A STUDY ON PERFORMANCE IMPROVEMENT OF EXPANSIVE SOIL USING RESIDUAL SOIL AND LIME

A THESIS

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of

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CERTIFICATE

This is to certify that the thesis entitled **"A Study on Performance Improvement of Expansive Soil Using Residual Soil and Lime"** submitted by **Md. Monowar Hussain** Roll No. 05610404 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 my supervision and guidance. The thesis work, in my 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

Keywords: expansive soil, stabilization, residual soil, lime, plasticity, compaction, swell-shrink, strength, micro structure.

The soils which show volumetric changes due to changes in their moisture content are referred to as *expansive soils*. With increase in moisture content, these soils swell and with decrease in moisture content undergo shrink. This leads to seasonal movements causing heave in rainy season and subsidence in summer, giving rise to high differential settlement in the structures founded on them thereby causing distress and damage to it. Stabilization of expansive soils with additives has been used with great success. In the recent past several investigations have been reported highlighting the beneficial use of lime for performance improvement of clay soils (Davidson and Handy, 1959; Thompson, 1966; Bhasin, 1978; Bell, 1988; Sivapullaiah et al., 1998, 2000; to name a few). In spite of these studies there is still need of further investigation to understand the mechanism of lime induced modifications of soils more clearly.

From literature review it is observed that lime generally improves the performance of clayey soils in terms of reduced swelling and increased strength. However, in some cases, depending on the type of soil, amount of lime added, curing period etc. the performance improvement reduces. In view of this an attempt is made to carry out a systematic study through careful variation of above parameters, to develop an understanding of the mechanisms involved.

Primarily an expansive soil (ES) and a residual soil (RS) that represent the extreme types of soil are used in the present study. The non swelling and high strength residual soil was added to the expansive soil to improve its performance. Besides to cover a wide range of plasticity these two soils are mixed in different proportion (i.e. 100%ES, 80%ES +

20%RS, 60%ES + 40%RS, 40%ES + 60%RS, 20%ES + 80% RS, 100%RS) to prepare six different soil samples. Subsequenty these soils were treated with lime of varied quantity under varied curing period.

The test results indicate that the liquid limit continues to reduce till 3% lime content, beyond which the increased lime content has marginal effect on it. However, at very high lime content (i.e. 13%) and long curing period (i.e. 28 days) the liquid limit of the expansive soil has shown an increasing trend. This increasing trend gradually grows more prominent with increased percentage of residual soil and curing period. It is of interest to note that irrespective of soil type the liquid limit is large for increased curing period.

The plastic limit increases with increases in lime content. The increase is relatively faster till lime content reaching about 3%. From 3 to 5% of lime content the rate of increase in plastic limit is relatively slow. Beyond 5% lime content visible increase in plastic limit is noticed only for increased percentage of residual soil and higher curing period.

In general soils have shown an immediate decrease in plasticity index upon addition of lime. For 100%ES, increasing the lime content beyond 5% had a marginal effect in further reducing the plasticity index. This is in line with earlier observations that immediately upon addition of lime the liquid limit reduces and plastic limit increases. In general the plasticity index increases with increase in curing period however it is more prominent for the increased percentage of residual soil. This is attributed to the silica gel that enhances the water holding capacity of the soil.

For all soils, as the percentage of lime increases the maximum dry density (MDD) reduces. This continues till lime content reaches 3%, beyond which the MDD continues to increase, with increase in lime content, though at a relatively smaller rate. Correspondingly the optimum moisture content (OMC) increases till 3% of lime for expansive soil and expansive soil-residual soil mixtures. Beyond this the OMC decreases with increase in lime content. However, incase of residual soil the maximum OMC is observed at 5% of lime, though the difference between the values for 3% and 5% lime content is marginal. Based on the present study it can be said that, relatively higher quantity of lime (i.e. more than 3%) should be added to soils to obtain better compaction density.

It is observed that for the expansive soil (100%ES), even with addition of 1% lime the swelling has been substantially reduced. With further increase in lime content swelling continues to reduce to reach a practically negligible value. The corresponding log time versus percent swell responses shows that the slope of the responses has undergone reduction with increased percent of lime. This indicates that the rate of swelling too reduces with increase in lime content. The reduction in swelling and rate of swelling with lime is attributed to the increased electrolytic concentration giving rise to reduced thickness of the diffuse double layer. However it is of interest to note that after this initial reduction, the swelling of soils has once again increased with further addition of lime. This phenomenon has manifested in greater prominence in case of soils having higher percentage of residual soil. This is attributed to fabric change of the soil. At higher lime content the pH value increases, that forms flocculated in structure in soil. As the flocculated structure holds more water it induces higher swelling.

Initially the shrinkage limit has increased sharply till about 5% lime content beyond which the rate of increase has reduced with the reduction being more at less curing period. The initial increase is attributed to aggregation of particles due to addition of lime. With increased electrolyte concentration, in the pore fluid, the diffuse double layer thickness reduces and thereby the repulsion between the clay particles reduces. As a result of which the particles come closer and from aggregated clusters. These aggregated clusters offer increased resistance against capillary suction induced volumetric shrinkage leading to increased shrinkage void ratio and thereby the water content (i.e. shrinkage limit).

The volumetric shrinkage responses indicate that similar to the untreated soil the lime treated soil too shows three stage shrinkage (i.e. initial, primary and residual). Among these three stages, maximum shrinkage has taken place under primary shrinkage, indicating that the general mechanism of shrinkage too prevails in case of lime treated soils. Also, it is observed that, in general, shrinkage magnitude has reduced with increase in lime content. This is attributed to the combined effect of aggregation, flocculation and cementation that has taken place due to lime addition.

At 9% lime content and 28 days curing period the strength of the expansive soil has gone upto 3000 kPa against 250 kPa for untreated soil indicating a 12 fold increase. With adequate percent of lime and prolonged curing period there is increased formation of the cementitious compounds leading to increased strength. It is of interest to note that beyond 9% lime, there takes place reduction in strength with further increase in lime content. This effect is more pronounced at increased curing period. It has been suggested by earlier researchers that since lime itself has neither appreciable friction nor cohesion (Bell, 1996) an excess amount of lime serves as a lubricant to the soil particles and thereby reduces strength. However under the present study it has been brought out that at higher lime content the liquid limit increases due to formation of cementitious gels having substantial volume of pores that hold the water within. Hence it can be said that with higher lime content the soil structure tends to be more porous that counteracts the strength gain due to cementation. At very high lime content due to formation of large quantity of gel material there occurs an overall decrease in strength.

The X-ray diffraction analysis shows the formation of cementitious compounds such as Gyrolite [$2Ca.3SiO_2.H_2O$], Calcium Silicate Hydrate (CSH) [$5Ca_2SiO_4.6H_2O$], Calcium Aluminum Silicate Hydroxide Hydrate (CASHH) [$Ca_5Si_5Al(OH)O_{17}.5H_2O$] etc. The scanning electron micrograph indicate increase in size of the soil clusters. This is attributed to flocculation and aggregation of clay particles caused due to lime treatment. The white patches, observed, are the cementitious gels formed due to lime induced pozzolanic reactions. The cementitious gels apart from bonding the clay particles together, fill in the voids between the aggregated particles leading to a denser fabric. These microchanges are believed to have enhanced the performance of the expansive soil in terms of enhanced strength and reduced swell-shrink etc.

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ABBREVIATIONS

ASTM	American Society for Testing and Materials
BET	Brunauer, Emmet and Teller
CEC	Cation exchange capacity
CH	Clay with high plasticity
CI	Clay with medium compressibility
CL	Clay of low compressibility
CSH	Calcium silicate hydrate
CAOH	Calcium aluminum hydroxide
CASH	Calcium aluminum silicate hydrate
CASHH	Calcium aluminum silicate hydroxide hydrate
EDX	Energy Dispersive X-ray
ES	Expansive soil
FSI	Free swell index
ICL	Initial consumption of lime
IS	Indian Standard
LL	Liquid limit
MDD	Maximum dry density
MH	Silt of high plasticity
ML	Silt of low compressibility
OMC	optimum moisture content
PI	Plasticity index
PL	Plastic limit
SL	Shrinkage limit
RS	Residual soil

SEM	Scanning Electron Microscope
SSA	Specific surface area
UCS	Unconfined Compressive Strength
XRD	X-ray diffraction

NOTATION

English Symbols

A_s	surface area
a	intercept
a_m	molecular projected area of the adsorbate, N_2
b	slope of the straight line
C	BET constant
e	void ratio at any stage of shrinkage
e_0	void ratio of swollen soil
e_s	void ratio at maximum swell
e_{is}	void ratio at the end of initial shrinkage
e_{ps}	void ratio at the end of primary shrinkage
e_{rs}	void ratio at the end of residual shrinkage
G	specific gravity of soil
h_i	initial thickness of the sample
N_A	Avogradros number
P	applied pressure
P_0	saturation vapor pressure of N_2
P_s	swell pressure
R^2	correlation coefficient
S	percent swell
S_I	initial shrinkage
S_P	primary shrinkage
S_p	swell potential
S_R	residual shrinkage
t	time
V_{ads}	volume of N_2 adsorbed
V_d	equilibrium sediment volume of the soil
V_m	volume of monolayer coverage
V_0	volume of swollen soil specimen
w_0	water content of the swollen soil specimen
Δw	reduction in weight of the soil specimen due to drying

w	water content of the soil specimen at any stage of drying
W	total dry weight of soil
W_{ES}	weight of expansive soil
W_l	weight of lime
W_{RS}	weight of residual soil

Greek Symbols

γ_w	unit weight of water
γ_d	dry density of soil specimens at any stage of drying (i.e. ratio of weight of soil mass, w_s, which is a constant to the volume of specimen at that stage).
γ_{dmax}	maximum dry density
γ_{dPL}	density of the soil at the plastic limit water content
Δe	reduction in void ratio of the soil specimen due to shrinkage
Δh	increase in specimen thickness at a given time
ΔV	reduction in volume of soil specimen due to shrinkage

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

The soils which show volumetric changes due to changes in their moisture content are referred to as swelling soils. With increase in moisture content, these soils swell and with decrease in moisture content undergo shrinkage. This leads to seasonal movements causing heave in rainy season and subsidence in summer, giving rise to high differential settlement in the structures founded on them thereby causing distress and damage to it. Movement of foundations, upheaval of floors and pavements, breakage of slab on grade members, channel and reservoir linings, irrigation systems; and bursting of pipes are some of the problems commonly encountered in constructions on these soils. Fig. 1.1 depicts distress and damage in a typical building, founded on swelling soil deposit, located in northern Oman (Al-Rawas et al. 2005). Fig. 1.2 shows the distress in a highway pavement due to expansive soil underneath.

Fig. 1.1 Structural damage due to ground heave (Al-Rawas et al. 2005)

Fig. 1.2 Damage in pavement founded on expansive soil.

Natural expansive soils have been found in several countries across the world i.e. India, Africa, Australia, Israel, USA, Arab, Oman, Jordan, Ethiopia etc. In India these soils are generally refereed to as *Black cotton soil.* It covers an area of more than 2, 00,000 square miles which is about 20% of the total land area spreading over state of Maharastra, Madhya Pradesh, Gujarat, Orissa, Andra Pradesh, Karnataka and Tamilnadu (Subba Rao et al. 1985). The annual cost of damage to civil engineering structures is estimated at 150 million pounds in the U.K.; 1,000 million dollars in U.S. and many billions of dollars world wide (Gourly et al. 1993). 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 (Jones and Holtz 1973).

Various innovative techniques such as special foundations that include belled piers, drilled piers, friction piers and moisture barriers have been developed to mitigate the problems posed by expansive soils (e.g. Chen 1975). Besides, an altogether a different technique, stabilization of expansive soils with additives has been used with great success. Soil stabilization is the modification of soils to enhance its properties to meet specific engineering requirements, such as reduction of swell and shrink to the permissible limit, modify plasticity and improve workability; improvement of strength etc. Over the last fifty years, significant research has been performed to develop various treatment methods to stabilize the expansive soils. Based on the mechanisms involved, stabilization methods can be either physical, mechanical, or chemical (Mitchell and Katti 1981; Hausmann 1990). Among these, chemical stabilization of expansive clays, being generally cost effective, is widely used by geotechnical engineers.

Raju (1991) studied the efficacy of various chemicals (i.e. NaCl, KOH, KCl, $Ca(OH)_2$, $CaCl_2$, $Ba(OH)_2$, $BaCl_2$, MgO, $MgCl_2$) for stabilization of expansive soils. The test results indicate that among the different chemicals studied, $Ca(OH)_2$ (i.e. lime) gives maximum performance improvement.

In the recent past several investigations have been reported highlighting the beneficial use of lime for performance improvement of clay soils (Davidson and Handy, 1959; Davidson, Mateos and Barnes, 1960; Thompson, 1964, 1966, 1969; Bhasin, 1978; Bell, 1988; Cobbe, 1988; Sivapullaiah et. al., 1998, 2000; to name a few). Petry and Little (2002) has brought out a detailed review on lime stabilization of clays with specific reference to pavements and light loaded structures.

In spite of these studies there is still need of further investigation to understand the mechanism of lime induced modifications of soils more clearly. Besides it is important to look for new methods to bring out still higher performance improvement of clay soils through combined use of different techniques such as, mechanical stabilization and lime stabilization. To achieve these objectives a series of investigations have been carried out under the present study.

1.2 ORGANISATION OF THE THESIS

In squeal to the introductory chapter (i.e. **Chapter 1**), a detailed review of literature on stabilization of clay soils through addition of lime and some selected additives has been presented, in **chapter 2.** Subsequently, the scope of the present investigation has been brought out.

Chapter 3 presents the details of materials used, planning of experiments and the test procedures.

Chapter 4-8 present and discuss the results obtained. **Chapter 4** deals with the plasticity characteristics of the lime treated soils. **Chapter 5** presents the compaction characteristics. **Chapter 6** deals with swell-shrink behaviour. **Chapter 7** presents the strength characteristics of the lime treated soils. In **Chapter 8** the microstructural changes have been studied. **Chapter 9** presents the conclusions drawn from the present study.

CHAPTER 2

LITERATURE REVIEW AND SCOPE OF PRESENT STUDY

2.1 INTRODUCTION

When the soil at any particular site is found unsuitable that it can not provide stability and serviceability to the proposed structure, the potential remedial measures the designer may consider are, (a) avoiding the site (b) alter the foundation design (c) remove the insitu soil and replace with a high strength non swelling free draining granular material (d) modify the existing soil through ground improvement techniques so as to create a new material capable of meeting the desired requirements.

Although all the above methods have been tried but in case of expansive soil choice (c) and (d) are widely adopted by the practicing engineers. Choice (d) i.e. to improve the soil through ground improvement technique is popularly know as soil stabilization.

Soil can be stabilized by various methods such as mechanical stabilization, chemical stabilization, stabilization with additives, thermal stabilization and electrical stabilization etc. The choice of a particular method depends on the type of soil to be stabilized, the properties to be improved upon (i.e. shear strength, compressibility, swelling, shrinkage etc.), availability of equipment and time, expertise, environmental factors and cost.

It has been observed that for expansive soils, chemical stabilization through addition of lime is more suitable (Herrin and Mitchell, 1961; Bell, 1988; Raju, 1991; Sherwood, 1993; Greaves, 1996; Holt and Freer-Hewish, 1996; Mathew and Narasimha Rao, 1997 etc.). However, in many a cases stabilization through admixtures, such as fly ash and other non-swelling soils too have produced substantial performance improvement (Mateos and Davidson,1962; Katti, 1978; Abduljauwad, 1993; Keshawarz and Dutta, 1993; Katti and Katti,1996; Sivapullaiah et. al., 1996; Srivastava et al., 1997; Muttharam, 2000; Mishra et al., 2008; Cokca, 2001 etc.)

2.2 LIME SABLIZATION OF EXPANSIVE SOILS

2.2.1 Mechanism

The improvement in engineering performance of cohesive soil through addition of lime is primarily attributed to two basic mechanisms (Davidson and Handy, 1959):

(i) Physico-chemical reaction between the lime and clay minerals, leading to reduction in plasticity of cohesive soil, known as soil improvement or modification.

(ii) Soil-lime pozzolanic reactions, leading to increased strength of soil, known as lime stabilization

The first phase of the chemical reaction involves immediate changes in soil properties caused by cation exchange that takes place between ions of the soil and lime. The strong calcium cations produced by lime replace the weaker ions, such as sodium and hydrogen, on the surface of the clay particle. Besides, crowding of additional calcium cations of the lime in to the surface of clay, alters charge density on the surface of the clay particles leading to flocculation/agglomeration of the clay particles. The aggregated clay particles behave as silt with low plasticity that it is easily workable.

The second phase of the chemical reaction involves, a relatively slower, long-term pozzolanic reaction within the lime-soil mixture, resulting in the formation of cementing agents which increases strength and durability. When lime is added to soil the pH of the pore water increases. At high pH (i.e. >12), the silica and alumina from the clay become soluble and react with the calcium from the lime to form calcium silicate hydrate and calcium aluminate hydrate gels, which coat the soil particle and subsequently crystallize to bond them leading to increased strength.

In addition to the above two reactions, there takes place a third reaction called carbonation, between lime and atmospheric carbon-dioxide. It forms relatively weak cementing agents like calcium carbonate (Goldberg and Klein, 1952; Eades and Grim, 1960; Arman and Munfakh, 1970). This carbonate, though forms weak cementing compounds but also deters the pozzolanic reaction between silica, alumina and lime thereby prevents normal strength gains.

2.2.2 Effect of Lime on Index and Engineering Properties of Soil

2.2.2.1 Effect on Plasticity Characteristics

Decrease in liquid limit of clayey soils with addition of lime has been observed by many researchers (Wang et al., 1963; Jan and Walker, 1963; Mateos, 1954; Thompson, 1966; holtz, 1969). However, there are exception to this observation that liquid limit of some clayey soils increases with addition of lime (Clare and Cruchley, 1957; Zolkov, 1962; Ingles and Metcalf, 1972; Prakash et al., 1989). The decrease in liquid limit due to addition of lime is attributed to the reduction in thickness of diffuse double layer, which takes place due to cation exchange and flocculation-agglomeration reactions (Lambe, 1962; Thompson, 1966).

However, mechanisms leading to increase in liquid limit due to addition of lime has not been clearly understood yet, which needs further investigation.

The plastic limit of soils generally shows an increasing trend due to addition of lime. Greater is the amount of clay greater is the increase in plastic limit with largest increase taking in the case of montmorillonitic soils (Hilt and Davidsson, 1960). Irrespective of the liquid limit increasing or decreasing the plasticity index is usually reduced with the addition of lime (Herrin and Mitchell, 1961, Dumbleton, 1962). This is due to the increase in the plastic limit in presence of lime. But, Anon (1975) has reported increase in plasticity index for kaolinitic soils.

The plasticity characteristics are also influenced by the length of time (curing) the lime reacts with the soil and the type of lime (Prakash et al., 1989; Sivapullaiah et al., 2000).

Given the strikingly different observations it needs further investigation to better understand the mechanisms.

2.2.2.2 Effect on Compaction

It is a general observation that the addition of lime to clayey soils increases the optimum moisture content and reduces the maximum dry density for the same compactive effort (Dumbleton, 1962; Mateos, 1964; Prakash et al., 1989; Bell, 1996; Holt and Freer-Hewish, 1996; Sivapullaiah et al., 1998). However, there are few exceptions. Johnson (1948) reported that out of 11 different soils tested, in case of two soils the unit weight was found to increase with addition of lime. The optimum moisture content increases and maximum dry density reduces with increase in clay content. Croft (1964) found that higher compaction density was obtained in lime treated kaolinite clay than in soils in which clay minerals with expandable lattices predominated.

Sivapullaiah et al. (1998) through experiments on black cotton soil, observed that below the lime fixation point, lime addition steeply decreased the maximum dry density, while lime addition beyond lime fixation point caused no further change in compaction characteristics of the treated soil.

The time lag between wet mixing and compaction further reduces the maximum dry density and increases the optimum moisture content (Mateos, 1964; Prakash et al., 1989; Bell, 1996; Holt and Freer-Hewish, 1996; Osinubi, 1998). This change in compaction characteristic is due to the cementitious reactions between soil and lime during curing, leading to greater resistance to compaction (Diamond and Kinter, 1965; Verhasselt, 1960).

The difference in the observations on the lime added compaction characteristics (i.e. maximum dry density and optimum moisture content) pertaining to soils having different minerals and clay contents needs further investigation to understand the mechanism involved.

2.2.2.3 Effect on Swell

Lime tends to reduce the water induced increase in volume (i.e. swelling) of expansive soils (Mateos, 1964; Thompson, 1969; Uppal, 1969; Bhasin et al., 1978; Abduljauwad 1995; Al-Rawas et al., 2005; Guneya et al. 2005). Jones (1958) observed that 2% of lime completely arrested swelling of moist California soil.

Sivapullaiah et al. (2000) carried out free swell tests on black cotton expansive soil with addition of different percentages of lime. It was observed that (Sridharan and Rao, 1985; Sridharan et al., 1986) the free swell index decreases till 3% lime addition beyond which it starts increasing till 6% of lime. Further addition of lime has marginal influence on free swell of the black cotton expansive soil. The initial decrease in free swell index is attributed to reduction in thickness of diffuse double layer and the increase in free swell index at higher percentage of lime is attributed to flocculation.

Abduljauwad (1993) carried out the suction test by using filter paper technique, before and after swelling. The test results indicate that the lime stabilized soil absorb less water than the untreated soil. In general the reduction of swelling is mainly due to reduction of diffuse double layer thickness through ion exchange between lime and clay and increased pore salinity.

Abduljauwad (1995) observed that, through addition of 5% lime and without any curing, the swell potential of soil was reduced from 25% to 9%. However, after 7 and 30 days curing the swell potential marginally reduced to 8% and 7% respectively. Since majority of reduction in swell potential takes place immediately further reduction with increase in curing period is marginal (Thompson, 1969; Abduljawad, 1995).

Holtz (1969) observed that 6% of lime can reduce the swelling pressure of expansive soils as high as 350%. Al-Rawas et al. (2005) found that with the addition of 6% lime, both the swell and swell pressure of expansive soil reduced to zero. Abduljauwad (1993) reported that the optimum lime content required to substantially reduce the swell and swell pressure is about 3%, beyond which further improvement is marginal.

Swell potential and swell pressure is found to be a property of soil type, quantity of lime added, curing period etc. which needs a systematic study to broaden the understanding of different mechanisms involved.

2.2.2.4 Effect on Shrinkage

The shrinkage of clayey soils can be reduced substantially by adding lime to it (Spangler and Patel, 1949; Laguros et al., 1956; Wang et al., 1963; Mateos, 1964; Thompson, 1966; Holtz, 1969; Bhasin et al., 1978; Sivapullaha et al., 2000; Rao et al., 2001). With increase in the quantity of lime added the shrinkage limit of soil increases and the shrinkage void ratio decreases, indicating that the soils shrink less upon drying (Mateos, 1964). The amount of shrinkage limit upon drying is a measure of average particle orientation (Lambe, 1958). Any soil with a parallel arrangement of particles undergoes more volume reduction upon drying than the same soil with its particles in a random/ flocculent fabric. Addition of lime to soil, changes its fabric to flocculated one. This phenomenon is dependent upon type and amount of clay, the quantity of lime added and the duration of treatment. The shrinkage limit of kaolinite only increases slightly while for montmorillonite rich black cotton soil it is very high (Sivapullaha et al., 2000).

Shrinkage limit increases with curing of lime-soil mixture (Prakash et al., 1989; Shivapullaiah et al., 2000). This is attributed to the increased the flocculation of clay particles due to prolonged reaction between lime and soil.

The study of shrinkage limit of lime treated soils over a wide variation of plasticity characteristics of the soil needs a more detailed study. Besides, the relationships between shrinkage void ratio with water content, required for predicting the shrinkage induced deformation in soil, has not been studied yet for the case of lime treated soils. *This is proposed to be carried out under the present study.*

2.2.2.5 Effect on Strength

Lime addition can bring substantial increase in strength of soil (Bell, 1988, 1996; Narasimha Rao, 1990; Sivapullaiah et al. 1998; Rajasekaran and Narasimha Rao, 2000). This strength gain is influenced by several factors such as soil type, type of lime, amount of lime added, curing time and test method (temperature and water availability), moisture content, compaction density, and time elapsed between mixing and compaction (Ingles and Metcalf, 1972).

The amount of strength increase in a soil that can be produced by adding lime is dependent on the pozzolans such as alumina and silica present in the soil. When desirable pozzolans are available they react readily with the lime to improve the strength of the lime-soil mixture. If the soil has very small or no pozzolans, marginal strength improvement is observed due to addition of lime (Herrin and Mitchell, 1961). The optimum lime content tends to range from 4.5% to 8%, higher values occurring in soils with higher clay fractions. Through unconfined compression tests Bell (1996) observed that expansive clays such as montmorillonitc show a relatively rapid gain in strength, in presence of lime, than the kaolinitic clay. The optimum quantity of lime giving maximum strength increase is about 4% for montmorillonite, varied between 4% to 6% for kaolinite and between 4 to 8% for quartz. *The mechanism governing such variation with soil type needs further investigation.*

It is of interest to note that in many a cases further addition of lime beyond the optimum, retards the strength gain (Hilt and Davidson, 1960; Bell, 1988; Herrin and Mitchell, 1961, Al-Rawi, 1981, Bell, 1996). It is postulated that since lime itself has nether appreciable friction nor cohesion, excess of lime reduces strength. *However, this aspect needs further investigation which has been attempted in this study.*

Strength of lime stabilized soil depends on their moisture contents. Strength decrease with increase in moisture content. Lime-soil mixtures compacted at moisture contents above optimum attain, after brief periods of curing, higher strength than those compacted with moisture content less than optimum. This is probably because the lime is more uniformly diffused and occurs in a more homogeneous curing environment (Sabry and Parcher, 1979).

Saturation of lime stabilised soil specimens with water, that simulates the worst conditions to which a stabilized soil may be subjected, results in a reduction of strength. Al-Rawi (1981) reported that this reduction could be as high as 50%. However, results reported by Thompson (1970) showed that prolonged exposure to water produced only slight detrimental effect and that the ratio of soaked to unsoaked strength was high i.e. about 0.7 to 0.85.

The compaction conditions affects the lime stabilisation induced strength gain significantly. With the compactive effort increasing from standard to modified AASHO, the compressive strength of soil-lime mixtures was found to increase by 50% to 250% for both 7 and 28 days cured specimens (Mateos, 1964). This increase in strength was obtained by an accompanying increase in maximum dry density of about 10%. Bell, 1961; Mitchell and Hoper, 1961; Sivapullaiah et al., 1998; observed that there occurs substantial reduction in strength with increase in time interval between mixing and compaction.

Laguross et al. (1956) found that the curing induced strength of lime stabilized soils increases rapidly at first, notably during the first seven days of curing and then increases relatively slowly at a more or less constant rate for about 15 weeks. This is because the cementitious products, due to lime-clay reaction, form at an early stage of the process (Lees et al., 1983). Brandl (1981) observed that the time dependent increase of the strength is approximately linear with the logarithm of time. Locate et al. (1990) reported that at very high water content (i.e. above the liquid limit) significant increase in strength can still be obtained provided enough quantity of lime is added and longer period of curing is allowed. Laguros et al. (1956) found that 90% relative humidity gave the greatest ultimate strength gain when comparing different curing conditions.

The strength gain of lime stabilized soils are accelerated due to increased temperature of curing (Laguros et al., 1956; Mateos, 1964). The specimens cured at 35^0 C are found to have developed twice the strength of the one cured at 25^0 C.

2.2.2.5 Effect on Microstructure

The microstructural behaviour of lime treated soils has been studied by many a researchers (Eades and Grim, 1960; Wang et al., 1963; Diamond et al., 1963; Croft, 1967; Kawamura and Diamond, 1975; Joshi et al., 1981; Lee, et al., 1983; Rajasekaran and Narasimha Rao, 1997, 1998 and 2000; Kassim et al., 2005). The changes occurred in the lime treated soils are attributed to the cementation of the soil particles due to soil-lime reactions, and attempts were made to identify the new reaction products formed using X-ray diffraction (XRD). The results indicated the formation of cementitious compounds such as calcium silicate hydrate (CSH), calcium aluminate hydrate (CAH) and calcium aluminate silicate hydrate (CASH), in different lime treated soil systems. Croft (1967) reported that the development and hardening of gelatinous reaction products and intergrowths of crystalline, hydrous calcium silicates and aluminates are responsible for cementation in clay-soils stabilized with lime. Structural arrangements of reaction products and soil elements which contributed to rigidity of stabilized soil were determined from microscopic examination of sections through compacted soil-stabilizing agent mixtures after curing. Arabi and Wild (1986) reported that the bond which develops during curing between soil particles in the presence of lime and moisture is a result of development and growth of the newly formed cementitious compounds. Khoury et al. (2004) employed the reference intensity ratio (RIR) method to identify and quantify the mass percent of minerals and cementitious compounds in the mixtures. Results show the formation of cementitious compounds such as ettringite, gismondine, straetlingite, and tobermorite, among others, which are responsible for increased unconfined compressive strength (UCS). Besides, the cementitious compounds showed the same qualitative trend as the UCS with curing time. James et al. (2008) through XRD analysis showed the formation of reaction products such as calcium silicate hydrate (C-S-H), calcium aluminum silicate hydrate (C-A-S-H), and hydrotalcite (HT), which facilitated the strength increment, of the lime-slag treated clay.

Scanning Electron Microscope (SEM) can be used for qualitatively visualizing the reaction products formed and changes in fabric taking place in the lime-treated soil systems (Diamond et al., 1965; Kawamura and Diamond, 1975; Rajasekaran and Narasimha Rao, 2000). The aggregation and flocculation of the soil particles formed due to lime-soil reactions have been observed though the SEM micrographs. Locate et al.

(1990) reported that the cementitious compounds, CASH and CSH, are platy and reticular in structure respectively. It is also observed that at 10 days curing period, addition of quick lime first results in a rapid exothermic hydration process and a simultaneous cation exchange which flocculates the soil in to larger lumps. But after 100 days these lumps once again cemented together by the subsequent pozzolanic reaction products, such as platy CASH and reticular CSH. Such a fabric results in the reduction of shrinkage and swell of the soil system.

Energy Dispersive X-ray (EDX) analysis, shows that, lime usually concentrates in pores and on the surface of aggregates, therefore, has minimal effect on the inner structure of the aggregates (Shi Bin et al. 2007). Bell (1996) reported that, in the case of kaolinite treated lime, the major element present is silicon, followed by aluminum. Traces of calcium and to a lesser extent potassium was also noticed. The calcium presumably is part of the reaction products formed by attack of lime upon the kaolinite. While in case of montmorillonite the compounds formed due to lime are primarily of silicon and aluminum, and to a lesser extent of calcium and magnesium.

2.3 STABILIZATION OF EXPANSIVE SOIL THROUGH ADDITIVES

Stabilization of expansive soils through additives such as dune sand, fly ash, cohesive non-swelling soil etc, have been employed to improve its engineering performance in terms of plasticity characteristics, strength and durability.

The performance improvement of a calcareous expansive clay, through addition of sand has been studied by Abduljauwad (1993). It is observed that through addition of 8% of sand to the expansive soil, the liquid limit reduces from 173 to about 145% and the plasticity index reduced from 124 to about 100%. Correspondingly the swell and swell pressure are found to have reduced from 38% to about 15% and 650 kPa to about 200 kPa respectively. The addition of sand to active clay does not generate chemical reactions and its effect is mainly to reduce the clay content. A higher proportion of sand content, and correspondingly lower clay content, will reduce the tendency of the clay to swell due to larger capillary canals in the soil pores leading to reduction in the soil suction and hence better performance in terms of reduced swelling. The effect of percentage of sand and curing time on the swell and swell pressure is very small.

