilized soils and aggregates has been verified by extensive laboratory testing as well as field testing (Little, 1999). These tests show that, when lime is added to a reactive soil or aggregate, strength gain in excess of about 1,400 kPa is expected. This increase in strength provides significant structural enhancement to the pavement. In some soils, ultimate compressive strength values of as high as 7,000 to 10,000 kPa can be reached. Little (1999) also reported that strength gain continues with time even after periods of environmental or load damage (autogenous healing) providing long-term durability of lime-treated soil over decades of service, even under severe environmental conditions.

Khattab et al. (2007) reported that treatment with 4% lime resulted in an overall improvement of most of the mechanical properties of the expansive clay by increasing the shear strength. Working on several types of soils, Locat (1990) reported that the soils, including the high water content ones, attain significant strength increase when enough time or quicklime is provided. Although the initial controlling reaction parameters are grain size and specific surface area, with the development of pozzolanic reactions, the mineralogy becomes the chief parameter related to strength development. Presence of high level of carbonate, sulphate, chlorite and organic matter was found to have inhibiting effects on strength development. As the pozzolanic reactions continue, lime in solution is consumed and more must be dissolved to maintain solution equilibrium (Locat et al., 1990). Increase in quicklime concentration, at least up to 10%, is favorable in terms of strength development, despite the fact that less than 0.1% lime is sufficient to saturate the pore-water solution. In addition to the lime content, the dispersion of solid lime in excess may have a strong influence on stabilization. Mixing lime with soil vigorously yielded higher strength at equivalent lime concentration and time, this being more evident for high plastic soils. During mixing, a given lime dispersion is achieved. When pozzolanic reactions take place and consume lime in solution, equilibrium concentration gradients occur in the pore-water solution between soil and lime to maintain the pH at 12.4. So, higher is the lime content better is the dispersion of it, the shorter the average distance between the reacting soil and lime particles, thus producing a more efficient molecular diffusion of calcium within the interconnected and saturated portion of the porous system. With time, the diffusion process may be impeded by precipitates that slowly fill the pore space.

Unconfined compressive strength increases with lime content and time, provided sufficient lime is available in soil. The rate of increase of strength with increase in lime content however reduces (Wild et al., 1986; Al-Mukhtar et al., 2010), or the UCS stops increasing or even starts decreasing on further addition of lime (Osinubi and Nwaiwu, 2006).

2.2.5 Effect on Swelling

Khattab et al. (2007) have shown that the majority of expansive soils are typically medium to highly plastic soils (i.e., CL to CH) with swelling pressure values in the range of 50–500 kPa and swell potential in the range of 2–20% (Williams and Donaldson 1980). Swell and shrinkage characteristics of expansive soils are significantly improved by addition of lime. Bell (1996) reported that this is due to the decrease in moisture absorption capacity in lime treated soils. Swell pressure reduced to about 1/4th of the original value of about 160 kPa.

Afès and Didier (2000) described treatment of black cotton soil in Algeria with slaked lime. The swelling potential is significantly affected by the addition of lime, even within a short span of 7-days. With 3% of lime, the swelling potential of the soil dropped from about 6% to less than 1%, over a period of 7 to 90 days. It is opined that the percentage of lime added to the clay and the period of curing have an important effect on the physical characteristics of the soil, including its swelling potential. They also examined the influence of wetting and drying cycles. In the lime treated specimen, the free swell (i.e., volumetric changes) seems to stabilize after a few cycles at a value significantly lower than in the untreated specimen. However, with an initial drying the free swell is found to have progressively increased, during the cycles, and the treatment seems to have lost most of its beneficial effect. Such behavior is attributed to the interruption of the lime–clay reactions, mainly pozzolanic, during the initial drying.

Ameta et al. (2008) have observed the effects of gypsum ($CaSO_4.2H_20$) on the performance of the expansive soil of Jaisalmer, Rajasthan, that has a liquid limit of about 190%. On addition of gypsum, swelling pressure of this soil was found to have reduced by more than 70%.

Swamy (2006) examined the efficacy of stabilising black cotton soil using lime piles. The study compared the results of lime-soil mixtures (1 % and 3 %) and lime piles (75 mm and 25 mm diameters). It was observed that lime pile treatment in the field can substantially reduce the swell potential of the soil at least to a radial extent of 2 to 3 times the lime pile diameter. It was, however, found that mixing of lime with soil is more effective rather than the lime pile, wherein the lime interaction is mostly through diffusion, which is a delayed process. With 3 % lime added, the pH of the lime-mixed soil was found to be sufficiently high (i.e. in excess of 12) to form the cementation compounds.

2.2.6 Effect on Compressibility

The compressibility of pure clays under external load depends not only on the mechanical properties of clay minerals but also on the physicochemical properties of the pore fluid i.e. concentration of ions, valency of cations or dielectric constant of the pore fluid, etc. (Sridharan and Jayadeva, 1982; Bolt, 1956; Olson & Mesri, 1970; Mitchell, 1973; Sridharan & Rao, 1973). Sridharan & Rao (1973) established that basically two mechanisms control the volume change behaviour of clays. The first one is the shearing resistance at the region where two adjacent particles come to the closest. This resistance is different at different points of contact and volume changes occur by shear displacements between particles. The second controlling mechanism of compressibility is governed by the electrical repulsive forces due to the double layer. The factors involved in the double layer theory are, (i) soil properties represented by base exchange capacity and surface area (ii) fluid properties, i.e. ion concentration, cation valency, dielectric constant and temperature.

As ion concentration increases, the swelling pressure decreases because the double layer gets compressed. The equilibrium void ratio under any loading is directly proportional to the square root of the dielectric constant of the pore fluid (Sridharan and Jayadeva, 1982).

An increase in cationic valency reduces the compressibility of the clay material along with the liquid limit, while an increase in the hydrated ionic radius (for a constant valency) raises both the compressibility and liquid limit. Potassium and ammonium bentonites consolidate approximately five times faster than sodium and lithium bentonites. Comparatively, the divalent clays consolidate about ten times faster than the sodium and lithium bentonites and approximately 2-5 times quicker than the potassium and ammonium clays (Sridharan et al., 1986a).

Rajasekaran and Narasimha Rao (2002) found that the application of lime resulted in the reduction of compressibility of marine clay to 1/2 to 1/3 of the compressibility of the untreated soil within 30 to 45 days of treatment. There was significant increase in the preconsolidation pressure value from 36 kN/m^2 to 82 kN/m^2, with a corresponding decrease in the compression index value from 0.85 to 0.36.

Galvao et al. (2004) made an investigation of the effect of hydrated lime on the compressibility of two Brazilian soils - a brown saprolitic soil, and a red lateritic soil. On the addition of 4% lime, the resistance of the soil to compression improved substantially. Further addition of lime was not found to increase the less improvement.

Rao and Shivananda (2005) found that the one-dimensional compression behaviour of clay stabilized with saturated lime is similar to saturated cemented soils. The compression curves have an initial region where no axial strains occur on loading the specimens, followed by initial yield where some axial deformation of the specimens occur that are largely elastic in nature. The initial yield is followed by a more prominent second yield where large plastic strains occur due to general bond failure. Presence of cementation bonds imparts yield stress ranging from 3900 to 5200 kPa to the lime-stabilized specimens. The artificially cemented specimens exhibited relatively low magnitudes of elastic strains (3–4%) in the pre-yield stress region and large magnitudes of plastic strains (12–18%) in the post-yield stress regions. However, the stabilised specimens exhibited similar strain magnitudes per unit pressure increase in the pre-yield and post-yield stress regions.

Salehi and Sivakugan (2009) have investigated the compressibility behaviour of dredged mud from the Port of Brisbane, Australia, and found that the addition of 4% lime brings about maximum flocculation resulting in maximum void ratio and permeability. The coefficient of consolidation drastically increased in both compression and recompression states. In the compression range, the increase of the coefficient of consolidation was up to 10-fold with an increase of lime up to 4%. In the compression range, the compression index increased with increasing lime content, whereas in the recompression range, the value of the recompression index gradually decreases with increasing percentage of lime. This indicates that with increasing percentage of lime, the magnitude of primary consolidation in the NC state increases while in the OC state it decreases. The secondary compression index, in both the compression and recompression range, decreases with increasing percentage of lime.

2.3 TREATMENT OF SOIL WITH FLY ASH

Based on the self-cementitious properties, fly ashes are divided in to two categories – Class C and Class F (ASTM C618-08a). Both types are pozzolanic. Class C fly ash has generally higher calcium content, measured as CaO, (more than 10%) and is self-cementitious. Although many investigators thought that the self-cementitious properties were the results of the presence of free CaO in fly ash, Joshi (2000) reported that it is the calcium in the glassy phase i.e. fly ash particles which produces cementitious compounds on hydration even without the presence of free lime.

Cocka (2001) has examined the effect of high-calcium and low-calcium, class C fly ashes, for stabilization of an expansive soil in Turkey. The study compared the behavior of the soil-lime, soil-cement, and soil-fly ash mixes. The test soil had liquid limit and plastic limit of 74% and 22%, respectively. Lime and cement were added to the expansive soil in the range of 0–8%. It was found that class C fly ash can be effectively used for improvement of expansive soils. Addition of 20% fly ash, substantially decreased the swelling potential of the expansive soil. Further increase in fly ash content was not beneficial. It has also been observed that the beneficial effect of 20% fly ash, in reducing the swell potential, is similar with that of 8% lime.

Phanikumar and Sharma (2004) have studied the effect of a low calcium fly ash on the engineering properties of an expansive soil. Addition of fly ash reduced the plasticity characteristics of the expansive soil. The liquid limit decreased and the plastic limit increased with increase in the fly ash content. With 20% fly ash, the free swell index could be reduced by about 50%. Both the swell potential and swell pressure were reduced by 50% at 20% fly ash. With an increase in fly ash content the optimum moisture content decreased and the maximum dry unit weight increased, that the addition of fly ash is akin to increased compactive effort. The hydraulic conductivity decreased with an increase in fly ash content. The undrained cohesion (c_u) of the expansive soil blended with fly ash increased with the increase in fly ash content. At water content of 20% the increase in undrained shear strength was about 27% when the fly ash content was 20%.

Kumar et al. (2007) have studied the effects of lime stabilization on the geotechnical characteristics of expansive soil-fly ash mixtures. The expansive black cotton soil had liquid limit and plastic limit of 68% and 58% respectively. Lime and fly ash were added to the expansive soil in the ranges of 1–10% and 1–20% respectively and cured for 7, 14, and 28 days, after which, they were tested for unconfined compression tests and split tensile tests. With the increase in lime content, the maximum dry density of soil-lime mixes decreased and optimum moisture content increased. The fall in density was more significant at lower percentages of lime. When fly ash is added to soil-lime mixture, maximum dry density decreased further and optimum moisture content increased. Time of curing did not produce much increase in strength up to 4% of lime content. With the increase in the percentage of fly ash keeping amount of lime as constant, strength increased and reached a certain maximum value and thereafter it started decreasing, but was always higher than that of the respective soil-lime mixture. The optimum value of lime content and fly ash content in the fly ash-soil-lime mixtures were found to be 8% and 15%, respectively.

Phanikumar and Sharma (2007) have studied the effects of mixing a low-calcium fly ash on the volume change behaviour of expansive and non-expansive clays. For the type of fly ash and expansive clays used, 20% fly ash content reduced the free swell, swell potential, and swelling pressure by about 50%. The reduction in the swelling characteristics was basically, by replacement of plastic fines of clay by nonplastic fines of fly ash. The reduction in swelling was also attributed to the flocculation and cementation effects developed due to the fly ash. The compression index of both expansive and non-expansive clays decreased by about 50%, when fly ash content was 20%, indicating that the effect of fly ash is more pronounced on the compressibility behavior of expansive clays.

Secondary consolidation characteristics of fly ash-blended clays also showed improvement in comparison to those of untreated clays. The volume change due to creep and slippage of particles after the end of primary consolidation was better resisted by clay blended with fly ash. The time required for the end of primary consolidation and the beginning of secondary consolidation was shortened in clays blended with fly ash. This indicates that the amount of settlement of structures built on fly ash-amended expansive clays decreases and the rate of settlement increases reducing the time required for reaching the final settlement.

Nalbantoglu and Gucbilmez (2001) have investigated the stabilizing effect of a high calcium fly ash on a calcareous expansive soil. The soil had 17% montmorillonite content, with liquid limit and plasticity index of 67.8% and 22.2% respectively. The fly ash had 16% CaO content. The study indicates that fly ash improves the swell characteristics of the expansive soil, but is more time-dependent. However, adding 3% of lime along with fly ash improved the swelling properties instantly and significantly. Fly ash, with relatively high percentage of lime, provided a better source of pozzolans for the soil to react with lime and form the cementitious compounds.

Ghosh and Subbarao (2001) reported that the interaction between fly ash and lime is complex and pozzolanic reaction is very slow. The X-ray diffraction study indicated the formation of new gel-like crystalline phases of CSH1 after 4 weeks of curing, and also the presence of hydrated calcium aluminate C_4AH_{13}. Similarly, Luxan et al. (1989) identified the formation of calcium aluminium hydrate (C_4AH_{13}), carboaluminate (C_4ACH_{11}), monosulfoaluminate (C_4ASH_{12}), and calcium silicate hydrate (C-S-H) as pozzolanic products of fly ash and calcium hydroxide.

Joshi (2000) reported that the pozzolanic nature of the fly ash and hence its reactivity with lime depends on the degree of strain in the alumino-silicious glassy particles.

Prabhakar et al. (2004) conducted a series of tests on three types of soil mixed with fly ash at various proportions to study the influence of compaction and swelling, as well as development of shear strength and penetration resistance. It is found that for all the three soils, on varying fly ash content from 0 to 46 per cent, the maximum dry density decreased and the optimum moisture content increased. The CBR value too increased consistently with increase in fly ash content. The cohesion intercept obtained from shear tests was found to increase with fly ash content in the case of two soils i.e. clayey silt and gravelly silt, while it was found to decrease in case of the silty clay. The angle of internal friction increased gradually for fly ash content up to about 41 per cent. However, at fly ash content of 46%, the angle of internal friction decreased for the clayey silt. In general the free swell decreased with increase in fly ash content.

Kaniraj and Havanagi (1999) mixed Rajghat fly ash from Delhi, India with Yamuna sand, and Baumineral fly ash from Bochum, Germany with the Rhine sand, in different proportions. Cement, varying from 3-9% was added to stabilize these fly ash-soil mixtures. Unconfined compression tests were conducted on these samples prepared at optimum moisture content and maximum dry density and were cured for different duration. Correlations for unconfined compressive strength and secant modulus as functions of curing time, fly ash content, and cement content have been established. Correlations for water content as functions of curing time and cement content were also established. It is observed that the gain in unconfined compressive strength and secant modulus of fly ash-soil mixtures with time can be assumed to be hyperbolic. The gain in strength and modulus increase as cement content increases, but decrease as fly ash content increases. The cement content has a significantly higher influence on strength than the fly ash content. The water content decreases as curing time and cement content increase. The influence of cement content is more pronounced than that of the curing time.

Goswami (2004) has used fly ash and lime for improvement of engineering properties of residual soil. The strength of soil-fly ash mix was found to be less than the strength of soil alone. However, after sufficient curing, the strength of the soil-fly ash mix surpassed that of the soil. Strength and stiffness of the soil increased with addition of lime, however with addition of higher percentages of fly ash, the strength was found to reduce. With fly ash alone the angle of internal friction was found to reduce, when tested under undrained condition. Addition of about 2% lime increased the angle of internal friction and induced cohesion as well. Further addition of lime significantly increased both friction and cohesion.

Lav and Lav (2000) have reported that the pozzolanic reactivity of fly ash is influenced by the factors such as phase composition, chemical composition, fineness, morphology, and loss on ignition.

Dermatas and Meng (2003) have experimentally explored the possibility of using fly ash for stabilization and solidification of heavy metal contaminated soils. The experiments were conducted with artificial soils composed of kaolinite-quartz fine sand and montmorillonite-quartz fine sand, mixed with fly ash and quick lime. On increasing the clay content up to about 30%, strength continued to increase. This is attributed to enhanced pozzolanic reactions caused by the alumina and silica present in the soil. Addition of Class C fly ash increased the strength manifolds, which further went up when quick lime was added. For soil with 5% kaolinite, the UCS increased from 12.5 kPa to 3830 kPa, on addition of 25% fly ash. However, the same soil with 25% fly ash when added with 10% quick lime, the strength increased from 145 kPa to 6662.5 kPa. The X-ray diffraction analyses of the lime treated samples revealed that pozzolanic products formed are mainly, calcium silicate hydrate (CSH) and calcium silicate hydroxide. When fly ash was used, additionaly ettringite was also formed. This is attributed to the presence of sulphate within the fly ash matrix. Ettringites cause swelling on contact with water. However, this swelling was overcome due to the cementing action of the fly ash.

Ferguson (1993) reported that ash treatment can effectively reduce the swell potential of clay soils and thereby increase subgrade support capacity of pavements. Ash hydration occurs rapidly and therefore the delay between incorporation of the ash and final compaction is to be limited, preferably, to less than 2 hours. Compressive strengths of ash treated materials are very much dependent on the moisture content at time of compaction and hence strict moisture control is required during construction.

Lav and Lav (2000) have studied the microstructural development in fly ash of New South Wales, Australia, treated with cement or lime. The XRD analysis of unstabilized fly ash revealed the presence of quartz and mullite as the crystalline components. The study also found the XRD patterns of cement-stabilized fly ash and lime-stabilized fly ash to be almost similar, differing only in the intensity, which implies that similar hydration products are formed. The authors investigated the development of microstructures under SEM on stabilization of fly ash after 28 days. It was noted that the reaction products in cement-stabilized fly ash were similar to those in hydrated Portland cement. Just after stabilization, there was no sign of significant hydration, but after 7 days of curing the fly ash particles were found to be the nucleation sites for hydration products. Further, Ettringite was formed using the fly ash spheres as nucleating sites. Fibrous hydration products were found, indicating CSH gel and ettringite rods being joined together, resulting in strength gain in the cement-stabilized fly ash matrix.

2.4 CONCLUDING REMARKS AND OBJECTIVE OF PRESENT STUDY

From literature review it is found that the individual application of fly ash and lime have been extensively studied, for the improvement of soils, but the combined application of both has not been dealt in detail. This aspect has been taken up under the present research work. The objective of the study is to develop an understanding of the performance improvement of expansive soil through combined application of fly ash and lime.

2.5 SCOPE OF WORK

Following are the important scopes of the present study:

- To study the influence of lime-fly ash induced changes in the index properties of the expansive soil. Such a study would help in evaluating the workability of the amended soil, an important requirement for field applications.
- Review of literature indicates that the liquid limit of soils, important index property, tends to be different when is evaluated through the two different widely used methods i.e. percussion method and cone penetration method. This aspect has been carefully reinvestigated under the present study.
- The lime-fly ash amended soil is generally compacted, for geotechnical applications, to achieve a desired density and thereby the strength. Therefore, the compaction behaviour of soil-fly ash-lime composite is an important aspect which has been studied herein.
- The stabilised soil, when subjected to loading, such as in foundations, like any other material, is expected to undergo settlement. In view of this the influence of lime and fly ash on the compressibility behaviour of the expansive soil has been investigated.
- Swelling of soils leads to distortion and destabilisation in the structures founded over it. Lime and fly ash through chemical alteration are expected to modify the soil plasticity. Besides the induced cementation, over a period of time, would lead to solidification of the soil mass. These two phenomenons are expected to reduce the swelling of the soil which needs to be carefully investigated.
- Strength gain of lime-fly ash amended soils is primarily dependent on pozzolanic reactions and therefore is highly influenced by the quantity of the pozzolanic materials present and the curing period. It is therefore proposed to study the effect of quantity of lime, fly ash added and the period of curing on the strength of the amended expansive soil.

CHAPTER 3

MATERIALS AND METHODS

3.1 MATERIALS

This chapter describes the soil under investigation and the properties of the other materials viz. fly ash, lime used to improve the soil. Besides, the description of the investigations conducted are also presented in detail.

Expansive soil (ES)

The expansive soil used in this study is a bentonite, available commercially. The soil is light brown in colour and of smooth texture. This is typically a highly expansive soil, the swelling constituent of which is montmorillonite. The soils were mixed thoroughly, oven dried, lumps broken and sieved through 425 micron sieves and stored in polythene bags for testing. Characterization of the test expansive soil is presented in the next section of this chapter.

Fly ash (FA)

The fly ash used in the experiments is a waste product of Farakka Thermal Plant, National Thermal Power Corporation Ltd., West Bengal, India. This is a non-self cementing fly ash. The fraction passing 425 micron was used for various tests in this study. Characterization of the test fly ash is presented in the Section 3.3 of this chapter..

Lime

The lime used is laboratory grade CaO, available in air-tight packings of 500g in the powder form. The lime used has a minimum assay of 95%, with a specific gravity of 3.1.

3.2 CHARACTERIZATION OF EXPANSIVE SOIL

3.2.1 Physico-Chemical Properties of Expansive Soil

Specific gravity of expansive soil was determined using specific gravity bottle as per IS 2720 Part 3 Section 1. Kerosene was used with expansive soil because of high affinity of the expansive soil to water. The average value of specific gravity of the ES is found to be 2.54.

Particle size analysis was done in a laser particle size analyzer (Hydro 2000G, Malvern instrument U.K.). The usual sedimentation method of particle size analysis was not employed, as bentonite particles would get swollen in water resulting in higher sizes of particles. As per the results from the analyzer, the particle sizes of the expansive soil ranged from 1.12 μ to 399 μ. 97% of the particles were found to be finer than 75μ.

The specific surface area (*SSA*) was measured using (i) Brunauer, Emmet and Teller (BET) method and (ii) desiccator method described by Sridharan and Rao (1972). The specific surface area of the expansive soil obtained using the desiccator method is 127.7m^2/g. Sridharan and Rao (1972) reported a method to modify the dessiccator SSA value to its equivalent BET value. The BET equivalent specific surface area obtained from the desiccator method is 73.6 m^2/g, while the measured BET value is 65.79 m^2/g.

Scanning electron microscopy (SEM) is performed to find out the microstructure and texture of the expansive soil. SEM was done in moisture-less environment. In spite of the dry condition, the aggregation persisted. Very compact sub-rounded aggregated particles are formed mainly by scaly clay particles and some non-scaly particles (Fig. 3.1 and 3.2) as well.

Cation exchange capacity (*CEC*) is the measure of a soil to retain readily exchangeable cations which neutralize the negative charge of soil particle surfaces. *CEC* of expansive soil was determined using an ammonium replacement method described by Horneck, et al. (1989). The expansive soil used in the study has *CEC* of 45 meq as measured by this method. For comparison, montmorillonite has *CEC* value in the range of 80-150 (Mitchell and Soga, 2005). The soil under study has montmorillonite as one of the components, but not entirely montmorillonite, so this reduced value.

The lowest percentage of lime that gives a pH of 12.4 at 25°C is the approximate lime percentage for stabilizing the soil (ASTM 6276). The solubility of lime in water decreases at higher temperatures. In this study, the measurement temperature was 30°C and above, so the maximum pH value attained was 12.23 at 4% or more lime content (Fig. 3.3). So, the initial consumption of lime (ICL) for the expansive soil is 4%. The pH value without lime solution is 8.79, so the expansive soil is basic.

3.2.2 Composition of Expansive Soil

Energy Dispersive X-Ray (EDX) was performed during Scanning Electron Microscopy to find out the elemental composition of the expansive soil. It gives estimates of atomic percentages as well as percentage by weight of the elements present in the sample. Fig. 3.4 shows an EDX spectrum of expansive soil. Table 3.1 lists the elements observed in expansive soil based on 5 EDX observations. The ranges of percentages of various elements in expansive soil, as found through EDX are:

O – 64.01-70.32%	Si – 16.74-18.42%	Al – 7.44-9.22%
Fe – 2.08-4.01%	Na – 0-2.29%	Mg – 0-01.55%
Ca – 0-0.66%	Cl – 0-0.51%	

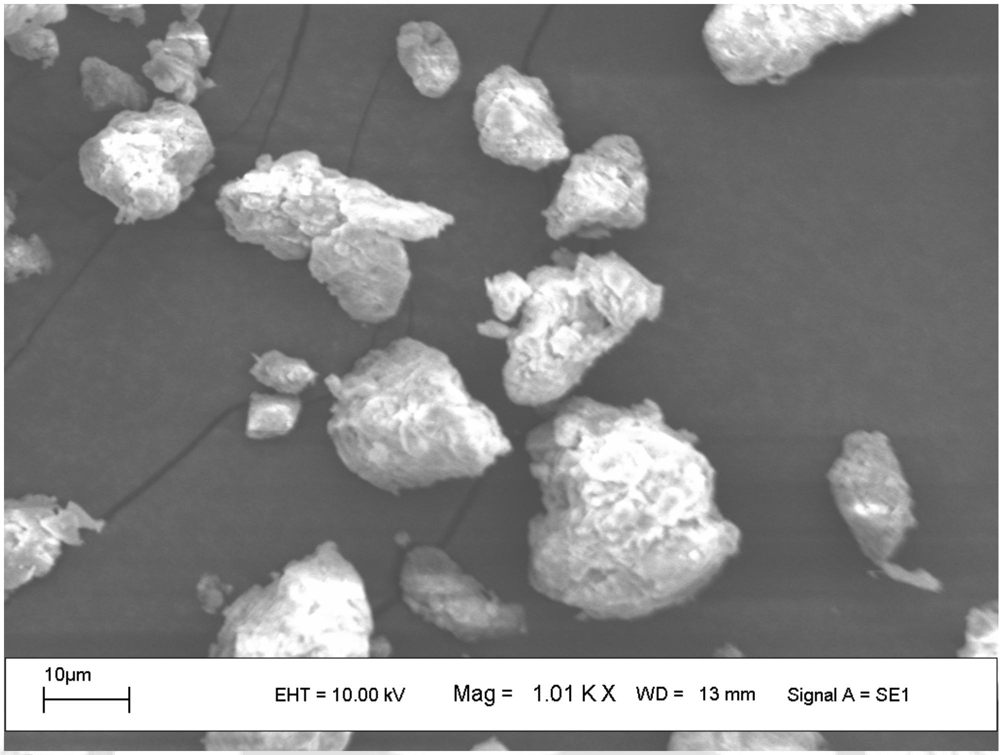


Fig. 3.1 SEM microphotograph of expansive soil.

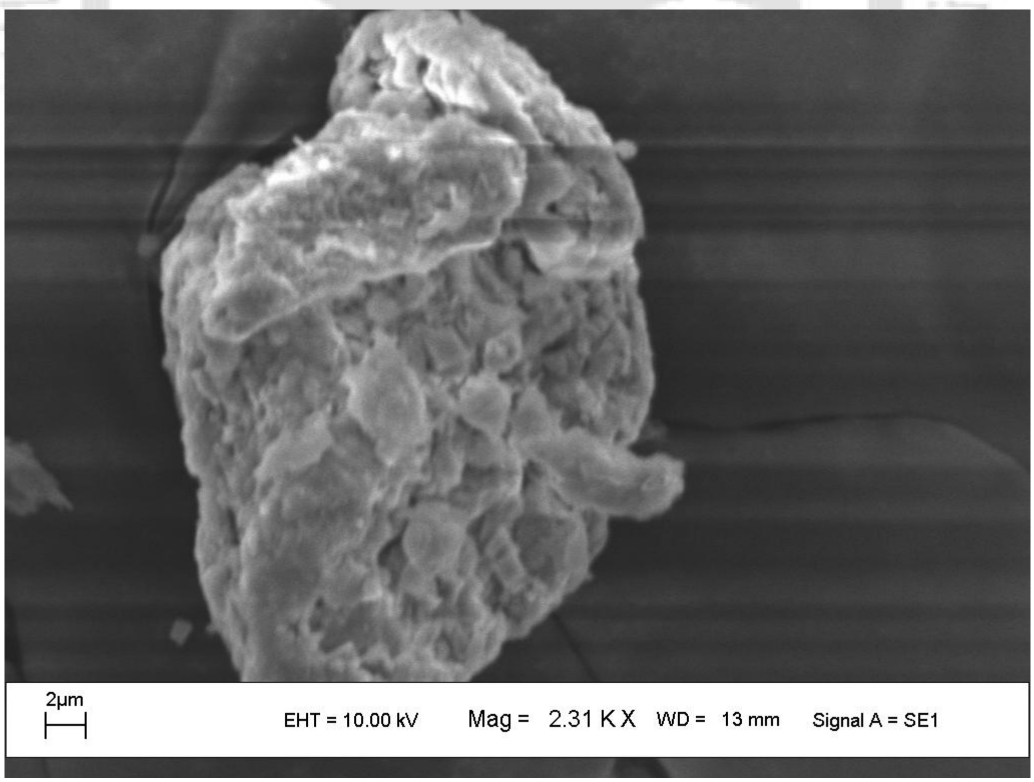


Fig. 3.2 SEM microphotograph of a particle of expansive soil at higher magnification.

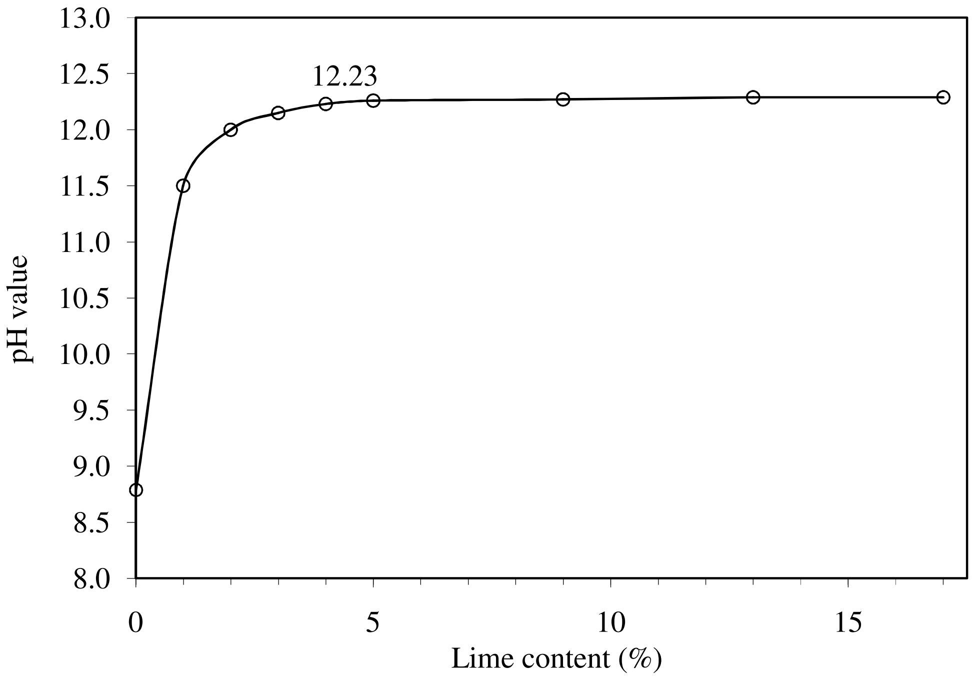


Fig. 3.3 pH values of lime-mixed expansive soil.

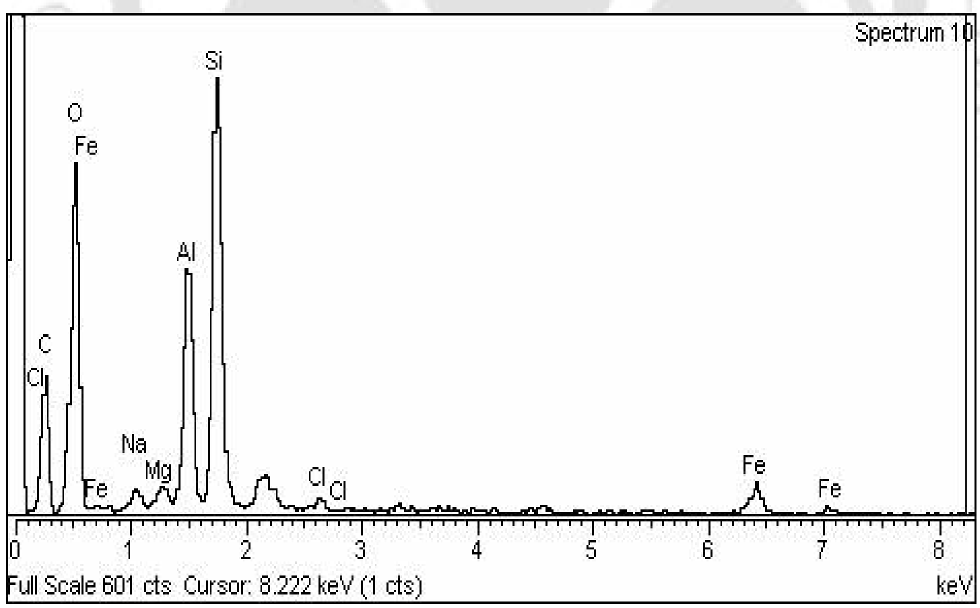


Fig. 3.4 EDX spectrum of an expansive soil sample.

Table 3.1 Elemental composition of ES as observed from EDX.

Element	Obs. 1		Obs. 2		Obs. 3		Obs. 4		Obs. 5	
	Weight %	Atomic %	Weight %	Atomic %	Weight %	Atomic %	Weight %	Atomic %	Weight %	Atomic %
O	53.36	68.95	47.79	64.01	51.42	67.31	55.21	70.32	53.16	67.54
Na	2.46	2.22	2.35	2.19	2.51	2.29	-	-	1.73	1.53
Mg	-	-	1.75	1.55	-	-	-	-	1.05	0.88
Al	10.27	7.87	11.61	9.22	9.58	7.44	11.74	8.87	12.00	9.04
Si	22.74	16.74	23.01	17.56	24.75	18.46	24.30	17.63	25.45	18.42
Ca	-	-	1.24	0.66	-	-	-	-	-	-
Ti	0.27	0.65	1.78	0.80	1.54	0.67	-	-	-	-
Fe	9.67	3.58	10.46	4.01	10.20	3.83	8.75	3.19	5.72	2.08
Cl	-	-	-	-	-	-	-	-	0.89	0.51

X-Ray Diffraction (XRD) tests were conducted to identify the minerals present in the expansive soil. A Bruker AXS D8 machine was used for the purpose. Major peaks in the XRD patterns were identified by comparing with standard patterns of minerals by search and match method. By trial and match, the minerals identified in the expansive soil from the positions of the peaks of the diffraction pattern are: SiO_2, montmorillonite, kaolinite, illite, TiO_2 and iron silicate (Fe_2SiO_4). The strongest peak matches with SiO_2, but it also coincides with illite and TiO_2.

3.2.3 Index Properties of Expansive Soil

Dry sieving only was done in the case of expansive soil. Wet sieving was not done as expansive soil particles absorb water, swell and become sticky. Sedimentation analysis was not done on the sub-75µ fraction as the swelling of expansive soil particles can yield

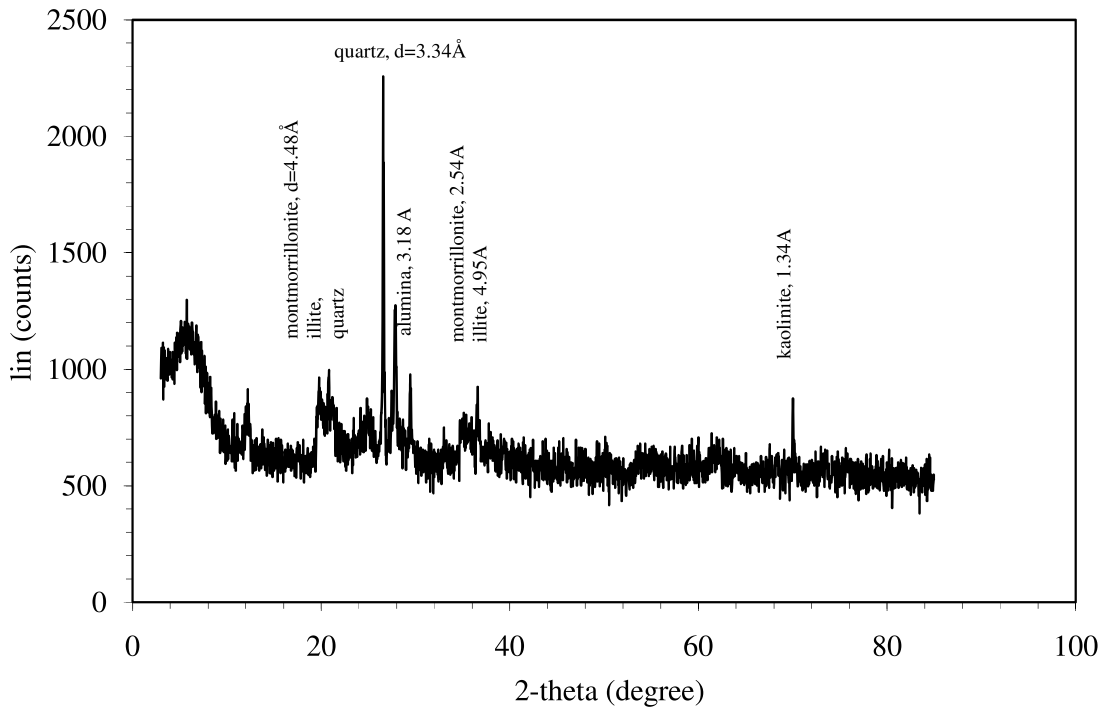


Fig. 3.5 X-Ray Diffraction pattern of expansive soil.

erroneous particle size distribution. Moreover, expansive soil suspension appears like gel and remains stable for a long time. The fractions of soil passing 425 micron and 75 micron sieves are presented in Table 3.2.

Table 3.2 Grain size of expansive soil.

Size	Fraction passing (by weight)
425 μ	100.0%
75 μ	73.7%

The Liquid limit (*LL*) tests on fly ash-clay mixtures were done using both Cassagrande's apparatus and cone penetration methods. Plastic limit (*PL*) tests are done by thread-rolling method. The Atterberg limits of the expansive soil are presented in Table 3.3.

Table 3.3 Atterberg limits for expansive soil.

Method	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Shrinkage Limit (%)
Cassagrande	265	42	223	33
Cone penetration	224		182	

Based on the liquid limit and plasticity index, the expansive soil is classified as clay of high compressibility (CH) as per the Plasticity Chart.

3.3 CHARACTERIZATION OF FLY ASH

3.3.1 Physico-Chemical Properties of Fly Ash

Specific gravity of the fly ash was determined using density bottle and water, as per IS 2720, Part 3. It is found to be 2.12.

Specific surface area of fly ash is measured using (i) desiccator method described by Sridharan and Rao (1972) and (ii) BET method. The obtained values are presented in Table 3.4.

Table 3.4 Specific surface area of fly ash.

Specific Surface by desiccator method (m^2/g)	BET equivalent Sp.Surface by desiccator method (m^2/g)	Actual BET result (m^2/g)
6.1	3.4	0.92

Fig. 3.6 shows a typical microstructure of the fly ash as observed under electron microscope. It can be seen that the fly ash has spherical particles of varied sizes in the micron scale. Some of the particles are broken indicating that the spheres are hollow.

The pH value of the fly ash is 9.0, showing its basic character. On addition of 1% lime, the pH value has steeply rised to 12.0. This indicates that apparently lime has not been consumed by fly ash for any short-term reactions, and almost all the lime has contributed to raise the alkalinity of the soil-water system. On further addition of lime, pH has shown marginal increase (Fig. 3.7).

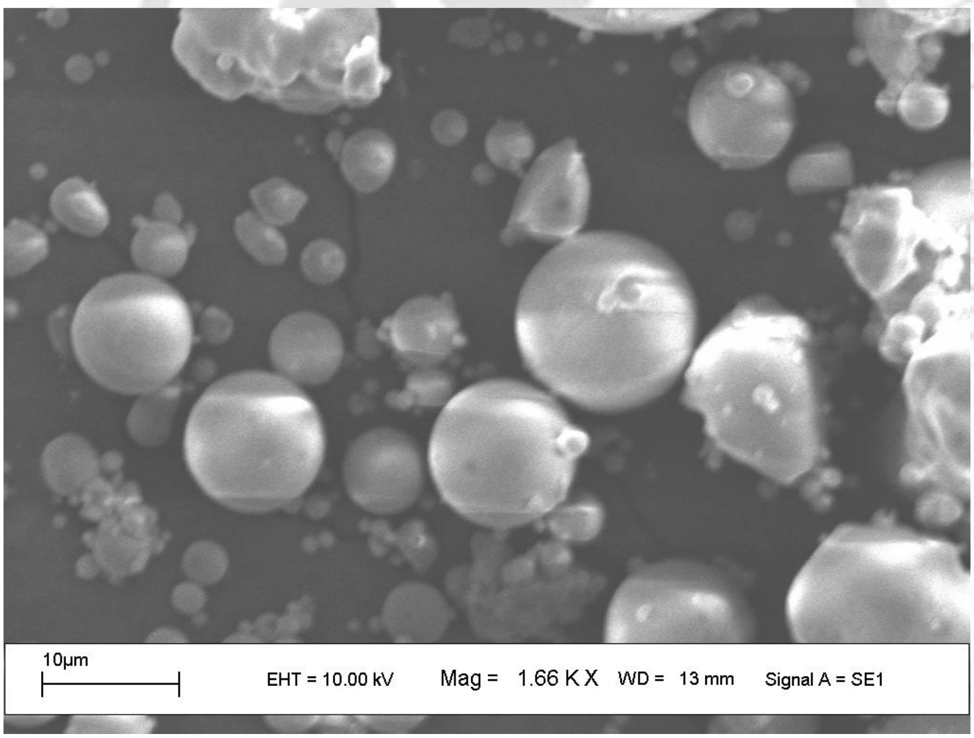


Fig.3.6 Electron photomicrograph of fly ash.

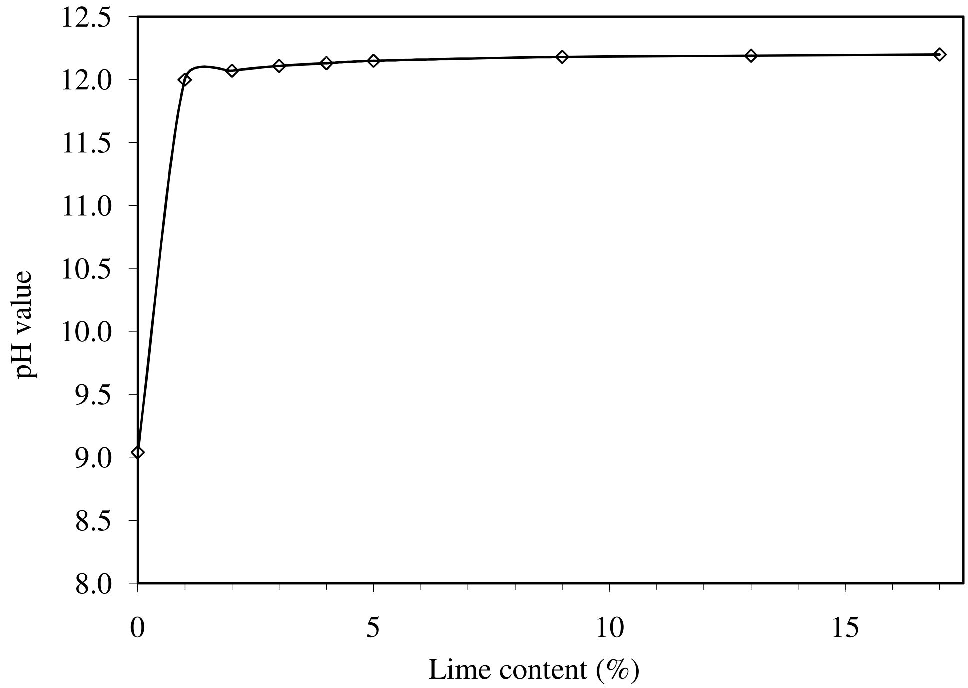


Fig.3.7 pH values of lime-treated fly ash.

3.3.2 Composition of Fly Ash

Energy Dispersive X-Ray (EDX) analysis was done in Scanning Electron Microscope to find elemental composition of the fly ash. An EDX spectrum generated thus is presented in Fig. 3.8. Average percentages of different elements, based on 3 observations, are presented in Table 3.5. Since fly ash is a combustion product, it can be assumed that all the above elements are present in the oxide form. Considering this, the molecular percentages of the compounds (oxides) by of the above elements are estimated as :

SiO_2 – 49.5-54.5%	Al_2O_3 – 25.3-33.8%
CaO – 0-1.3%	Fe_2O_3 – 6.8-8.0%
K_2O – 0.8-1.7%	TiO_2 – 2.4-3.2%
CuO – 3.6-7.6%	

Based on this estimate, the silica to alumina ratio of the fly ash ranges from 1.55 to 2.22.

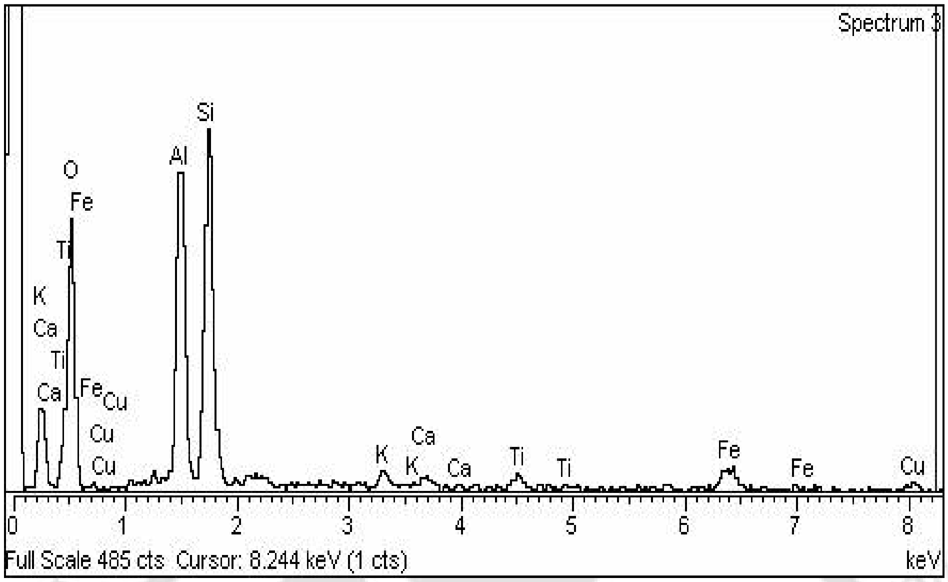


Fig. 3.8 An EDX spectrum of fly ash.

Table 3.5 Elemental composition of FA as observed from EDX.

Element	Obs.1		Obs.2		Obs.3	
	Weight %	Atomic %	Weight %	Atomic %	Weight %	Atomic %
O	49.49	65.39	44.41	61.57	47.37	63.81
Al	17.08	13.38	13.91	11.43	15.81	12.63
Si	22.04	16.59	26.42	20.87	24.22	18.59
Ca	0.84	0.44	1	0.55	-	-
Fe	4.53	1.72	5.76	2.29	5.1	1.97
K	1.33	0.72	0.7	0.4	1.25	0.69
Ti	1.8	0.8	1.49	0.69	1.76	0.79
Cu	2.87	0.96	6.3	2.2	4.5	1.53

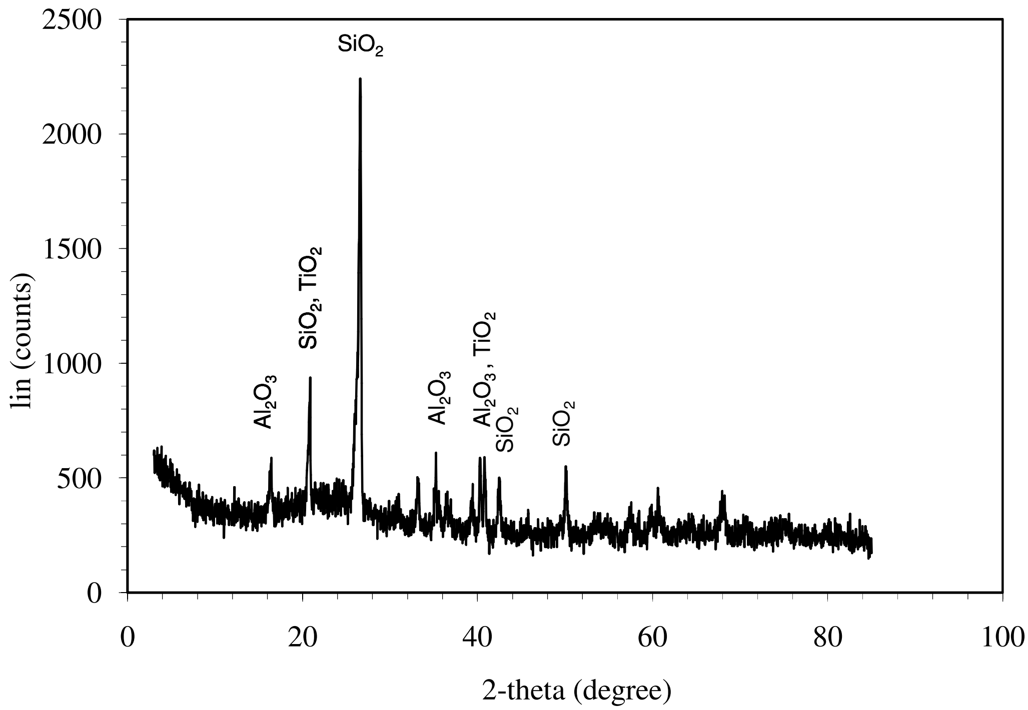


Fig. 3.9 X-Ray Diffraction pattern of fly ash.

It can be observed that the combined percentage of SiO_2, Al_2O_3 and Fe_2O_3 exceeds 70%. Moreover, the CaO content is also much less than 10%. In addition, setting did not take place when the fly ash was mixed with water and kept for 7 days. Based on these observations, the fly ash is classified as Class F (ASTM C 618 – 08a).

The X-Ray Diffraction (XRD) pattern of the fly ash, obtained from powder diffraction, is shown in Fig. 3.9. It shows a very prominent peak at $2\theta = 26.6°$ indicating SiO_2 to be the main constituent in the fly ash. The other minerals in the fly ash identified from XRD peaks are Al_2O_3, Fe_2O_3 and TiO_2.

3.3.3 Index Properties of Fly Ash

The grain size distribution curve of fly ash obtained by combining data from wet sieving and sedimentation analysis is as shown in the Fig. 3.10. It could be observed that the fly

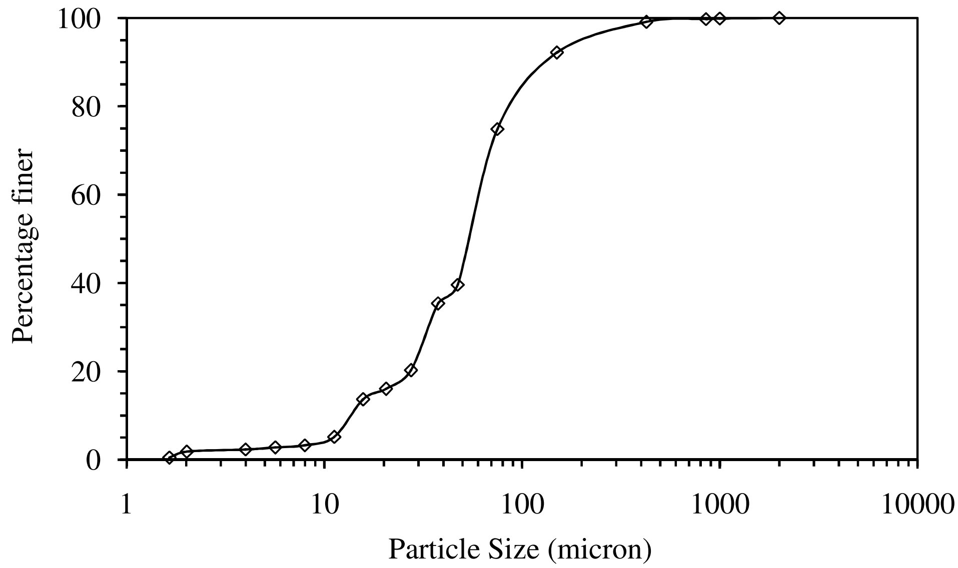


Fig. 3.10 Grain size distribution of fly ash.

ash is silt-sized, and the gradation curve is well-graded. The D_{90}, D_{50} and D_{10} of the test fly ash are 120μ, 51μ and 12μ respectively.

The liquid limit of the fly ash is found to be 27%, as tested in the cone penetration test. Fly ash being non-plastic, the other Atterberg limits viz. plastic limit, plasticity index and shrinkage limit were not determined.

3.4 PLANNING OF EXPERIMENTS

Five series of experiments have been carried out under the present investigation, to study the physico-chemical and engineering behaviour of expansive soil mixed with fly ash and treated with lime. The details of these test series are presented in Table 3.6.

In each test series, fly ash was added with expansive soil at different percentage (i.e. 0%, 20%, 40%, 60%, 80%). For comparison purpose, tests were also carried out on 100% fly ash. All these six soil samples were added with different percent of lime (0, 1, 3, 5, 9, 13 and 17 per cent by weight of dry soil). Each of the samples thus prepared were mixed

Table 3.6 Details of tests.

Test series	Soil	Soil designation	Lime added (%)	Curing period (day)	Test detail
1	100%ES	F0	0, 1, 3, 5, 9, 13, 17	0, 1, 3, 7, 15, 30	(i) Plasticity characteristics (*LL*, *PL*) (excluding F100) (ii) Specific surface area (for untreated soil only)
	80%ES + 20%FA	F20			
	60%ES + 40%FA	F40			
	40%ES + 60%FA	F60			
	20%ES + 80%FA	F80			
	100%FA	F100			
2	100%ES	F0	0, 1, 3, 5, 9, 13, 17	Nil	Compaction
	80%ES + 20%FA	F20			
	60%ES + 40%FA	F40			
	40%ES + 60%FA	F60			
	20%ES + 80%FA	F80			
	100%FA	F100			
3	100%ES	F0	0, 1, 3, 5, 9, 13, 17	Test started immediately after sample preparation. Curing takes place during the test	i) Free swell ii) Odeometer swell
	80%ES + 20%FA	F20			
	60%ES + 40%FA	F40			
	40%ES + 60%FA	F60			
	20%ES + 80%FA	F80			
	100%FA	F100			
4	100%ES	F0	0, 1, 3, 5, 9, 13, 17	Test started immediately after sample preparation. Curing takes place during the test	Consolidation
	80%ES + 20%FA	F20			
	60%ES + 40%FA	F40			
	40%ES + 60%FA	F60			
	20%ES + 80%FA	F80			
	100%FA	F100			
5	100%ES	F0	0, 1, 3, 5, 9, 13, 17	0, 1, 3, 7, 15, 30, 60, 90	Unconfined compressive strength
	80%ES + 20%FA	F20			
	60%ES + 40%FA	F40			
	40%ES + 60%FA	F60			
	20%ES + 80%FA	F80			
	100%FA	F100			

with desired quantity of water and was subjected to different curing periods (0, 1, 3, 7, 15 and 30 days) before the tests were carried out.

3.5 TEST METHODS

3.5.1 Tests for Physico-Chemical Properties

Specific gravity

Specific gravity for expansive soil, fly ash and their mixes is determined using specific gravity bottle as per IS 2720 Part 3 Section 1. Kerosene is used in case of expansive soil because of its high affinity to water. In case of lime-treated soil, soil and lime are properly mixed, sufficient water is added beyond saturation, and then kept for 2 hours. By this time, all CaO has been converted to $Ca(OH)_2$ and initial lime reactions has taken place with soil. The soil is then oven dried and then specific gravity is determined by the usual procedure.

Specific Surface Area (SSA)

Specific surface of the expansive soil - fly ash mixes is measured using (i) BET method and (ii) method described by Sridharan and Rao (1972). Sridharan and Rao's method is used in this study because of its simplicity. It does not require any sophisticated equipment – only a desiccator and a sensitive balance is required. It is based on the assumption that unimolecular layers of water would be formed around soil particles at partial pressure of 0.20. An aqueous solution of sulphuric acid with density of 1.4789 g/cc at 25°C is used to maintain the partial pressure. A small quantity of the soil (2-5 g) is measured to the accuracy of 0.0001 g and spread evenly on an aluminium dish. The dish is kept in a desiccator where an aqueous solution of H_2SO_4 of the specified density is kept at the bottom of the desiccator. The soil absorbs water vapour from the air inside the desiccator. The soil sample is kept there for 24 hours and weighed after every 24 hours until its weight stops increasing. The specific surface is obtained using the following formula:

$$S = w/M.N/10^4.A.10^{-16} = 3612w$$

Where,

w = equilibrium moisture content

N = Avogadro's number = 6.023×10^{23}

M = molecular weight of water = 18.016

A = area of a water molecule in square angstrom = 10.8

In the BET method (after Brunauer, Emmett, and Teller), adsorption of nitrogen gas by soil particles in monolayers is measured in a surface area analyzer. The analyzer uses BET adsorption isotherms for reporting the surface area. An analyzer of make Beckman Coulter and model SA3100 was used.

Scanning Electron Microscopy (SEM)

Scanning electron microscopy is done to find out the microstructure and texture of the expansive soil and fly ash. A Leo 1430vp equipment is used. The samples are made free of moisture by subjecting them to high vacuum, and then mounted on substrate before putting them in the microscope. Energy dispersive X-ray spectroscopy was performed in the same equipment. So, SEM gives the microstructure as well as elemental composition of test soils.

Cation exchange capacity (CEC)

CEC is the measure of a soil to retain readily exchangeable cations which neutralize the negative charge of soils. *CEC* was determined using an ammonium replacement method.

This method involves saturation of the cation exchange sites with ammonium, equilibration, removal of the excess ammonium with ethanol, replacement and leaching of exchangeable ammonium with H^+ ions from HCl acid (Horneck, et al.1989). The method detection limit is approximately 1.0 meq/100 gm and is generally reproducible within ± 10%. 10 g of soil is treated with 50 ml of 1N ammonium acetate solution and shaken for 30 minutes. The suspension is transferred on to a funnel with Whatman No.5 or equivalent filter paper. The sample is leached with 175 ml of ammonium acetate solution. The cations are leached away and their places are taken by ammonium ions. The excess ammonia is removed by leaching the sample with 200 ml of 95% ethanol. The sample is then leached with 225 ml of 0.1N HCl. The exchangeable ammonium is leached away. The leachate is collected and made to 250 ml adding de-ionised water. The leachate is analyzed for ammonium concentration in a liquid chromatograph. An ion chromatograph (model 792 BASIC IC of Metrohm make) was used to analyze the ammonium ion concentration. Using the ammonium concentration, the CEC is calculated from the following formula:

CEC in meq per 100 g of soil = (mg/L of NH_4^- in leachate)×0.25/14×100/sample size (g)

Initial consumption of lime (ICL)

The lowest percentage of lime that gives a pH of 12.4 is the approximate lime percentage for stabilizing the soil (ASTM 6276). Soil samples are mixed with different percentage of limes. A slurry is made with CO_2-free distilled water at soil water ratio of 1:5. The slurry is agitated intermittently and after 1 hour the pH value is measured with the help of standard electrode pH-meter. There may be soils in which the pH is greater than 12.4. If this occurs, the lowest percentage of lime where the higher pH value does not rise for atleast two successive test samples, at increasing lime percentages, is selected. This pH value is the maximum for the saturated solution of $Ca(OH)_2$ in CO_2 free water at 25°C.

Solubility of lime decreases at higher temperatures. So, pH value of 12.4 may not be achieved at higher temperatures. In this study, the measurement temperature was 30°C and above, so the maximum pH value attained was less than 12.4°C.

3.5.2 Tests for Composition

Elemental composition

Energy Dispersive X-Ray (EDX) was performed during Scanning Electron Microscopy to find elemental composition of expansive soil and fly ash. It gives estimates of atomic percentages as well as percentage by weight of the elements present in the sample.

Mineralogical composition

X-Ray Diffraction (XRD) tests were conducted to identify the minerals present in the expansive soil and fly ash. A Bruker AXS D8 machine was used for the purpose. Major peaks in the powder diffraction patterns were identified by comparing with standard powder diffraction patterns of minerals by search and match method.

3.5.3 Tests for Index Properties

Grain size distributions of samples were made as per IS: 2720 Part 4, 1985. Dry sieving was done in case of expansive soil. Wet sieving was not done for ES as these particles absorb water, swell and become sticky. Sedimentation analysis was not done on the sub-75µ fraction of ES as swelled particles can give erroneous particle size. Moreover, ES suspension appears like gel and remains stable for a long time.

In case of fly ash, the grain size analysis for the size range above 75 µ was done by wet sieving and that for the range below 75 µ was done by sedimentation analysis using hydrometer method.

Liquid limit (*LL*) and plastic limit (*PL*) tests are done as per IS:2720 Part 5, 1985. The *LL* tests on fly ash-clay mixtures were done initially using both Cassagrande's apparatus and cone penetration methods. The Cassagrande's method involves imparting an impact load to the wet soil, while in the cone penetration method, a metal cone is allowed to push in to the soil under its own load. The differences between the measurements from the two methods were observed and investigated. Data from *LL* tests on other soil types viz. ES and river silt mixes, red soil, potters clay were used for studying the differences between the two methods. In both methods, the soil is dried from higher water content to a lower water content to test at different water contents. This is done to avoid non-uniform mixing of water in clayey soil if water is added to increase water content from a lower value to a higher value. The details of the investigation is presented in Chapter 4.

Further *LL* tests on expansive soil and soil-fly ash mixes treated with lime are done using Cassagrande's apparatus. *LL* tests on fly ash samples are done using cone the penetration method.

Plastic limit tests are done by thread-rolling method. Fly ash being non-plastic, no *PL* tests were done on it.

3.5.4 Tests for Engineering Properties

Compaction behaviour

Light compaction tests are done on samples of expansive soil, fly ash and their mixtures. These tests are conducted using a small compaction apparatus developed and verified by Prashanth (1998). The schematic diagram of this compaction apparatus is produced in Fig. 3.11.

The main features of this mini-compactor vis-à-vis the Standard Proctor (light compaction) test are:

i. For each test, approximately 200 g of dry soil is required and for each set of 6 moisture content conditions, approximately 1.2 kg of soil is required. For Standard Proctor test, approximately 2.5 kg of soil is required.

ii. Each test of each set is done with fresh soil, i.e. soil is not re-used.

iii. The soil is compacted in three layers using a 0.8 kg drop weight falling through 160 mm, as compared to 2.6 kg weight falling through 310 mm. Number of tampings in each layer is 45 compared with 25 tampings in Standard Proctor test.

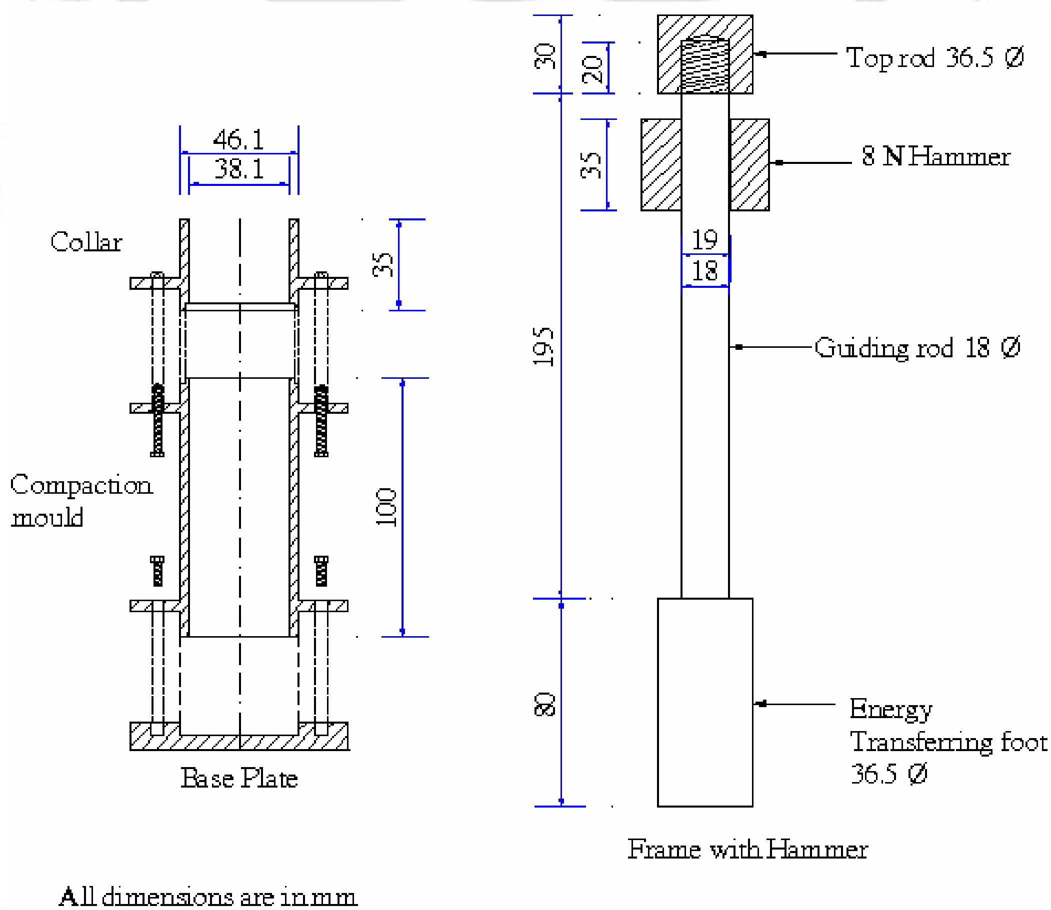


Fig. 3.11 Compaction apparatus (Prashanth, 1998).

iv. There is substantial saving in test material. Since sample mixing time is more than the testing time for most of the tests, this results in substantial reduction in sample preparation time.

v. Unlike the Proctor test, the sample in the mini compaction device is completely confined at its top by the hammer foot, the bulging during compaction is prevented.

vi. The drop weight does not fall on the soil directly, but the load is transferred through a foot. Thus some amount of kinetic energy of the falling weight is lost by way of sound, vibration, etc. For this, more energy is required for compaction of the same quantity of soil using this test (1.49 J per cm^3 of compacted soil) compared to that required in Proctor's test (0.592 J per cm^3 of compacted soil).

vii. The internal diameter of the compaction cylinder is 38 mm. Hence, 38 mm diameter compacted samples can be obtained from this test for other tests such as UC test, triaxial test, etc.

The moisture content vs. dry density curves are plotted to find the optimum moisture content (*OMC*) and maximum dry density (*MDD*) in the same fashion as in the case of Proctor test.

For verification of the miniature compaction apparatus, some tests were carried out using the Standard Proctor compaction and the mini compaction apparatus. It may be observed that the compaction curves from both the devices almost match each other (Fig. 3.12). It establishes that the mini compaction apparatus can be used satisfactorily in lieu of the Standard Proctor test. Since this mini compaction apparatus requires much less soil, time and effort compared to the conventional one, in the present study it is used for all compaction tests.

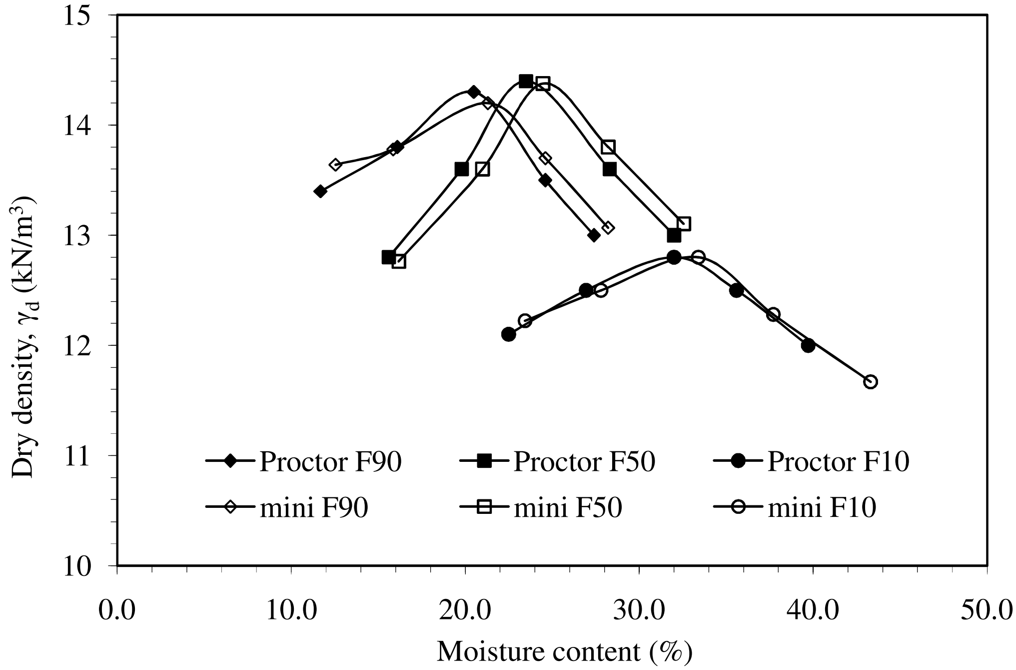


Fig. 3.12 Comparison of Standard Proctor and mini compaction test results.

At first, OMC and *MDD* are determined for the expansive soil-fly ash mixes without addition of lime, with fly ash contents of 0%, 20%, 40%, 60%, 80%, and 100%. Then *OMC* and *MDD* are found out for the lime-treated soil-fly ash mixes for the same fly ash contents and with lime contents of 1%, 3%, 5%, 9%, 13% and 17%. For lime treated soils, a 1-hour period was kept between the start of water mixing and start of compaction test to allow for the initial lime reactions with soil to take place. This delay was adopted because it is not possible to perform compaction tests exactly immediately after mixing. The process of mixing water to the prepared soil takes about 15 minutes. As water is sprayed gradually for mixing, some particles will get more reaction time than others if compaction is done immediately, i.e. after about 15 minutes from the start of mixing. If tests are done with some delay, say of 1 hour, the difference of reaction time of different portions of the sample will be leveled off. This is also expected to contribute to the uniform spread of moisture in the sample. To find out the differences in *OMC* and *MDD* due to delay, some tests were also done without delay.

The ES-FA mixes with 1% and 3% lime were tested immediately after mixing (within 15 minutes of starting mixing) and also with a delay of 1 hour to see the difference in compaction due to delay effect. The results are shown in Fig.3.13 and Fig.3.14.

In the case of samples with 1% lime content, the *OMC* was found to be slightly higher for 1 hour delay than immediate values. This was contrary to the findings of Osinubi and Nwaiwu (2006). In case of 3% lime addition, *OMC* of 1 hour delay tests were slightly higher in case of 0-40% fly ash contents. For higher fly ash soils, the results followed findings of Osinubi and Nwaiwu (2006). The differences in *MDD* for immediate and 1h delay tests are marginal. In both cases of 1% lime and 3% lime, the *MDD* was slightly less in case of delayed compaction, except for mixes with higher fly ash contents with 3% lime.

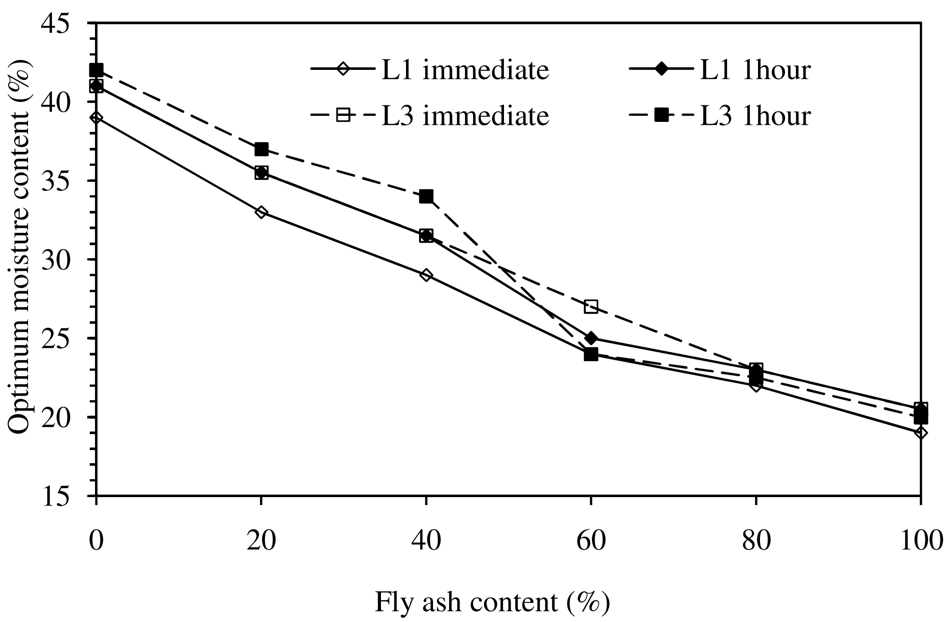


Fig. 3.13 Delay effect on OMC for ES-FA mixes with 1% and 3% lime.