Sivapullaiah et al. (1996) studied the swelling behaviour of expansive soil (i.e. betonite) mixed with different percentages and types of non-swelling soil. The non-swelling soils used are kaolinite, silt, fine sand and coarse sand. It is observed that for the same percentage of expansive clay, the total swelling decreases significantly with the increase in the size of the nonswelling soil added to it.

Azam (2007) carried out test on clay mixed with sand in different proportions to find out its suitability for earthwork constructions such as roads and landfills. The test results indicate that consistency limit of clay-sand mixes depend on the weighted average of the constituents in the clay-sand mixes. 10% clay through 40% clay captured the transition from sand like behaviour to clay like behaviour. In the sand matrix, initial swelling is the dominant phenomena on the overall response while in the clay matrix majority of swelling took place in the subsequent primary stage.

Sridharan and Prakash (2000) studied the shrinkage limit of soil mixes such as, clay-clay, clay-non-cohesive soils and non-cohesive soil. It is observed that the shrinkage limit of a soil does not depend upon its plasticity characteristics, but is primarily governed by the relative grain size distribution of the soil. For the soil systems having the same grain size distribution, the one which has higher shearing resistance, at the particle level, will shrink less.

Test data reported by Mishra et al. (2008) indicates that the rate of swell and shrinkage are reduced when the soil changes from expansive black cotton soil to non-expansive residual soil.

Fly ash can be effectively used as an additive to improve the performance of clayey soils (Mateos and Davidson, 1962; Sivapullaiah et al., 1996; Cokca, 2001; Keshawarz and Dutta, 1993; Al-Rawas, 2004). Addition of fly ash with lime to the expansive soils is found to produce enhanced performance improvement as compared to fly ash alone (Nicholson and Kashyap, 1993; Srivastava and Joshi, 1997; Srivastava et al., 1998).

Providing a cushion of different forms of materials i.e. sand, sand and boulder, cohesive non swelling soil (CNS) etc. to absorb the swelling potential of expansive soils is one popular method in India (Satyanarayan, 1969; Katti, 1979; Mussa, 1985; Katti and Katti, 1996; Subba Rao, 2000).

2.4 CONCLUDING REMARKS AND SCOPE OF THE PRESENT WORK

From literature review it is generally observed that lime improves the performance of clayey soils in terms of reduced swelling and increased strength. However, it has been reported that in some cases; depending on the type of soil, amount of lime added, curing period etc. the performance improvement reduces. In view of this an attempt is made to carry out a systematic study through careful variation of above parameters, to develop an understanding of the mechanisms involved in lime stabilization of soils.

It is a general practice to improve the performance of expansive soil beds by providing a cushion of non-expansive soils such as sand, gravel, residual murhom soil etc. Higher the swelling potential of the expansive soil thicker is the cushion required leading to larger volume of such non-swelling soils to be borrowed. This at times makes the project too costly. Since both lime and non-expansive soils improve the performance of the expansive soils a blend of these two materials is expected to produce better performance improvement. Hence an attempt has been made in this thesis to study this aspect. In India the residual soils are easily available non-expansive soils. Therefore in the present study this has been used along with lime in expansive soils for the performance improvement.

CHAPTER 3

MATERIALS AND METHODS

3.1 INTRODUCTION

The details of the different test materials, i.e. soils, lime their characterization and the procedures of the various tests carried out in this study, are presented in this chapter. Planning of experiments, preparation of samples and details of test procedures are presented and discussed.

3.2 MATERIALS

3.2.1 Soils

Primarily an expansive soil (ES) and a residual soil (RS) that represent the extreme types of soil are used in the present study. Besides to cover a wide range of properties these two soils are mixed in different proportion (i.e. 100%ES, 80%ES + 20%RS, 60%ES + 40%RS, 40%ES + 60%RS, 20%ES + 80% RS, 100%RS) to prepare six different soil samples. The details of the two primary soils are presented below.

3.2.1.1 Expansive Soil

Expansive soil used in this study is a commercially available bentonite. This soil, a typical of highly expansive clay, has montmorillonite as chief clay mineral. The soil is abbreviated as ES. The bentonite (ES) being finer than 425 μm is directly used in the experiments. The expansive soil being high swelling bentonite its particle size analysis through hydrometer method that uses sedimentation analysis is extremely difficult. However, in view of the extremely high plasticity characteristic, without much error, the soil can be assumed to have 100% clay size particles. The specific gravity, liquid limit, plastic limit and shrinkage limit of this soil are found to be 2.63, 459.94%, 53.7% and 7% respectively. As per ASTM D2487, the soil is classified as clay with high plasticity (CH).

The specific surface area of the soil as obtained through BET method is 86.45 sq.m/g. Its pH and cation exchange capacity are found to be 8.3 and 69.12 meq/100g of soil respectively. The pH value being more than 7, the soil is alkaline.

The photo micrograph of the expansive soil obtained using Scanning Electron Microscope (SEM) is presented in Fig. 3.1. The micrograph clearly shows that the soil has a sheet like structure. The soil particle surfaces have a rough texture. Data from Energy Dispersive X-ray spectrometer (EDX) analysis of the expansive soil is depicted in Fig. 3.2. The major element present are; Na, Mg, Al, Si, K, Ca, Fe, Ti and O. The presence of Na-Al-Mg-Si-O represent the montmorillonite mineral, while Si, Al represent quartz and aluminum oxide which is further confirmed through X-ray diffraction analysis (XRD). The X-ray diffraction pattern with the mineralogical details for the expansive soil is shown in Fig. 3.3. It could be observed that the expansive soil primarily consists of montmorillonite, quartz and aluminum oxide.

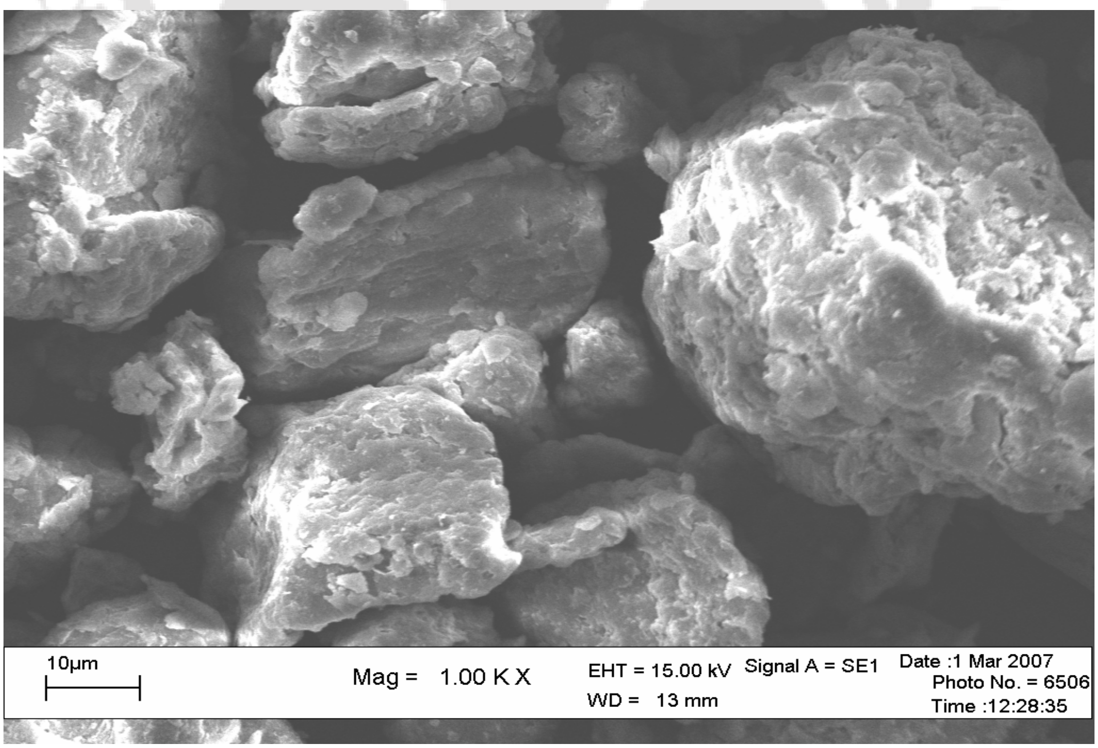

Fig. 3.1 Scanning electron photo micrograph of expansive soil (powder sample)

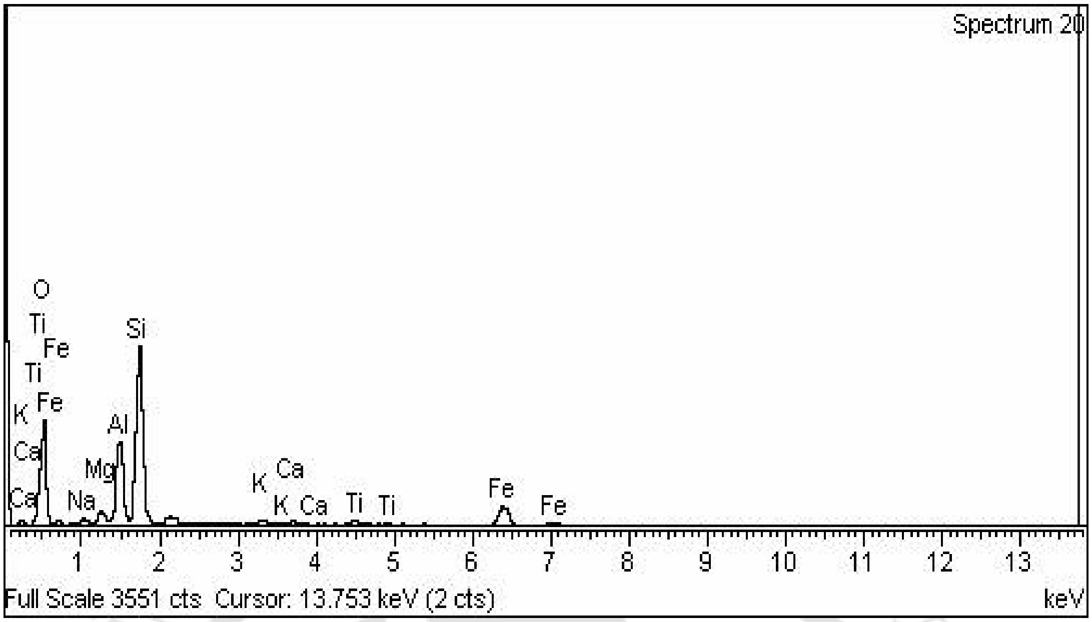

Fig. 3.2 EDX spectrum of expansive soil

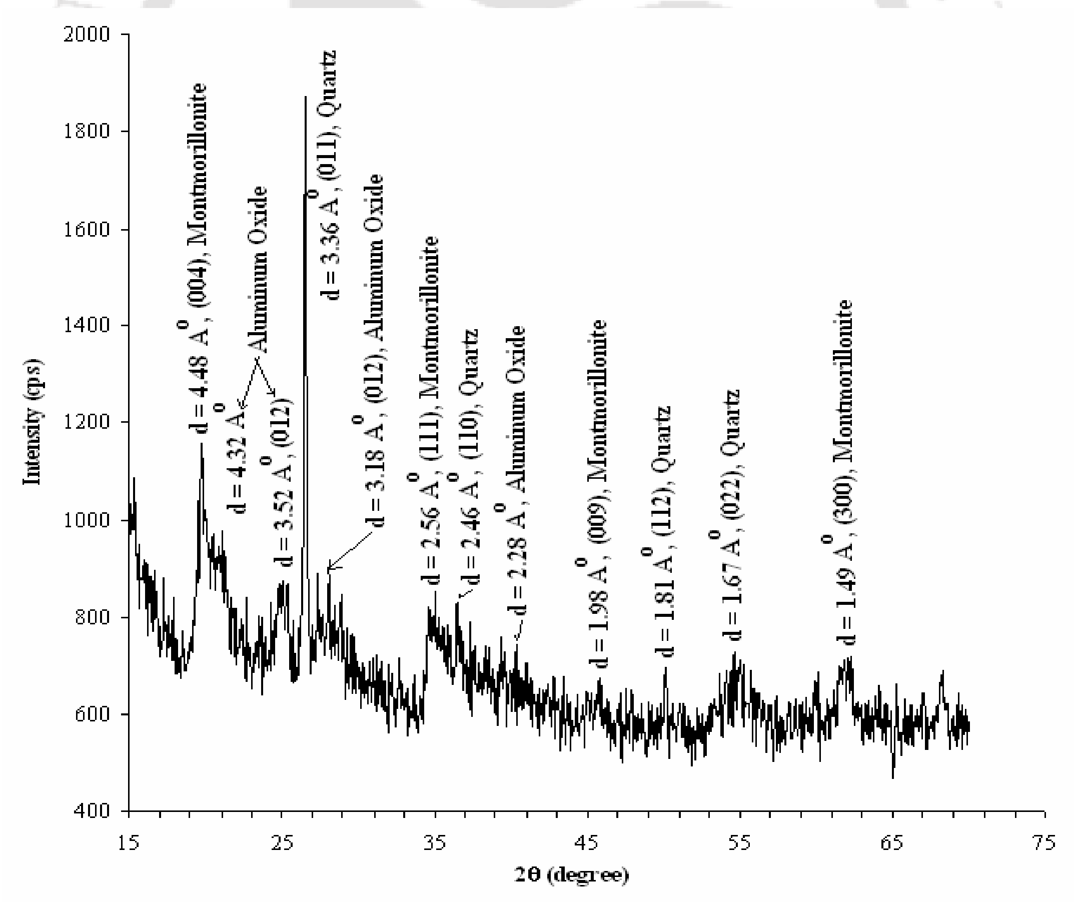

Fig. 3.3 X-ray diffraction pattern of expansive soil

3.2.1.2 Residual Soil

Disturbed residual soil samples were obtained from IIT Guwahati campus. The soil was collected through open excavation from a depth of about half of meter. The collected residual soil was air-dried and sieved through IS sieve of 425 μm size. The soil passing through the 425 μm sieve was used in all the experiments. In this thesis, at many places it is referred through the abbreviated term RS.

The particle size distribution was determined as per IS: 2720 (Part-4)-1985. For particles of size more than 75 μm sieve analysis was carried out and for the particles of size finer than 75 μm hydrometer test was used. Fig. 3.4 shows the particle size distribution curve of the residual soil. it is predominantly fine grained consisting of 83% fines in which silt is predominant (71.74%).

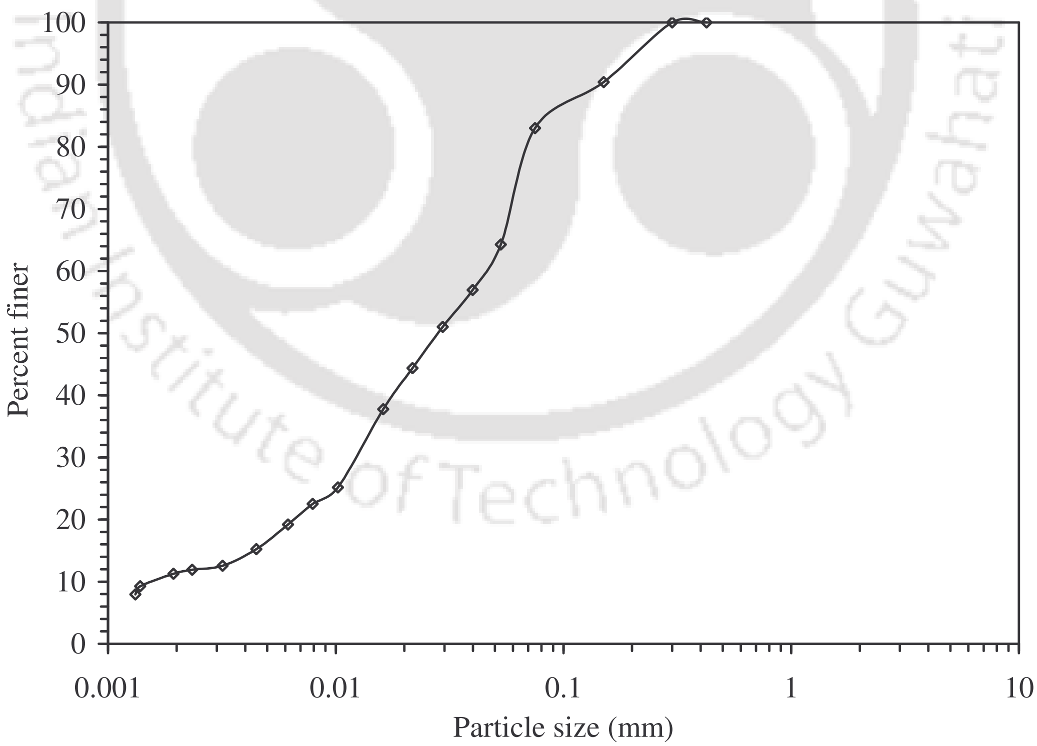

Fig. 3.4 Particle size distribution of residual soil

The specific gravity, liquid limit, plastic limit and shrinkage limit of this soil are found to be 2.67, 45.33%, 25.99% and 25.45% respectively. As per ASTM D 2487, the soil is classified as clay with medium compressibility (CI).

The specific surface area of the soil as obtained through BET method is 38.76 sq.m/g. Its pH and cation exchange capacity are found to be 4.8 and 12.43 meq/100g of soil respectively. The pH value being less than 7, the soil is acidic.

The photo micrograph of the residual soil obtained using Scanning Electron Microscope (SEM) is presented in Fig. 3.5. The micrograph clearly shows that the soil particle has an angular structure. The soil particle surfaces have a rough texture. Data from Energy Dispersive X-ray spectrometer (EDX) analysis of the residual soil is depicted in Fig. 3.6. The major elements present are; Si, Al, Fe, K, and O. The presence of Al-Si-O-H represent the aluminum silicate hydroxide which is the kaolinite mineral. Si and Fe

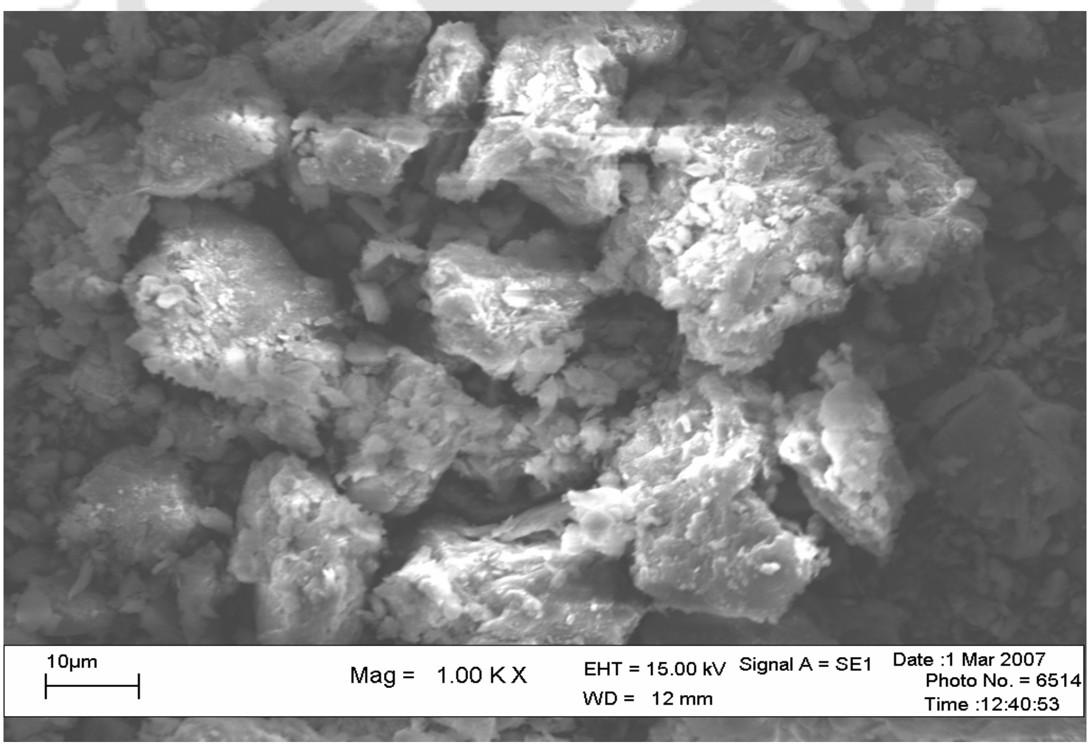

Fig. 3.5 Scanning Electron photo micrograph of residual soil (powder sample)

represent quartz and magnetite which has further been analyzed through X-ray diffraction (XRD). The X-ray diffraction pattern with the mineralogical details for the residual soil is shown in Fig. 3.7. It could be observed that the residual soil primarily consists of kaolinite mineral, quartz and magnetite.

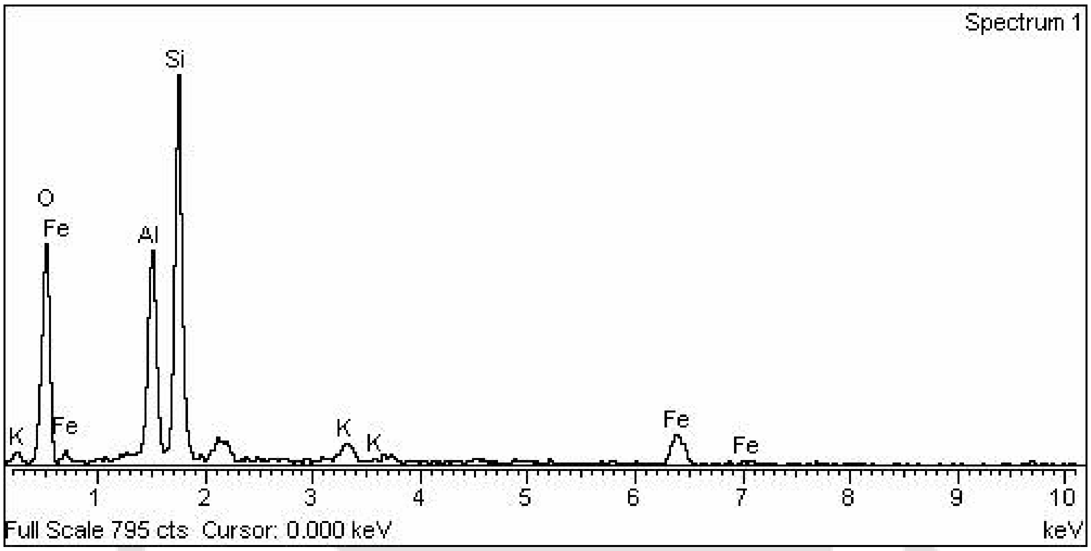

Fig. 3.6 EDX spectrum of residual soil

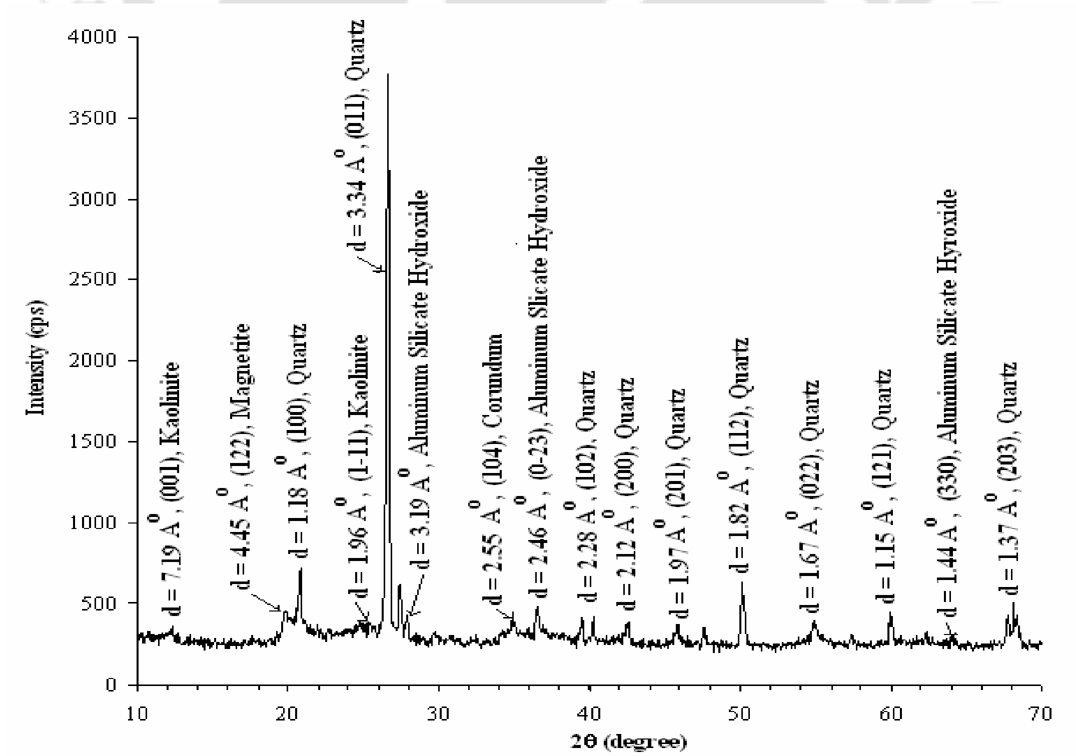

Fig. 3.7 X-ray diffraction pattern of residual soil

3.2.2 Lime

Laboratory reagent grade quick lime (CaO) obtained from S.D. Fine-Chem. Ltd. Mumbai, India, is used in this investigation. Its composition is given in Table 3.1

Table 3.1 Composition of lime used in the present study

Property	Value
Chemical formula	CaO
Molecular weight	56.08
Specific gravity	3.1
Minimum assay	95 %

3.3 PLANNING OF EXPERIMENTS

Under the present investigation six different series of experiments under a systematically planned scheme, have been carried out, to study the physico-chemical and engineering behaviour of residual soil-lime treated expansive soil. The details of these test series are presented in Table 3.2.

In each test series expansive soil was admixtured with residual soil of different percentage (i.e. 0%, 20%, 40%, 60%, 80% and 100%). All these six soil samples were added with different percent of lime (0, 1, 3, 5, 9 and 13% 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, 21 and 28 days) before the tests were carried out.

Tests under series 1 were conducted to study the plasticity characteristics through index properties of soils/lime treated soils, such as; liquid limit, plastic limit and shrinkage limit. Besides a limited number of tests were carried out to determine the specific surface area of the soils alone (i.e. not the lime treated soils). Under series 2 influence of lime on compaction behaviour of different soil samples were studied. The swelling behaviour of residual soil-lime treated expansive soil was studied, in series 3, using free swell and

Table 3.2: Details of tests

Test series	Soil	Lime added (%)	Curing period (day)	Test detail
1	100%ES 80%ES + 20%RS 60%ES + 40%RS 40%ES + 60%RS 20%ES + 80%RS 100%RS	0, 1,3,5,9,13	0, 3, 7, 21, 28	(i) Plasticity characteristics (LL, PL, SL) (ii) Specific surface area (for untreated soil only)
2	100%ES 80%ES + 20%RS 60%ES + 40%RS 40%ES + 60%RS 20%ES + 80%RS 100%RS	0, 1,3,5,9,13	Nil	Compaction
3	100%ES 80%ES + 20%RS 60%ES + 40%RS 40%ES + 60%RS 20%ES + 80%RS 100%RS	0, 1,3,5,9,13	Test started immediately after sample preparation. Curing takes place during the test	i) Free swell ii) Oedometer swell
4	100%ES 80%ES + 20%RS 60%ES + 40%RS 40%ES + 60%RS 20%ES + 80%RS 100%RS	0,1,3,5,9,13	Test started immediately after sample preparation, Curing takes place during the test	Volumetric shrinkage
5	100%ES 80%ES + 20%RS 60%ES + 40%RS 40%ES + 60%RS 20%ES + 80%RS 100%RS	0,1,3,5,9,13	3,7,21,28	Unconfined compressive strength
6	100%ES 60%ES+40%RS 100%RS	0,1,3,5,9,13	28	Microstructural characteristic (SEM, EDX, XRD)

oedometer swell tests. The volumetric shrinkage response of the same was studied under series 4. Improvement in performance of the expansive soil in terms of increase in strength was studied using unconfined compressive strength tests (Test series 5). Microstructural characteristics of selected samples were studied through SEM, EDX and XRD, under test series 6.

3.4 TEST DETAILS

3.4.1 Atterberg Limits

3.4.1.1 Sample Preparation

Table 3.2 gives the details of the expansive soil-residual soil-lime mixtures for tests conducted. For the preparation of different soil samples (i.e. 100%ES, 80%ES + 20%RS, 60%ES + 40%RS, 40%ES + 60%RS, 20%ES + 80%RS and 100%RS) the required amounts of oven dried residual soil and expansive soil were measured and mixed together thoroughly in the dry state. If lime was not used, then dry expansive soil-residual soil mixture was mixed with required amount of distilled water. In case of lime addition, desired percentage of lime was added to the soil and was dry mixed thoroughly. The quantity of lime added was in percentage of dry weight of soil ($W = W_{ES} + W_{RS}$, W it total weight of soil sample, W_{ES} is weight of expansive soil, W_{RS} is weight of residual soil). Then expansive soil-residual soil-lime mixture was mixed with water. A large amount of water (i.e. greater then liquid limit) was added to the soil/ soil-lime mixture, and mixing was done frequently by hand. Care was taken to prepare homogeneous mixtures at each stage of mixing.

Soil samples thus prepared were kept in polythene bags and sealed. These sealed packets were kept in desiccator under 100% relative humidity and were cured for predetermined intervals of time at room temperature. After curing is done the samples were again mixed thoroughly before the tests were done.

3.4.1.2 *Liquid Limit Test*

The liquid limit of soil sample were obtained uniformly for all soils, with and without additives by Percussion method (Casagrande method, ASTM D 4318). The liquid limit tests were carried out to obtain a minimum of five points for plotting the flow curve. The number of blows in between 15 to 35 were considered. The entire operation was repeated for each soil specimen and the average of the two liquid limit value was reported.

3.4.1.3 *Plastic Limit Test*

The plastic limit of soil/ soil-lime specimens was determined by the 3 mm rolling thread method as outlined in the ASTM D 4318. The plastic limits reported are an average of three determinations.

3.4.1.4 *Shrinkage Limit Test*

The tests for shrinkage limit were carried out as per the ASTM D 427. The entrapped air in the soil sample in the shrinkage dish was carefully expelled out. Cracking during drying was prevented by first allowing the soil pat to dry very slowly in the air under controlled condition followed by oven drying to a constant mass. The shrinkage limit reported is the average of three of trials.

3.4.2 Specific Surface Area (SSA)

Nitrogen (N_2) gas adsorption Technique

The specific surface area, SSA of a soil sample are determined by the Brunauer, Emmet and Teller (BET) method using Coulter surface area analyzer (Model No. SA 3100, USA). The method is based on the adsorption of nitrogen (N_2), at temperature of 77 K (Brunauer, et al., 1938). The surface area is taken as the area for monolayer coverage. 0.2 gram of the air-dried soil sample was poured in to a glass-cell and degassed at 200^0C, for a period of 3hour. Later, the sample was exposed to N_2 corresponding to different relative pressure (P/P_0), where P and P_0 are applied pressure and saturation vapor pressure of N_2 respectively. The volume of N_2 adsorbed (V_{ads}) on the sample at pressure P was recorded and adsorption isotherm is developed. A typical such isotherm is depicted in Fig. 3.8.