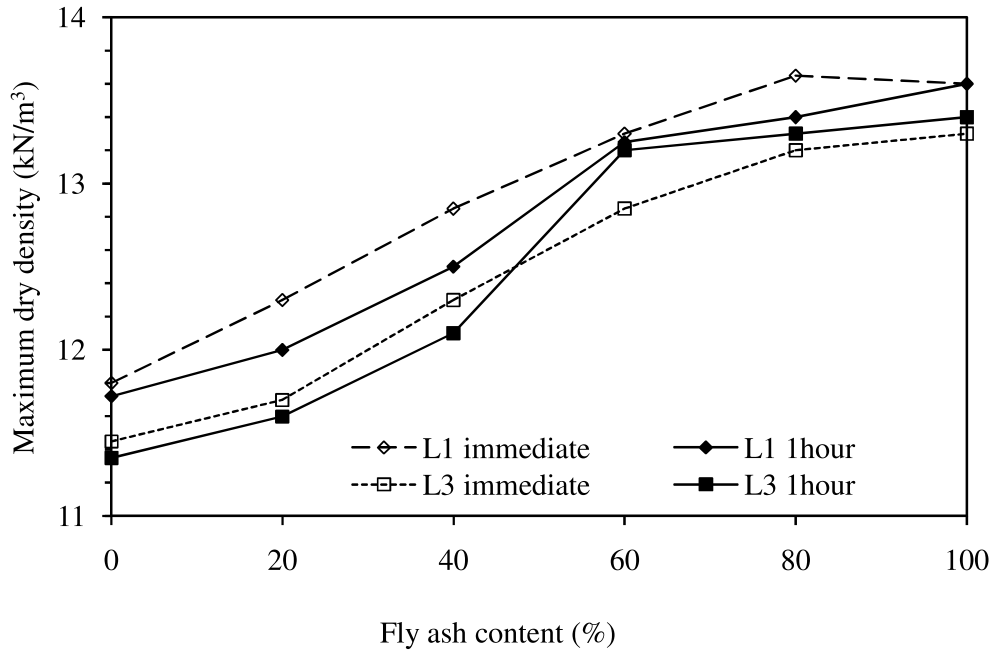


Fig. 3.14 Delay effect on *MDD* for ES-FA mixes with 1% and 3% lime.

Consolidation behaviour

One-dimensional consolidation tests are done on mixes of expansive soil and fly ash with or without lime treatment. The tests were done according to IS:2720 (Part 15) – 1986. Fixed ring type oedometers were used. All specimens were prepared at *OMC* and *MDD*. The pressures applied were 10, 20, 40, 80, 160, 320, 640 and 1240 kPa above a seating pressure of 5 kPa. Each pressure was sustained for 24 hours and time-deformation records obtained. Total settlement was recorded for each pressure, from which pressure-settlement characteristics were obtained. Compression index of each of the mix was determined from the voids ratio versus log(pressure) plots.

Swell behaviour

Free Swell Indices of soil-fly ash mixes with and without lime treatment are found out as per IS:2720 (Part 40) – 1977. The free swell index is the ratio of volume of 10 g of soil submerged in water in an 100 ml measuring cylinder to the volume of same quantity of soil submerged in kerosene. Swelled volume is also calculated and reported as volume per g of swelled soil without comparing with volume in kerosene.

Swelling was also observed under a seating pressure of 5 kPa in samples remoulded and mounted on oedometers as per IS:2720 (Part 41) – 1977. Samples were prepared at *OMC* and *MDD*. The increase in sample height has been recorded until swelling is stabilized. The swell percentage is then determined as the ratio of increase in sample thickness to the original thickness expressed in percentage. The soil after exhibiting the maximum swell has been incrementally reloaded to bring it back to its original thickness of 20 mm. The cumulative load at which the swelled soil attains the original thickness has been used to determine the swelling pressure of the soil.

Strength

Unconfined compressive tests are conducted as per IS: 2720 (Part 10) – 1991. Samples of soil-fly ash mixes with or without lime treatment are prepared at *OMC* and *MDD*. The samples with lime are cured for different periods (0, 1, 3, 7, 15, 30, 60 and 90 days), to see the effect of curing on strength. The 38mm diameter specimens are compressed at a strain rate of 1.25 mm/min. The peak compressive stress attained by a specimen is considered as its unconfined compressive strength, and the corresponding strain is considered as the failure strain.

3.6 SUMMARY

In this chapter the test materials and their characterization have been described. The method of sample preparation and test procedures are discussed. The details of planning of the experiments have been presented. The results of the experiments are analysed and discussed in the subsequent chapters.

CHAPTER 4

PLASTICITY BEHAVIOUR

4.1 INTRODUCTION

Plasticity behaviour of a soil primarily depends on the quantity and quality of the fine grained component present in it. This behavior can be assessed using index properties such as liquid limit (*LL*), plastic limit (*PL*) and plasticity index (*PI*). Besides, these index properties can be correlated with the engineering properties such as compressibility, shear strength, permeability etc. (Sridharan and Prakash, 2000).

The expansive soils being highly plastic are difficult to handle. This problem can be overcome by amending the soil through addition of non-plastic soil and chemicals such as lime, fly ash, cement etc. This chapter presents the results of the experiments aimed at understanding the plasticity behaviour of a highly expansive soil (ES) amended with the combined addition of fly ash (FA) and lime. The influence of the time-dependent effects of the additives on the plasticity characteristics of the expansive soil are also brought out. Presently there are two different methods i.e. percussion method and cone penetration method being used for determining the liquid limit. Initially, a critical evaluation of these two methods has been carried out.

4.2 EVALUATION OF LIQUID LIMIT DETERMINATION METHODS

The two well established methods for the determination of liquid limit (*LL*) are percussion method and cone penetration method. The percussion method developed by Casagrande, determines *LL* as the water content at which the soil slumps and closes a cut groove under a specific dynamic impact loading (Casagrande, 1958). The major limitation of percussion method is that it is empirical and sometimes difficult to cut the groove in low plastic soils (i.e. silty soils). The cone penetration method defines *LL* as the water content at which a cone of particular cone angle and mass induces static penetration at a specified time rate into the soil mass (BS 1377; IS 2720 Part 5; IS 11196; Christaras, 1991; Prakash and Sridharan, 2006). Though this method is simple and consistent in determining *LL*, several researchers have highlighted that the obtained *LL* may not be the true representation of soil plasticity as it is predominantly based on the undrained shear strength of the soil (Silvestri, 1997; Sridharan and Prakash, 1999; Prakash and Sridharan, 2006). Therefore, this method may yield *LL* even for a non-plastic soil.

As discussed above, the principle based on which *LL* is measured by using the two methods are entirely different. Therefore, several researchers have compared the *LL* values determined by both the methods (Christaras, 1991; Leroueil and Le Bihan, 1996; Feng, 2001; Prakash and Sridharan, 2006). One of the general observations is that the two methods do not yield comparable results for high *LL* soil (> 100 %). In most of the cases, percussion method gives higher *LL* as compared to cone penetration method. However, both the methods yield almost identical results for soils with low *LL*. In some cases, even the high *LL* soils also gave similar results (Leroueil and Le Bihan 1996; Feng 2001). Though the reported literature discusses a few reasons for the above mentioned observations, there is a need to understand clearly the parameter which influences the discrepancy for high *LL* soil. One such parameter which has not been extensively investigated is the swelling characteristics.

The results reported in the literature for *LL* using percussion and cone penetration methods were assimilated and plotted, as depicted in Fig. 4.1. In the figure, LL_P and LL_{CP}

denotes liquid limit obtained by using percussion and cone penetration methods, respectively. It is quite explicit from the figure that there is a difference between LL_P and LL_{CP}. For critical evaluation, the graphical plot is divided into 4 different ranges such as (a) $LL<50$; (b) $50< LL<100$; (c) $100< LL<300$; and (d) $LL>300$. For better understanding, results for $LL<100$ have been re-plotted as depicted in Fig. 4.2. It can be noted that both the methods match very well for $LL<50\%$, beyond which LL_P is moderately greater than LL_{CP} upto 100%. For majority of the data > 100%, the difference is high with an exception of a very few data. The difference between LL_P and LL_{CP} is maximum for the range greater than 300.

The difference between LL_P and LL_{CP} is mainly attributed to the governing mechanism and the mobilized shear resistance of the soil, in both the methods (Sridharan and Prakash, 2000; Prakash and Sridharan, 2006). However, there are not many discussions and reasoning presented in the literature for higher value of LL_P as compared to LL_{CP} for high LL soils. The present study attempts to explain this aspect based on three factors (a) manner of loading (b) viscous shear resistance and (c) water retention characteristics of the soil.

a) Manner of loading: In percussion method, soil is subjected to repeated dynamic impact loading due to the tapping of the metallic cup against the solid base. Such a loading creates undrained conditions in the soil, followed by the tendency to flow out towards the groove (open space) created in the soil (Atkinson and Bransby, 1978). This phenomenon is similar to the dilatancy test conducted for the preliminary investigation of the soil, where the repeated tapping of the soil pat on the palm results in bleeding. The flowable tendency of water results in slumping of the soil mass at the point where it is subjected to tapping.

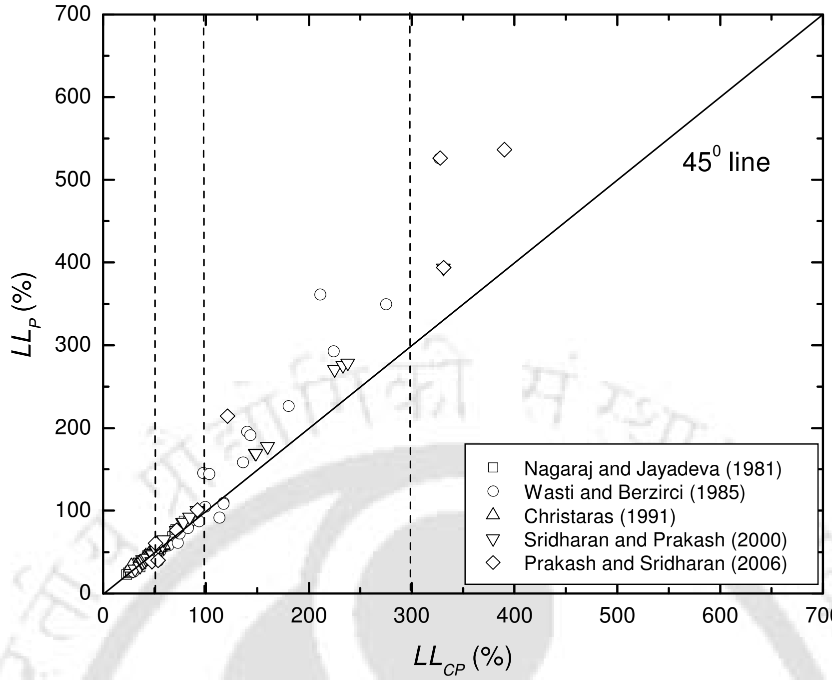


Fig. 4.1 Comparison of percussion and cone penetration methods reported in the literature.

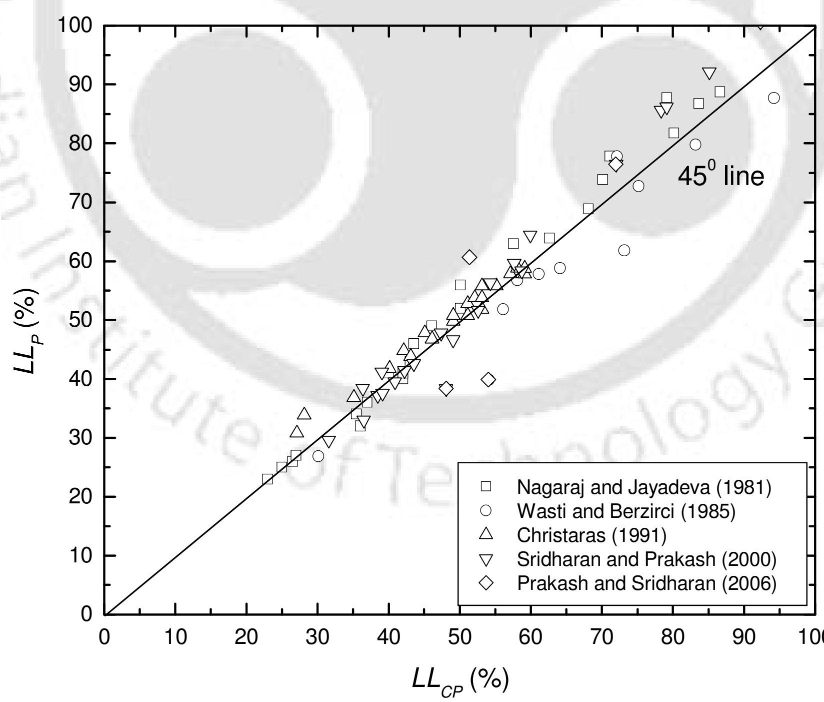


Fig. 4.2 Comparison of *LL* values (≤ 100%) reported in the literature.

On the contrary, the cone penetration method follows a static undrained penetration within a very small time of 5 seconds (BS 1377). The undrained condition developed in the soil would be more severe in the case of cone penetration as compared to percussion method. Since the dissipation of undrained condition is easier in the case of low *LL* soil (permeability is higher), the slump failure of the soil and achieving required penetration in the soil takes place at comparable water content. In the case of soil with high *LL*, there is hardly any time for the dissipation of undrained condition in the case of cone penetration test resulting in the early loss of resistance to penetration at water content lower than LL_P. However, for the percussion method, the time required for 25 blows is 12.5 seconds (Casagrande, 1958), which is more than that in the cone penetration method and hence, there is a chance for slow dissipation of undrained condition in percussion method. Therefore, the slump failure of the soil takes place at higher water content resulting in higher value of LL_P.

b) Viscous shear resistance: In general, both the methods induce shearing in the soil leading to mobilization of viscous shear resistance. It should be mentioned here that *LL* refers to water content at which soil has an undrained shear strength of about 2.5 kPa (Casagranade, 1932). In the case of cone penetration, the soil mass is subjected to cone penetration in 5 seconds which results in low mobilized viscous shear resistance. While in percussion method, the loading is at a slower rate, resulting in higher magnitude of mobilized viscous shear resistance. This indicates that greater amount of water content is required to reduce the viscous shear resistance in percussion method than cone penetration method. The role of viscosity is more predominant for high plastic soils (high *LL* soils) and hence the difference between LL_P and LL_{CP} will be more due to the difference in mobilized viscous shear resistance. For low plastic soils (low *LL* soils), viscosity being low, the difference in mobilized shear resistance between both the methods is marginal and hence, *LL* obtained by both the methods are identical.

c) Water retention property of the soil: It is an established fact that the water retention of high *LL* soil is greater than low *LL* soil (Marinho, 2005). A high water retention property of soil results in less flowability and better resistance to slump failure. Hence, high water content is required to cause slump failure in the case of percussion method. However, higher water retention property results in quick loss of resistance to penetration in the case of cone penetration method. This enables the cone to achieve the required penetration at a water content less than LL_P for high *LL* soil. It must be noted that such a discrepancy would be negligible for a low *LL* soil due to its less water retention capacity. Hence, LL_P and LL_{CP} are comparable for low *LL* soils.

In general, *LL* represents flowable consistency of the soil where its shear strength becomes negligible. Following this, it is clear that LL_P would yield a true representation of water content resulting in flowable consistency and better representation of plasticity characteristics. This statement is in agreement with the observations reported by previous researchers (Sridharan and Prakash, 2000). Therefore, there is a need to revisit and re-evaluate the methodology adopted for cone penetration method for obtaining *LL* values comparable to LL_P in the case of high *LL* soils. With this in view, effort has been made to re-evaluate LL_{CP} based on free-swell index property of the soil.

It can be noted from the literature that most of the soils having LL>100% contain swelling clay minerals (Prakash and Sridharan, 2004). Based on this, free-swell index, which is a simple and easily measurable soil property, has been used in this study for re-defining the cone penetration limit of high *LL* soils.

A locally available non-swelling red soil (RS) and the commercially available expansive soil (CS) were used for re-evaluation of cone penetration method. These soils are different from the soils used in the main study of this thesis (i.e. performance improvement of expansive soil). The properties of these soils are summarized in Table 4.1 along with the soil classification (ASTM D 2487). The values of *LL* reported in this table correspond to percussion method. It can be noted that both the soils exhibit entirely different characteristics and belong to two different classes of soils. For the sake of completeness, properties of other soils and FA used for reevaluation study are also presented in Table 4.1.

The two soils RS and CS were mixed in different proportions to obtain synthesized soils with wide range of *LL* and *FSI* characteristics. The details of mix proportions and designations of the soils are listed in Table 4.2 along with its *FSI*. It must be noted that only particles finer than 0.425 mm present in the parent soils were used to obtain the synthesized soils. The synthesized soils were used for *LL* determination by percussion method and cone penetration method by following the procedure reported in the literature (ASTM D 4318; BS 1377; IS 2720-part V; IS 11196). The cone employed in this study has a mass of 80g and a cone angle of 30^0 (BS 1377). For the same time duration (5 sec.), the penetration of the laboratory cone will be different for different water content due to the difference in shear resistance of the soil sample. Also, for the same water content shear resistance offered by different soils will be different. This is the basic principle of laboratory cone penetration discussed in this study. All the synthesised soils were then subjected to *PL* determination by thread rolling method (ASTM D 4318).

Table 4.3 presents the details of percussion and cone penetration *LL* values obtained for all the soil samples. For the sake of completeness, the *PL* and *PI* values are also listed in the table. The LL values obtained by both the methods have been plotted together as depicted in Fig. 4.3. It can be noted that for all the soil samples LL_P is greater than LL_{CP} and the difference increases with an increase in percentage of CS. However, both the methods yield identical LL for low LL soil such as S11. This observation confirms with those reported in the literature and as presented in Figs. 4.1 and 4.2. This indicates that cone penetration method underestimates the LL of soils containing expansive clay minerals or active clay minerals. To obtain a unique LL by both the methods, a modified penetration value has been derived for high LL soil based on free-swell index property of the soil as discussed in the following section.

Linear trends have been fitted for water content versus cone penetration response as depicted in Fig. 4.4. The details of these linear fits are presented in Table 4.4, along with regression coefficient R^2. Based on these linear equations, the penetration value corresponding to LL_P (designated as h_m) for each soil sample is determined and listed in Table 4.4. Further, h_m is plotted as a function of FSI as shown in Fig. 4.5. It can be clearly noted from the figure that h_m increases with FSI up to a maximum of 300 % and remains almost constant for $FSI > 300\%$.

Based on this observation, modified penetration limits have been set for soils as presented in Table 4.5. For all soils whose FSI is less than 300%, the required penetration value in cone penetration test is calculated based on the linear equation and for soils with FSI greater than or equal to 300%, the penetration value is fixed as 26 mm. This modified penetration limits were then used to re-calculate LL_{CP} of soils S1 to S11 by using the slope and intercept details presented in Table 4.4. The modified LL_{CP} values were compared with LL_P along with the original values as depicted in Fig. 4.6.

Table 4.1 Properties of the soils used in the evaluation of *LL* determination methods.

Soil Property	Red soil (RS)	Expansive Soil 1 (CS)	Fly Ash (FA)	Expansive Soil 2 (ES)	Natural clay (NC)
G	2.62	2.24	2.123	2.82	2.51
% Sand size					
Coarse (4.75-2.0 mm)	14	0	0	0	0
Medium (2.0-0.425 mm)	20	0	1	0	5
Fine (0.425-0.075 mm)	10	0	24	6	10
% Silt size (0.075-0.002 mm)	27	28	76	54	45
% Clay size (<0.002 mm)	29	72	0	40	40
LL (%)	45	460	-	272	70
PL (%)	26	54	-	51	29
PI (%)	19	406	-	221	41
USCS Classification*	ML	CH	Class F**	CH	CH
Free-swell index (%)	10	1233	0	427	33
Minerals present	Quartz, Magnetite	Montmorillonite, Illite, Quartz	Quartz	Montmorillonite, Quartz	Illite, Quartz

* ASTM D2487   ** ASTM C618-08a

Table 4.2 Details of the synthesized soil samples.

Designation	% of RS	% of CS	*FSI* (%)
S1	0	100	1233
S2	10	90	900
S3	20	80	733
S4	30	70	600
S5	40	60	433
S6	50	50	367
S7	60	40	300
S8	70	30	233
S9	80	20	167
S10	90	10	67
S11	100	0	10

Table 4.3 Plasticity characteristics of the synthesized soil samples.

Soil Sample	LL_P (%) (a)	*PL* (%) (b)	*PI* (%) (a-b)	LL_{CP} (%)
S1	459.94	53.70	406.24	355.91
S2	385.26	52.44	332.82	300.24
S3	351.90	49.37	302.53	284.55
S4	331.42	43.35	288.07	266.48
S5	248.19	38.06	210.13	192.09
S6	203.81	32.72	171.09	166.09
S7	178.84	30.72	148.12	136.25
S8	129.17	30.28	98.89	100.95
S9	99.19	28.68	70.51	82.36
S10	60.42	27.65	32.77	57.51
S11	45.33	25.99	19.34	45.04

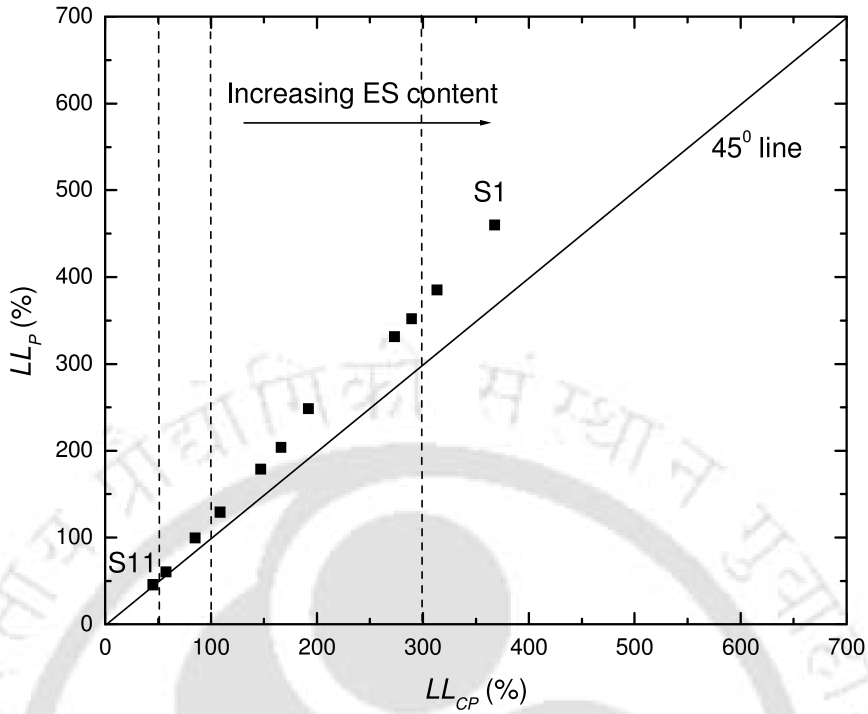


Fig. 4.3 Comparison of *LL* obtained using percussion and cone penetration method.

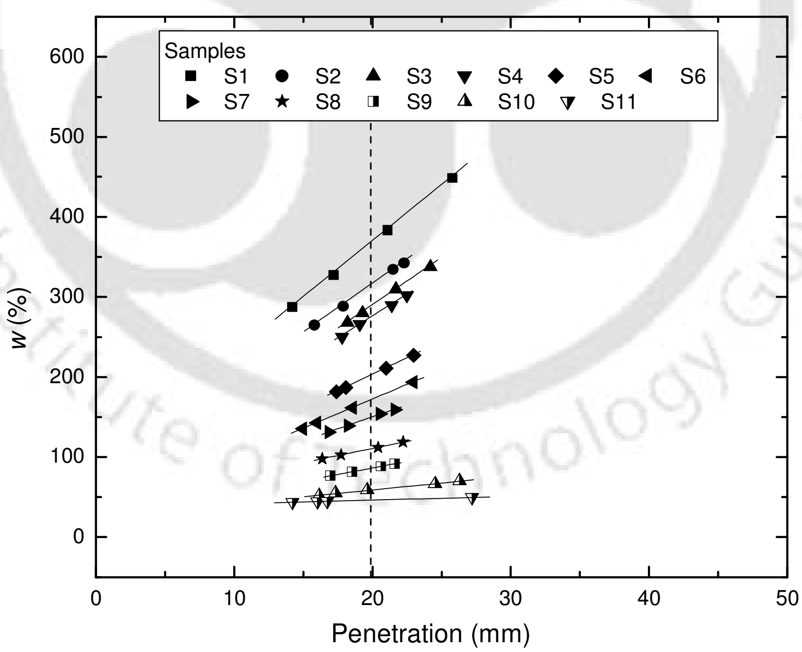


Fig. 4.4 Details of cone penetration and water content response.

Table 4.4 Modified penetration values corresponding to percussion liquid limit.

Soil Sample	Slope	Intercept (%)	R^2	h_m (mm)
S1	14.1	85	0.9927	26.6
S2	12.1	72	0.9895	25.9
S3	11.5	58	0.9987	25.6
S4	11	55	0.9632	25.1
S5	8.3	36	0.9778	25.6
S6	7	25	0.999	25.5
S7	6	28	0.9962	25.1
S8	3.7	35	0.9938	25.5
S9	3.2	21	0.9863	24.4
S10	1.71	24	0.9983	21.3
S11	0.46	36	0.9975	20.7

It can be clearly noted from Fig. 4.6 that LL_{CP} corresponding to modified penetration limit matches well with LL_P. This indicates that measuring LL_{CP} based on the modified penetration h_m value compares very well with LL_P, even for higher *LL* soils. Therefore, the proposed methodology would be helpful to obtain consistent and comparable values of *LL* by both the methods.

For the purpose of independent evaluation of the proposed methodology, the commercially available expansive soil (ES) and a non-plastic fly ash (FA), with its physical and mineralogical characteristics presented in Table 4.1, were mixed together in different proportions as listed in Table 4.6. In addition, a locally available natural clayey soil (S18) is also used for the validation. The mixed soils (S12 to S17) and S18 represent a wide range of plasticity characteristics and hence can be used to ensure the generality of the proposed methodology. By using the *FSI* values listed in Table 4.6, the modified penetration limits (h_m) were computed by following the guidelines listed in Table 4.5. The cone penetration *LL* values obtained corresponding to 20 mm penetration and h_m is compared with LL_P, as depicted in Fig. 4.7. It can be noted that LL_{CP} values obtained by using modified penetration matches well with LL_P. This indicates that the proposed methodology for determining cone penetration *LL* yields comparable results with the percussion method for all ranges of plasticity. Hence, the present study recommends the determination of *FSI* of the soil for deciding the appropriate penetration value, before employing cone penetration method for *LL* determination.

In view of the above findings, it can be said that apart from giving reliable values of *LL*, the percussion method is easier and more straight forward than the simpler looking cone penetration method. Further, all the results are for expansive soils or mixes where *LL* is better represented by the percussion method. Hence the *LL* values presented in the successive sections are obtained by percussion method.

Table 4.5 Proposed cone penetration value based on *FSI* .

FSI (%)	h_m (mm)
< 300	0.02**FSI* + 20
≥ 300	26

Table 4.6 Details of the soil samples used for validation.

Designation	% of FA	% of CS	*FSI* (%)	h_m (mm)
S12	0	100	427	26.0
S13	20	80	220	24.4
S14	40	60	140	22.8
S15	60	40	83	21.7
S16	70	30	81	21.6
S17	90	10	25	21.1
S18	Natural Clay		33	20.7

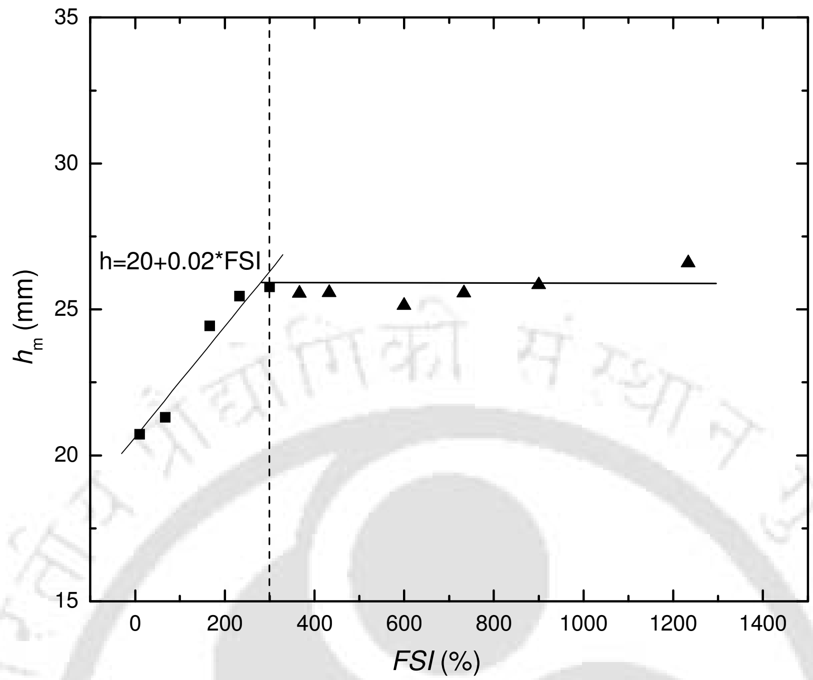


Fig. 4.5 Relationship between modified cone penetration and free-swell index.

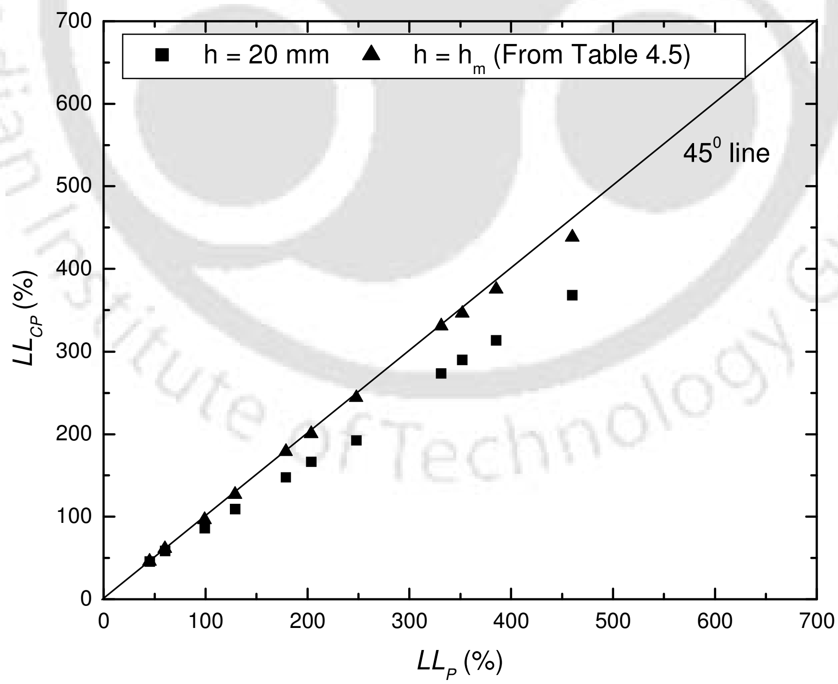


Fig. 4.6 Comparison of original and modified cone penetration liquid limit.

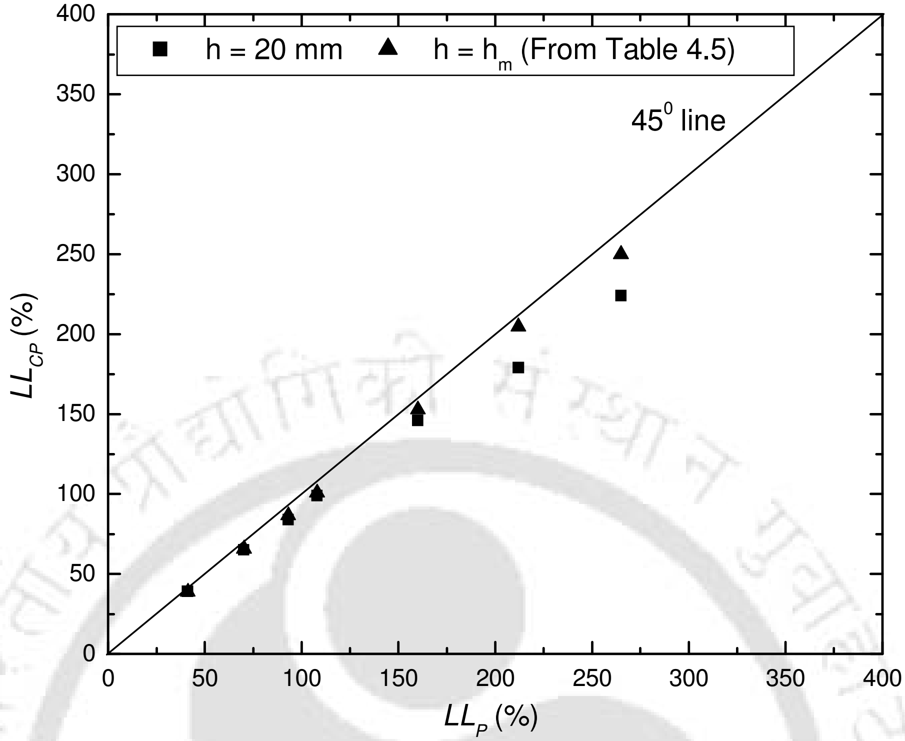


Fig. 4.7 Validation of the proposed methodology.

4.3 PLASTICITY OF ES-FA MIXES

The *LL*, *PL* of ES-FA mixes are presented in Table 4.7. It could be observed that values of these consistency limits (*LL*, *PL*) of the expansive soil have visibly reduced with addition of FA. Correspondingly the *PI* has also undergone marked reduction. With addition of fly ash, the fine clay minerals in the soil get replaced by the relatively coarse fly ash. The reduced clay content and thereby the reduced specific surface area (*SSA*), as shown in Fig. 4.8, results in reduced water holding capacity (adsorption) of the soil giving rise to reduced plasticity. However, the plasticity characteristics of these soils depicted in the plasticity chart (Fig. 4.9) indicates that even with addition of 80% FA, the soil still remains as high plastic clay (CH), and therefore is prone to swelling. This necessitates further treatment by chemical alteration of soil properties, which has been achieved through lime treatment as has been presented and discussed in the following sections.

Table 4.7 Consistency limits of ES-FA mixes.

Soil	Liquid limit (%)	Plastic limit (%)	Plasticity Index (%)
F0	265	42	223
F20	212	37	175
F40	169	31	138
F60	108	24	84
F80	70	23	47

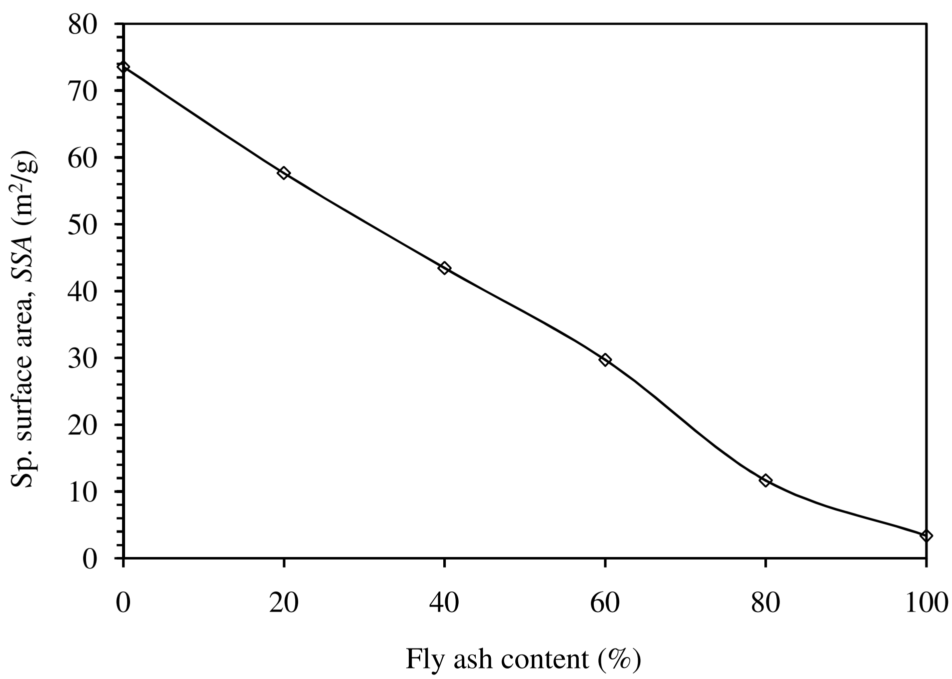


Fig. 4.8 Change in specific surface area with addition of fly ash in expansive soil.

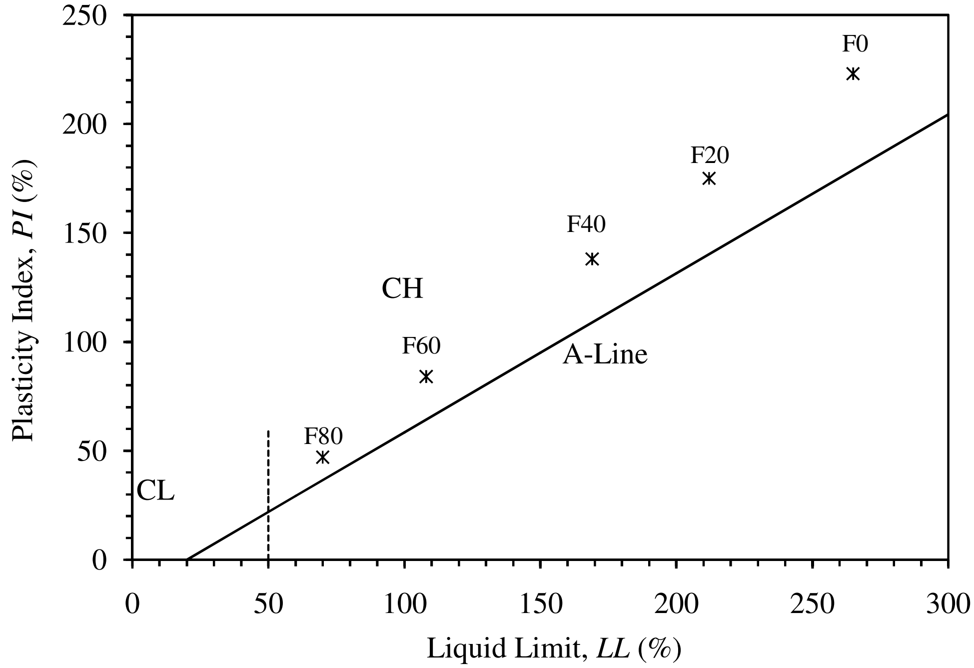


Fig. 4.9 ES-FA mixes indicated on the plasticity chart.

4.4 PLASTICITY BEHAVIOUR OF ES-FA-LIME MIXES

4.4.1 Liquid Limit

The change in *LL* of ES-FA mixes due to addition of different quantities of lime has been presented in Fig.4.10. It can be observed that with addition of lime, initially there is a steep reduction in *LL*, beyond which further change is marginal. This reduction is maximum for expansive soil (F0) and gradually reduces with increased content of FA. A very important point to be noted here is that *LL* of ES reduces at the most to 100% by addition of lime. It is only with the combined addition of FA and lime, *LL* can be reduced below 100%. Addition of lime increases the cation (i.e. Ca^{2+}) content giving rise to increased charge concentration in the porewater, as a result of which the quantity of porewater, held tightly onto the clay surface in order to neutralize the negative charge on it, reduces. This in turn reduces the overall water holding capacity of the soil leading to reduced *LL*. The water under the influence of surface charge, held on clay surface, is viscous in nature that induces plasticity into the soil, which makes it a difficult material in handling. With lime treatment the quantity of this viscous water reduces, the overall plasticity of the soil reduces thereby it turns out to be more workable. It could be observed from Fig. 4.10 that the lime content at which the *LL* ceases to reduce is about 3% in case of expansive soil (F0), while it is about 1% in case of ES-FA mixes.

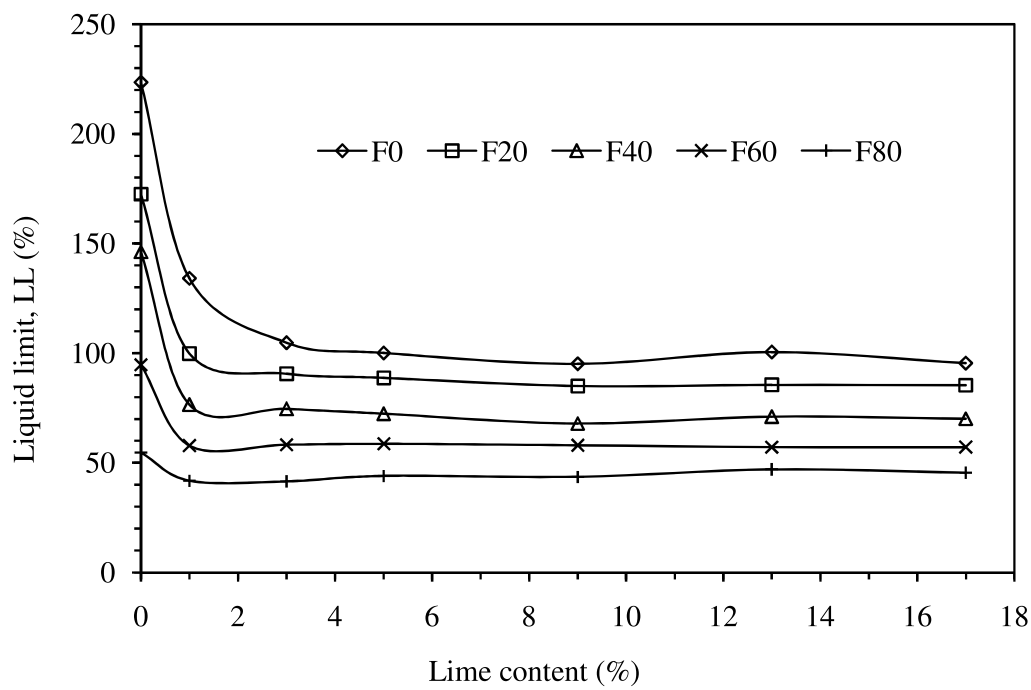


Fig. 4.10 Liquid limit Vs lime content for ES-FA mixes (without curing).

Similar tests were conducted on ES-FA mixes with curing period varying from 1 day to 30 days. The variations of *LL* with lime content and curing period for these soils are presented in Fig. 4.11 to Fig. 4.15. At higher lime content and long curing periods, the *LL* of the expansive soil has shown an increasing trend. This is more prominent in case of ES-FA mixes, as depicted in Figs. 4.16–4.21. This implies that with large amount of lime, some time-dependent phenomenon has taken place that has caused the increase in LL. The pH value of the soil-water system is found to have increased with increase in the quantity of lime added to it (Fig. 4.22). When the lime quantity is above the lime fixation point, the pH value of the soil-water-lime system goes beyond 12.2. Such high pH creates strong alkaline condition around the silica present in the soil that undergoes accelerated pozzolanic reactions yielding cementitious gels of Calcium Silicate Hydrate (C-S-H). This C-S-H gel being highly porous holds large quantity of water onto it. The water attached to the gel (called gel water) can be as high as 28% of the volume of the gel (Neville and Brooks, 1987). This gel water along with the physico-chemically adsorbed water, on the clay surface, has caused the increase in the *LL* of the soils. As the fly ash contains relatively large amount of silica, with increase in its quantity in the soil mix has produced increased quantity of silica gel, leading to increased *LL*. Similarly over a prolonged curing and thereby increased reaction period, more pozzolanic gels are formed giving rise to increased *LL*.

From Fig.4.11-4.15 it could be observed that with lime content increasing to 17%, the *LL* has slightly reduced. With large percentage of lime, formation of grits, in the ES-FA samples was observed. These grits are produced due to carbonation of $Ca(OH)_2$ by CO_2 in atmosphere, forming $CaCO_3$. At 17% lime content, the grit formation was more prominent. As the grits are coarse in nature, with their formation the water-holding capacity of the soil reduces leading to reduced *LL*.

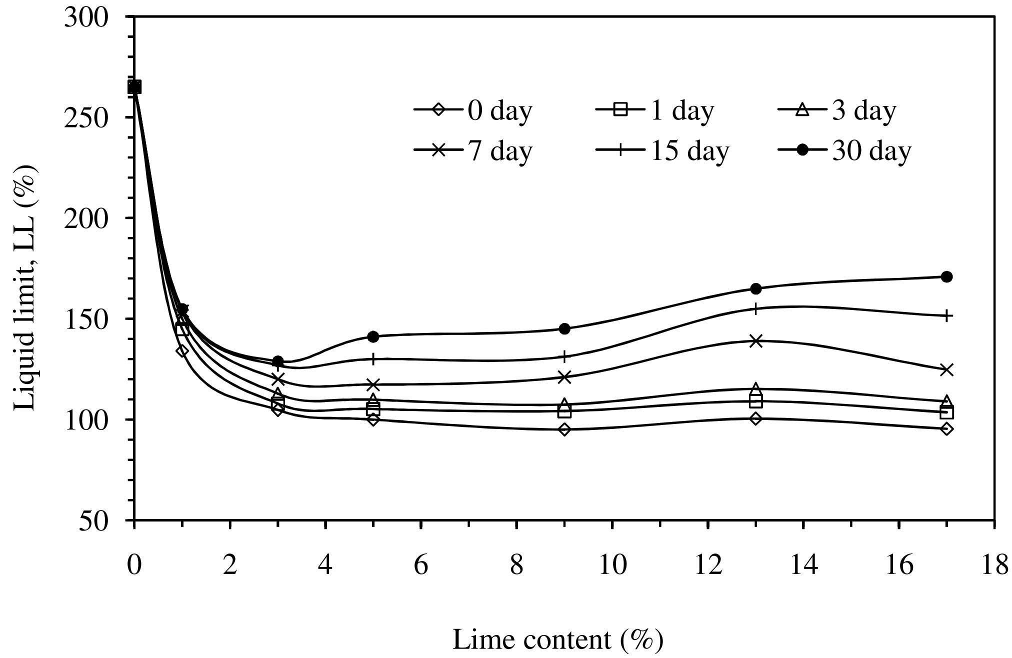


Fig. 4.11 Liquid limit Vs lime content for expansive soil (F0) at different curing periods.

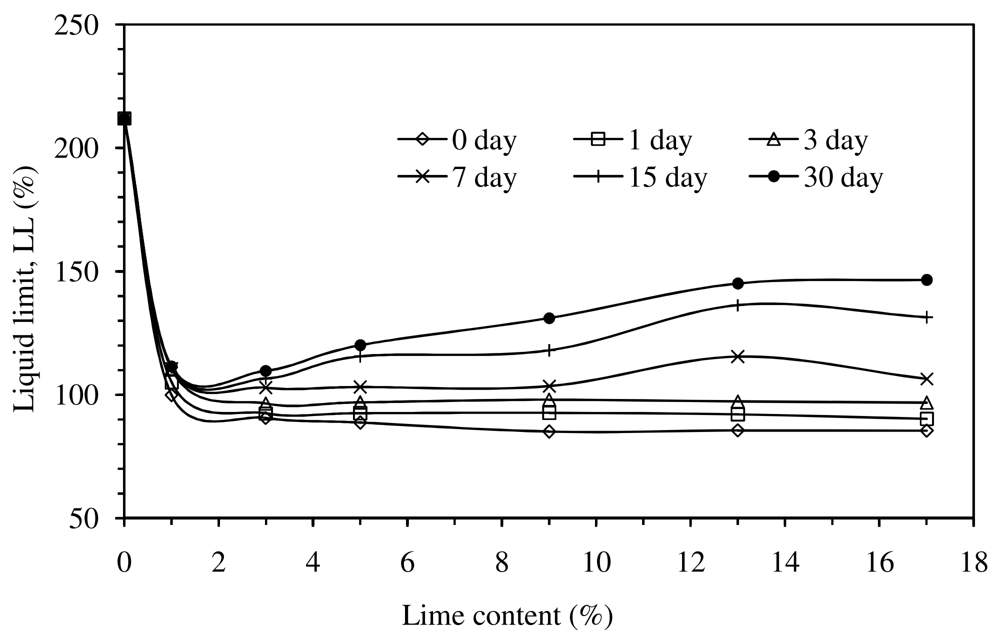


Fig. 4.12 Liquid limit Vs lime content for ES-FA mix F20 at different curing periods.

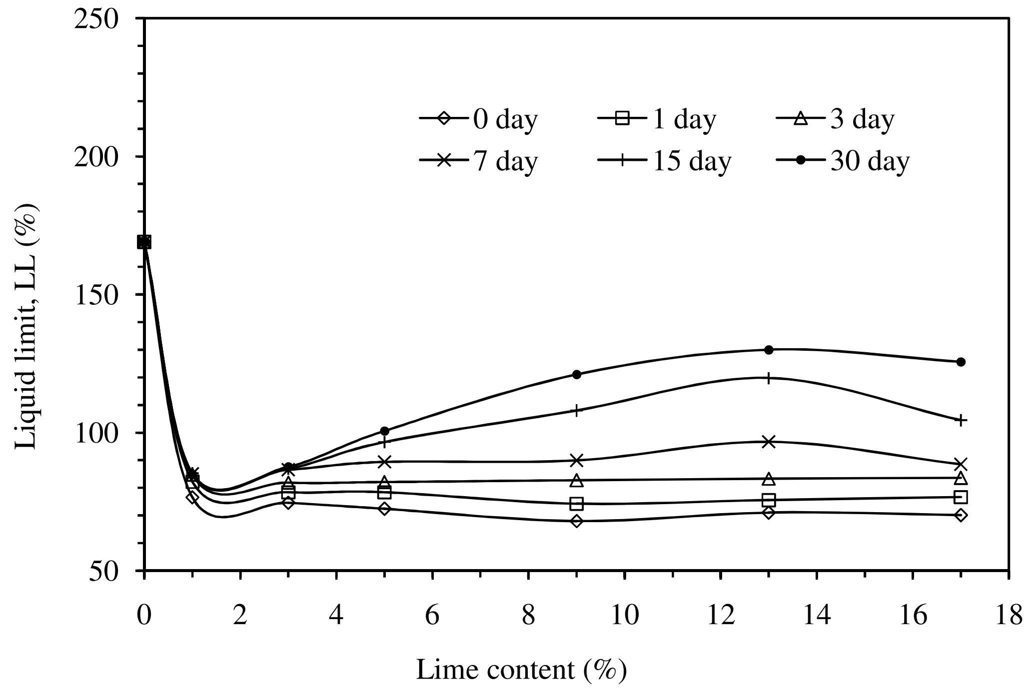


Fig. 4.13 Liquid limit Vs lime content for ES-FA mix F40 at different curing periods.

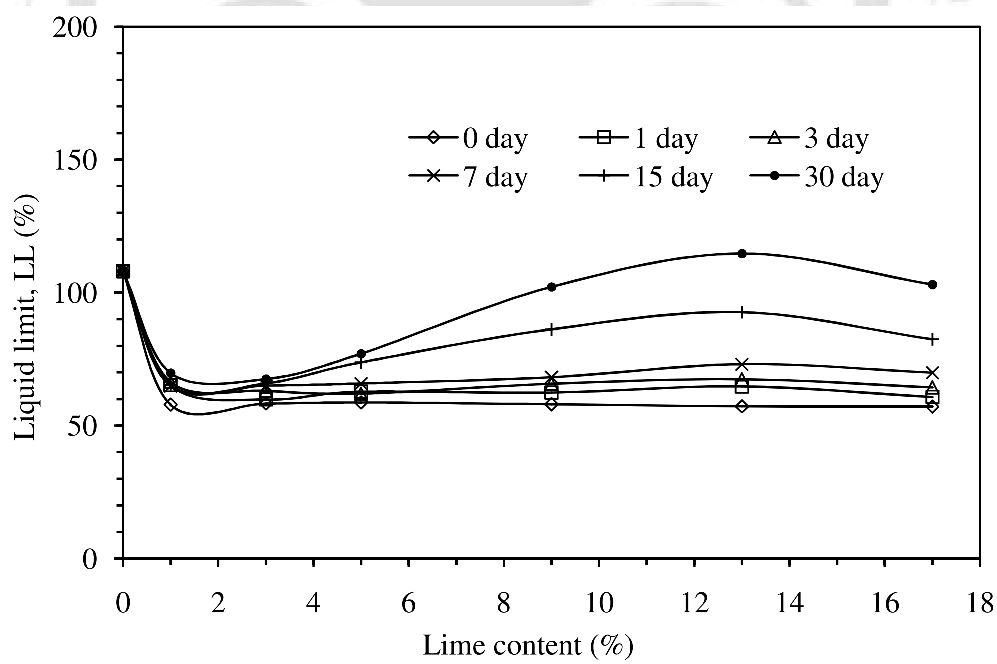


Fig. 4.14 Liquid limit Vs lime content for ES-FA mix F60 at different curing periods.

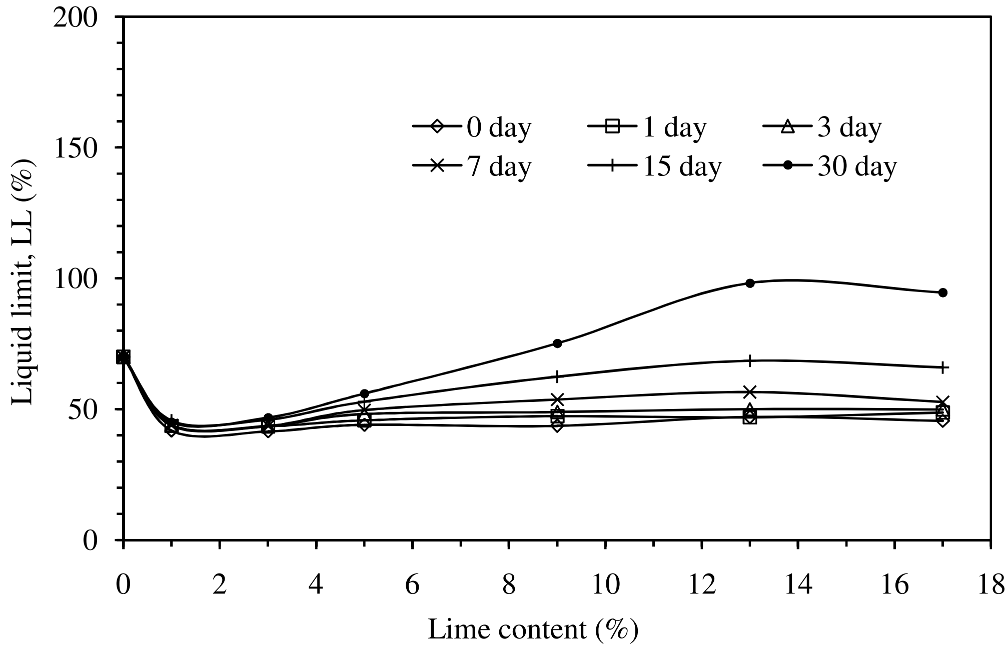


Fig. 4.15 Liquid limit Vs lime content for ES-FA mix F80 at different curing periods.

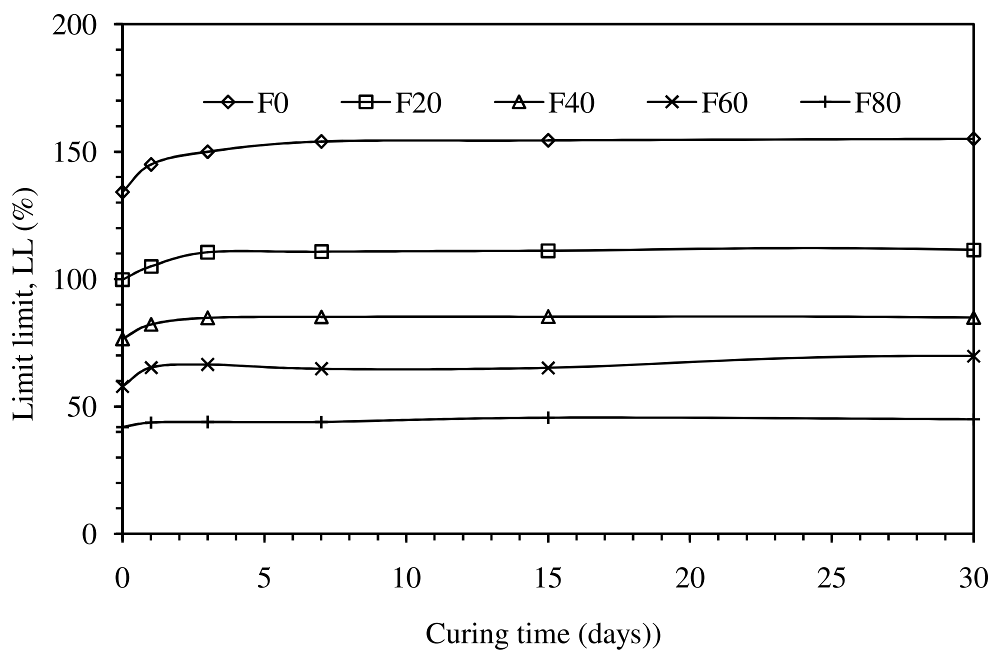


Fig. 4.16 Liquid limit Vs curing time for ES-FA mixes with 1% lime.

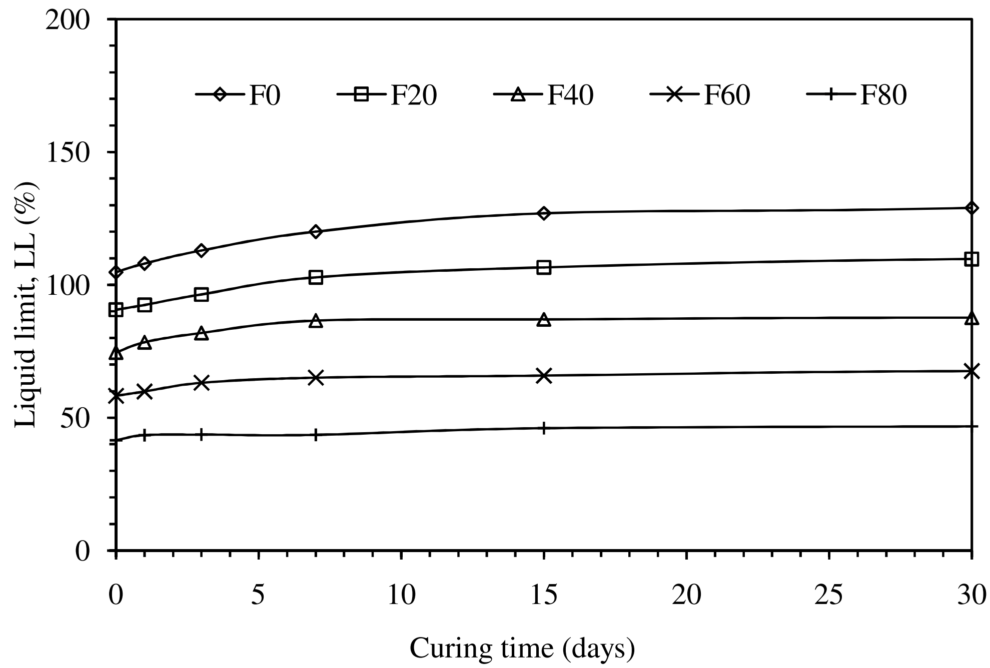


Fig. 4.17 Liquid limit Vs curing time for ES-FA mixes with 3% lime.

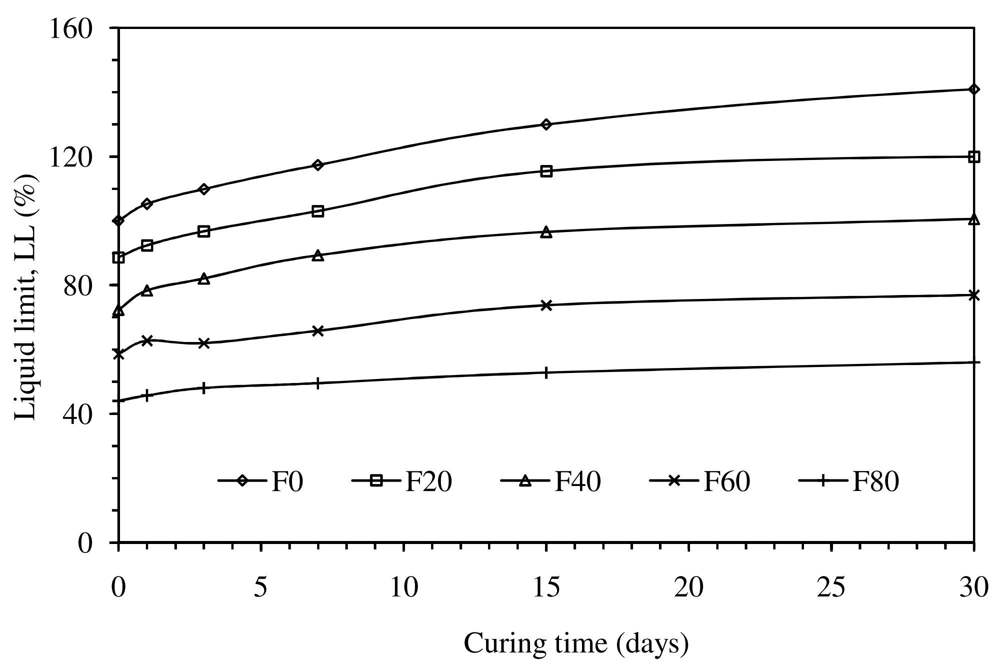


Fig. 4.18 Liquid limit Vs curing time for ES-FA mixes with 5% lime.

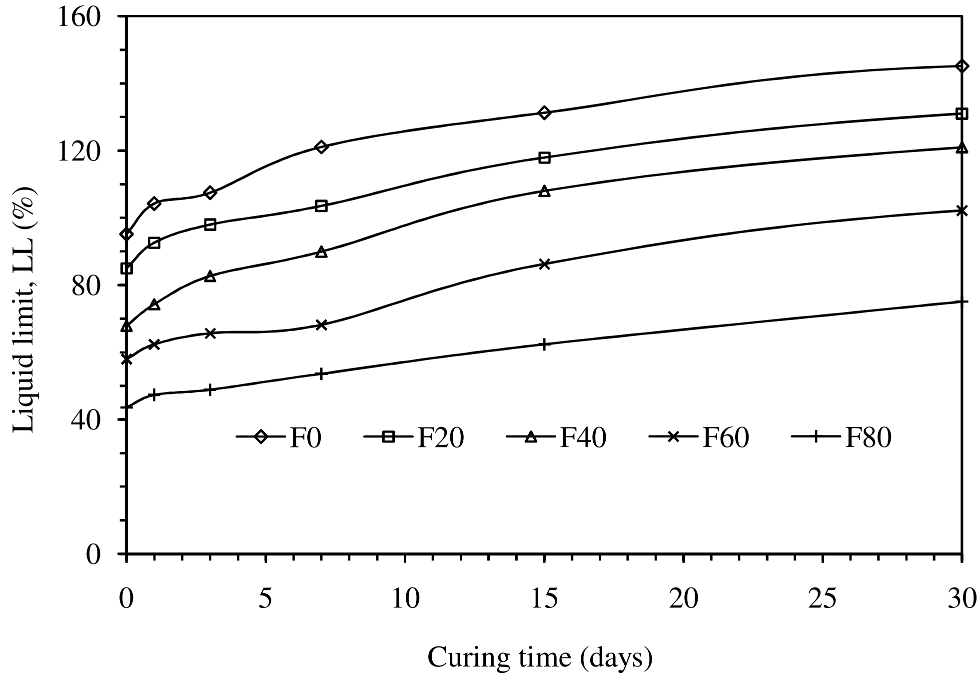


Fig. 4.19 Liquid limit Vs curing time for ES-FA mixes with 9% lime.

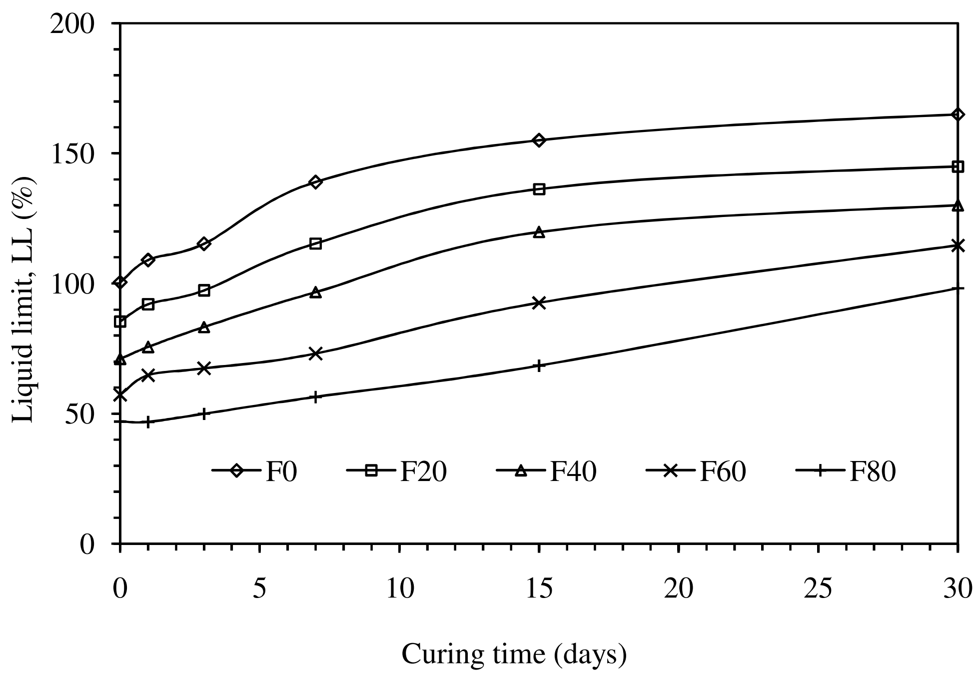


Fig. 4.20 Liquid limit Vs curing time for ES-FA mixes with 13% lime.

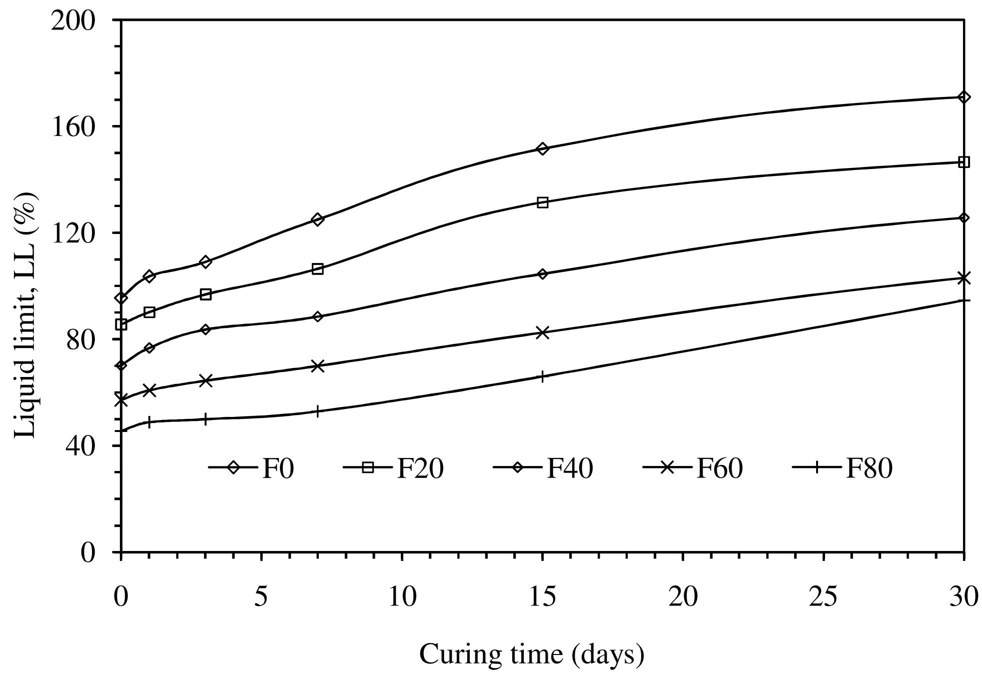


Fig. 4.21 Liquid limit Vs curing time for ES-FA mixes with 17% lime.

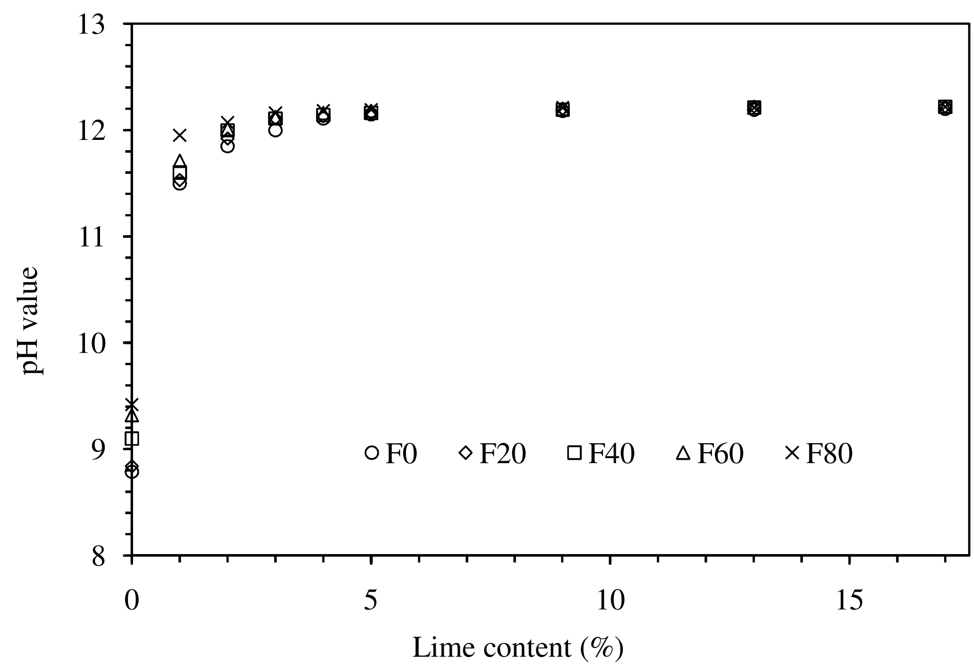


Fig. 4.22 Variation of pH of soil-water with lime content.

4.4.2 Plastic Limit

The influence of quantity of lime added and the time period of curing, on the *PL* of ES-FA mixes are depicted in Figs. 4.23–4.34. It is observed that the *PL* of all the soil samples has increased due to lime treatment. *PL* is the lowest water content at which the cohesion between the soil particles should be low enough that the particles can have relative movements, but high enough to maintain the soil mass in the remolded position (Yong and Workentin, 1975; Mitchel and Soga, 2005). It is also the water content of the soil when it attains a certain shear resistance in the range of 100-300 kPa and in average 170 kPa (Sharma and Bora, 2003; Mitchel and Soga, 2005). With addition of lime the thickness of the diffuse double layer reduces. However near the state of *PL*, the inter-particle distance, and hence pore water content is much less compared to the *LL* state. So, for the same quantity of lime, the concentration of ions in the pore fluid is much more than in the *LL* condition. This causes the viscosity of the pore fluid to increase, which in turn increases the inter particle shear resistance leading to a sharp increase in the *PL* of the soil. Besides, application of lime tends towards alkaline environment (Fig. 4.22) that induces increased negative charge at edge of the soil particles leading to flocculation. With increased flocculation the inter-particle resistance against movement increases leading to increased *PL*. However, beyond the lime fixation point, further lime addition does not increase the pH value of the pore water (Fig. 4.22). Therefore no further flocculation takes place and *PL* does not increase further (Fig.4.23).