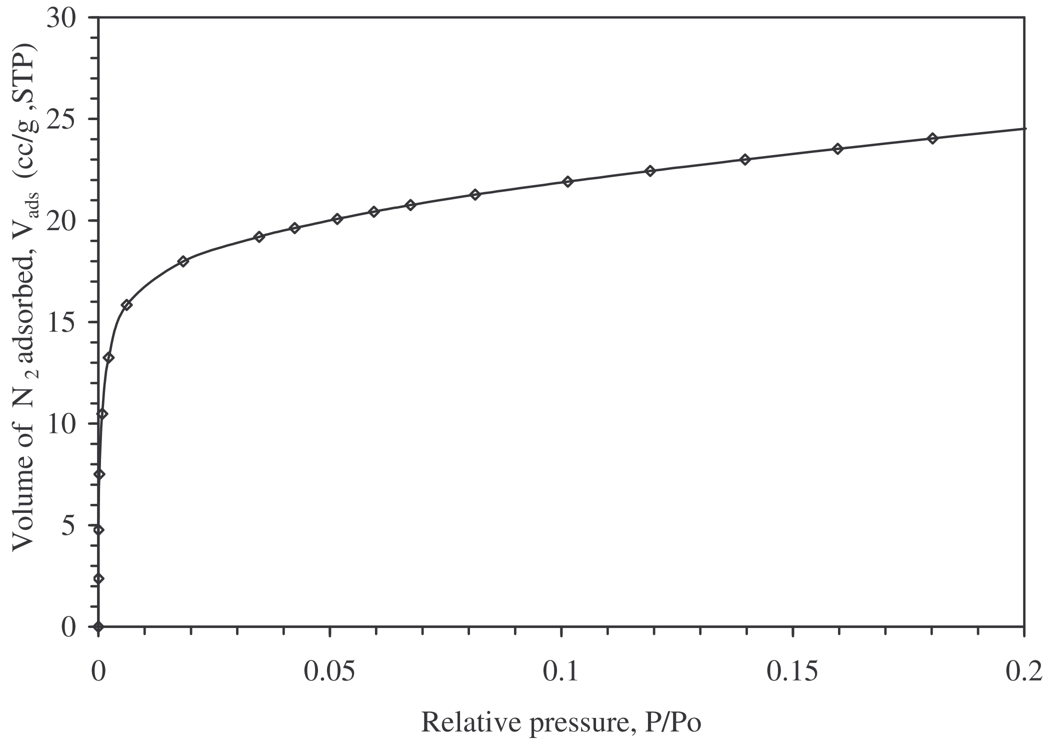

Fig. 3.8 BET adsorption isotherms for expansive soil

Eq. 3.1 is the BET equation generally used to estimate the monolayer coverage (V_m) required to estimate the specific surface area of soil (Brunauer et al., 1938; Santamarina et al., 2002).

$$\frac{P}{V_{ads}(P_0 - P)} = \frac{1}{V_m C} + \frac{C-1}{V_m C}\frac{P}{P_0} \qquad 3.1$$

Where, C is the BET constant. The relative pressure (P/P_0) range of the validity of the above equation 3.1 is in the range of 0.05 to 0.3.

A plot of P/ [$V_{ads}(P_0$-P)] versus P/P_0 would yield a straight line, with slope of (C-1)/V_mC and intercept of 1/V_mC. Fig. 3.9 presents this plot corresponding to the data in Fig. 3.8.

Once the slope and intercept of this line are known, C and V_m can be obtained from the following two equations:

$$\text{Slope} = \frac{(C-1)}{V_m C} \qquad 3.2$$

$$\text{Intercept} = \frac{1}{CV_m} \qquad 3.3$$

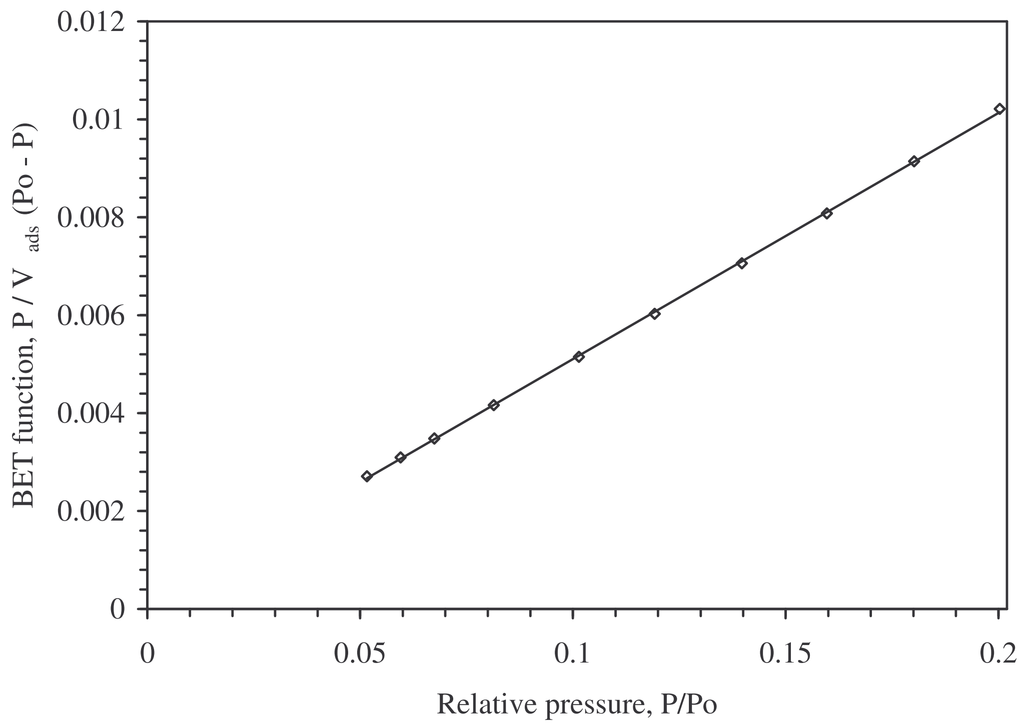

Fig. 3.9 BET function vs. relative pressure for expansive soil

Once V_m is obtained the surface area (A_s) of the soil is then determined from the following expression:

$$A_s = V_m . N_A . a_m \qquad 3.4$$

Where, N_A is Avogradros number; a_m is the molecular projected area of the adsorbate, N_2 (= 16.2×10^{-20} m^2/molecule).

3.4.3 Cation Exchange Capacity (CEC)

The cation exchange capacity (CEC) of soils are determined as per the method proposed by Horneck et al. (1989), which is based on ammonium replacement technique. This method involves saturation of the cation exchange sites on the soil surface with ammonium, equilibration, and removal of the excess ammonium with ethanol, replacement and leaching of exchangeable ammonium with protons from HCl acid. The concentration of ammonium in the final leachate is measured using Ion Chromatograph (Metrohm, 792 Basic IC) and CEC is calculated using Eq. 3.5.

$$CEC(meq./100gmofsoil) = (ammoniumconcentration \times \frac{0.25}{14} \times \frac{100}{samplesize(gm)}) \qquad 3.5$$

3.4.4 pH test

The pH of expansive soil-residual soil mixtures were obtained by the electrometric method as per ASTM D 4972. In this method pH test is carried out by using pH meter. This instrument consists of a glass electrode with a temperature setting arrangement. The accuracy of pH measurement is ± 0.05 units. The instrument was standardized with two standard buffer solutions (pH of 4.0 and 9.2). The expansive soil-residual soil mixtures of 10 gram passing 425 µm sieve were equilibrated with distilled water (Solids: Water ratio = 1:2.5). The suspension is stirred well and allowed to come to room temperature before taking the pH measurement.

3.4.5 Initial Consumption of Lime (ICL)

Initial consumption of lime test for expansive soil-residual soil mixtures was determined as per ASTM C 977–03. A saturated solution of lime (calcium hydroxide) in distilled water completely free of carbon dioxide has a pH value of 12.40 at 25 ^{0}C. This pH is required to maintain reaction between lime and any reactive components in the material to be stabilized. Soil specimens passing 425 µm sieve were therefore mixed with different proportions of the lime in water medium. Test specimens (20g) of a given soil were mixed with different percentages of lime (1, 3, 5, 9 and 13%) on dry weight basis of the soil specimen. In each test, 100mL of distilled water was added to the soil-lime mixtures. The suspensions were stirred intermittently for 1h. The pH of the suspensions was measured after 1 h ± 5 min. The pH readings do not go beyond 12.30, hence the lowest percentage of lime that gave pH of 12.30 was expressed as the ICL of the soil specimen. Fig. 3.10 presents typical ICL curves for expansive soil. The ICL values of expansive soil correspond to 3%.

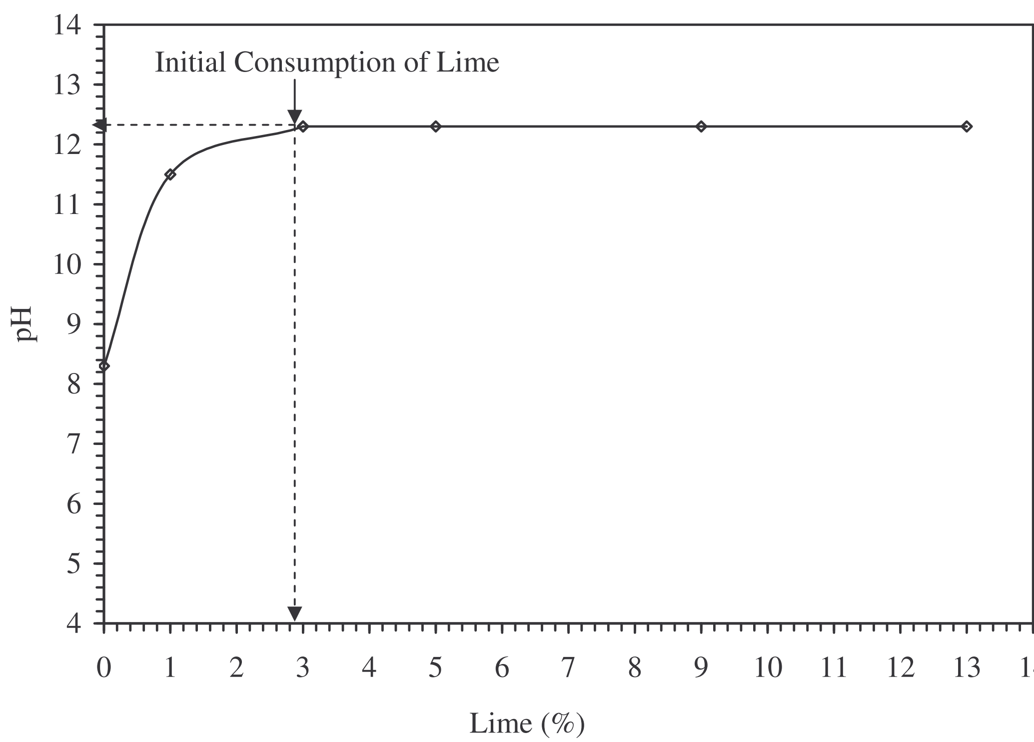

Fig. 3.10 Initial consumption of lime curve for expansive soil.

3.4.6 Specific Gravity

The specific gravity of the soil sample, G, has been determined as per the guidelines provided by ASTM D 854-06 e1. The tests were carried out using kerosene. A vacuum pump is used to removing the entrapped air from the kerosene added soil sample. For accurate determination, three sets of observation are taken, the mean of which is reported as the final value of G.

3.4.7 Compaction Test

For standard Proctor compaction test, large quantities of soil, (i.e. for each test 120 to 150 N), is required. Further, these tests involve lot of effort and time particularly in case of expansive soil. To overcome these problems a mini compaction apparatus has been designed by Prashanth (1998), Sridharan and Shivapullaiah (2005). Details of this mini compaction apparatus is presented below.

Fig. 3.11 shows the line diagram of the mini compaction apparatus. The photographic view of different components of the device is shown in Fig. 3.12. It consists of a brass mold and a steel drop hammer with guide frame. The mold is 38.1 mm in internal diameter and 100 mm in height. It has a detachable base plate and a removable collar of 35 mm height. The hammer assembly consists of a guiding frame and drop weight. The guiding frame consists of three detachable portions: top, middle, and bottom- as shown in Fig. 3.11 and Fig. 3.12. The arrangement is such that the drooping weight is a floating weight between the bottom of the top portion and top of the bottom portion of the guiding frame. The bottom portion of the guiding rod is a pedestal of 80 mm in length and 36.5 mm in diameter, which acts as an energy transferring foot. The top portion of the guiding rod is a cap of 30 mm in length and 36.5 mm in diameter which is used to hold the hammer in position before dropping it. The central portion of the guiding frame is 195 mm in length and 18 mm in diameter. The vertical rod in the middle portion acts as a guide for the hammer. The hammer is 35 mm in height and 64 mm in diameter with a central bore of 19 mm, and falls freely through a height of 160 mm over the energy transferring foot. It has a weight of 8 N.

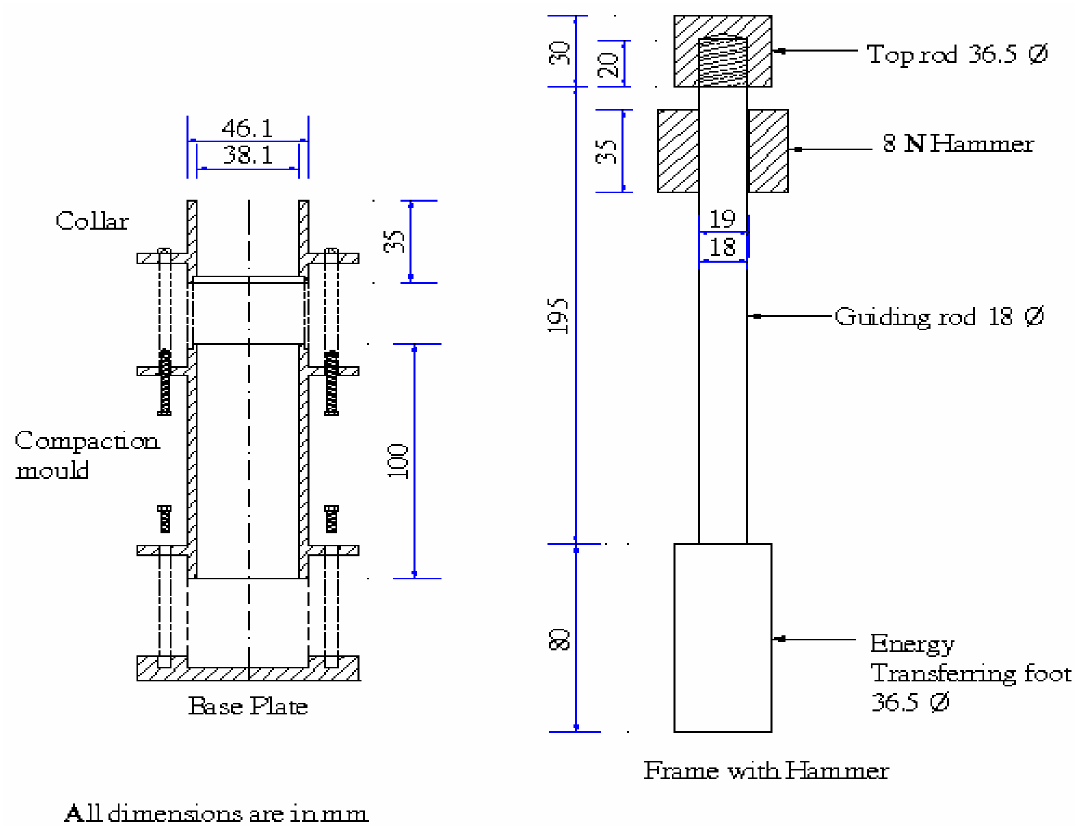

Fig. 3.11 Mini compaction apparatus (Prashanth, 1998; Sridharan and Shivapullaiah, 2005)

Fig. 3.12 Components of mini compaction apparatus

In Proctor compaction test, the hammer directly falls on to the soil and compacts it. In the present apparatus, the hammer falls on the top of the energy transferring foot of the frame and thus the kinetic energy of the hammer is transferred to the frame and the frame compacts the soil.

To determine the number of blows required to achieve Proctor compaction condition in the mini compaction apparatus; Prashanth (1998), Sridharan and Shivapullaiah (2005) carried out a seris of tests with varied number of blows. The soils used in the test program had wide variation in their physical and index properties. It was observed that invariably for all soils, 45 blows per layer (Note: total 3 layers) with hammer of 8N weight (correspondingly it is 36 blows per layer for hammer of 10 N weight), is required to achieve the Proctor compaction curve. In the present tests since hammer of 8 N weight is used, therefore to achieve Proctor compaction 45 blows per each of the 3 layer of soil in the mini compaction device was adopted.

For further verification, few tests both in standard Proctor compaction and the mini compaction apparatus were carried out in the present study. The typical match of the compaction curves from both the devices depicted in Fig. 3.13, further establishes the mini compaction apparatus. Since this mini compaction apparatus is relatively simple and needs much less soil, time and effort compared to the conventional one, in the present study it is used for all compaction tests.

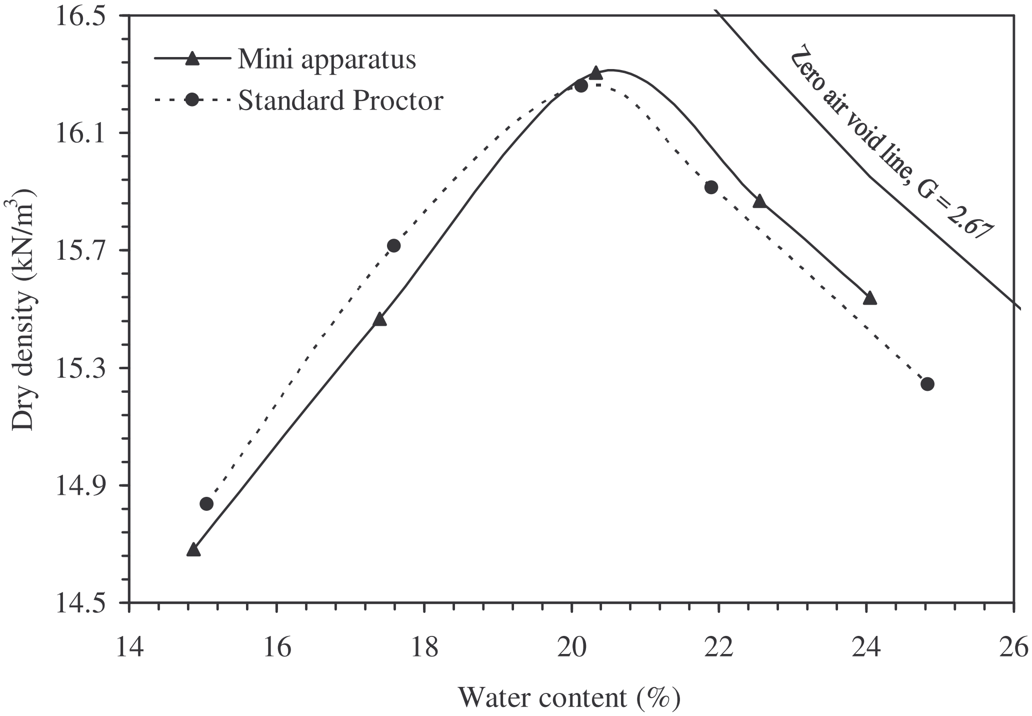

Fig. 3.13 Compaction curves of residual soil from standard Proctor and mini apparatus (present study).

The general expression for the total dry weight W of a expansive soil-residual soil–lime mixture is

$$W = W_{ES} + W_{RS} + W_l \quad 3.6$$

Where, W_{ES}, W_{RS}, W_l = weight of expansive soil, residual soil and lime respectively.

Table 3.2 gives the details of the expansive soil-residual soil-lime mixtures for the tests conducted. For each compaction test about 200 g of dry soil is required. First the desired amounts of oven dried residual soil and expansive soil were measured and mixed together in the dry state. If lime was not used, then dry expansive soil-residual soil mixture was mixed with required amount of water. If lime was used as stabilizer, first lime was dry mixed with the expansive soil-residual soil mixture. The soil was spread over a glass plate and thoroughly mixed in hand using a thin metallic plate. Then desired amount of water was added to the soil with a spray bottle and mixing was done frequently through the metal plate, manually. Care was taken to prepare homogeneous mixtures at each stage of mixing.

The mold was cleaned, dried, and greased lightly to reduce the sidewall friction. It was then fixed to the base plate. The mold with the base plate is placed on a rigid platform. The soil is compacted in the mold in three layers. Approximate quantity of the soil required for the first layer is put in the mold and then 45 numbers of blows is applied to the soil by dropping the 8 N weight hammer on the energy transferring foot of the frame. Care was taken when the hammer striked the energy transferring foot that the frame (top rod) was not in contact with hand. After the required number of blows was applied, the soil surface was scarified. The mold was filled with the soil for the second layer and again compacted. After the compaction of second layer, top collar was positioned on the mold and the third layer is placed and compacted. After the compaction, the collar is removed and excess soil is trimmed off to make even with the top of the mold. The weight of the compacted soil together with the mold is measured and the weight of the compacted soil is determined. The bulk unit weight of the soil is computed from the weight of the compacted soil and the volume of the mold. Knowing the water content and bulk unit weight, the dry unit weight is calculated. The test was repeated for the soil mixed with different water contents. A fresh sample was used every time, to obtain each compaction point of the compaction curve at different water content, both for untreated and lime treated soil.

3.4.8 Swell Test

3.4.8.1 Free Swell Test

The free swell tests were carried out as per the procedure suggested by Sridharan et al. (1986). 3 gms of oven dried powder soil sample was poured into a 100 ml graduated glass cylinder containing 40 ml distilled water. However, for low swelling soils i.e. RS, 10 gms of soil sample was used. In case of lime addition the required quantity of lime (i.e. percent of dry weight of soil) was mixed with the soil of 10 gm weight. The soil-lime mixture was added to the 40 ml water in the graduated glass cylinder. The suspensions were stirred repeatedly to remove the entrapped air and homogenise the mixture. The suspension was allowed to equilibrate for 24 hours. After this the cylinders were filled with distilled water upto 100 ml mark. The cylinders were stoppered and left undisturbed. For the soils to swell/settle and attain equilibrium state of volume. After the equilibrium state the sediment volume of soil sample in water was noted. The free swell is expressed in terms of a parameter called free swell index which is defined as

$$\text{Free swell index} = \frac{V_d}{w} \quad \text{ml / gm} \tag{3.6}$$

Where, V_d is the equilibrium sediment volume of the soil sample recorded from the graduated glass cylinder expressed in ml. w is the weight of oven dry soil in gm.

It should be mentioned here that Swell index (i.e. difference in sediment volume in water and in a non polar liquid i.e. CCl_4) gives negative value in case of nonswelling kaolinitic soils leading to error. This is due to the property of kaolinitic mineral to occupy a higher sediment volume in nonpolar liquid than in water (Sridharan et al., 1985, 1986). In view of this Free Swell Index is preferred to the Swell Index. In this study the Free Swell Index is used.

3.4.8.2 Oedometer Swell Test

The time versus swell response of the soils with and without addition of lime was obtained through conventional oedometer apparatus. The oedometer rings were 60 mm in diameter and 20 mm in height. The amount of dry soil required to produce Proctor maximum density, in the oedometer ring with soil specimen of diameter 60 mm and height of 10 mm was weighed out. It was added with required amount of water through spraying, so as to achieve the Proctor OMC as obtained from compaction tests and thoroughly mixed. In case of lime addition, lime was first mixed with the dry soil and then water was added.

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

a) the choosen compaction optimum moisture content (i.e. OMC)

b) water required for slaking of quick lime

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

Quick lime, when used for stabilization, immediately takes up 32% of its own mass of water thus significantly reducing the moisture content of the natural soil (Greaves, 1996). Addition of lime also causes an increase in the plastic limit thus causing an apparent drying due to the slaking reaction of quick lime, which is highly exothermic and the heat generated causes further water loss due to evaporation. Hence, for the present study, whenever lime was used; an additional quantity of water equal to 32% of the mass of lime, was added to take into account for slaking of lime and loss due to evaporation. The moist soils samples thus prepared were transferred to oedometer ring and were static compacted to a thickness of 10 mm. The specimen were compacted to the thickness of 10 mm so that even after full swelling the soil sample is within the oedometer ring and lateral confinement is offered. The inside surface of the rings were lubricated to minimise side friction between the ring and the soil specimen. Filter papers were placed on top and bottom of soil samples. The soil specimen along with the filter papers were placed between two porous stones, at its top and bottom. Filter papers placed at top and bottom of the soil sample are for preventing the soil particles getting into the porous stones. The whole assembly was mounted in the consolidation cell and placed in the loading frame. The specimen was loaded to a normal pressure of 5 kPa, inundated in distilled water and was allowed to swell till 30 days. It was observed that for the case with 100%ES swell continued to take place even after 30 days. Since for lime added soil most of the pozzolinic reaction is expected to be over by 28 days. Swell test for all soils were carried out till 30 days uniformly. Swelling was recorded at regular intervals, using dial gauges of 0.002 mm least count. The percent swell at any interval of time is defined as follows

$$\% \text{ Swell} = \frac{\Delta h}{h_i} \times 100 \qquad 3.7$$

Where, Δh = increase in specimen thickness at a given time.

h_i = initial thickness of the sample.

A typical time versus percent swell plot for the soil (40%ES + 60%RS) is shown in Fig. 3.14.

After 30 days, the swollen specimens were loaded in stages with a load increment ratio of unity and the sample was allowed to compress under each load. The loading was continued till the soil sample reached the original thickness. A typical plot of change in sample height in compression with log of pressure applied, is presented in Fig. 3.15.

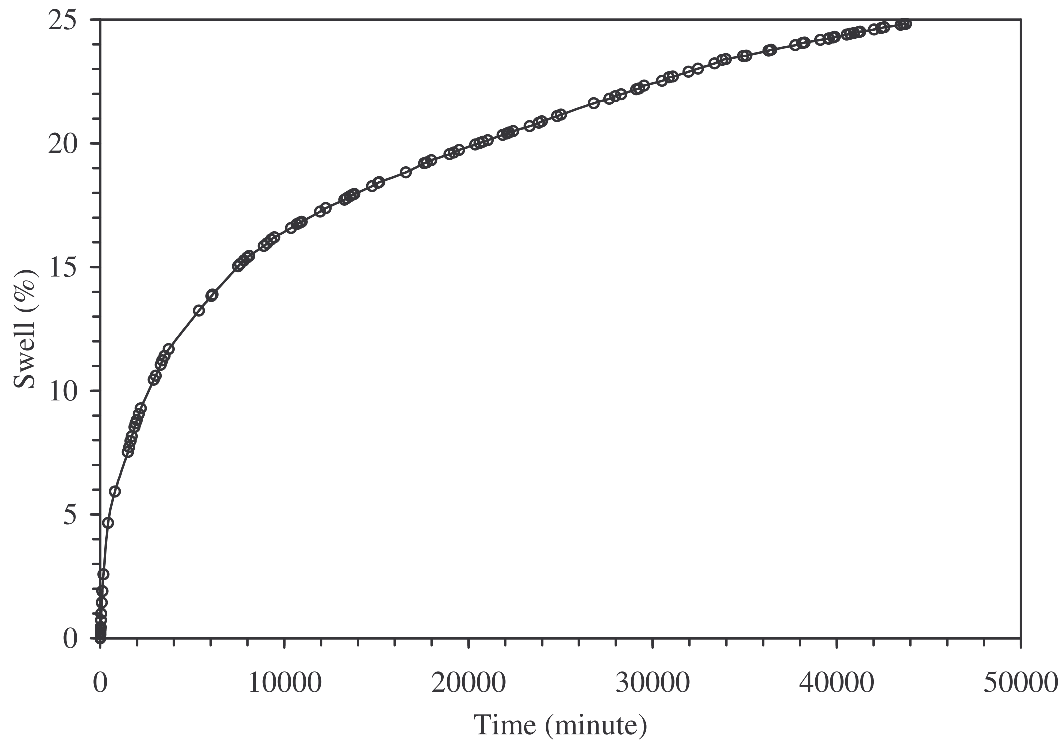

Fig. 3.14 Swell-time plot for soil (40%ES + 60%RS)

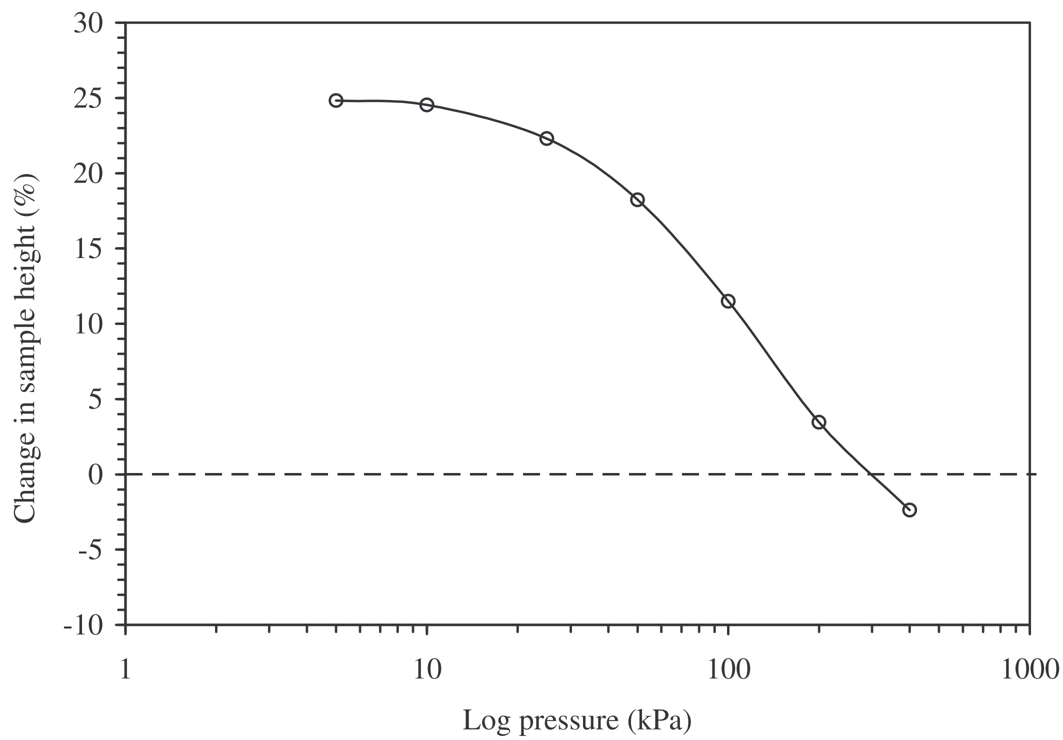

Fig. 3.15 Typical plots of percent change in sample height under compression versus pressure applied for soil (40%ES + 60%RS)

3.4.9 Shrinkage Tests

Soil specimens with and without addition of lime were prepared in the same procedure as in case of oedometer swell tests. The moist soils were transferred to PVC rings of 69 mm internal diameter and 40 mm height. The soil contained in the ring was placed on the metallic pedestal and was statically compressed to 20 mm thickness using a hand-operated static press. The weight of dry soil and water content were taken such that Proctor maximum dry density condition is attained. Dry filter paper were placed both at top and bottom of the compacted soil sample in the ring. Over the filter papers were placed, two perforated Perspex sheets having 2mm holes, one at each end, to allow free access of water. The whole assembly was placed in a tray and a normal surcharge of 5 kPa was applied onto it through dead weights. Distilled water was poured in the tray that the soil sample is submerged. The submerged soil was allowed to swell for 30 days. After this, water was taken out from tray and the swollen soil was allowed to dry at temperature of 30 ± 3^0C, while the surcharge pressure of 5 kPa was maintained. As shrinkage of soil progressed due to drying, the weight, height and diameter of the specimen were measured at every 24 hours. The diameter of the specimen was measured through vernier caliper and height through a table top dial gauge assembly. Measurement of height and diameter were taken at five different places, the average value of which are used for obtaining the volume of the specimen at different stages of shrinkage. The void ratio and water content of the soil samples, at regular intervals during shrinkage are obtained through the following relations.

$$e_0 = \frac{G\gamma_w}{\gamma_d} - 1 \qquad 3.8$$

$$\frac{\Delta V}{V_0} = \frac{\Delta e}{1+e_0} \qquad 3.9$$

$$e = e_0 - \Delta e \qquad 3.10$$

$$w = w_0 - \frac{\Delta w}{w_s} \times 100 \qquad 3.11$$

Where,

e_0 = void ratio of swollen soil

G = specific gravity of soil

γ_w = unit weight of water

γ_d = dry density of soil specimens at any stage of drying (i.e. ratio of weight of soil mass w_s, which is a constant to the volume of specimen at that stage).