The increased rate of *PL*, in the initial stage, is appreciably high with increase in lime content up to about 3-5%. Beyond this, for no curing and short curing periods further increase in *PL* is relatively less. However, for longer curing periods, such as 15 and 30 days, the *PL* continues to increase steadily with lime content up to about 13%. This is attributed to two different factors; (i) time-dependent adsorption of water onto clay surface, (ii) new products formed due to the time-dependent reactions between lime and soil. As has been already discussed in the Section 4.4.1, these new products are the gel-like materials that hold relatively large quantity of water. This has led to the increase in water content of the soil giving rise to increased *PL*. With increase in lime percentage there forms increased quantity of gel products. Therefore the increase in *PL* is more prominent in case of higher lime content (Figs. 4.29–4.34).

Further addition of lime (17%) is seen to reduce the *PL* somewhat. This is attributed to the formation of small grits of $CaCO_3$ due to carbonation of $Ca(OH)_2$. As $CaCO_3$ is a non-plastic material, with its formation the overall *PL* of the soil mass has reduced.

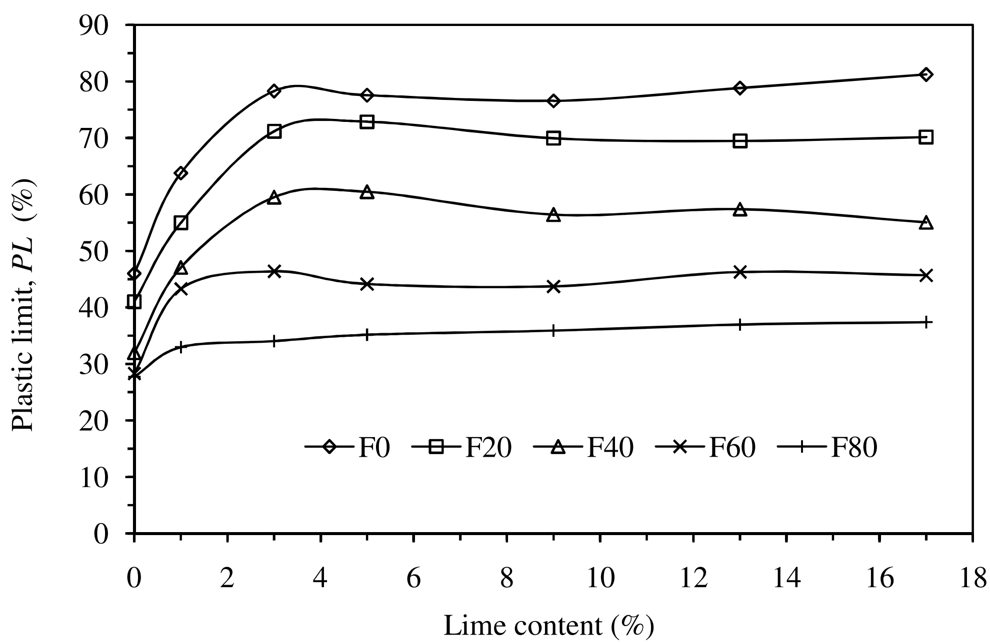


Fig. 4.23 Plastic limit Vs lime content for ES-FA mixes (without curing).

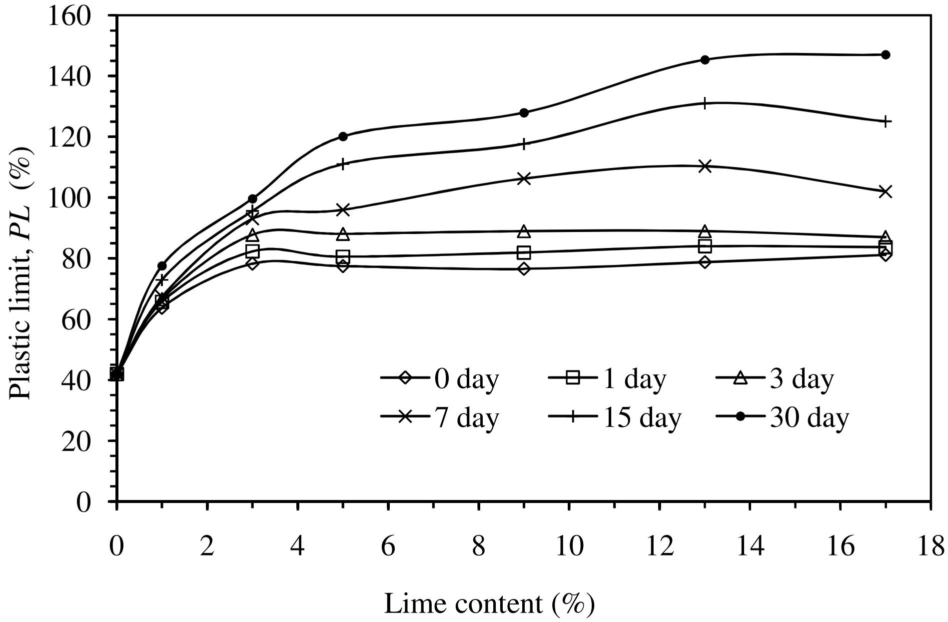


Fig. 4.24 Plastic limit Vs lime content for expansive soil (F0) at different curing periods.

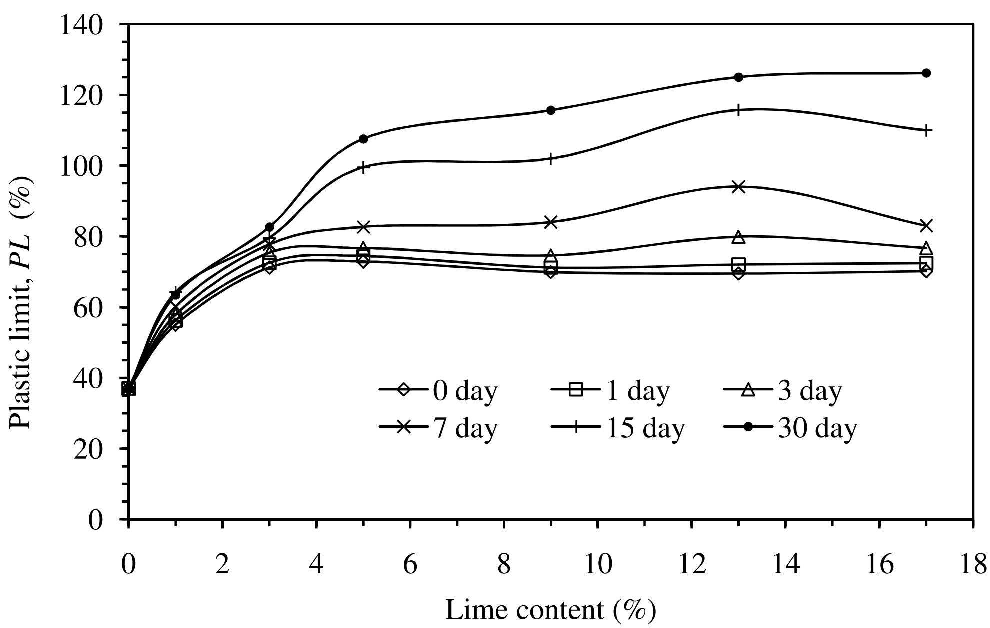


Fig. 4.25 Plastic limit Vs lime content for ES-FA mix F20 at different curing periods.

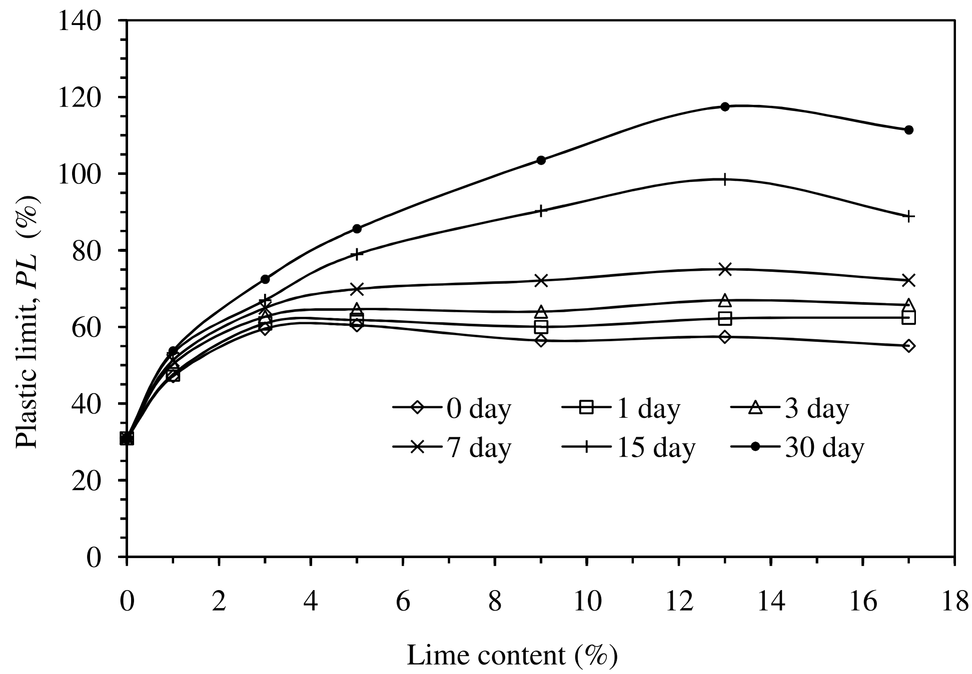


Fig. 4.26 Plastic limit Vs lime content for ES-FA mix F40 at different curing periods.

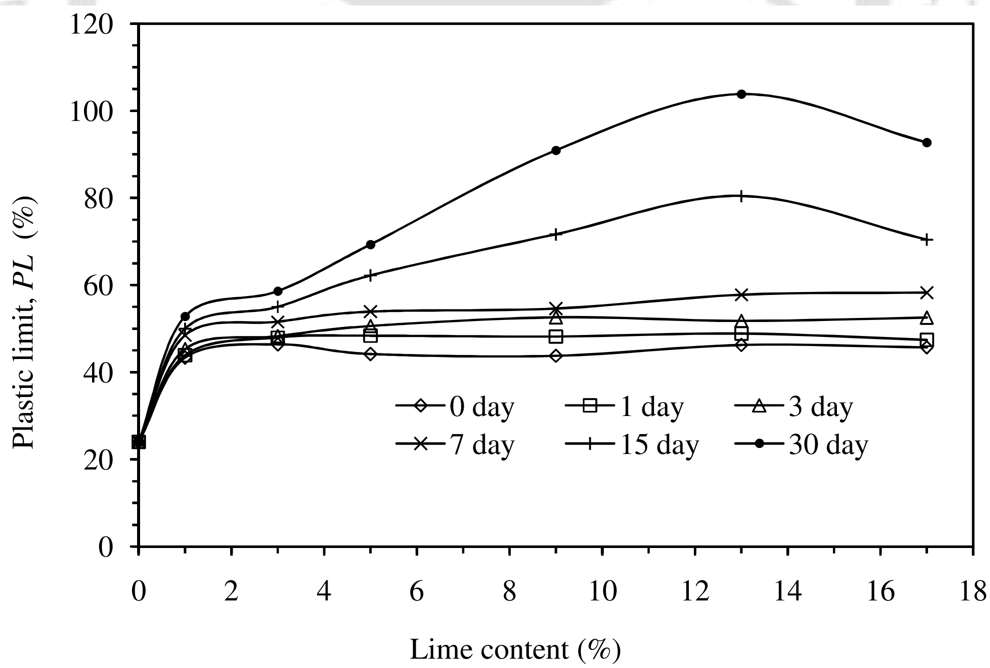


Fig. 4.27 Plastic limit Vs lime content for ES-FA mix F60 at different curing periods.

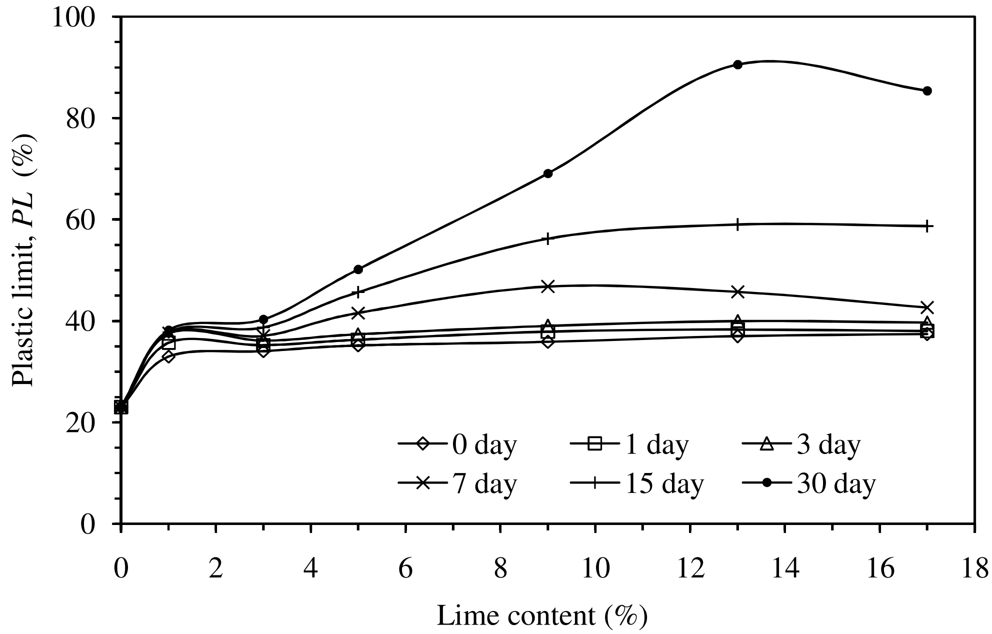


Fig. 4.28 Plastic limit Vs lime content for ES-FA mix F80 at different curing periods.

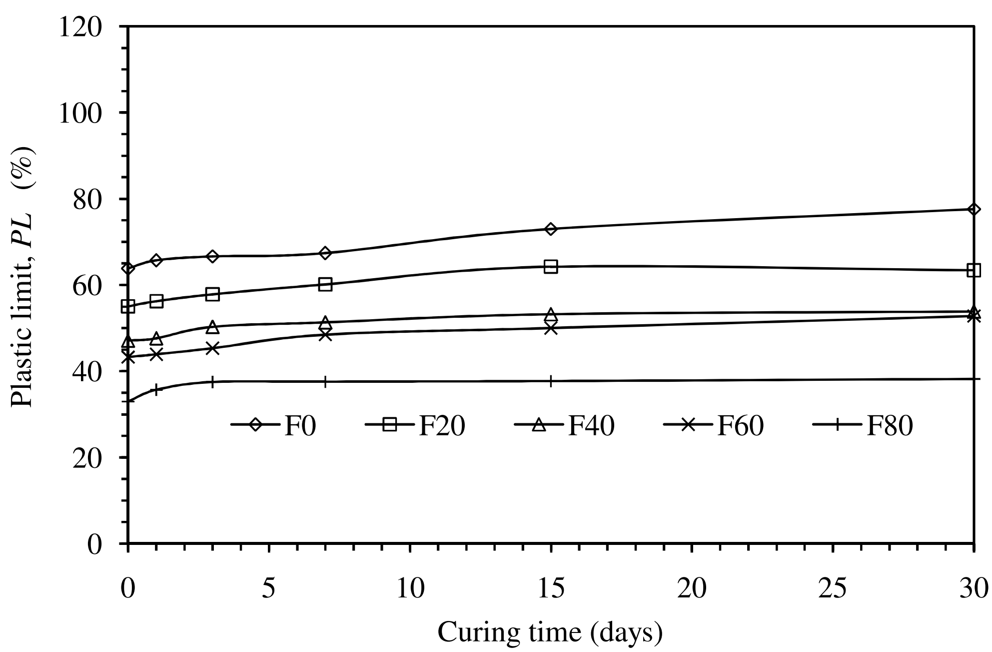


Fig. 4.29 Plastic limit Vs curing time for ES-FA mixes with 1% lime.

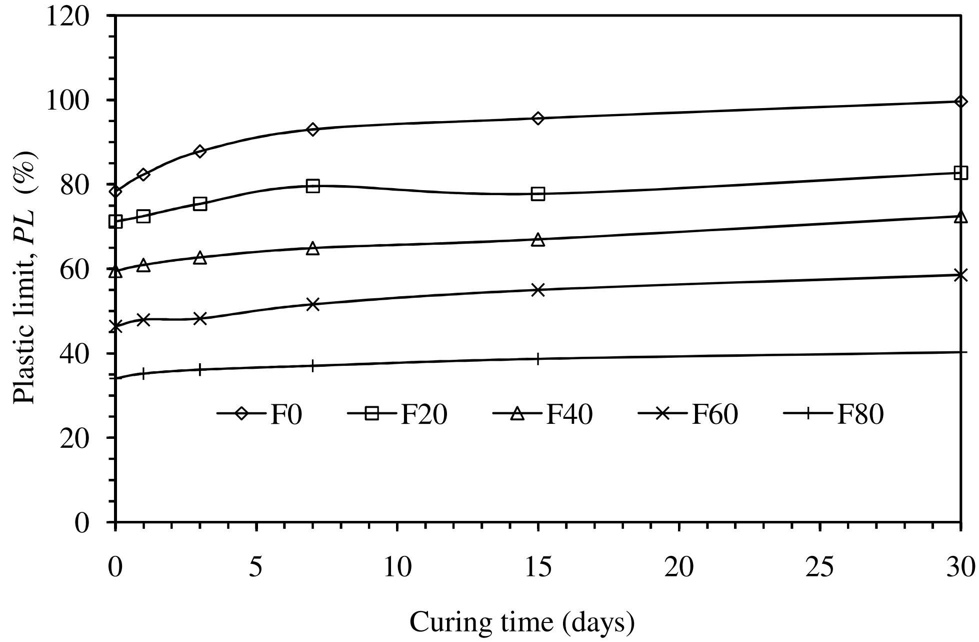


Fig. 4.30 Plastic limit Vs curing time for ES-FA mixes with 3% lime.

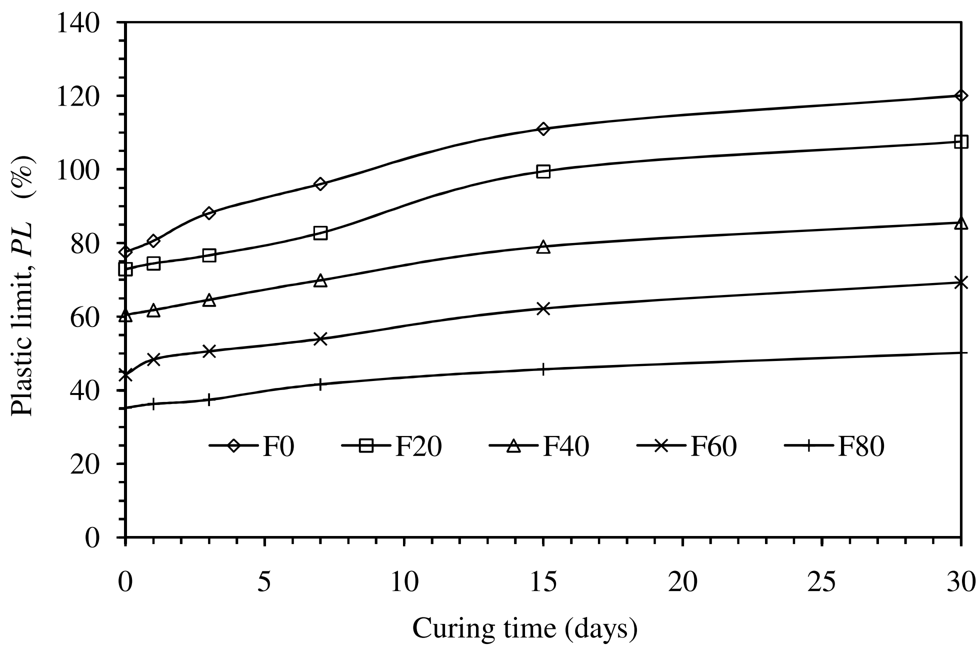


Fig. 4.31 Plastic limit Vs curing time for ES-FA mixes with 5% lime.

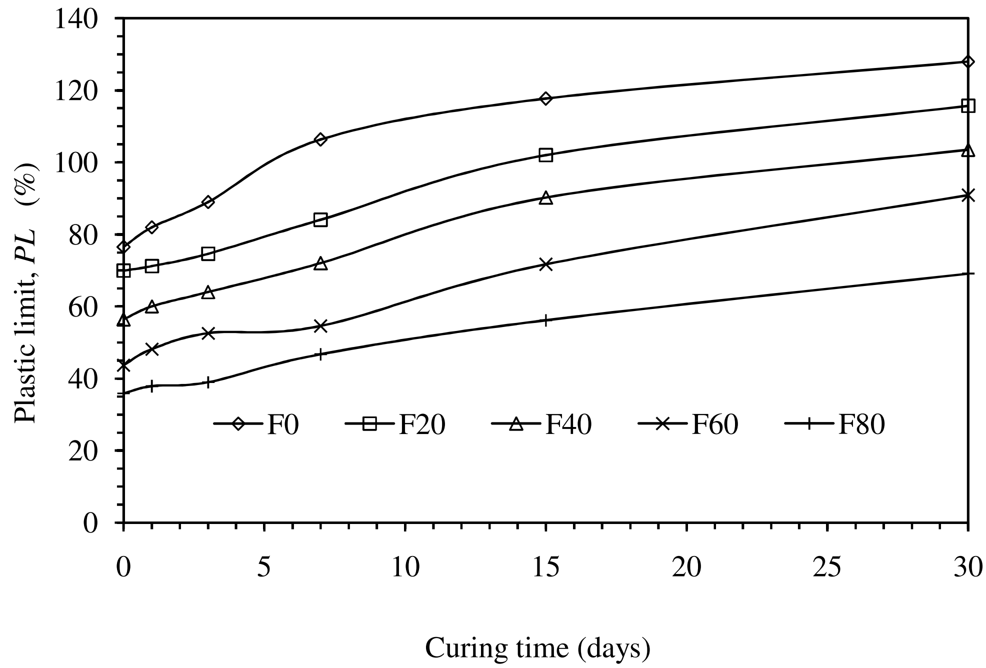


Fig. 4.32 Plastic limit Vs curing time for ES-FA mixes with 9% lime.

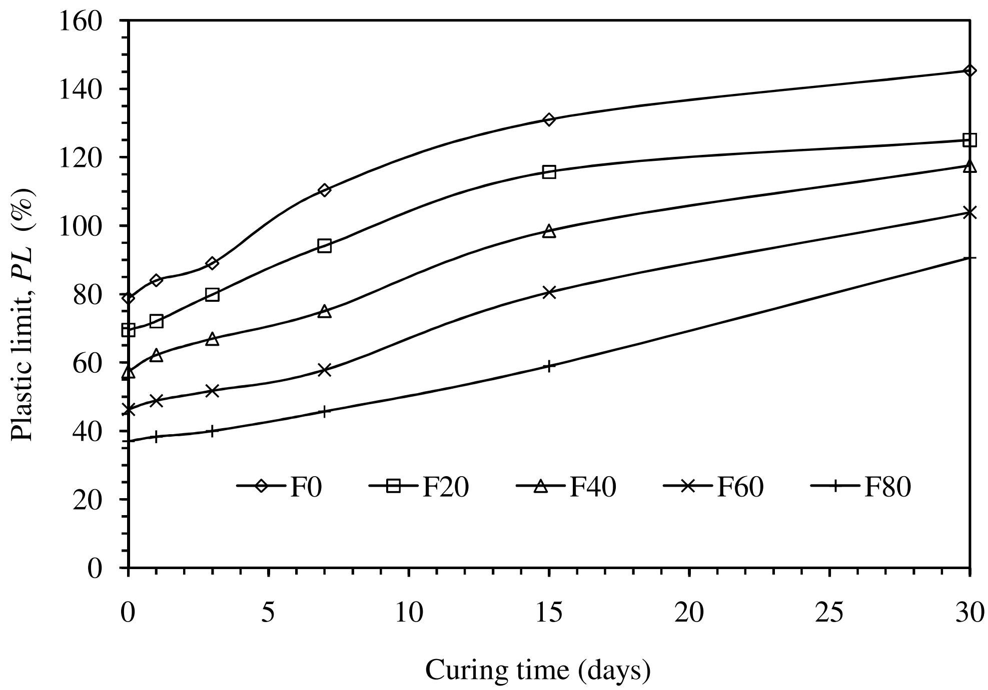


Fig. 4.33 Plastic limit Vs curing time for ES-FA mixes with 13% lime.

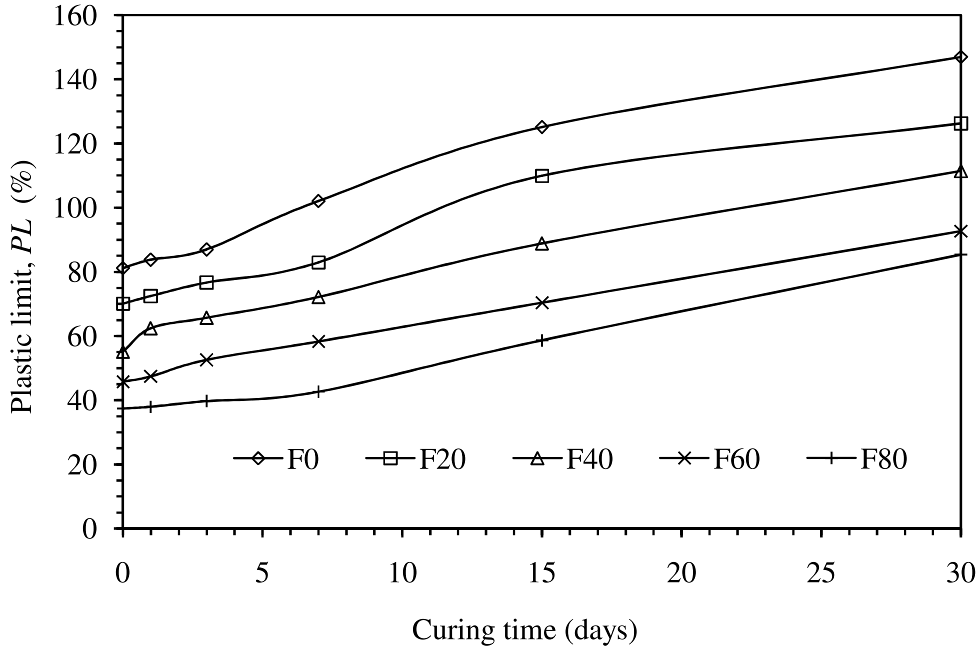


Fig. 4.34 Plastic limit Vs curing time for ES-FA mixes with 17% lime.

4.4.3 Plasticity Index

The *PI* of a soil indicates the range of water content within which the soil remains plastic. It determines the workability of a soil (Sivapullaiah et al., 1996). Since it is the difference of *LL* and *PL*, the factors influencing *LL* and *PL* of the soil also influences *PI*. The *PI* of the different ES-FA mixes are presented in Fig. 4.35 to Fig. 4.39. The *PI* of all these soil mixes are found to decrease, substantially, even with an addition of just 1% lime. The *PI* decreased further upto about 3% lime, beyond which further change was not significant. This indicates that from workability point of view, addition of 3% lime would be good enough. It is of interest to note that unlike *LL* and *PL*, the *PI* does not change much with increase in curing period. This is attributed to nearly equal change in *LL* and *PL* values with curing period, that their difference, the PI, remains unchanged.

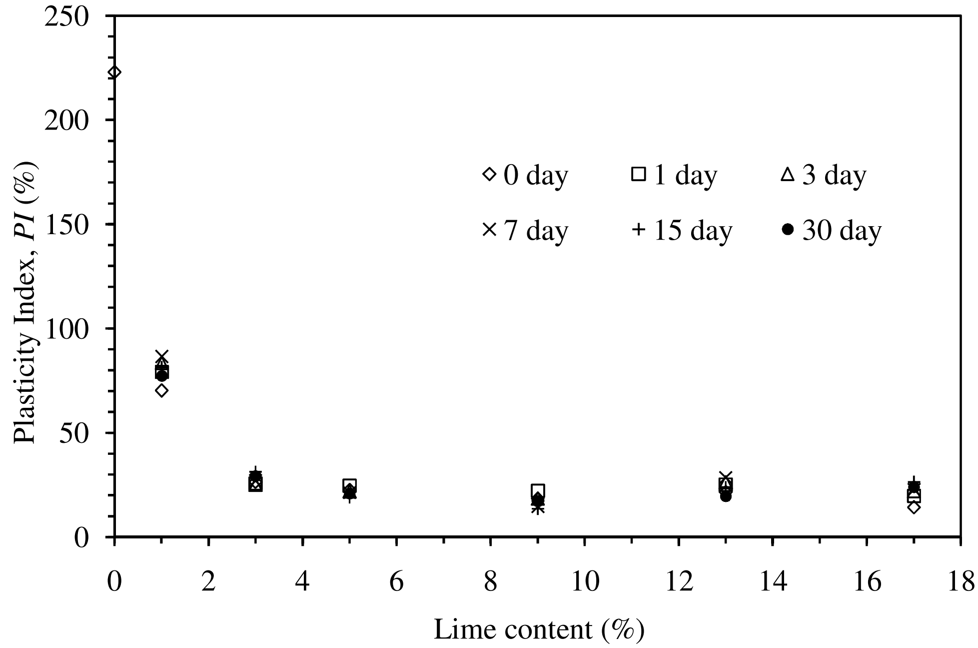


Fig. 4.35 Plasticity Index Vs lime content for expansive soil (F0) at different curing periods.

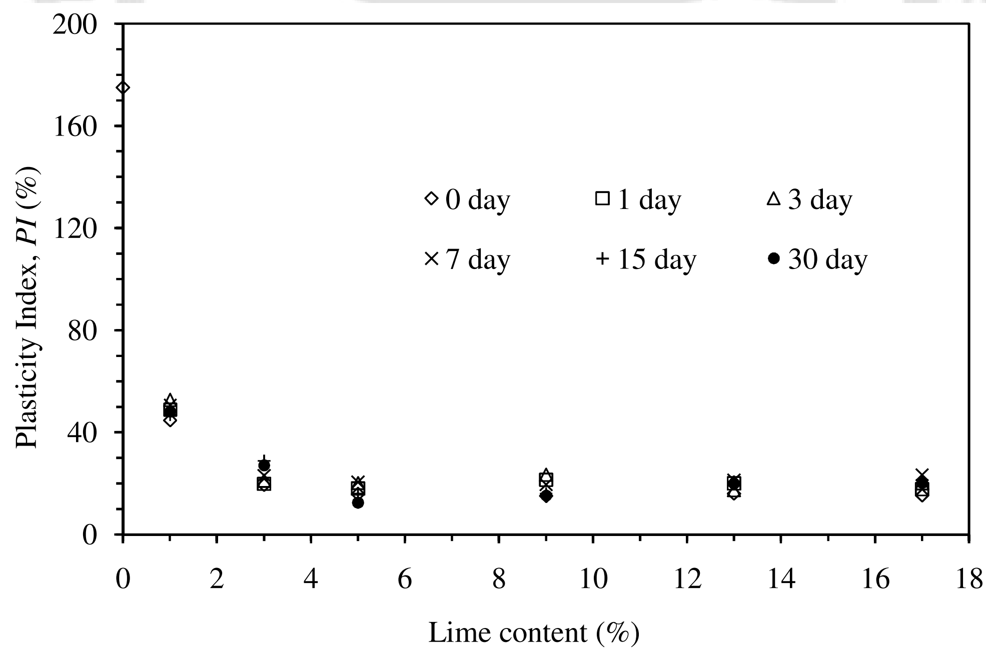


Fig. 4.36 Plasticity Index Vs lime content for ES-FA mix F20 at different curing periods.

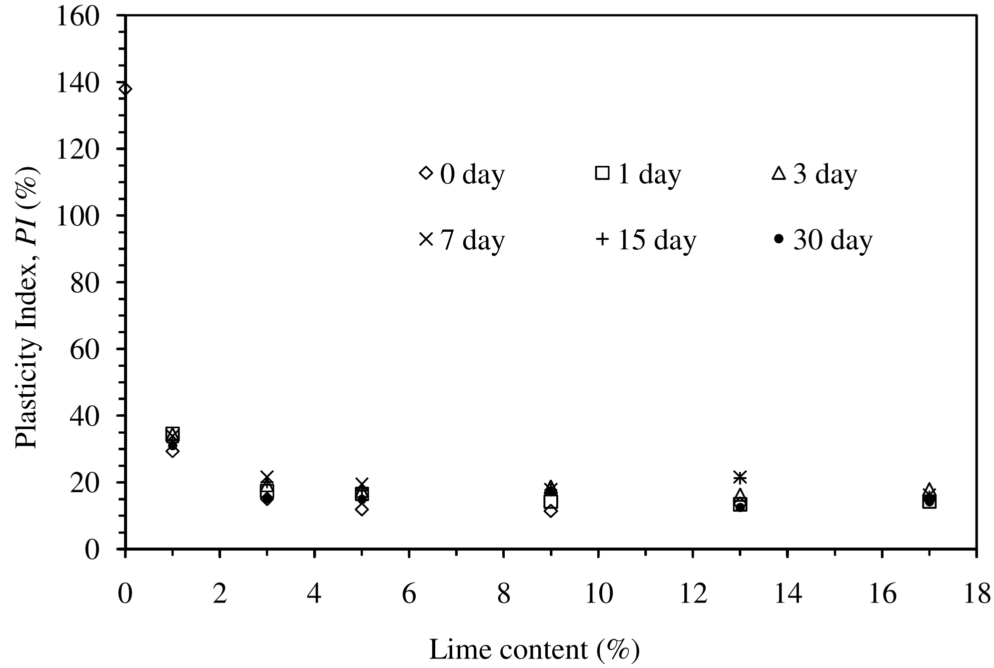


Fig. 4.37 Plasticity Index Vs lime content for ES-FA mix F40 at different curing periods.

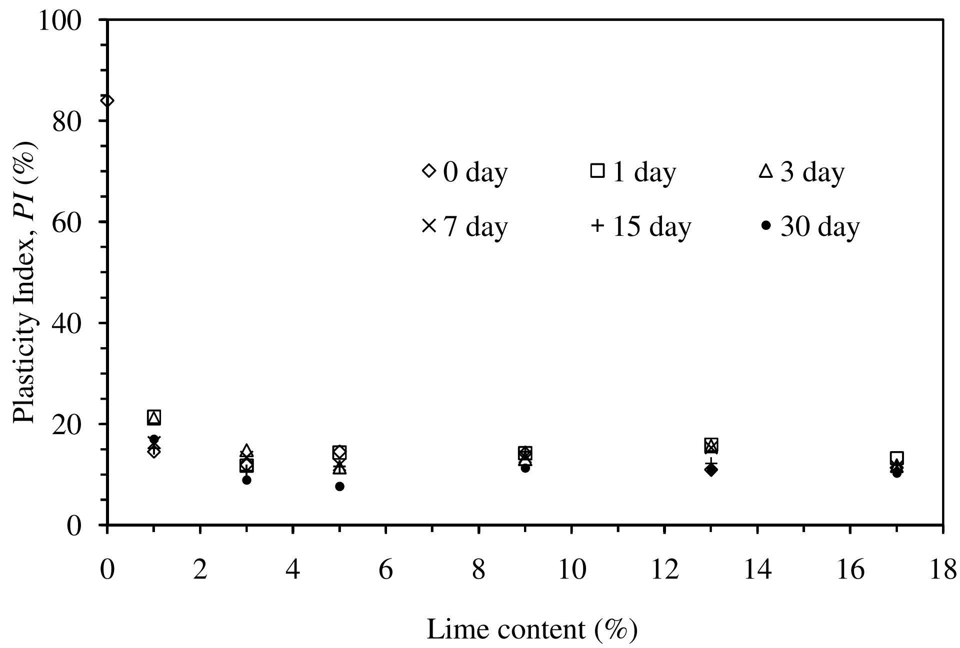


Fig. 4.38 Plasticity Index Vs lime content for ES-FA mix F60 at different curing periods.

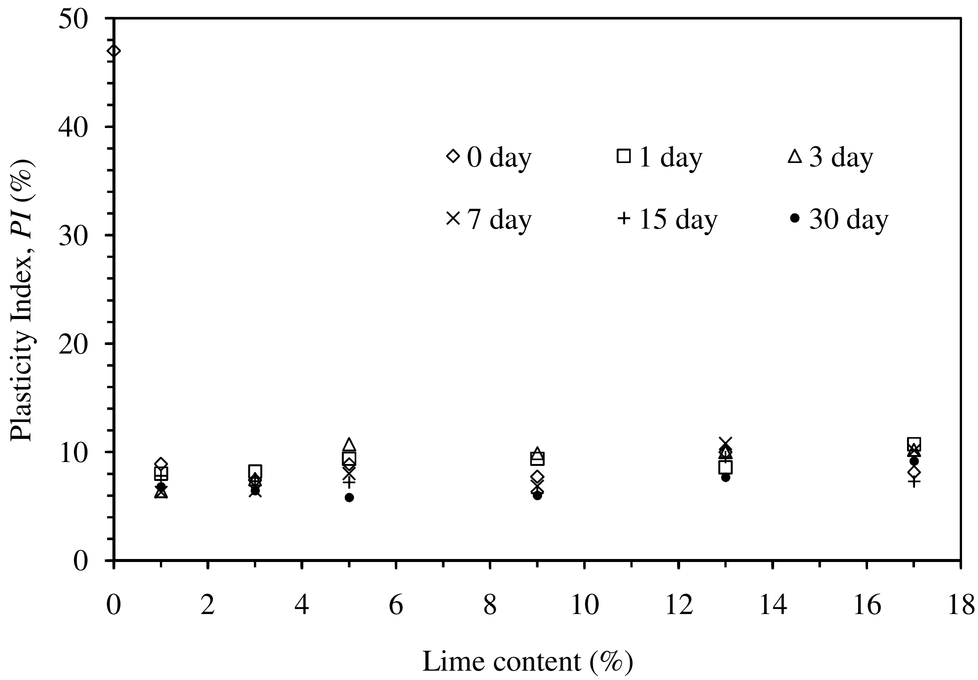


Fig. 4.39 Plasticity Index Vs lime content for ES-FA mix F80 at different curing periods.

The changes in the plasticity characteristics of the ES-FA soils due to application of lime have been depicted in Figs.4.40 and 4.41 as well as in Table 4.8. It can be seen that all ES-FA mixes, except that having 80% fly ash (F80), have been transformed from clay (CH) classification to silt (MH) classification, even without curing. The soils with 80% fly ash have been converted from CH to either ML or MH, depending on the lime content and curing time. For small lime content (1-3%) or shorter curing period (0-1 day), the F80 mixes have changed into ML. On the other hand, at longer curing (≥7 days), with relatively higher lime content (i.e. ≥5%), the F80 mixes turns to MH classification. This is attributed to their high LL (i.e. >50). Overall it can be said that even with a very small quantity of lime the workability can be improved substantially, which may not be possible even with very large quantity of fly ash. This point has more clearly been brought out in the following section.

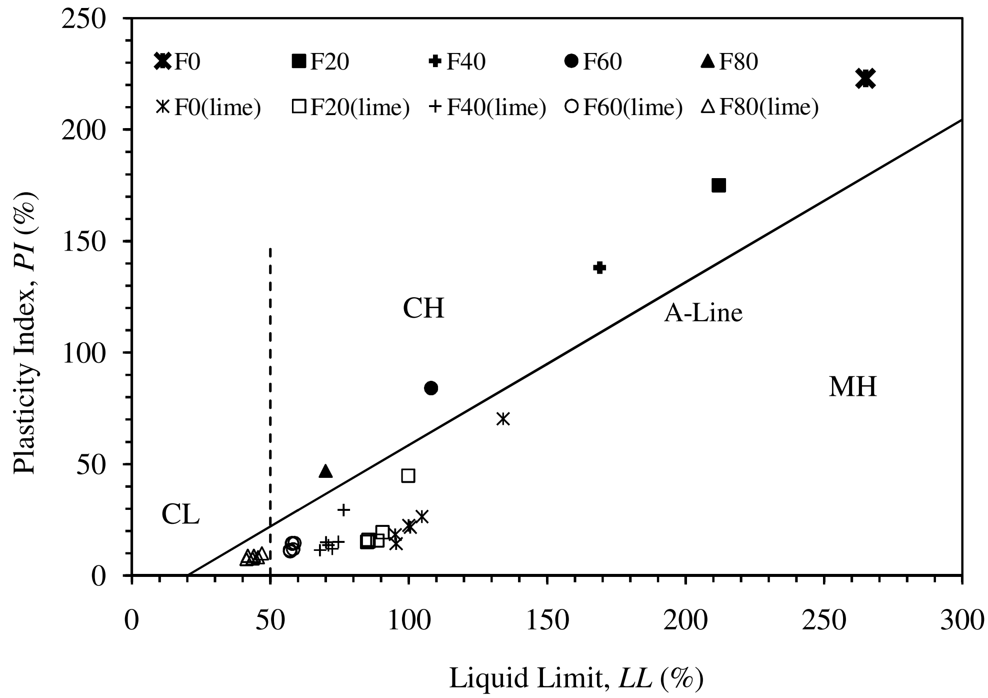


Fig. 4.40 Depiction of ES-FA-lime mixes in the plasticity chart (without curing).

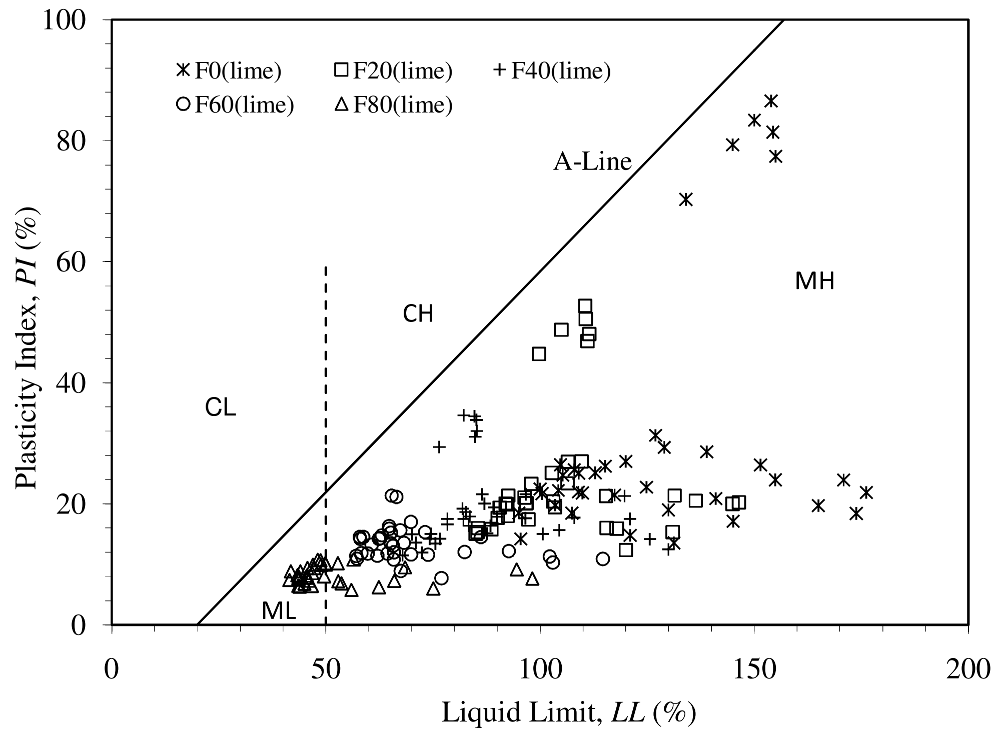


Fig. 4.41 Depiction of ES-FA-lime mixes in the plasticity chart (with curing).

Tables 4.8 Change in the classification of ES-FA due to lime treatment.

Soil mix	Classification						
	0% lime	1% lime	3% lime	5% lime	9% lime	13% lime	17% lime
			No Curing				
F0	CH	MH	MH	MH	MH	MH	MH
F20	CH	MH	MH	MH	MH	MH	MH
F40	CH	MH	MH	MH	MH	MH	MH
F60	CH	MH	MH	MH	MH	MH	MH
F80	CH	ML	ML	ML	ML	ML	ML
			1 day Curing				
F0	CH	MH	MH	MH	MH	MH	MH
F20	CH	MH	MH	MH	MH	MH	MH
F40	CH	MH	MH	MH	MH	MH	MH
F60	CH	MH	MH	MH	MH	MH	MH
F80	CH	ML	ML	ML	ML	ML	ML
			3 days Curing				
F0	CH	MH	MH	MH	MH	MH	MH
F20	CH	MH	MH	MH	MH	MH	MH
F40	CH	MH	MH	MH	MH	MH	MH
F60	CH	MH	MH	MH	MH	MH	MH
F80	CH	ML	ML	ML	ML	MH	MH
			7 days Curing				
F0	CH	MH	MH	MH	MH	MH	MH
F20	CH	MH	MH	MH	MH	MH	MH
F40	CH	MH	MH	MH	MH	MH	MH
F60	CH	MH	MH	MH	MH	MH	MH
F80	CH	ML	ML	MH	MH	MH	MH
			15 days Curing				
F0	CH	MH	MH	MH	MH	MH	MH
F20	CH	MH	MH	MH	MH	MH	MH
F40	CH	MH	MH	MH	MH	MH	MH
F60	CH	MH	MH	MH	MH	MH	MH
F80	CH	ML	ML	MH	MH	MH	MH
			30 days Curing				
F0	CH	MH	MH	MH	MH	MH	MH
F20	CH	MH	MH	MH	MH	MH	MH
F40	CH	MH	MH	MH	MH	MH	MH
F60	CH	MH	MH	MH	MH	MH	MH
F80	CH	ML	ML	MH	MH	MH	MH

4.5 COMPARATIVE ANALYSIS

Figs. 4.42–4.47 compare the individual influences of FA and lime on the change in the *LL*, *PL* and *PI* of ES, without curing and with 30 days of curing. It can be observed that, the reduction in *LL* of ES, that can be achieved by mixing with fly ash is almost linear in trend. On the other hand, the reduction in *LL* due to lime treatment is relatively steep upto about 2% lime, but further increase in lime content does not change the *LL* much.

In contrast to the change in *LL*, the effects of addition of fly ash and lime, on the *PL* of the ES are opposite in trend (Figs. 4.44–4.45). The *PL* reduces on fly ash addition, whereas it increases with lime addition.

The *PI* reduces with fly ash and lime addition, but the effect of lime addition is more prominent (Figs. 4.46–4.47). Like the *LL*, *PI* visibly decreases even with small content of lime, and then reaches nearly a constant value of about 20%.

It is of interest to note that with addition of about 10-12% of lime to the ES, one can achieve similar reduction in *LL* as that of adding about 50-60% of fly ash (Figs. 4.42 and 4.43). In terms of *PI* the results are more encouraging (Figs. 4.45 and 4.46). Similar reduction in *PI* obtained by the addition of 80% of FA to the expansive soil, can be achieved with just 2% of lime. Hence it can be said that, in order to achieve substantial performance improvement, it is advantageous to add a little of lime to the ES-FA mix.

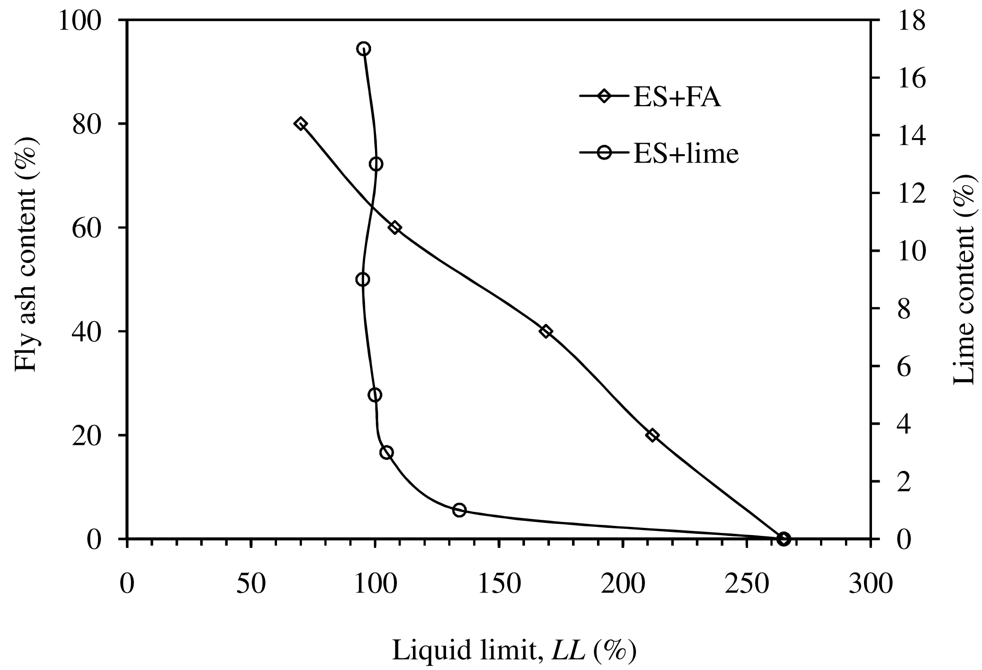


Fig. 4.42 Comparison of effects of fly ash and lime on liquid limit of ES (without curing).

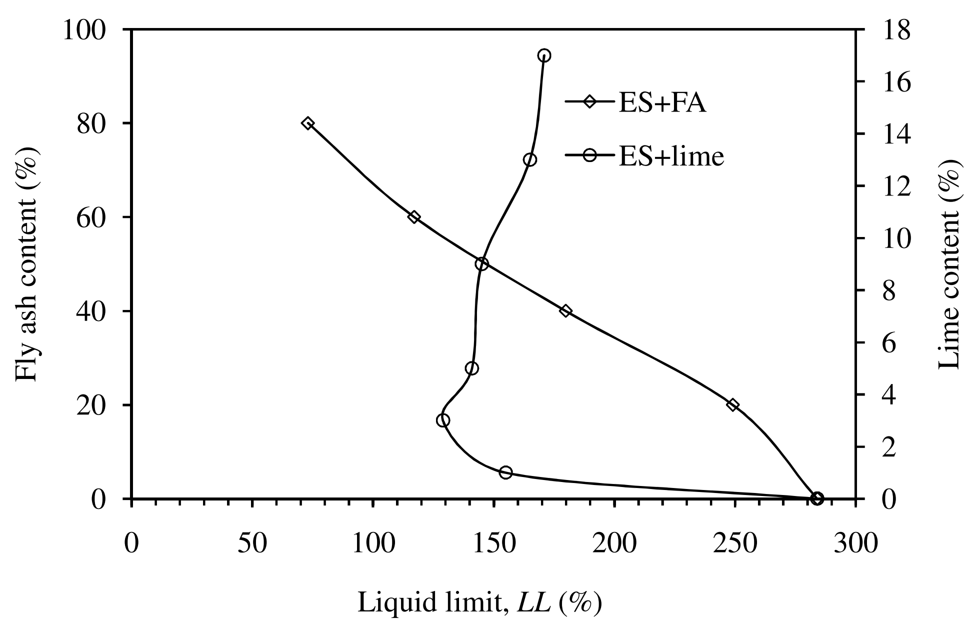


Fig. 4.43 Comparison of effects of fly ash and lime on liquid limit of ES (30 days curing).

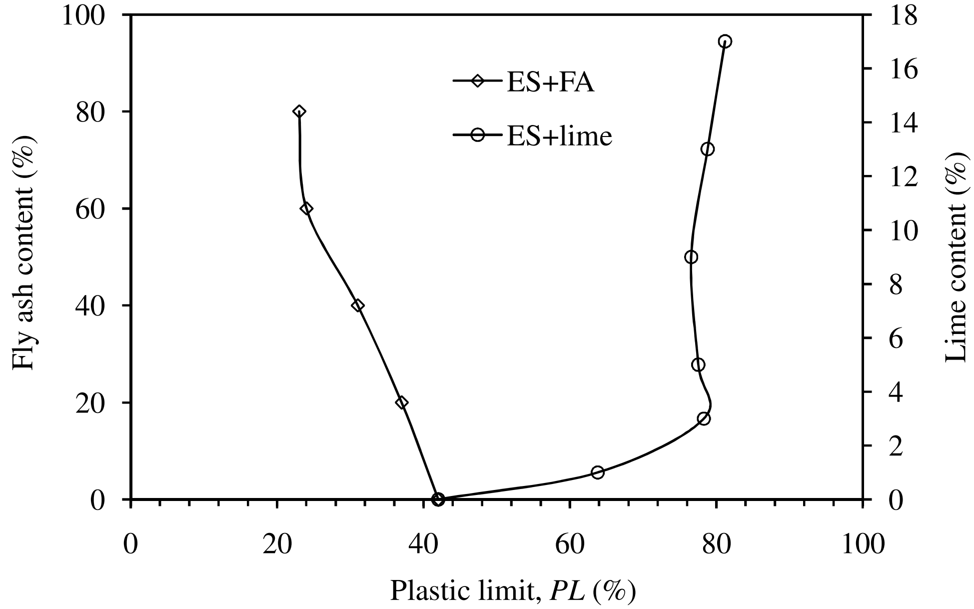


Fig. 4.44 Comparison of effects of fly ash and lime on plastic limit of ES
(without curing).

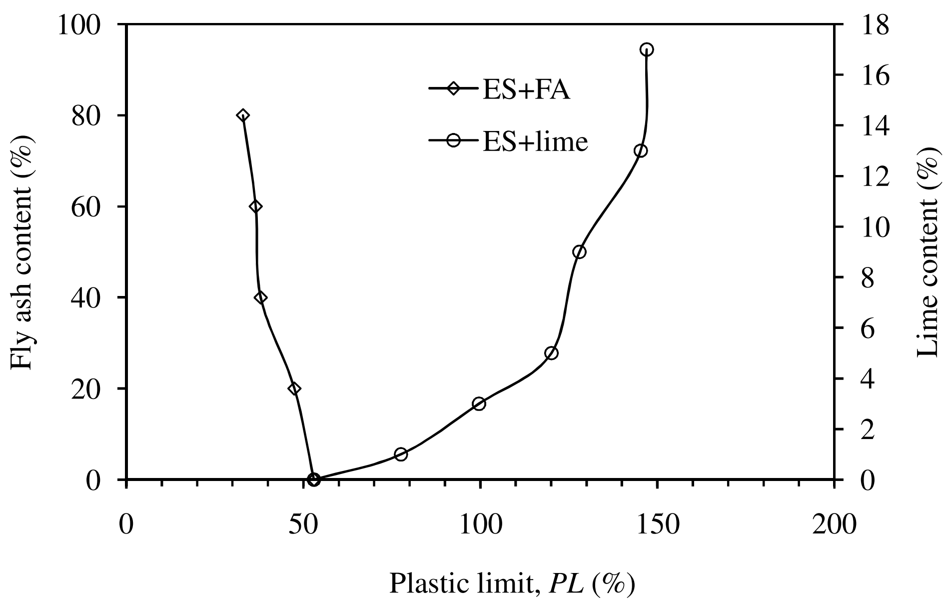


Fig. 4.45 Comparison of effects of fly ash and lime on plastic limit of ES
(30 days curing).

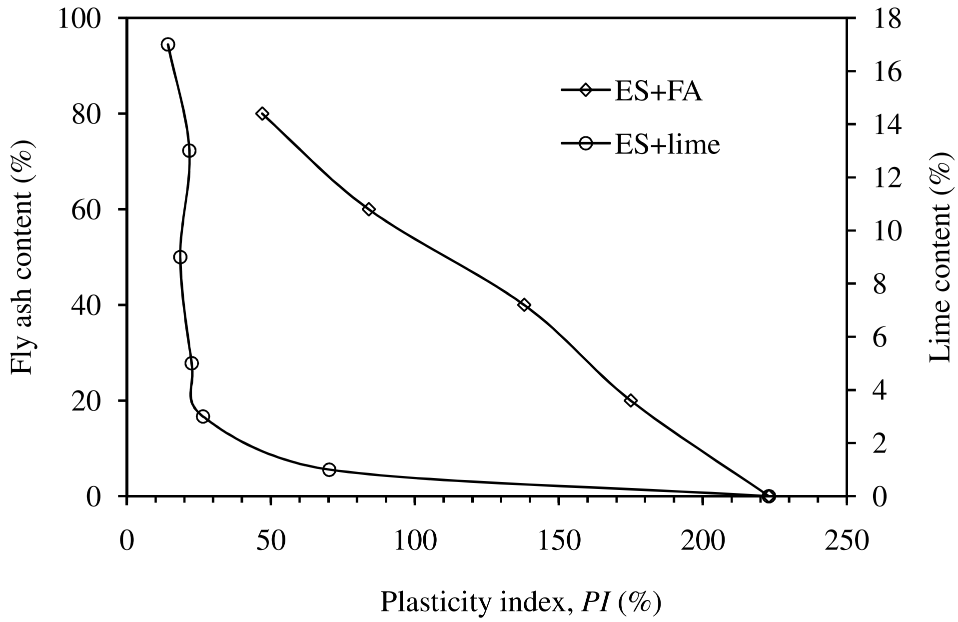


Fig. 4.46 Comparison of effects of fly ash and lime on plasticity index of ES (without curing).

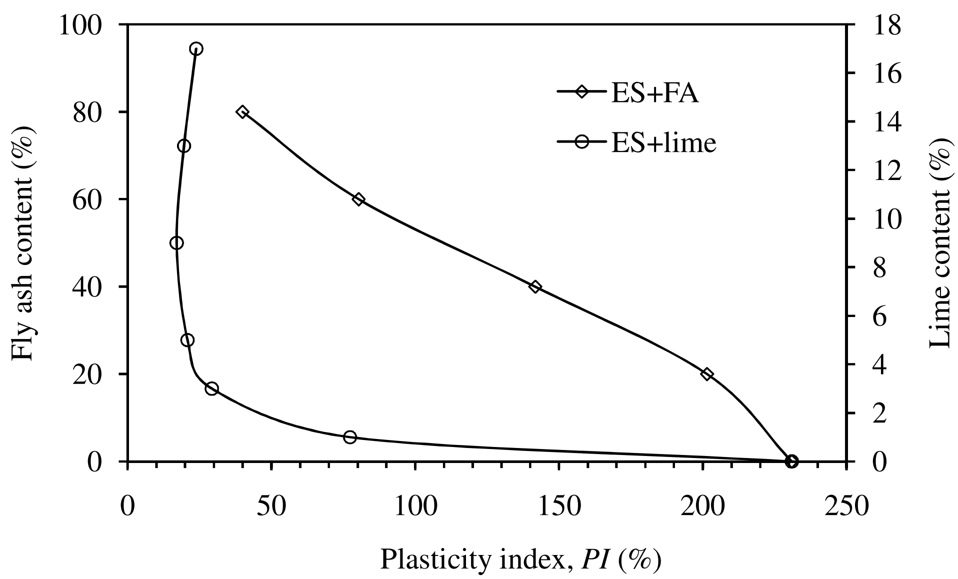


Fig. 4.47 Comparison of effects of fly ash and lime on plasticity index of ES (30 days curing).

4.6 SUMMARY

It is observed that there is an underestimation of liquid limit obtained by using cone penetration method as compared to percussion method, especially for high liquid limit soils. Also, previous studies indicate that percussion method is a better representation of plasticity characteristics of the soil. In view of this, a modified methodology has been proposed based on the free-swell index property of the soil for obtaining cone penetration liquid limit values, which are comparable with that of percussion method. To achieve this, modified penetration limit has been derived based on the free-swell index of the soil. The cone penetration liquid limit determined based on modified penetration value matches well with the percussion liquid limit even for high plastic soils. The proposed methodology has been validated by an independent soil data set. Based on this observation, it is found that the free-swell index of the soil can be used to modify the value of penetration for calculating liquid limit determination.

Performance improvement of expansive soil, in terms of its plasticity characteristics, through addition of fly ash and lime has been studied through a series of tests. It is observed that addition of fly ash reduces plasticity of the expansive soil-fly ash mix, but the soil still behaves as high plastic even with 80% of fly ash. However with addition of just 1% of lime, changes it to silt classification. This indicates that in order to achieve significant increase in workability of expansie soil-fly ash mix it is advantageous to add a little of lime onto it. Curing further improves the workability. High lime content (e.g. 17%) combined with long curing period is found to increase the liquid limit and plastic limit. However, this does not increase the plasticity index significantly.

While addition of FA to ES has reduced the liquid limit, plastic limit and plasticity index gradually, the effect of lime addition on these properties is significant. The change in plasticity (i.e. *LL, PL* and *PI*) is visibly high till lime content of about 2% beyond which further change is marginal.

CHAPTER 5

COMPACTION BEHAVIOUR

5.1 INTRODUCTION

Compaction is an important process in geotechnical engineering, by which the volume of voids in an unsaturated soil medium is reduced by applying external load. On compaction, soils achieve increased dry density and shear strength and reduced settlement and permeability, which are ideal conditions for many of the geotechnical projects.

It is well understood that expansive soils are difficult to compact properly. This difficulty can be reduced, to some extent, by application of additives such as soils with low plasticity, lime, cement, fly ash, etc.

In this chapter, compaction properties of expansive soil (ES) amended with fly ash (FA) are studied. Subsequently, effects of lime on compaction properties of various expansive soil-fly ash (ES-FA) mixes have been studied.

5.2 COMPACTION BEHAVIOR OF ES-FA MIXES

5.2.1 Compaction characteristics

Compaction tests were carried out to find out moisture content-dry density relationships for expansive soil, fly ash and expansive soil-fly ash mixes by following the procedure discussed in section 3.5.4. The dry density-moisture content relationships obtained for ES-FA mixes are depicted in Fig. 5.1. For comparison purpose, the zero air-void line (ZAV line) for the expansive soil (i.e. F0) is also included in this figure. These plots clearly bring out the influence of FA addition on the compaction characteristics of the

ES. Overall, the compaction response of the ES has grown flatter with increased percentage of fly ash added to it. Hence it could be said that soil mix has turned more friable and that the target density can be achieved over a wider range of moisture content. This indicates that the workability of the expansive soil can be increased by addition of fly ash.

The maximum dry density (*MDD*) and optimum moisture content (*OMC*) obtained from Fig. 5.1 for various mixes are summarized in Table 5.1 and are plotted in Fig. 5.2. It can be seen that the *OMC* reduces and *MDD* increases as FA content increases in the ES. The addition of FA facilitates the dissipation of pore water pressure during compaction of the clay-ash mix, resulting in lower *OMC* and higher *MDD*. It can be observed that the change in *OMC* and *MDD* is significant till fly ash content of about 60%, beyond which, further change in these parameters is marginal. This is because in the initial stages when the percentage of fly ash is less, it is interspersed in the clay mass that the whole of soil mix behaves like a homogeneous mass and gets effectively compacted. The increased density is due to higher specific weight of the fly ash that has high silica content in it. With increased fly ash content the clay mass effectively fills the voids formed by the relatively coarse fly ash, giving rise to a compact structure and hence increased density. However when the fly ash quantity is relatively high (i.e. more than 60%) the fly ash forms a cluster-like structure (Murthy et al., 1985) that effectively resists the compaction. As a result, there is not much improvement in the overall density of the expansive soil-fly ash mix. Hence it can be said that 60% is the optimum FA content that gives maximum density for ES-FA mixes.

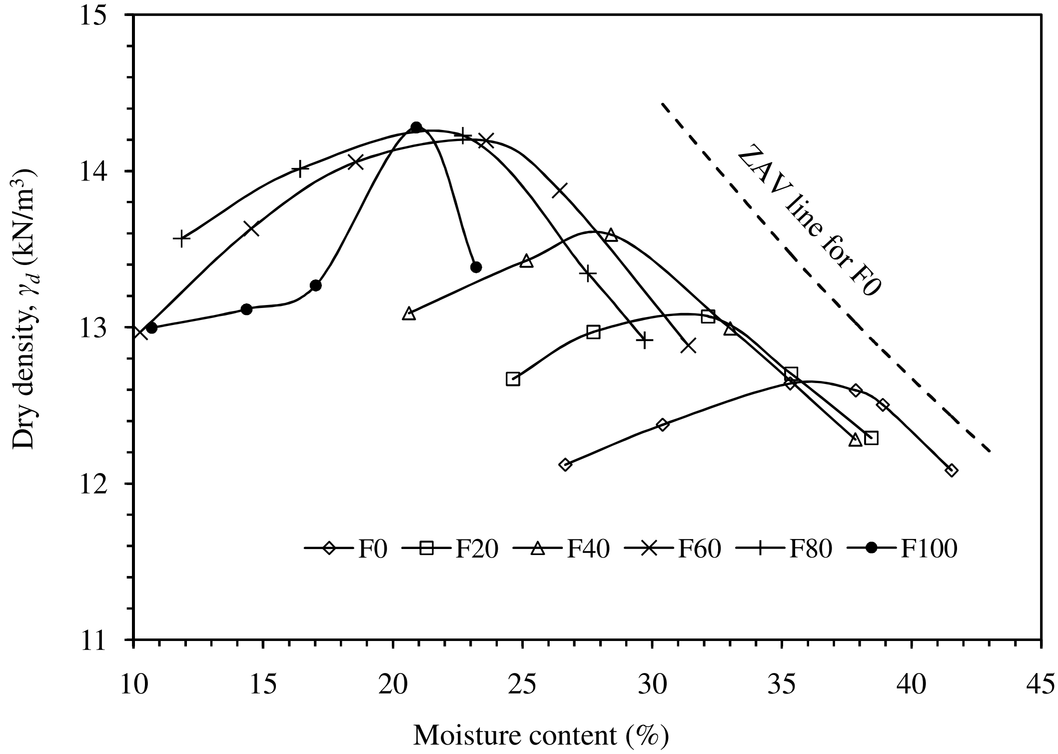


Fig. 5.1 Compaction curves for ES-FA mixes.

Table 5.1 Compaction characteristics of ES-FA mixes.

ES-FA mix designation	Fly ash content (%)	*OMC* (%)	*MDD* (kN/m^3)
F0	0	35.3	12.6
F20	20	32.2	13.1
F40	40	28.4	13.6
F60	60	23.6	14.2
F80	80	22.7	14.2
F100	100	20.9	14.3

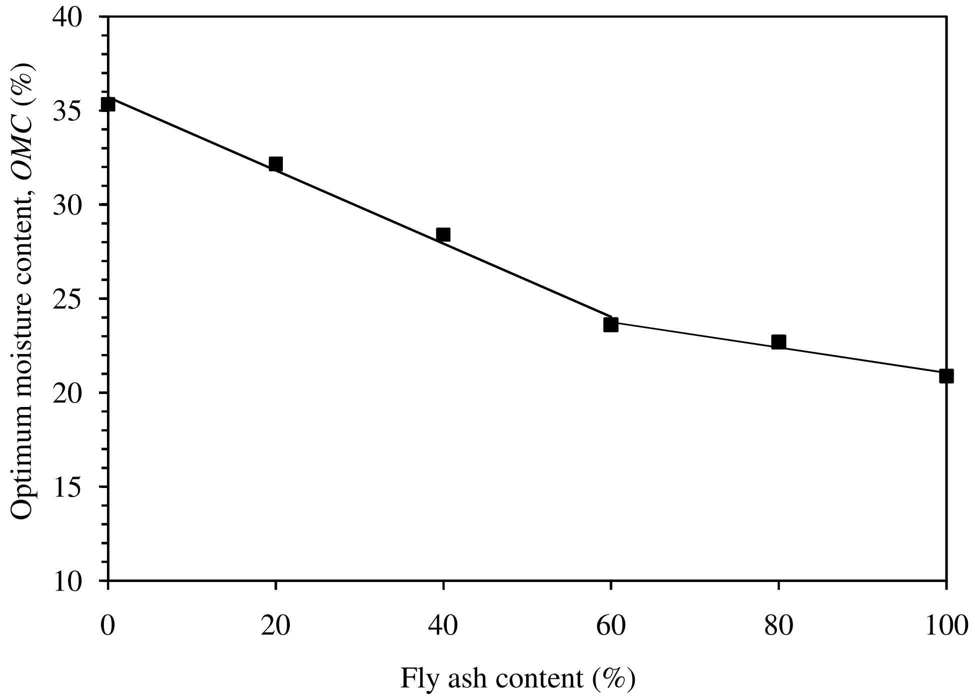


Fig. 5.2 Variation of optimum moisture content with fly ash content.

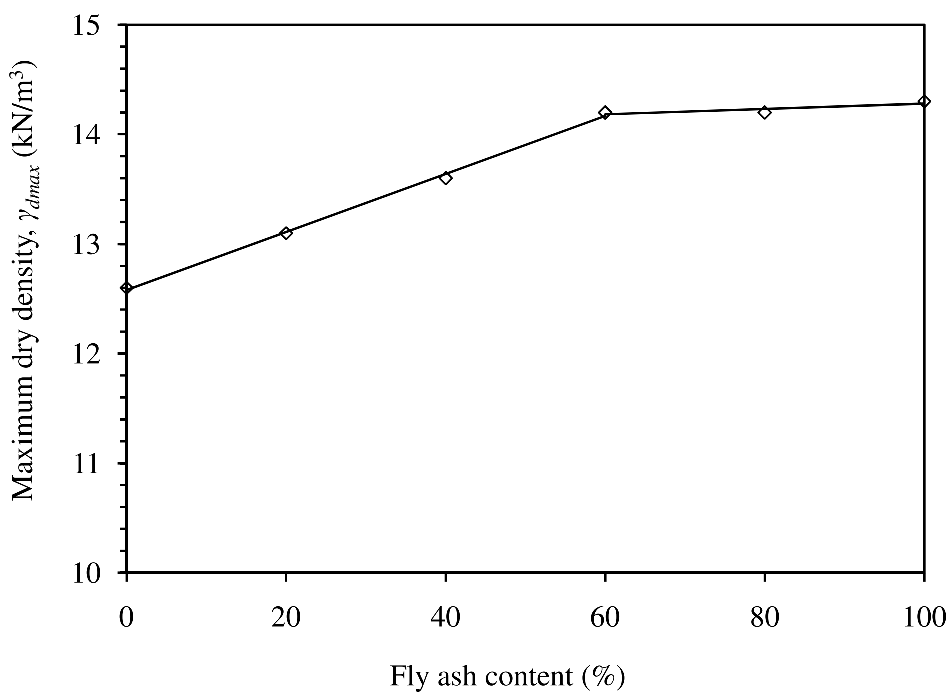


Fig. 5.3 Variation of maximum dry density with fly ash content.

The variation of *MDD* with fly ash content (Fig.5.3) is comparable with the results of Rao and Rao (2008) wherein it was observed that the *MDD* increased with increase in fly ash content till about 40%. Similar trend is reported by Phanikumar and Sharma (2004).

5.2.2 Correlations

A reliable correlation of the compaction characteristics with the index properties can be extremely useful for selection of soils for various geotechnical projects. Variation of *OMC* of different soil mixes with the corresponding liquid limit (*LL*), plastic limit (*PL*) and plasticity index (*PI*) are depicted in Fig. 5.4 to 5.6 respectively. Correspondingly, variation of *MDD* with these index properties is presented in Fig. 5.7 to 5.9 respectively. It can be seen that with increase in plasticity (i.e. *LL*, *PL* and *PI*) the *OMC* increases and the variation is almost linear. Similar observation has been reported by Phanikumar and Sharma (2004). This is attributed to the increased water holding capacity of the soil, with increase in its plasticity. On the contrary, *MDD* has reduced with increase in the plasticity of the soil. Initially when fly ash is large in quantity, large percentage of the soil water gets absorbed by the highly porous ash particles. As the water is now held within the fly ash particles, it does not play much role in the compaction process; therefore, the maximum dry density does not change much. With increase in clay content leading to increase in plasticity, large amount of water is held on to the soil surface through diffuse double layer. Being incompressible it effectively resists compaction leading to reduced dry density. The following correlations have been obtained for the *OMC* and plasticity characteristics:

$$OMC = (0.07)LL + 17.1 \text{ (for } LL \geq 70\%) \tag{5.1}$$

$$OMC = (0.66)PL + 7.7 \text{ (for } PL \geq 23) \tag{5.2}$$

$$OMC = (0.08)PI + 18.2 \text{ (for } PI \geq 47) \tag{5.3}$$

The correlation coefficients (R^2) of these equations are 0.98, 0.99 and 0.98 respectively. Similarly the correlations for the maximum dry density (γ_{dmax}) in kN/m^3 are found to be :

$$\gamma_{dmax} = (-0.01)LL + 15.3 \text{ (for } LL > 100\%) \tag{5.4}$$

$$\gamma_{dmax} = (-0.08)PL + 16.2 \text{ (for } PL > 20\%) \tag{5.5}$$

$$\gamma_{dmax} = (-0.01)PI + 15.2 \text{ (for } PI > 80\%) \tag{5.6}$$

All these equations have a correlation coefficient (R^2) equal to 0.99.

It is of interest to note that among the different index properties of the ES-FA specimens, plastic limit correlates most with the compaction parameters i.e. maximum dry density and optimum moisture content. This is established through the high value of the correlation coefficient (R^2) i.e. 0.99.

Correlations of *OMC* with *PL* have been reported in the literature (Gurtug and Sridharan, 2002; Sridharan and Nagaraj, 2005) for fine grained soils as:

$$OMC = (0.92)PL \tag{5.7}$$

In the present study, compaction data has been assimilated from the literature (Prakash et al., 1989; Blotz et al., 1998; Gurtug and Sridharan, 2002; Sridharan and Nagaraj, 2005; Sridharan and Suvapullaiah, 2005 and Horpibulsuk et al., 2008) to obtain a relationship between *OMC* and *PL* (Fig. 5.10), which is represented by:

$$OMC = (0.84)PL \tag{5.8}$$

It may be noted that the correlation of *OMC* with *PL* in this study (Eq.5.2) has an intercept, whereas, the correlations from the literature for natural soils pass through the origin. When forced through the origin to compare with the literature data, the correlation between *OMC* and *PL* of our study (Fig.5.11) becomes:

$$OMC = (0.89)PL \tag{5.9}$$

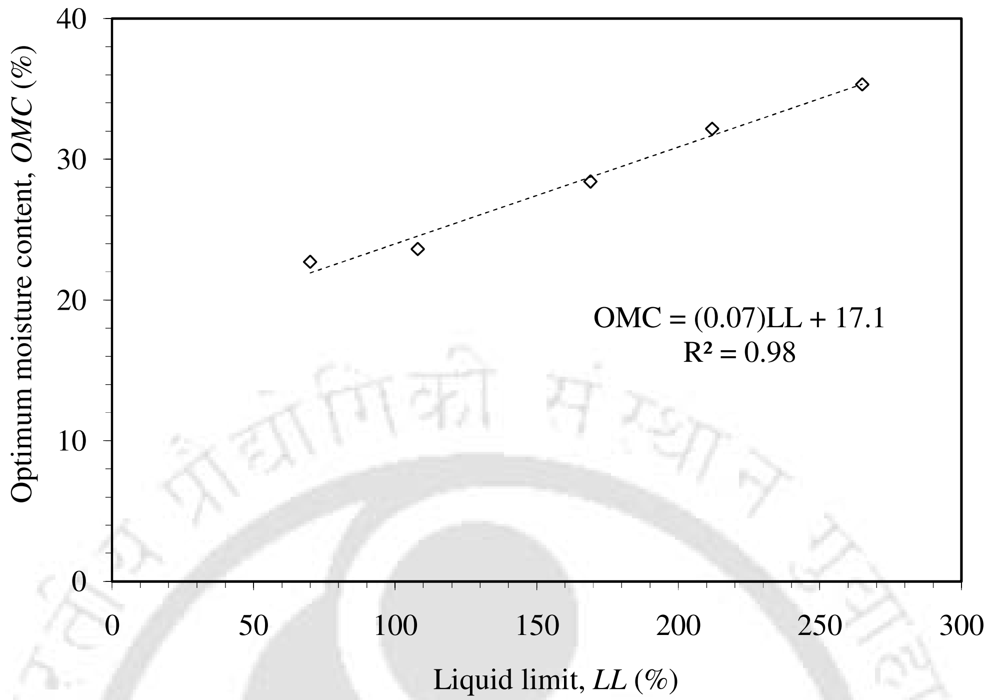


Fig. 5.4 Correlation of optimum moisture content with liquid limit for ES-FA mixes.

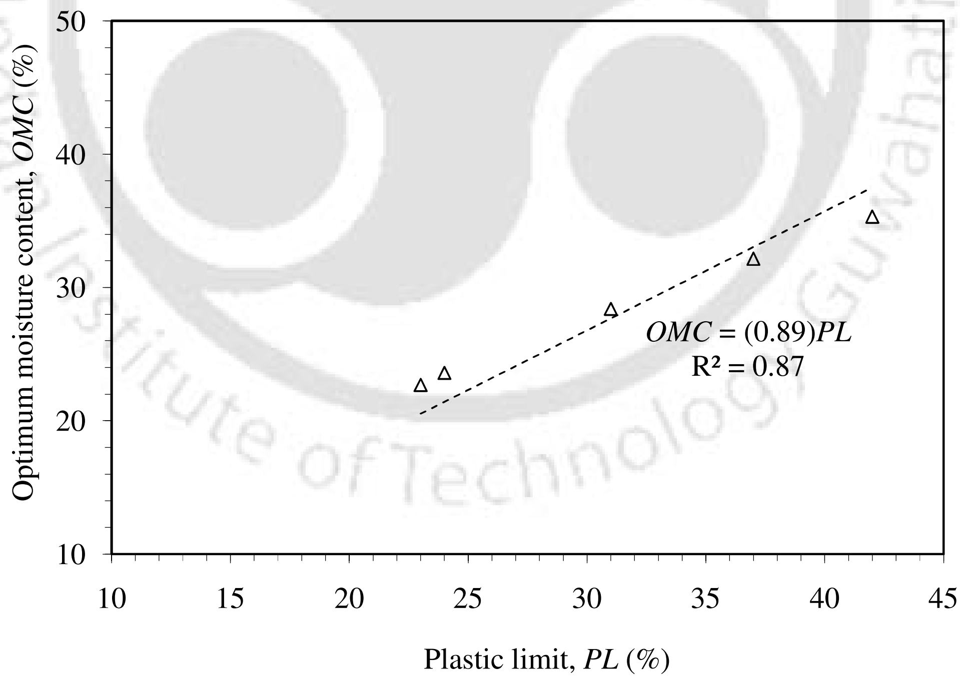


Fig. 5.5 Correlation of optimum moisture content with plastic limit for ES-FA mixes.

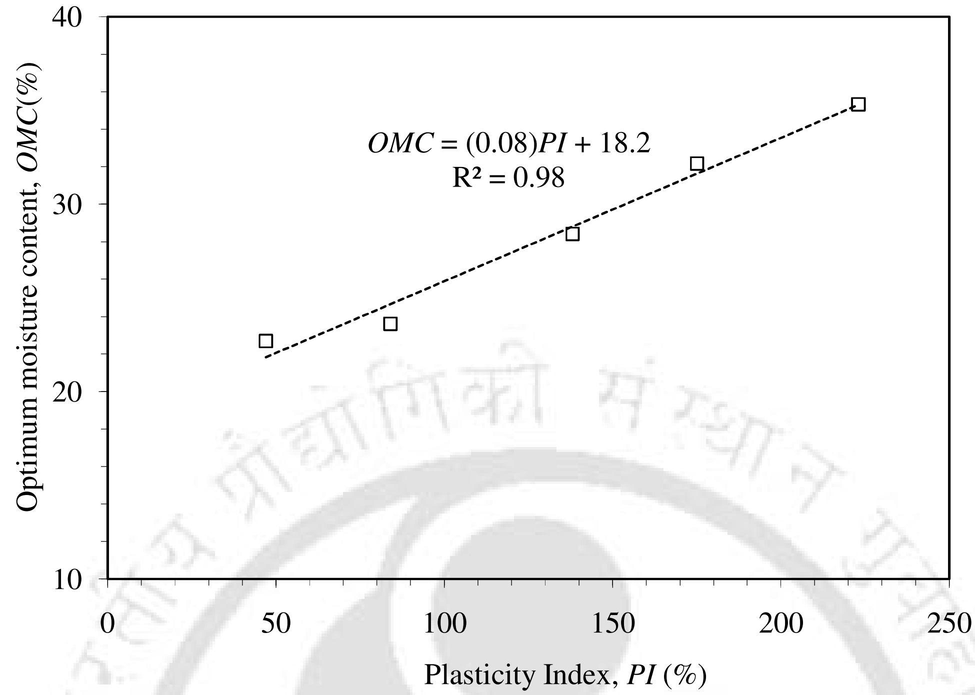


Fig. 5.6 Correlation of optimum moisture content with Plasticity Index for ES-FA mixes.

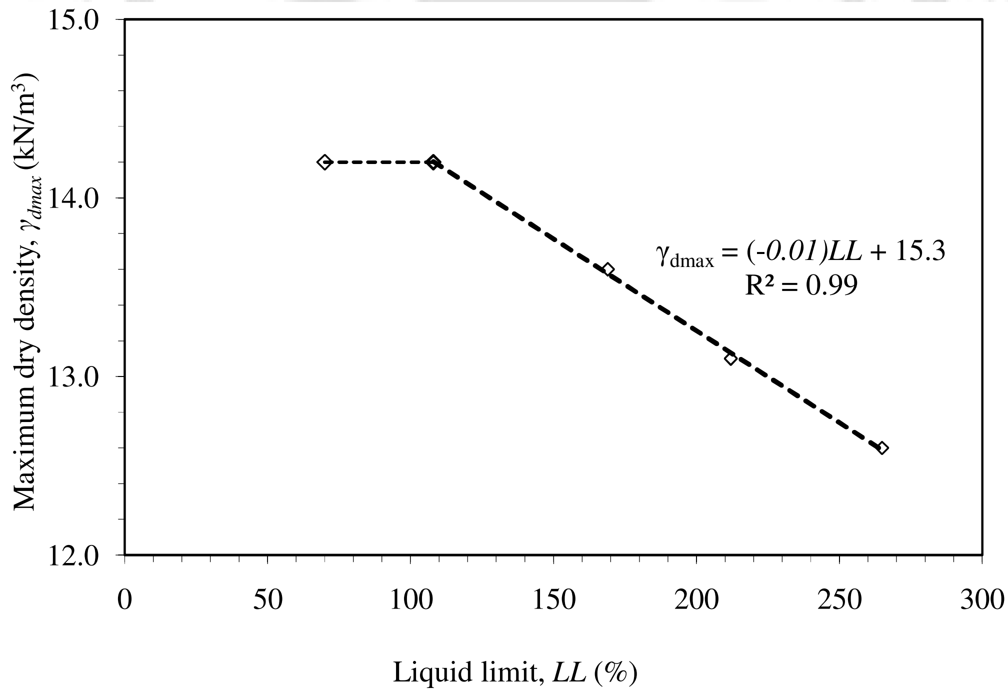


Fig. 5.7 Variation of maximum dry density with liquid limit for ES-FA mixes.

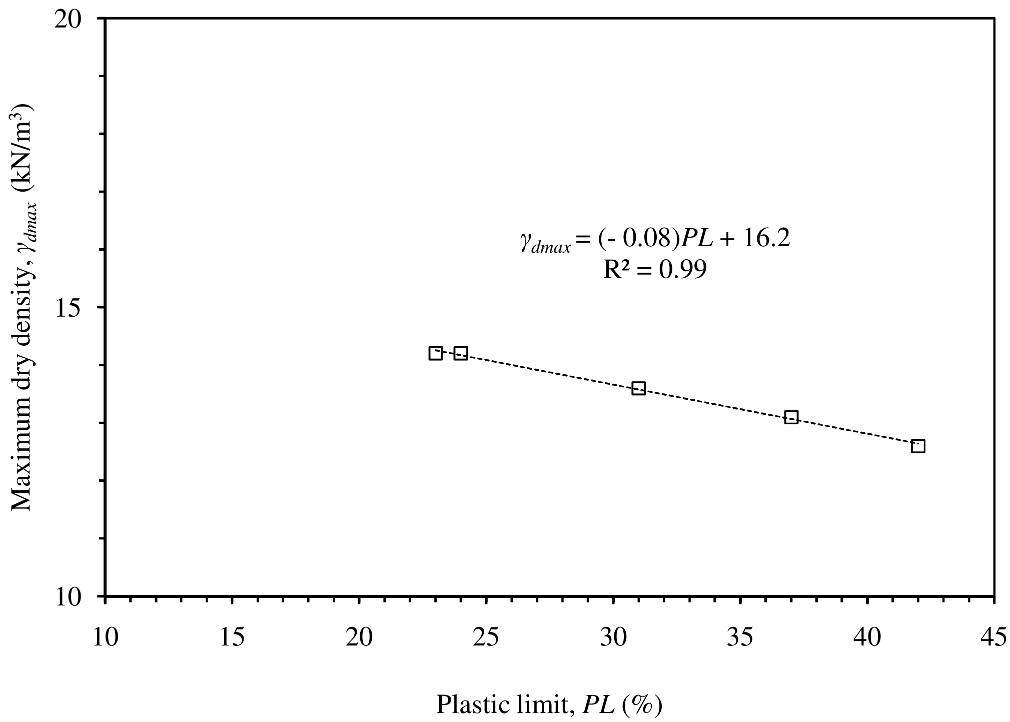


Fig. 5.8 Variation of maximum dry density with plastic limit for ES-FA mixes.

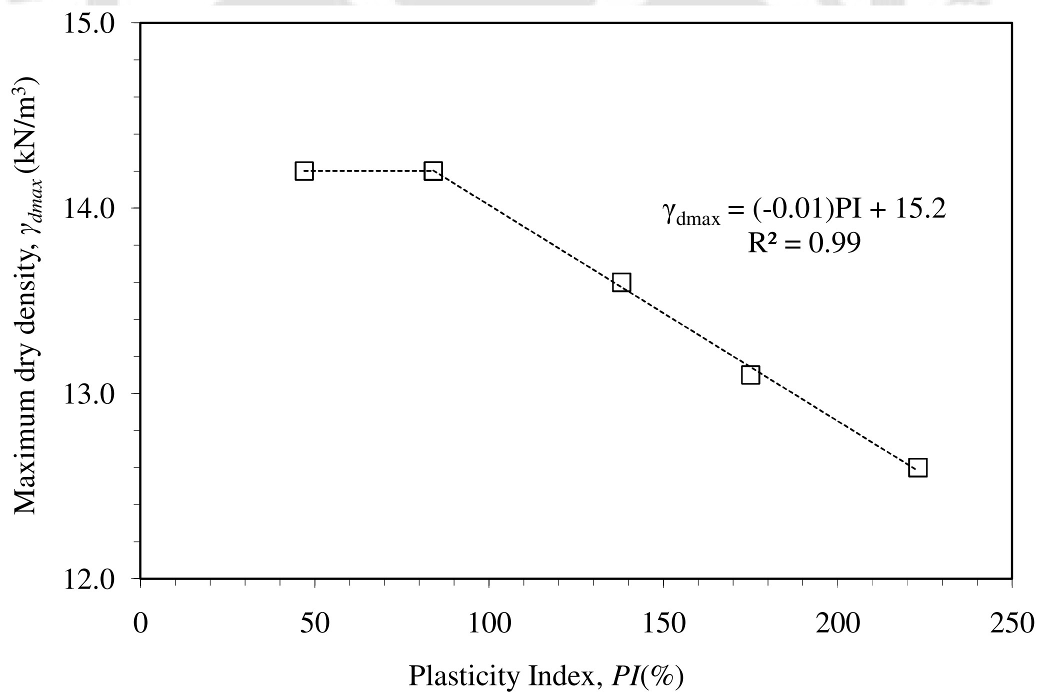


Fig. 5.9 Variation of maximum dry density with Plasticity Index for ES-FA mixes.

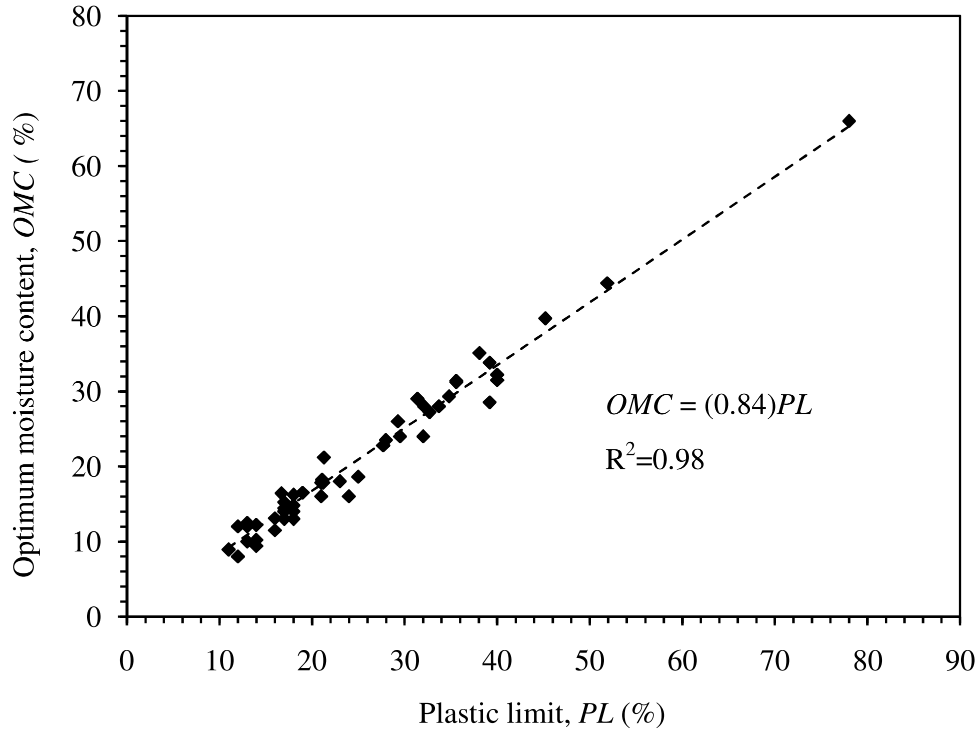


Fig. 5.10 Correlation of optimum moisture content with plastic limit based on literature data.

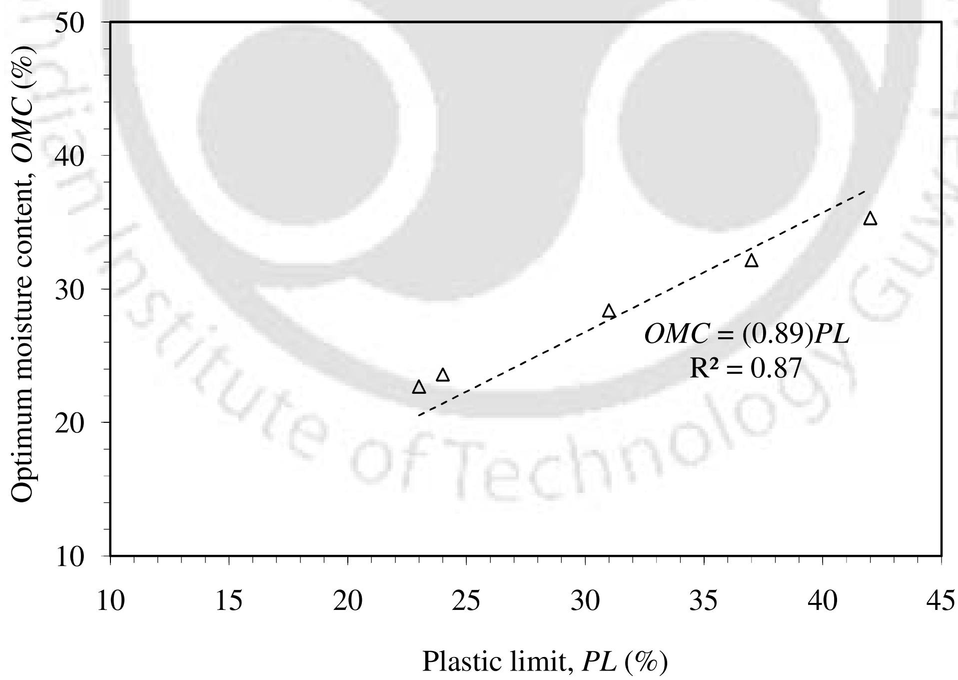


Fig. 5.11 Correlation of optimum moisture content with plastic limit for ES-FA mixes, with zero intercept.

The Eq. 5.9 obtained for ES-FA mixes matches closely with the correlation obtained from the literature data for natural soils (Eq. 5.8). This indicates that the inclusion of FA in ES does not affect much *OMC-PL* correlation of natural soils.

Assuming the soil to be fully saturated at the *PL* the corresponding dry density (γ_{dPL}) is obtained. Fig. 5.12 presents the variations of the maximum dry density (γ_{dmax}) with the dry density of the soil at its plastic limit (γ_{dPL}). The variation is almost linear and the correlation is:

$$\gamma_{dmax} = (0.64)\gamma_{dPL} + 4.79 \tag{5.10}$$

The correlation coefficient for this equation is 0.99.

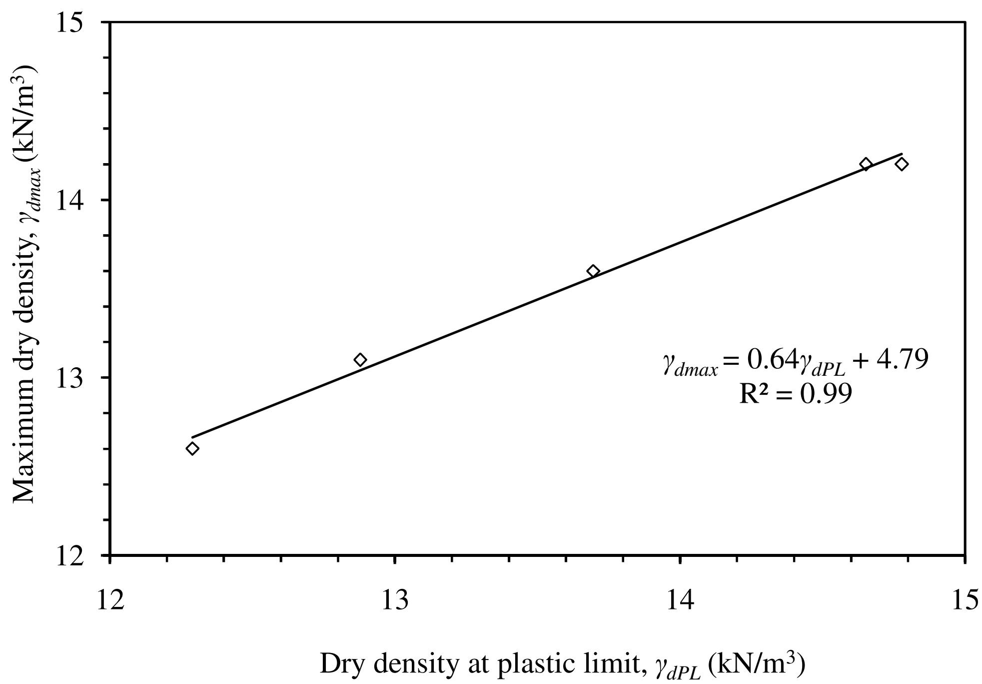


Fig. 5.12 Variation of maximum dry density with dry density at plastic limit for ES-FA mixes.

Gurtug and Sridharan (2002) for fine-grained soils have observed that the maximum dry density is about 98% that of the dry density at plastic limit water content, i.e.

$$\gamma_{dmax} = 0.98\ \gamma_{dPL} \tag{5.11}$$

The difference is attributed to the fly ash used in the present investigation.

5.3 COMPACTION BEHAVIOUR OF LIME TREATED ES-FA MIXES

The effects of lime on compaction response of soil-fly ash mixes, with varied fly ash contents, are presented in Figs.5.13 to 5.18. In the figures, legends are designated based on FA and lime content, e.g. F20L1 means, the specimen was prepared by mixing 20% FA and 80% ES, and it had 1% lime by weight.

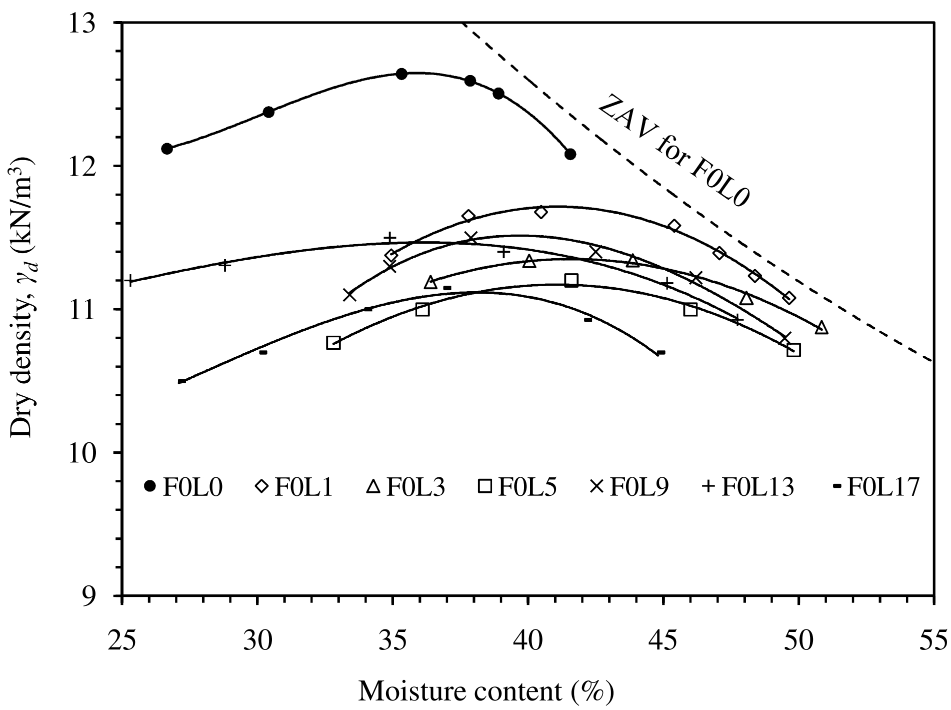


Fig. 5.13 Effect of lime on compaction response of expansive soil (F0).

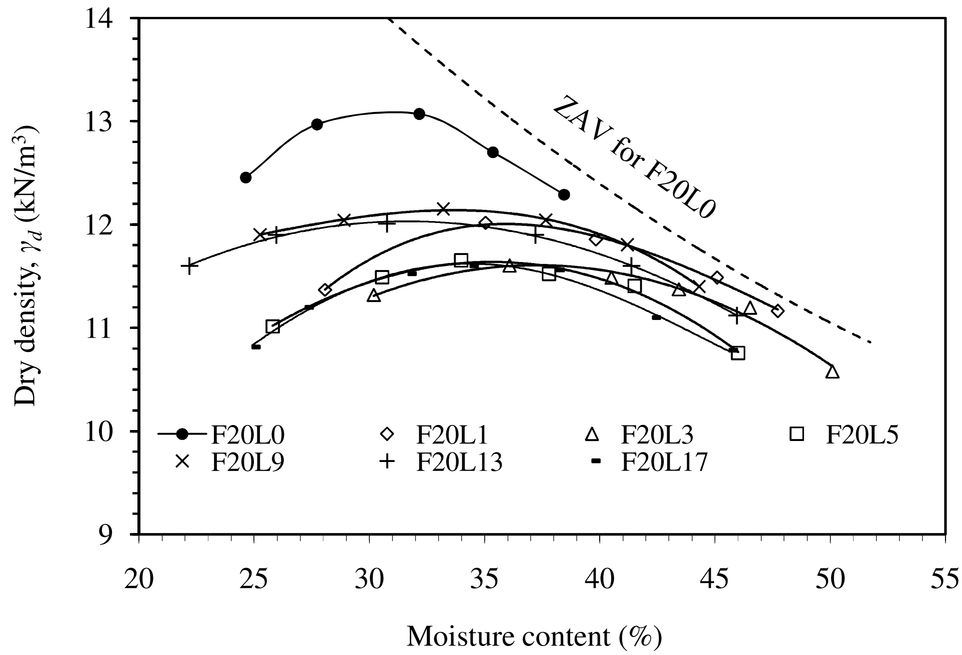


Fig. 5.14 Effect of lime on compaction response of ES-FA mix F20.

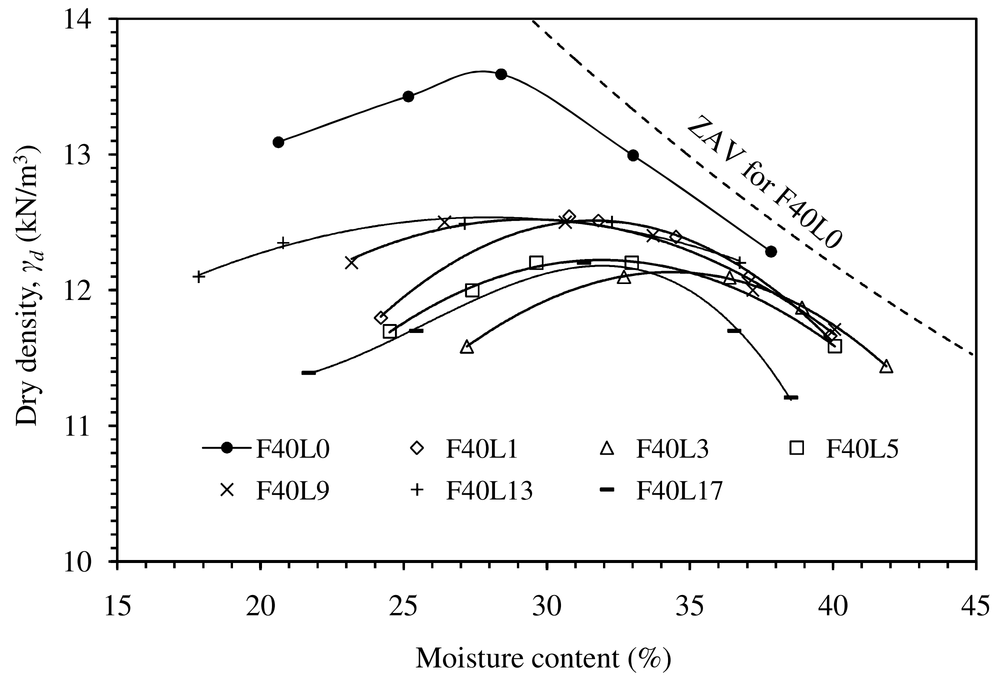


Fig.5.15 Effect of lime on compaction response of ES-FA mix F40.

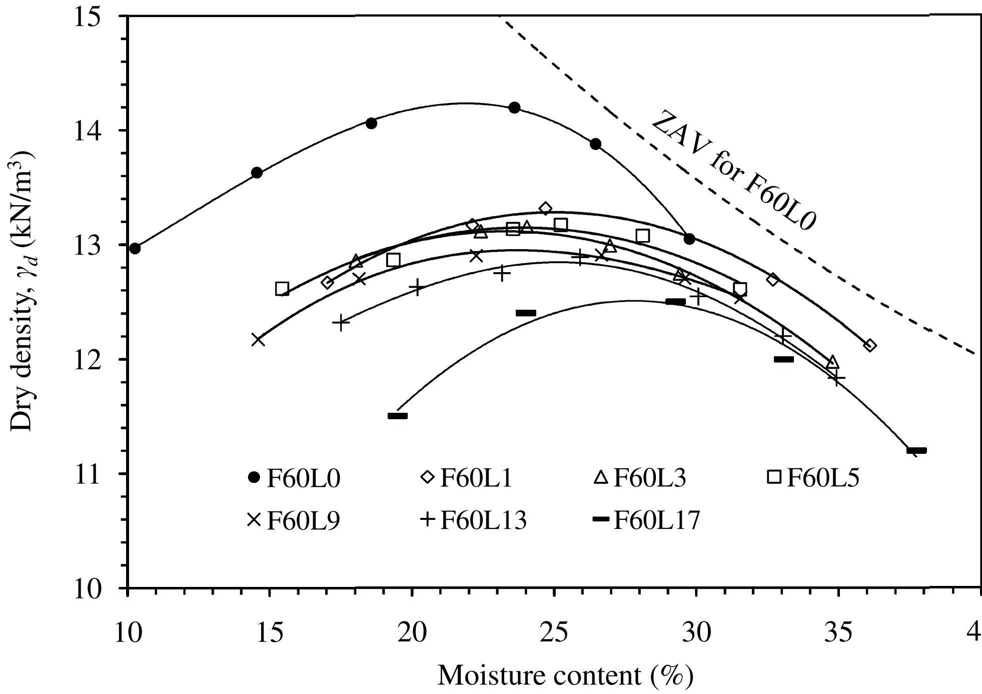


Fig. 5.16 Effect of lime on compaction response of ES-FA mix F60.

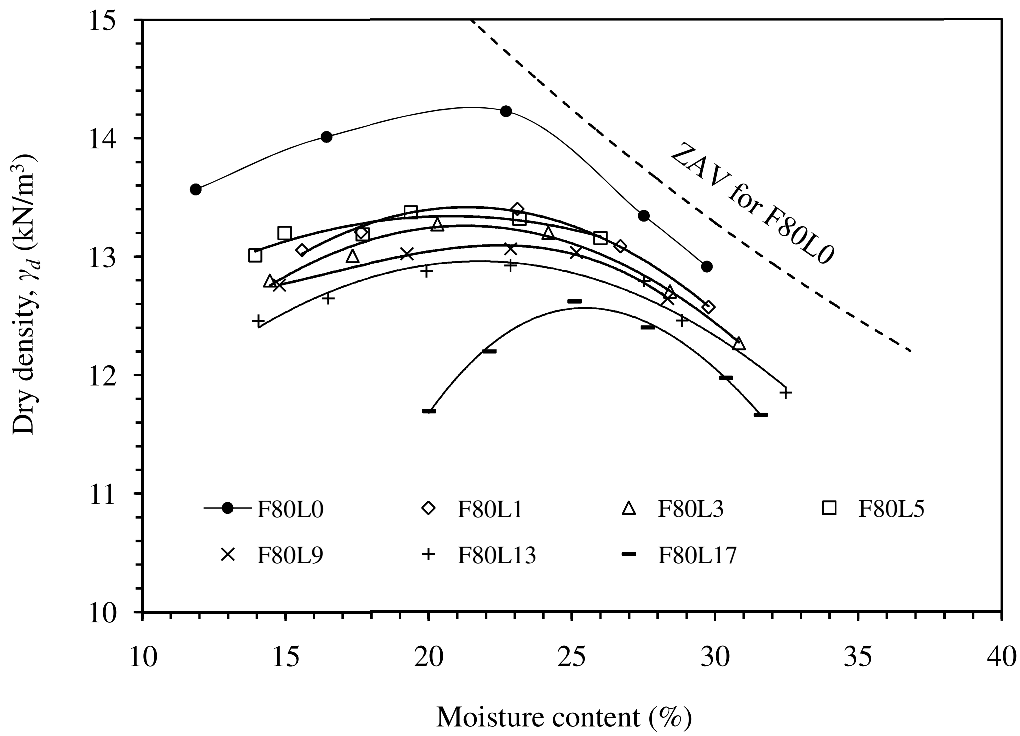


Fig. 5.17 Effect of lime on compaction response of ES-FA mix F80.

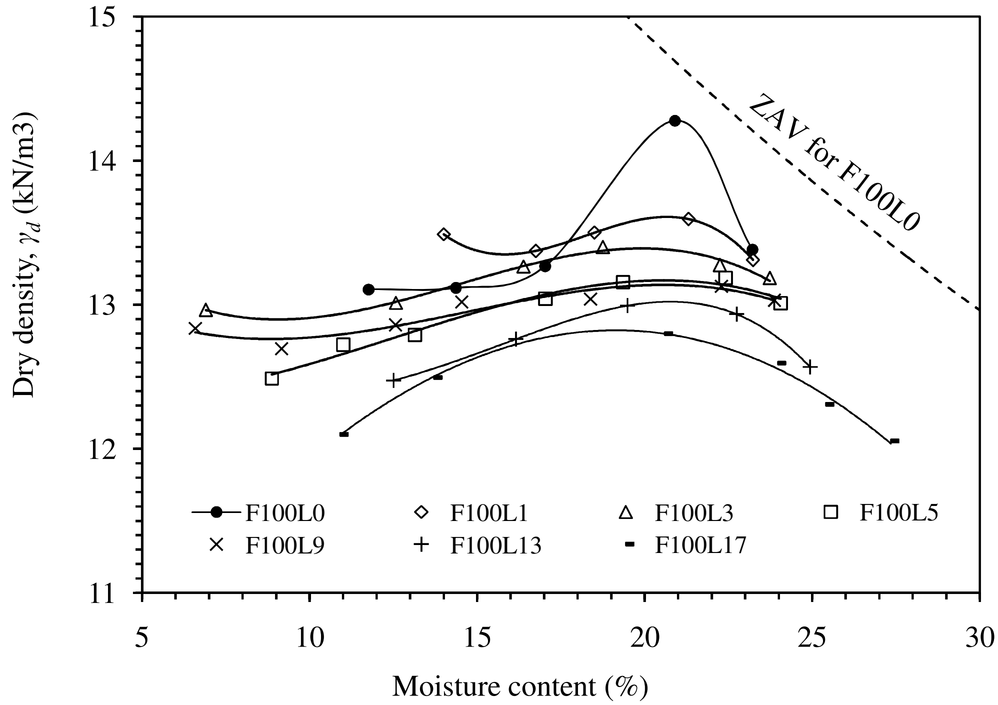


Fig. 5.18 Effect of lime on compaction response of fly ash (F100).

It could be observed that, in general, with addition of lime, the compaction curves have become flatter. It can be said that lime treatment can further improve the workability of the soils and hence the target density can be achieved over a wide range of water content. In other words it can be said that the plasticity of the soil has been reduced that it has turned relatively friable. This is attributed to the lime induced modifications of the diffuse double layer surrounding the soil particles. With reduced thickness of the double layer, achieved through lime treatment, the volume of the viscous pore water has reduced. As a result of which the plasticity of the soil mass has reduced leading to improved workability. The *OMC* and *MDD* for different soils at varied lime contents, as obtained from Figs. 5.13–5.18, are presented in Figs. 5.19 and Fig. 5.20 respectively. It could be observed that when clay content is relatively more, ≥60% (i.e. F0, F20 and F40), the *OMC* marginally increases with lime content and then reduce and stabilize at higher percentage of lime (i.e. ≥13%). Correspondingly, the dry density of the soils, initially reduces till about 3% lime content and subsequently increases with increased lime content to nearly stabilize at higher percentage of lime. However, when FA is the dominant component, i.e. F80 and F100, the trends are a bit different. The optimum moisture content continues to increase and correspondingly the dry density continues to reduce with the increase in the lime content.

For clay and clay rich soils, as the electrolyte concentration of pore fluid increases due to increase in lime content, the thickness of the double layer reduces. This leads to increased Van der Waals attraction between particles (Guven, 1992; Sogami and Ise, 1984) giving rise to flocculation and hence a cardhouse type of structure. This cardhouse structure effectively resists the compaction leading to reduced density of soil (Sweeney et al., 1988). Besides, being an open structure, it holds large amount of water that has produced increased *OMC*. With lime content increasing further, the concentration of cations in pore fluid adjacent to the clay surface increases. This difference in charge leads to osmosis that tends to equalize the charge through diffusion of free water towards the clay surface (Mitchel and Soga, 2005). This causes separation of clay particles giving rise to dispersed soil structure. As the dispersed structure is easily compacted, the overall density of the soil matrix has shown an increasing trend. At very large lime content, this process almost stabilizes that the *OMC* and *MDD* remains practically unchanged.

The difference in the trend of variation of *OMC* and *MDD* in case FA and FA-rich soils is attributed to the pozzolanic products (CSH) formed out of reaction between the lime (CaO) and the silica present in the fly ash. It should be mentioned here that in order to simulate the time lag that generally takes place in the field, the compaction tests were carried out with nearly one hour time lag. As the pozzolanic products are gels and therefore viscous in nature, they induce higher shear resistance against compaction leading to reduced density and increased water content of the soil mass. Besides, the gel water too is expected to have increased the *OMC* of the soil mass.

Figs. 5.19 and 5.20 show the variation of *OMC* and *MDD* with FA content for different percentages of lime added. It could be observed that at all percentage of lime, FA has significant influence on the *OMC* and *MDD* of the soil. In comparison to the data presented in Figs. 5.19 and 5.20, it is of interest to note that while change in *OMC* and *MDD* stabilizes at about 15% of lime, with FA the change is significant upto a very large percentage, as high as 80% (Figs. 5.21 and 5.22). Hence it can be said, that as compared to the lime, the variation of *OMC* and *MDD* is influenced more by the FA content. It can also be noted that the influence of lime content on *OMC* is more for low FA percentage as shown in Fig. 5.20.

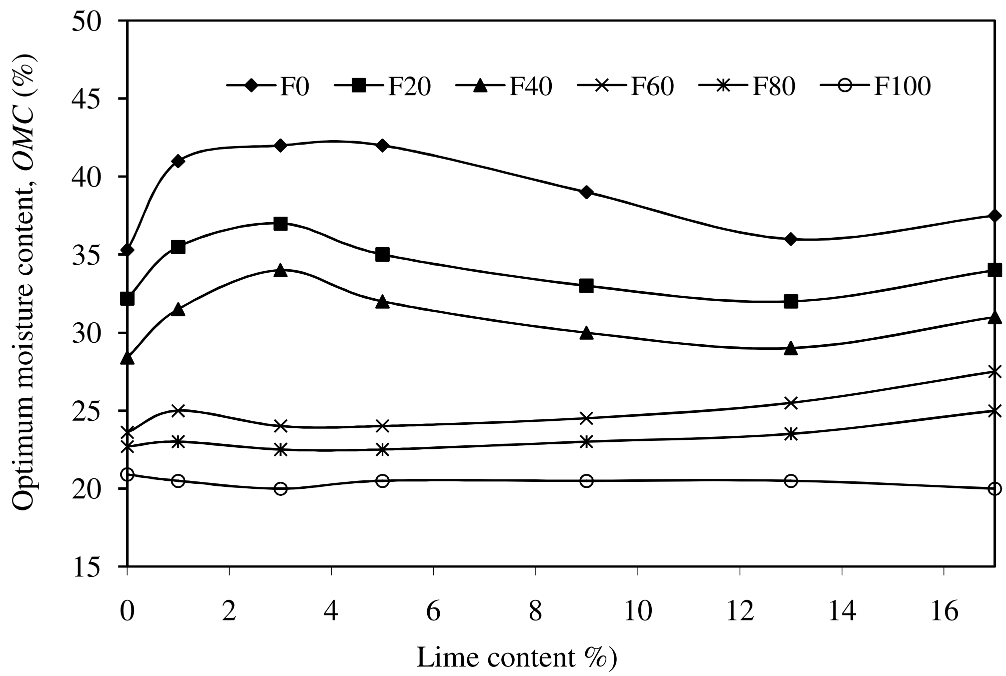


Fig. 5.19 Variation of optimum moisture content with lime content in ES-FA mixes.

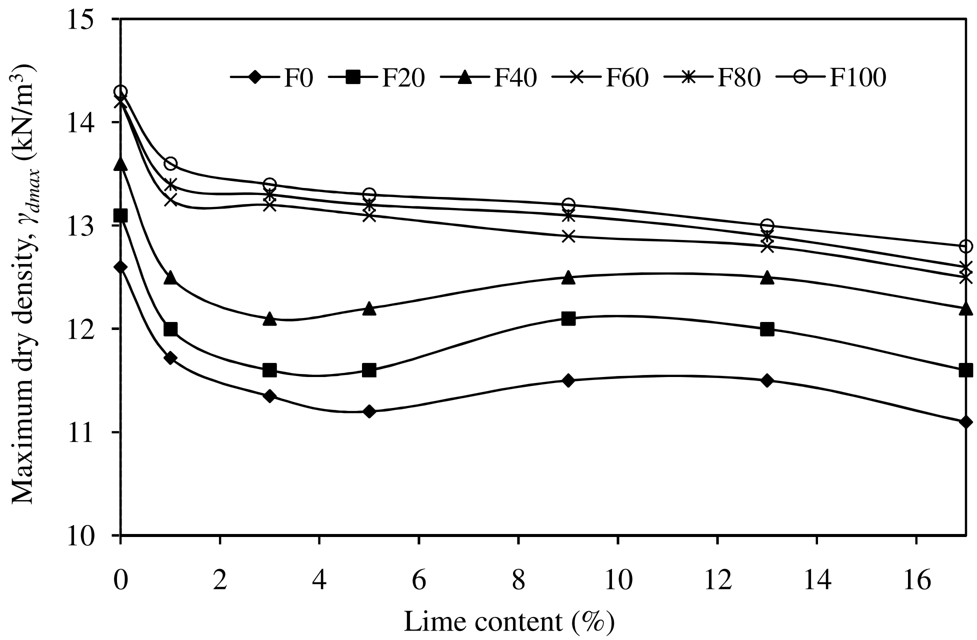


Fig. 5.20 Variation of maximum dry density with lime content in in ES-FA mixes.

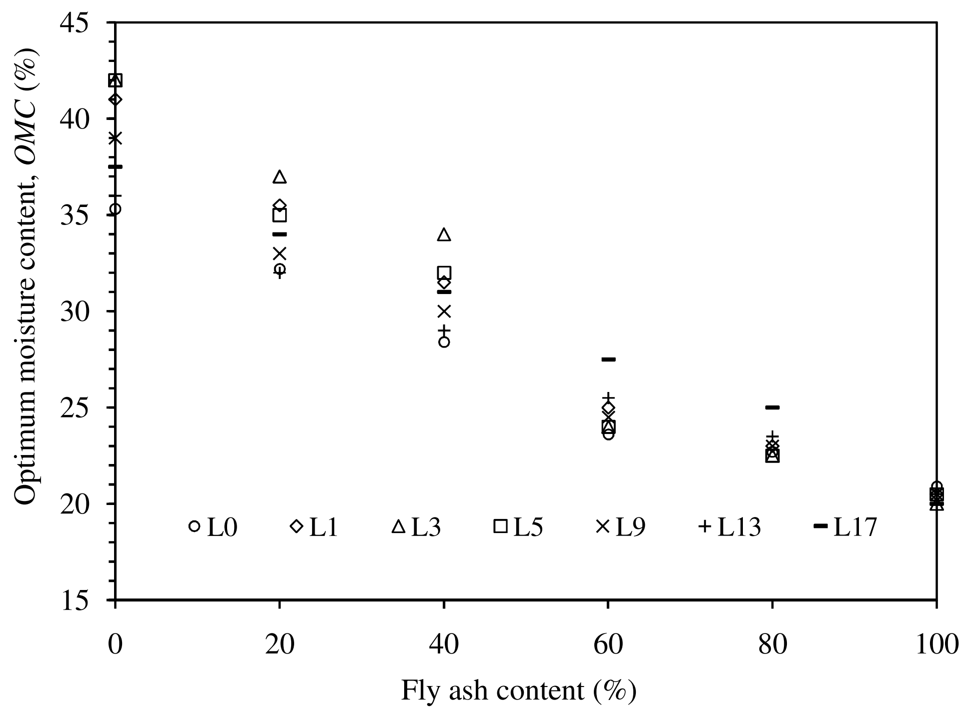


Fig. 5.21 Variation of optimum moisture content with fly ash content in lime treated soils.

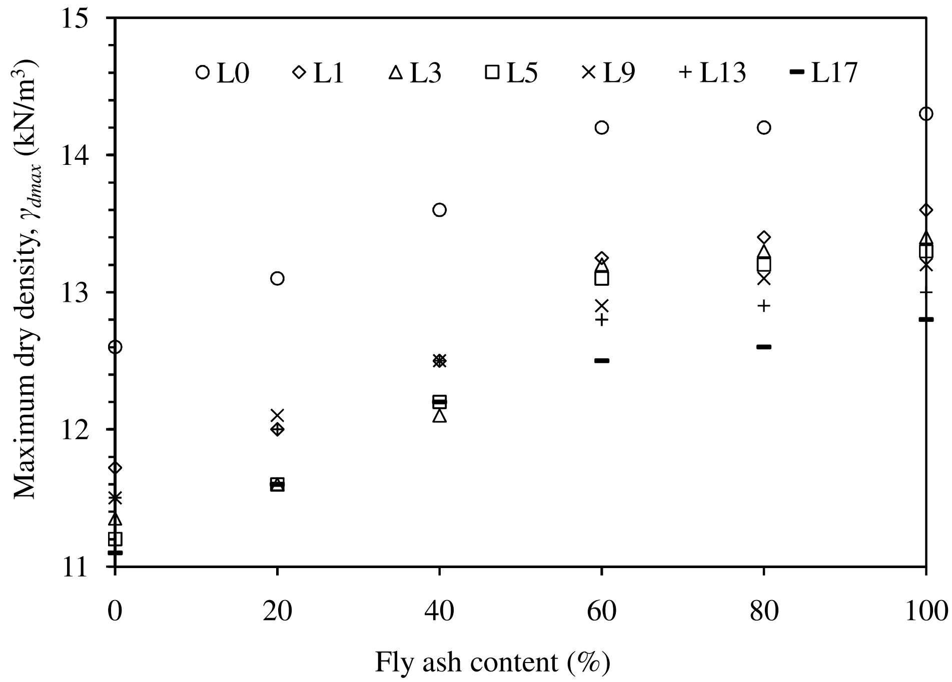


Fig. 5.22 Variation of maximum dry density with fly ash content in lime treated soil.

5.3.1 Correlations

Variations of *MDD* with *OMC* for different soils (i.e. ES, ES+FA, FA) with varied lime content, is depicted in Fig.5.23. It could be observed that *MDD* decreases with increase in *OMC* and the variation is almost linear. The correlation equation of these two parameters is found to be:

$$\gamma_{dmax} = (-\,0.1)OMC + 15.3 \tag{5.12}$$

The correlation coefficient (R^2) for this equation is 0.9.

The variation of *OMC*, with *LL*, *PL* and *PI* are shown in Fig. 5.24 to 5.27. It could be observed that the *OMC* has increased almost linearly with the increase in the liquid limit and the plastic limit. This is attributed to increased water holding capacity of the specimens, indicated through increased liquid limit and plastic limit. The correlations for *OMC* with *LL* and *PL* are found to be as in Eq. 5.13–5.14.

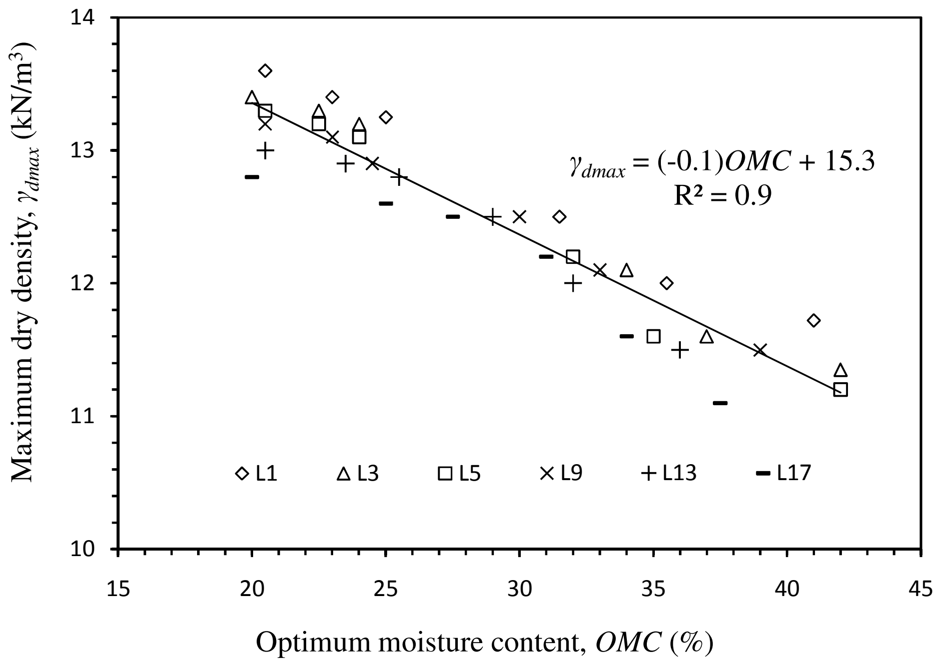


Fig. 5.23 Variation of maximum dry density with optimum moisture content for varied percentage of lime in ES-FA mixes.

$$OMC = (0.26)LL + 11.7\,, \qquad R^2 = 0.89 \text{ (for } LL > 40\%) \tag{5.13}$$

$$OMC = (0.37)PL + 10.1\,, \qquad R^2 = 0.83 \text{ (for } PL > 30\%) \tag{5.14}$$

$$OMC = (0.53)PL, \qquad R^2 = 0.64 \text{ (if intercept is set to zero)} \tag{5.15}$$

Eq. 5.15 has been presented to compare the *OMC–PL* relationship of lime treated ES-FA with those of natural fine-grained soils (Eq. 5.7 and 5.8) and ES-FA mixes without lime treatment (Eq. 5.9). It is obvious that *OMC–PL* relationship for lime-treated ES-FA mixes is not similar to that of natural fine-grained soils and ES-FA mixes. This indicates that the correlation presented in this study need to be used for expansive soils treated with lime.

It is interesting to note from Fig.5.27 that, the *OMC-PI* relationship for ES-FA mixes with 1% lime stands apart from the same with lime content > 1%. For comparison, the trend line for correlation of *OMC-PI* for ES-FA mixes without lime is also included in the figure. The trend line for correlation for 1% lime is positioned in between the trend lines for lime >1% and lime = 0%. In this context, it can be said that ES-FA mixes with 1% lime are in the transition stage of modification from lime = 0% and lime > 1%. The *OMC-PI* correlations for ES-FA mixes are denoted by:

$$OMC = (0.30)PI + 21.28, \quad R^2 = 0.97 \text{ (for lime content = 1\%)} \tag{5.16}$$

$$OMC = (1.14)PI + 14.34, \quad R^2 = 0.75 \text{ (for lime content > 1\%)} \tag{5.17}$$

The *MDD* is found to decrease with increase in the liquid limit and plastic limit of the specimen (Fig.5.28–5.29). The parameters have a nearly linear correlation and the equations are found to be:

$$\gamma_{dmax} = (-0.03)LL + 14.2, \quad R^2=0.76 \text{ (for } LL > 40\%\text{)} \tag{5.18}$$

$$\gamma_{dmax} = (-0.04)PL + 14.7, \quad R^2=0.89 \text{ (for } PL > 30\%\text{)} \tag{5.19}$$

Sridharan and Nagaraj (2005) obtained the following correlations between *OMC-PL* and *MDD–LL*:

$$OMC = 0.37(LL + 12.46) \tag{5.20}$$

$$OMC = 0.92PL \tag{5.21}$$

$$\gamma_{dmax} = 0.09(218 - LL) \tag{5.22}$$

$$\gamma_{dmax} = 0.23(93.3 - LL) \tag{5.23}$$

The above correlations (Eq. 5.20–5.23) obviously differ from the correlations obtained here for ES-FA mixes treated with lime, owing to the fact that the correlations in the literature were for soils without lime treatment. Based on the above comparisons, it is explicit that correlations for *OMC* and γ_{dmax} presented in this study (Eqs. 5.13-5.19) will be more appropriate for lime treated soils.

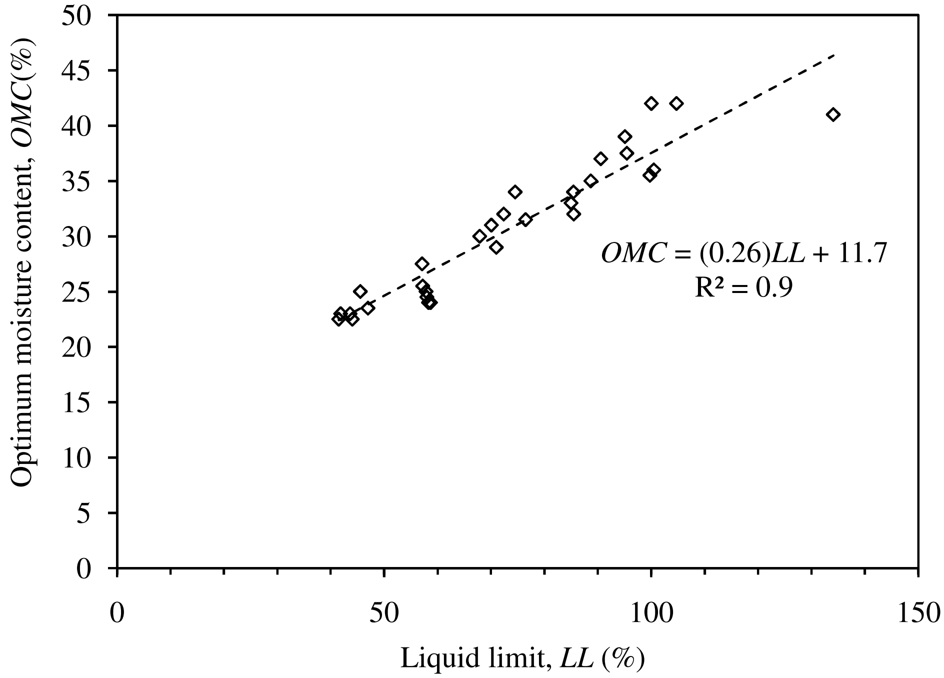


Fig. 5.24 Correlation of optimum moisture content with liquid limit for lime-treated ES-FA mixes.

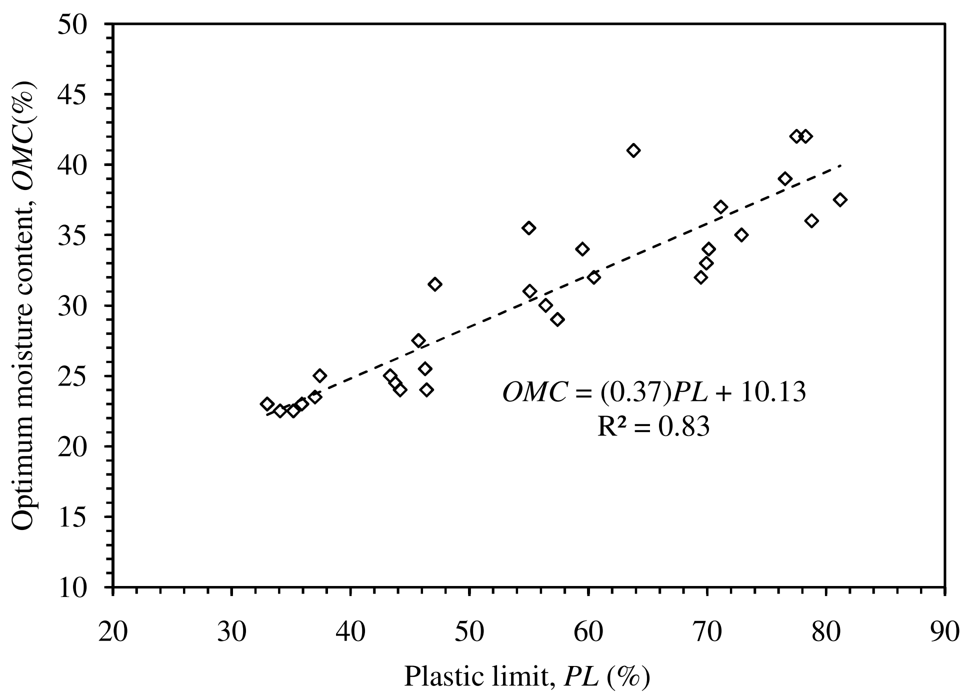


Fig. 5.25 Correlation of optimum moisture content with plastic limit for lime-treated ES-FA mixes.

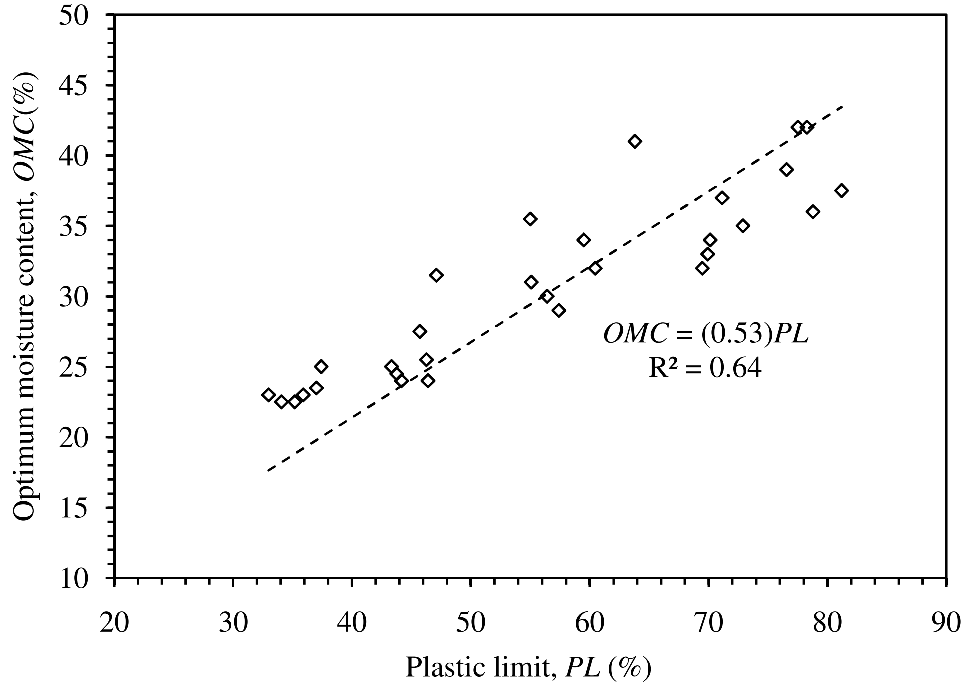


Fig. 5.26 Correlation of optimum moisture content with plastic limit for lime-treated ES-FA mixes, with intercept set to zero.

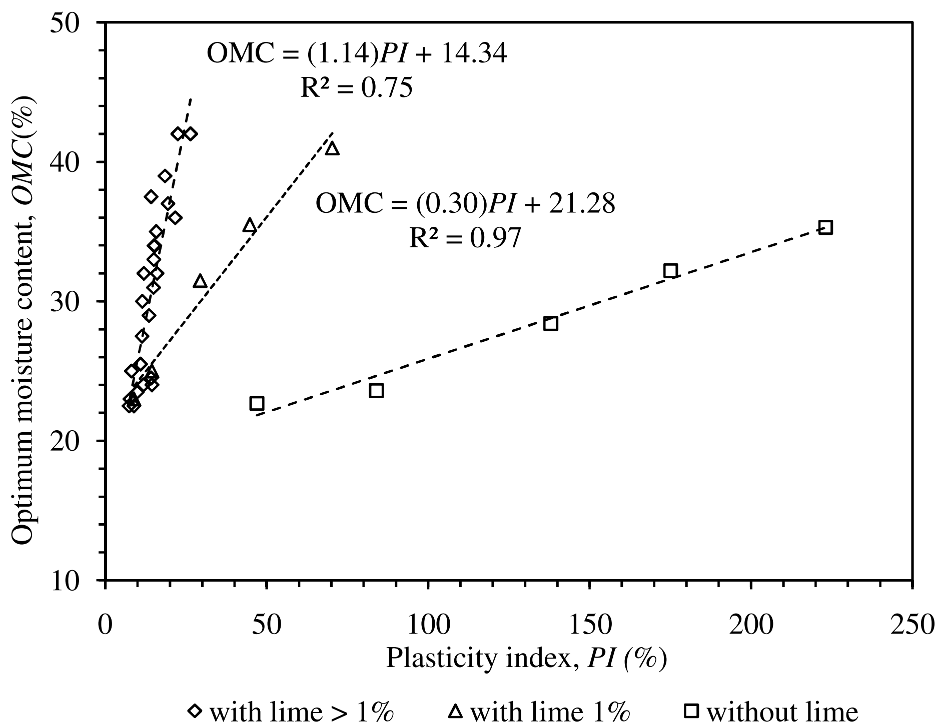


Fig. 5.27 Correlation of optimum moisture content with plasticity index for lime-treated ES-FA mixes.

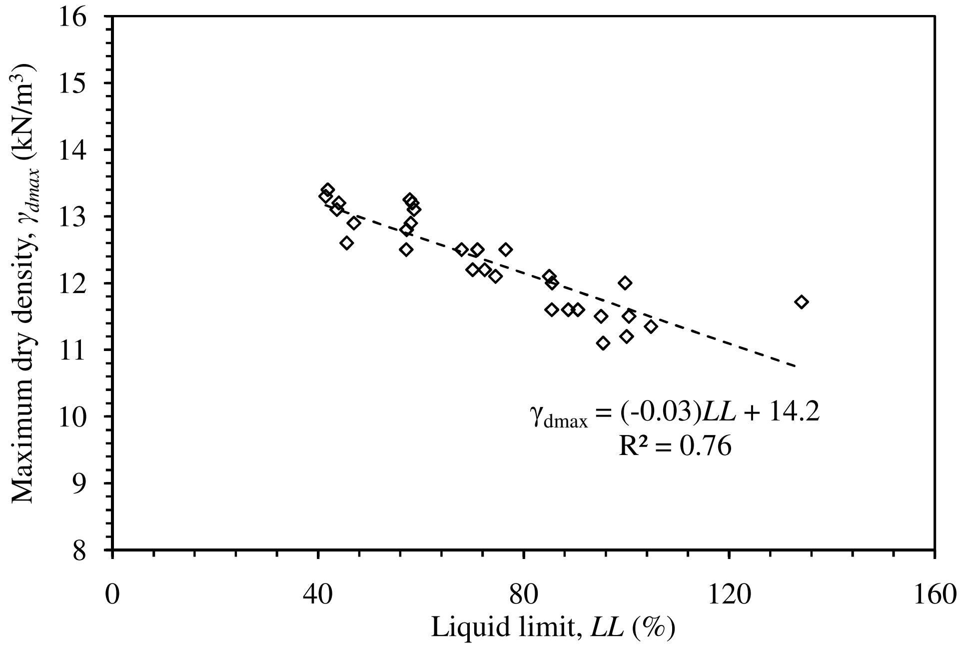


Fig. 5.28 Correlation of maximum dry density with liquid limit for lime-treated ES-FA mixes

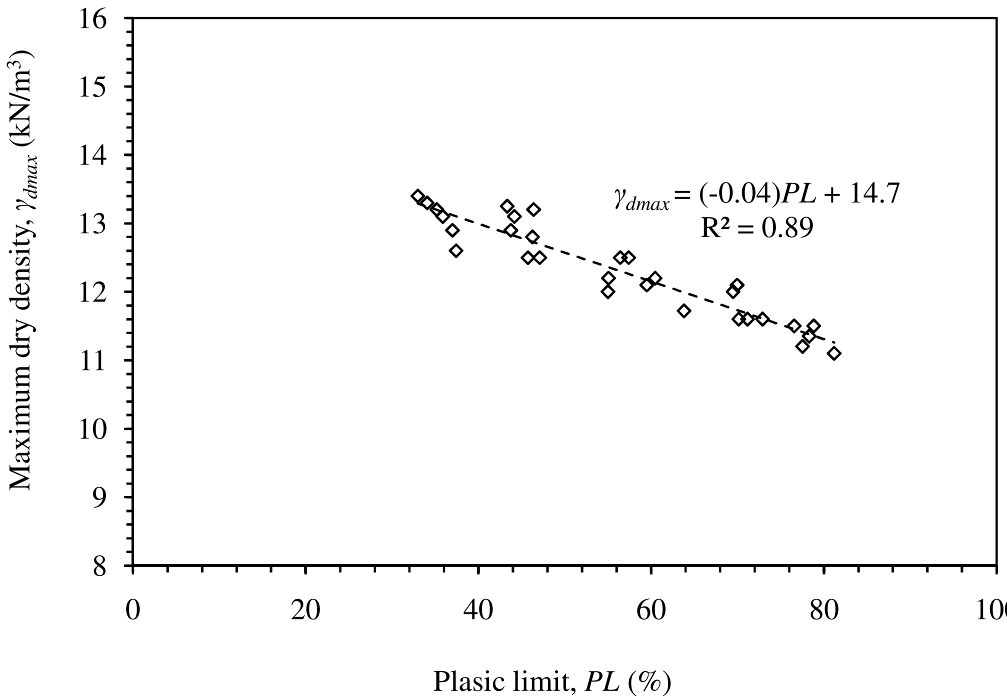


Fig. 5.29 Correlation of maximum dry density with plastic limit for lime-treated ES-FA mixes

5.4 SUMMARY

In this chapter, the compaction behavior of expansive soils amended with fly ash and lime are studied. It is observed that fly ash has increased the maximum dry density and reduced optimum moisture content. With addition of lime, the maximum dry density of the specimens reduce, the reduction depending on the quantity of lime and the amount of expansive soil in the amended soil. When the portion of expansive soil in the mixes is more, the optimum moisture content initially increases on application of lime.

The various index properties such as liquid limit and plastic limit are found to correlate well with the compaction parameters i.e. optimum moisture content and maximum dry density for fly ash amended expansive soil, with or without treatment by lime. It is noted that correlations for *OMC* and γ_{dmax} presented in this study will be more appropriate for lime treated soils. Inclusion of only fly ash in expansive soils (no lime treatment) is found to have negligible influence on such correlations obtained for natural soils.

CHAPTER 6

SWELL CHARACTERISTICS

6.1 INTRODUCTION

The change in moisture content causes large volume changes in expansive soils. The resulting heave and settlement cause the damage to the structures founded on such soils. Swelling of clays has been mainly attributed to the electrical double layer. The exchangeable cations in the clay-water system remain at some distance from the particle surface. The cations are attracted towards the clay surface because of the negative charge on the clay surface, but their thermal energy and concentration gradient makes them diffuse away from the surface. This attraction and thermal diffusion creates a diffused layer of cations with its concentration highest adjacent to the surface. The concentration gradually decreases with increase in distance from the clay surface (Sridharan, et al, 1986). The interaction of diffuse ion layers of adjacent particles gives an explanation for the properties of swelling.

The dipolar water molecules are adsorbed on the clay surface in multiple layers. The bonds between the clay mineral plates are weak in the case of expansive soils. Therefore, different layers of dipolar water molecules accumulating between the plates try to push them apart. Polarity of water molecules also enable them to hydrate the ions adsorbed on to clay surface. Komine and Ogata (1996) put forward a model for estimating swelling of montmorillonite, as depicted in Figs. 6.1 and 6.2.

When there is no interlayer water between the clay mineral plates, the distance between the two parallel montmorillonite plates is considered to be equal to the non-hydrated dia-

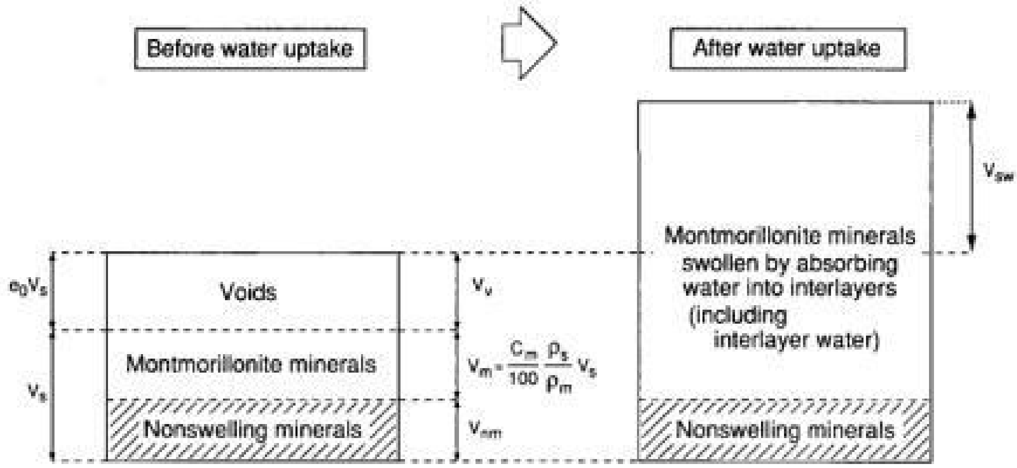


Fig.6.1 Swelling of montmorillonite (Komine and Ogata, 1996).