V_0 = volume of swollen soil specimen

ΔV = reduction in volume of soil specimen due to shrinkage

Δe = reduction in void ratio of the soil specimen due to shrinkage

e = void ratio at any stage of shrinkage

w_0 = water content of the swollen soil specimen

Δw = reduction in weight of the soil specimen due to drying

w = water content of the soil specimen at any stage of drying

The drying process was terminated when the weight of soil specimens became nearly constant. Depending on the type of soil and lime content the time required for completion of shrinkage test varied between 15 to 30 days. A typical shrinkage response obtained from the present study is depicted in Fig. 3.16.

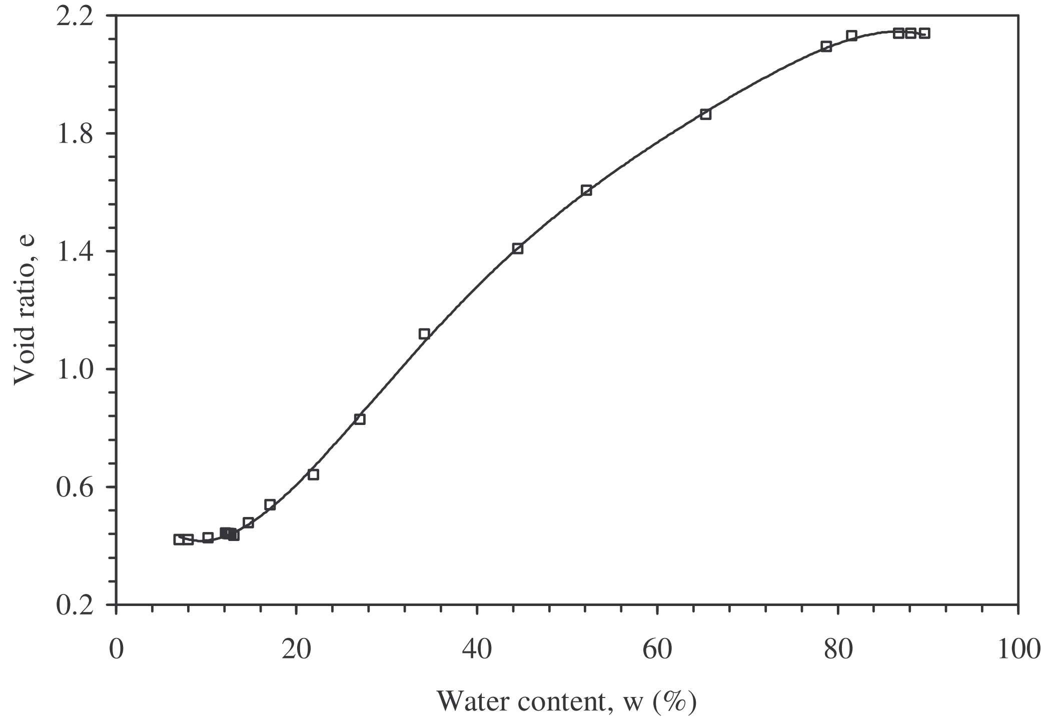

Fig. 3.16 Shrinkage response (i.e. water content vs. void ratio for soil, 80%ES+20%RS)

3.4.10 Unconfined Compressive Strength Test

The unconfined compressive tests for different soils both with and without addition of lime were carried out at their respective Proctor maximum density and optimum moisture content condition. First, the amount of dry soil required to produce Proctor maximum dry density in a sample of 38 mm diameter and 76 mm length was weighed out. It was mixed with desired amount of water and mixed thoroughly. The details of the same has already been explained in detail in the sections for compaction test and oedometer swell test.

For preparation of specimens for unconfined compressive strength test, a simple miniature static compaction tool was designed and fabricated in the laboratory to exert equal static compactive effort from both ends of the specimen (Fig. 3.17 and Fig. 3.18).

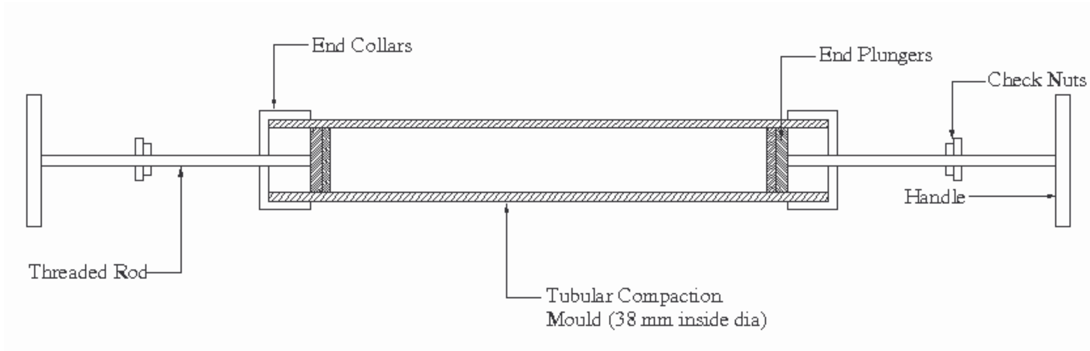

Fig. 3.17 Line sketch of the miniature static compaction tool

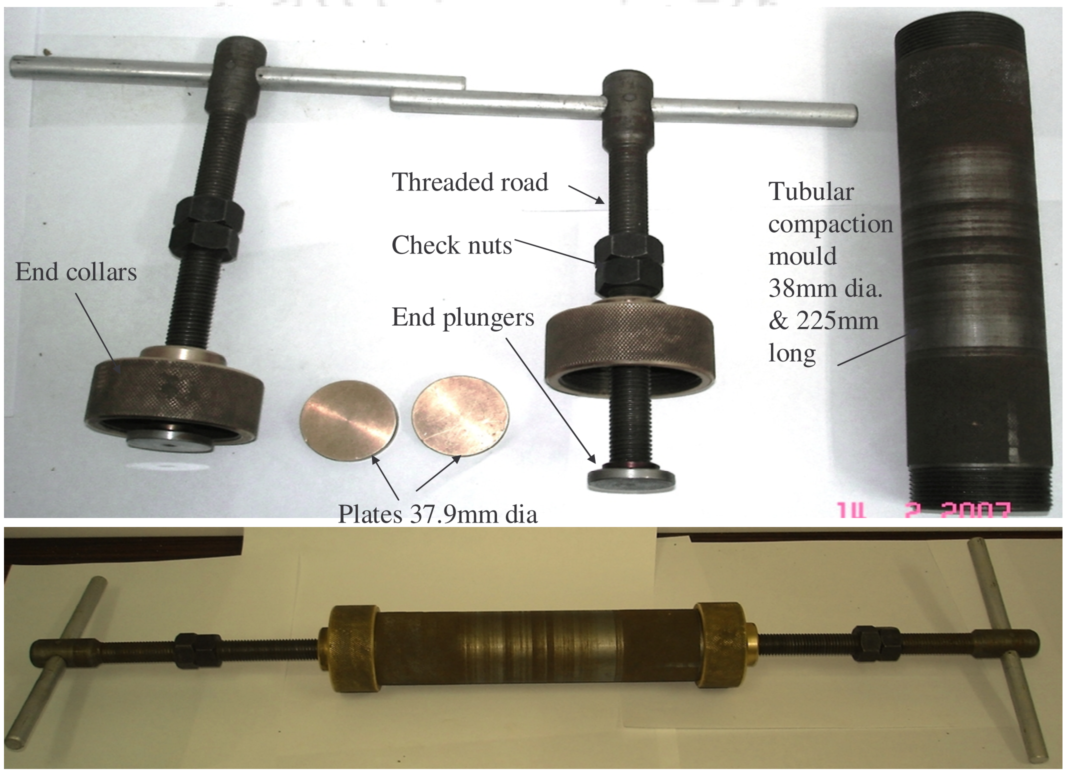

Fig. 3.18 Photographic view of the miniature static compaction tool

The tool is comprised of a tubular mould, two end-collars, and two end plungers, each connected to the handle through a threaded rod. The mould is of 38 mm internal diameter, 225 mm long, and approximately 6 mm thick with both open ends threaded externally. The end-collars are threaded internally to fit onto the mould. Each of the end-collars has a threaded central hole through which the threaded rod of the end-plunger can travel as the handle is rotated. The lengths of travel of the plungers, which in turn determines the length of the compaction specimen, can be controlled with the help of two check nuts mounted on each end of the threaded rod. Two dummy steel plates of 37.9 mm diameter are also used, one at each end, so as to isolate the ends of the specimen from the rotary movement of the plunger plate, which may otherwise cause specimen disturbance. After fixing an end-collar to one end of the mould, the required quantity of wet soil is introduced from the other end. Then, the second end collar is screwed on. Thereafter, both the end-plungers are advanced simultaneously from either mould end so as to compact the soil to the required density. After extrusion, the compacted specimens were of 38 mm in diameter and 76 mm in length.

The unconfined compressive strength test was carried out after curing the specimen for required curing period. The prepared specimen was closely wrapped in a polyethylene bag (Kaniraj et al., 2001) to prevent moisture loss and placed in a desiccator at 100% humidity. Depending on test requirement the specimens were cured for varied periods i.e. 3, 7, 21 and 28 days.

The unconfined compressive strength tests were conducted as per ASTM D 2166-06, under a constant strain rate of 1.25 mm/min. Load and deformation readings were taken upto failure of the specimen. Graph of stress versus strain were plotted and the peak stress was obtained. Typical stress-strain responses of expansive soil and residual soil obtained in the present tests are presented in Fig. 3.19.

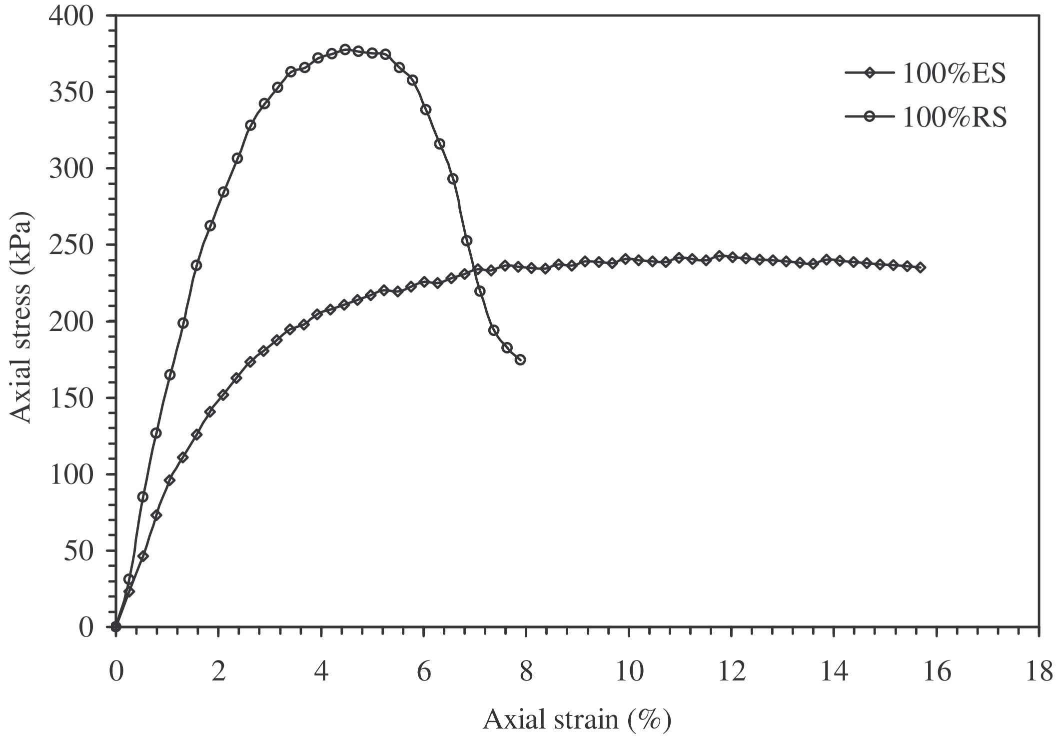

Fig. 3.19 Typical stress-strain plots of expansive soil and residual soil

3.4.11 Microstructural Test

3.4.11.1 Scanning Electron Microscope (SEM)

The Scanning Electron Microscope (SEM) with Energy Dispersive X-ray analysis (EDX) was employed to study the micro-structural characteristics in the matrix of the soil and lime stabilized soils used in the present study.

A piece of soil sample was taken out from the compacted soil specimens and was oven dried completely. The dried soil samples were mounted in phenolic resin base with the help of a mounting device. The mounted sample were coated with a thin layer of gold palladium to provide surface conductivity. Polaron sputter coater, operating under a vacuum of 10^{-2} bar and 4mA current, was used for this purpose. The coated specimens were mounted in a commercial SEM-EDX instrument (LEO, 1430VP). Both spot and overall EDX analysis were performed on the samples. The SEM-EDX was operated with an acceleration potential of 15 kV and each spectrum was collected for about 100s (acquisition time). The photomicrographs were taken at different magnifications. Typical photomicrograph and EDX pattern are shown in Fig. 3.1 and Fig. 3.2 respectively.

3.4.11.2 X-ray Diffraction (XRD)

In the present investigation, the X-ray diffraction technique has been used for identification of various minerals present in the lime treated and untreated expansive soil-residual soil mixture. Powder X-ray diffraction (XRD) tests were carried out at room temperature using a commercial X-ray diffractometer (Phillips 2404, Holland) by employing graphite monochromator and CuKα radiation (1.5418Å). In the present investigation, all the XRD data were collected with the setting of 30 mA current and 40 kV voltage for X-ray generator. The instrument is based on the Bragg-Brentano geometry. In this geometry, the source to sample distance and the sample to detector distance are kept equal. A perspex sheet with rectangular groove was used for sample mount where the powder sample was filled uniformly in the groove. The data were collected in a usual θ-θ scan with an angular speed 2^0 per minute and a step size of 0.03-0.05. Data obtained by the diffractometer were analyzed with Jade 3.1, a X-ray powder diffraction analytical software, produced by the Materials Data, Inc. *(Jade 3.1, Manual, Materials Data, Livermore, CA, 94550, Copyrights, 1995-1999).*

Tests were conducted on representative samples after the completion of the Unconfined Compressive Strength (UCS) test. The material was air dried powdered and used for XRD analysis. Typical XRD pattern are shown in Fig. 3.3 in section 3.2.1.1.

3.5 SUMMARY

In this chapter the test materials, evaluation of their properties and characterization has been presented. The details of sample preparation and test procedures are discussed. The details of planning of tests have been presented. The results are analysed and discussed in the subsequent chapters.

CHAPTER 4

PLASTICITY BEHAVIOUR

4.1 INTRODUCTION

The workability of a soil is closely related with its plasticity characteristics that primarily depends on the water holding capacity of the soil and is quantified through index properties such as liquid limit, plastic limit and plasticity index. The expansive soils due to their large water holding capacity are highly difficult material to be handled in construction sites. This problem can be overcome by altering the soil through addition of non expansive soil and chemicals such as lime, cement etc. This chapter presents the results of the experiments aimed at understanding the plasticity behaviour of a highly expansive bentonite soil (ES) modified through addition of a non expansive residual soil (RS) and lime. The influence of the time-dependent effects of pozzolanic reactions on the plasticity characteristics are brought out through a series of tests carried out at different curing periods.

4.2 PLASTICITY BEHAVIOUR OF ES-RS MIXES

The liquid limit (LL), plastic limit (PL) and plasticity index (PI) of expansive soil-residual soil samples are depicted in Table 4.1. It could be observed that the consistency limits (LL, PL) of expansive soil have reduced with addition of residual soil. The liquid limit has shown a sharp reduction while the reduction in plastic limit is relatively mild. Correspondingly the plasticity index has undergone visible reduction. Primarily the plasticity characteristics of soils are governed by the amount and quantity of clay minerals present in the soil. With replacement of the highly charged montmorillonite clay mineral present in the expansive soil by the relatively less charged kaolinitic clay mineral present in the residual soil, the soil (ES-RS mix) tends to have less charge deficiency thereby reducing the thickness of diffuse double layer and hence reduced plasticity (Sridharan et al., 1986). Indeed the data presented in Table 4.2 shows that the cation exchange capacity of the soil has reduced with increase in residual soil content. The reduced fines content and specific surface area (Table 4.2) indicates reduced water holding (adsorption) capacity of the soil giving rise to reduced plasticity. However, the plasticity characteristics of the soils depicted in the plasticity chart (Fig. 4.1) indicates that even with 80% residual soil content (20% expansive soil) the soil is a high plastic clay (CH), prone to high swelling and shrinkage. Further improvement can be achieved through addition of lime that causes considerable change in the pore fluid charge and hence the diffuse double layer. This aspect is explored in the following section.

Table 4.1 Index properties of expansive soil-residual soil mixes

Soil	Liquid limit (%)	Plastic limit (%)	Plasticity Index (%)
100%ES	459.9	53.7	406.2
80%ES+20%RS	351.9	49.4	302.5
60%ES+40%RS	248.2	38.1	210.1
40%ES+60%RS	178.8	30.7	148.1
20%ES+80%RS	99.2	28.7	70.5
100%RS	45.3	25.9	19.4

Table 4.2 Physico-chemical properties of expansive soil-residual soil mixes

Soil	Fines content (%)	SSA (m^2 /gm)	CEC (meq/100gm soil)
100%ES	100	86.45	69.12
80%ES+20%RS	96.6	83.24	67.9
60%ES+40%RS	93.2	65.41	51.75
40%ES+60%RS	89.8	55.32	42.1
20%ES+80%RS	86.4	49.15	29.64
100%RS	83	38.76	12.43

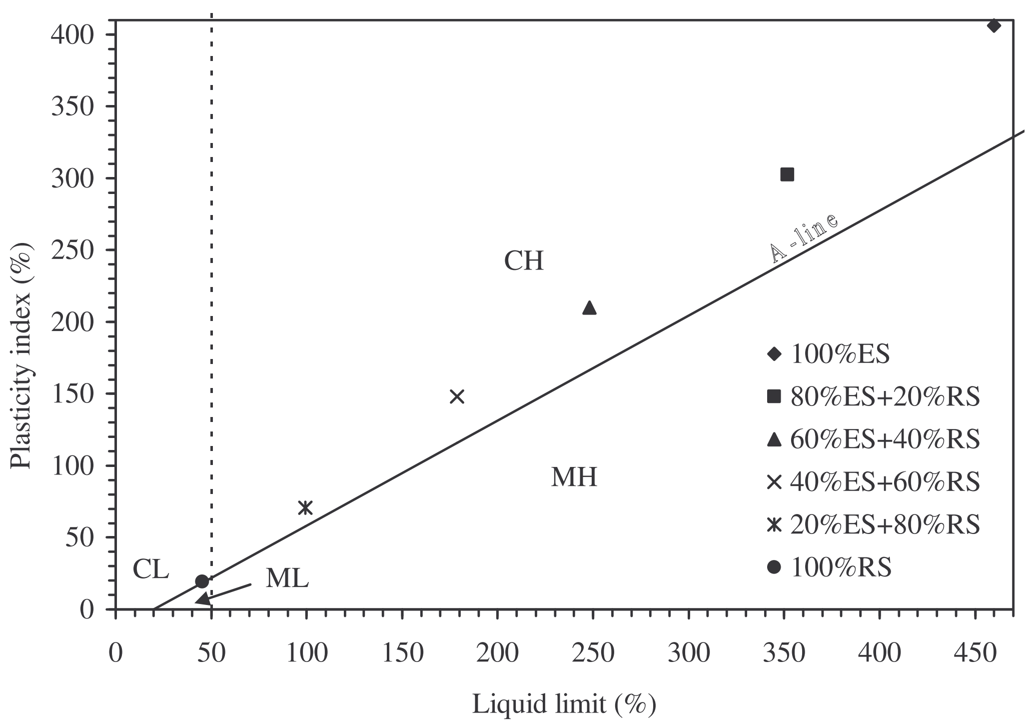

Fig. 4.1 Plasticity characteristics of expansive soil-residual soil mixes (ASTM D2487).

4.3 PLASTICITY BEHAVIOUR OF ES-RS-LIME MIXES

4.3.1 Variation of Liquid Limit

The variation of liquid limit with lime content and curing period for the soils; 100%ES, 80%ES+20%RS, 60%ES+40%RS, 40%ES+60%RS, 20%ES+80%RS and 100%RS are depicted in Fig. 4.2 to Fig. 4.7 respectively. It could be observed that, in general, for all soils initially there is a decrease in liquid limit with increases in lime content. This reduction is maximum for expansive soil (100%ES) and gradually reduces with increased content of residual soil. With addition of lime (i.e. CaO) Ca^{+} ions are released into the pore fluid. As a result of which the electrolyte concentration of the pore fluid increases that reduces the thickness of the diffused double layer leading to reduced liquid limit.

For expansive soil the liquid limit continues to reduce till 3% lime content, beyond which the increased lime content has marginal effect on the liquid limit. However, at very high lime content (i.e. 13%) and long curing period (i.e. 28 days) the liquid limit of the expansive soil has shown an increasing trend. This increasing trend gradually grows more prominent with increased percentage of residual soil and curing period (Fig. 4.3 to Fig. 4.7). It is of interest to note that irrespective of soil type the liquid limit is large for increased curing period. Hence it could be said that it is the residual soil which in association with lime has increased the liquid limit. Since this occurs after certain period of curing it is expected that certain physico-chemical reaction between soil and lime is responsible for this phenomenon.

Fig. 4.8 shows that with addition of lime the pH of soils increases. With about 3% of lime the pH value is about 12.2 beyond which it remains nearly same. In the initial stages the lime induced calcium ions are used to neutralise the charge on clay surfaces. Once this requirement is over the additional calcium ions, due to increased lime content, move freely in the pore fluid increasing its pH. At one stage saturation takes place and the pH continues to remain at about 12.2. In an alkaline environment with pH value above 12, there occurs formation of Calcium Silicate Hydrate (C-S-H) gel, which consists of solid products of hydration and water, that is held physically or adsorbed on surface of the hydrates. This water called gel water is located between the solid products of hydration in the gel pores of very small size (i.e. about 2μm). It has been established that the volume of gel water is as high as 28 percent of the volume of gel (Neville and Brooks, 2004). In addition to gel water there exists water which is combined chemically or physically with the hydrates. This large amount of water significantly marginalizes the influence of the double layer reduction induced decrease in water content and thereby the liquid limit. Indeed, the increase in liquid limit with increase in lime content is more incase of residual soil that has high silica content leading to formation of higher quantity of CSH gel thereby the increase in gel water content and hence the liquid limit. Hence it can be said that soil having high silica content are prone to have increased liquid limit, due to lime induced pozzolanic reactions. With increased duration of curing thereby increased duration of pozzolanic reaction produces increased quantity of the water holding gelatinous products leading to increased liquid limit.

The other factor responsible for increase in liquid limit could be attributed to a possible change in soil fabric. The increase in pH due to addition of lime imparts increased negative charge to the particle edges (Taylor, 1959). The increased negative charge at the edge induces increased edge to face attraction of clay particles leading to flocculated structure. The flocculant fabric being a relatively open structure holds more water leading to increased liquid limit. Similar behaviour has been reported by Sivapullaiah et al. (2000) and Raju (1991), Mathew and Narasimha Rao (1997). This fabric change induced increase in the liquid limit will be discussed in more detail in chapter 6.

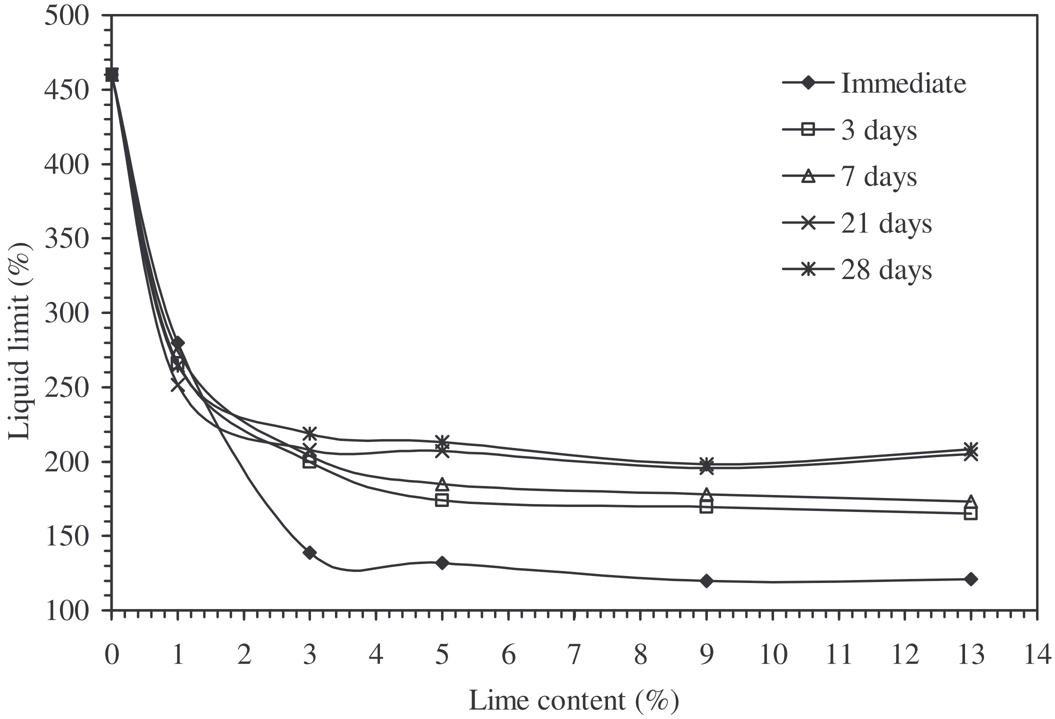

Fig. 4.2 Variation of liquid limit with lime content for expansive soil (100%ES)

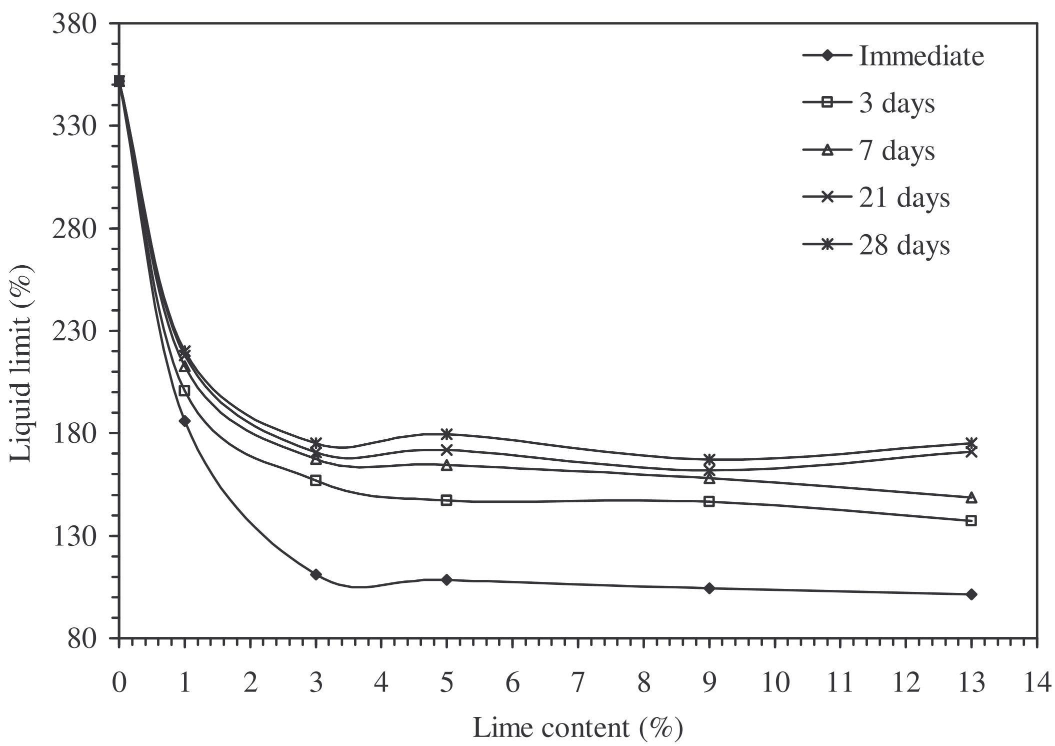

Fig. 4.3 Variation of liquid limit with lime content for expansive soil-residual soil mix (80%ES+20%RS)

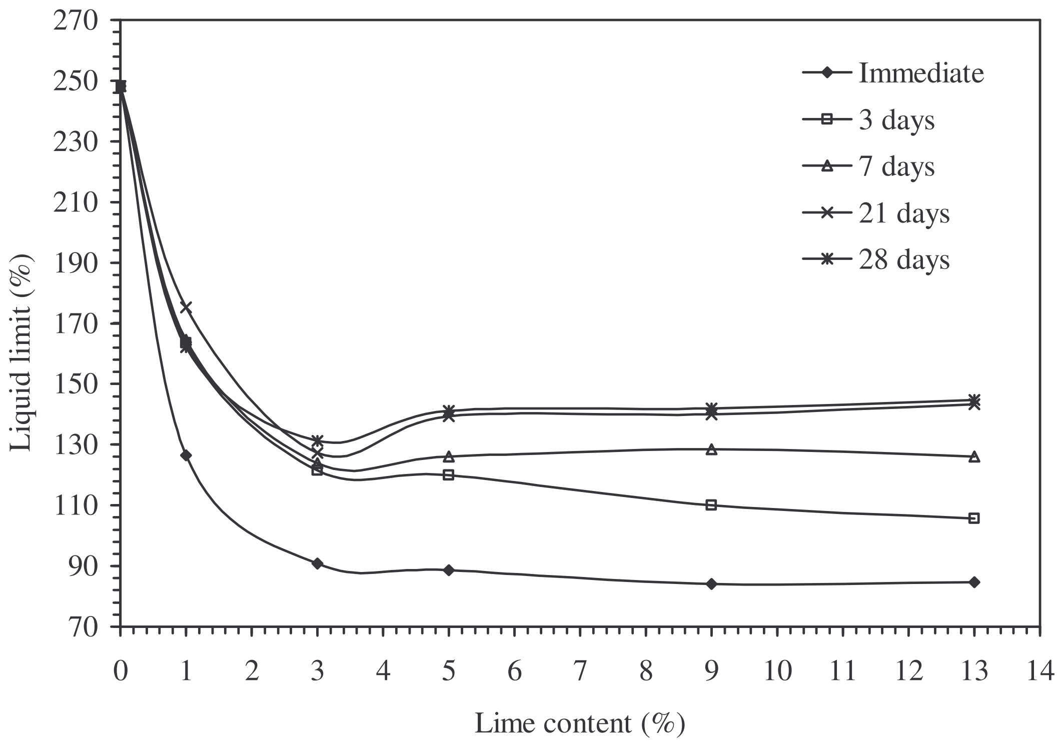

Fig. 4.4 Variation of liquid limit with lime content for expansive soil-residual soil mix (60%ES+40%RS)

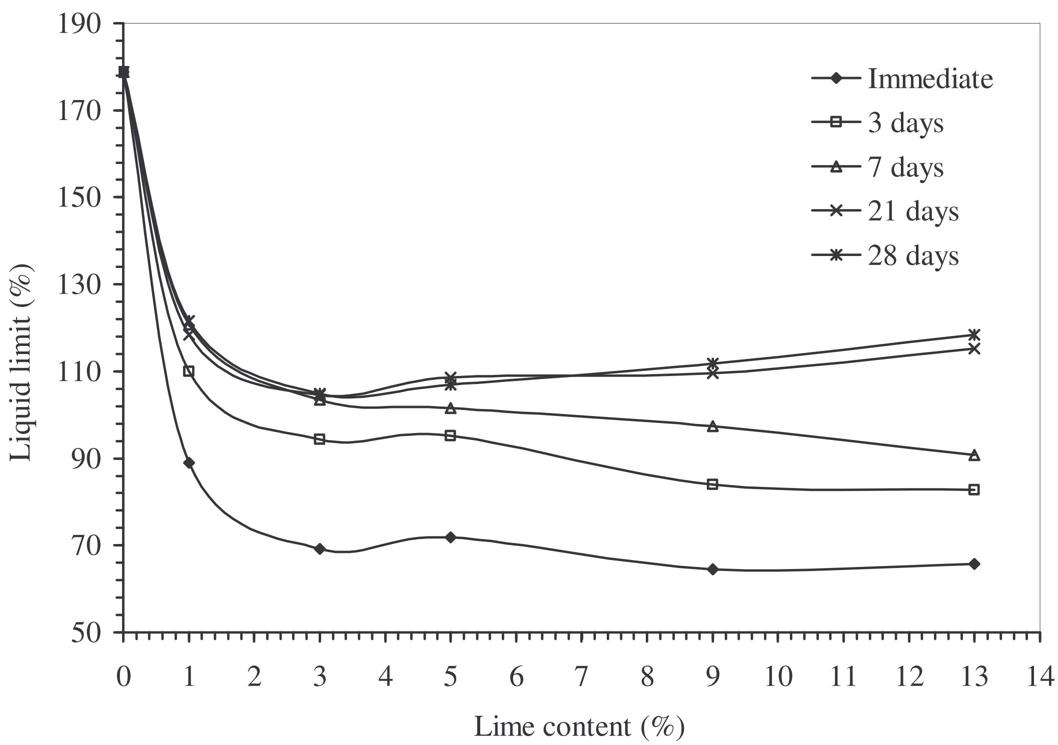

Fig. 4.5 Variation of liquid limit with lime content for expansive soil-residual soil mix (40%ES+60%RS)

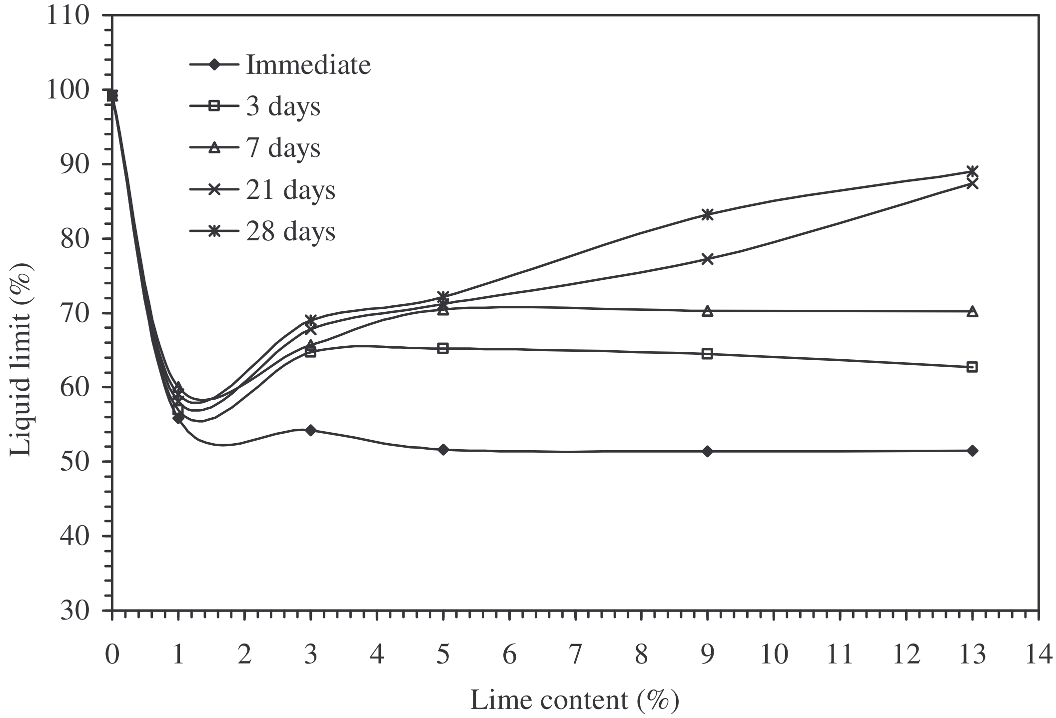

Fig. 4.6 Variation of liquid limit with lime content for expansive soil-residual soil mix (20%ES+80%RS)

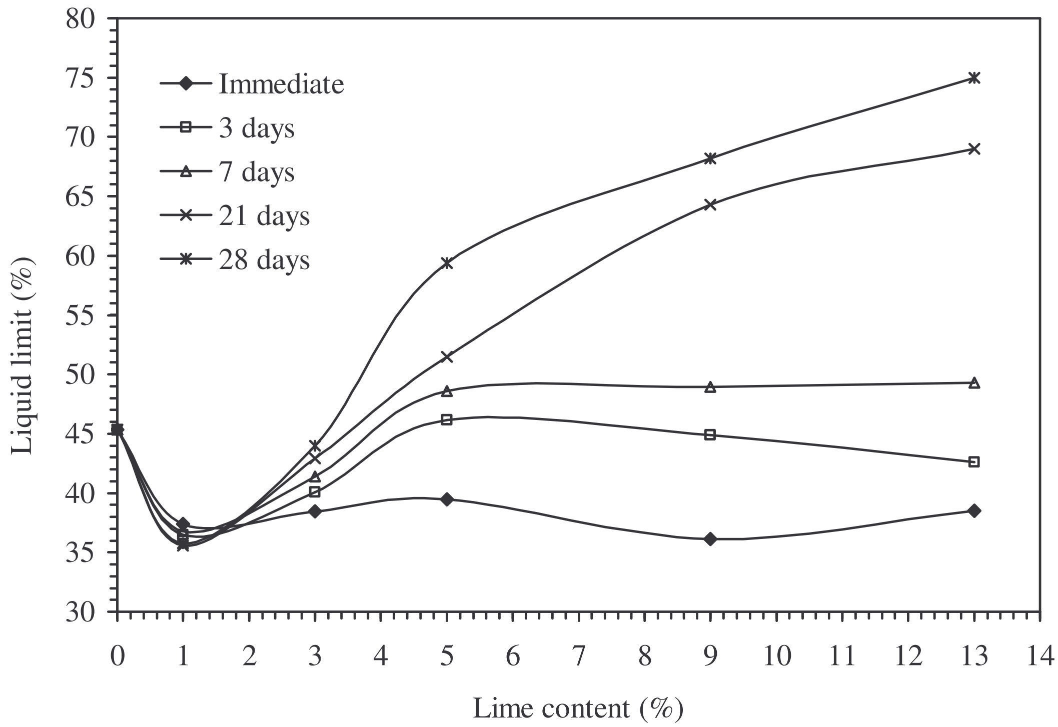

Fig. 4.7 Variation of liquid limit with lime content for residual soil (100%RS)

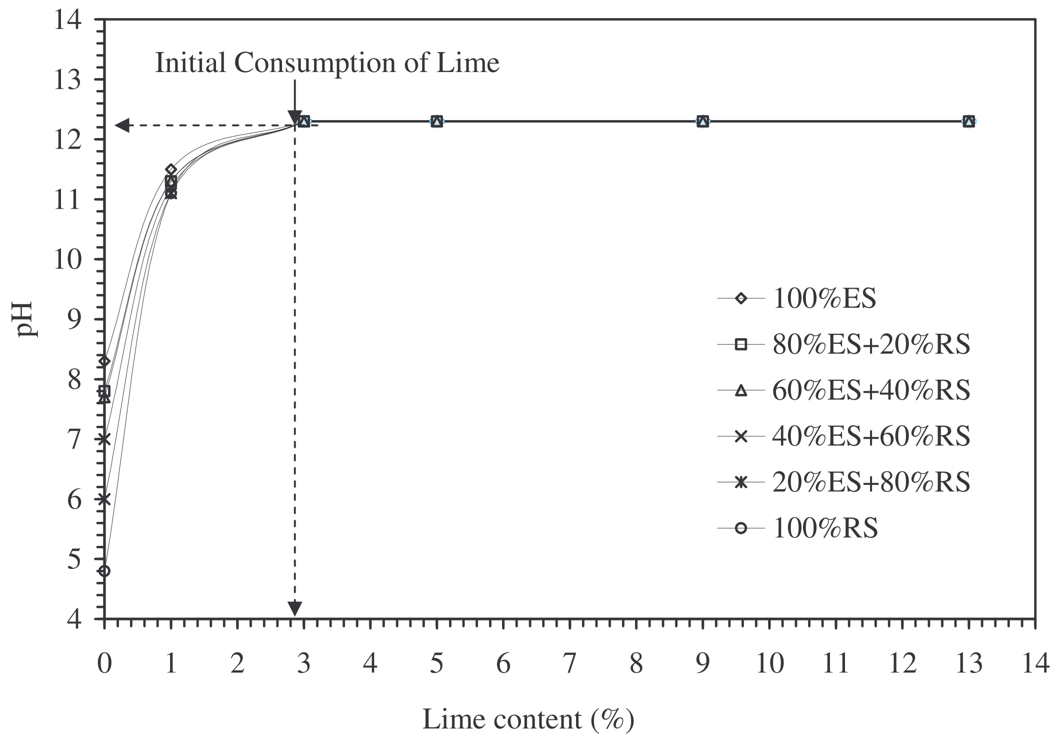

Fig. 4.8 Initial consumption of lime curves for soils.

Hence it can be said that the liquid limit behaviour of lime treated soil has three different phases. The first phase is reduction in thickness of diffuse double layer which takes place quickly leading to reduced liquid limit. The second phase is the increase in liquid limit due to fabric changes giving rise to flocculated structures. The last phase is the pozolanic reactions producing water holding gelatinous materials inducing increased liquid limit. This phenomenon is dominant incase of silica rich soils.

4.3.2 Variation of Plastic Limit

Fig. 4.9 to Fig. 4.14 depict the influence of quantity of lime added and time period of curing on the plastic limit of expansive soil (100%ES)-residual soil (100%RS) respectively. It could be observed that the plastic limit for all soils increases with increase in lime content. The increase is relatively faster till lime content reaching about 3%. From 3 to 5% of lime content the rate of increase in plastic limit is relatively slow. Beyond 5% lime content visible increase in plastic limit is noticed only for increased percentage of residual soil and higher curing period. For other cases (RS < 80%, curing period < 21 days), increase in plastic limit beyond 5% lime content is marginal.

Plastic limit is a measure of cohesion of the soil particles against cracking when the soil is worked with (Yong and Workentin, 1975). The cohesion and hence the shear strength between the soil particles should be low enough that the particles slide part over each other at ease. However, at the same time the inter particle shear strength should be high enough to hold the soil mass in the remolded position. Hence it could be said that the plastic limit is a measure of the water content of the soil when it approaches a certain shear resistance.

With addition of lime the thickness of the diffuse double layer reduces, however, due to increased charge concentration the viscosity of the pore fluid increases. This increases the inter particle shear resistance leading to a sharp increase in the plastic limit of the soil. With further addition of lime beyond 3% the pH of the pore water increases above 12. The alkaline environment gives rise to increased negative charge at edge of the soil particles that leads to flocculation. With increased flocculation the interparticle resistance against movement increases leading to increased plastic limit, with lime content increasing beyond 3%. Beyond 5% lime content the threshold has reached and therefore the plastic limit do not change much. Hence practically 5% lime content can be considered as lime fixation point, that can provide substantial increase in the workability of the soil. The increase in plastic limit beyond this limit (i.e. lime content > 5%) for the silica rich soils, is attributed to the formation of water holding cementious gelatinous compounds (CSH). With increase in curing period the pozzolanic reaction continues to form more such products leading to increased plastic limit.

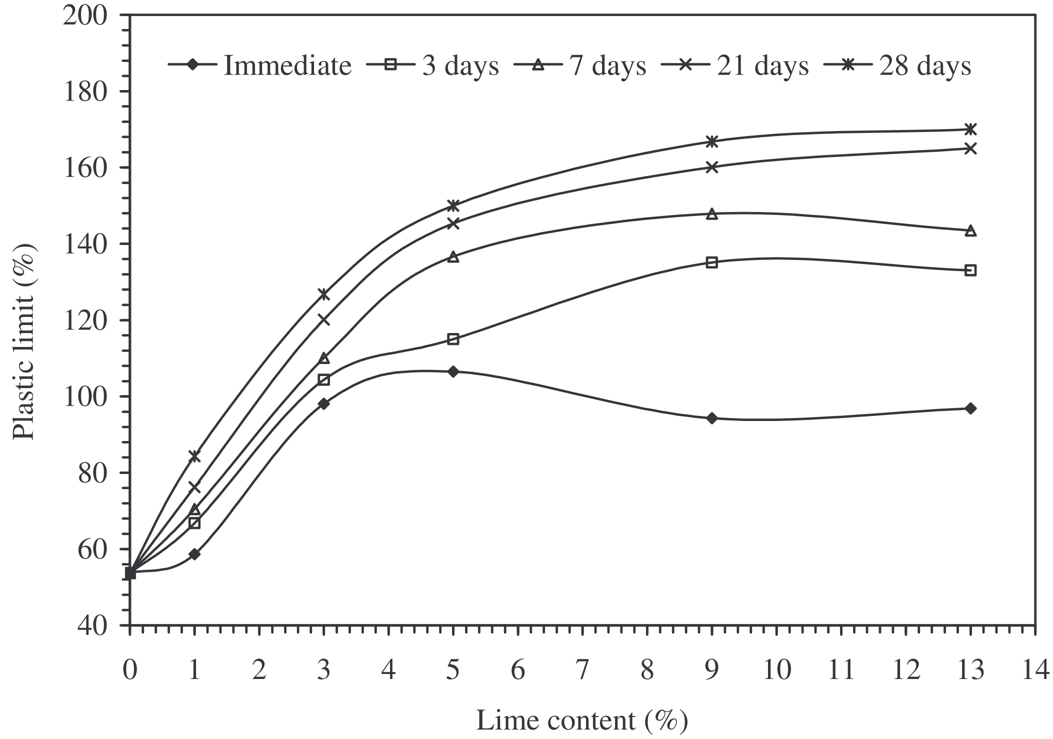

Fig. 4.9 Variation of plastic limit with lime content for expansive soil (100%ES)

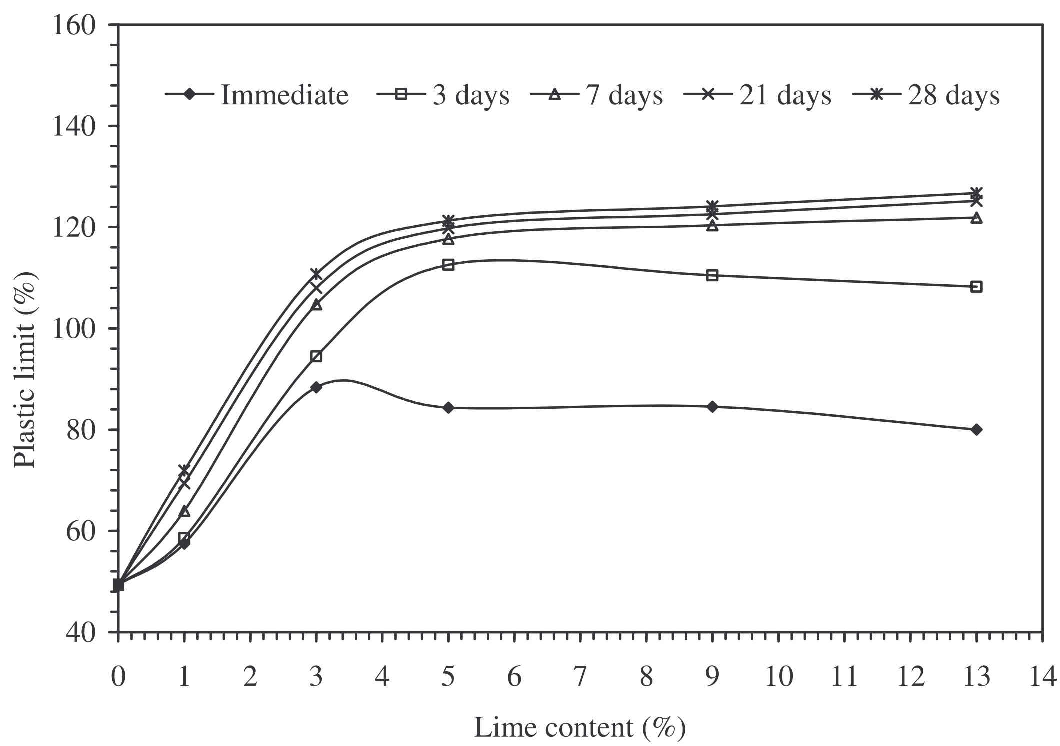

Fig. 4.10 Variation of plastic limit with lime content for expansive soil-residual soil mix (80%ES+20%RS)

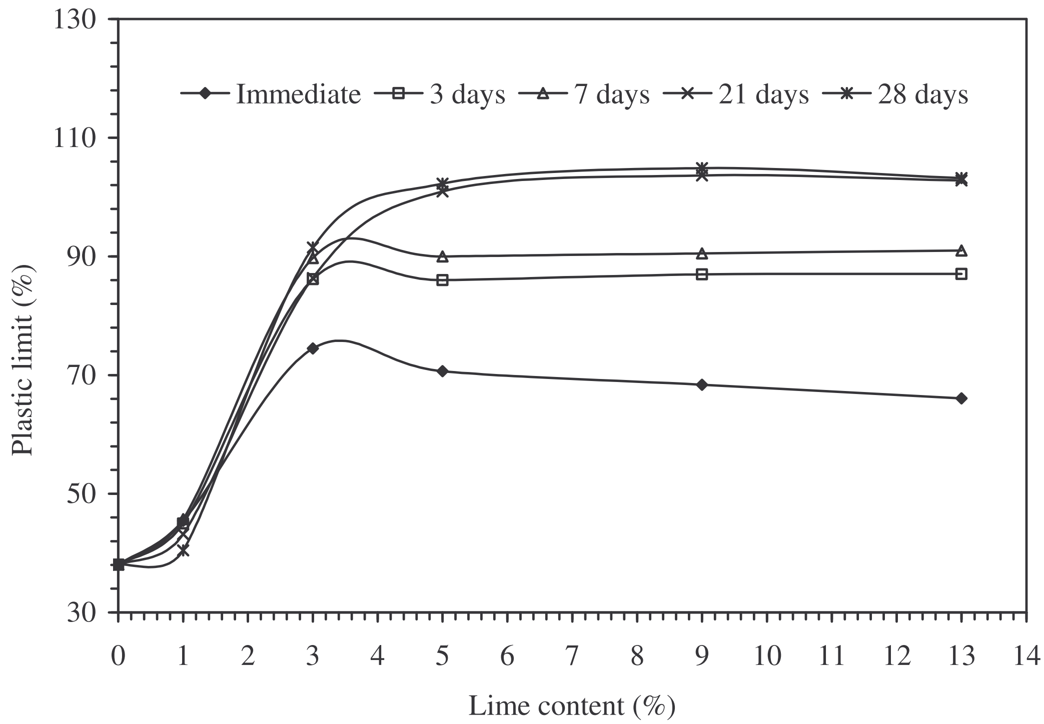

Fig. 4.11 Variation of plastic limit with lime content for expansive soil-residual soil mix (60%ES+40%RS)

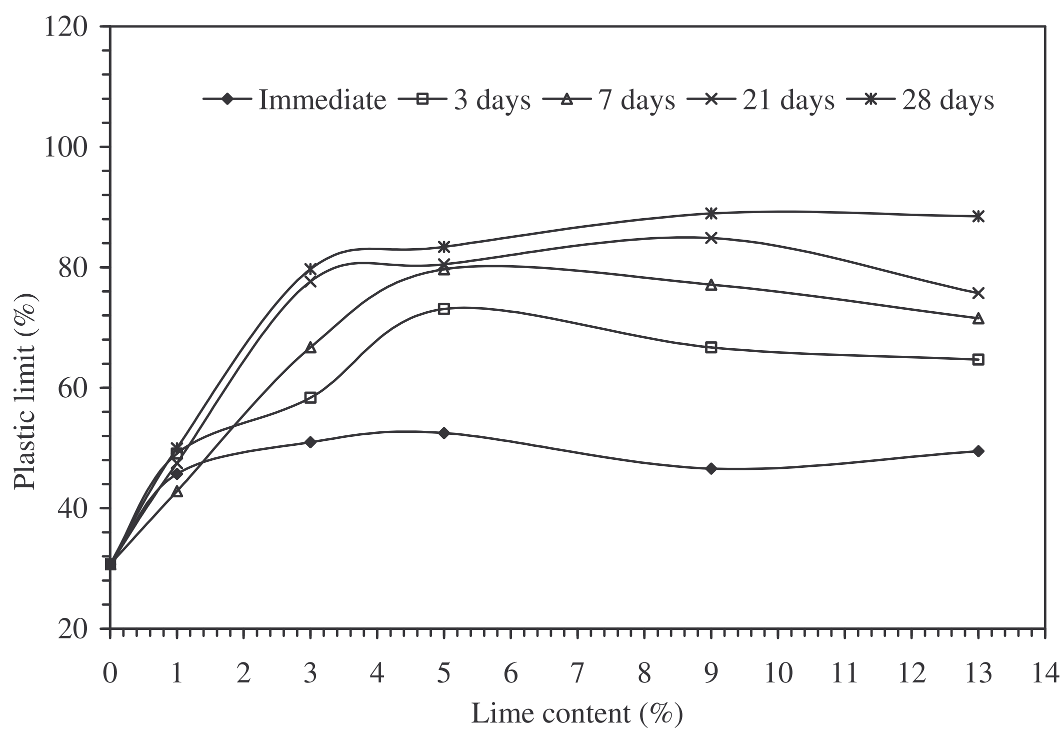

Fig. 4.12 Variation of plastic limit with lime content for expansive soil-residual soil mix (40%ES+60%RS)

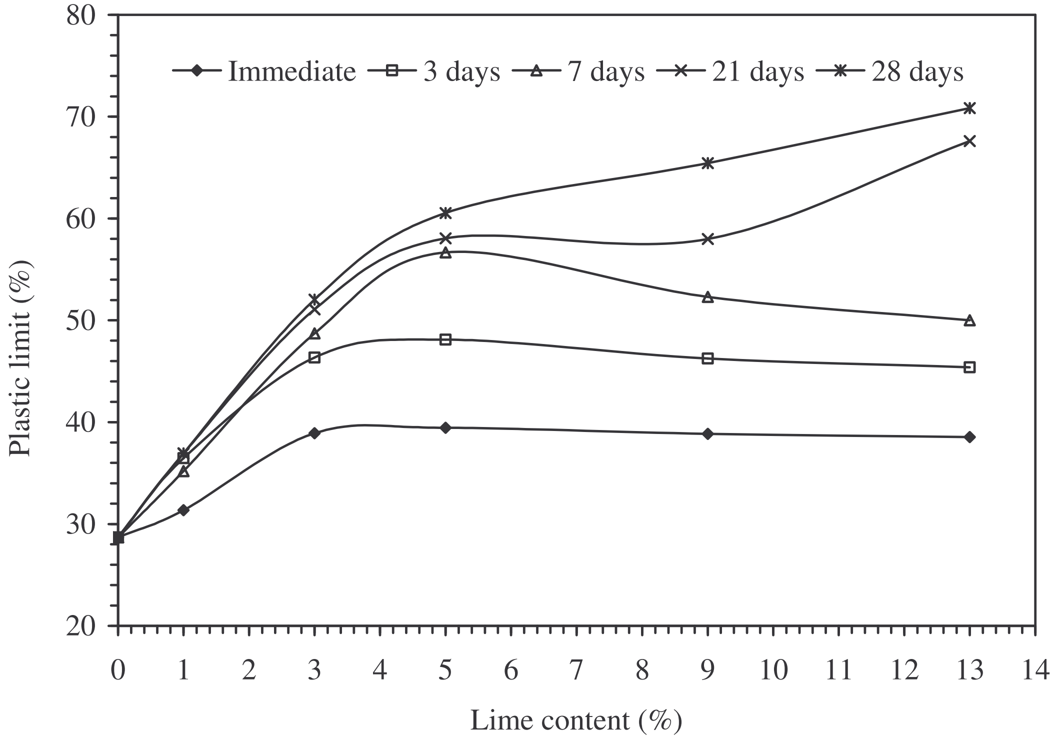

Fig. 4.13 Variation of plastic limit with lime content for expansive soil-residual soil mix (20%ES+80%RS)

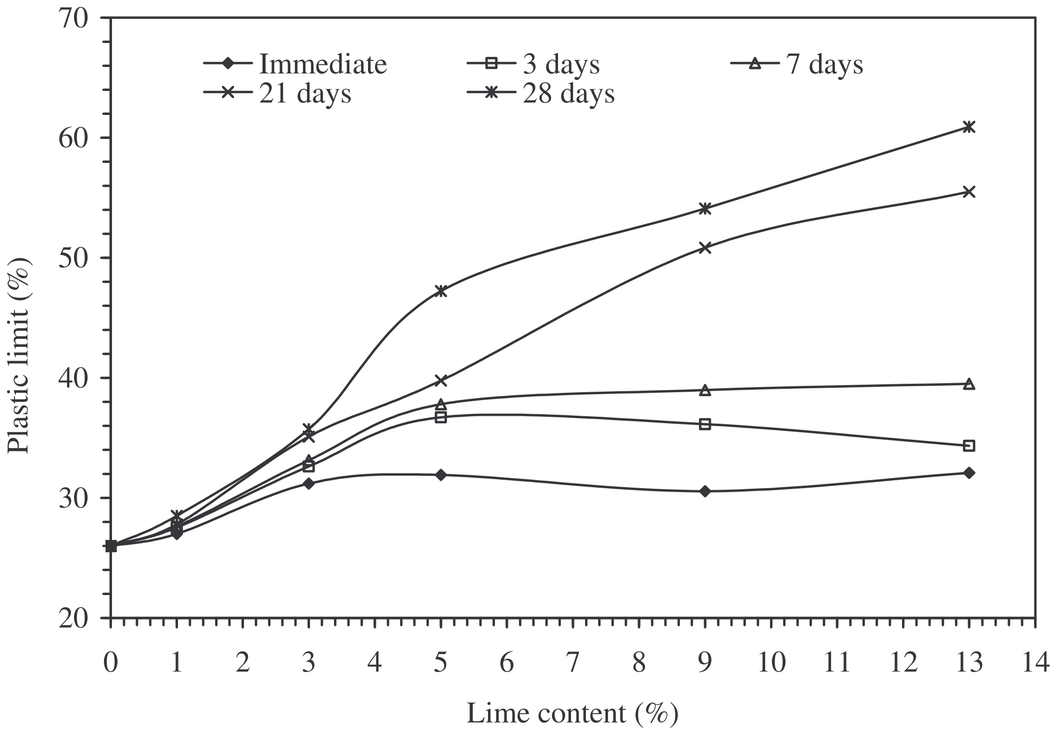

Fig. 4.14 Variation of plastic limit with lime content for residual soil (100%RS)

4.3.3 Variation of Plasticity Index

The plasticity index (PI) of a soil indicates its water holding capacity and thereby is a measure of its workability. In general the decrease in plasticity index indicates an improvement in the workability of the soil (Sivapullaiah et al., 1996). Since plasticity index is derived from liquid limit and plastic limit (PI = LL – PL) the mechanisms for liquid limit and plastic limit hold good for the plasticity index. The plastic limit variations for different soils are depicted in Fig. 4.15 to Fig. 4.19 respectively.

It could be observed that, in general, soils have shown an immediate decrease in plasticity index upon addition of lime. For 100%ES and 80%ES+20%RS, increasing the lime content beyond 5% had a marginal effect in further reducing the plasticity index (Fig. 4.15 and Fig. 4.16). This limit is about 3% for 100%RS soil (Fig. 4.20). This is in line with earlier observations that immediately upon addition of lime the liquid limit reduces and plastic limit increases. In general the plasticity index increases with increase in curing period however it is more prominent for the increased percentage of residual soil. This is attributed to the silica gel that enhances the water holding capacity of the soil.

With lime treatment the plasticity index of the expansive soil has reduced from about 400% to 50%. This substantial reduction in plasticity index suggests that the soil itself has changed. Indeed Fig. 4.21 shows that in most of cases the lime treated soils have crossed the A-line to the silt (M) region indicating that the clay soil upon lime treatment has transformed itself to silt. Exceptions were for the specimens with 1% lime indicating that the low quantity of lime is not sufficient enough to convert the high plastic clays (CH) to silt. The detailed classifications reported in Table 4.3 shows that the soils, 100%ES to 20%ES+80%RS, which were initially high compressible clay (CH), with 3% lime, even with no curing, turn to silt of high plasticity (MH). Further curing does not change the soil classification. Contrary to this the 100%RS soil which was originally clay of low compressibility (CL), initially changes itself to silt of low compressibility (ML), however, with higher quantity of lime and prolonged curing period it changes again to silt with high compressibility (MH). This is due to the formation of silica gel which increases the liquid limit of the soil and hence increases its compressibility.