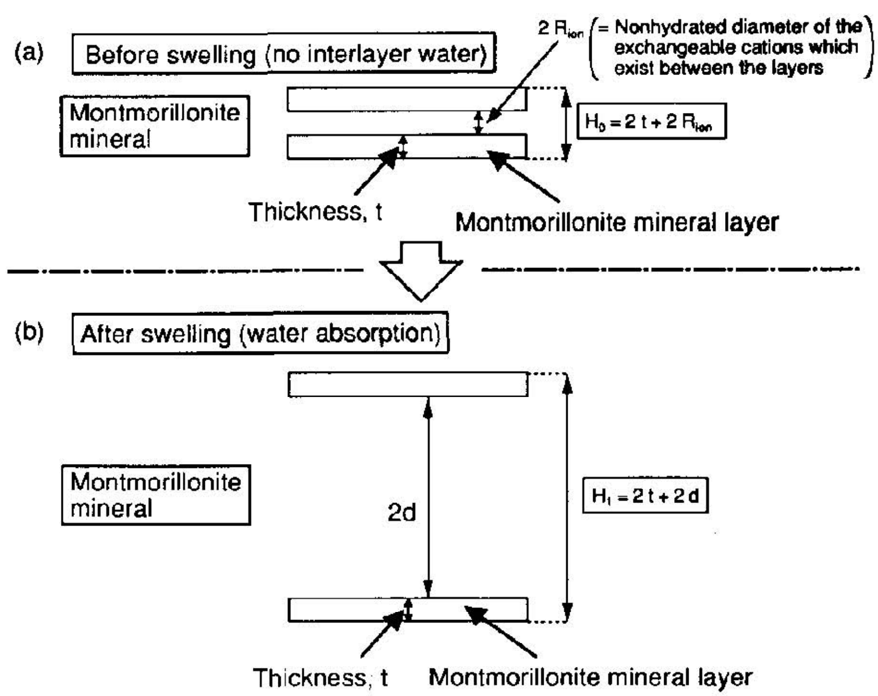


Fig.6.2 Mechanism of swelling (Komine and Ogata, 1996).

meter of the exchangeable cations ($2R_{ion}$) that exist between the two layers. After swelling the distance between the plates become 2d. The swelling volumetric strain of the swelling mineral is given by:

$$\varepsilon_{sv}(\%) = \frac{d - R_{ion}}{t + R_{ion}} X100 \tag{6.1}$$

Experiments have been conducted in this study according to the procedure discussed in Section 3.5.4 to examine the reduction in swelling of expansive soil by the combined addition of FA and lime. In Chapter 4, it was observed that plasticity of ES-FA-lime mixes initially decreased with lime content and then again increased at higher lime content. In view of this, the swell responses of ES-FA-lime mixes at higher lime content have also been examined. Efforts were also made to evaluate whether the methodology for swelling pressure estimation works well for lime and fly ash amended expansive soil. The obtained results are presented and discussed in the following sections.

6.2 SWELL BEHAVIOUR OF ES-FA MIXES

6.2.1 Free Swell of ES-FA Mixes

Free swell index (*FSI*) of ES-FA mixes is presented in Table 6.1 along with the index properties. Fig. 6.3 shows the variation of free swell index of ES with different FA contents after 24 hrs of soaking. The *FSI* of ES decreases from 367 % to 140 % when FA content varies from 0 to 80%. The 24 hrs swelling in terms of volume per gm of the mixes is also presented in the same figure. From the figure it can be noted that the reduction in *FSI* is most prominent with the addition of 40% of FA in the soil. Further addition of FA does not bring any significant reduction in *FSI*.

The relationship of free swell of ES-FA mixes has been plotted as a function of *LL* and *PI* as presented in Fig.6.4. Similarly, the relationship between free swell and specific surface area of the mixes has been presented in Fig.6.5.

Table 6.1 – Free swell index of ES-FA mixes with index properties.

Fly ash content (%)	*FSI* (%)	Swelled volume (cm^3 per gm of soil)	Liquid limit (%)	Plastic limit (%)	Plasticity Index (%)	Sp. surface area (m^2/gm)
0	367	4.9	265	42	223	73.6
20	205	3.2	212	37	175	57.7
40	167	2.8	169	31	138	43.5
60	160	2.6	108	24	84	29.7
80	140	2.4	70	23	47	11.7

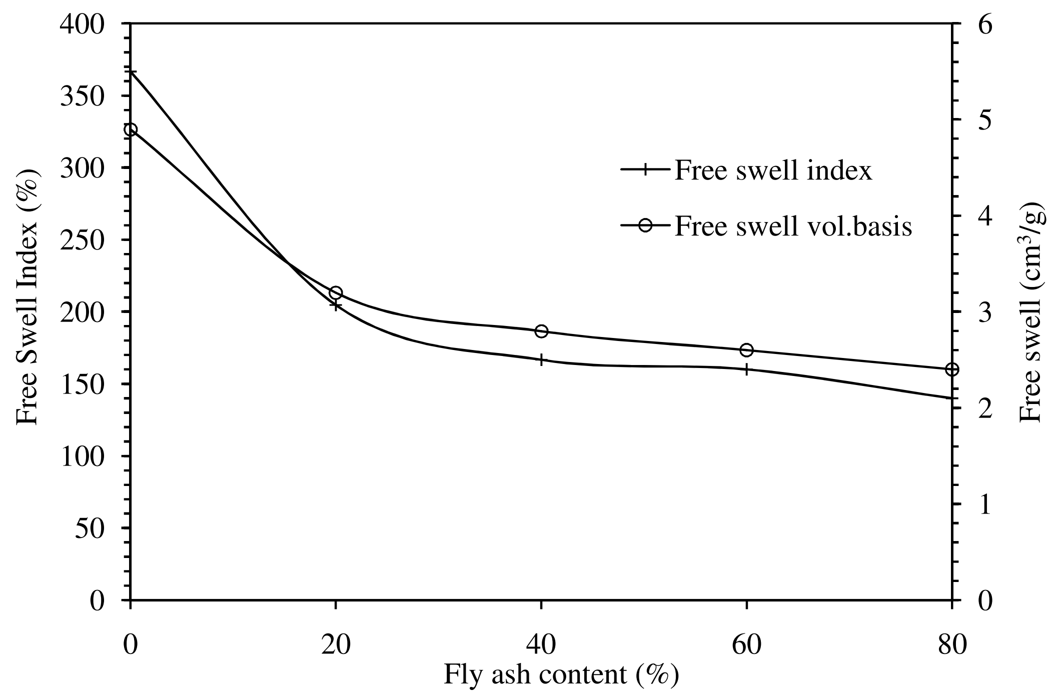
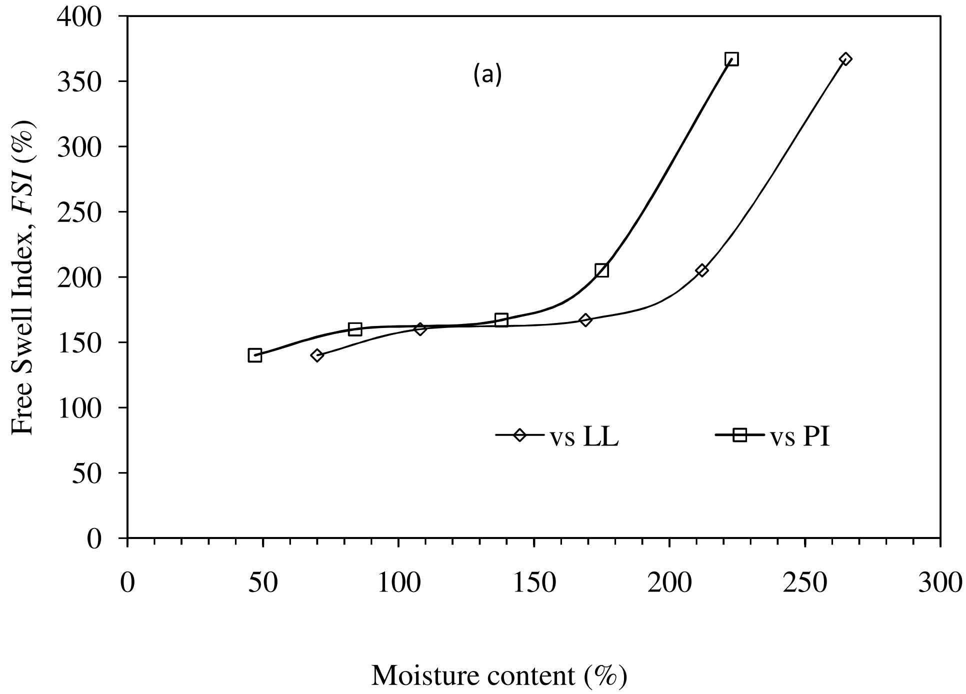


Fig. 6.3 Free swell of ES-FA mixes expressed as Free Swell Index and swelled volume per gm of soil.



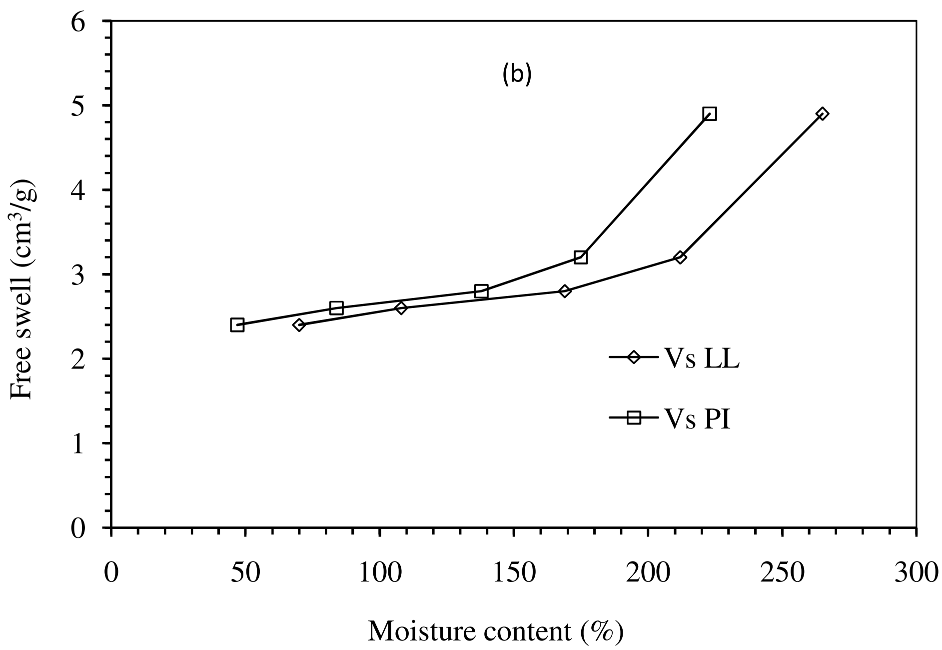


Fig.6.4 Variation of (a) Free Swell Index and (b) free swell on volume per gm with liquid limit and Plasticity Index of ES-FA mixes.

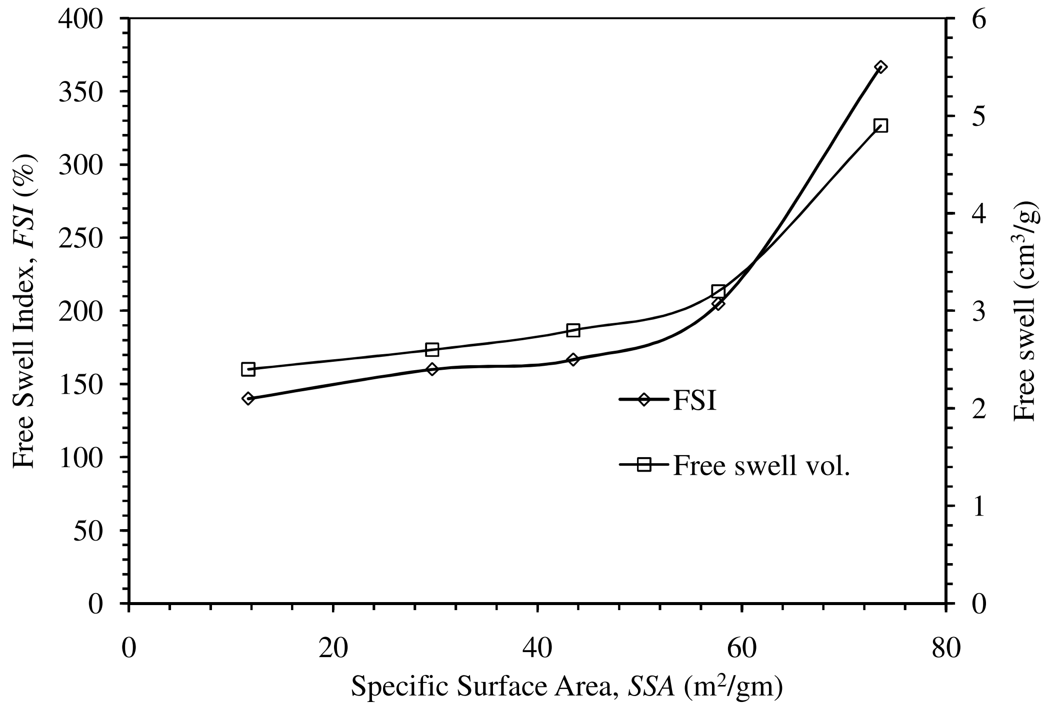


Fig.6.5 Variation of *FSI* and free swell volume per gm with *SSA* of ES-FA mixes.

From the Figs. 6.4 and 6.5, it can be observed that the free swell of ES-FA mixes increases with an increase in *LL*, *PI* and *SSA*. However, this increase is substantial for mixes with FA less than 40%. For FA content greater than 40%, the increase in free swell property with plasticity characteristics and SSA is not very prominent. This indicates that 40 % replacement of ES by FA is effective in appreciably reducing the free swell of the former.

6.2.2 Swelling Characteristics of ES-FA Mixes in Oedometer

Swell percentage (ε) of ES-FA mixes was studied using the oedometer method for a period of 28 days, as described in Section 3.5.4. The essential difference between ε and *FSI* is that the former is conducted on samples compacted at *OMC* where as the latter is performed on loosest state of the ES-FA mix. Fig. 6.6 shows the variation of ε with time for different ES-FA mixes.

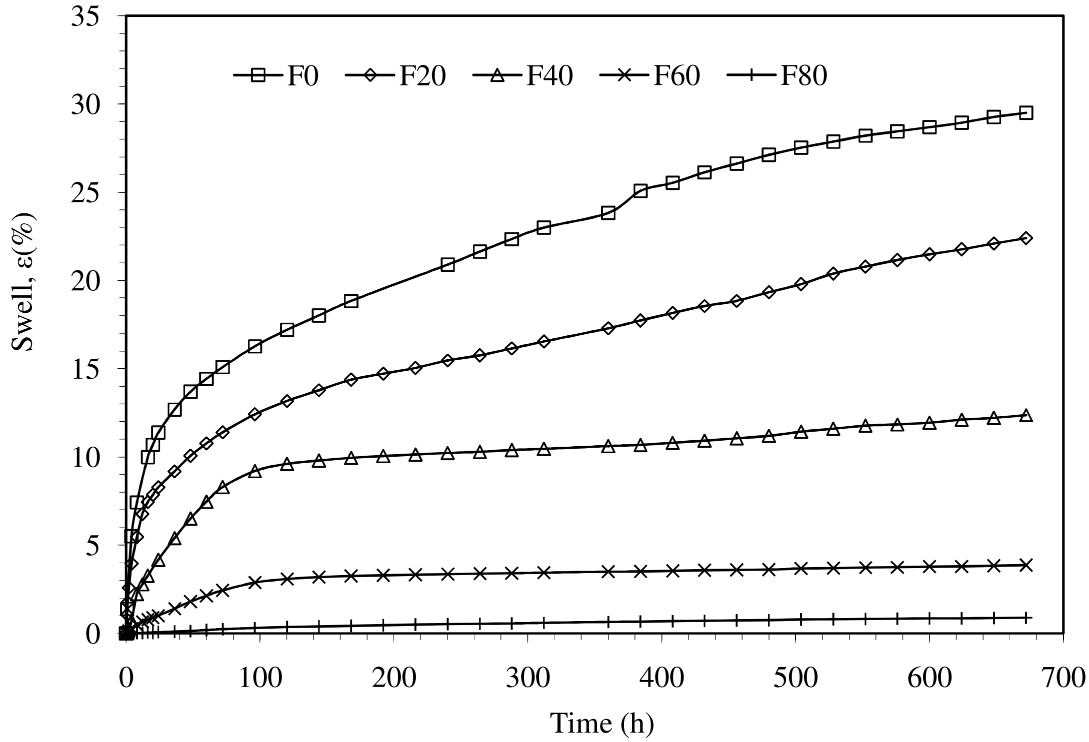


Fig.6.6 Percentage swelling of ES-FA mixes in oedometer.

It can be noted that the maximum *ε* of ES recorded at the end of 28 days is 30%, which reduces to 1% for F80. It must be noted that swell percentage has not reached equilibrium condition even after 28 days for those mixes with less than 40% FA content. For mixes with high FA content (> 40%), *ε* has reached equilibrium value within 7 days. For the mix with 40 % FA, at the end of 28 days, there is a tendency to swell, however it is very less.

The initial higher rate of swelling is the result of immediate adsorption of water layers around clay particles. The slow rate of swelling at a later stage is attributed to expansion of diffused double layers of adjacent particles (Dakshanamurthy, 1978). The shapes of the swelling curves can be considered similar to that of a rectangular hyperbola. If these results are plotted in terms of t/ε vs t, the curves become linear, as depicted in Fig.6.7.

The equation of the straight line has been used to represent these trends as given by Eq. 6.2.

$$\frac{t}{\varepsilon} = a + bt \tag{6.2}$$

Where, t = time

ε = swell (%)

a = intercept of the straight line on the t/ε axis, and

b = slope of the line

From the above equation, $\varepsilon = \frac{1}{b+\frac{a}{t}}$. As $t \rightarrow \infty$, $\varepsilon = 1/b$.

Therefore, the maximum swell percentage (ε_{max}) of the ES-FA mixes can be estimated from the inverse of the slope of the respective linear characteristics (Dakshanamurthy, 1978). The highest swelling soil in this type of plot will have the least slope, and vice versa. For F80, the initial non-linear portion of the curve (presented in Fig. 6.7) has not been considered while estimating ε_{max}. This non-linear response is attributed to the readjustment of the relatively coarse fly ash particles and the associated voids. The ε_{max} of ES-FA mixes has been estimated using the above procedure and has been compared with the measured result observed at the end of 28 days as shown in Fig.6.8. It can be noted that the estimated and measured values of ε matches very well for mixes with FA content ≥ 60%. For these mixes, swelling has reached an equilibrium state within 28 days as shown Fig.6.6. Therefore, ε_{max} estimated is expected to match with the measured results. This clearly indicates that the procedure proposed by Dakshanamurthy (1978) for the estimation of ε_{max} works well for ES-FA mixes. For mixes with FA < 60 %, the measured swell percentage has not reached equilibrium at the end of 28 days (refer Fig. 6.6). Hence, there is a difference in measured and estimated ε_{max} for these mixes, as depicted in Fig. 6.8. The difference is minimal for F40 but relatively more for F20 and F0. Hence it can be said that by using Dakshanamurthy's method and short term oedo-

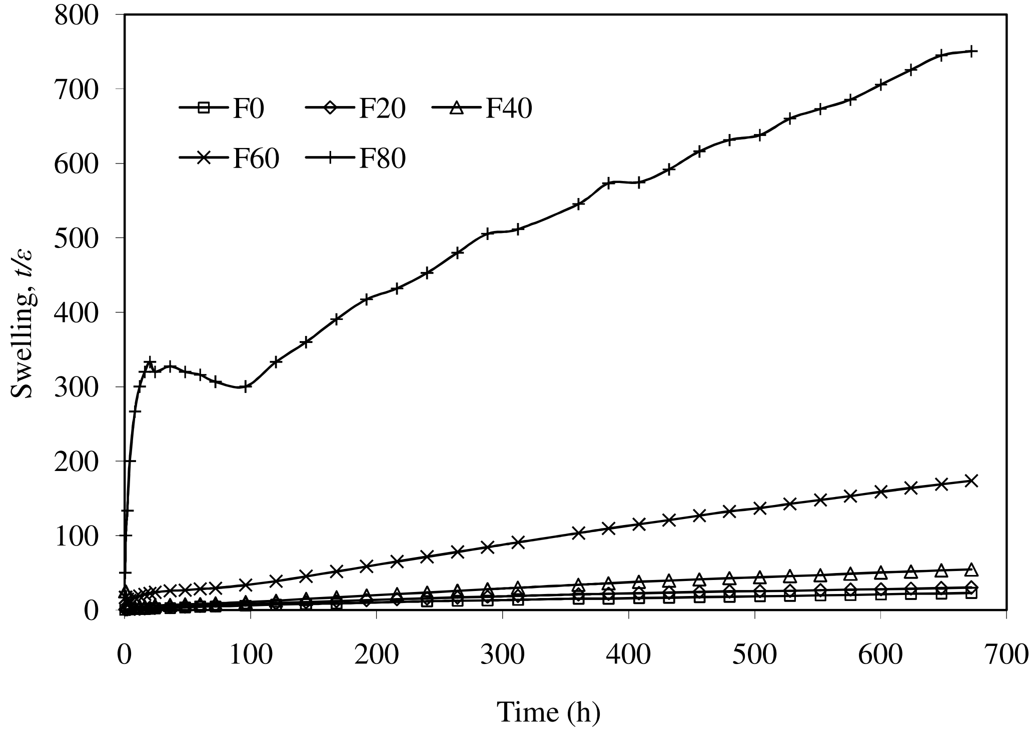


Fig.6.7 Linearized plots for swell percentage of ES-FA mixes in oedometer.

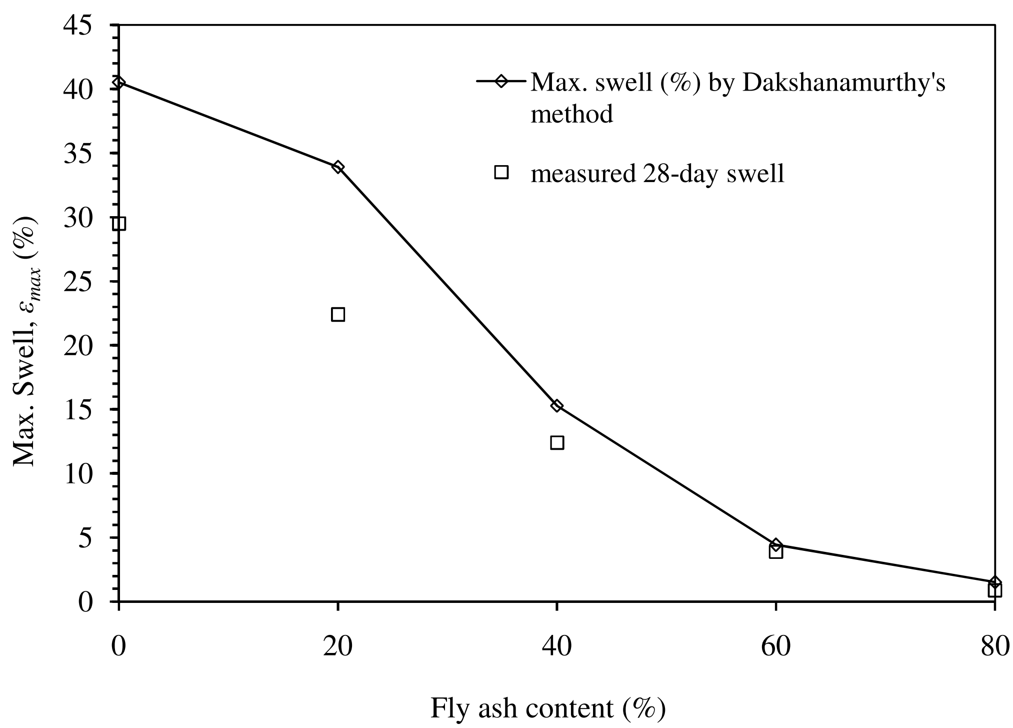


Fig.6.8 Swelling potential of ES-FA mixes using Dakshanamurthy's (1978) method.

meter swell test data, the ε_{max} of ES-FA mixes can be estimated reasonably. The study clearly indicates the usefulness of Dakshanamurthy's (1978) method for estimating ε_{max} of ES-FA mixes.

The ε_{max} and *FSI* of different ES-FA mixes has been plotted as a function of FA content as shown in Fig.6.9. It can be observed that both the characteristics do not depict similar trends. The swell behavior in oedometer is for sample compacted at *OMC* and hence indicates the maximum swell potential of compacted sample. Also, the samples are subjected to a confining seating load of 5 kPa. On the other hand, *FSI* indicates the maximum swelling of soil in the loosest possible state. Therefore, Fig. 6.9 depicts the maximum (loose dry soil mixed with adequate water) and relatively minimum (compacted at OMC) possible limits of swelling for different ES-FA mixes. However, the minimum possible swelling of the compacted mixes is not conclusive from this study due to the fact that the tests were not conducted on compacted states other than *OMC*. As expected, *FSI* shows high values as compared to ε_{max} obtained from oedometer. It may be noted that even with 80% replacement of ES with FA, the *FSI* remains above 100%. For compacted state, the ε_{max} is 1%. Holtz and Gibbs (1956) stated that soils with *FSI* as low as 100% may also undergo considerable expansion in the field.

The ε_{max} of different mixes obtained as per the above procedure is listed in Table 6.2 along with other index properties. Variation of ε_{max} of ES-FA mixes in oedometer has been plotted as a function of *LL, PL, PI* and *SSA* as depicted in Figs. 6.10 to 6.12. It can be noted that ε_{max} exhibits "S" shaped variation with *LL, PI* and *SSA* whereas a linear trend is obtained for PL. The linear trend with PL is given by:

$$\varepsilon_{max} = (2.11)PL - 47.11 \tag{6.3}$$

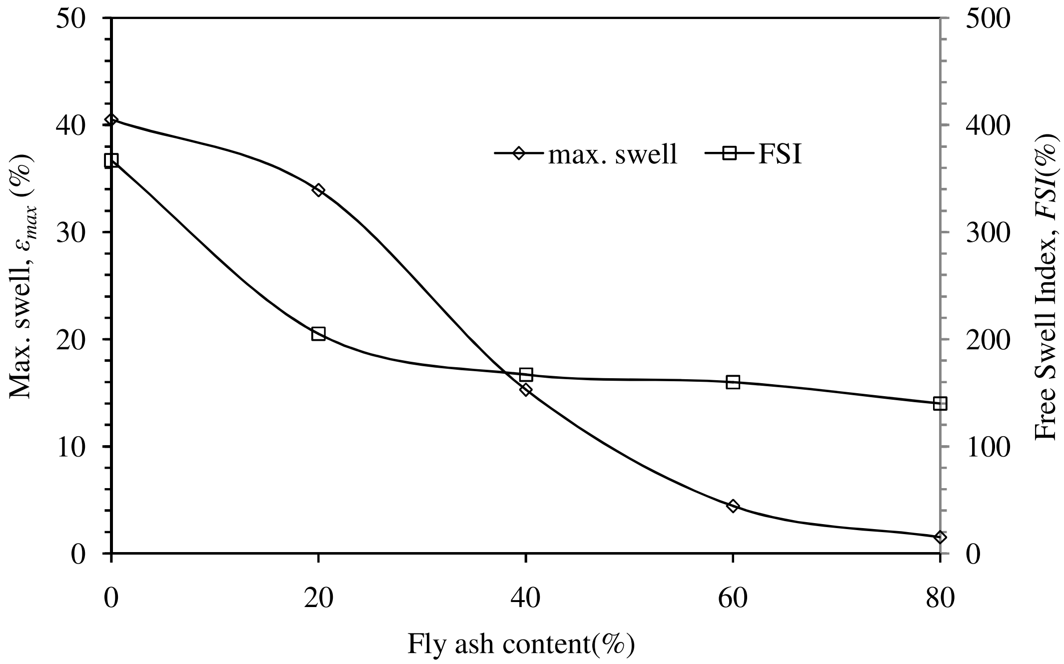


Fig.6.9 Comparison of swelling in oedometer and FSI for ES-FA mixes.

Some of the correlations for ε_{max} reported in the literature are presented as follows:

$\varepsilon = (0.00216)PI^{2.44}$ (Seed et al, 1962) (6.4)

Log $\varepsilon = \{(0.44)LL - w_0 + 5.5\}/12$ (Vijayvergiya and Sullivan, 1973) (6.5)

Log $\varepsilon = (0.9)\ (PI/w_0) - 1.19$ (Shneider and Poor, 1974) (6.6)

where, w_0 = initial moisture content.

It can be noted that the reported correlations are in terms of *PI* and *LL*. Since the present study has obtained a non-linear sigmoidal trend for these parameters, only *PL* has been considered to obtain a simple correlation for $\varepsilon_{max.}$ However, the appropriateness of the linear correlation in terms of PL needs to be verified for other swelling materials.

It is noted from Figs. 6.10 and 6.12 that the variation of ε_{max} is relatively steeper, for the range of parameters $100 < PI < 170$; $110 < LL < 210$ and $30 < SSA < 60$.

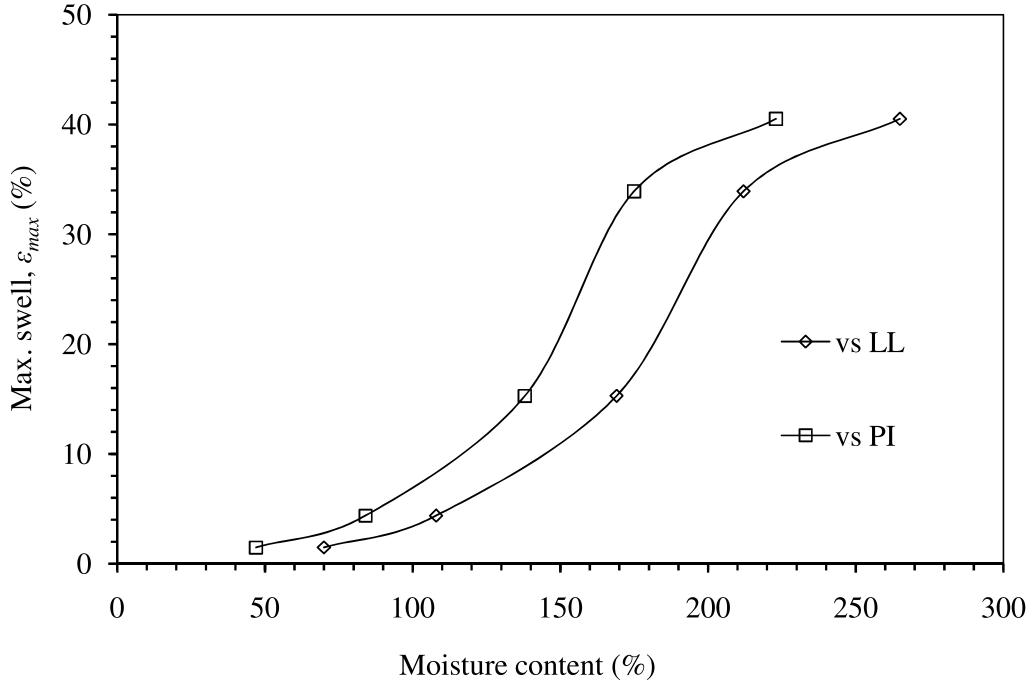


Fig.6.10 Variation of ε_{max} with *LL* and *PI* for ES-FA mixes.

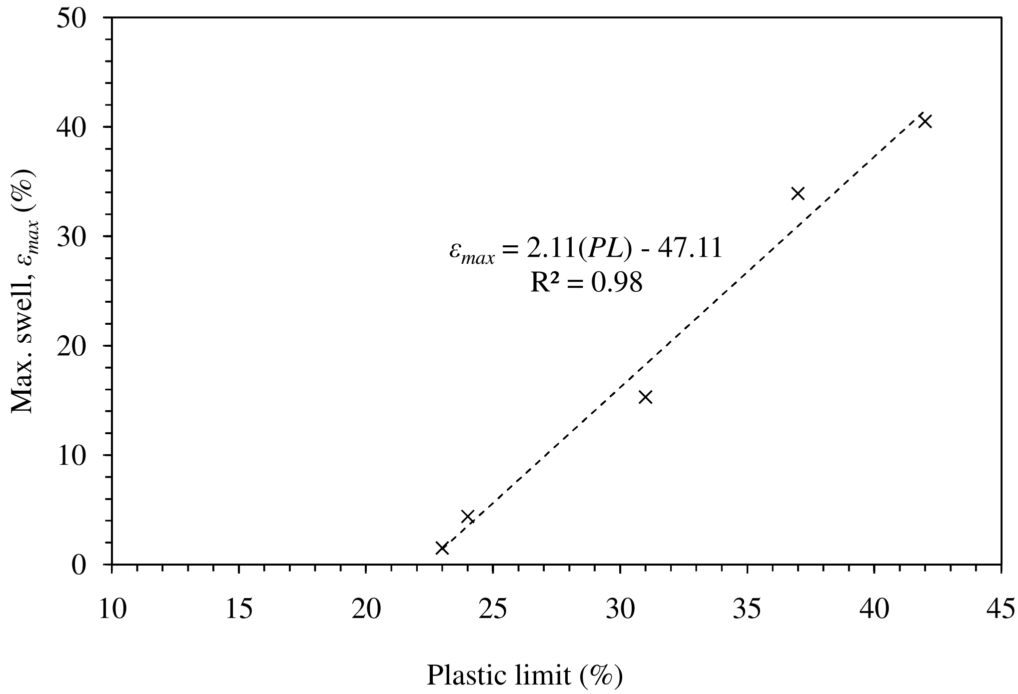


Fig.6.11 Variation of ε_{max} with plastic limit of ES-FA mixes.

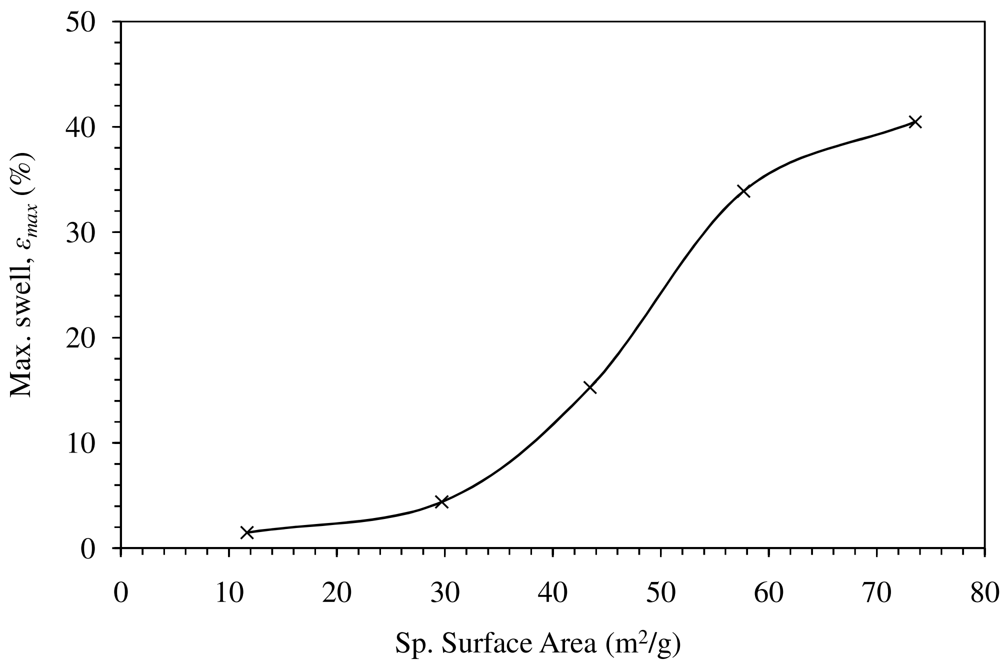


Fig.6.12 Variation of ε_{max} with *SSA* for ES-FA mixes.

However, the trends reported here correspond to only ES-FA mix compacted at *OMC* condition. Further, investigations are required to confirm the trend for other compaction states as well.

6.2.3 Swell Pressure of ES-FA Mixes

The same oedometer set up used for determining ε_{max} has been used to find out the swell pressure of the ES-FA mixes. After the samples reached maximum swell, they were subjected to incremental compressive loads and allowed to stabilize under each increment. The total load required to bring the sample to zero swell is used to determine swell pressure of the mixes. The swell pressure for different ES-FA mixes has been presented in Fig. 6.13 and the same are listed in Table 6.2. It must be noted that for samples F0 and F20, the swell pressure obtained from this study would be lesser than the actual value due to the fact that maximum swell percentage for these samples are higher than the 28 days observation (Fig. 6.8). The 28 days swelling pressure is plotted as a function of FA content as shown in Fig. 6.14. It can be noted from the figure that swell pressure of ES (= 600 kN/m^2) reduces almost linearly to 10 kN/m^2 due to 80% addition of FA. The swell pressure is found to increase linearly with *LL, PL, PI, SSA* and ε_{max} as shown in Figs. 6.15 to 6.17. The linear trends obtained are presented on the figure along with regression co-efficient. These regression equations are valid only for high plastic soils (*LL* > 70; *PI* > 47). The lower limit for validity of regression equations are fixed based on the results of F80. Such a limit is justified due to the fact that most of the swelling soils exhibit high plasticity characteristics, possibly greater than that of F80. However, a conclusive statement can be drawn only after further experimental investigation with wide range of swelling soils.

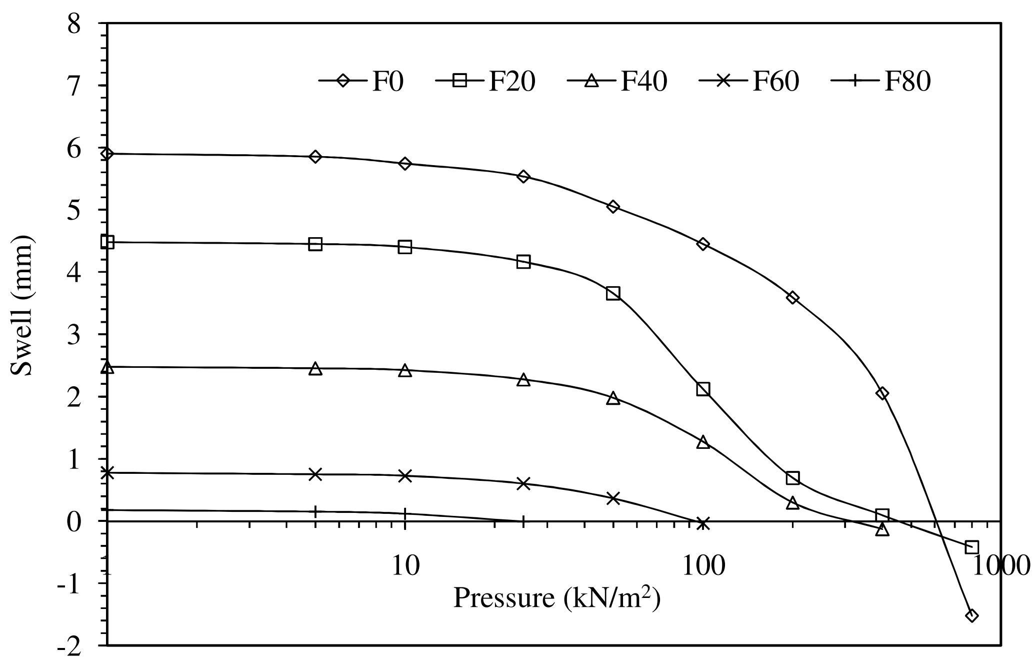


Fig.6.13 Determination of swell pressures of ES-FA mixes from oedometer tests.

Table 6.2 *FSI*, maximum swell, swell pressure and index properties of ES-FA mixes.

Fly ash content (%)	*FSI* (%)	Max. swell (%)	Swell pressure (kN/m^2)	Liquid limit (%)	Plastic limit (%)	Plasticity Index (%)	*SSA* (g/m^2)
0	367	41	600	265	42	223	73.57
20	205	34	450	212	37	175	57.66
40	167	15	300	169	31	138	43.46
60	160	4	100	108	24	84	29.73
80	140	1	25	70	23	47	11.66

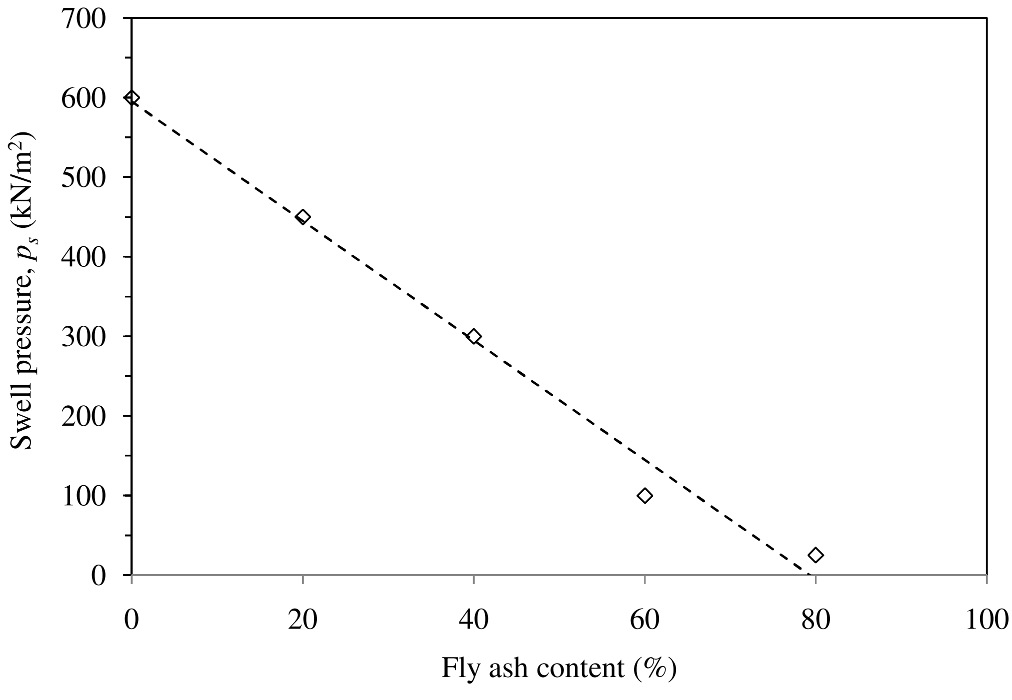


Fig.6.14 Variation of swell pressure with fly ash content.

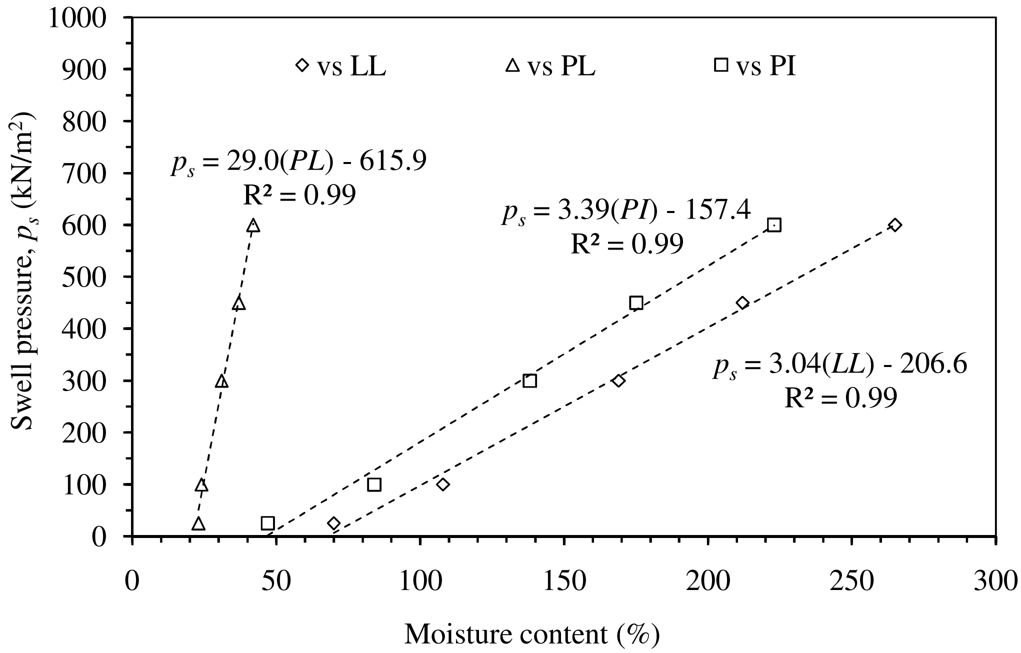


Fig.6.15 Variation of swell pressure with *LL*, *PL* and *PI* of ES-FA mixes.

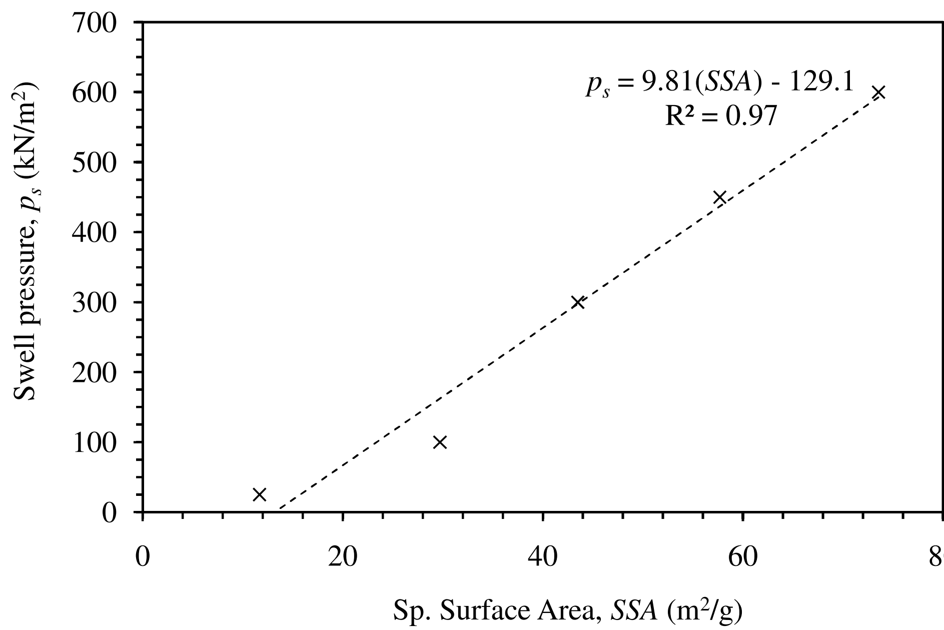


Fig.6.16 Variation of swell pressure with specific surface area of ES-FA mixes.

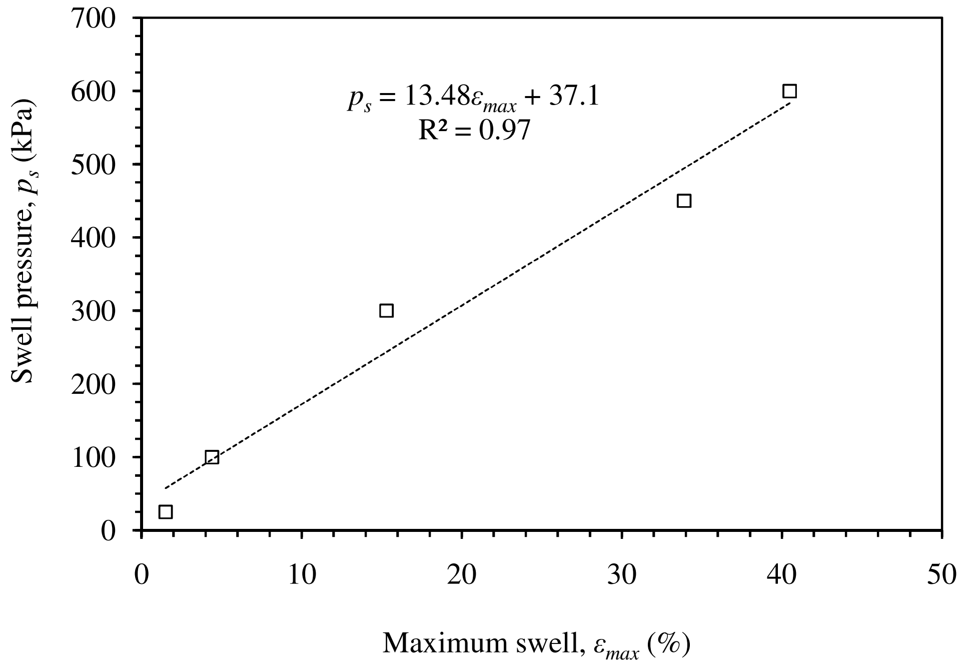


Fig.6.17 Variation of swell pressure with ε_{max} for ES-FA mixes.

The correlations obtained in this study for swelling pressure (p_s) has been summarized as follows:

$$p_s = (3.04)LL - 206.6 \tag{6.7}$$

$$p_s = (29.01)PL - 615.9 \tag{6.8}$$

$$p_s = (3.39)PI - 157.4 \tag{6.9}$$

$$p_s = (9.81)SSA - 129.1 \tag{6.10}$$

$$p_s = (13.48)\varepsilon_{max} + 37.1 \tag{6.11}$$

Komornik and David (1969) have presented a multiple regression equation for determining p_s (Eq. 6.15).

$$\log p_s = -2.132 + (0.0208)LL + (0.000665)\gamma_d - (0.0269)w_0 \tag{6.15}$$

where, w_0 = initial moisture content, γ_d = dry density

It must be noted that the correlation obtained in this study correspond to only samples compacted at *OMC* with Standard Proctor effort. Therefore, the contribution of water content and density does not appear in Eqs. 6.7 to 6.11. Also, the validity of regression equations used for p_s determination is defined above.

6.3 SWELL BEHAVIOUR OF ES-FA MIXES TREATED WITH LIME

Fig. 6.9 and Fig 6.14 reveal that ε_{max} and p_s of ES gradually reduce with addition of FA. If the required reduction in swell percentage is high, then considerable amount of ES need to be replaced by FA. Therefore, for an optimal replacement of ES by FA and also meeting the objective of minimizing swelling, it is proposed to use lime with ES-FA mix. The following section discusses in detail the combined influence of lime and FA on swell characteristics of ES.

6.3.1 Free Swell of ES-FA-Lime Mixes

Fig. 6.18 shows the free swell index of ES-FA mixes treated with different percentage of lime. It is observed that the maximum reduction in free swell is achieved within 3% addition of lime content. Further, *FSI* remains more or less constant up to 13% addition of lime. For more than 13 % lime content, *FSI* is found to increase marginally. This may be due to the fact than the settled soil gets flocculated and has tendency to occupy more space. The effect becomes more prominent as the lime content increases. This is also followed by formation of small lumps of $CaCO_3$ at higher percentage of lime content, which is normally formed under influence of the atmospheric CO_2 (Hafez et al, 2008; Lav and Lav, 2000; Arabi and Wild, 1989; Arman and Munfakh, 1970). The results indicate that *FSI* test can be misleading for soils with higher percentage of lime. Another observation worth noting is that the *FSI* of all the mixes is below 50% within 3%

addition of lime. According to Nelson and Miller (1992), soils with *FSI*<50% are not considered to exhibit appreciable swelling in the field.

The observed results encourage to use very less percentage of lime (<3%) with ES-FA mixes, which enhance the utility of waste FA and at the same time restrict the swelling within permissible limit. To investigate this proposition, swell percentage and swelling pressure of these mixes were determined as described in Sections 6.3.2 and 6.3.3.

6.3.2 Swell Percentage of ES-FA-Lime Mixes in Oedometer

The swell percentage, ε, ES-FA-lime mixes has been determined by oedometer tests as discussed before. It can be noted from Figs. 6.19 and Fig. 6.20 that addition of only 1% lime reduced ε_{max} of ES and ES-FA mixes to less than 4%. Unlike plasticity characteristics and *FSI*, ε_{max} has not increased with higher lime content (> 13%). It needs to be mentioned here that the rectangular hyperbola method proposed by Dakshanamurthy (1979) need not be applied for lime treated soil. Since ε is quite low with the addition of lime, the same can be measured directly. The ε_{max} of ES-FA-lime mix is plotted as a function of *LL* as shown in Fig. 6.21. There is no variation of maximum swell with liquid limit up to *LL* value of about 90%; beyond which, maximum swell appears to increase linearly with *LL*. All the mixes of ES-FA-lime having *LL* > 90% are F0 and F20. The maximum swell can be correlated with *LL* as given by Eq. 6.16.

$$\varepsilon_{max} = (0.05)LL - 3.66 \qquad (6.16)$$

Another important observation is that ε_{max} for all lime-treated soils have reduced to less than 5% even with 1% lime. Therefore, addition of 1% lime to ES-FA mixes would help to maximize the use of FA (depending on the requirement and availability of FA) to reduce the swelling characteristics of ES in the field.

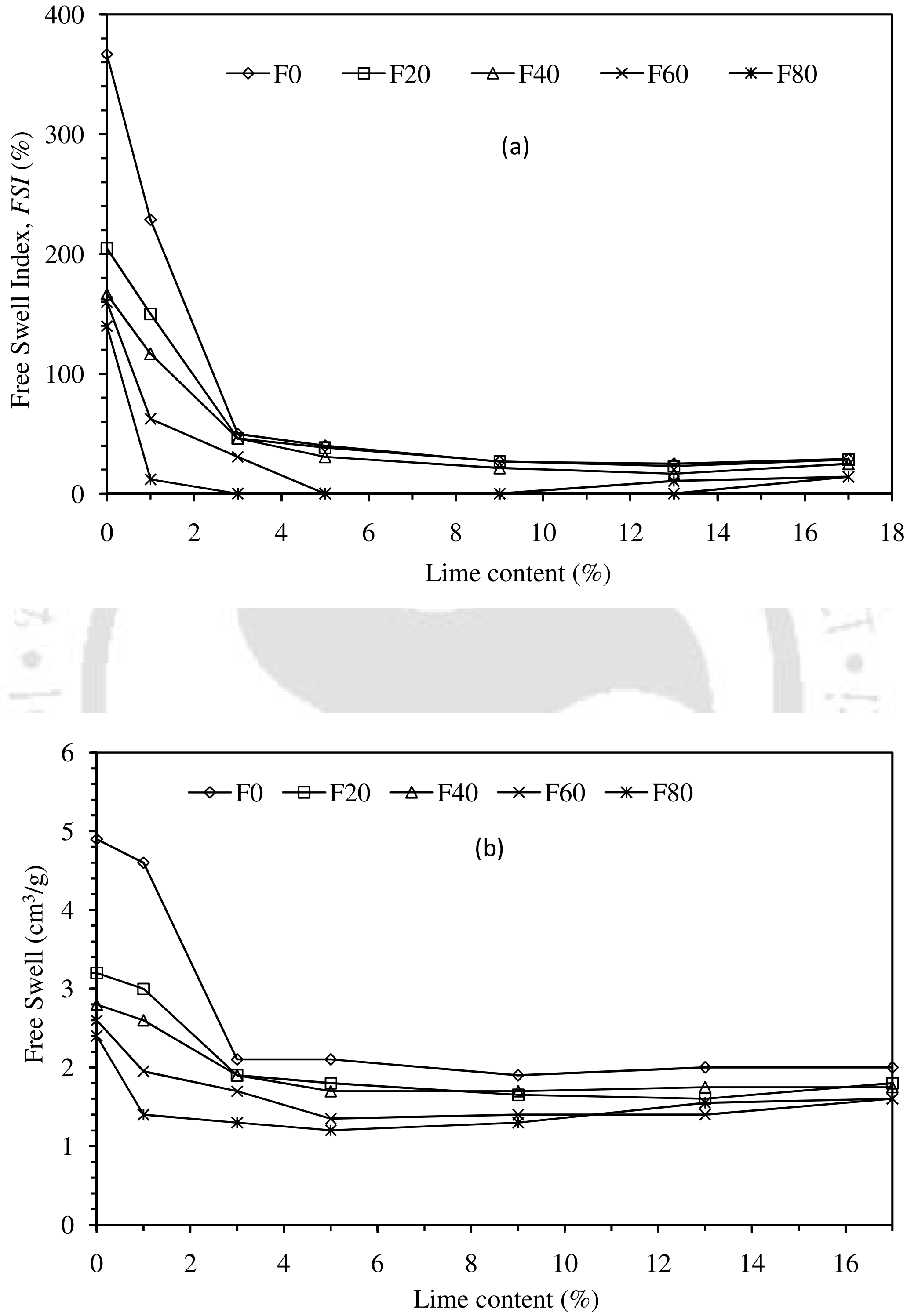


Fig. 6.18 Free-swell of ES-FA treated with lime (a) *FSI* and (b) free swell vol. per gm.

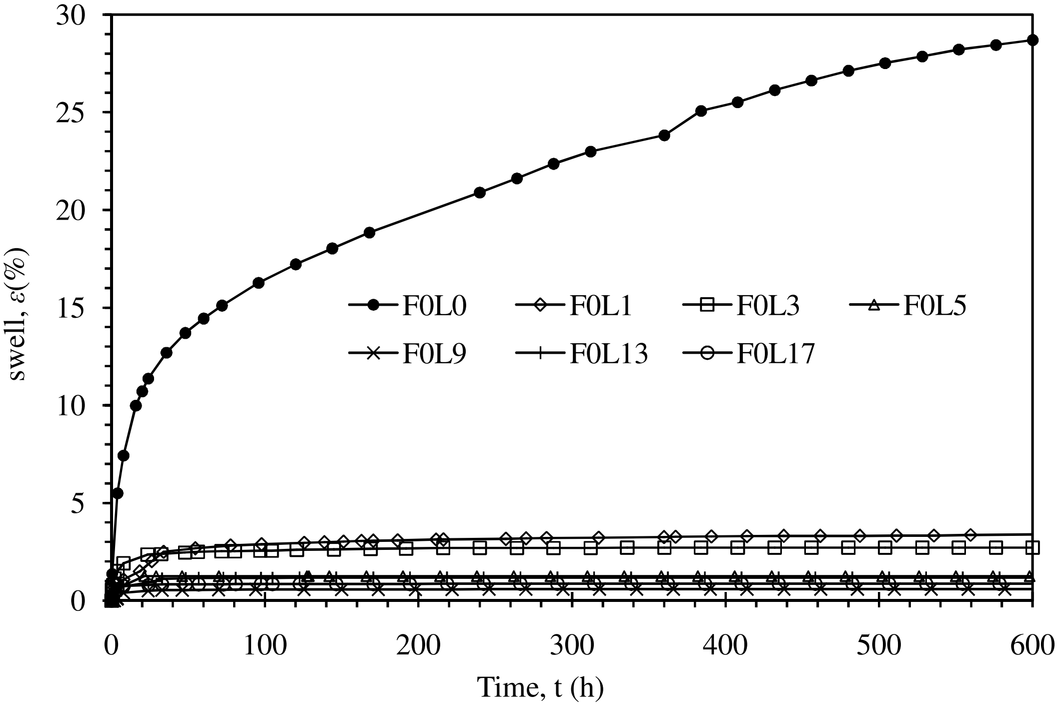


Fig. 6.19 Swell percentage in oedometer for lime-treated ES.

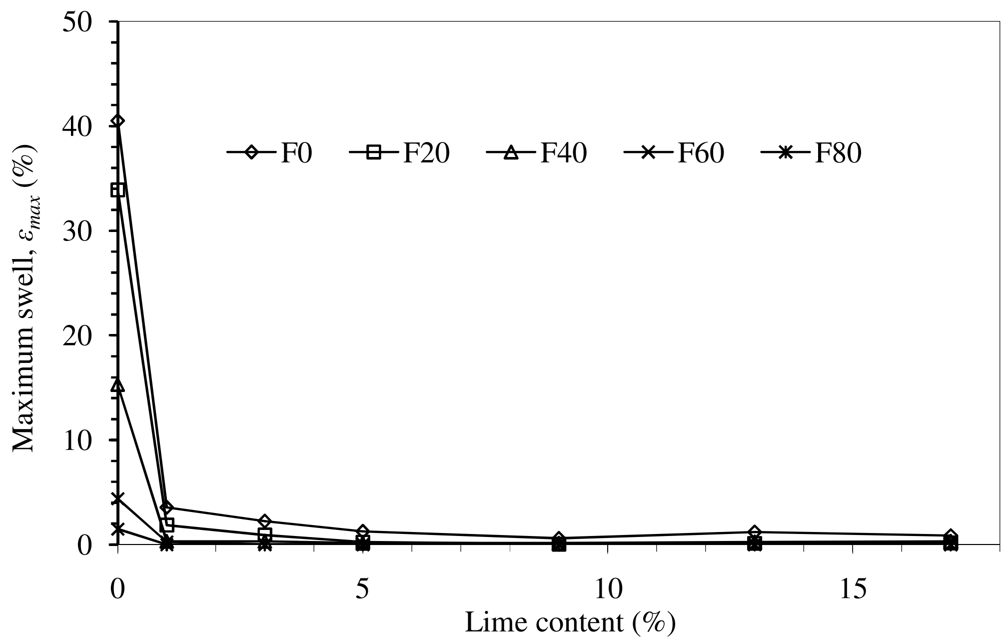


Fig. 6.20 Maximum swell Vs lime content for ES-FA mixes.

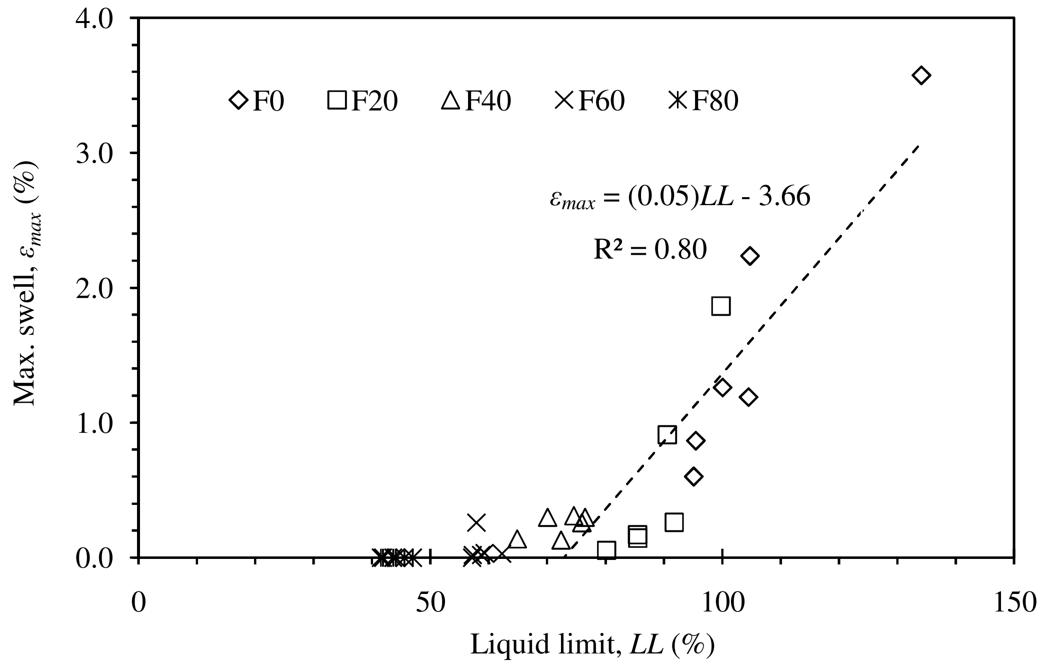
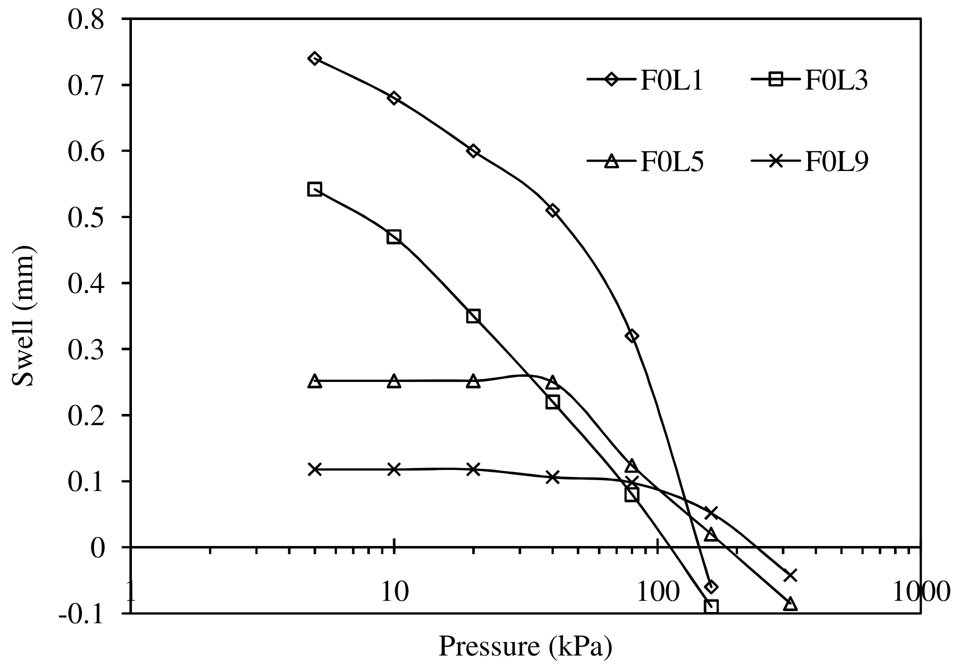


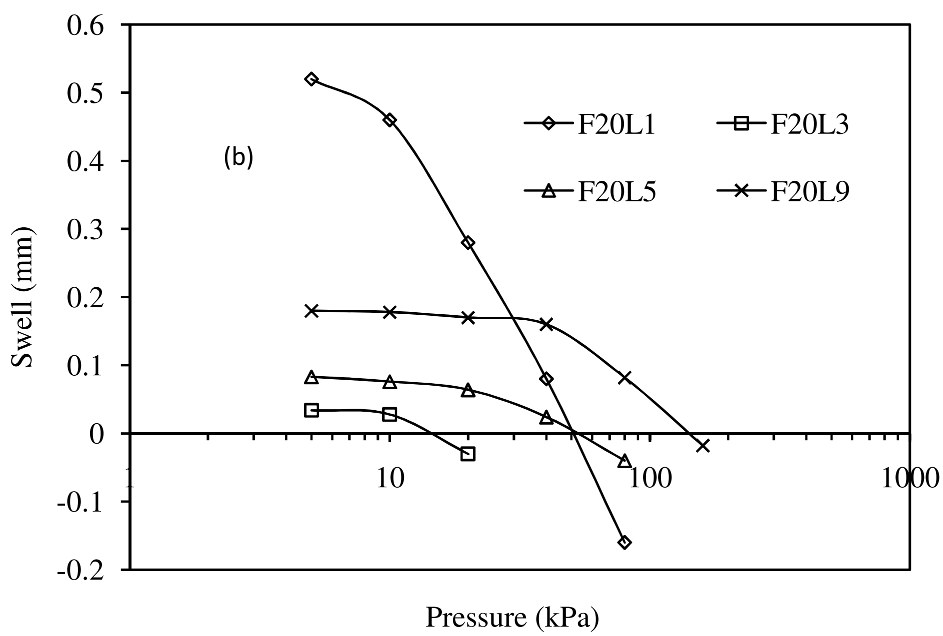
Fig. 6.21 Variation of ε_{max} with *LL* for lime treated ES-FA mixes.

6.3.3 Swell Pressure Measurement of ES-FA-Lime Mixes in Oedometer

In case of ES-FA mixes, swell pressure was measured by gradually reducing swell with successive increments of load, after reaching ε_{max}, in the oedometer test. Fig. 6.22 depicts lime treated ES and F20 specimens loaded back to their original thicknesses. It can be noted that when 1% lime was used with ES, the maximum swell is 0.74 mm and the pressure required to bring it to the original thickness (zero swell) is 140 kPa. With addition of 3% lime, the swell is reduced to 0.54 mm and the pressure required to bring it to 0 is 110 kPa. The maximum swell for ES with 5% lime addition is 0.292 mm and the swell pressure is 190 kPa. It must be noted here that even though the maximum swell for 3% lime is greater than 5% lime addition, the swell pressure of latter is seen to be greater. This is not as per expected because higher swelling is always associated with high swelling pressure. Similarly, swell pressure for sample with 9% lime content is



(a)



(b)

Fig. 6.22 Swell pressure measurement of ES-FA mixed with lime (a) ES and (b) F20.

much higher in spite of its low maximum swell. The same trends have been observed for F20 also.

Unlike untreated soils, lime reactions alter the characteristics of the soil and ultimately result in cementation. There are two processes: one is modification and the other is solidification that can occur in lime treated soil (Boardman et al, 2001). There is no irreversible reaction that takes place for modification where as irreversible cementation reaction occurs for solidification. Low lime content results in modification only whereas high lime content results in both modification and solidification. The reason for high swell pressure in spite of low maximum swell is attributed to the initiation of cementation reaction. Therefore, swell pressure for ES mixed with high lime content is influenced by cementation reaction whereas not for low lime content. For bringing the swelled soil to its original volume, the applied loads have to break the cementation (solidification) forces. This indicates that it would be a gross overestimate of swell pressure of ES mixed with lime, when it is determined by conventional oedometer method (IS 2720, Part 41). In view of this, swell pressure has not been determined for mixes other than F20 and for lime content greater than 9%. It must be noted that the entire duration of swell and swell pressure test is 40 days. The present study does not appraise the reaction occurring in FA and lime treated soil for time period greater than 40 days.

The results presented in this section clearly indicates that bonding or solidification occurs in ES with lime content greater than 3%, which resulted in higher pressure to bring it back to zero swell. The swell pressure can therefore be conveniently used as an indicator to identify the optimal lime content in soil that results in only modification reactions or combined modification-solidification reactions. As long as swell pressure measured using the oedometer decreases with decrease in swell, the soil has only been modified, and no solidification has taken place. However, if the swell pressure is quite high in spite of its low swell, it indicates that solidification of the soil has taken place along with modification. According to this study, 3% lime can be considered as the optimal lime content that decides between modification and modification-solidification reaction that occurs in ES and ES-FA mix. This optimal percentage of lime may vary depending upon its quality and type of swelling soil used. Further investigations are required to appraise the optimal lime content using the proposed methodology for different type of swelling soils.

6.4 COMPARATIVE ANALYSIS

Fig. 6.23 compares the individual effects of fly ash and lime on the swell behavior of ES-FA mixes in oedometer. It can be seen that the swell potential of expansive soil has reduced by the same amounts on application of either 60% fly ash or 1% lime. The reduction in swell potential of expansive soil due to application of fly ash is gradual, whereas the same for application of 1% lime is abrupt. The plot again assert the previous observation that 1% lime would be sufficient for reducing swelling of ES-FA mix, thereby ensuring maximum usage of FA based on its availability.

6.5 SUMMARY

In this chapter, effort has been made to explore the possibility of using waste fly ash (FA) for reducing swelling characteristics of an expansive soil (ES). 20 to 80 % of ES has been replaced by FA to understand the variation in swelling property. Even though the swelling of ES has been reduced due to the addition of FA, the rate of reduction is

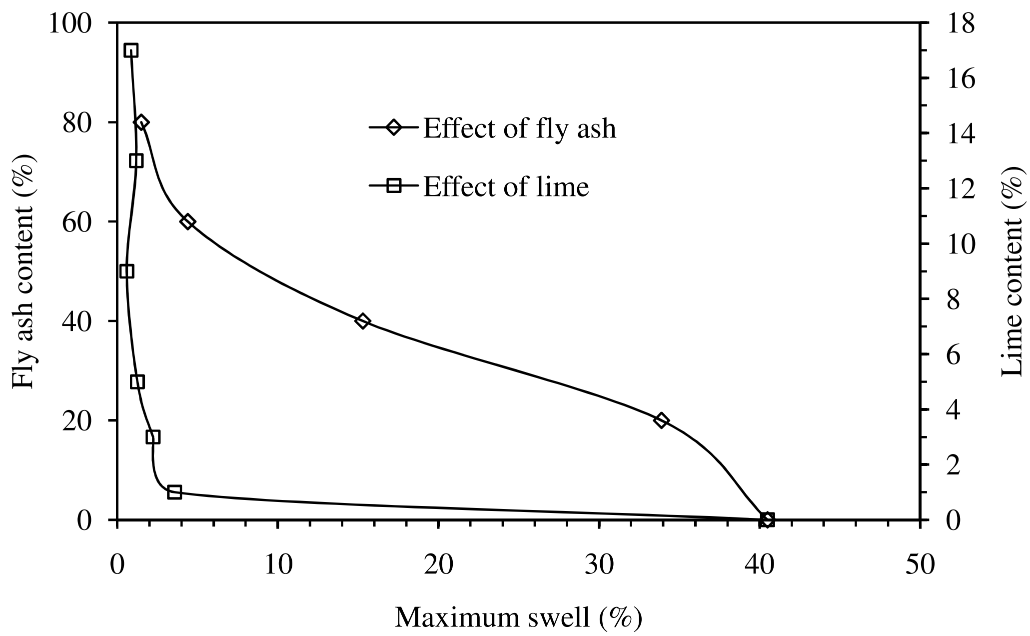


Fig. 6.23 Comparison of effects of fly ash and lime on maximum swell in oedometer for expansive soils.

very gentle. Therefore, different percentage of lime has been added to ES-FA mix and swelling characteristics evaluated. There are not many studies that deal with the swelling characteristics of ES-FA mixes treated with lime. The main purpose of this study is to maximize the use of FA to reduce swelling of ES with minimal use of lime. It is observed that 1% lime would be sufficient for reducing swelling of ES-FA mix, thereby ensuring maximum usage of FA based on its availability.