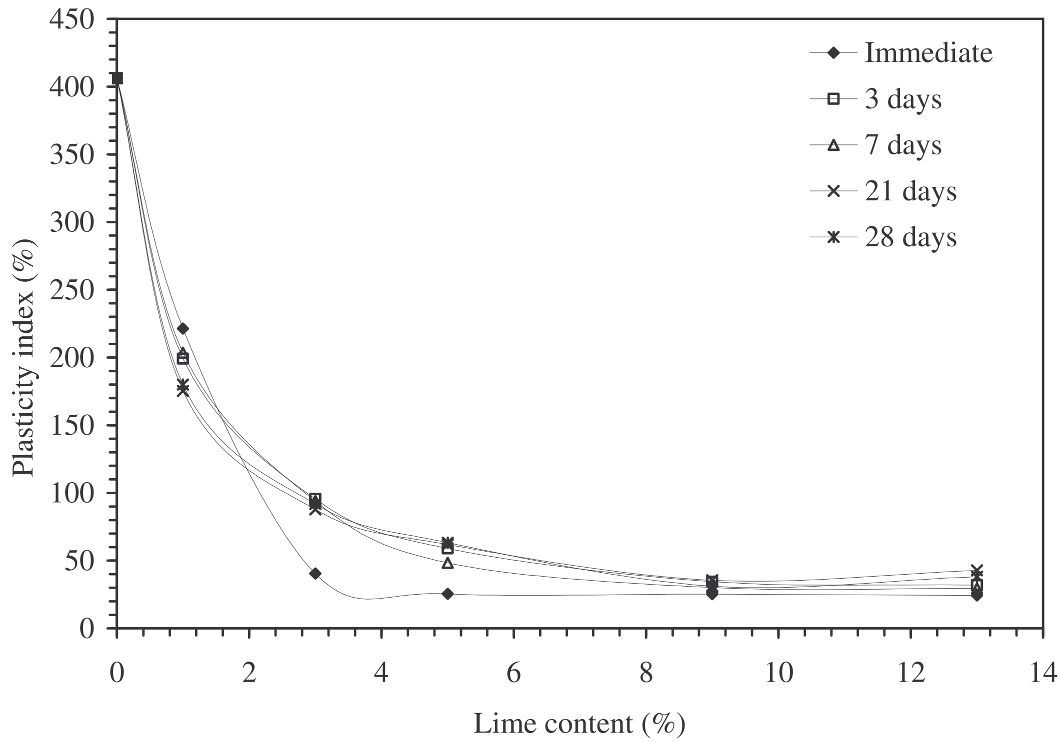

Fig. 4.15 Variation of plasticity index with lime content for expansive soil (100%ES)

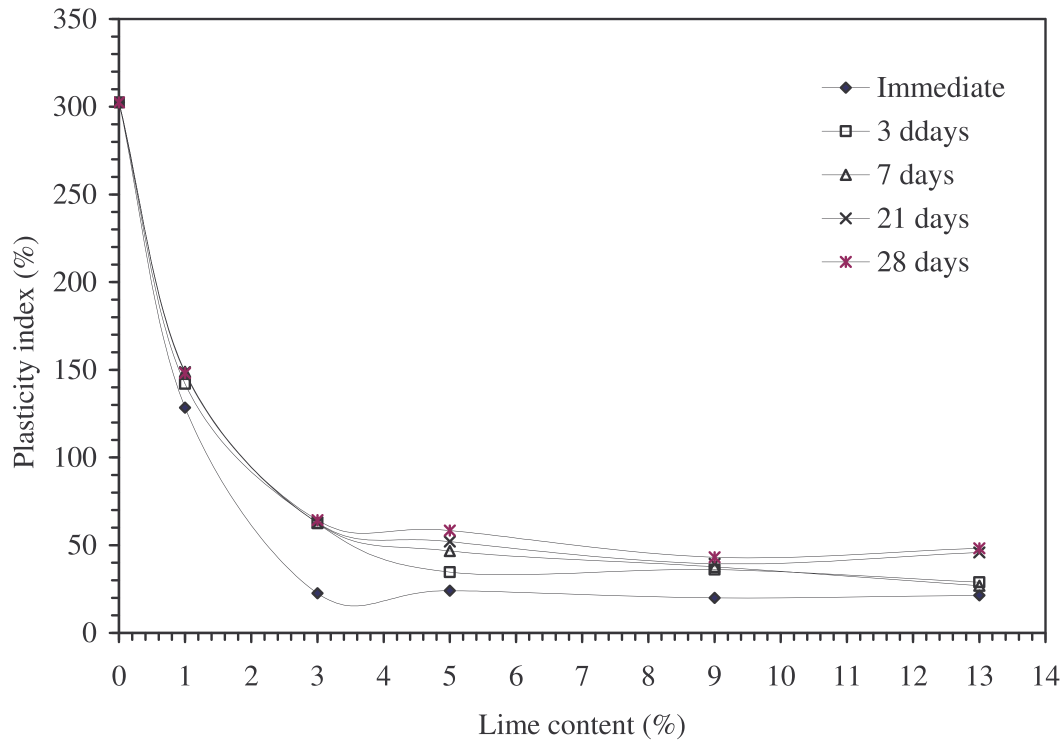

Fig. 4.16 Variation of plasticity index with lime content for expansive soil-residual soil mix (80%ES+20%RS)

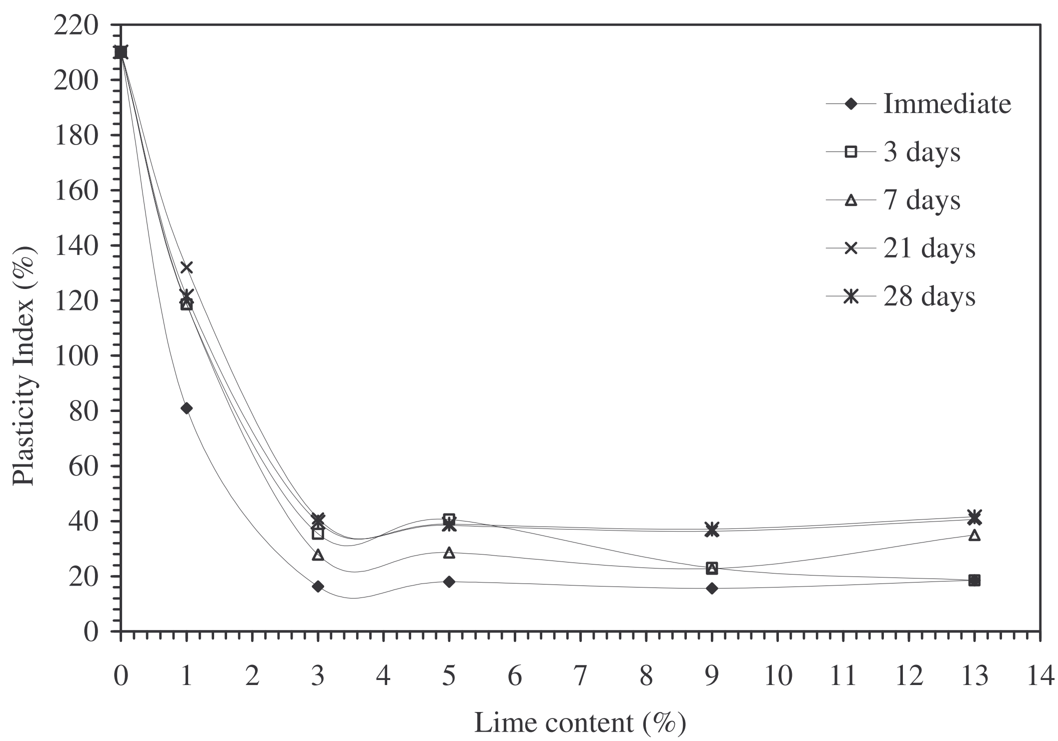

Fig. 4.17 Variation of plasticity index with lime content for expansive soil-residual soil mix (60%ES+40%RS)

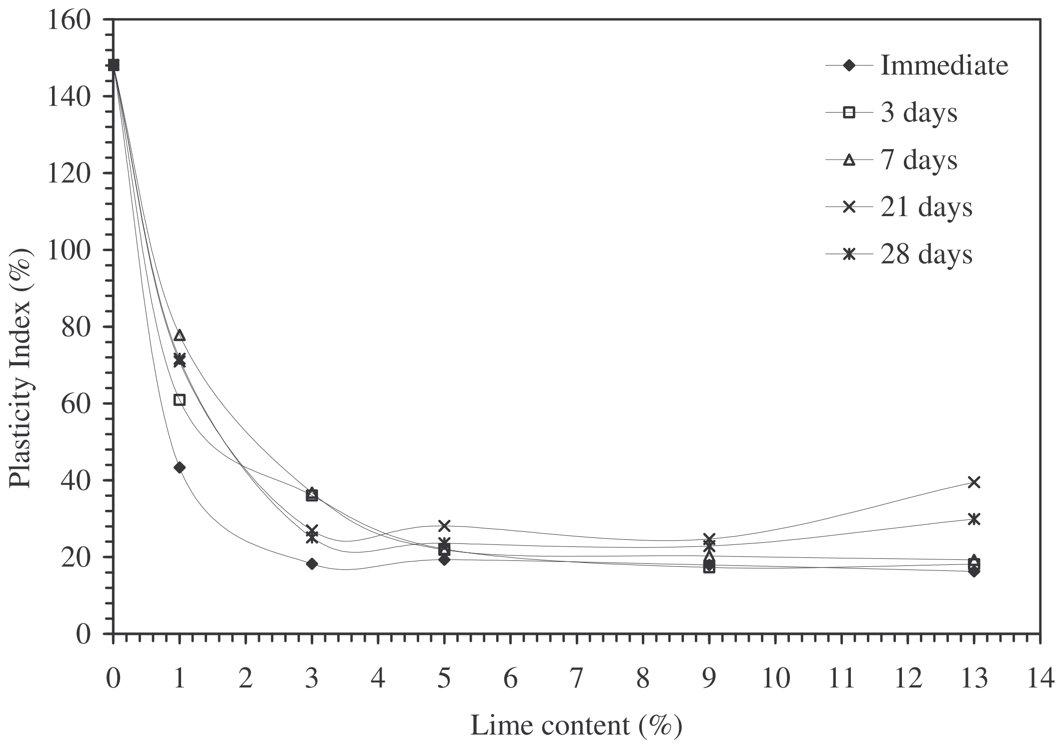

Fig. 4.18 Variation of plasticity index with lime content for expansive soil-residual soil mix (40%ES+60%RS)

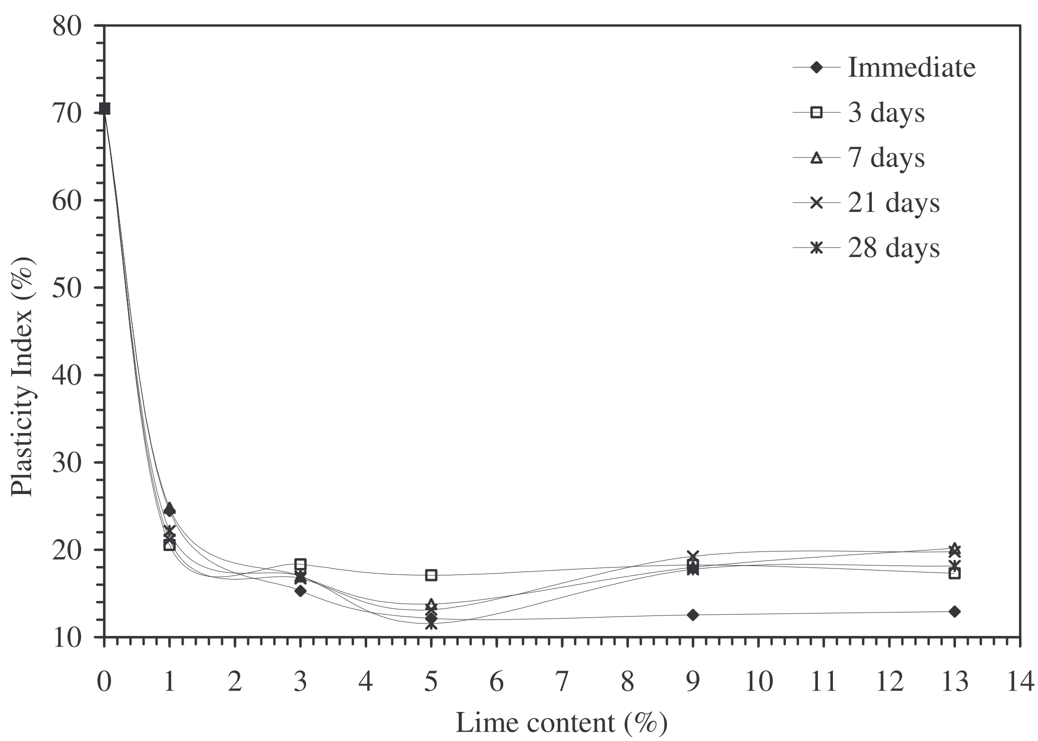

Fig. 4.19 Variation of plasticity index with lime content for expansive soil-residual soil mix (20%ES+80%RS)

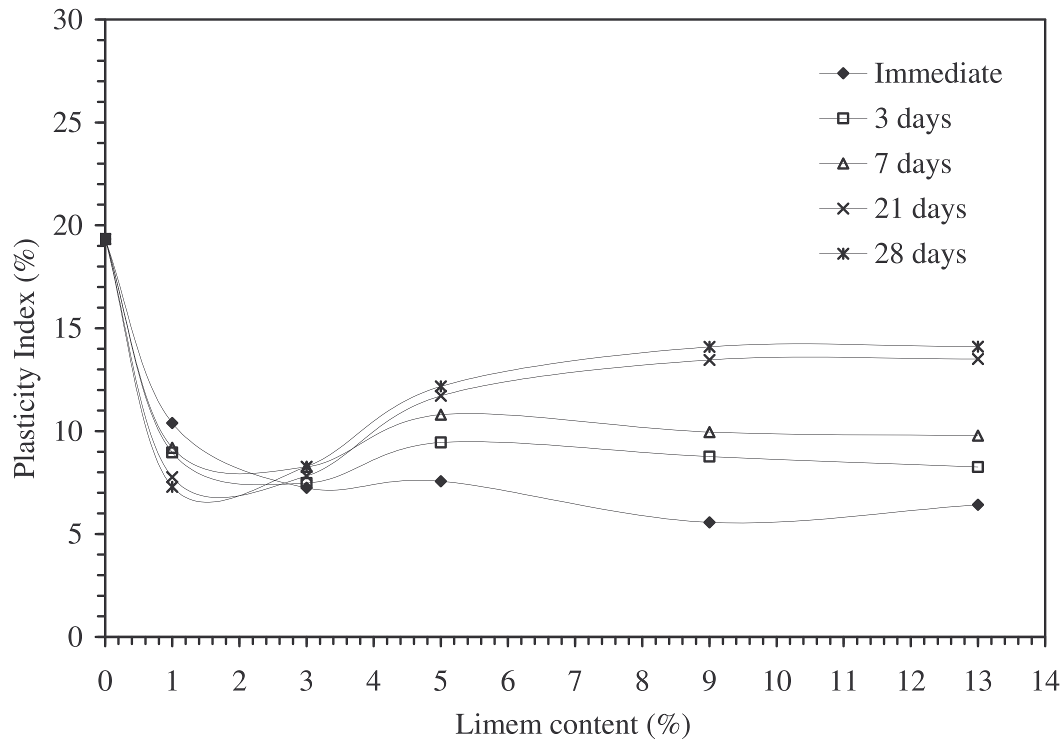

Fig. 4.20 Variation of plasticity index with lime content for residual soil (100%RS)

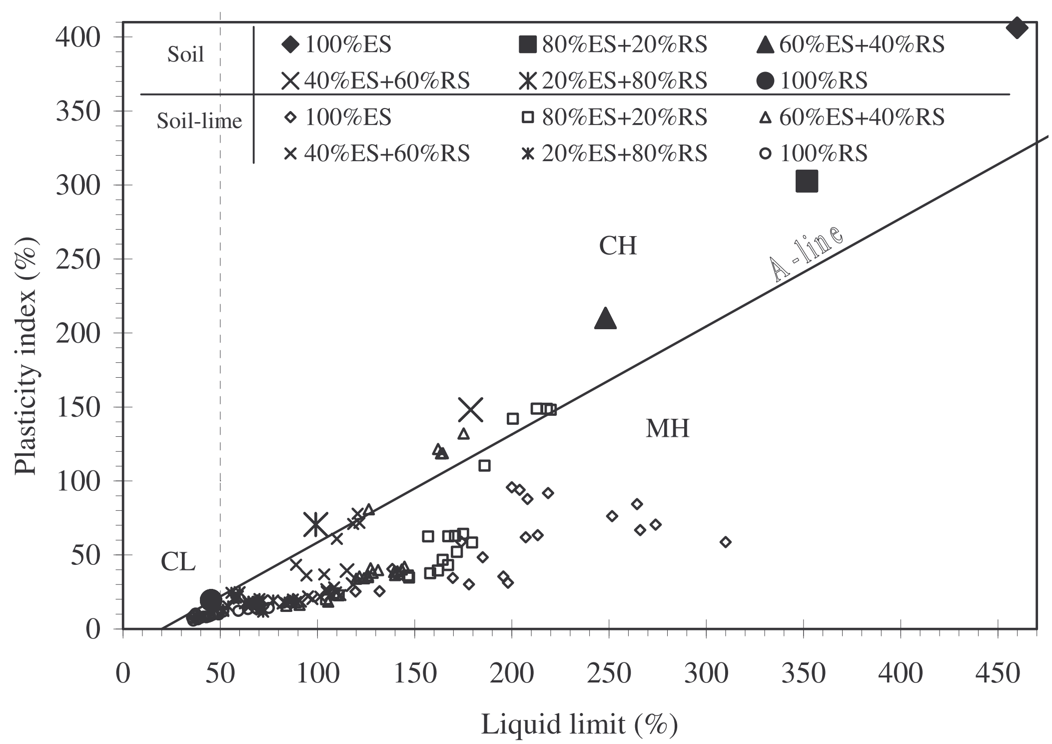

Fig. 4.21 Effect of lime on plasticity of soils

Table 4.3 Classification of lime treated soils

Soil	Classification of soil					
	0%Lime	1%Lime	3%Lime	5%Lime	9%Lime	13%Lime
	Curing period: Immediate					
100%ES	CH	CH	MH	MH	MH	MH
80%ES + 20%RS	CH	MH	MH	MH	MH	MH
60%ES + 40%RS	CH	CH	MH	MH	MH	MH
40%ES + 60%RS	CH	MH	MH	MH	MH	MH
20%ES + 80%RS	CH	MH	MH	MH	MH	MH
100%RS	CL	ML	ML	ML	ML	ML
	Curing period: 3 days					
100%ES	CH	CH	MH	MH	MH	MH
80%ES + 20%RS	CH	CH	MH	MH	MH	MH
60%ES + 40%RS	CH	CH	MH	MH	MH	MH
40%ES + 60%RS	CH	MH	MH	MH	MH	MH
20%ES + 80%RS	CH	MH	MH	MH	MH	MH
100%RS	CL	ML	ML	ML	ML	ML
	Curing period: 7 days					
100%ES	CH	CH	MH	MH	MH	MH
80%ES + 20%RS	CH	CH	MH	MH	MH	MH
60%ES + 40%RS	CH	CH	MH	MH	MH	MH
40%ES + 60%RS	CH	CH	MH	MH	MH	MH
20%ES + 80%RS	CH	MH	MH	MH	MH	MH
100%RS	CL	ML	ML	ML	ML	ML
	Curing period: 21 days					
100%ES	CH	CH	MH	MH	MH	MH
80%ES + 20%RS	CH	CH	MH	MH	MH	MH
60%ES + 40%RS	CH	CH	MH	MH	MH	MH
40%ES + 60%RS	CH	MH	MH	MH	MH	MH
20%ES + 80%RS	CH	MH	MH	MH	MH	MH
100%RS	CL	ML	ML	MH	MH	MH
	Curing period: 28 days					
100%ES	CH	CH	MH	MH	MH	MH
80%ES + 20%RS	CH	CH	MH	MH	MH	MH
60%ES + 40%RS	CH	CH	MH	MH	MH	MH
40%ES + 60%RS	CH	MH	MH	MH	MH	MH
20%ES + 80%RS	CH	MH	MH	MH	MH	MH
100%RS	CL	ML	ML	MH	MH	MH

4.4 SUMMARY

In this chapter test results have been analysed to study the plasticity behaviour of expansive soil mixed with varied proportions of residual soil and lime. Addition of residual soil though reduces plasticity but the soils still behave as clay. However with addition of lime they change to silt. This indicates increase in workability of very high order for the high plastic clays due to lime treatment. Curing further improves the workability.

CHAPTER 5

COMPACTION BEHAVIOUR

5.1 INTRODUCTION

Compaction is a process by which the volume of a three phase system consisting of solid, water and air is reduced by expulsion of air without changing the quantity of water or solids through the momentary application of mechanical energy. Thus compaction is basically a volume change phenomenon. The purpose of compacting earth fills in dams and embankments is to produce a soil mass that will meet the two basic requirements, i.e. reduction in settlement, and increase in shear strength. Many other engineering structures constructed on soils, such as highways, railways and airfield pavements, also, require compaction. The compaction induced strength of soils, increases the bearing capacity of foundations. Besides it reduces the settlement of structures and increases the stability of slopes of embankments.

Expansive soils, due to their high plasticity characteristics, are extremely difficult to compact properly. This can be to some extent, overcome by adding additives such as soil with low plasticity, lime, cement, fly ash etc.

In the construction of many earth structures, such as embankments, highway and railway subgrades, it is essential to assess the suitability of a soil with respect to the compaction characteristic. For preliminary assessment of the suitability of soils required for such project, involving extremely large quantity of soil from varied sources, empirical correlations between compaction properties with simple and easily determinable index properties are of great need. Hence, the compaction behaviour is an important engineering property that needs to be properly understood. In this chapter influence of residual soil and lime on the compaction behaviour of expansive soil is studied.

5.2 COMPACTION BEHAVIOUR OF ES-RS MIXES

Fig. 5.1 shows compaction curves for the expansive soil, residual soil and their mixtures. For all the soils the dry density increases with water content, reaches maximum and then decreases. For the expansive soils thickness of diffuse double layer increases with water content (Lambe, 1958). Due to increase in thickness of diffuse double layer particles mobilize higher repulsive pressure thereby resist the compactive effort leading to lower density. Further, an increase in water content decreases the effective negative pore water pressure. Thus with increase in water content the negative pore water pressure decreases that tends to increase the dry density. In the dry side of optimum, the increase in dry density with water content shows that effect of decrease in negative pore water pressure is more than the influence of increased thickness of the diffused double layer. Besides, under compaction, particles come closer to mobilize higher repulsive pressure to resist the compactive effort leading to increased density. However, on the wet side of optimum, the water content being high the compactive effect is resisted by the mobilized positive pore water pressure. This positive pore water pressure coupled with double layer repulsive pressure prevents the soil particles to come closer leading to reduction in the dry density.

As seen in Fig. 5.1 dry density of expansive soil < dry density of expansive soil-residual soil mixtures < dry density of residual soil. Since sodium ions present in the expansive soil are monovalent, their hydrated size is relatively large thereby mobilizes higher repulsive pressure in presence of water. Besides the expansive soil particles being finer than the residual soil the negative pore water pressure developed in the expansive soil too would be high. This higher repulsive pressure and negative pore water pressure resist the compactive effort leading to reduced dry density.

It can be seen from Fig. 5.1 that the expansive soil has shown relatively flat response that only at very high water content it has shown decrease in dry density. This is attributed to the large thickness of diffuse double layer that creates proportionately higher volume of viscous water in the soil pores. This viscous water negates the compactive effort leading to marginal change in fabric.

For residual soil (i.e. kaolinite mineral is predominant), the contribution from the diffused double layer induced repulsive pressure is negligible and it is the shear resistance at the particle level which controls its engineering behaviour (Sridharan and Rao, 1973). As the water content increases negative pore water pressure decreases. Therefore the soil particles come closer to mobilize the required shear resistance against the compactive effort. This continues till the optimum point is reached. On the wet side of optimum, the shearing resistance along with the positive pore water pressure developed, prevent the soil particles to come closer. With increase in water content the positive pore water pressure increases that offer increased resistance against compaction leading to decrease in dry density.

Residual soil has shown higher dry density and lower optimum water content than the other soils. Particles of residual soil are much coarser than the other soils. Therefore the size of inter particle pores is also larger. Hence the pore water pressure is considerably lower. This has resulted in the higher density for residual soil.

For expansive soil-residual soil mixtures, the dry density increases as the percentage of residual soil increases. Particles of residual soil are much coarser than the expansive soil. The finer particles of the expansive soil are replaced by coarser particles of residual soil. Therefore the size of pores grow larger. Hence the pore water pressure is considerably reduced. This has resulted in reduced optimum moisture content and increased dry density. The relatively higher specific gravity of the residual soil is another factor that has led to increased density.

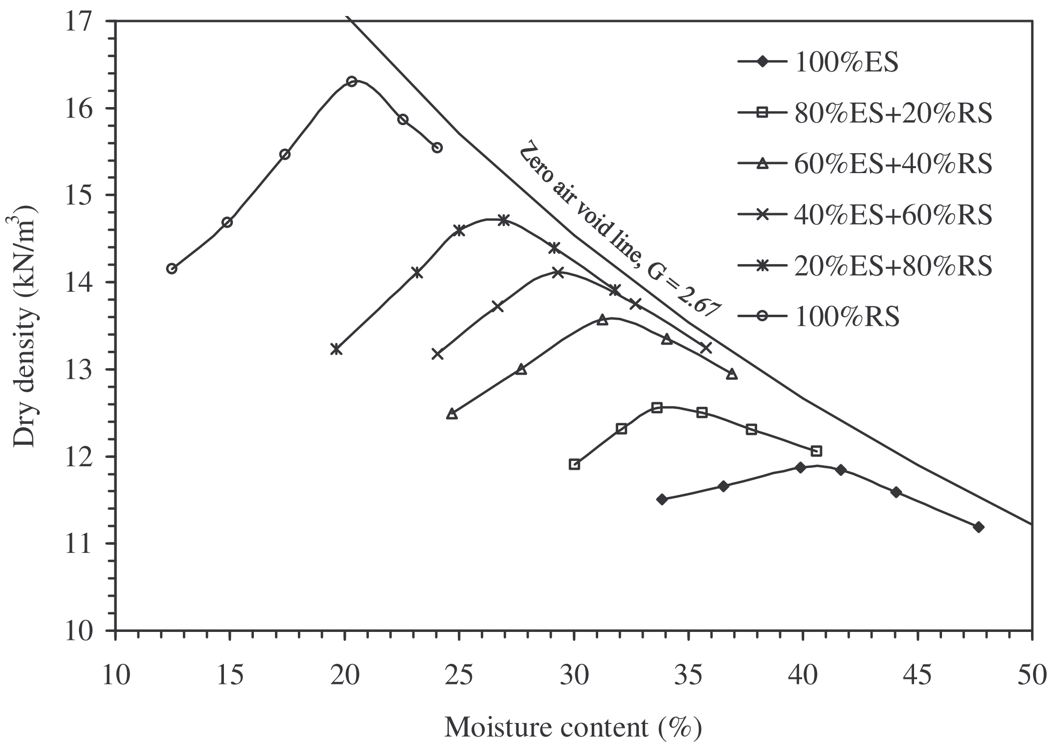

Fig. 5.1 Dry density-moisture content relationship of different soils

5.2.1 Correlations

Variation maximum dry density of these soils with their liquid limit and plasticity index are shown in Fig. 5.2 and Fig. 5.3 respectively. Correspondingly variation of optimum moisture content with these parameters is depicted in Fig. 5.4 and Fig. 5.5 respectively. It could be observed that with increase in the liquid limit and plasticity index the maximum dry density is found to decrease and the optimum moisture content increases. This is due to the increase in thickness of diffuse double layer leading to increased water holding capacity of the soil (Lambe, 1958). Due to increase in thickness of diffuse double layer particles can mobilize higher repulsive pressure and resist the compactive effort leading to lower density. It is also observed that up to liquid limit of about 100 % and plasticity index of 70% there is a steep slope in the responses (Fig. 5.2 to Fig 5.5), beyond they take relatively flatter slope. Therefore, it can be said that there is a phase change for liquid limit and plasticity index in the range of 100% and 70% respectively. This is attributed due to the fines content. The finer particles of the expansive soil initially fill up the voids between the relatively coarser particles of the residual soil. In this arrangement, the residual soil matrix prevents the entrapped fine soil mass against compaction leading to sharp reduction in overall density. With increased quantity of expansive soil the residual soil matrix breaks and the structure now is the residual soils floating in the expansive soil mass. This leads to effective compaction giving rise to reduced rate of decrease in density.

Fig. 5.6 presents the maximum dry density (γ_{dmax}) and the dry density of the soil at the plastic limit water content (γ_{dPL}), assuming it to be saturated. It shows a good co-relation and the equation is

$$\gamma_{d\max} = 0.76\gamma_{dPL} + 3.7 \quad 5.1$$

Where γ_{dmax} and γ_{dPL} are in kN/m^3. The correlation coefficient for this equation is 0.92.

However, Gurtug and Sridharan (2002) have found the following co-relation between γ_{dmax} and γ_{dPL}.

$$\gamma_{d\max} = 0.98\gamma_{dPL} \quad 5.2$$

The difference is attributed due to the difference in the ranges of plasticity used in both the studies.