The Dakshanamurthy (1978) method for estimating maximum swell has been verified and found to be suitable for ES and ES-FA mixes. The method is not useful for lime treated mixes due to the fact that the swell is quite less. The maximum swell percentage is found to exhibit a linear relationship with plastic limit. The swell pressure of ES and mixes is found to vary linearly with liquid limit, plastic limit, plasticity index, and specific surface area.

The present study also demonstrates the use of swell pressure as an excellent indicator for modification or modification-solidification reaction taking place in lime added ES or ES-FA mix. The optimal lime content required to initiate solidification reaction has been identified for the ES and ES-FA mix used in this study. The study also indicates that swell pressure of lime added soils determined using oedometer method can be misleading if solidification reaction occurs.

CHAPTER 7

COMPRESSIBILITY BEHAVIOUR

7.1 INTRODUCTION

Compressibility of soil is determined by conducting one-dimensional consolidation test. Consolidation is a time-dependent process wherein the voids present in the soil decreases under sustained loading by expulsion of water. There are two important aspects of this process, which is of interest to geotechnical engineers: (i) total settlement of soil mass under loading and (ii) rate of settlement. Total settlement potential of a soil is mostly defined by determining compression index (c_c) and/or co-efficient of volume compressibility (m_v). Rate of settlement is determined using co-efficient of consolidation (c_v). In this study, only total settlement potential of remolded expansive soil (ES) due to the application of fly ash (FA) and lime has been investigated. Rate of settlement is mostly important for undisturbed soil and hence has not been considered in this study.

7.2 CONSOLIDATION BEHAVIOUR OF ES MIXED WITH FLY ASH

Typical pressure-displacement response of the expansive soil, as obtained from consolidation test, is shown in Fig. 7.1. The corresponding variation of void ratio of the soil with the applied pressure, is presented in Fig. 7.2. For the sake of brevity, the responses of other specimens (i.e. expansive soil-fly ash mix) are not presented here. Unlike the normal practice of determining c_c over an average range of pressure, the values herein have been obtained corresponding to different ranges of pressure, based on Fig. 7.2 for ES. Similarly, c_c has been obtained for various ES-FA mixes for different pressure increments and the results are plotted as depicted in Fig. 7.3. It can be noted that

c_c increases with increase in pressure upto 640 kPa, beyond which it remains relatively constant. The increase in c_c for pressure less than 640 kPa is substantially high, for higher percentage of ES. Such a steep response indicates higher settlement susceptibility of the material when there is change in pressure. It also indicates that FA modified ES has lower settlement susceptibility when the pressure on the soil increases. This can further be analysed through the variation of c_c with FA content, as shown in Fig.7.4. In this figure, c_c corresponding to pressure range of 320-640 kPa has been considered. The c_c is found to decrease from 0.6 to 0.13 when the FA content increases from 0 to 100 %. There is a reduction of 78 % in c_c of the ES, with addition by about 80%. Such a considerable amount of reduction in the compressibility of ES indicates the potential of using FA for such application. Therefore, results presented in this study can be used to obtain desired percentage of FA, based on the required reduction in c_c. Further investigations are required to understand such reduction for different other sources of FA.

In Fig. 7.4, c_c values estimated based on liquid limit relation (Terzaghi and Peck, 1967) are also presented. It can be noted that the c_c values obtained from the consolidation tests are much less than the estimated. This is primarily due to the fact that the initial state in case of the estimated c_c, corresponds to liquid limit of the soil; whereas, the initial state in case of the consolidation tests, corresponds to the optimum compaction condition (i.e. *OMC* and *MDD*). The initial void ratio and water content would be quite high for liquid limit state of the soil as compared to the compacted state and hence the settlement potential of the former is high. It can also be noted that the difference between the experimentally obtained and the estimated c_c decreases with increase in FA content. For fly ash alone (FA), c_c could not be estimated due to the fact that it is a non-plastic material and hence *LL* is not valid.

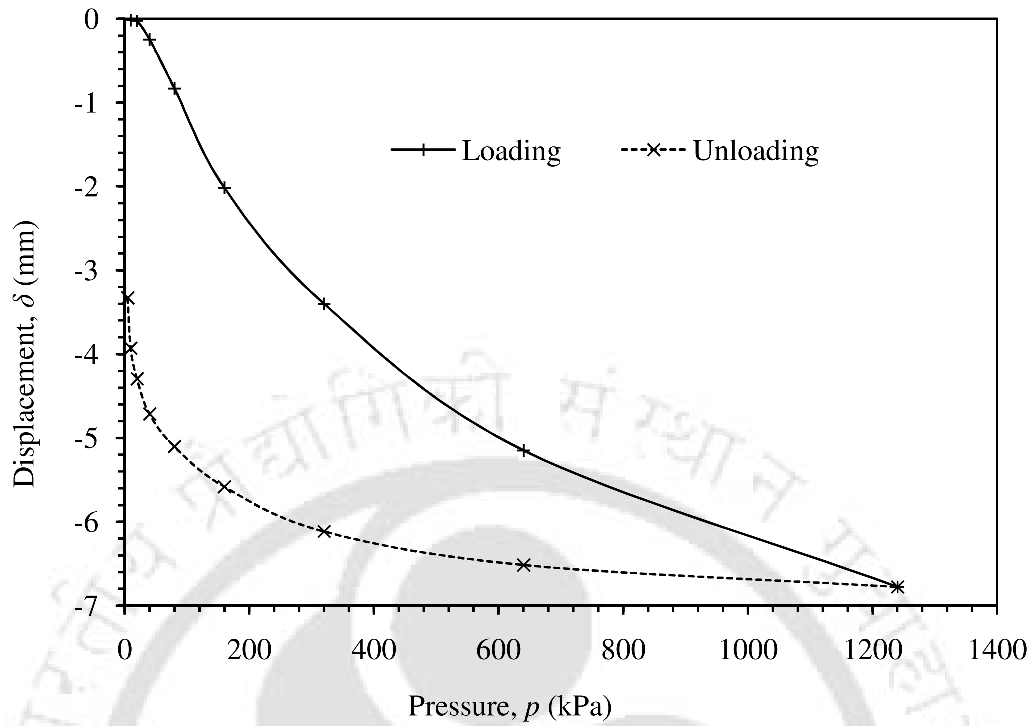


Fig.7.1 Settlement versus pressure response for ES.

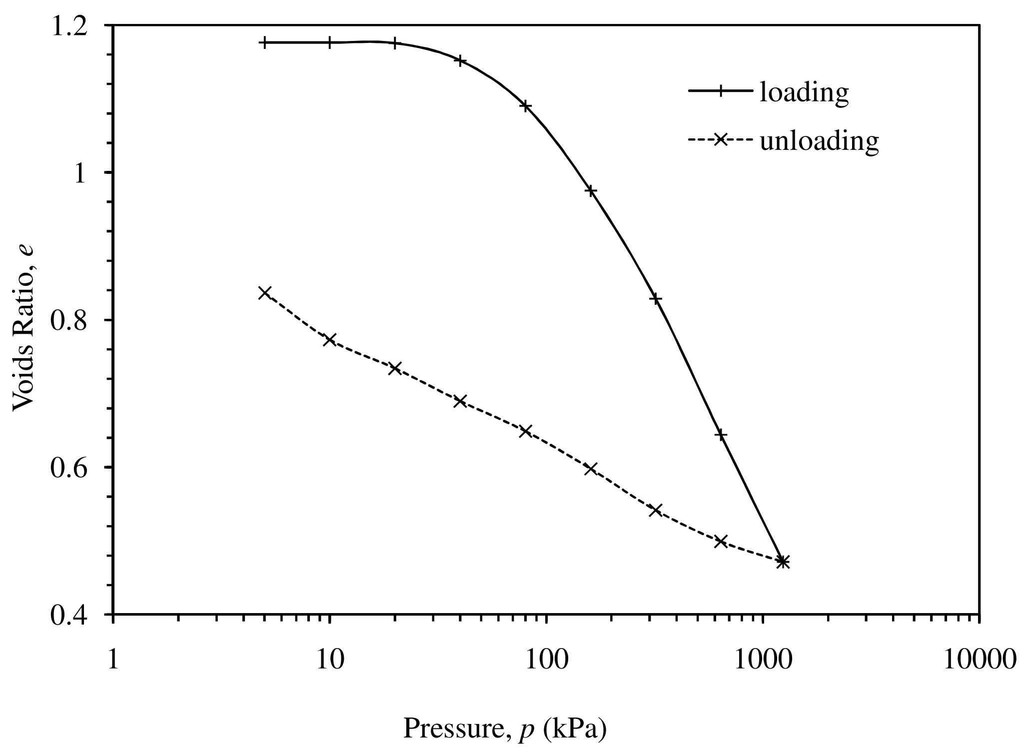


Fig.7.2 Void ratio versus pressure response for ES.

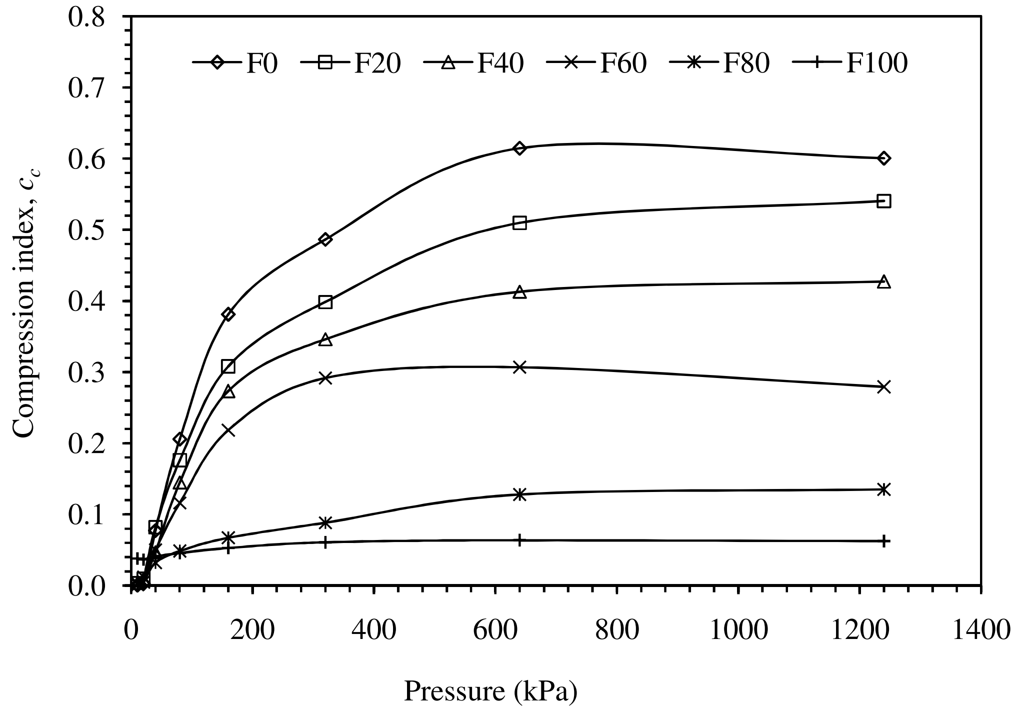


Fig.7.3 Compression index corresponding to different pressure ranges for ES-FA mixes.

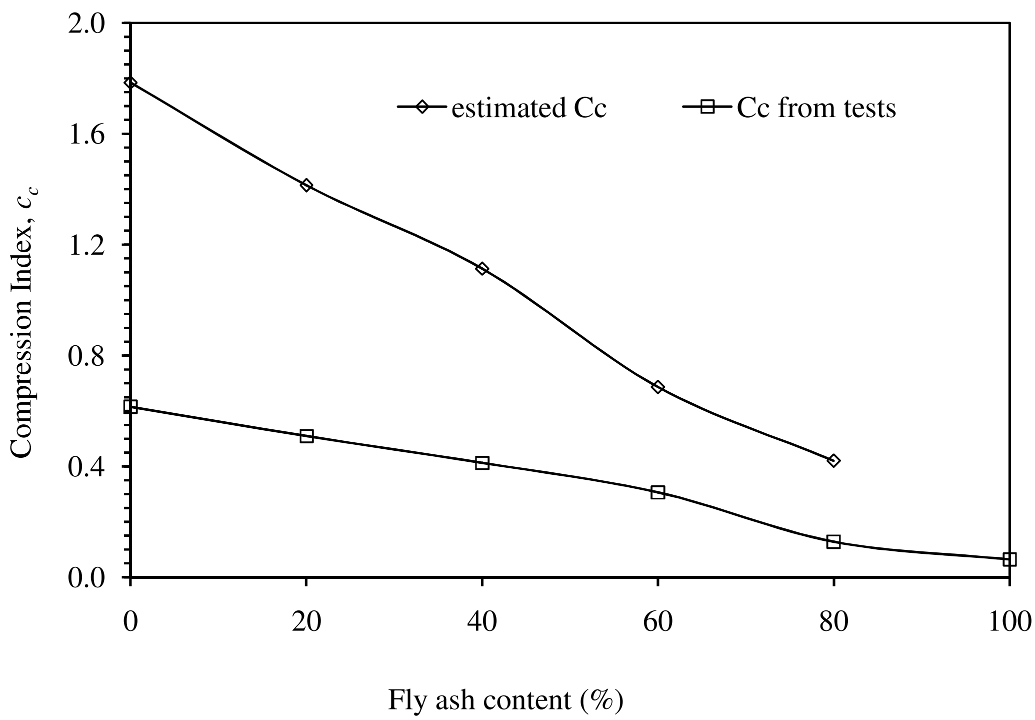


Fig.7.4 Variation of compression index with FA content.

Fig. 7.5 shows the variation of coefficient of volume compressibility (m_v) corresponding to different pressure. It can be noted that, for all the mixes m_v is maximum in the pressure range of 40-80 kPa and decreases thereafter. This indicates that the maximum sensitivity of ES-FA mixes to the pressure, in terms of compressibility, occurs in the range 40-80 kPa. The decrease in m_v with pressure beyond 80 kPa can be attributed to the increased densification of the sample as pressure increases. It can be noted that, for high FA content mixes m_v approaches a constant value with increase in pressure. However, for low FA content mix, m_v continues to reduce for the whole range of pressure considered in this study.

The peak m_v value, which is exhibited in the range of 40 to 80 kPa has been plotted as a function of FA content, as depicted in Fig. 7.6. It can be noted that there is visible reduction in m_v of ES when FA content increases beyond 20 %. This data can be utilized to obtain the required quantity of FA to be added to the ES, in order to achieve a desired value of m_v by addition of suitable quantity FA to ES. Additionally, it can also be of use

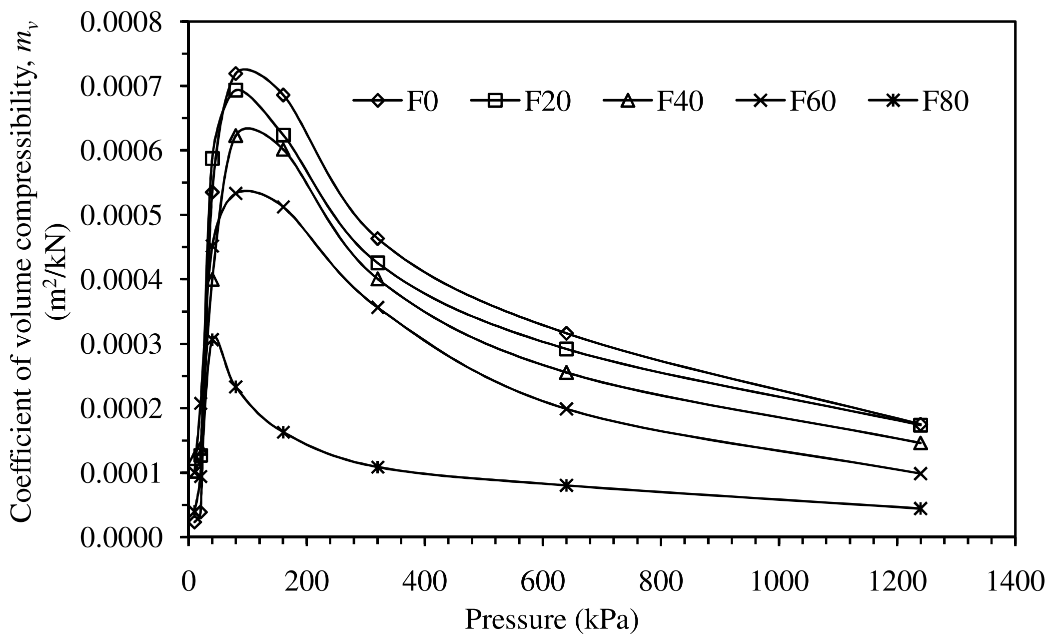


Fig. 7.5 Variation of m_v with pressure for different ES-FA mixes.

in maximizing the FA utility. There is 68% reduction in m_v of ES due to 80% addition of FA. This indicates increased resistance of ES to compression due to the addition of FA. The percentage reduction in m_v of ES due to FA addition corresponding to different ranges of pressure has been shown in Fig. 7.7. It can be noted m_v of ES reduces drastically upto 80% of fly ash addition. Also, there is no significant variation in percentage reduction of m_v for different pressure ranges, investigated in this study. The percentage reduction in m_v of ES due to FA addition corresponding to different ranges of pressure has been shown in Fig. 7.7. It can be noted that m_v of ES reduces drastically up to 80 % of FA addition. Also, there is not significant variation in percentage reduction of m_v for different pressure ranges, investigated in this study.

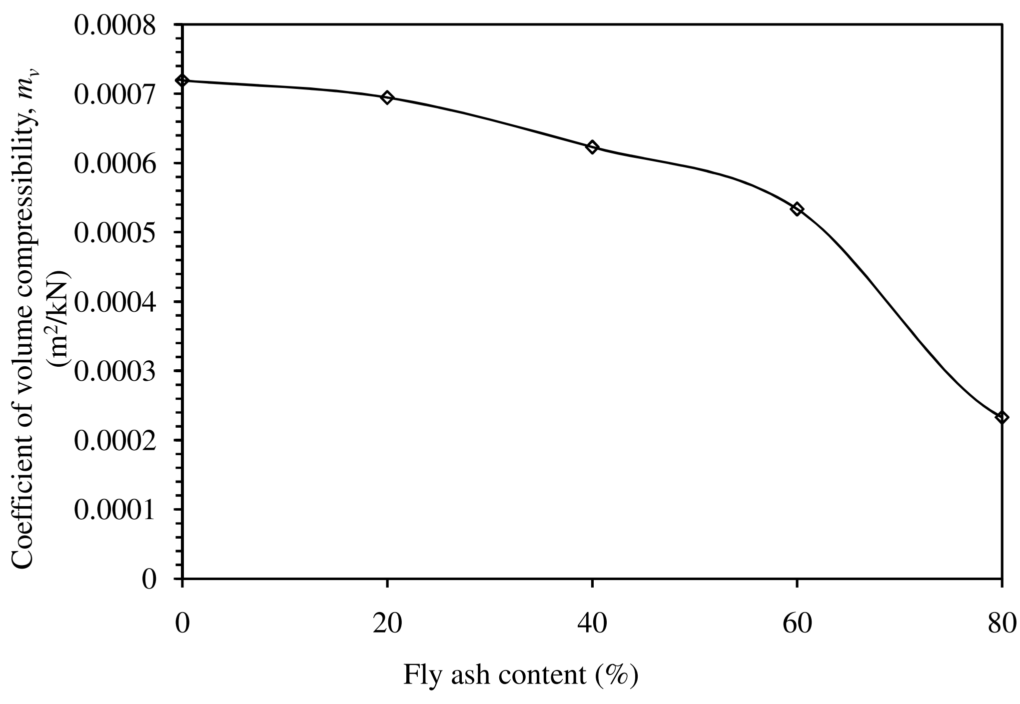


Fig.7.6 Variation of peak m_v of ES-FA mixes with FA content.

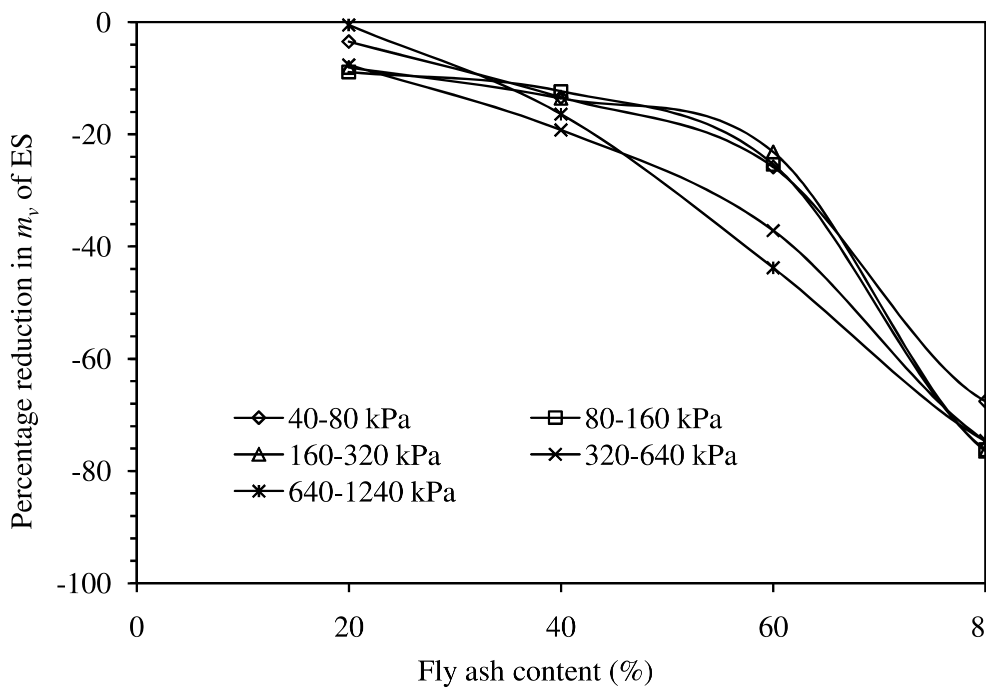


Fig.7.7 Percentage difference in m_v of ES due to FA addition for different ranges of pressure.

7.3 CONSOLIDATION BEHAVIOUR OF ES-FA MIXES TREATED WITH LIME

Fig. 7.8 depicts the variation of void ratio (e) with the pressure applied, for the ES treated with different percentage of lime (i.e. 1% to 17%). It can be noted that addition of lime makes the soil (ES) more stiff and resistant to compression. The variation of c_c and m_v of ES with addition of lime for different pressure ranges are depicted in Figs. 7.9 and 7.10, respectively. It could be observed that the c_c of ES is considerably reduced by the addition of lime. However, c_c is found to increase with pressure, for all the lime contents investigated in this study. Another important observation is that the c_c, in the higher pressure range (> 640 kPa), is relatively more for 17 % lime content as compared to that with 13%. Such a reversal of trend has been observed for liquid limit of the soil composition mentioned above. This indicates that high percentage of lime content can induce undesirable soil characteristics. This may be attributed to the high alkalinity of the soil combined with aging effect of the sample corresponding to 1240 kPa (this addition of loading is done after 7 days). That the silica present in the soil has adequately dissolved to produce substantial amount of CSH gel, leading to increased compressibility of the material. However, further investigations are required to explain fully the reversal of compressibility trends at higher pressure range and high soil alkalinity. The variation of m_v with pressure also follows the same trend as that of c_c (Fig. 7.10).

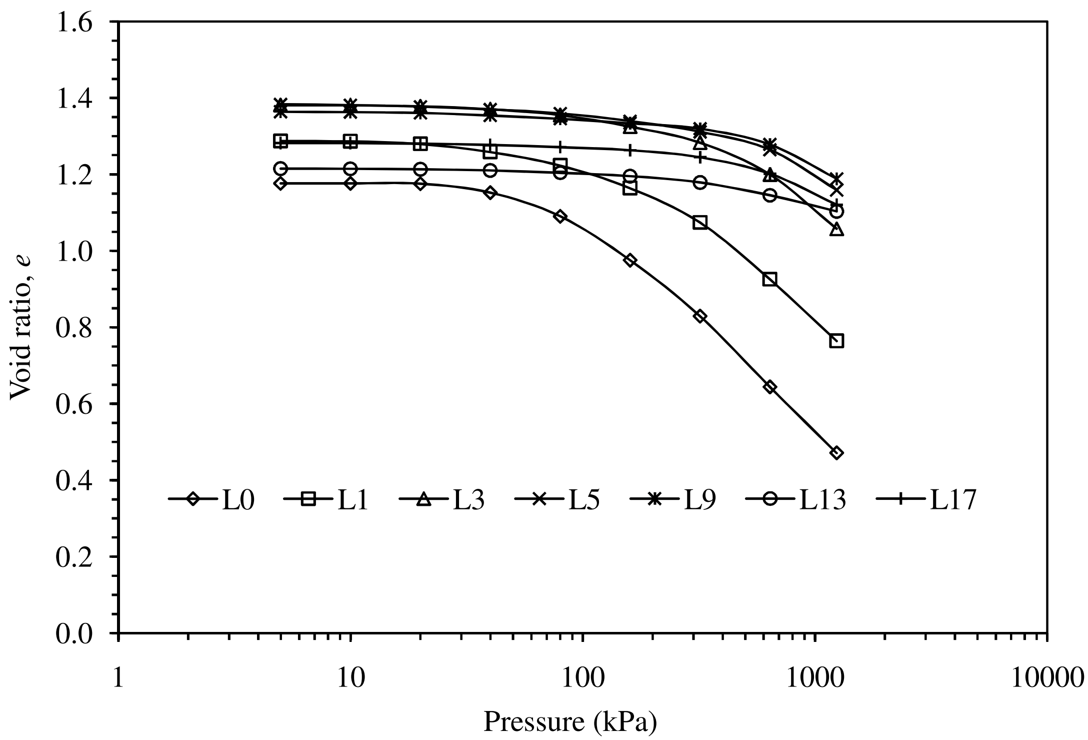


Fig.7.8 Variation of void ratio with pressure for ES treated with different percentage of lime content.

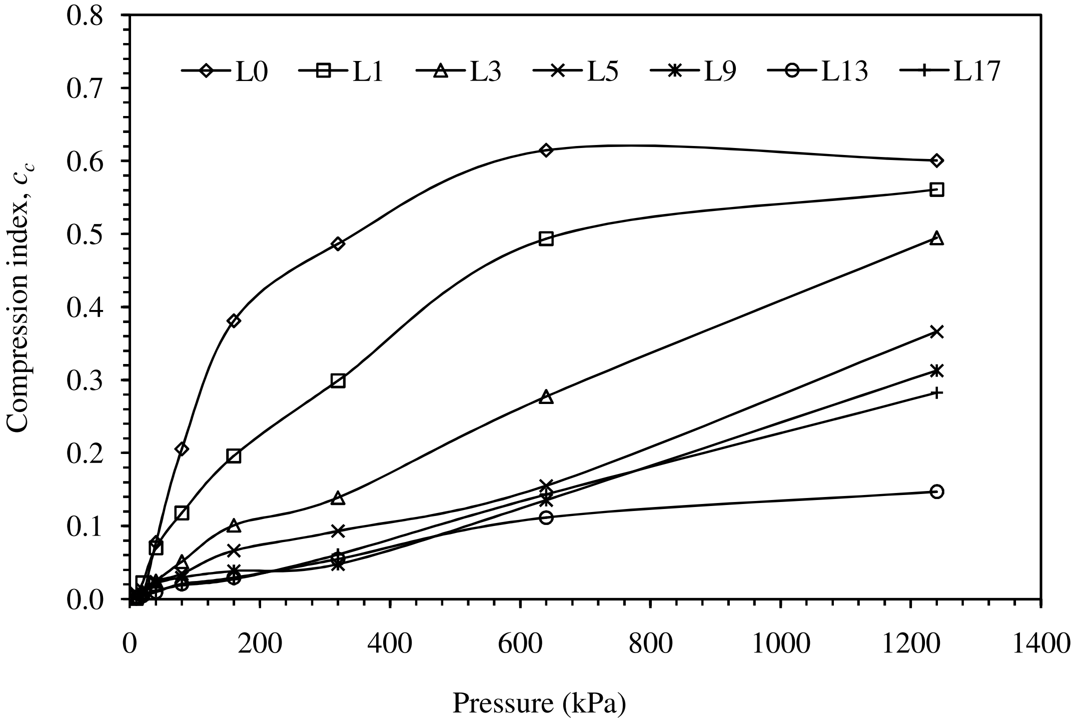


Fig. 7.9 Compression index of lime treated ES.

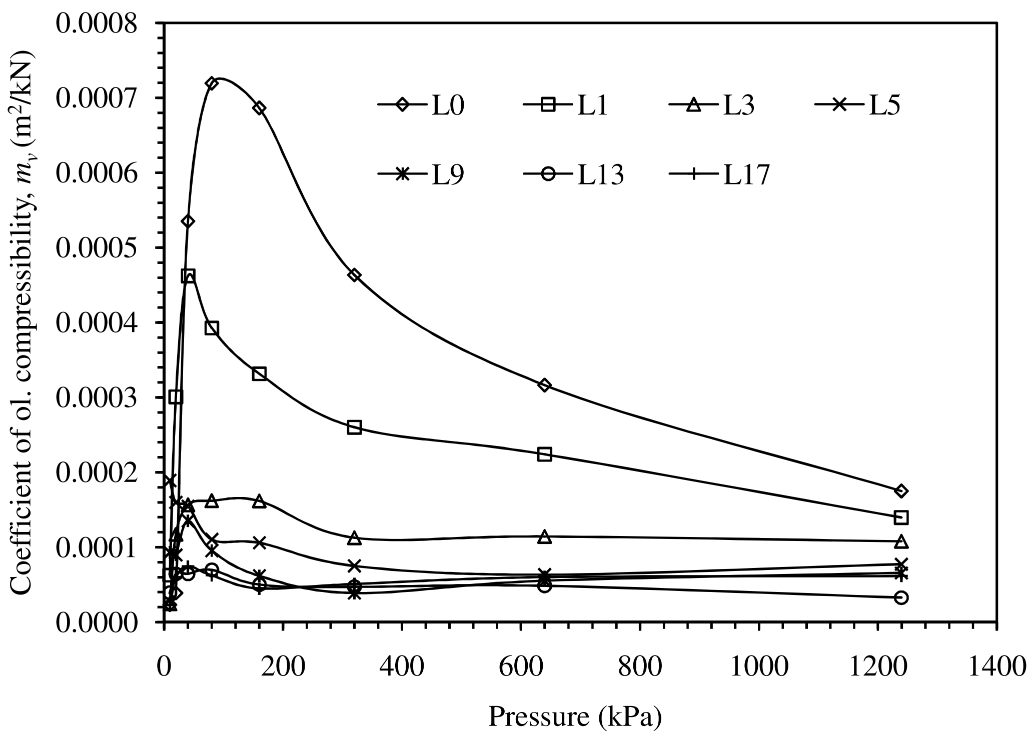


Fig. 7.10 Coefficient of volume compressibility of lime-treated ES.

Figs. 7.11 to 7.18 depict the variation of c_c and m_v with the applied pressure for FA and lime amended ES. In general, it can be noted that the simultaneous addition of lime and FA further reduces the compressibility of ES as compared to the FA amended ES. From Fig. 7.11, depicting response of ES with 20% FA, it can be seen that there is an increase in c_c when lime percentage is greater than 13%, specifically in the pressure range of 1240 kPa. However, as noted from Fig. 7.12 such a trend is not explicit for m_v. Studies reported in the literature indicate the possibility of zeolitization of soils under alkaline conditions (Sivapullaiah and Reddy, 2009) resulting in the collapse of the porous media. It needs to be investigated further, whether such reactions are responsible for reversal of trends for c_c when percentage of lime increases. Such reactions may also be time dependent and hence more visible at the end stage of consolidation loading (1240 kPa). Further detailed geochemical experiments are required to corroborate the hypothesis. As discussed in the previous section, the influence of lime addition on m_v is more visible in the lower range of 40 to 80 kPa. Beyond, 3% there is not much influence of lime addition on m_v. The trends observed for F20 is true for other FA amended ES also. However, the major observation from Fig. 7.13 is that for F40 there is an increase in c_c for 17% lime addition for pressure of 640 kPa as well. As shown in Fig. 7.15, for F60 the increase in c_c occurs as early as 320 kPa and the value is higher than that obtained for 3% lime addition. For F80, the c_c value is much higher for 17% lime addition as depicted in Fig. 7.17. An increase in m_v is also visible for F80 as shown in Fig. 7.18. These results indicates that higher percentage of lime addition (>13%) induces undesirable reactions in FA amended ES resulting in the increase of compressibility. The effect of high lime percentage on c_c of FA amended ES increases with an increase in FA content. Such an observation strengthens the hypothesis of zeolitization occurring with time in high alkaline state of the FA amended ES.

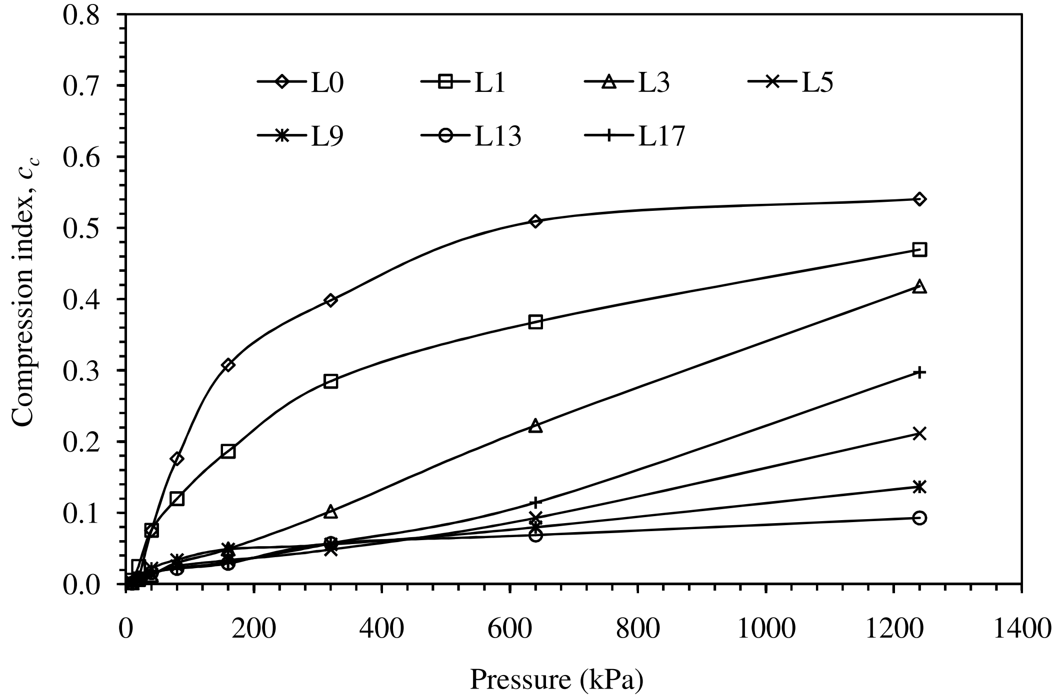


Fig. 7.11 Compression index of lime treated F20.

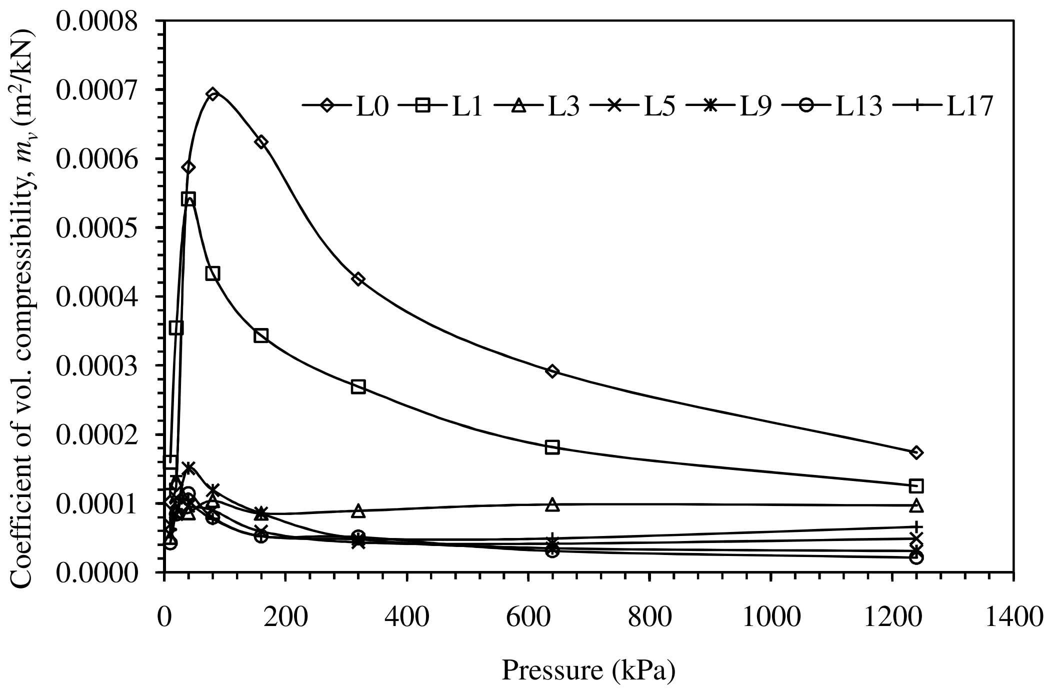


Fig. 7.12 Coefficient of volume compressibility of lime treated F20.

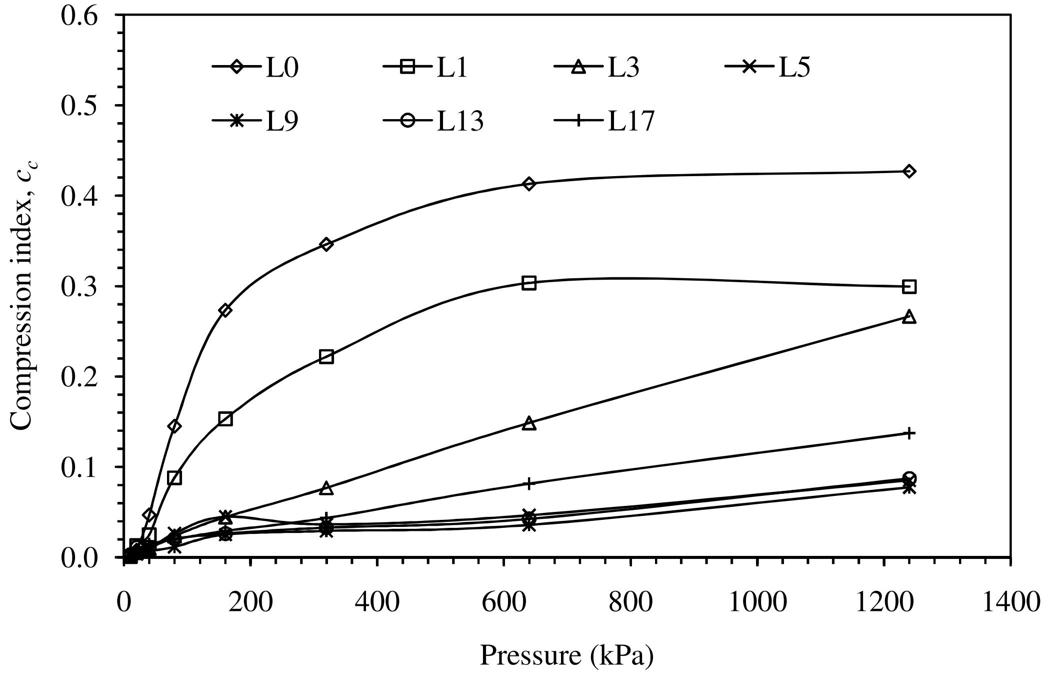


Fig. 7.13 Compression index of lime treated F40.

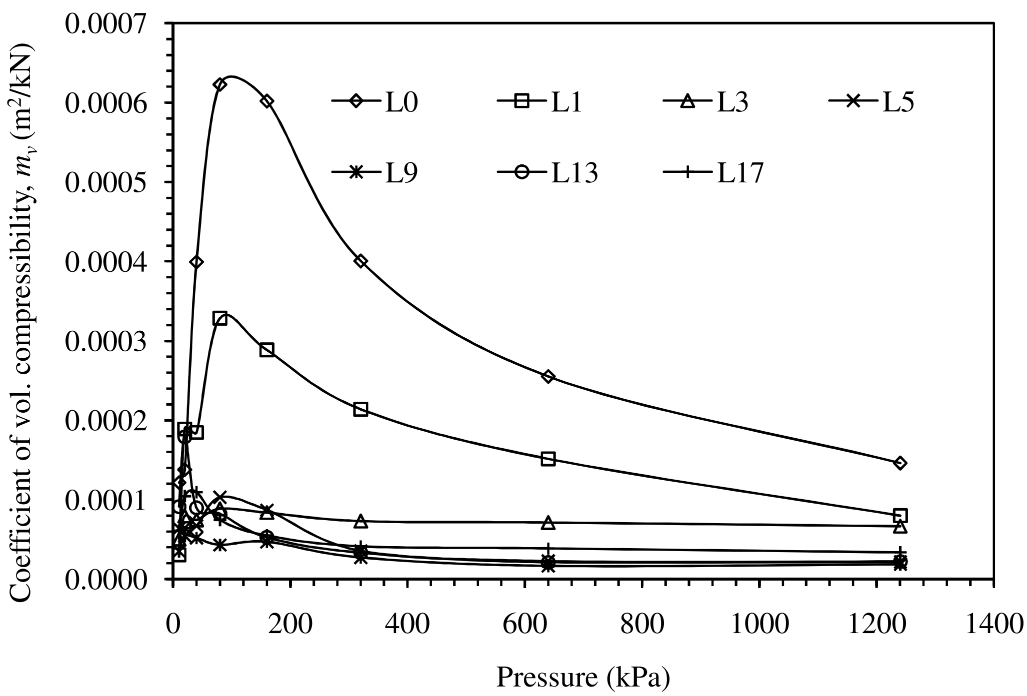


Fig. 7.14 Coefficient of volume compressibility of lime treated F40.

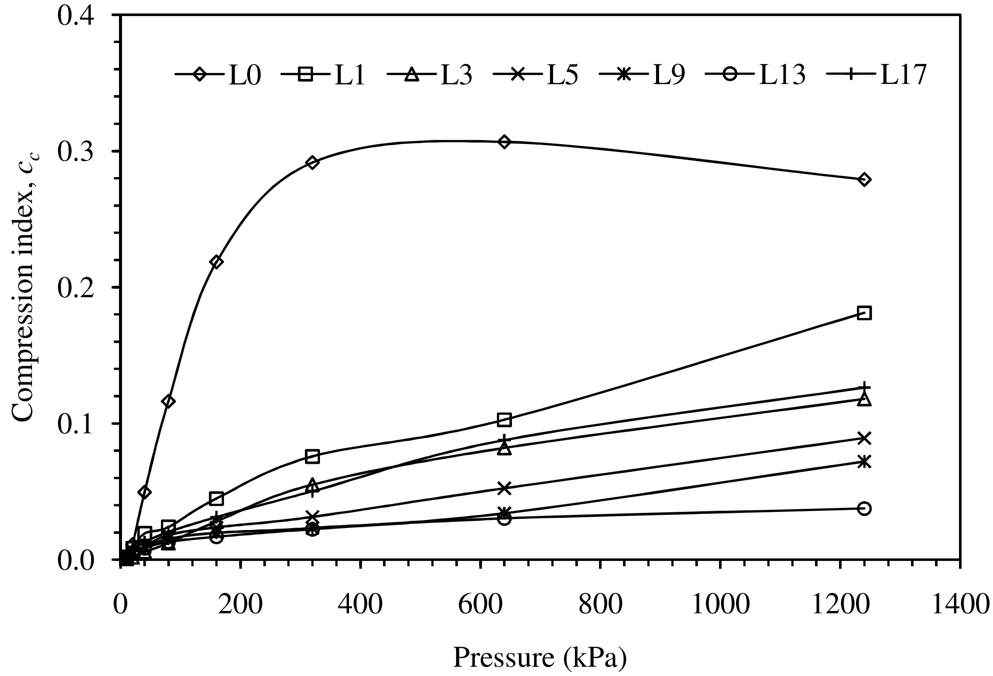


Fig. 7.15 Compression index of lime treated F60.

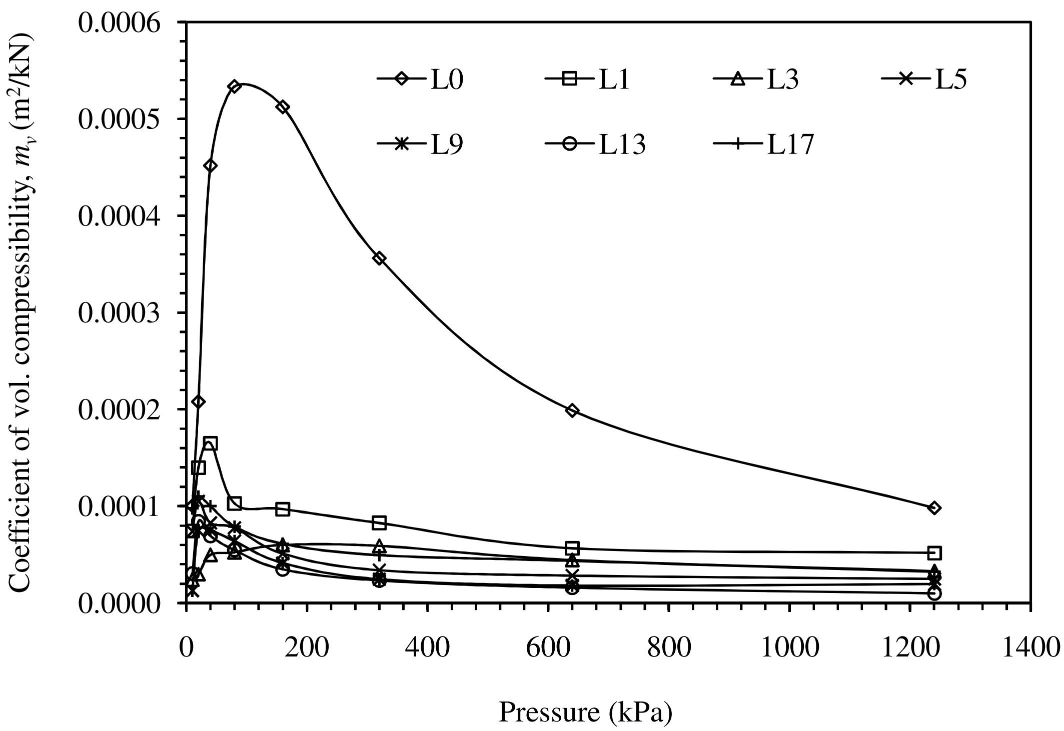


Fig. 7.16 Coefficient of volume compressibility of lime treated F60.

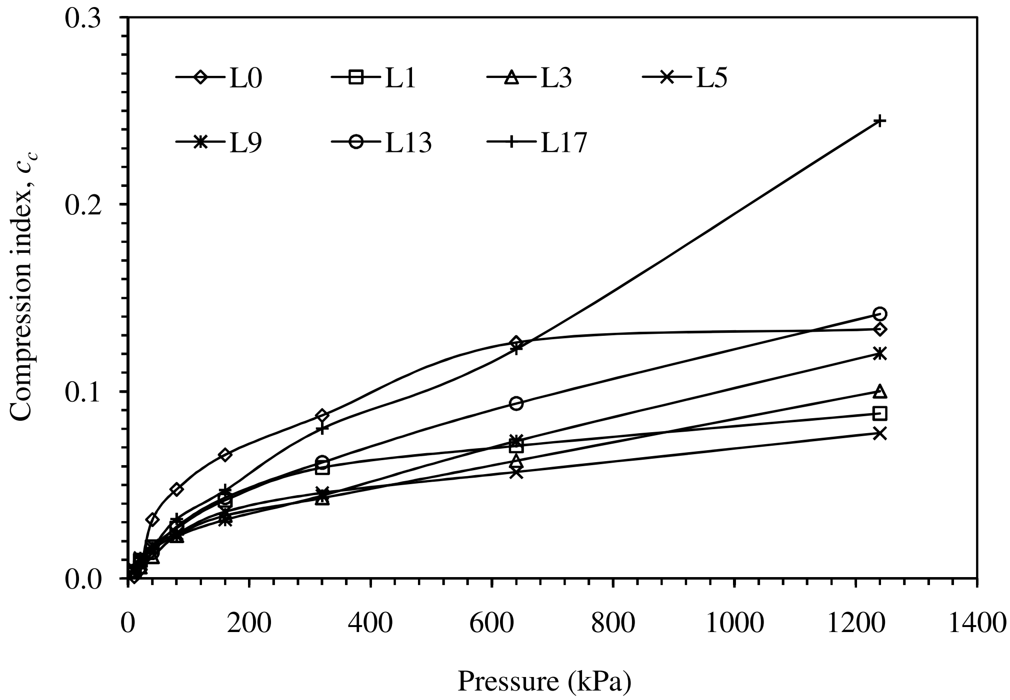


Fig. 7.17 Compression index of lime treated F80.

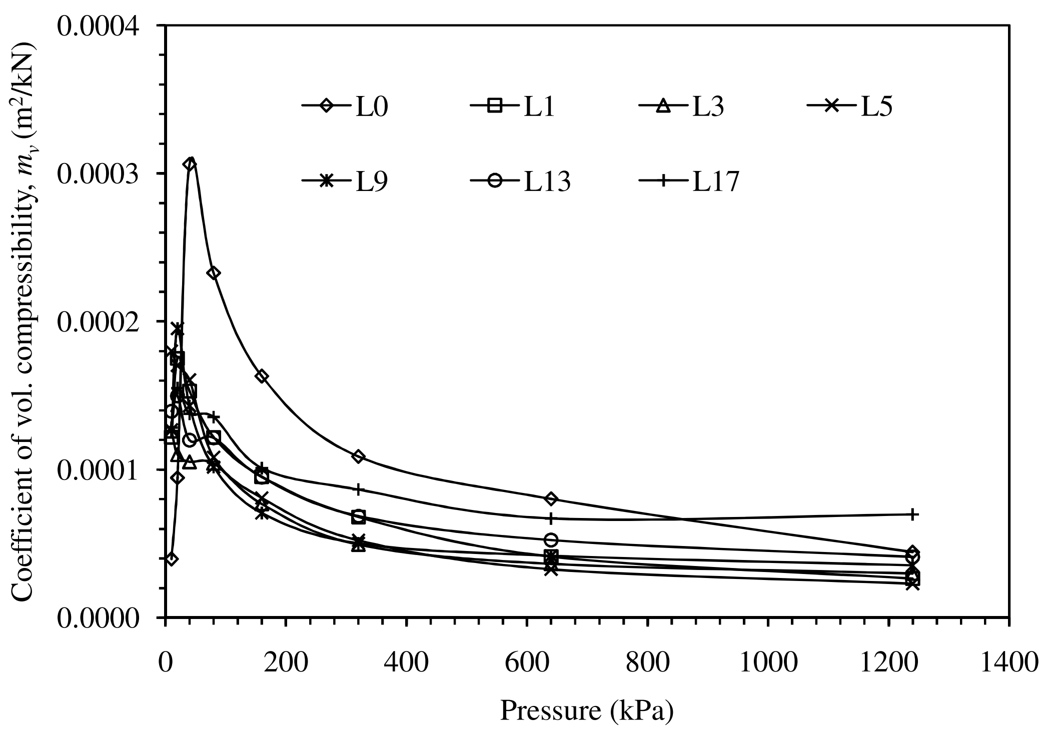


Fig. 7.18 Coefficient of volume compressibility of lime treated F80.

For better quantifying the effect of lime addition on FA amended ES, c_c and m_v corresponding to 320 kPa and 640 kPa have been plotted as a function of lime percentage, as depicted in Figs. 7.19 to 7.22. It is worth noting from these figures that, there is no desirable change in c_c and m_v for lime addition beyond 5%. For all practical purpose, the maximum lime modification of FA amended ES can be limited to 5% for obtaining the desired reduction in compressibility. It can be further noted that the reversal of trend for c_c for lime addition greater than 13% is more explicit for 640 kPa as compared to 320 kPa.

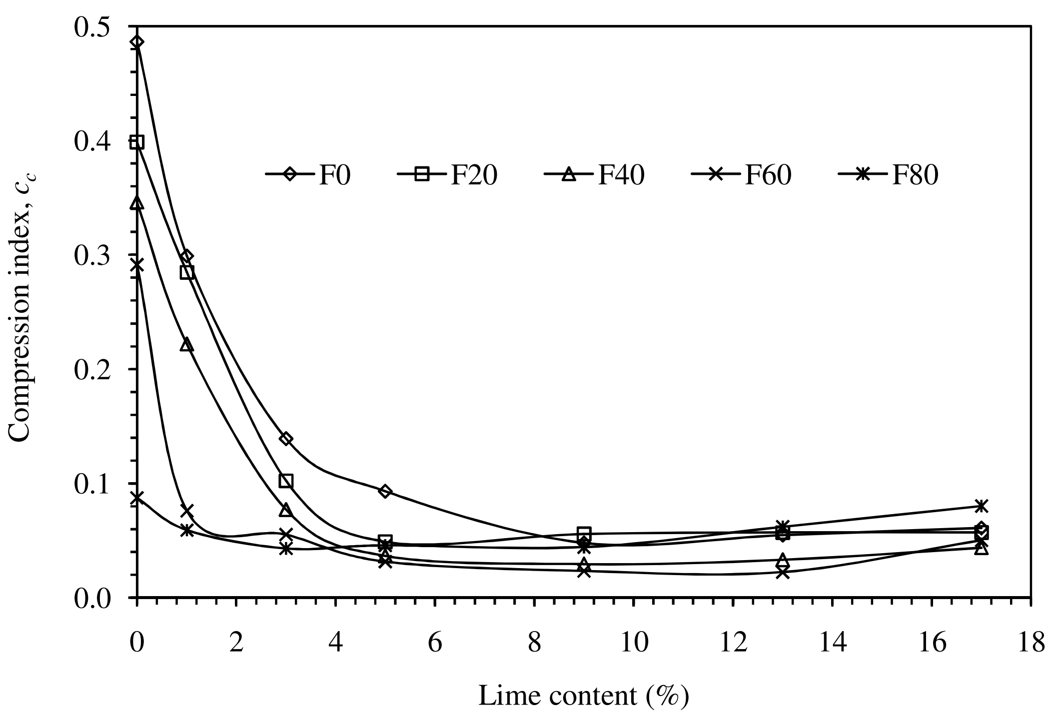


Fig.7.19 Variation of compression index with lime percentage at 320 kPa for ES-FA mixes.

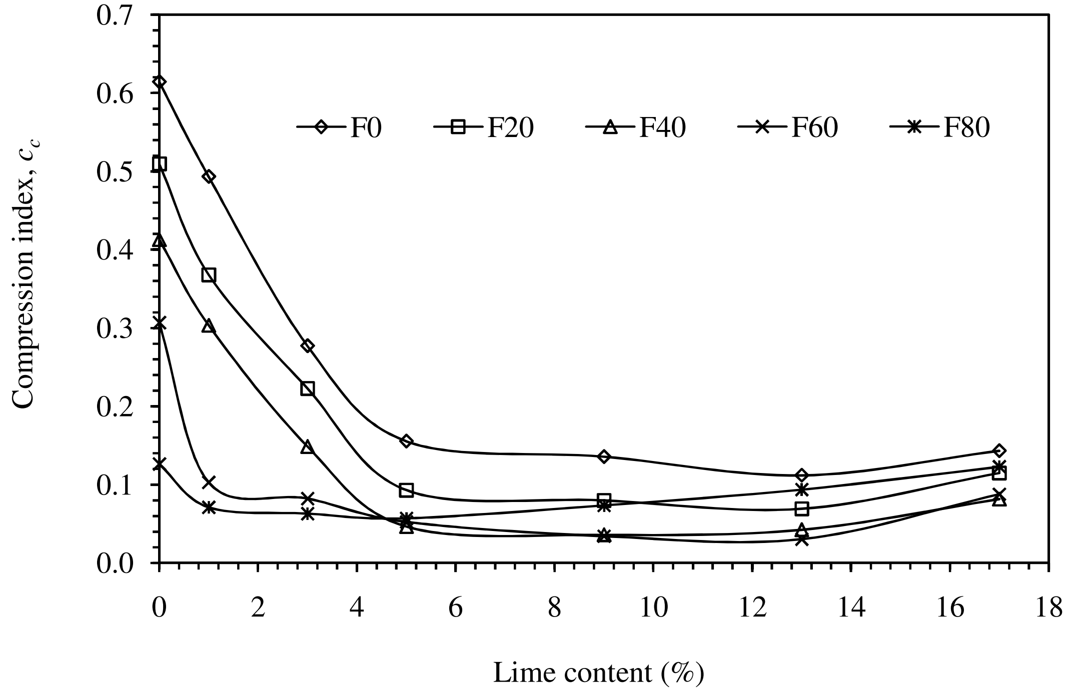


Fig.7.20 Variation of compression index with lime percentage at 640 kPa for ES-FA mixes.

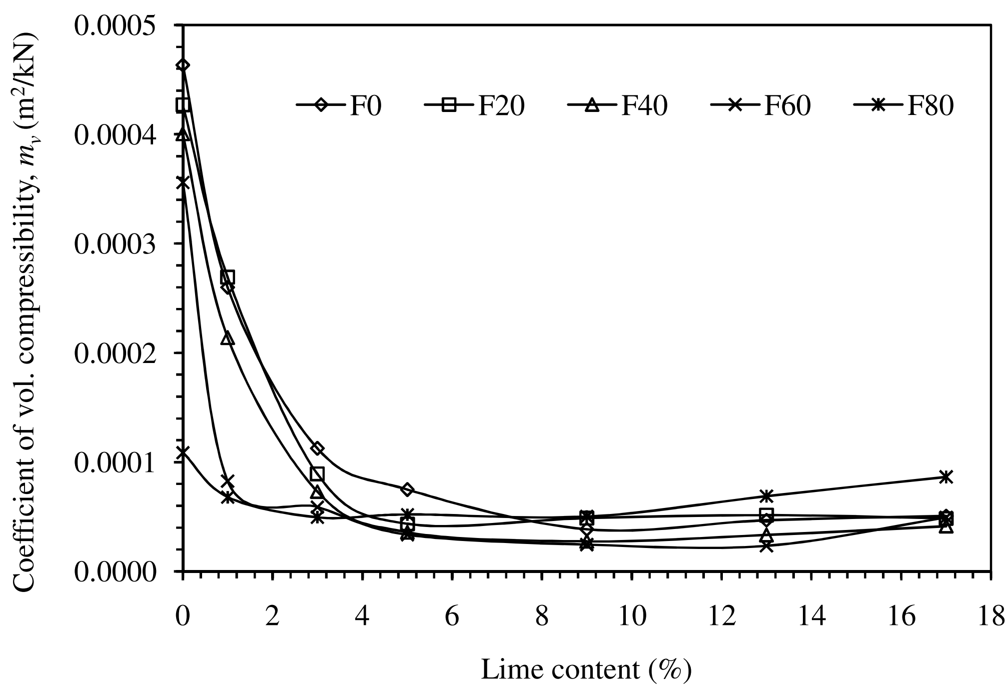


Fig.7.21 Variation of m_v with lime percentage at 320 kPa for ES-FA mixes.

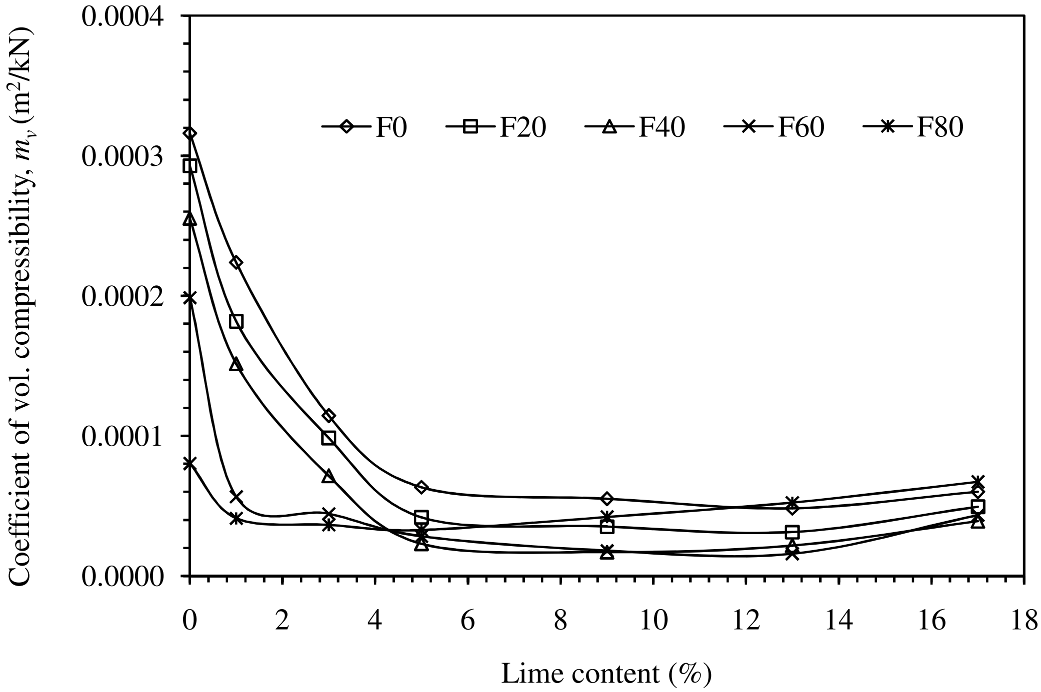
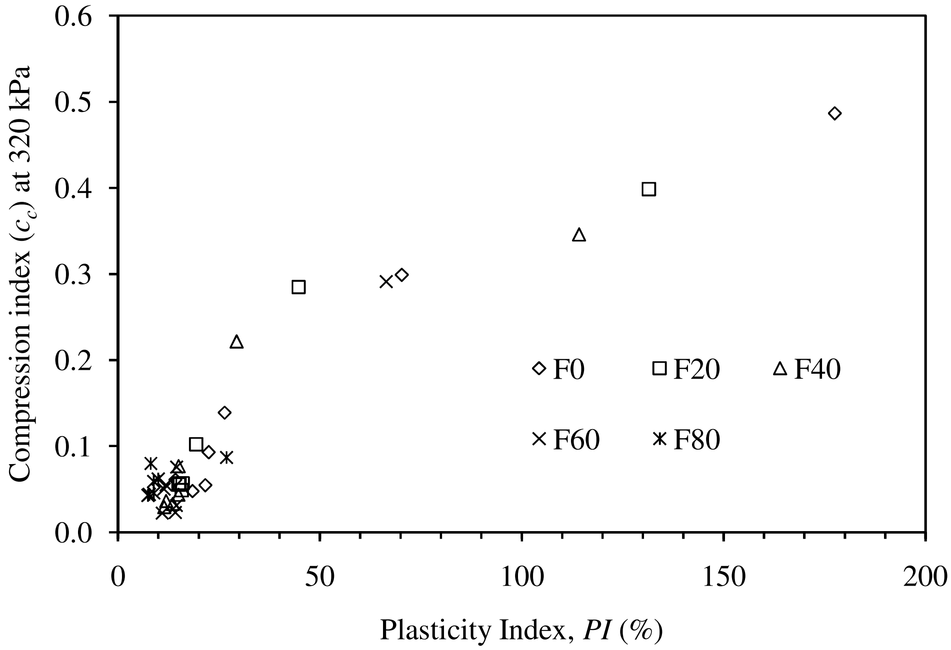


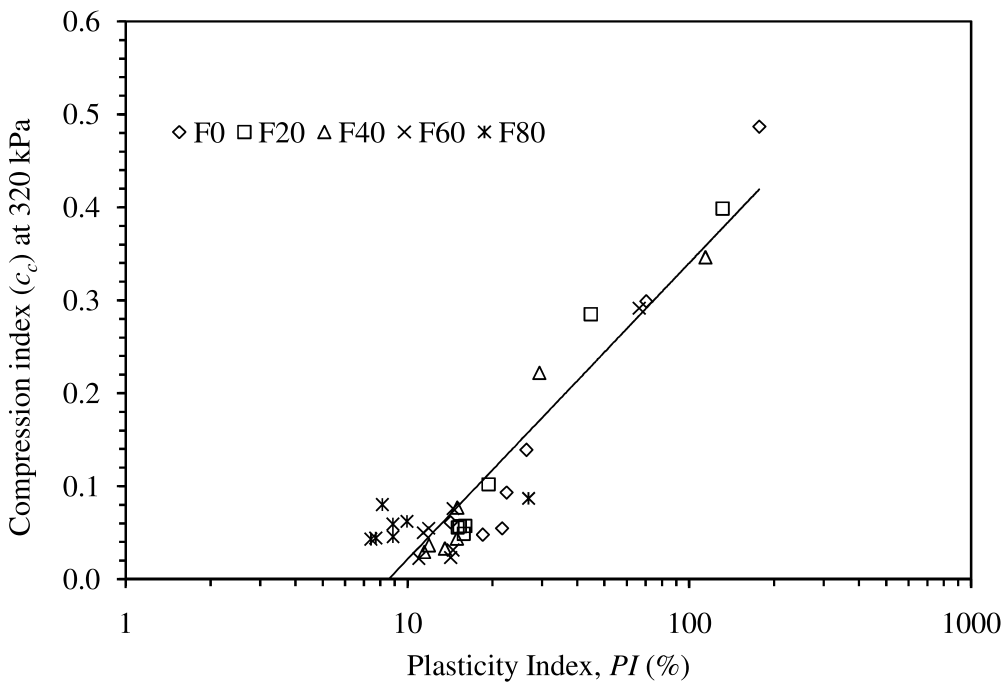
Fig.7.22 Variation of m_v with lime percentage at 640 kPa for ES-FA mixes.

7.3.1 Variation of c_c and m_v with Plasticity Index

The values of c_c and m_v, for different specimens, corresponding to 320 kPa and 640 kPa has been plotted as a function of plasticity index (*PI*) in Figs. 7.23 to 7.26 on (a) linear scale and (b) semi-log scale. It can be noted that there is a unique relationship between c_c, m_v and *PI* irrespective of the different combination of ES, FA and lime. It can be specifically noted that in the semi-log plot, c_c and m_v increases linearly with *PI* for *PI* > 10%. The summary of these linear relationships are listed in Table 7.1.

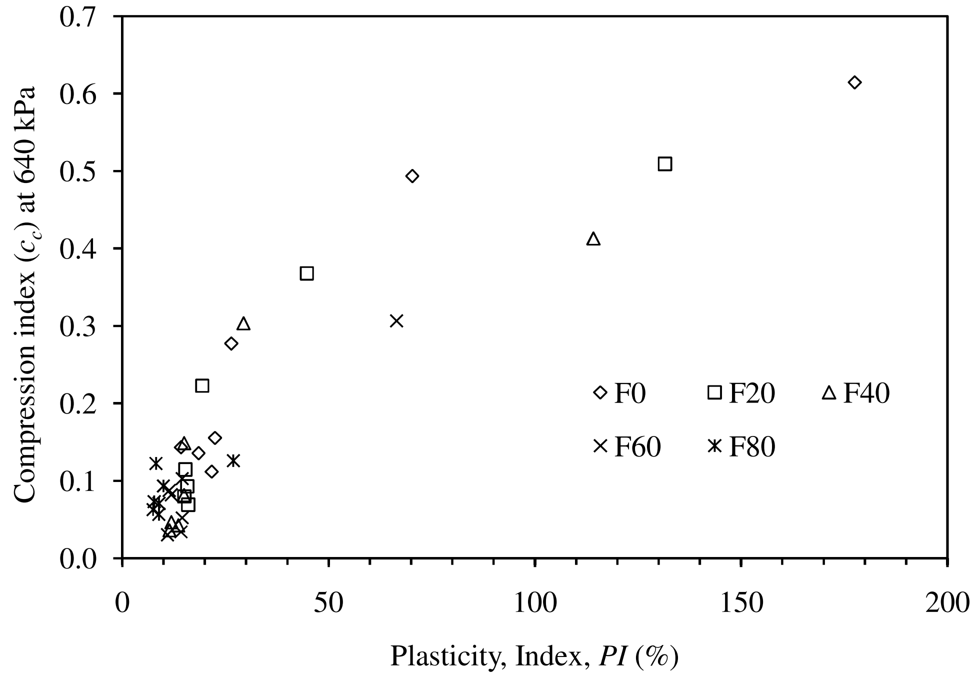


(a) Linear scale

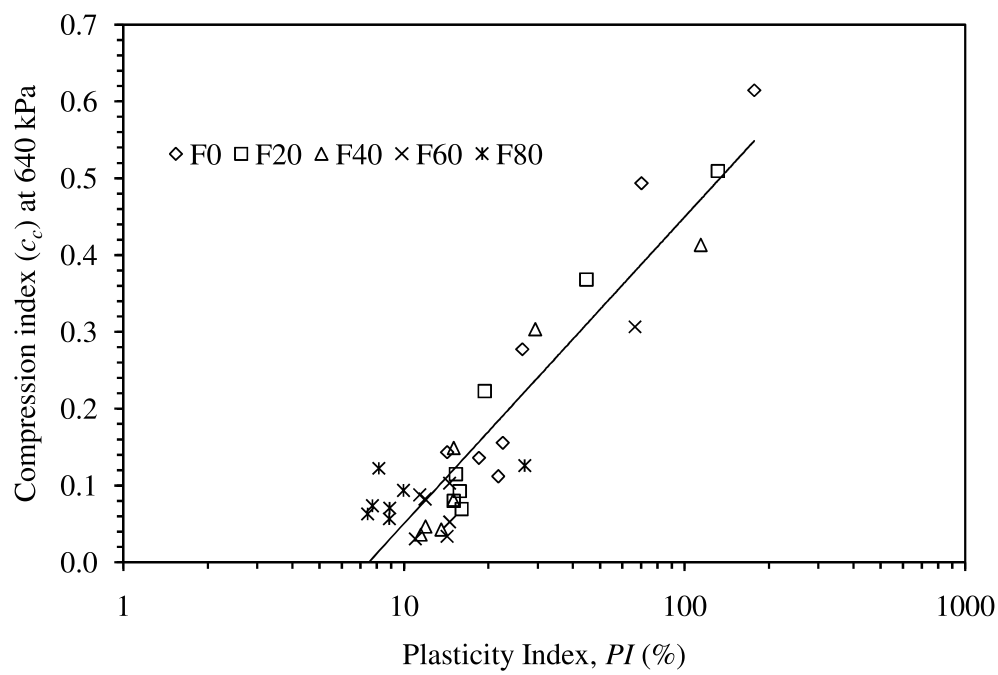


(b) Semi-log scale

Fig.7.23 Variation of compression index at 320 kPa with plasticity index.