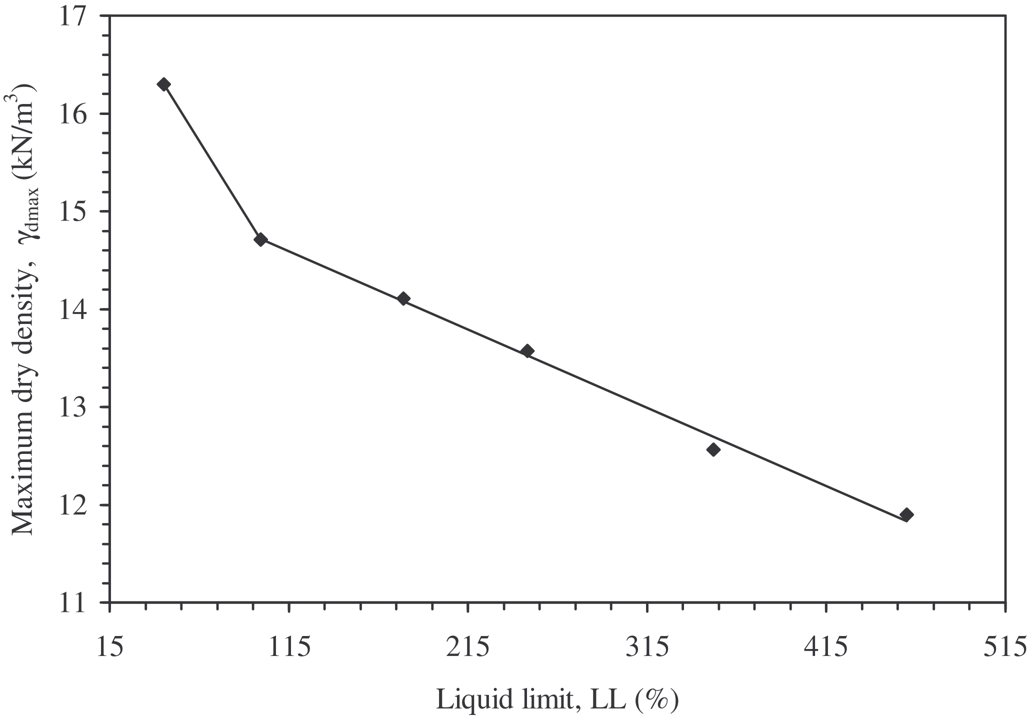

Fig. 5.2 Variation of maximum dry density with liquid limit

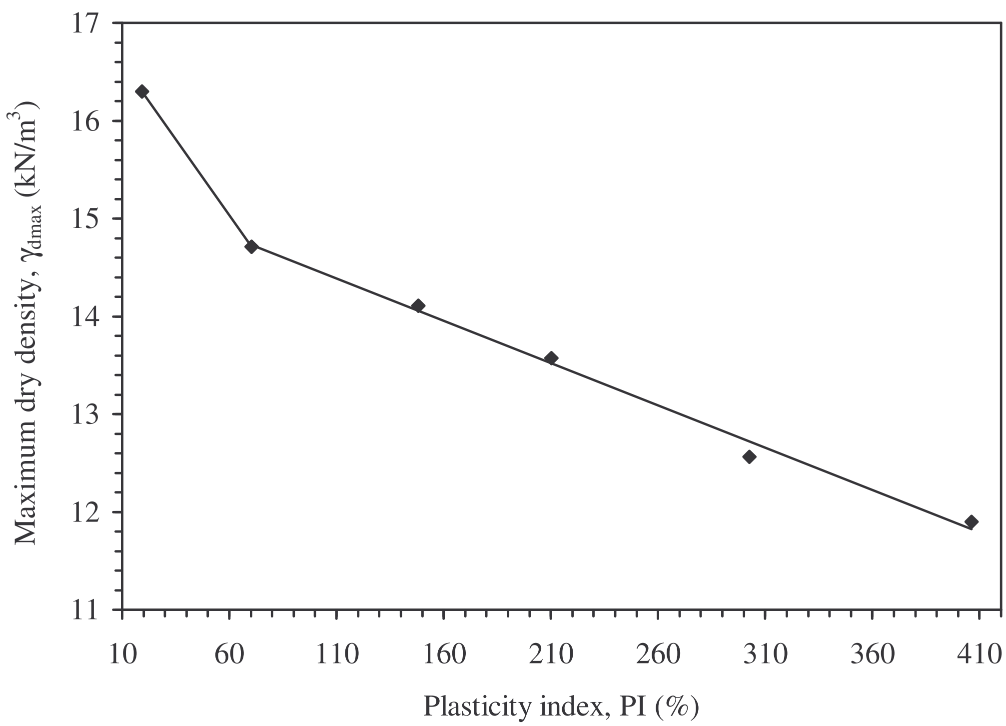

Fig. 5.3 Variation of maximum dry density with plasticity index

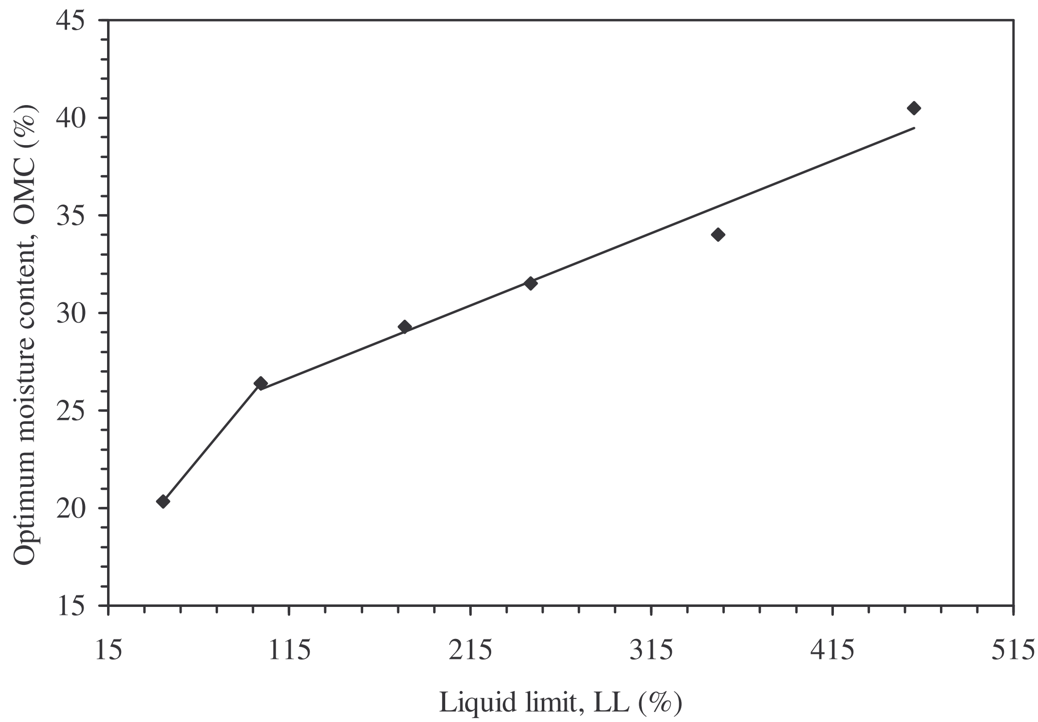

Fig. 5.4 Variation of optimum moisture content with liquid limit

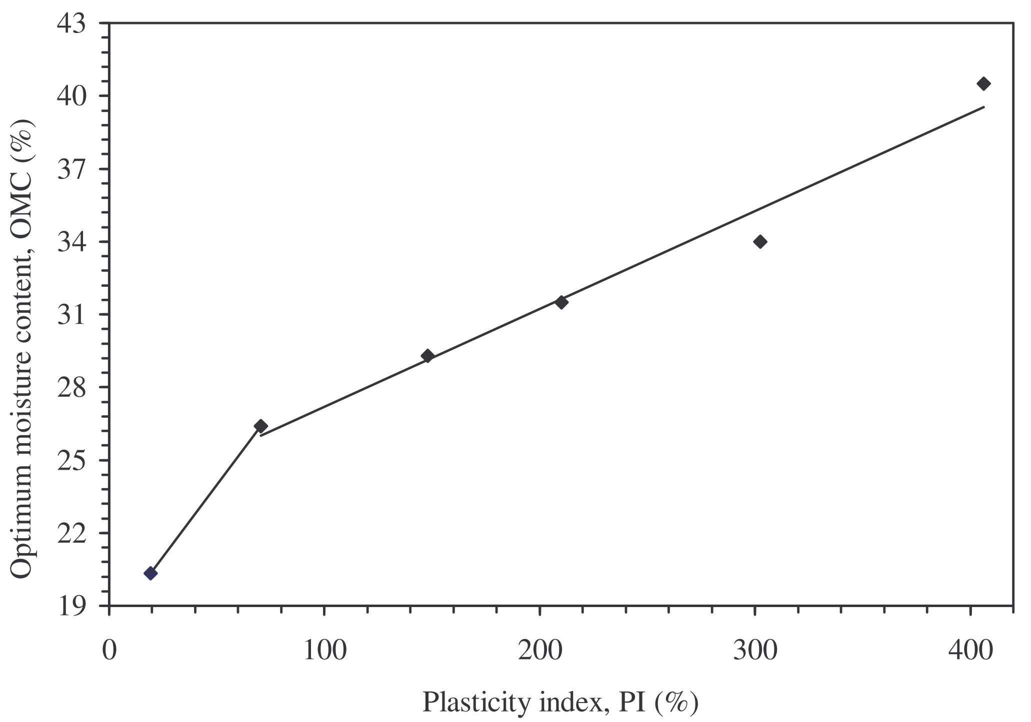

Fig. 5.5 Variation of optimum moisture content with plasticity index

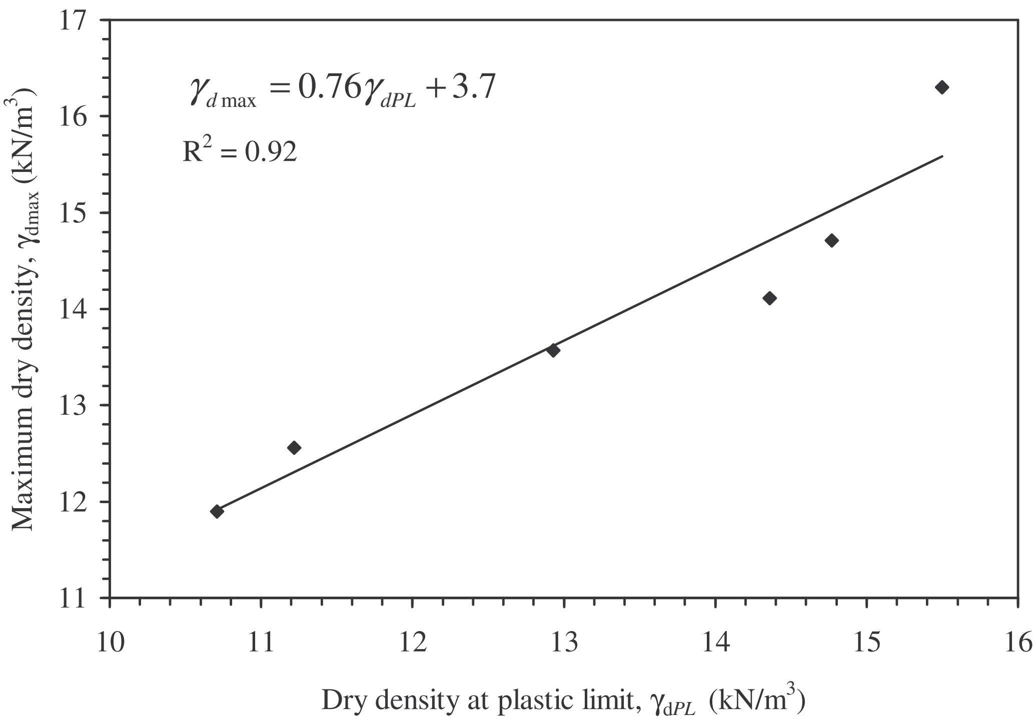

Fig. 5.6 Variation of maximum dry density with dry density at plastic limit

5.3 COMPACTION BEHAVIOUR OF ES-RS-LIME MIXES

The compaction responses of the soils; 100%ES, 80%ES+20%RS, 60%ES+40%RS, 40%ES+60%RS, 20%ES+80%RS and 100%RS, with varied lime contents, are depicted in Figs. 5.7, 5.8, 5.9, 5.10, 5.11 and 5.12 respectively. In these plots the compaction responses of the soils without lime, are included for comparison purpose.

It could be observed that for all the soils irrespective of their plasticity characteristics (i.e. varying from very low to very high) addition of lime has reduced the maximum dry density. This is attributed to the flocculated structure of clay particles, formed due to addition of lime.

It is of interest to note that lime treatment has flattened the compaction curves, indicating that the density variation with moisture content is gradual. In other words the targeted density can be achieved over a relatively wider range of moisture content. This is attributed to the reduced thickness of diffuse double layer due to increased cation concentration through addition of lime, which has made the soil more friable. Hence it can be said that, through lime treatment, the soils have turned out to be better workable materials.

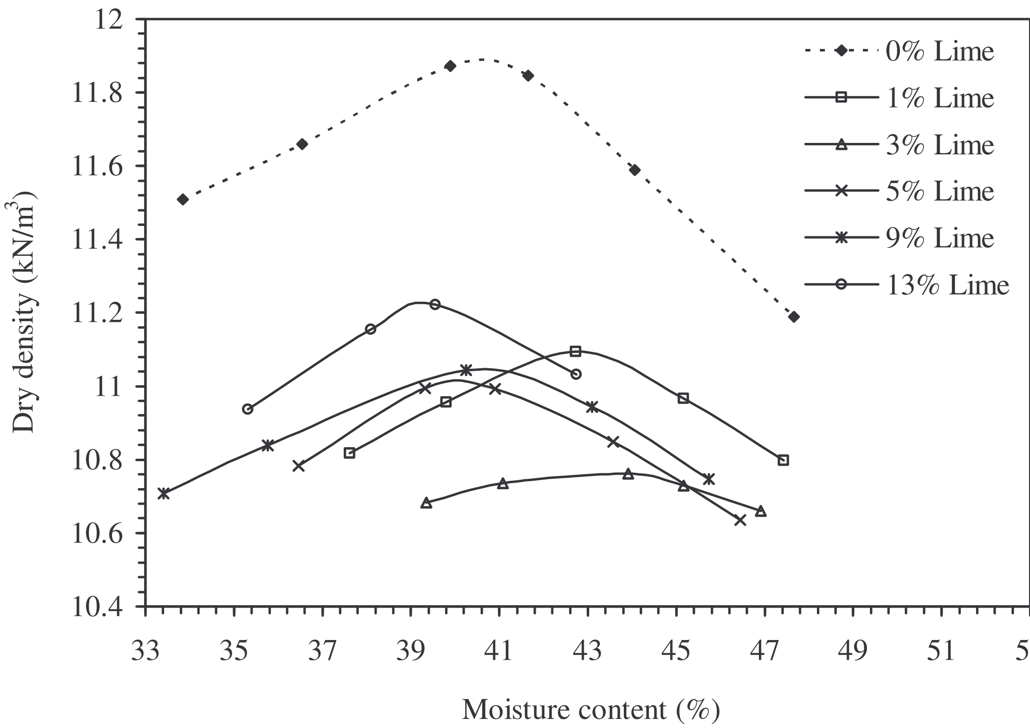

Fig. 5.7 Dry density-moisture content relationship of 100%ES at different percentage of lime.

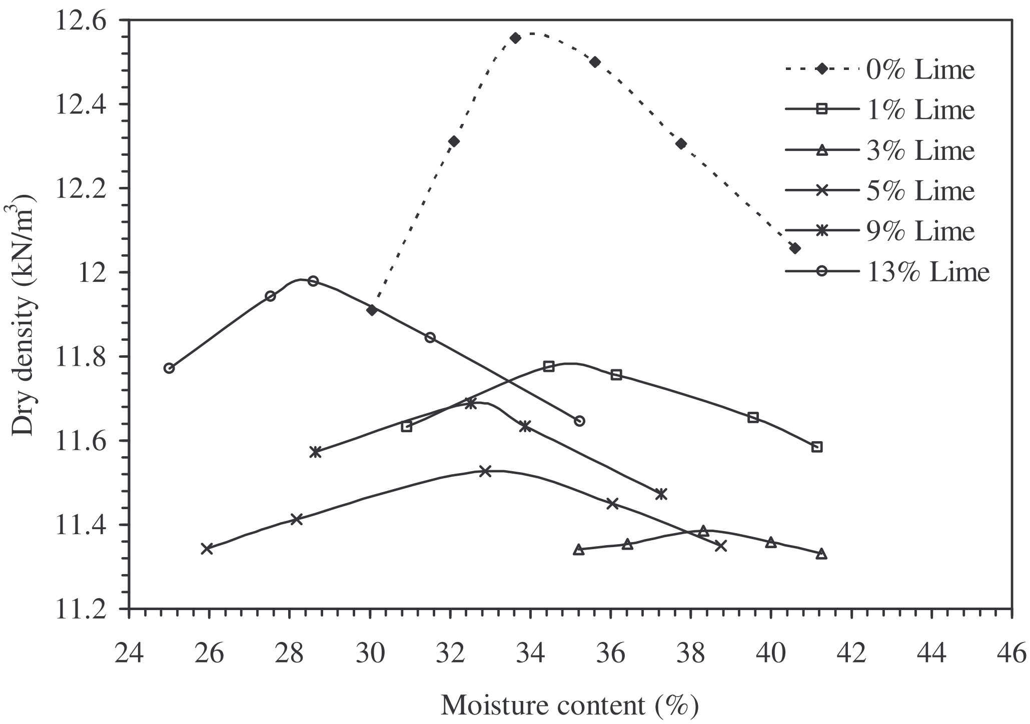

Fig. 5.8 Dry density-moisture content relationship of 80%ES+20%RS at different percentage of lime

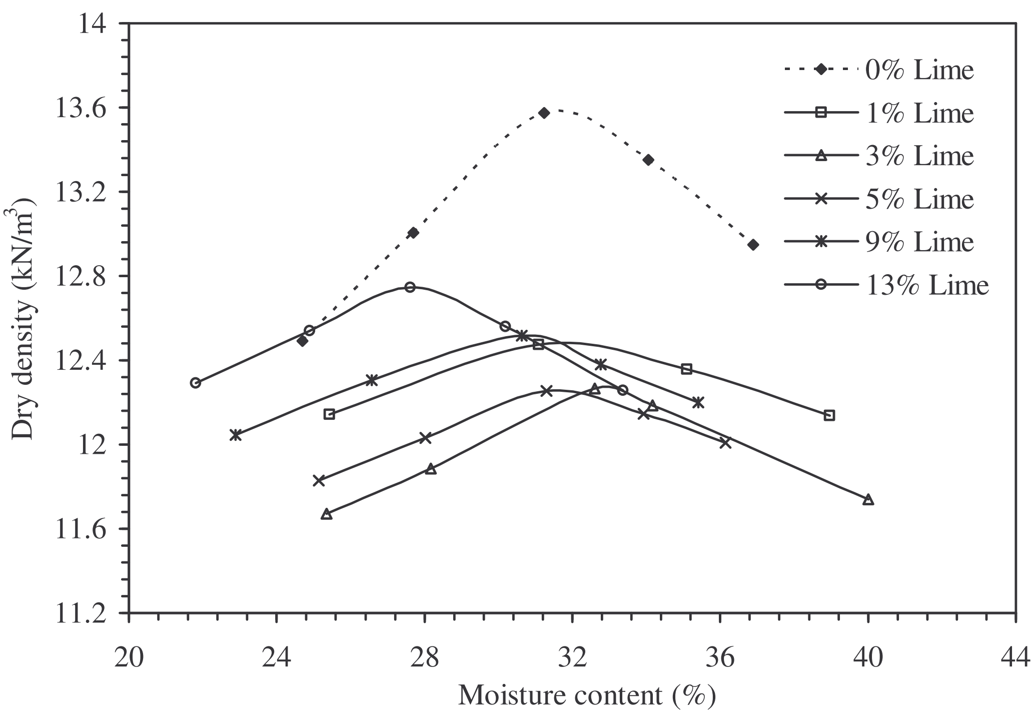

Fig. 5.9 Dry density-moisture content relationship of 60%ES+40%RS at different percentage of lime

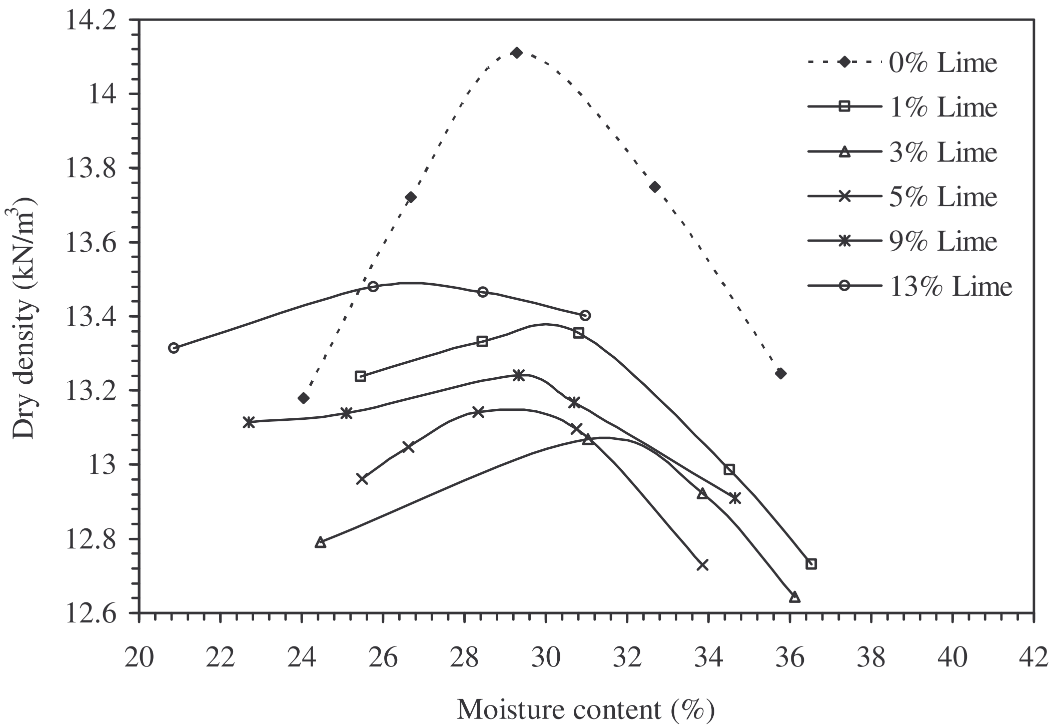

Fig. 5.10 Dry density-moisture content relationship of 40%ES+60%RS at different percentage of lime

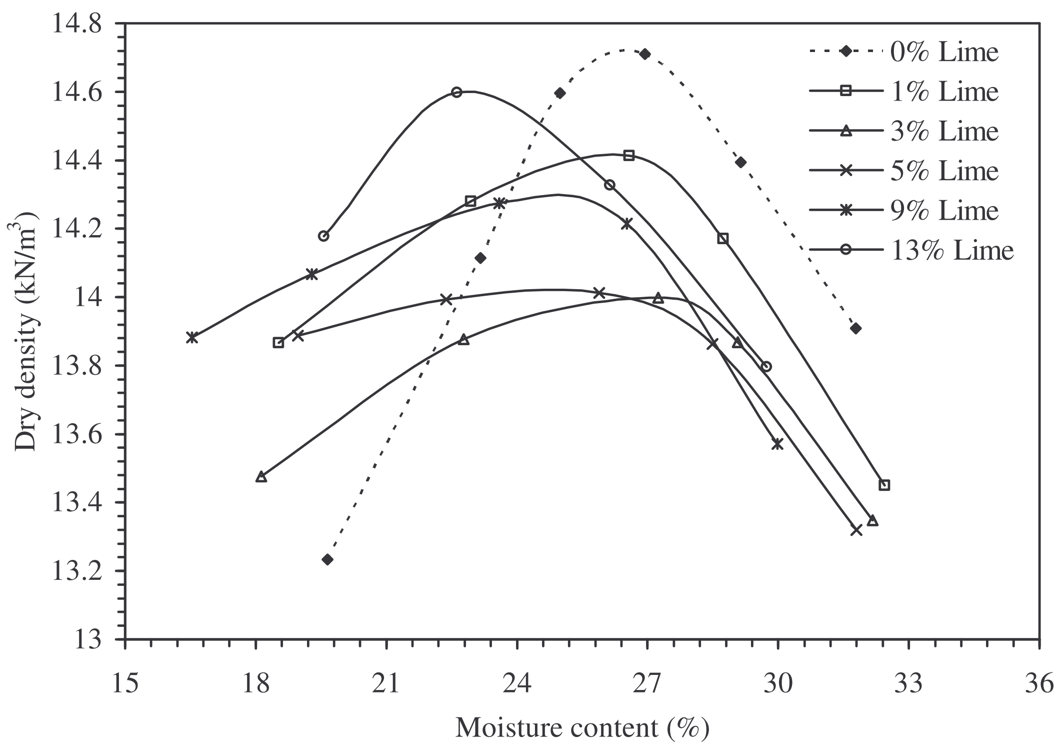

Fig. 5.11 Dry density-moisture content relationship of 20%ES+80%RS at different percentage of lime

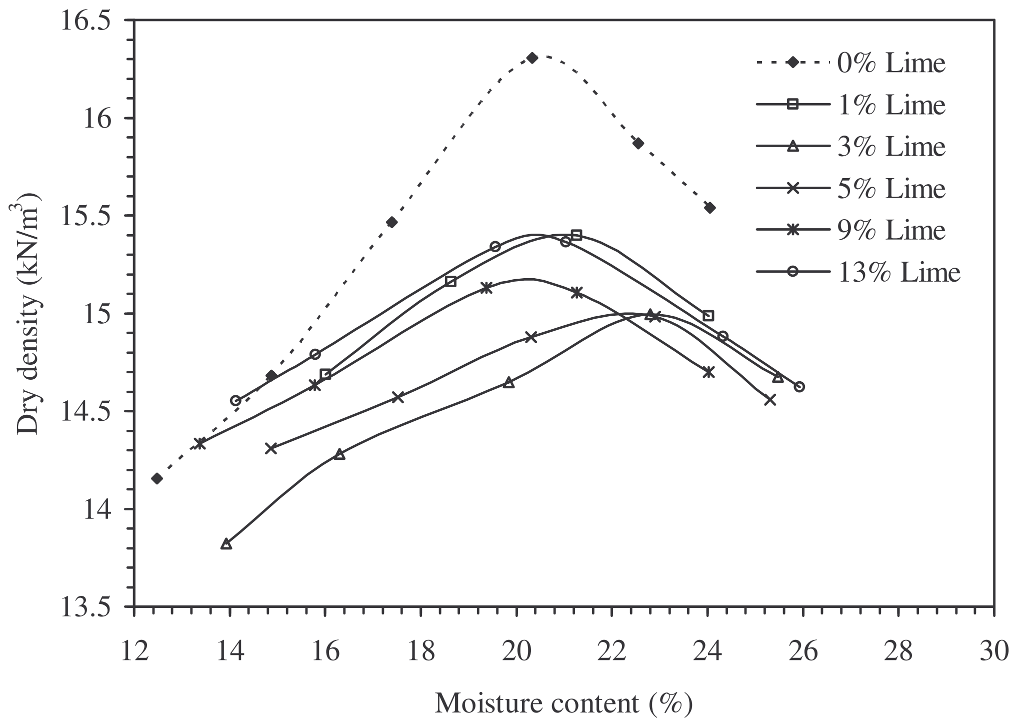

Fig. 5.12 Dry density-moisture content relationship of 100%RS at different percentage of lime

The test results depicted in Fig. 5.13 indicate that, for all soils, as the percentage of lime increases the maximum dry density (MDD) reduces. This continues till lime content reaches 3%, beyond which the MDD continues to increase, with increase in lime content, though at a relatively smaller rate. Correspondingly the optimum moisture content (OMC) increases till 3% of lime for expansive soil and expansive soil-residual soil mixtures. Beyond this the OMC decreases with increase in lime content (Fig. 5.14). However, incase of residual soil the maximum OMC is observed at 5% of lime, though the difference between the values for 3% and 5% lime content is marginal. This disparity could be due to experimental error given the accuracy of the tests. With increase in lime content the electrolyte concentration of the pore water increases leading to reduced thickness of double layer. As a result of which the clay particles move closer and Van der Waal's attraction becomes dominant producing flocculation and hence a card house type of clay structure. This card house structure of the clay matrix effectively resists the compaction effort giving rise to lower density and higher moisture content. With further increase in lime content the concentration of cations near to the negatively charged clay surfaces, increases. This difference of charged concentration in the pore water leads to osmosis. Since the ions are under influence of charge on clay surface they are restrained against diffusion. Therefore the free water molecules diffuse towards the clay surface to equalize charge concentration (Mitchell and Soga, 2005). This leads to separation of clay particles that produces relatively dispersed soil structure, thereby permits the particles to slide past over each other into a more oriented and hence a denser matrix. Based on the present study it can be said that, relatively higher quantity of lime (i.e. more than 3%) should be added to soil to obtain better compaction density. It should be mentioned here that the reduced density of soils due to lime is expected to give rise to reduced strength. However the large strength gain due to cementation process compensates this. Infact the increased strength is so high, as has been brought out in chapter 7, that the effect of reduced density turns out to be marginal.

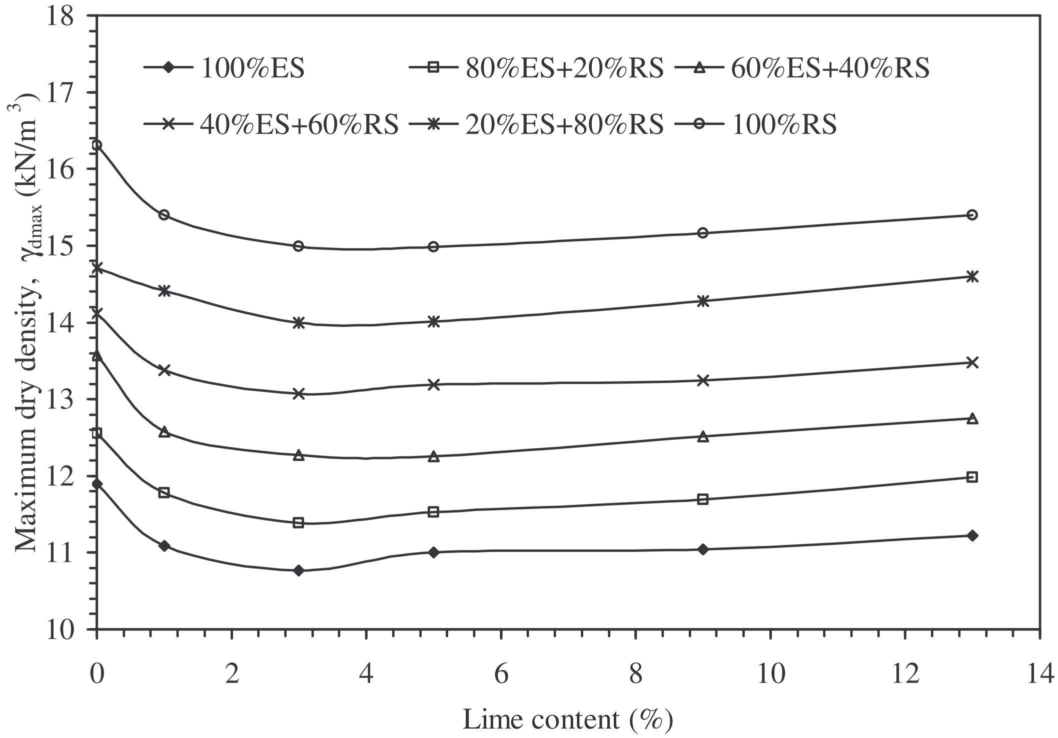

Fig. 5.13 Maximum dry density versus percentage of lime for different soils

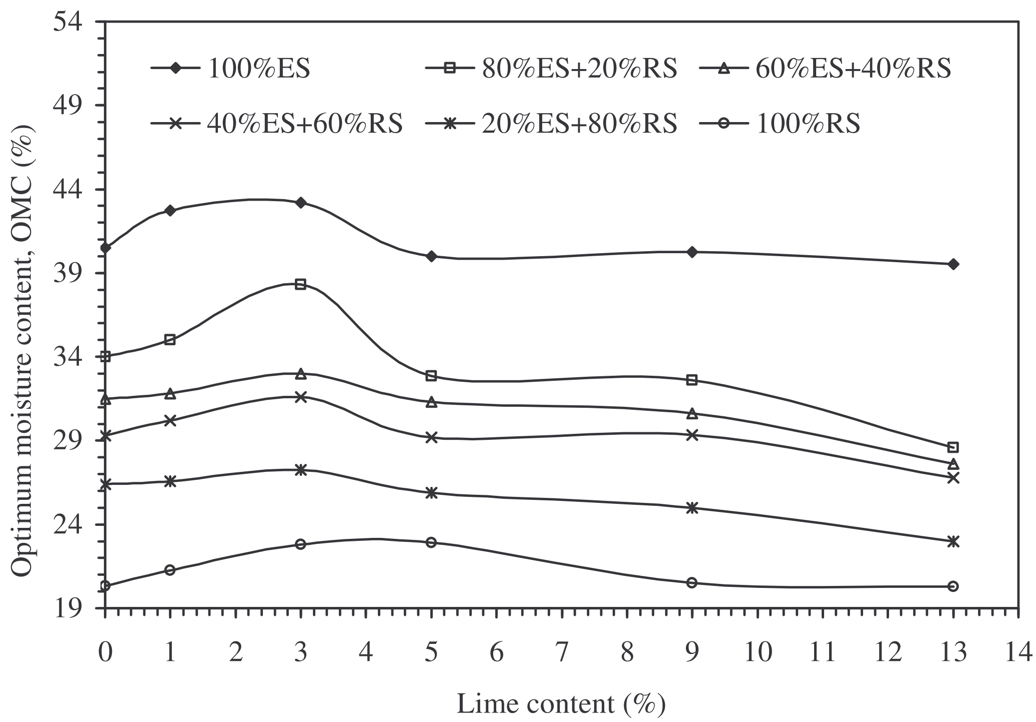

Fig. 5.14 Optimum moisture content versus percentage of lime for different soils

Fig. 5.15 and Fig. 5.16 show the effect of residual soil content on maximum dry density and optimum moisture content respectively, for different content of lime added. It could be observed that irrespective of quantity of lime added the maximum dry density increases and optimum moisture content decreases with increase in the residual soil content. In other words, compared to the lime treated expansive soil, the lime-treated residual soil attains higher compaction density. Major reason for this is the higher demand of expandable clay minerals for adsorbed and lubrication water than that of the non swelling residual soil (Croft 1964, Bell 1996), which is also valid for lime treated soils.