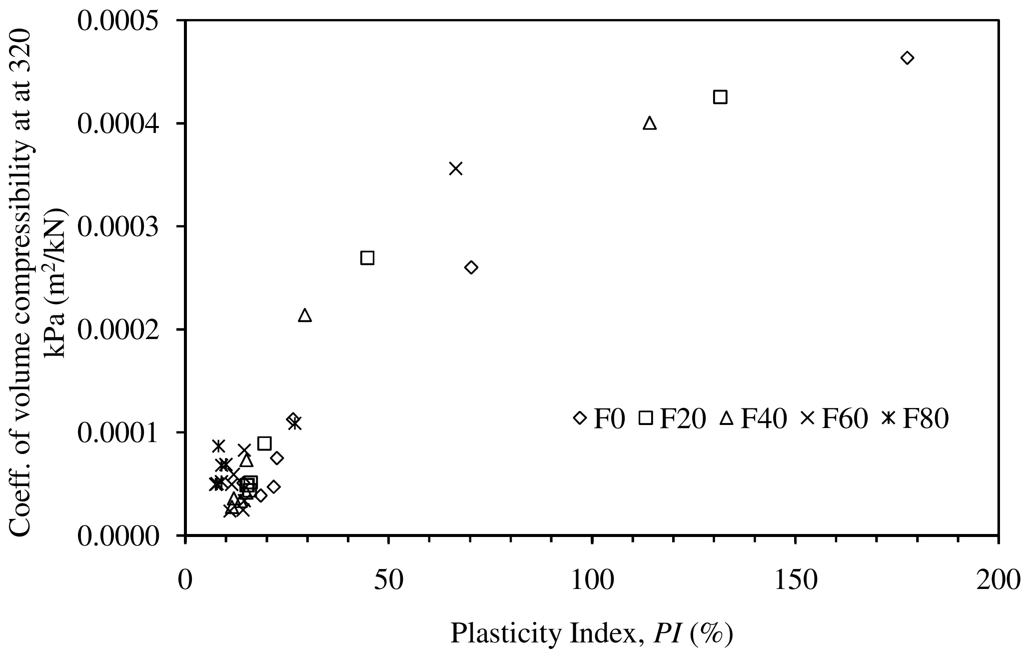


(a) Linear scale

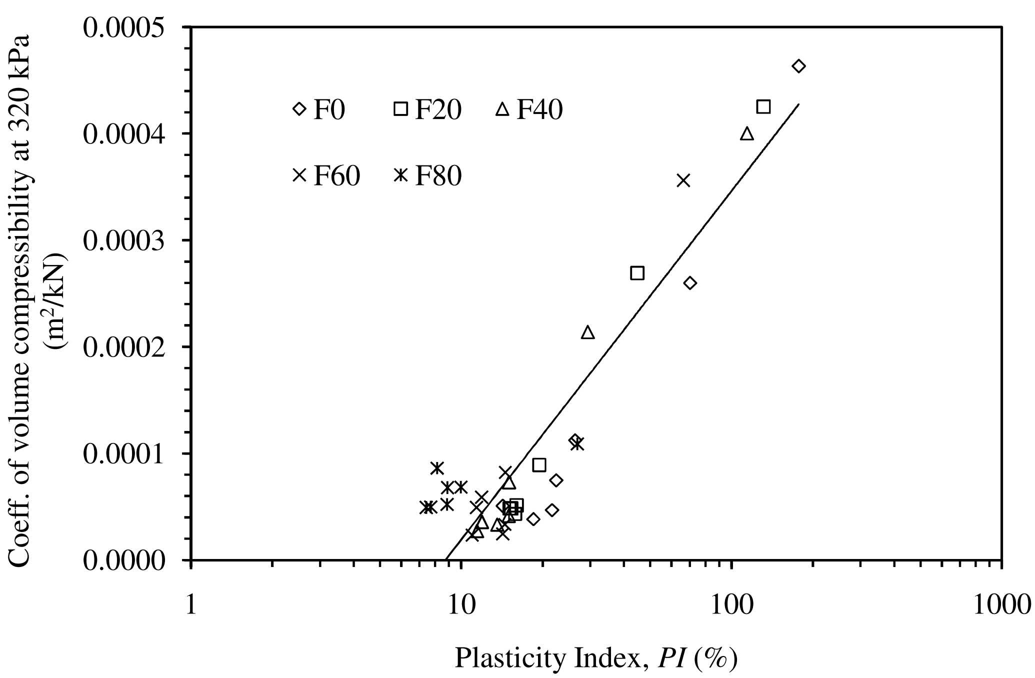


(b) Semi-log scale

Fig.7.24 Variation of compression index at 640 kPa with plasticity index.

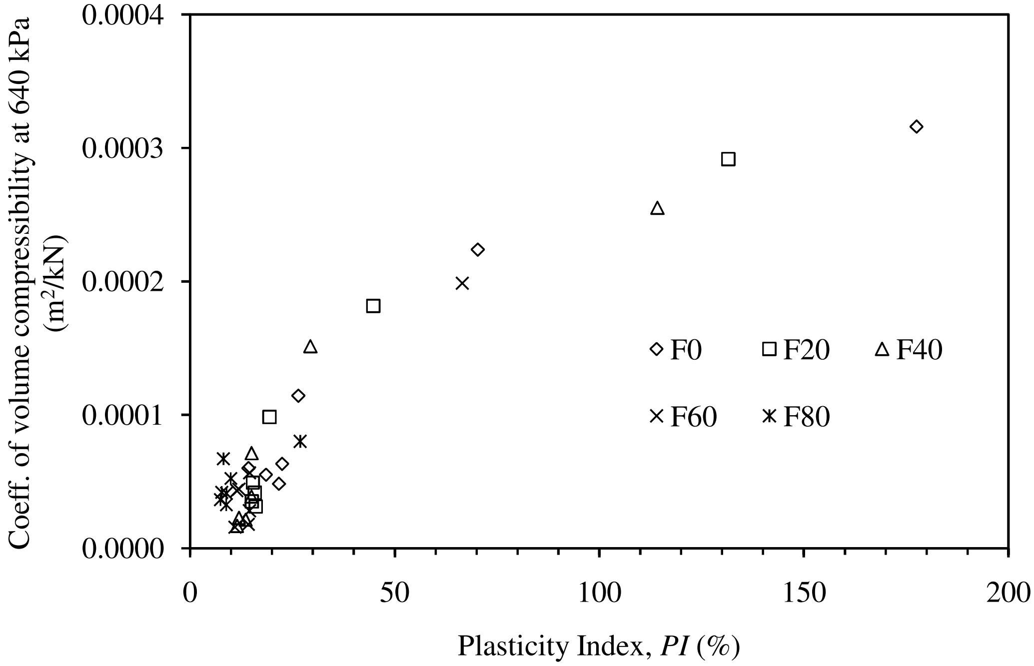


(a) Linear scale

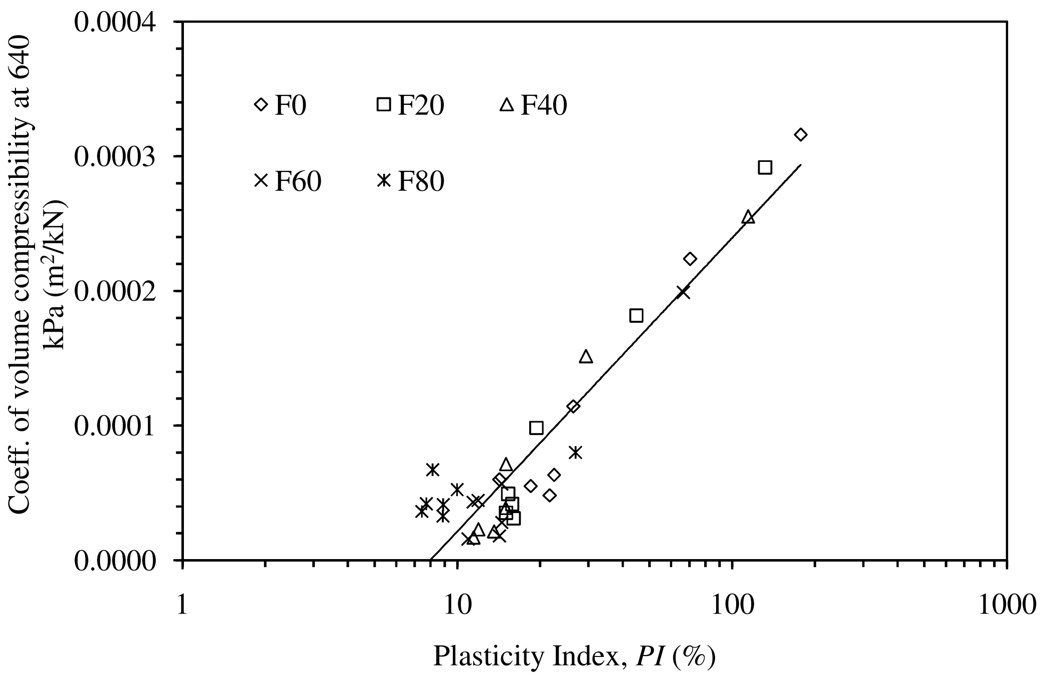


(b) Semi-log scale

Fig.7.25 Variation of coefficient of volume compressibility (m_v) at 320 kPa with plasticity index.



(a) Linear scale



(b) Semi-log scale

Fig.7.26 Variation of coefficient of volume compressibility (m_v) at 640 kPa with plasticity index.

Table 7.1 Trend equations for correlation of cc and m_v with plasticity index of lime-treated ES-FA mixes.

Consolidation pressure	Correlation equation	R^2
320 kPa	$c_c = 0.06\log(PI) - 0.298$	0.88
	$m_v = 0.00006\log(PI) - 0.00031$	0.86
640 kPa	$c_c = 0.075\log(PI) - 0.349$	0.86
	$m_v = 0.00004\log(PI) - 0.00020$	0.89

7.4 COMPARISON OF INDIVIDUAL EFFECTS OF LIME AND FLY ASH ON COMPRESSIBILITY OF ES

Figs. 7.27 to 7.30 show comparison of individual effects of fly ash and lime on c_c and m_v of ES, for two typical pressures i.e. 320 kPa and 640 kPa. Increasing fly ash content gradually reduces c_c and m_v where as there is a sharp reduction in these properties within limiting lime content of about 5%, beyond which further addition of lime is not beneficial in reducing compressibility. For similar reduction in compressibility (i.e. c_c and m_v) corresponding to lime content of 5%, around 80% of FA needs to be added to the ES. From environmental perspective, it is always recommended to use only FA with ES for the required reduction in compressibility and at the same time ensuring maximum utility of FA. Combined effect of 5% lime and 20-40% FA addition would give most suitable benefit in reducing compressibility of ES.

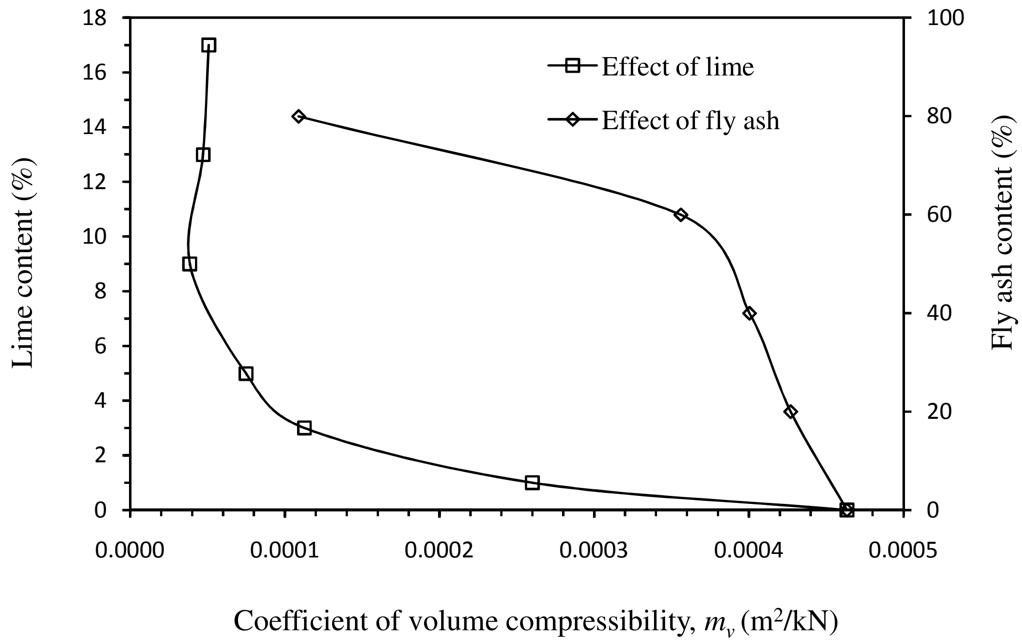


Fig. 7.27 Effect of lime and fly ash on m_v of ES, at 320 kPa.

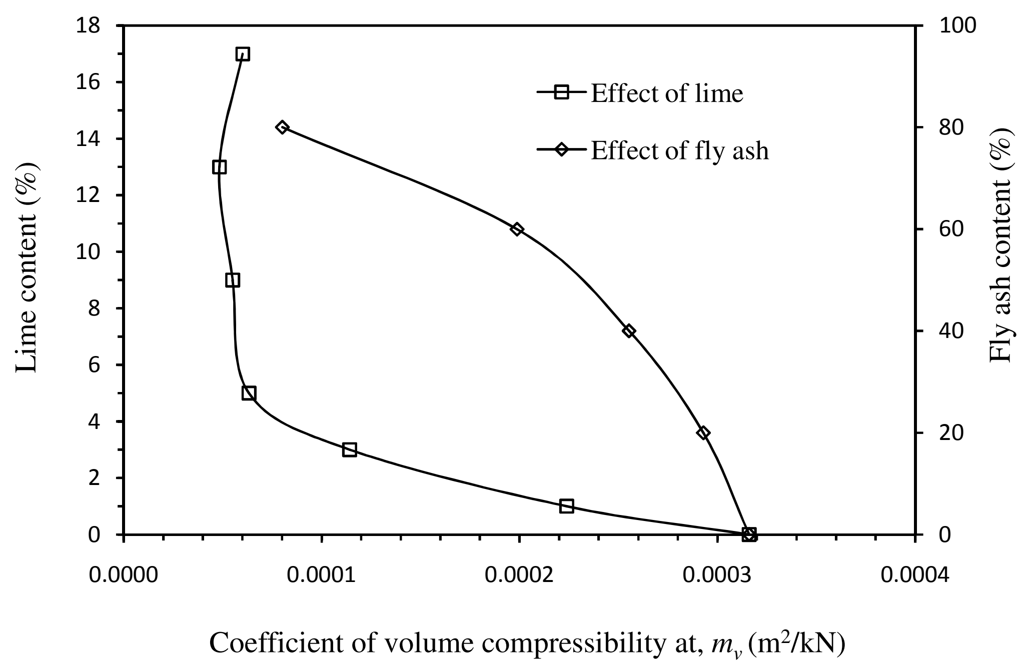


Fig. 7.28 Effect of lime and fly ash on m_v of ES, at 640 kPa.

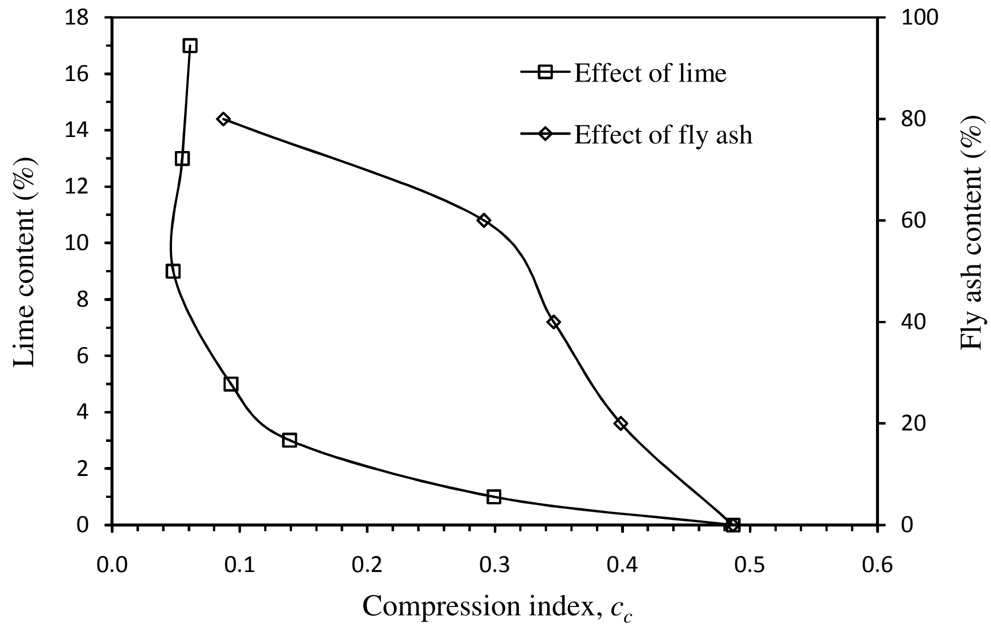


Fig. 7.29 Effect of lime and fly ash on c_c of ES, at 320 kPa.

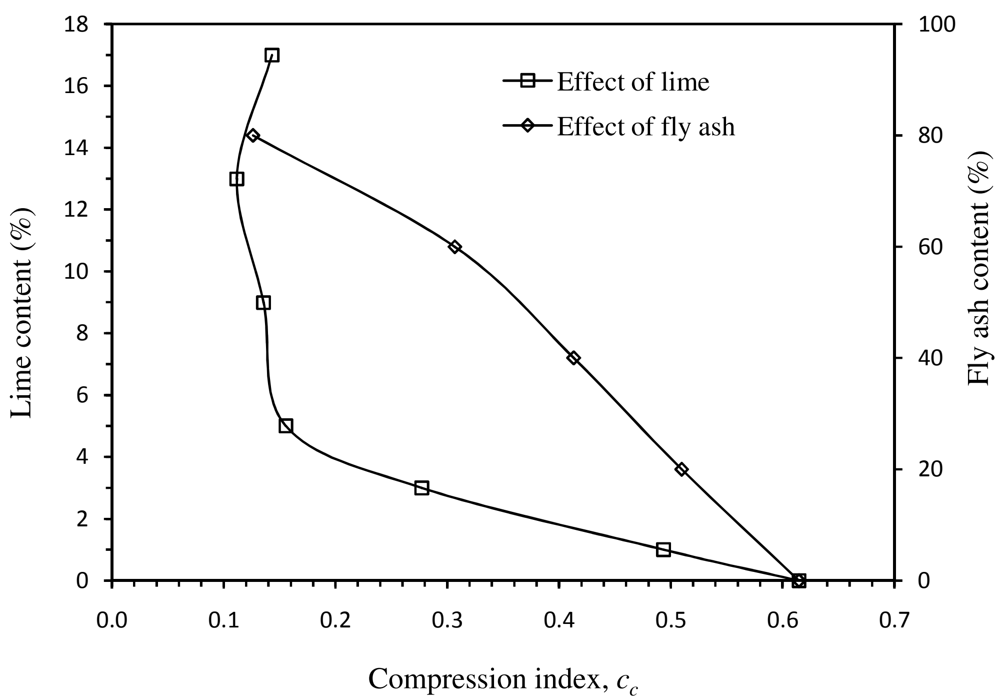


Fig. 7.30 Effect of lime and fly ash on c_c of ES, at 640 kPa.

7.5 SUMMARY

It can be noted that c_c increases with pressure up to 640 kPa for ES-FA mixes. The maximum value of c_c more or less remains constant for pressure greater than 640 kPa. Addition of fly ash reduces compressibility of expansive soil, which is reflected in the reduction of both compression index (c_c) and coefficient of volume compressibility (m_v). This indicates that settlement susceptibility of ES reduces with an increase in FA content. With addition of 80% of FA, there is 78% reduction of c_c of the ES. However, the reduction in m_v is relatively less i.e. 68%. Also, there is no significant variation in percentage reduction of m_v for different pressure ranges investigated in this study.

As expected, addition of lime to ES-FA mixes further reduces the compressibility. This reduction is steep for lime content up to 3% and then moderate for lime content up to 5%. Beyond this lime content, there is no significant reduction in compressibility. For all practical purpose, the maximum percentage of lime modification of FA amended ES can be limited to 5% for obtaining optimum reduction in compressibility. On the contrary, compressibility increases at high lime content >1% as compared to lower lime content soils. The increase in c_c with high lime percentage is more visible for mixes with high FA content. This may be due to the formation of CSH gel in large quantity, under the alkaline environment, resulting in the increase compressibility. This aspect need to be investigated in detail. It can be noted that there is a unique relationship between c_c, m_v and *PI*, irrespective of the different combination of ES, FA and lime. It can be specifically noted that in the semi-log plot, c_c and m_v increases linearly with *PI* for *PI*>10%.

While comparing the individual effects of FA and lime in reduction of compressibility, it is found that nearly the maximum reduction of compressibility is achieved with addition of 5% lime, while similar reduction can be obtained by replacement of expansive soil with as high as 80% fly ash. Therefore the resent study recommends a combination of 2% lime and 20-40% FA to ES for appreciable reduction in compressibility. This suggestion is mainly from environmental perspective to ensure maximum utility of FA, and at the same time meeting the main objective of reduction in compressibility.

CHAPTER 8

STRENGTH BEHAVIOUR

8.1 INTRODUCTION

Expansive soils are subjected to large increase in void ratio upon imbibition of water, resulting in diminished particle-to-particle contact, and hence reduced strength. In view of this, various treatments are being done to improve the performance of such soils. In order to quantify the performance improvement it is usual to measure strength of soils before and after the treatment. Unconfined compression (UC) tests provide a reliable and quick method of measurement of strength particularly when a large number of samples are to be tested (Sariosseiri and Muhunthan, 2009; Kalkana and Akbulut, 2004; Awoleye et al, 1991). In this chapter, results of unconfined compression tests on ES-FA mixes and lime-treated ES-FA mixes have been reported. Since lime stabilization of soils is dependent on time-dependent pozzolanic reactions, the UC tests were conducted on samples cured for different curing periods viz., 0, 1, 3, 7, 15, 30, 60 and 90 days.

8.2 STRENGTH BEHAVIOUR OF ES-FA MIXES

The stress-strain responses of the ES-FA mixes are presented in Fig. 8.1. It can be observed that for fly ash content upto 60% (F60) the specimens have shown ductile behavior, whereas, for higher contents of fly ash (F80 and F100) the behavior is brittle. Fig. 8.2 shows the typical failure patterns in the soil samples, recorded after the tests. It could be observed that while the expansive soil has undergone bulging, the fly as has shown clear shear failure.

Variation of peak strength termed as unconfined compressive strength (*UCS*) and the corresponding strain (i.e. failure strain) with different percent of fly ash, are shown in

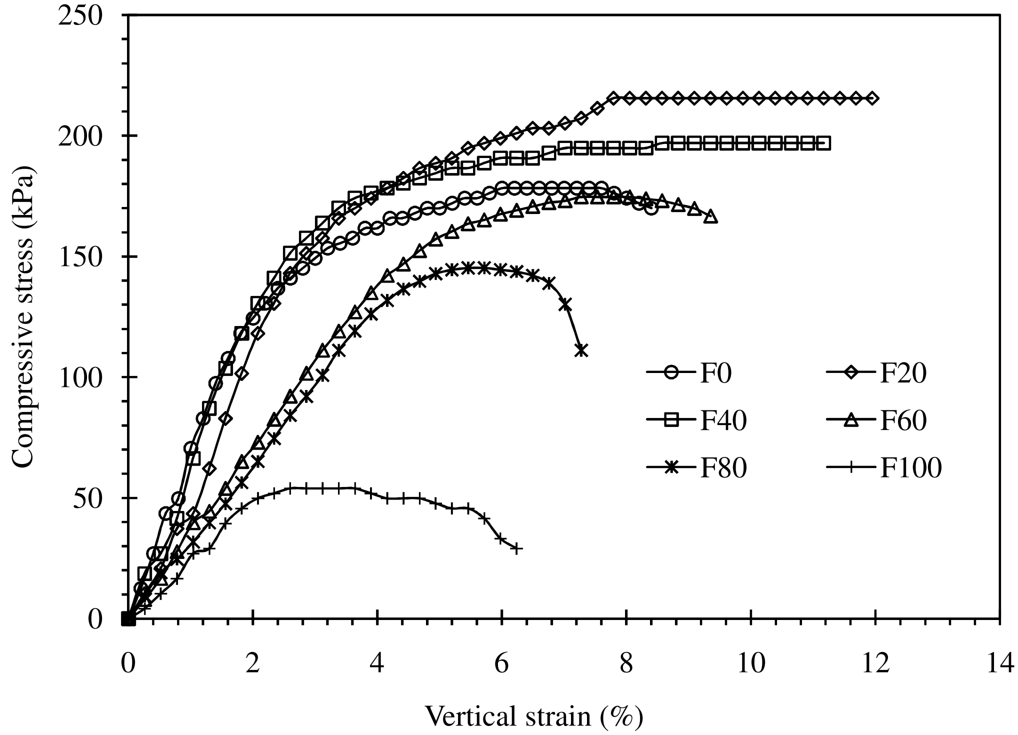


Fig. 8.1 Stress-strain response of expansive soil-fly ash mixes.

Fig. 8.3. It can be seen that the expansive soil (F0) attained a peak strength of 178 kPa at about 7% axial strain. With addition of 20% fly ash the compressive strength has increased to 216 kPa, resulting in a 20% rise in strength over that of the expansive soil. This is believed to be due to better packing of soil-fly ash particles leading to a coherent structure that sustains higher loading. Further addition of fly ash gradually reduces the peak strength as well as the failure strain. When FA content is more than about 60%, the strength has reduced to less than that of the soil alone. With higher FA content, cohesion component of strength is reduced. On the other hand, friction component of strength does not increase much as the fly ash particles are mostly round and comparable in size with that of the clay particles. The sample with only fly ash (F100) has a very low compressive strength. Since fly ash is non-cohesive, this small *UCS* is the result of the suction, developed through the small quantity of water (*OMC*) added during preparation of the specimen.

(a) (b)

Fig.8.2 Failure patterns under unconfined compression (a) expansive soil, (b) fly ash.

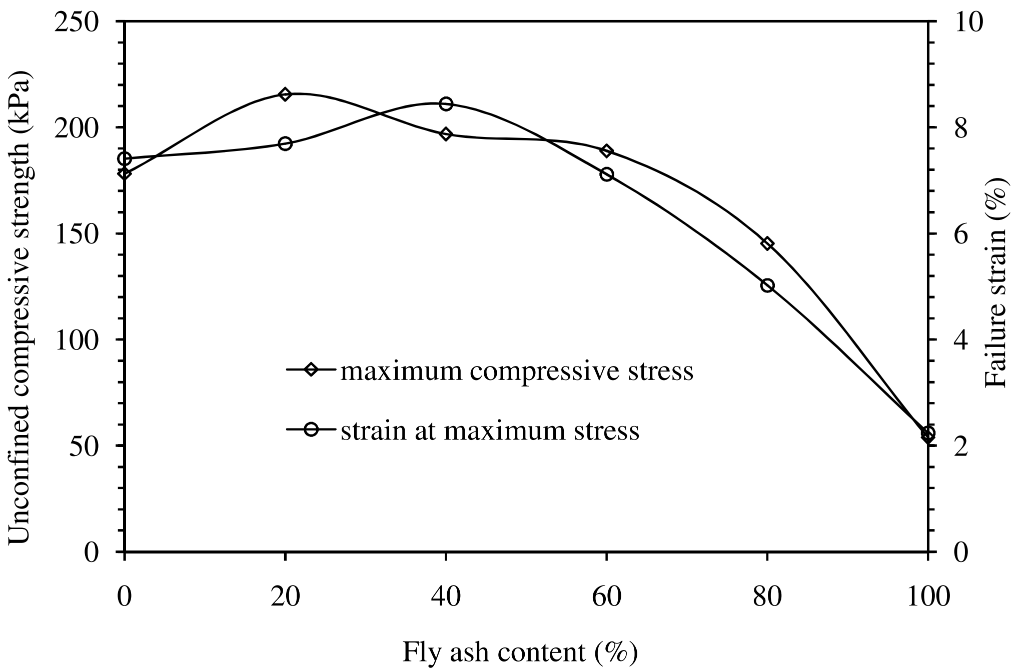


Fig. 8.3 Variation of *UCS* and failure strain of ES-FA mixes with fly ash content.

8.3 STRENGTH BEHAVIOUR OF ES-FA-LIME MIXES

The stress-strain responses of lime-treated expansive soil, for lime content of 1%, 3%, 5%, 7%, 13%, 17% and varying curing periods, are shown in Fig. 8.4, 8.to 8.9. It can be seen that invariably at all percentage of lime (i.e. both low and high) the influence of curing is visibly felt. However, the improvement is not linear and is more prominent at higher percentage of lime and increased curing period. Besides, as lime content increases, the failure behaviour has gradually shifted from ductile to brittle. Indeed the post test observed failure patterns for the low lime content (1%) specimens depicted in Fig. 8.10 show bulging, akin to ductile failure. Whereas, at higher lime content (9%, Fig. 8.11) the specimens have undergone brittle failure with visible cracking and shearing. This is attributed to the cementation of soil particles caused by the pozzolanic reactions. At high pH levels induced by the lime, alumina and silica get dissolved from the edges of the clay particles. These react with calcium and hydroxyl ions to form substances classified as calcium silicate hydrates (CSH) and calcium aluminate hydrates (CAH). These are gels which crystallize with time to become hard that cements the soil particles (Eades and Grim, 1960; Rajasekaran and Narasimha Rao, 1998).

Fig.8.12 depicts the variation of unconfined compressive strength of ES for different lime content and curing periods. With 1% lime, there is only marginal increase in *UCS*, irrespective of the curing period. A small quantity of lime is mostly used for the initial requirement of ES in altering its plasticity characteristics, through reduced thickness of the diffuse double layer. In the process, the soil tends to be more friable. As a result, the friction component of strength increases, leading to the increase in the *UCS*. Since a very small quantity of lime is left out for the remaining long term pozzolanic reaction, the strength does not increase appreciably with increased curing. With 3% lime, the strength improvement is found to be better. At 1% lime content, ES attains a pH value of 11.5, and at 3% lime content, pH becomes 12.0, coming close to the threshold value for starting the pozzolanic reactions. Therefore, at 3% lime content, pozzolanic reactions begin to take place leading to strength gain (i.e. from 36 kPa for untreated soil to 580 kPa with lime treatment). However, this lime content is not sufficient to cover all the soil particles and therefore the strength is not very high. With further increase in the lime content to 5%, the pH value increase to 12.15, that more calcium ions are available for pozzolanic reactions. Therefore, more soil particles participate in the pozzolanic reactions leading to increased strength (1224 kPa). In the similar way, at 90 days curing, *UCS* of ES has increased to 2633 kPa with lime content of 9% and 4540 kPa with lime content of 13%. However, when lime content exceeds 13%, the strength gain is found to reduce, particularly in case of higher curing period. This may be because of excess $Ca(OH)_2$, which remains unused in the reactions. It may be noted that $Ca(OH)_2$ itself does not have cohesion or significant friction (Bell, 1996). So, the excess of $Ca(OH)_2$ has a deleterious affect on strength. Another cause of reduction of strength may be the excess formation of CSH compounds, products of pozzolanic reactions between lime, water and silica. These are gel-like material which is highly porous and thereby holds a large amount of water leading to reduced strength. Beyond a certain stage the loss of strength due to such porous structure counteracts the strength gain due to cementation leading to overall reduction in strength. Besides, the excessive gel water might have served as a lubricant to reduce the strength. In view of this, for the present expansive soil, it can be said that 13% lime content is the optimum amount that can give maximum strength improvement. Similar behavior has also been observed in case of liquid limit and plastic limit tests (Chapter 4) where *LL* and *PL* values have shown visible increase when lime content is greater than to 13%.

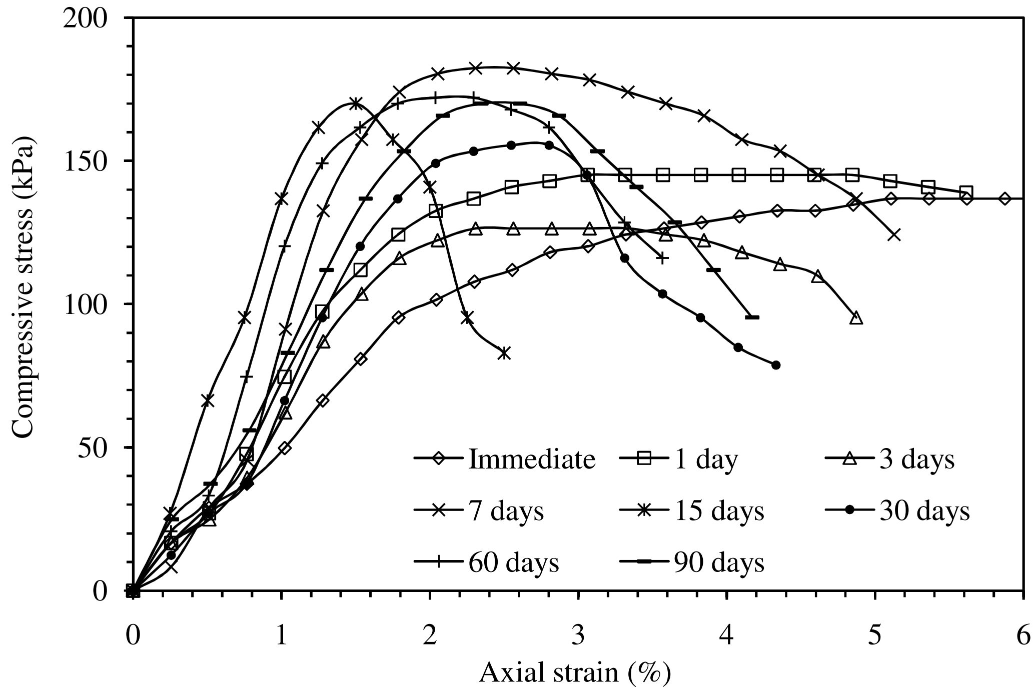


Fig. 8.4 Stress-strain responses of ES treated with 1% lime.

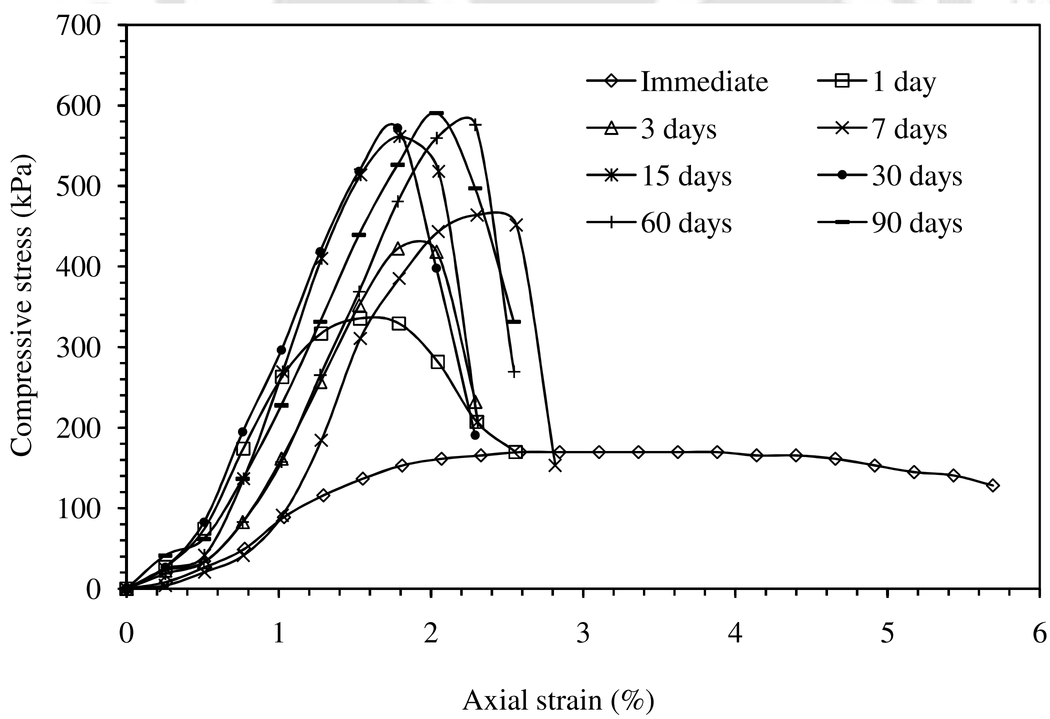


Fig. 8.5 Stress-strain responses of ES treated with 3% lime.

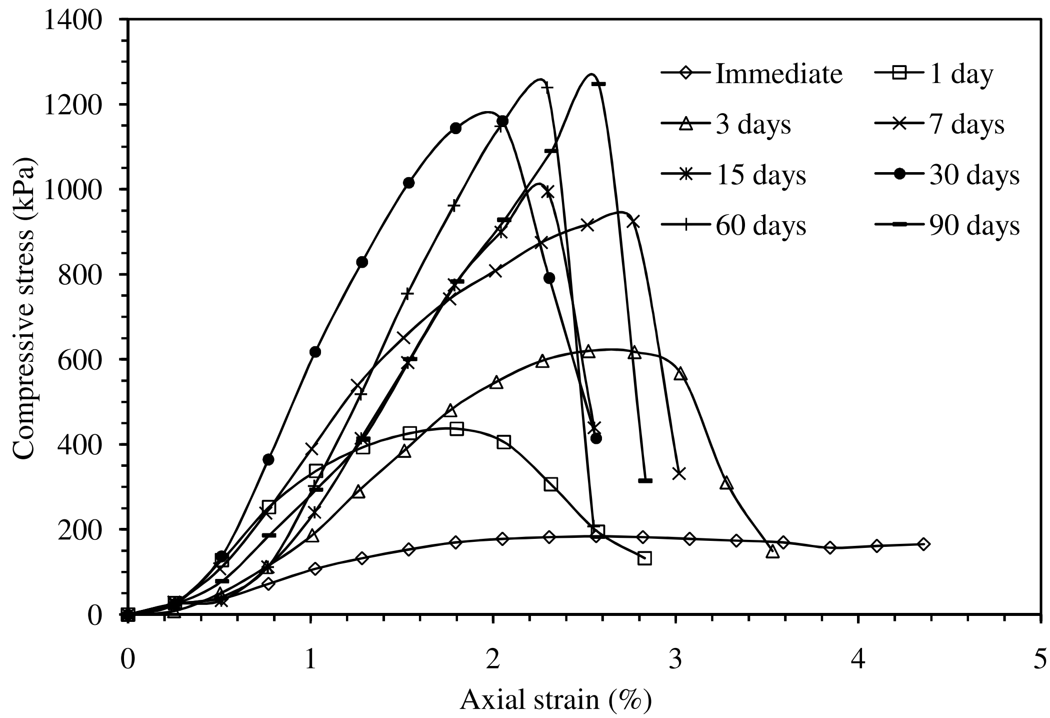


Fig. 8.6 Stress-strain responses of ES treated with 5% lime.

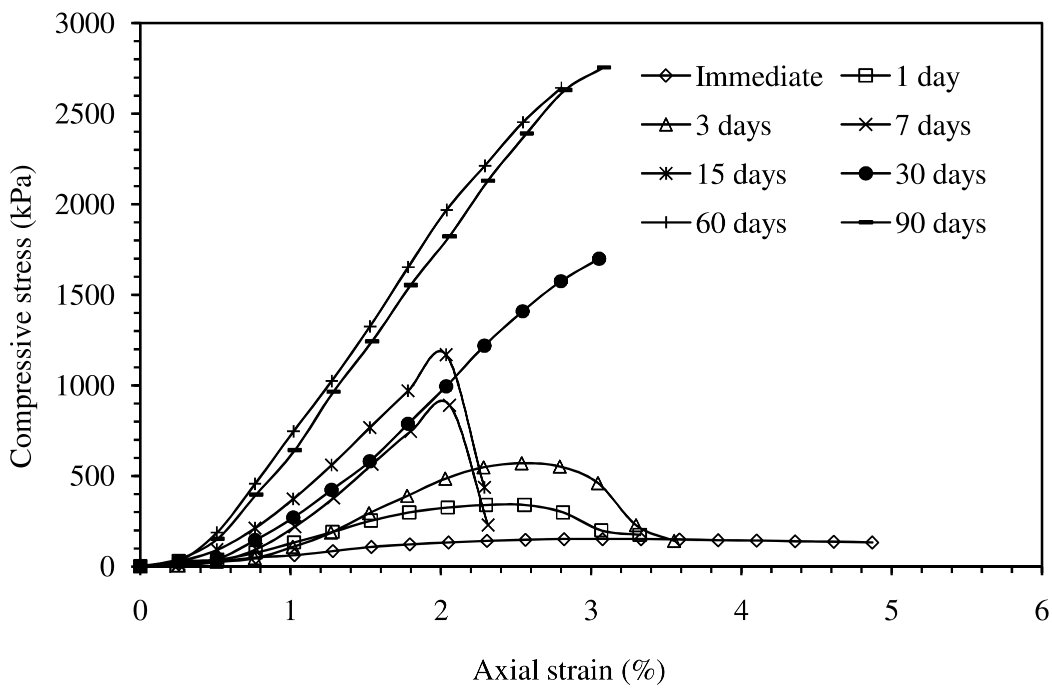


Fig. 8.7 Stress-strain responses of ES treated with 9% lime.

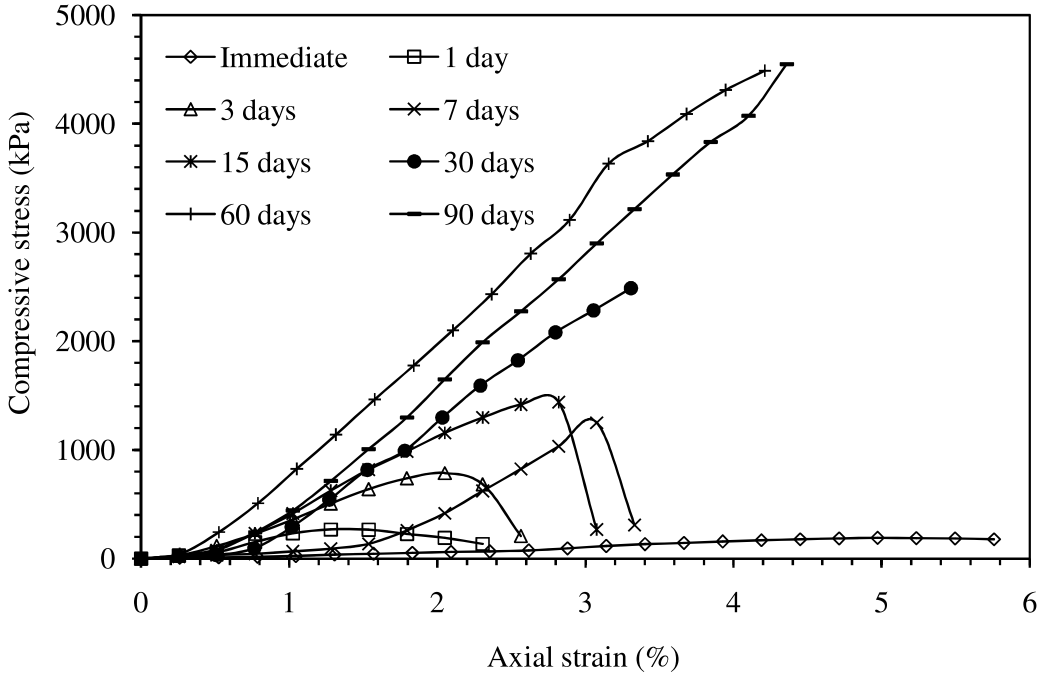


Fig. 8.8 Stress-strain responses of ES treated with 13% lime.

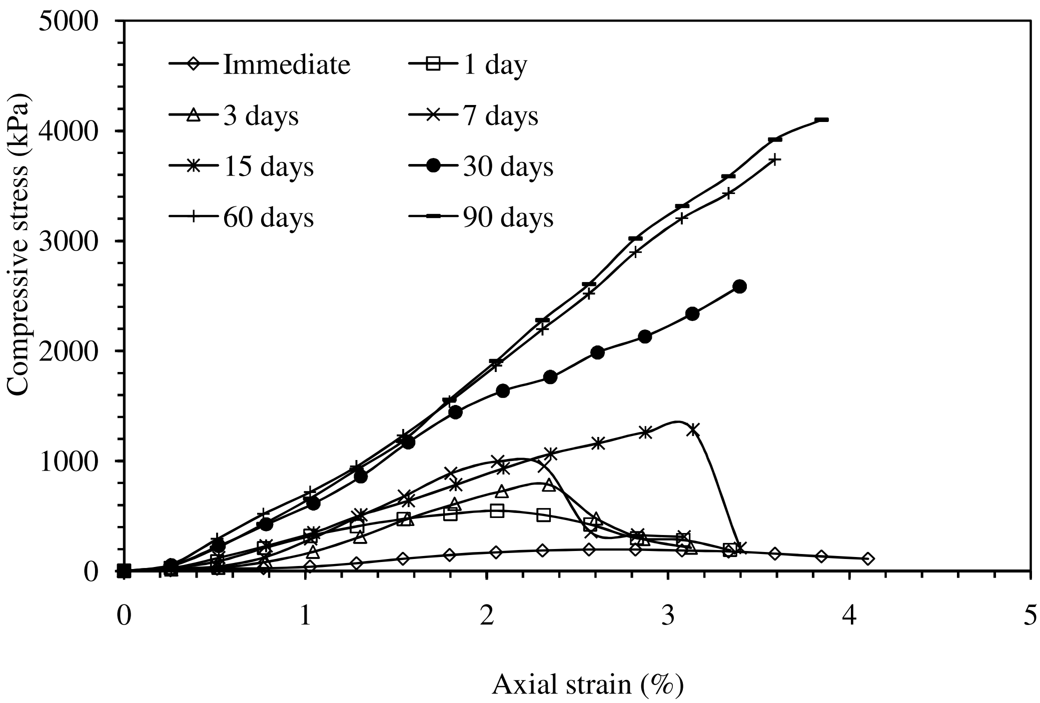


Fig. 8.9 Stress-strain responses of ES treated with 17% lime.

(a) (b) (c)

Fig. 8.10 Typical failure patterns of treated ES with low lime content (1%) and different curing periods – (a) Immediate, (b) 3 days (c) 15 days.

(a) (b) (c)

Fig. 8.11 Typical failure patterns of treated ES with high lime content (9%) and different curing periods – (a) Immediate, (b) 3 days (c) 15 days.

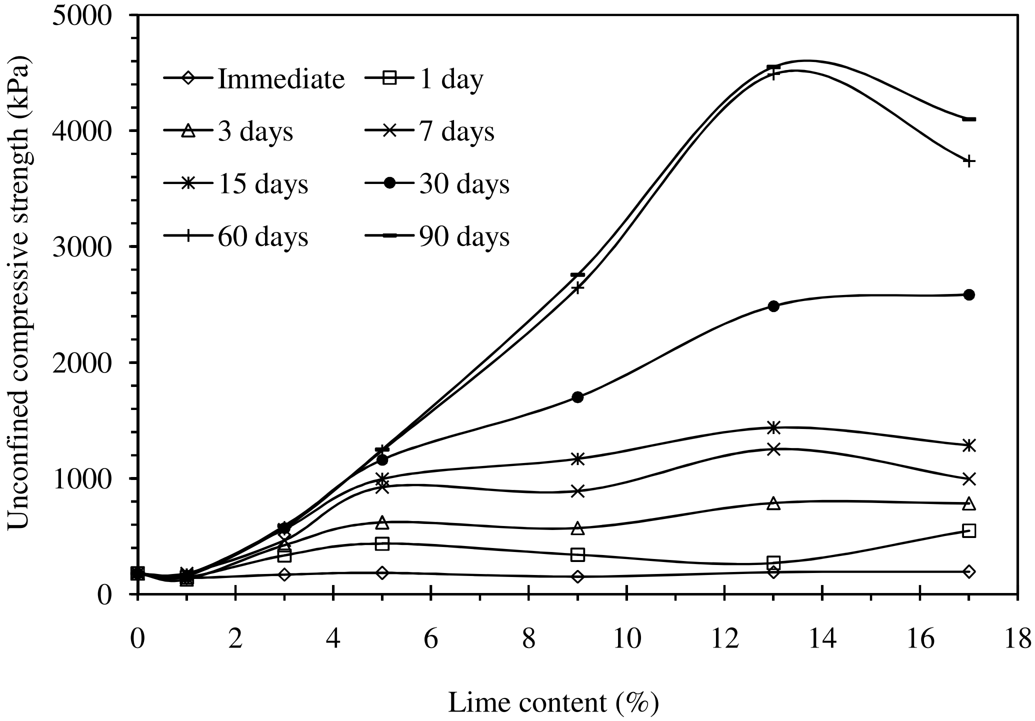


Fig.8.12 Effect of lime and curing on *UCS* of expansive soil.

The stress-strain responses of lime treated ES-FA mixes are presented in Figs. 8.13-8.36. It could be observed that in case of no curing (i.e. Immediate) and relatively short curing period (i.e. 1, 3 days) the responses are mostly ductile. However, with higher period of curing, in majority of cases, the strength after reaching a peak value, suddenly drops to zero, typical of brittle behaviour. This trend is more prominent with increased percentage of FA and lime. The strength and stiffness increase of a clay soil due to lime treatment is dependent on the pozzolans present in it (Bell, 1996). Fly ash, because of the reactive silica present in it, is a good pozzolanic material. With increased quantity of lime and curing period more of cementitious compounds are formed that gives rise to stiffened response of the material. As cemented soil cracks under loading, brittle failure is observed thereof.

The stress-strain response of the fly ash with varied percentage of lime and curing period are depicted in Figs. 8.37-8.42. It can be seen that at relatively low percentage of lime the stress-strain responses have mostly exhibited ductile response and the strength gain is visibly less. Even with higher percentage of lime (i.e. ≥ 5%) and moderate curing, the specimens are found to have shown ductile response. Only after long curing period (i.e. 60-90 days) that the specimen has turned brittle and the strength gain is tangibly high. This is attributed to the high content of ashes in the fly ash, that being inert in nature needs large quantity of cementitious compounds to bind them effectively. Therefore, with higher lime content and prolonged curing period, that gives rise to large quantity of pozzolanic cementitious products, resulted in visible improvement in strength of the fly ash. This is in contrary to the behavior of the ES-FA mixes, wherein lime induced strength gain is relatively quick. This is because the relatively stronger soil particles are effectively bridged through cementation. Hence it can be said that compared to fly ash alone, the soil fly-ash mix is a better material for lime treatment.

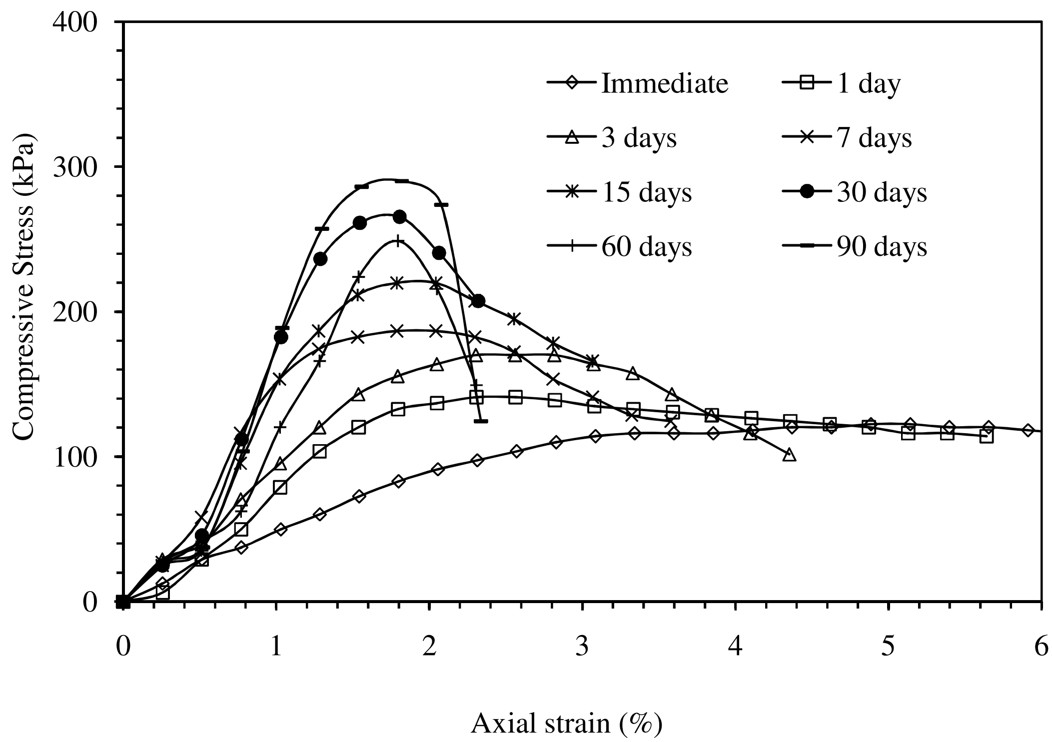


Fig. 8.13 Stress-strain responses of ES-FA mix F20 treated with 1% lime.

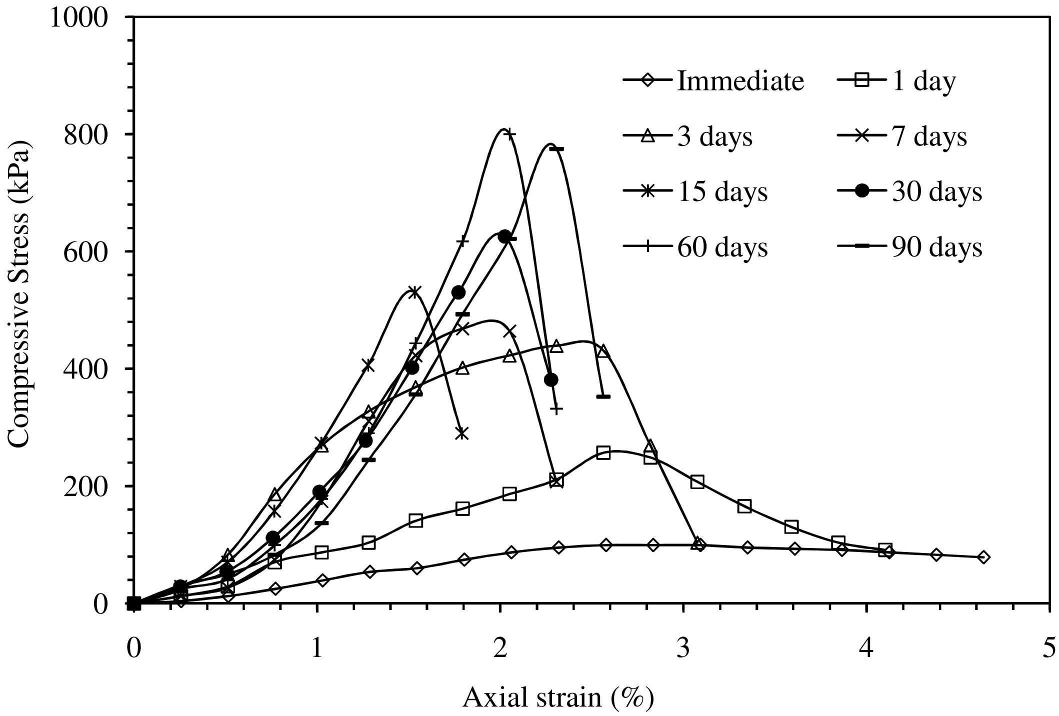


Fig. 8.14 Stress-strain responses of ES-FA mix F20 treated with 3% lime.

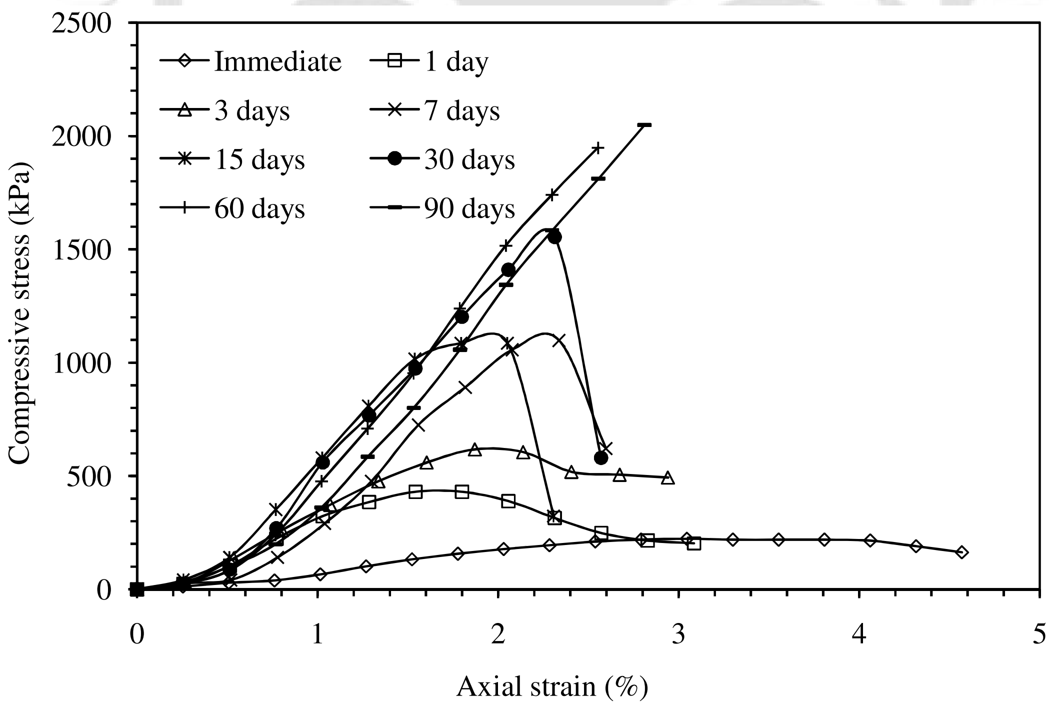


Fig. 8.15 Stress-strain responses of ES-FA mix F20 treated with 5% lime.

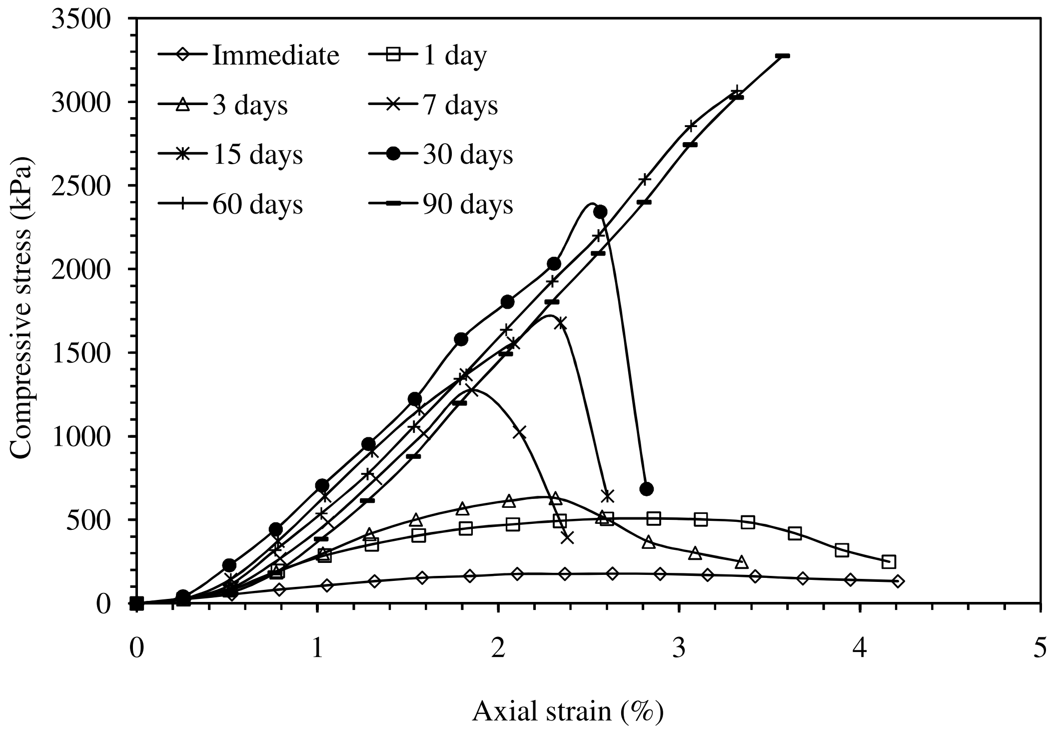


Fig. 8.16 Stress-strain responses of ES-FA mix F20 treated with 9% lime.

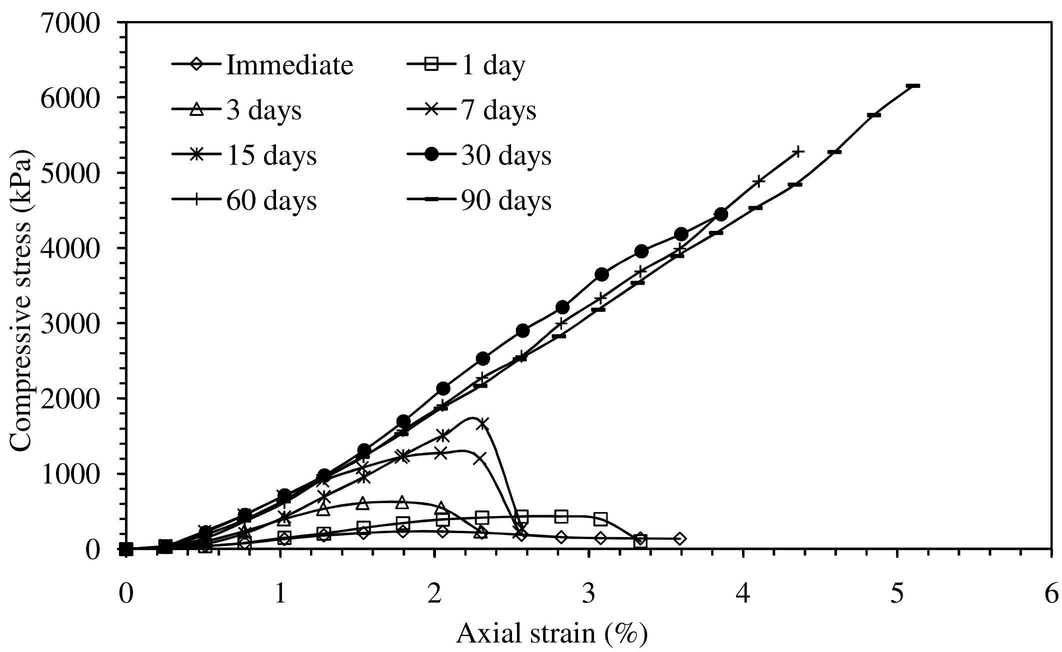


Fig. 8.17 Stress-strain responses of ES-FA mix F20 treated with 13% lime.

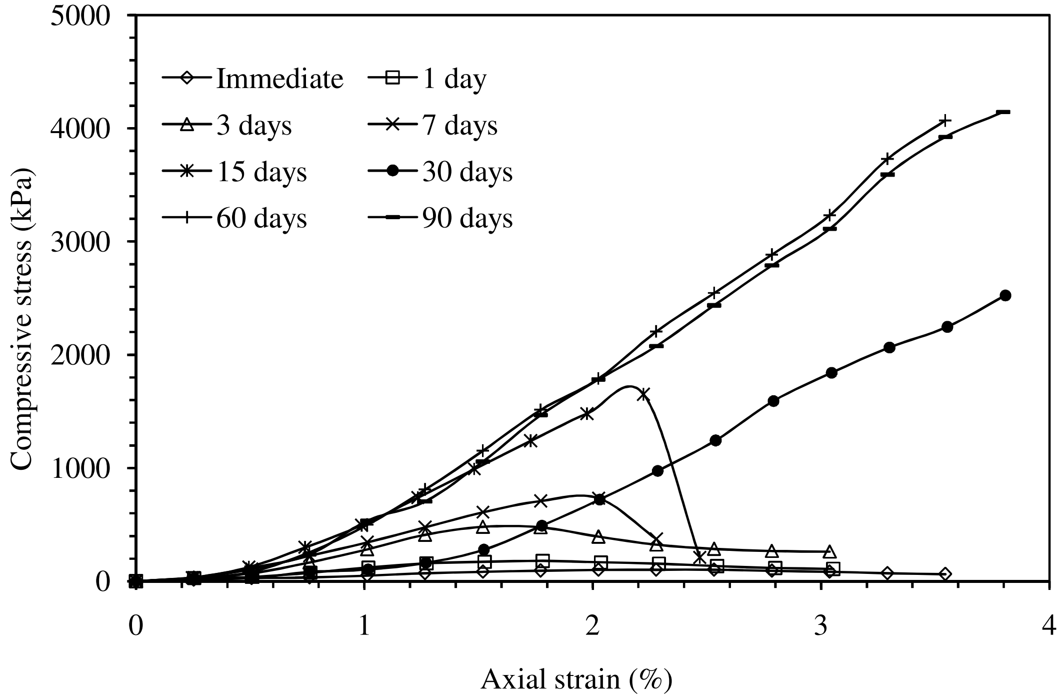


Fig. 8.18 Stress-strain responses of ES-FA mix F20 treated with 17% lime.

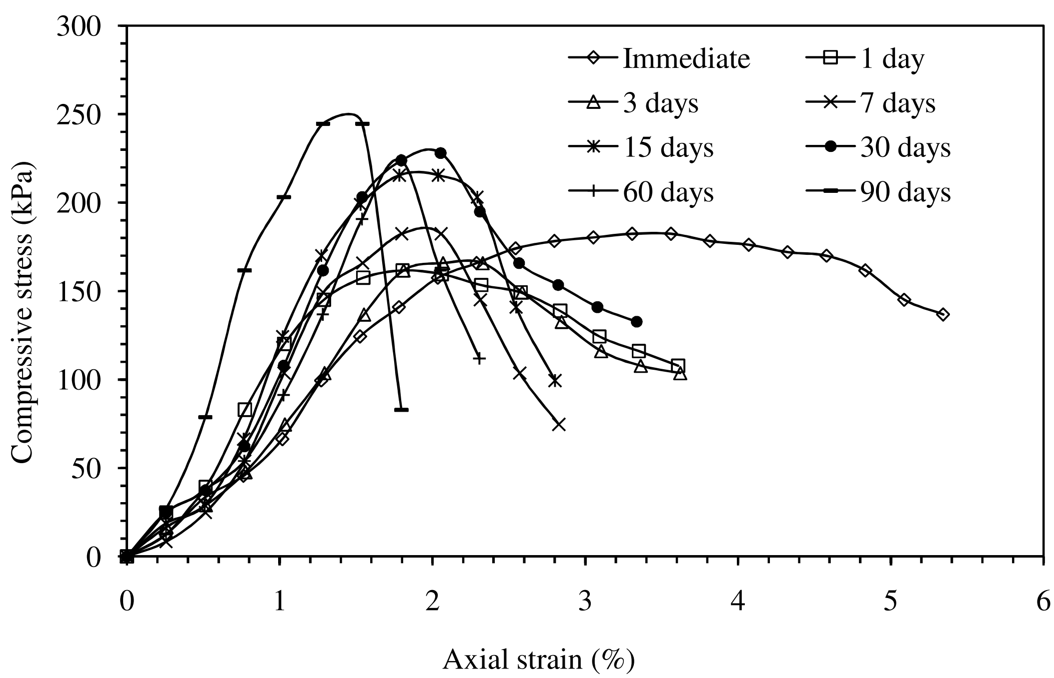


Fig. 8.19 Stress-strain responses of ES-FA mix F40 treated with 1% lime.

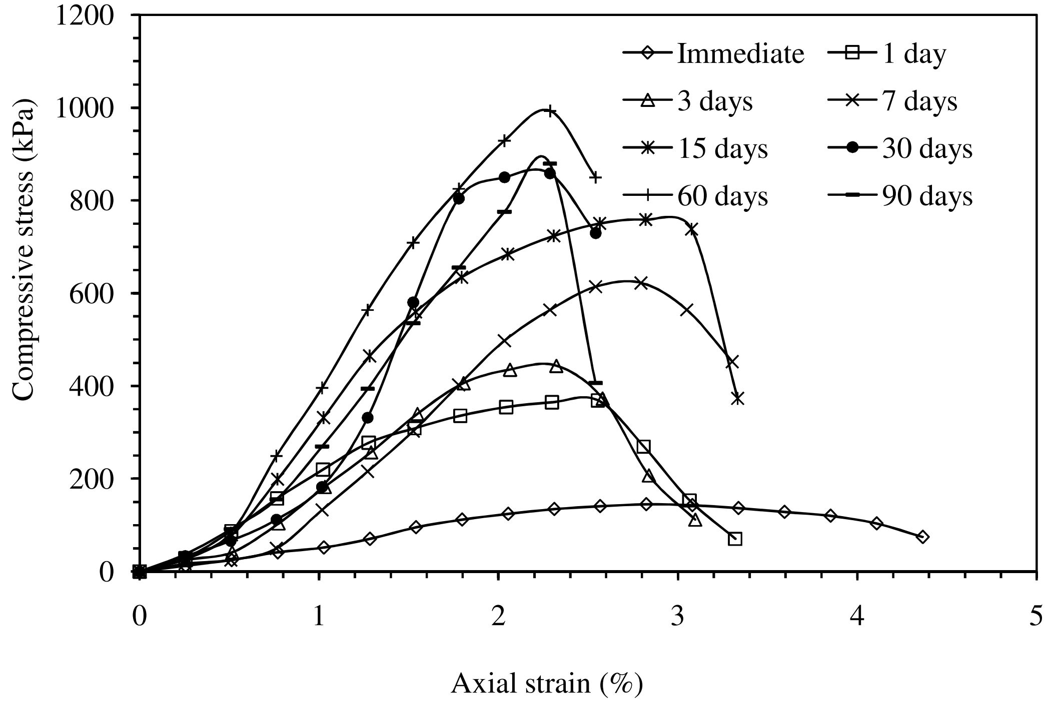


Fig. 8.20 Stress-strain responses of ES-FA mix F40 treated with 3% lime.

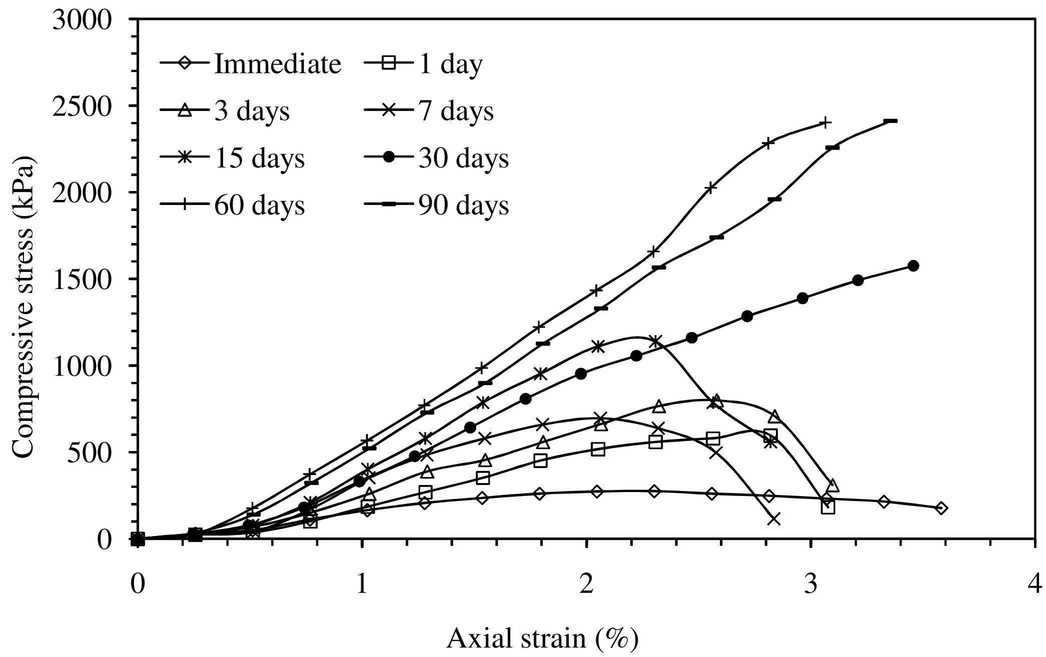


Fig. 8.21 Stress-strain responses of ES-FA mix F40 treated with 5% lime.

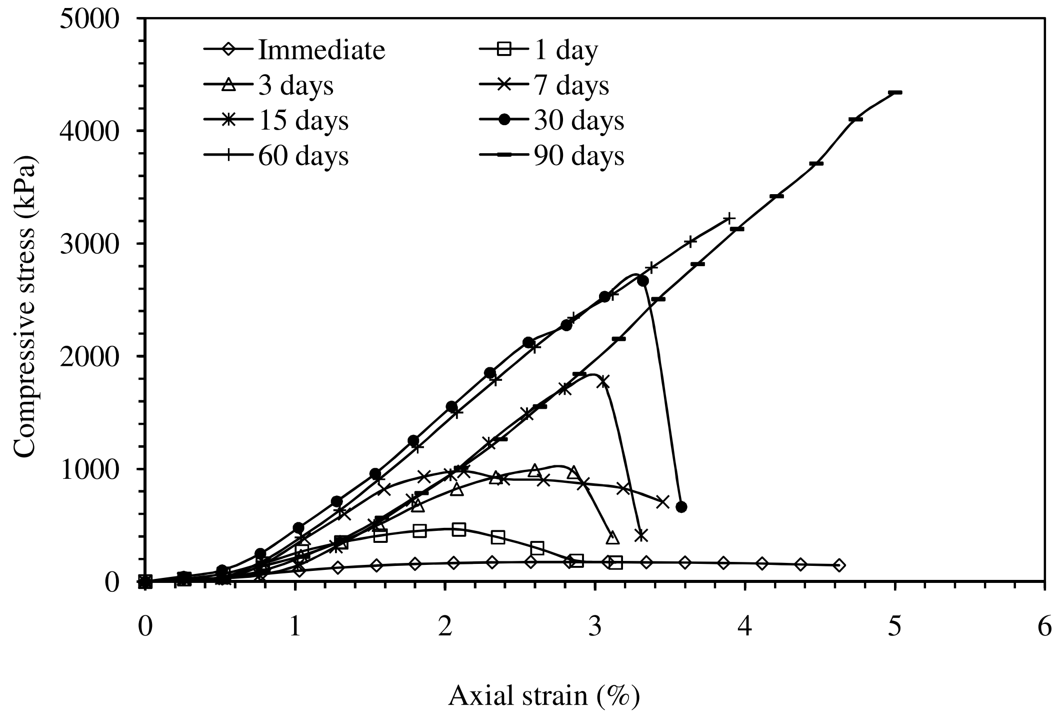


Fig. 8.22 Stress-strain responses of ES-FA mix F40 treated with 9% lime.

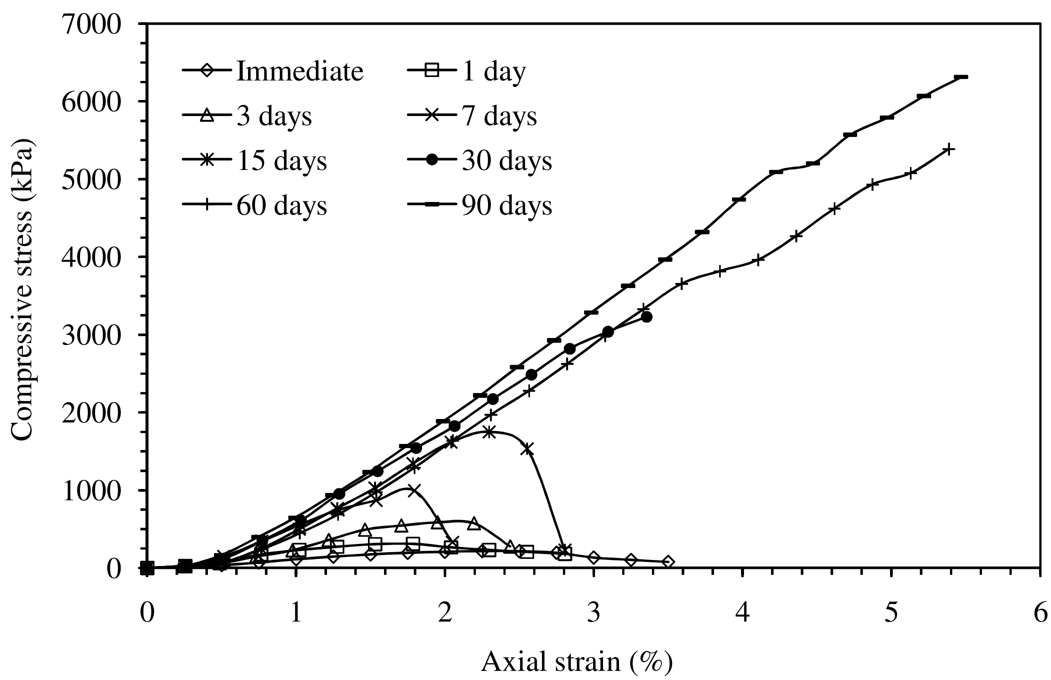


Fig. 8.23 Stress-strain responses of ES-FA mix F40 treated with 13% lime.

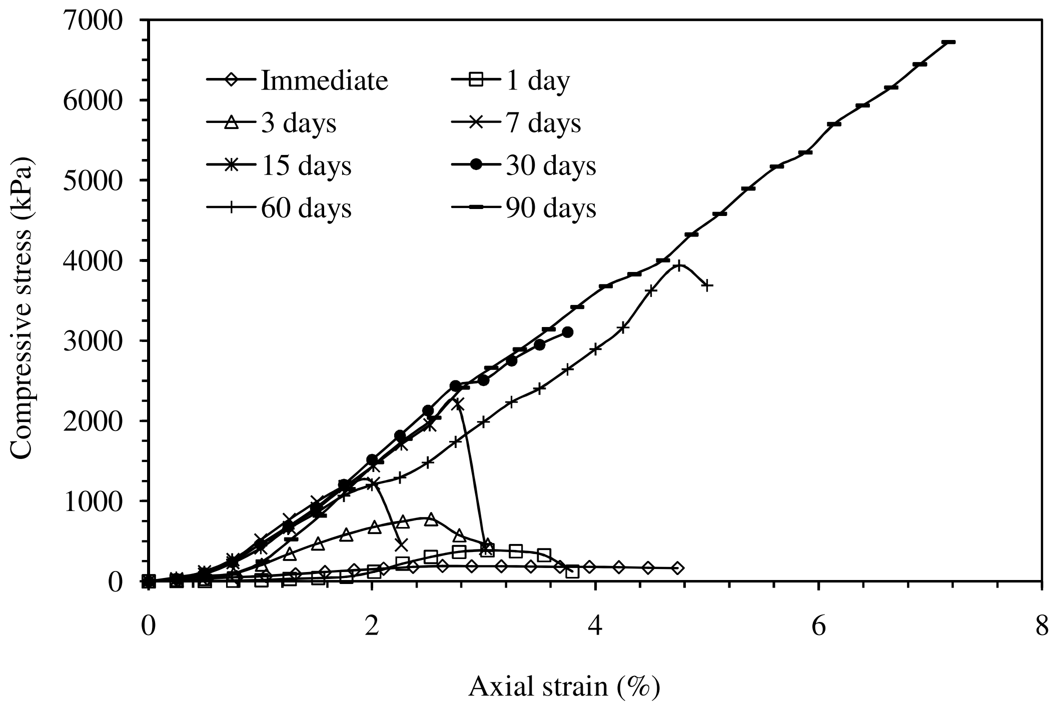


Fig. 8.24 Stress-strain responses of ES-FA mix F40 treated with 17% lime.

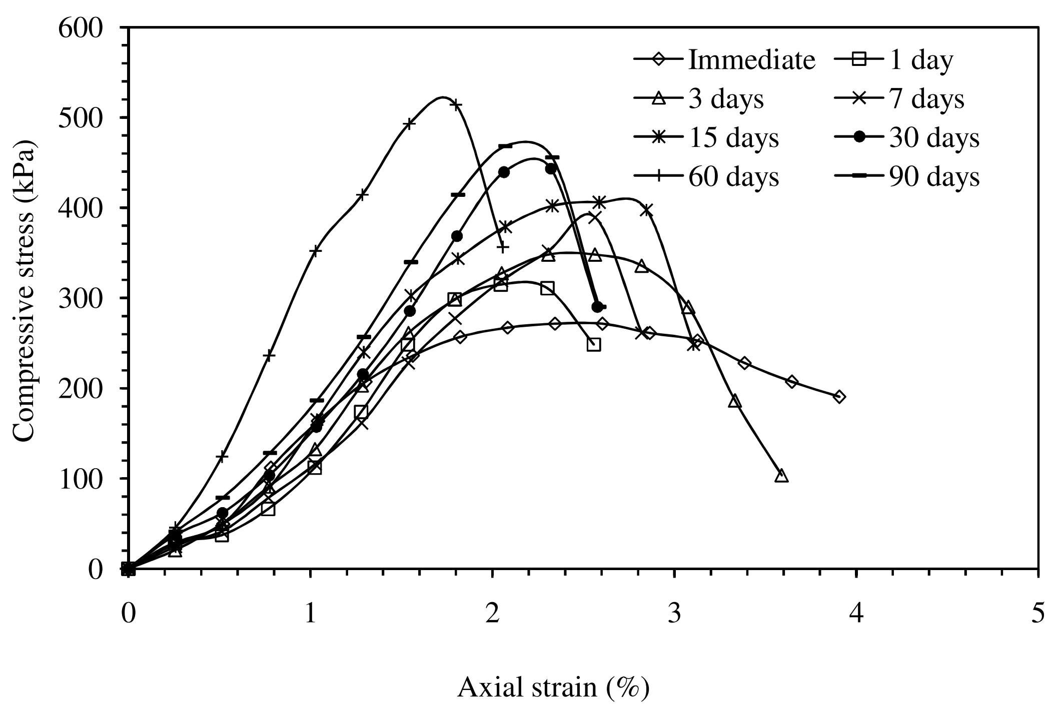


Fig. 8.25 Stress-strain responses of ES-FA mix F60 treated with 1% lime.

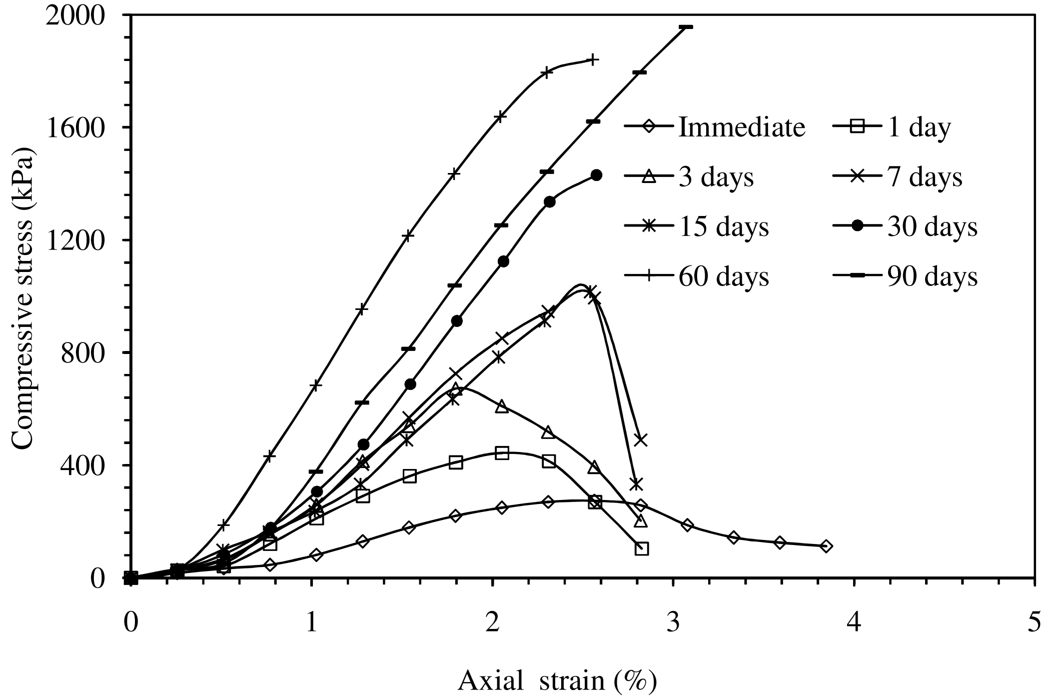


Fig. 8.26 Stress-strain responses of ES-FA mix F60 treated with 3% lime.

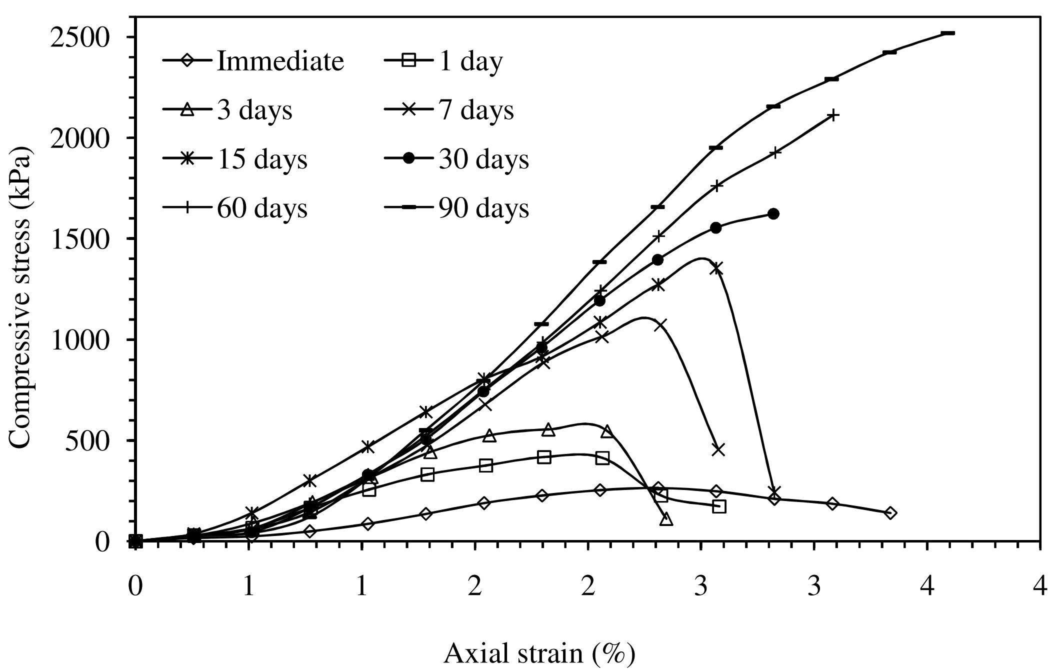


Fig. 8.27 Stress-strain responses of ES-FA mix F60 treated with 5% lime.

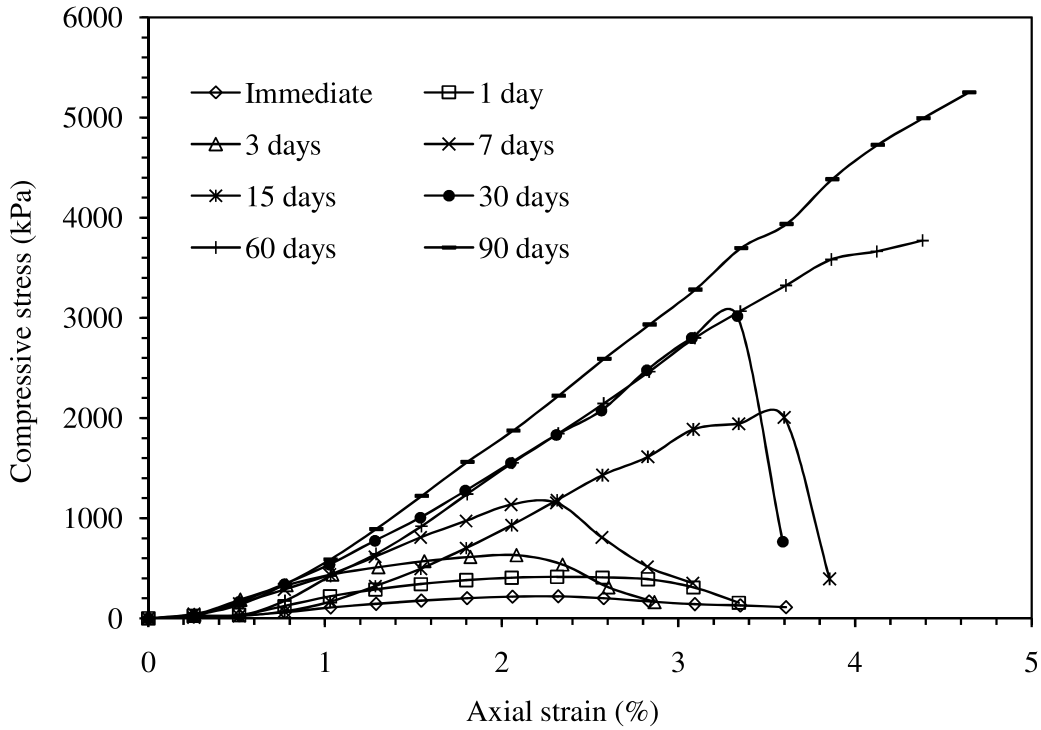


Fig. 8.28 Stress-strain responses of ES-FA mix F60 treated with 9% lime.

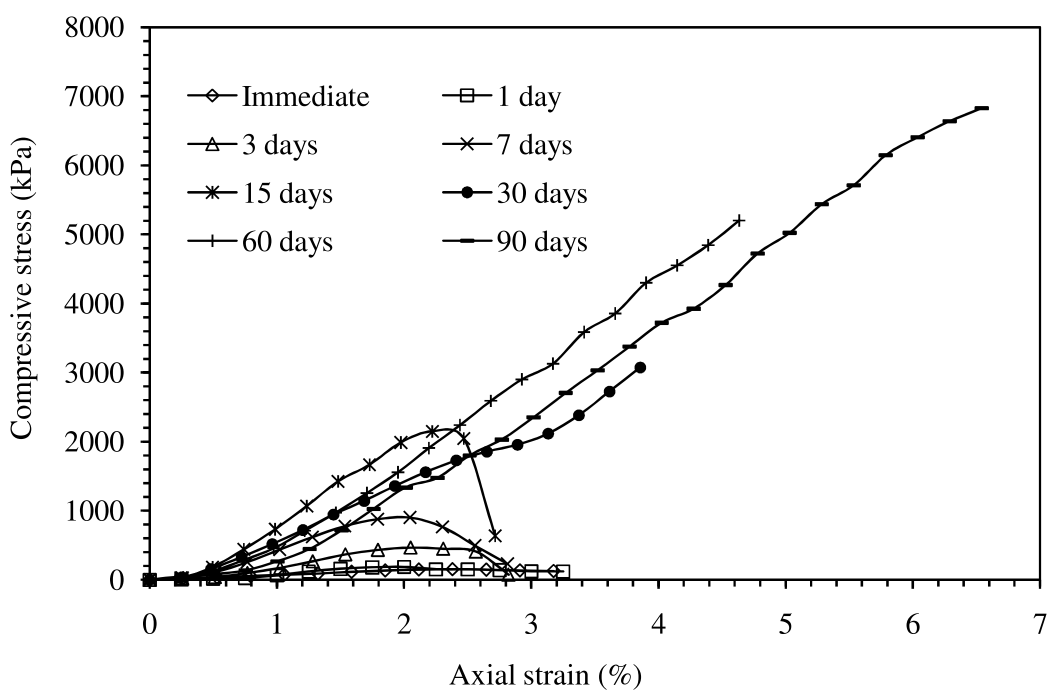


Fig. 8.29 Stress-strain responses of ES-FA mix F60 treated with 13% lime.

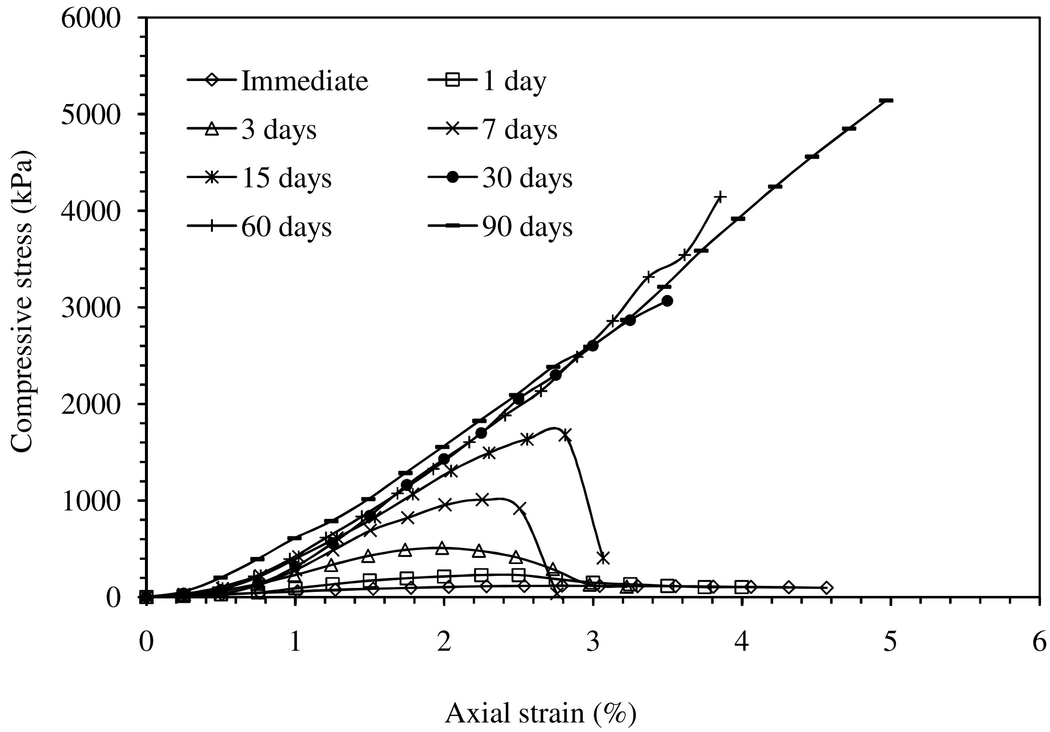


Fig. 8.30 Stress-strain responses of ES-FA mix F60 treated with 17% lime.

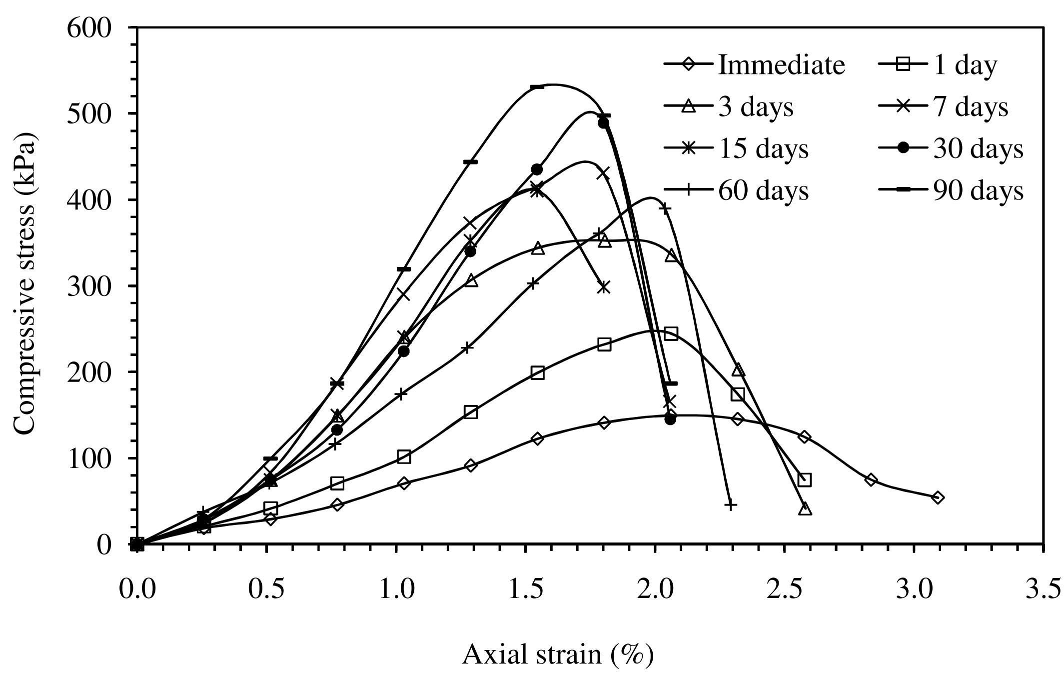


Fig. 8.31 Stress-strain responses of ES-FA mix F80 treated with 1% lime.

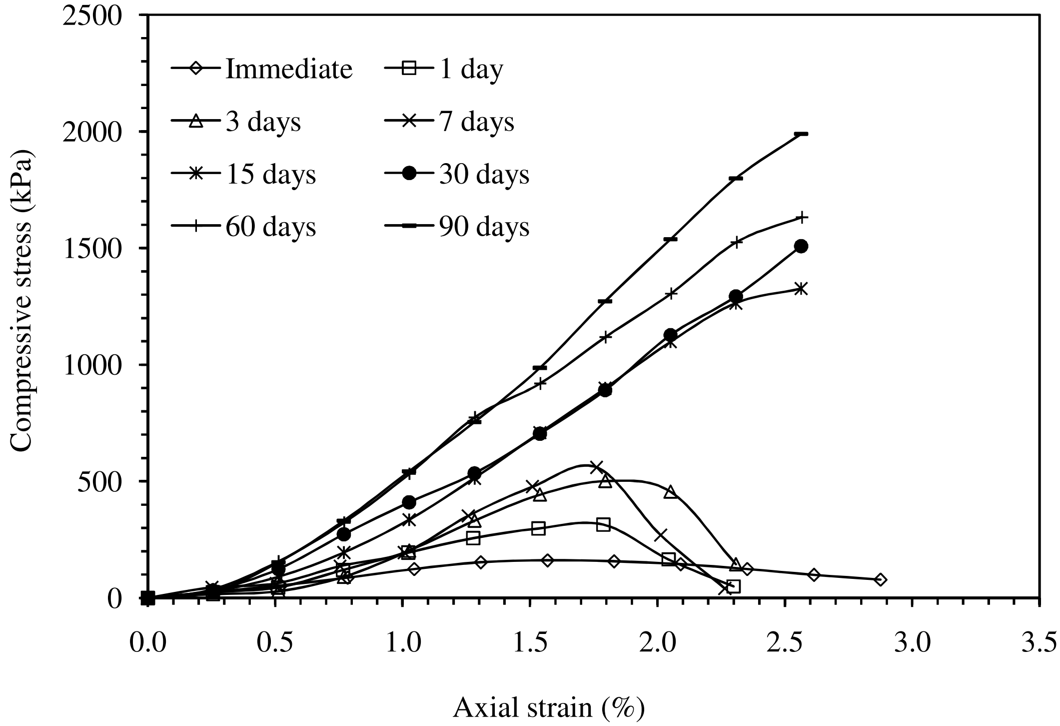


Fig. 8.32 Stress-strain responses of ES-FA mix F80 treated with 3% lime.

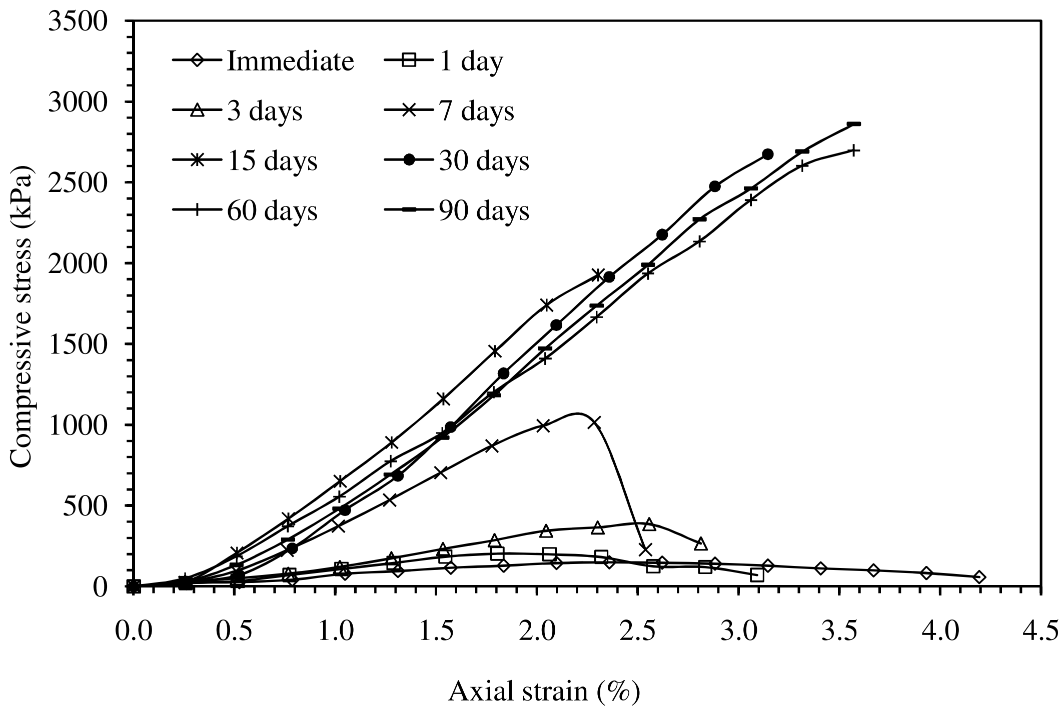


Fig. 8.33 Stress-strain responses of ES-FA mix F80 treated with 5% lime.

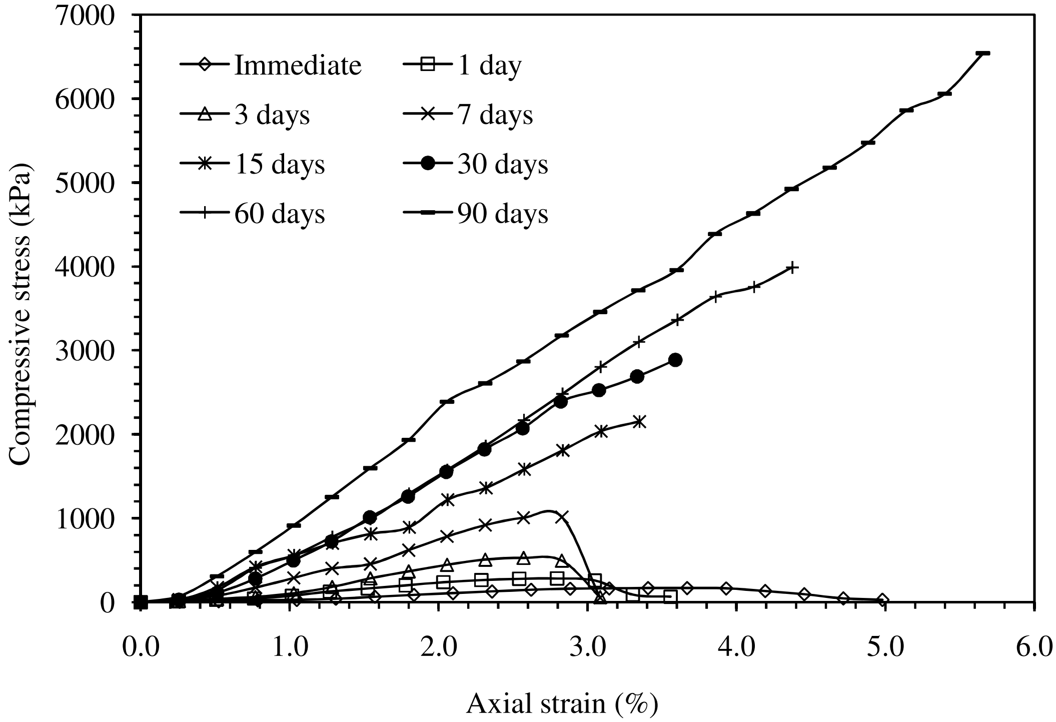


Fig. 8.34 Stress-strain responses of ES-FA mix F80 treated with 9% lime.

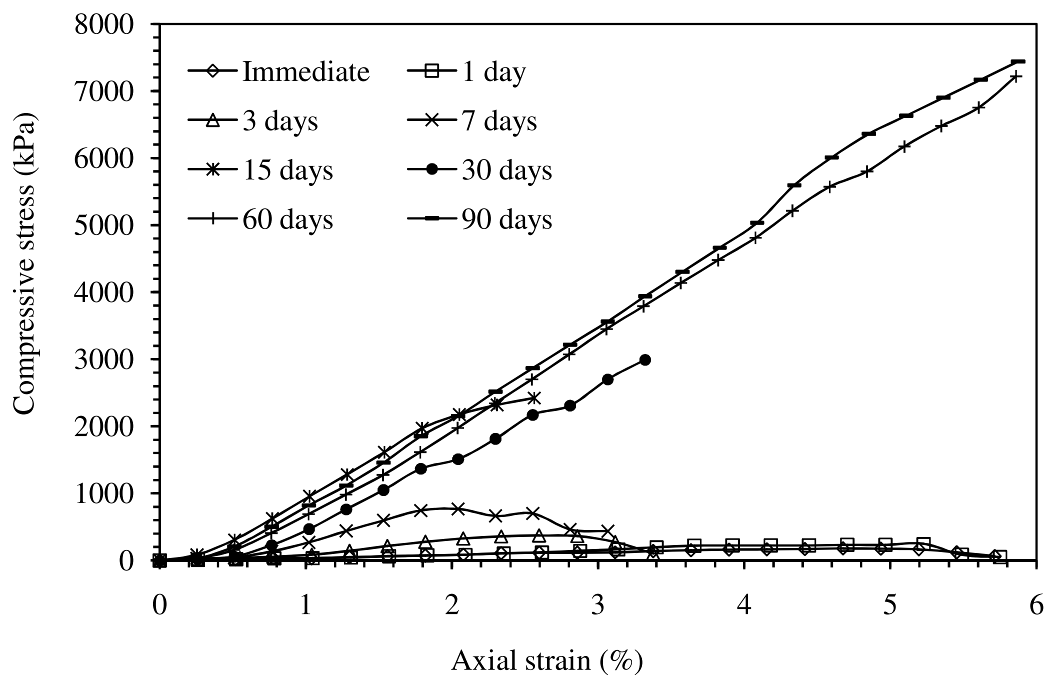


Fig. 8.35 Stress-strain responses of ES-FA mix F80 treated with 13% lime.

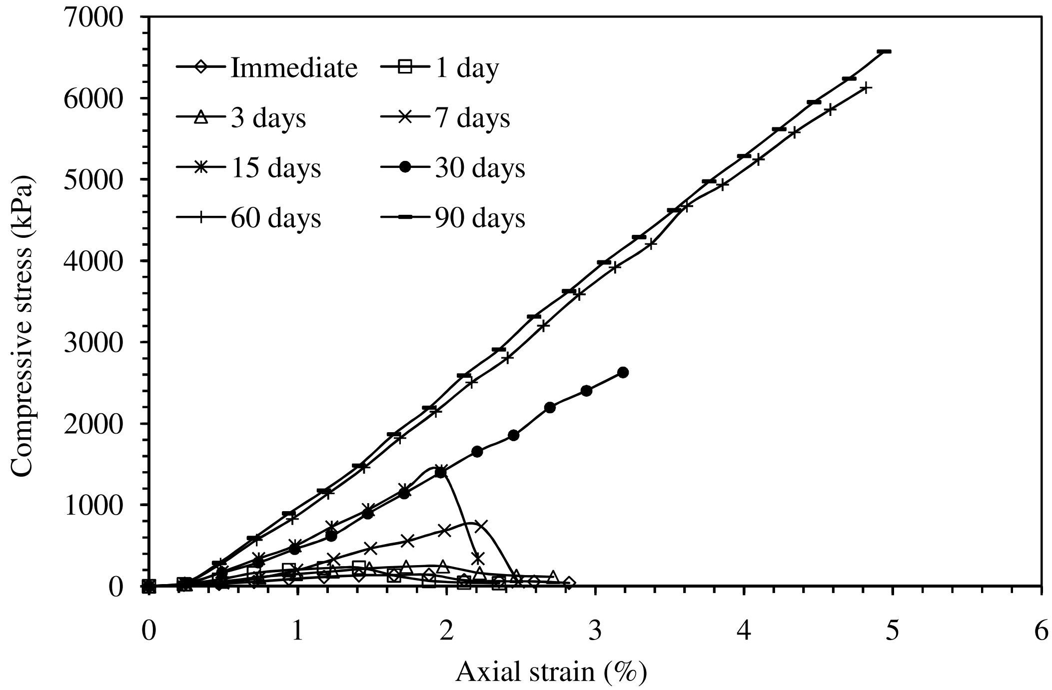


Fig. 8.36 Stress-strain responses of ES-FA mix F80 treated with 17% lime.

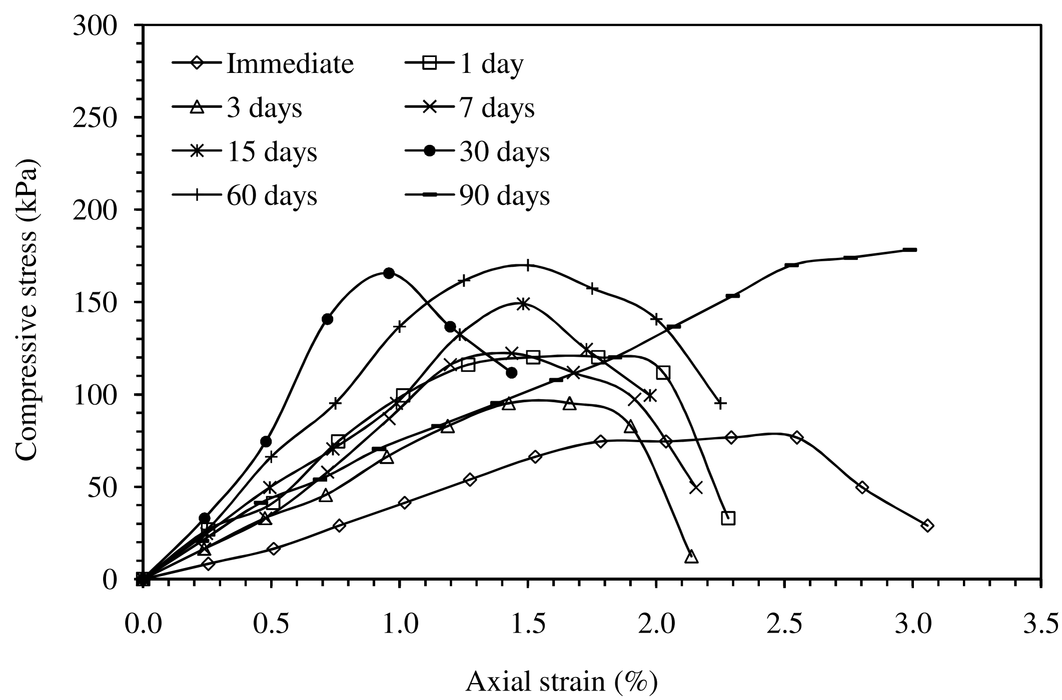


Fig. 8.37 Stress-strain responses of FA treated with 1% lime.

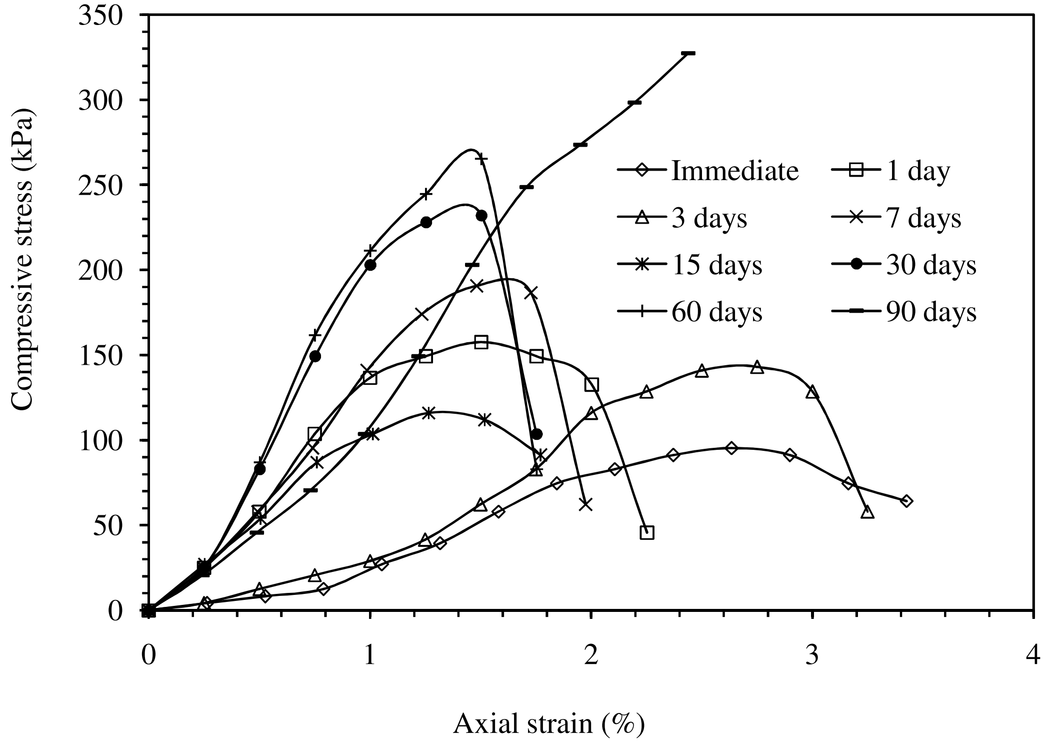


Fig. 8.38 Stress-strain responses of FA treated with 3% lime.

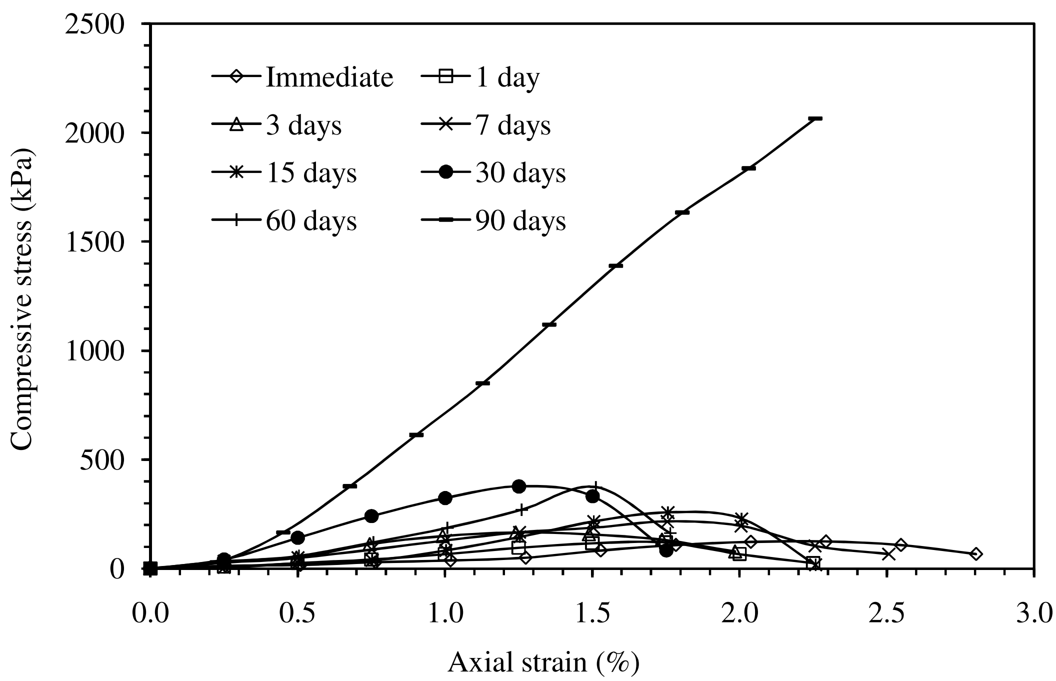


Fig. 8.39 Stress-strain responses of FA treated with 5% lime.

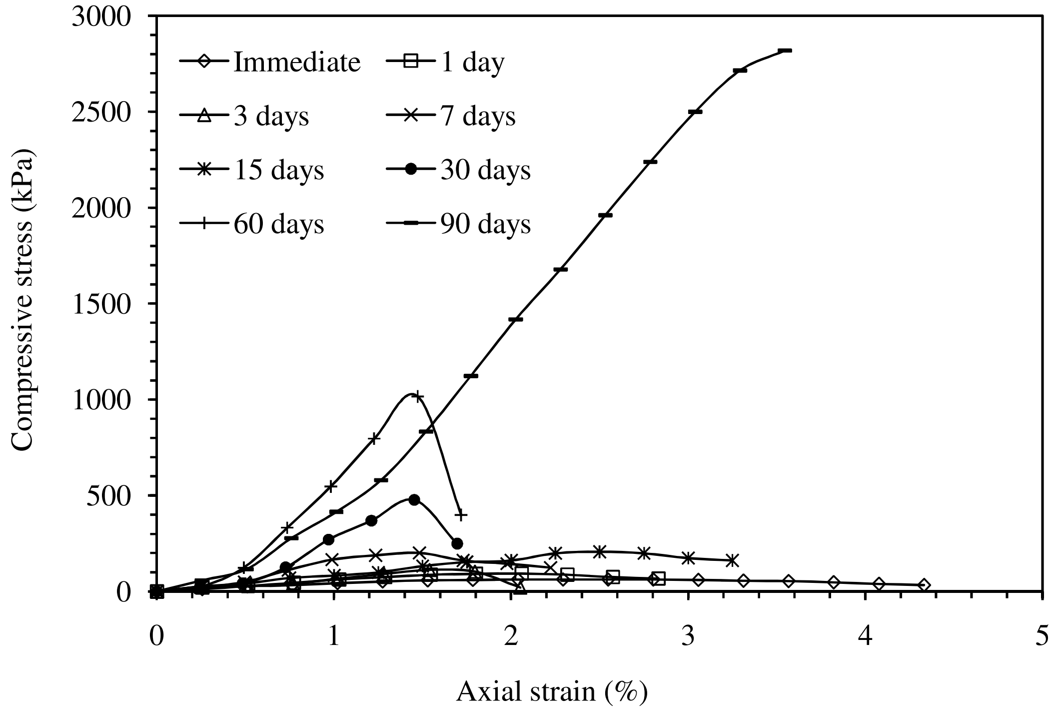


Fig. 8.40 Stress-strain responses of FA treated with 9% lime.

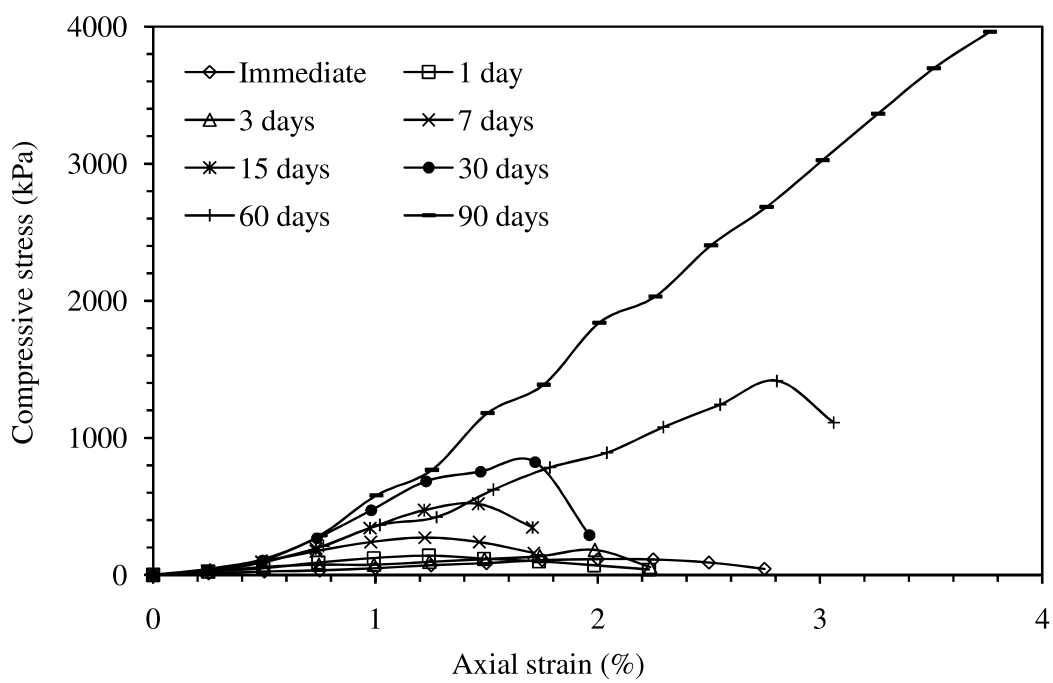


Fig. 8.41 Stress-strain responses of FA treated with 13% lime.

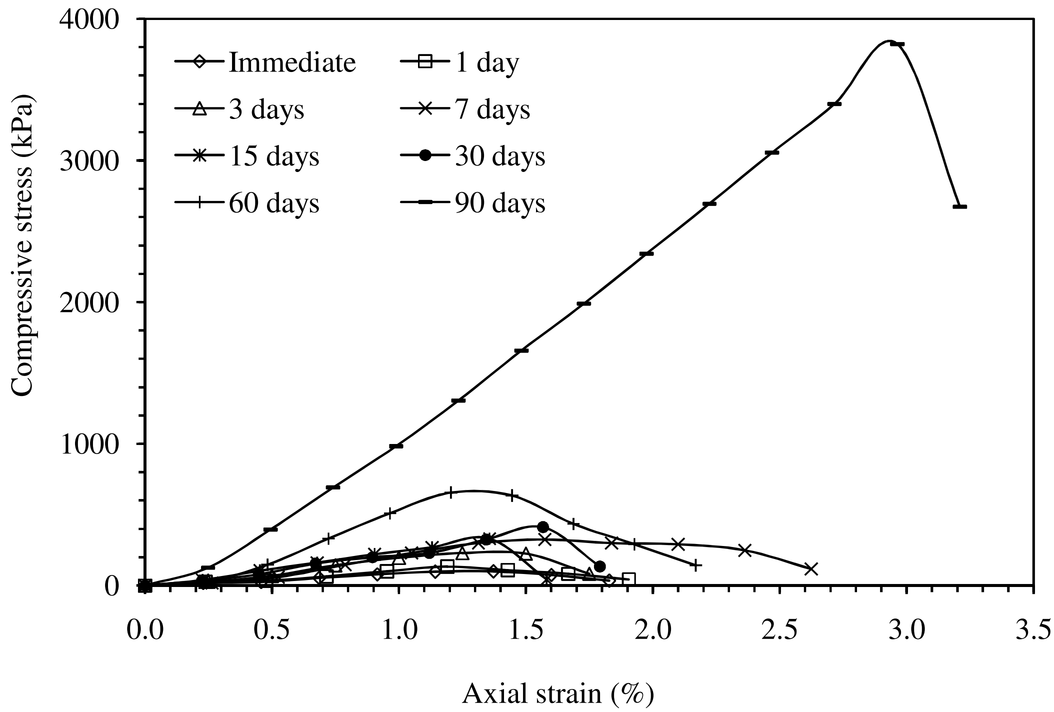


Fig. 8.42 Stress-strain responses of FA treated with 17% lime.

The variation of *UCS* with lime content for all the mixes, at different curing periods are presented in Fig. 8.43–8.47. It can be seen that, in general, with less curing time (i.e. ≤ 7 days), the strength improvement is marginal, even at very high content of lime. It is only after 15 days of curing, that visible increase in strength is observed. The *UCS* continues to increase with increase in lime content till about 13% beyond which further increase in lime content has reduced the strength. Hence it can be said that 13% lime content is the optimum amount that gives maximum increase in strength. The reduction in strength, at very high lime content (i.e. >13%), is attributed due to the formation of excessive C-S-H gel resulting from the high percentage of silica in the fly ash, leading to a relatively porous structure as has been explained earlier. The *UCS* for optimum content of lime (13%) for different mixes are depicted in Fig. 8.48. and 90 days curing, for the specimen with 100% ES, is 4550 kPa (Fig. 8.12). With 20% fly ash (F20, Fig.8.43) this strength has gone up to

6155 kPa. With 40% fly ash this is 6312 kPa (Fig. 8.44) and with 60% fly ash it is 6826 kPa (Fig. 8.45). When fly ash content is 80% (i.e. F80, Fig.8.46) the compressive strength has increased to the maximum i.e. 7440 kPa. With fly ash alone (i.e. FA, Fig. 8.47) the peak strength has reduced to 3875 kPa. Hence it can be said that 20% expansive soil with 80% fly ash is the best possible composition for maximum strength improvement through lime treatment. This finding opens up the possibility of utilizing the fly ash in large quantity for geotechnical applications such as building highway and railway embankments, filling up low lying areas, etc. specifically in the problematic expansive soil regions. Fly ash being pozzolanic in nature is a favourable material for cementation. Therefore, when mixed with in increased proportion with the expansive soil, in presence of lime, the overall strength increases, due to increased cementation effect. However, strength of lime amended fly ash alone is relatively less as compared to that of ES-FA mix, with 20% ES and 80% FA. This is because, a small fraction of expansive soil in the fly ash effectively fills in the voids within leading to a well compacted structure that sustains enhanced loading.

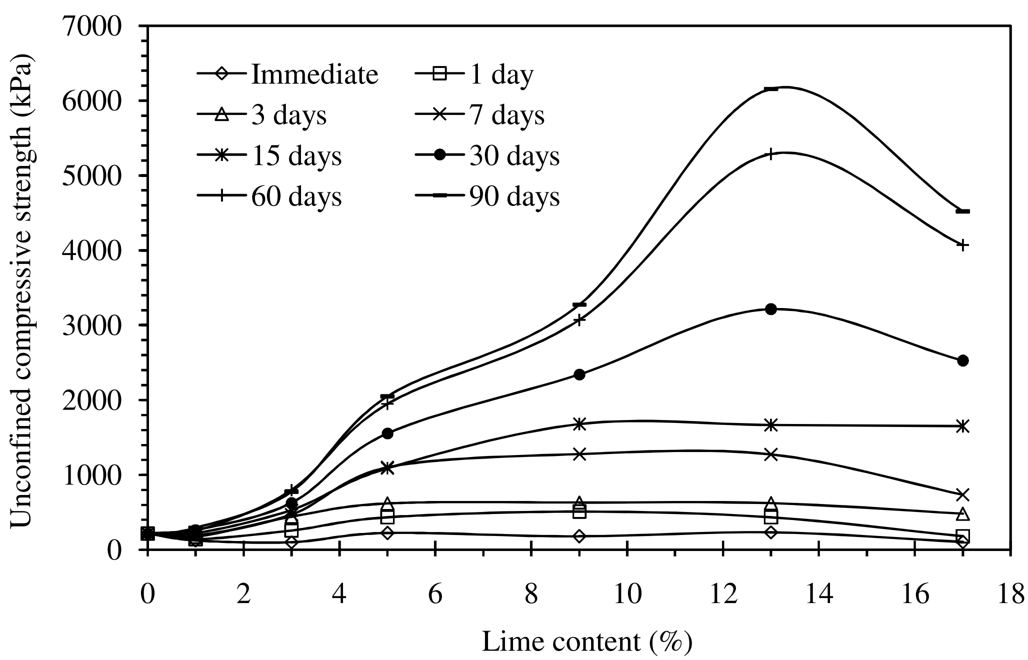


Fig.8.43 Unconfined compressive strength variations with lime content for F20.

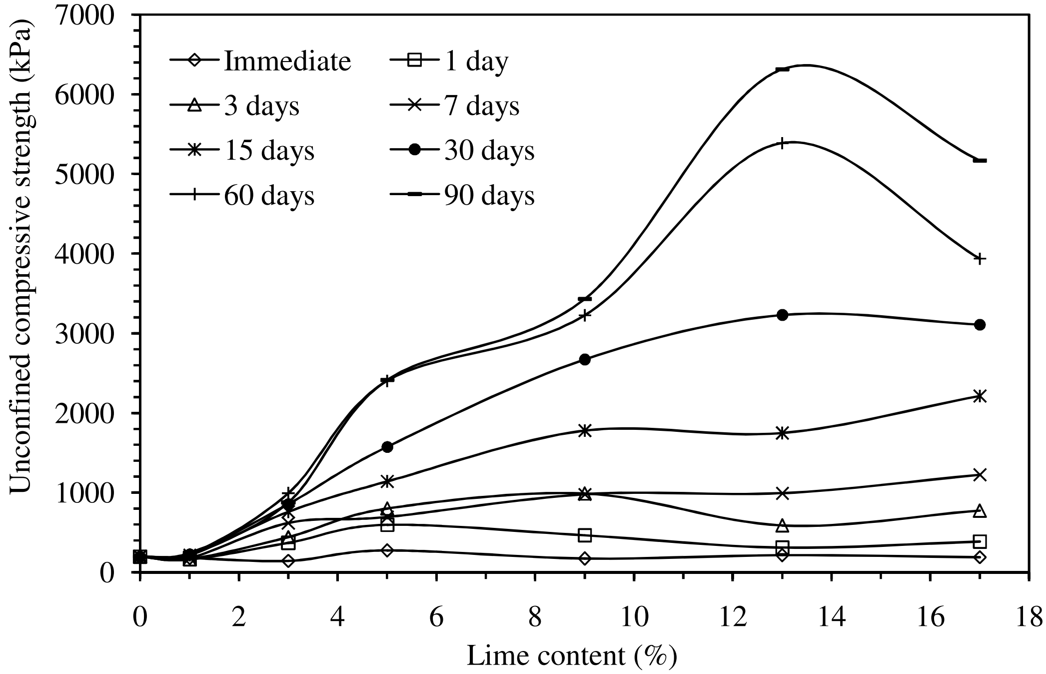


Fig.8.44 Unconfined compressive strength variations with lime content for F40.

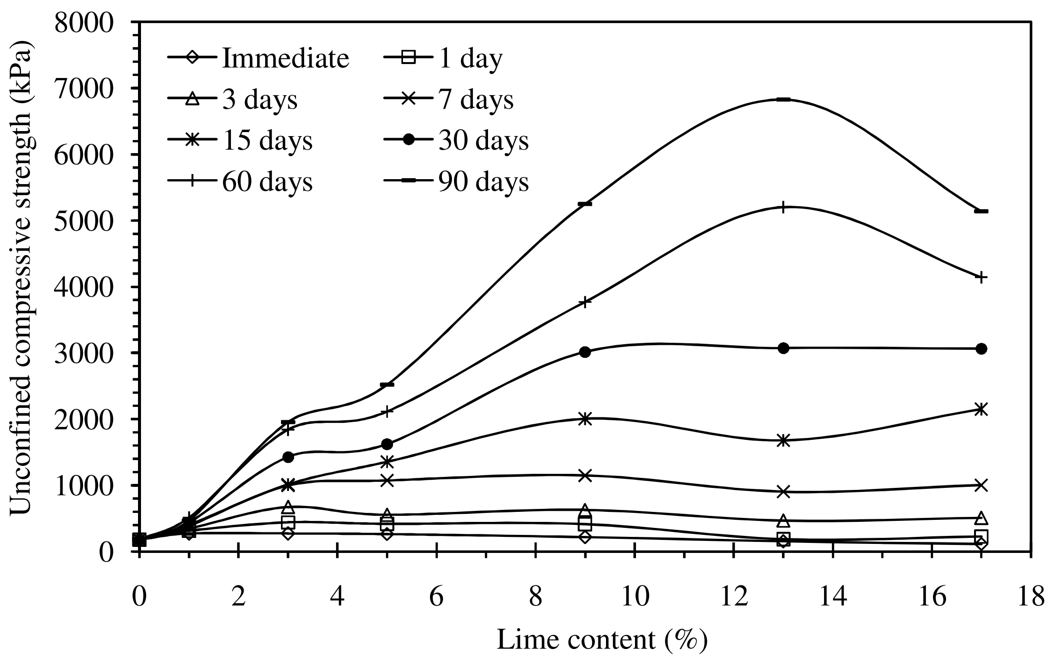


Fig.8.45 Unconfined compressive strength variations with lime content for F60.

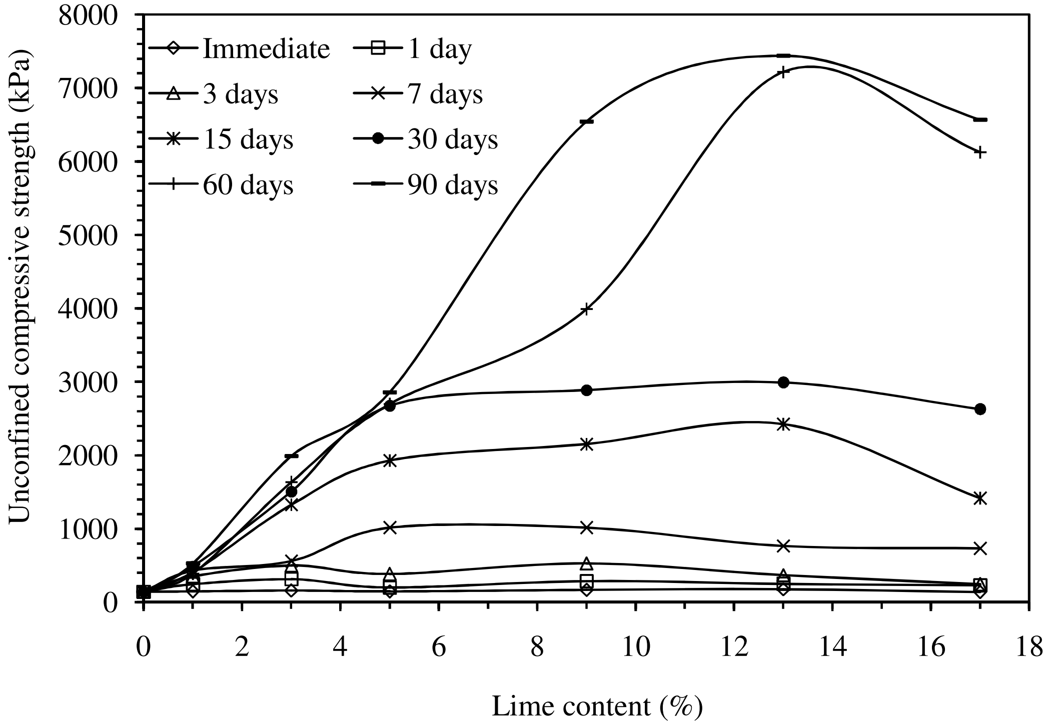


Fig.8.46 Unconfined compressive strength variations with lime content for F80.

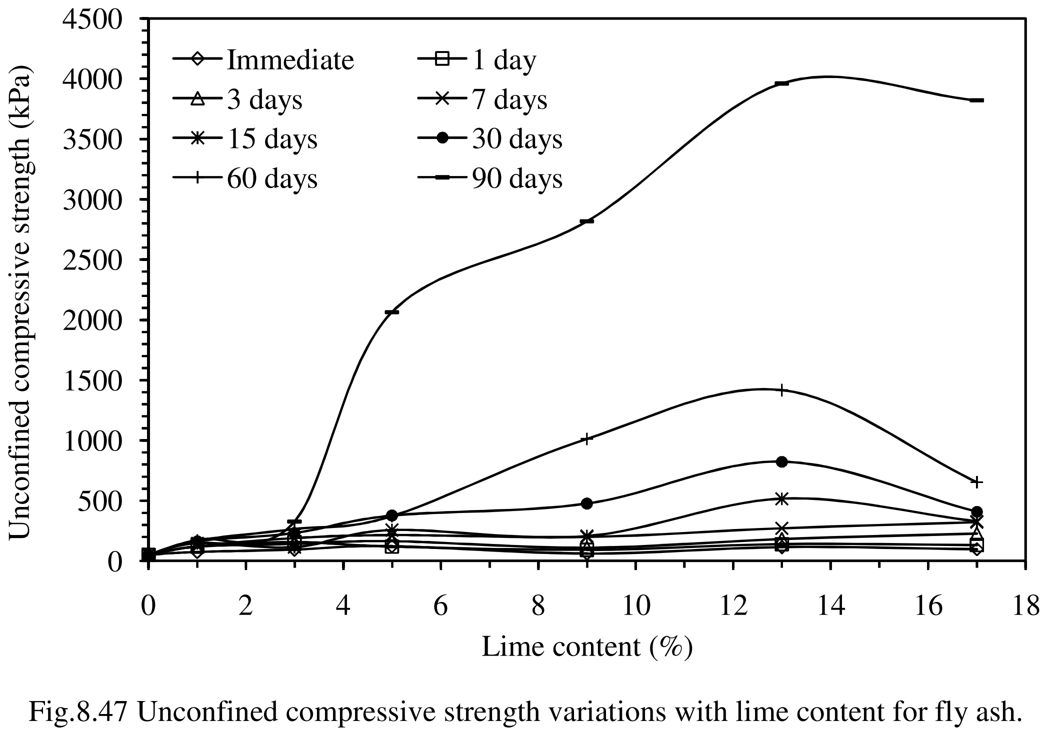


Fig.8.47 Unconfined compressive strength variations with lime content for fly ash.

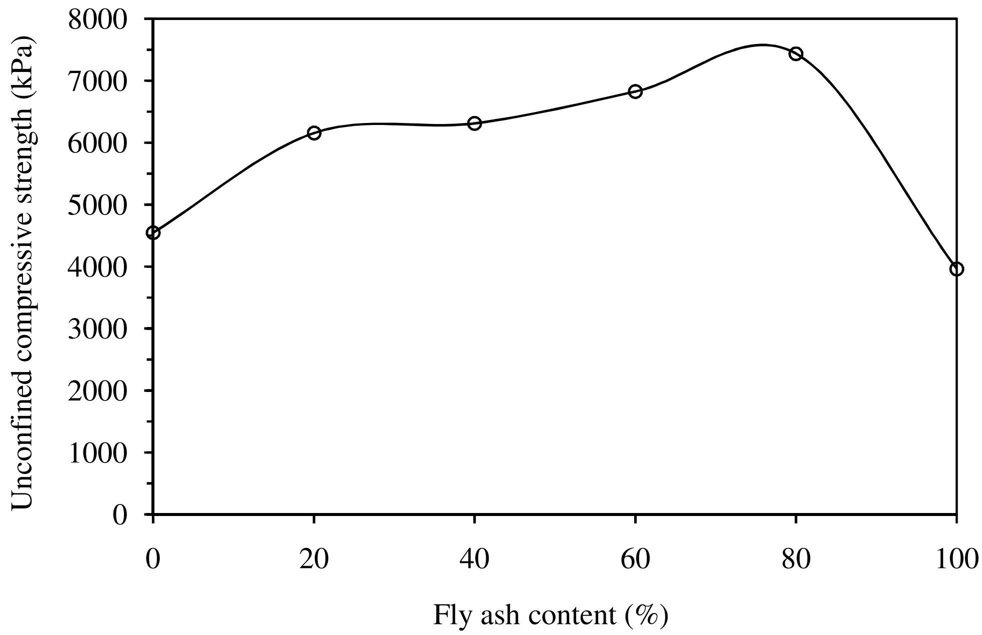


Fig.8.48 Unconfined compressive strength of ES-FA mixes corresponding to 13% lime and 90 days curing

8.4 SUMMARY

This chapter has brought out the influence of fly ash and lime on the strength improvement of expansive soil. A series of unconfined compression tests have been carried out by varying the fly ash content, percentage of lime added and the curing period. The test results indicate that there is an optimum percentage of lime and fly ash that gives maximum strength improvement. The *UCS* of ES is found to increase with fly ash addition. Improvement in strength is found to be more pronounced at higher percentage of lime and extended curing period. Strength improvement is marginal under 7 days of curing, even with high lime content. Treatment with 13% lime gives the maximum strength to ES-FA mixes. When lime content is > 13%, strength is found to decrease. As lime content and curing period increases, failure pattern changes from ductile to brittle. Out of the different ES-FA combinations, 20% expansive soil and 80% fly ash deliver the maximum strength on lime treatment.

CHAPTER 9

SUMMARY AND CONCLUSIONS

9.1 SUMMARY

Expansive soil deposits are found, spreading over vast stretches of land, in several countries across the world. These soils are problematic for engineering structures because of their volume change characteristics and the associated strength variations. It is reported that the damage due to expansive soils is much more than the damage caused by other natural disasters, including earthquakes and floods. Over the last fifty years, significant research has been performed to develop treatment methods for stabilizing the expansive soils. Among the various methods adopted, chemical stabilization of expansive soils is generally cost effective. However, in many cases stabilization through admixtures, such as fly ash and other non-swelling soils too has produced substantial performance improvement. Under the present study, combined use of fly ash and lime stabilization, has been attempted to improve the performance of expansive soils. Besides, a systematic study through careful variation of various parameters has been carried out, to develop an understanding of the various mechanisms involved.

Five different series of experiments under a systematically planned scheme, have been carried out, to study the physico-chemical and engineering behaviour of the expansive soil, treated with fly ash and lime. The details of these test series are presented in Chapter 3 (Table 3.2). In each test series expansive soil was mixed with fly ash at varying percentage (i.e. 0%, 20%, 40%, 60%, 80% and 100%). All these six expansive soil-fly ash mixes were added with different percent of lime (0, 1, 3, 5, 9, 13 and 17% by weight of dry soil). Each of the samples thus prepared were mixed with desired quantity of water and was subjected to different curing periods (0, 3, 7, 15 and 30 days) before the tests were carried out. Parameters studied are the plasticity characteristics, compaction behaviour, swell response, compressibility, and strength behaviour. Based on the obtained results, the following conclusions have been made.

9.2 CONCLUSIONS

1. An evaluation of available methods for determining the liquid limit of soils indicates that, there is an underestimation of the liquid limit obtained by cone penetration method as compared to that by percussion method. This discrepancy is more for high plastic soils. A modified methodology, based on free-swell index of soil, proposed for obtaining the cone penetration liquid limit is found to give comparable results as that from the percussion method.

2. Both the liquid limit and plastic limit values reduce with addition of fly ash to the expansive soil. While, with addition of lime the liquid limit of soils has undergone further reduction, the plastic limit has undergone a visible increase. With 10-12% of lime one can obtain similar reduction in liquid limit as that by adding 50-60% of fly ash to the expansive soil.

3. Overall the plasticity index has shown a sharp reduction till lime content of about 2 to 3%, beyond which further reduction is marginal. Therefore, the critical amount of lime, to provide the maximum increase in workability of soils (i.e. expansive soil, expansive soil- fly ash mixes), is about 3% by weight of dry soil.

4. At higher lime content and prolonged curing periods, the liquid limit of the expansive soil exhibit an increasing trend, which is more prominent in case of expansive soil-fly ash mixes.

5. With addition of fly ash and lime the compaction response of the expansive soil tends to be flatter. This indicates that the soil has turned more friable, that the target density can be achieved over a wider range of moisture content leading to improved workability.

6. Among different index properties of the expansive soil–fly ash–lime mixes, plastic limit is found to correlate well with the compaction parameters i.e. optimum moisture content and maximum dry density. The correlation equations for lime treated soils are observed to be different from those reported for natural soils. This is attributed to the lime induced chemical alterations of the soil.

7. When added with fly ash, the compressibility of the expansive soil is found to have reduced substantially. With 80% fly ash, the reduction in c_c is as high as 78% and that of m_v is 68%.

8. Lime has further reduced the compressibility of the expansive soil-fly ash mixes. However, there is no significant reduction in c_c and m_v, for lime content beyond 5%. Therefore, for all practical purpose, the maximum lime modification of the fly ash amended expansive soils can be limited to 5%.

9. The method proposed by Dakshanamurthy (1978) for estimating maximum swell of soils, is found to hold good for expansive soil-fly ash mixes as well. The maximum swell percentage is found to exhibit a linear relationship with the plastic limit. The swell pressure is found to vary linearly with liquid limit, plastic limit, plasticity index, and specific surface area.

10. The present study demonstrates the use of swell pressure as an excellent indicator for delineating modification or modification + solidification reaction taking place in lime amended expansive soil or expansive soil-fly ash mix.

11. The strength of expansive soil-fly ash mixes is found to increase with increase in lime content till about 13%, beyond which, further increase in lime content has reduced the strength. This is attributed to the formation of excessive amount of calcium silicate hydrate gels and the associated gel pores. Hence it can be said that 13% lime is the optimum one giving maximum strength improvement.

12. Compared to fly ash alone, the soil-fly ash mix is a better material for lime treatment. 20% expansive soil with 80% fly ash is the best possible composition that gives maximum strength improvement on lime treatment. This finding opens up the possibility of utilising the fly ash in large quantity for the geotechnical applications such as construction highway and railway embankments, filling up low lying areas etc., especially in the problematic expansive soil regions.

9.3 SCOPE FOR FURTHER STUDIES

- Extensive chemical characterization such as CEC, SEM, XRD, of the mixes subjected to different curing period needs to be performed for better understanding of the chemical reactions taking place.
- The flow and contaminant retention of these mixes need to be evaluated for its use in geoenvironmental projects.
- The influence of lime addition on rate of settlement need to be studied for compacted expansive soil.
- Influence of submergence on the strength of lime-fly ash treated expansive soil need to be studied.

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