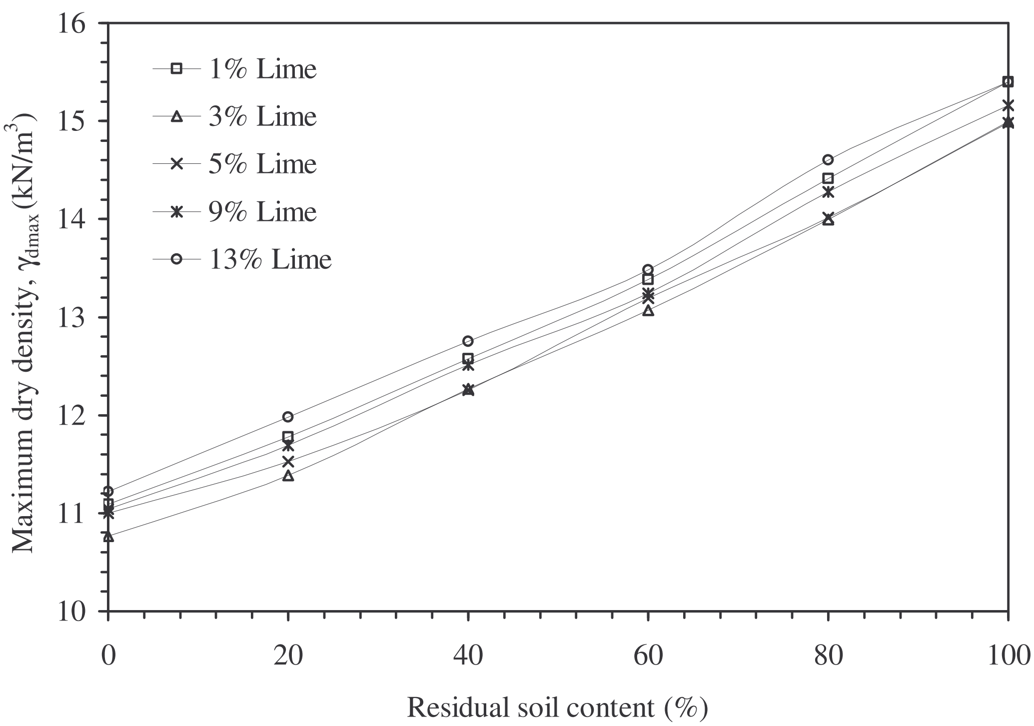

Fig. 5.15 Influence of residual soil on maximum dry density of lime treated soils.

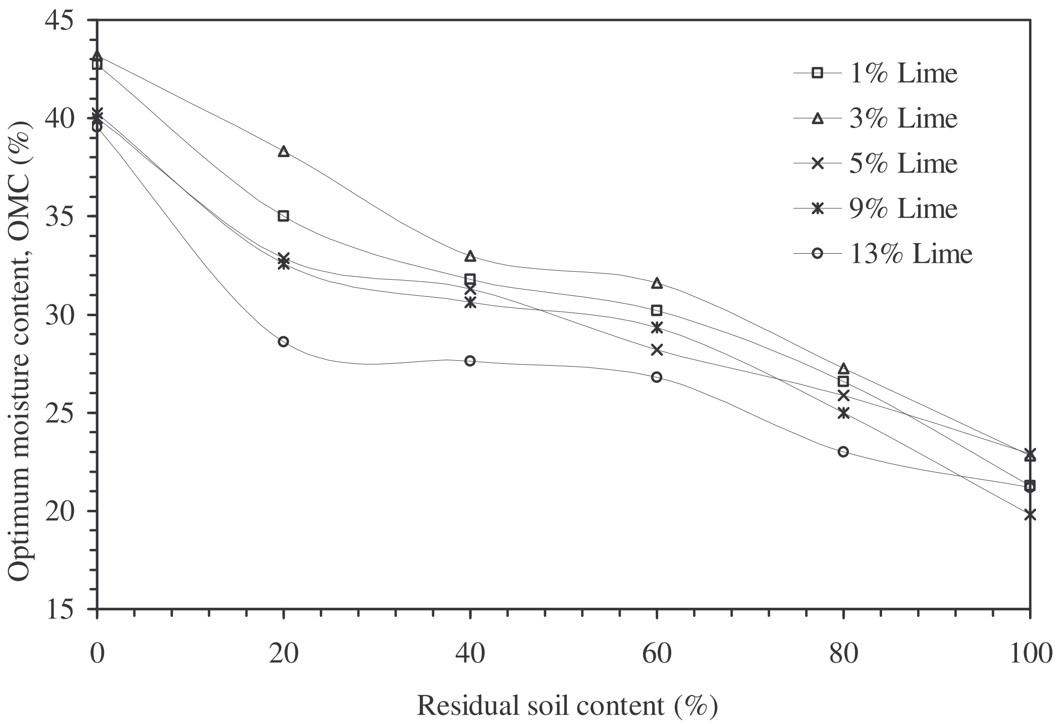

Fig. 5.16 Influence of residual soil content on optimum moisture content of lime treated soils

5.3.1 Correlations

Fig. 5.17 shows the variation of maximum dry density with optimum moisture content for different soils (i.e ES, ES+RS, RS) with varied lime content. It can be noted that there is a good correlation between the two parameters. The correlation equation obtained with a correlation coefficient (R^2) of 0.9 is

$$\gamma_{d\max} = 70.44(OMC)^{-0.497} \qquad 5.3$$

Where γ_{dmax} is in kN/m^3 and OMC is in percentage. This relation is similar to the one proposed by Hilf (1966) for untreated soils i.e $\gamma_{d\max} = 52.06(OMC)^{-0.396}$.

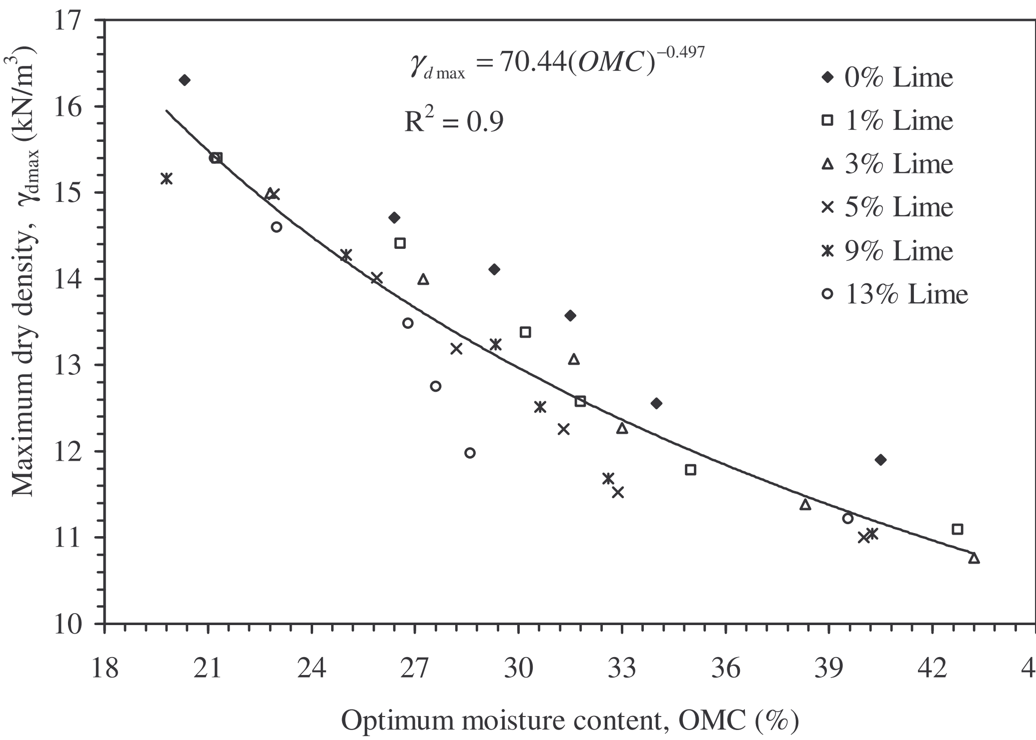

Fig. 5.17 Relationship between maximum dry density-optimum moisture content at different percent of lime for expansive soil-residual soil mixtures.

Fig. 5.18 and Fig. 5.19 show the correlation of the compaction parameters with liquid limit. In general the maximum dry density reduces and the optimum moisture content is found to increase with increase in liquid limit. However, the correlation is poor. Fig. 5.20 shows that the correlation between the maximum dry density with the plastic limit for the lime treated soils is very good. The proposed equation between these two parameters with correlation coefficient (R^2) of 0.9 is

$$\gamma_{d\max} = 35.2(PL)^{-0.25} \qquad 5.4$$

Where, γ_{dmax} is in kN/m^3 and plastic limit (*PL*) is in percentage.

Correspondingly the correlation between the optimum moisture content and plastic limit, depicted in Fig. 5.21 is found to be

$$OMC = 6(PL)^{0.4} \qquad 5.5$$

But there is some scatter in the data as indicated by a relatively low correlation coefficient $R^2 = 0.65$.

As suggested by Gurtug and Sridharan (2002) the dry density (γ_{dPL}) of the lime treated soil at their plastic limit were found out assuming the soils to be saturated at this condition. This parameter (γ_{dPL}) is plotted with the maximum dry density (γ_{dmax}) in Fig. 5.22. The correlation between the two is found to be good. The obtained correlation with R^2 value of 0.9 is

$$\gamma_{d\max} = 0.53\gamma_{dPL} + 7.15 \quad 5.6$$

Where, both γ_{dmax} and γ_{dPL} are in kN/m^3.

The variation of maximum dry density and optimum moisture content with shrinkage limit depicted in Fig. 5.23 and Fig. 5.24 respectively, shows that there is no correlation between the compaction parameters with the shrinkage limit.

From the above analysis it is observed that, for lime treated soils, among the different index properties, plastic limit best correlates with the compaction parameters, namely, maximum dry density and optimum moisture content. Similar such behaviour has been reported by Sridharan and Nagaraj (2005) for untreated soils. Hence, it can be said that both for lime treated and untreated soil, plastic limit state is nearly similar to optimum compaction state.

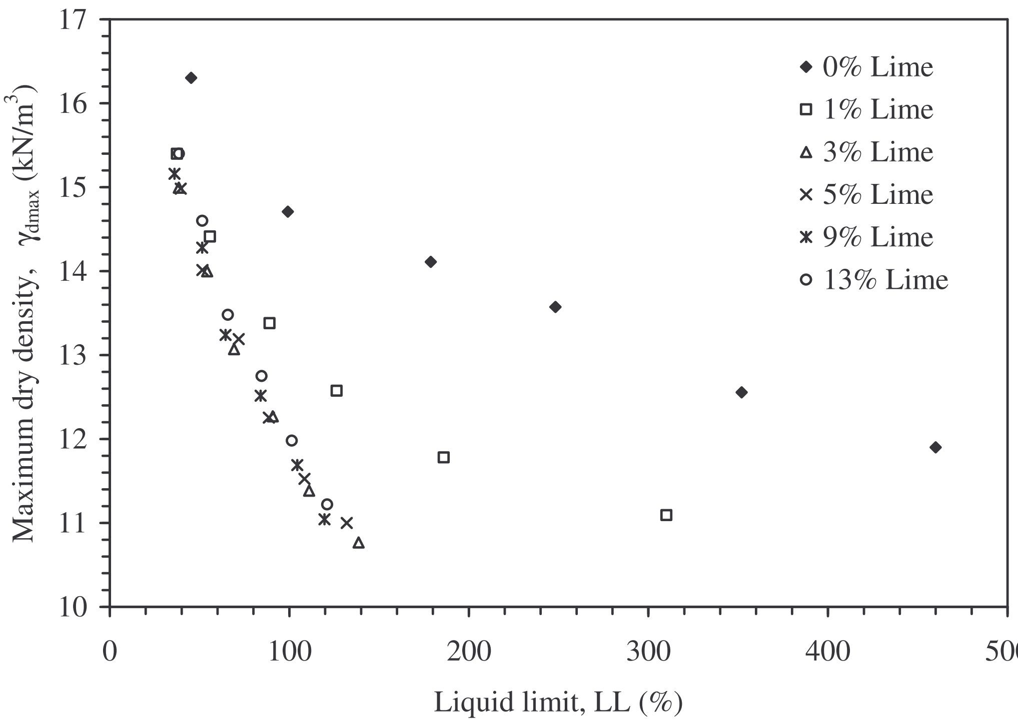

Fig. 5.18 Maximum dry density versus liquid limit relationship at different percent of lime for expansive soil-residual soil mixtures

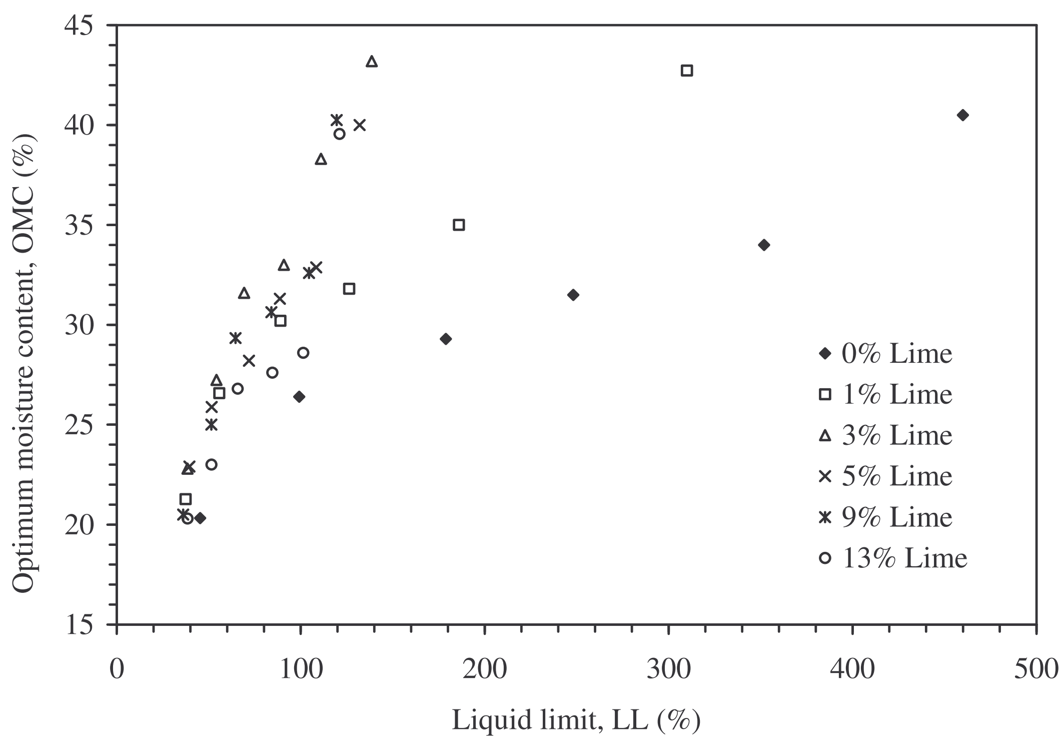

Fig. 5.19 Optimum moisture content versus liquid limit relationship at different percent of lime for expansive soil-residual soil mixtures

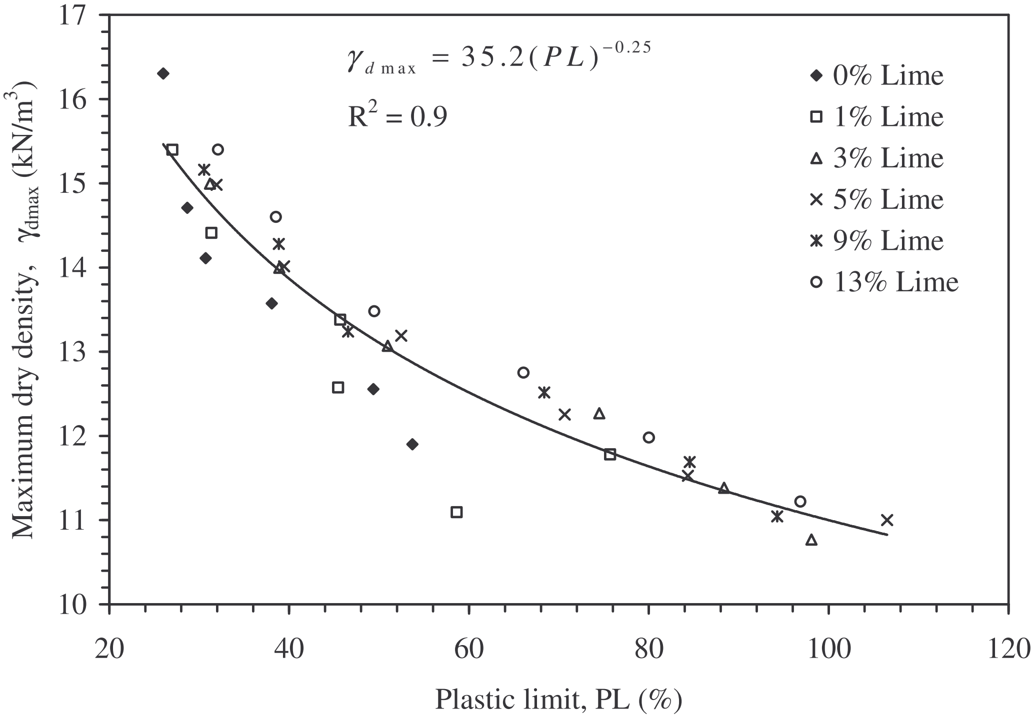

Fig. 5.20 Maximum dry density versus plastic limit relationship at different percent of lime for expansive soil-residual soil mixtures.

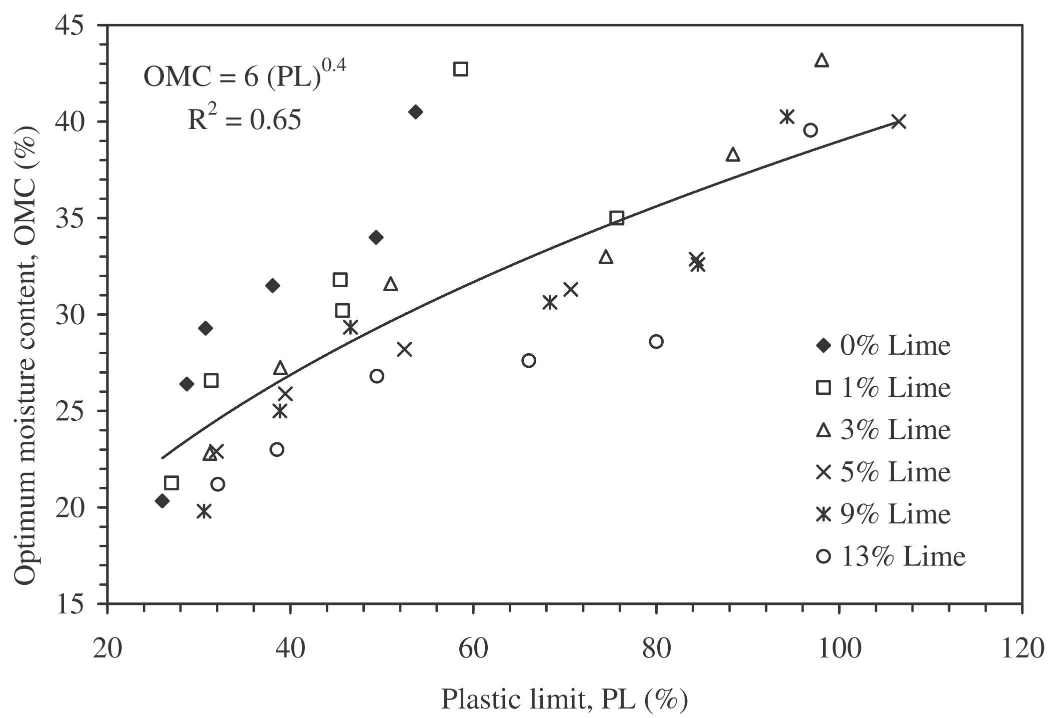

Fig. 5.21 Optimum moisture content versus plastic limit relationship at different percent of lime for expansive soil-residual soil mixtures.

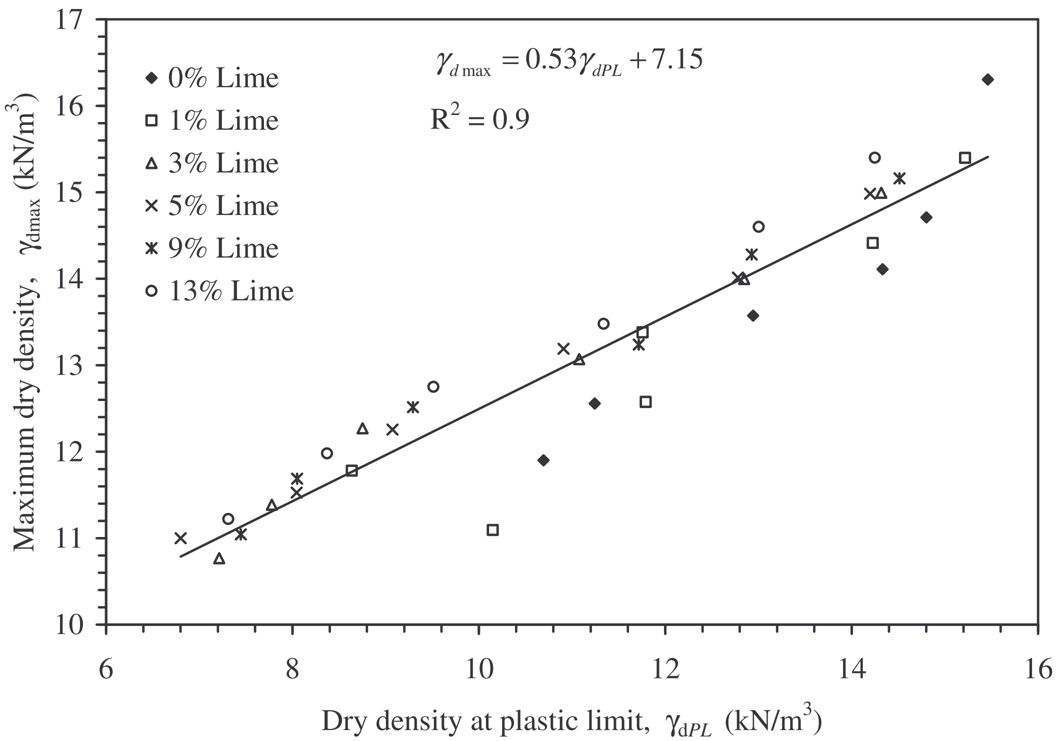

Fig. 5.22 Relationship between maximum dry density and dry density at plastic limit at different percent of lime for different soil.

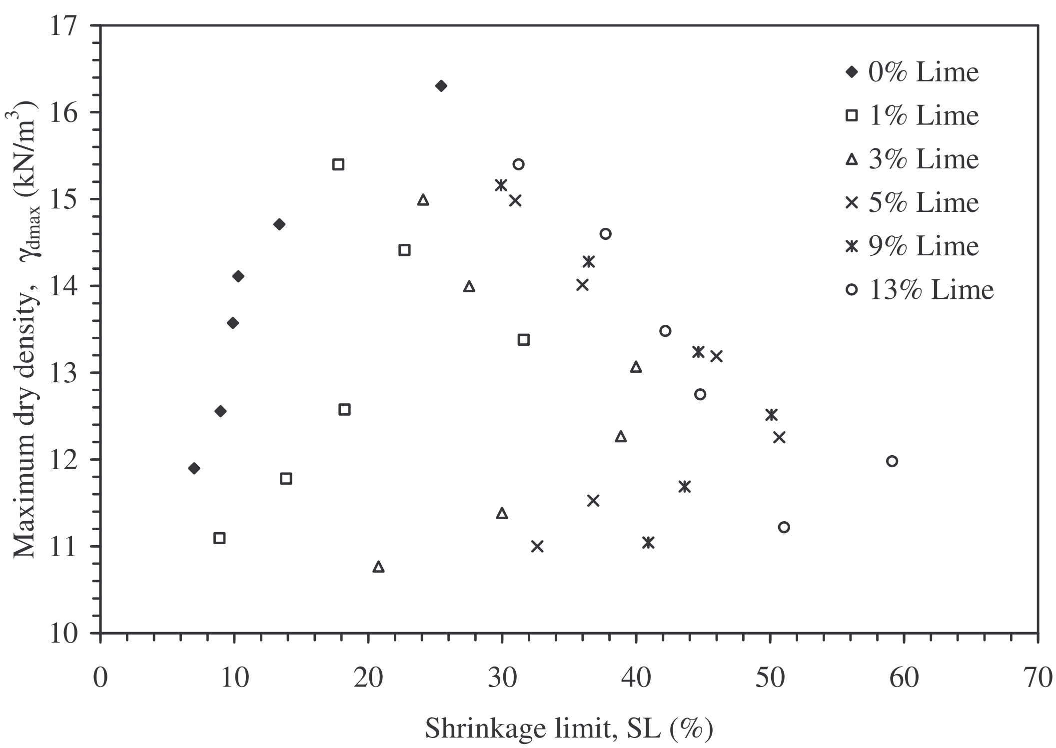

Fig. 5.23 Maximum dry density versus shrinkage limit relationship at different percent of lime for expansive soil-residual soil mixtures.

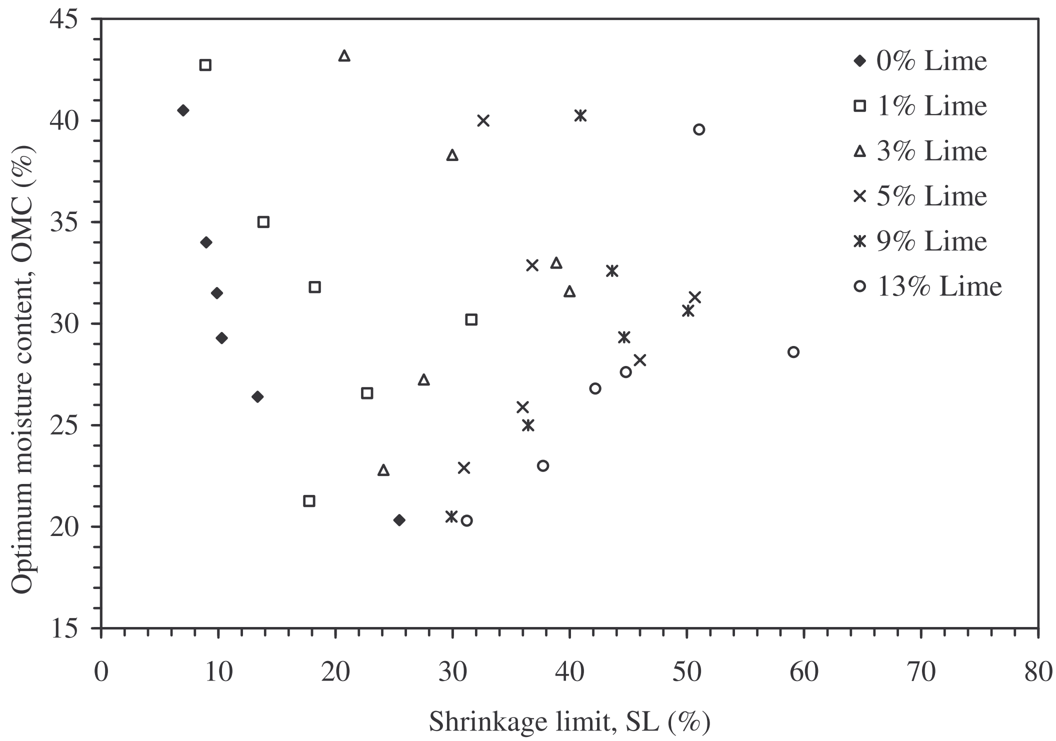

Fig. 5.24 Optimum moisture content versus shrinkage limit relationship at different percent of lime for expansive soil-residual soil mixtures.

5.4 SUMMARY

In this chapter the influence of lime on the compaction behaviour of expansive soils with varied content of residual soil has been studied. These soils have a wide range of plasticity characteristics (i.e. liquid limit varying from 45% to 460% and plastic limit varying from 26% to 53.7%). It is observed that as the lime content increases, the maximum dry density reduces and optimum moisture content increases till about 3% of lime. Beyond 3% lime content the maximum dry density tends to increase with increases in lime content. Among the different index properties, plastic limit is found to correlate most with the compaction parameters of the lime treated soils.

CHAPTER 6

SWELL-SHRINK BEHAVIOUR

6.1 INTRODUCTION

Expansive soils undergo large volumetric changes with change in moisture content. Increase in moisture content during wet season causes swelling and reduction in moisture content in dry season causes shrinkage in such soil. This leads to heaving and settlement that causes distress in the structures founded upon it such as; houses, pavements, rail tracks, canal lining, embankments etc. In the light of the present observation that the liquid limit and plasticity index, after initial decrease, again show an increasing trend with increased lime content (chapter 4) the swelling response needs to be relooked. Besides, though there has been studies to predict the swelling and shrinkage of untreated soils, but for lime treated soils it hasn't been attempted so far. In view of this a series of tests have been carried out to study the swelling and shrinkage behaviour of lime treated soils, the details of which are given in Table 3.2. Tests are also carried out for untreated soils. The obtained results are presented and discussed in the following sections.

6.2 SWELL BEHAVIOUR

6.2.1 Swell Behaviour of ES-RS Mixes

6.2.1.1 Free Swell

Free swell index (FSI) that represents the ratio of swelled volume of the soil in water per unit weight of the soil, is the simplest parameter to indicate swelling ability of soils. Table 6.1 shows the free swell index values for different soils along with other index properties. The free swell index shows a nearly linear variation with the index properties. The expansive soil that has high plasticity (i.e. LL, PL) and high specific surface area (SSA) shows higher free swell index. This is due to the high water holding capacity of this soil through its large volume of diffused double layer. With addition of residual soil the swelling montmorillonite mineral is replaced by the nonswelling kaolintic one that holds much less water onto its surface leading to reduced FSI. Little free swell observed incase of residual soil is attributed to its flocculated fabric that holds water in the void within.

Table 6.1 Free swell index (FSI) variation with index properties

Soil	Liquid limit (%)	Plastic limit (%)	Shrinkage limit (%)	Specific surface area, m^2/gm	Free swell index (ml/gm)
100%ES	459.9	53.7	7	86.45	20
80%ES+20%RS	351.9	49.4	8.97	83.24	12.50
60%ES+40%RS	248.2	38.1	9.89	65.41	8
40%ES+60%RS	178.8	30.7	10.29	55.32	6
20%ES+80%RS	99.2	28.7	13.36	49.15	4
100%RS	45.3	25.9	25.45	38.76	0.20

6.2.1.2 Oedometer Swell

Fig. 6.1 shows the time versus percent swell responses for the six different soils studied under the present investigation. It could be observed that the rate of swelling is rapid in the initial stages and reduces with increased time. Indeed the corresponding percent swell vs. log of time responses depicted in Fig. 6.2 shows that there is a clear change in rate of swell. The first stage of swelling is attributed to hydration of dry clay particles by virtue of which water is adsorbed in successive monolayers on the soil surface and pushes the particles away leading to swelling. While in the second stage swelling is predominantly due to repulsion of the diffuse double layers (Dakshinamurthy, 1978). Indeed in case of the non swelling kaolinite dominant soils (100%RS, 20%ES+80%RS) wherein the diffuse double layer effect is marginal, the second stage swelling is negligible and the percent swell-log t response is nearly horizontal. The soils with low plasticity though have shown asymptotic response the soils with high plasticity (ES> 20%) continue to swell even after 40000 minutes.

A careful look shows that the time vs. percent swell responses (Fig. 6.1) are of rectangular hyperbolic in shape. Hence it can be written as

$$\frac{t}{S} = a + bt \tag{6.1}$$

Where, t is time, S is percent swell, a and b are the intercept and slope of the straight line.

The nearly linear relationship obtained between time and time/swell (Fig. 6.3 and 6.4) establishes that the time-percent swell responses are rectangular hyperbolas in shape. The maximum swelling is the asymptotic value of the hyperbola which can be estimated as

$$S_{\max} = \lim_{t \to \infty} S(t) = \lim_{t \to \infty} \left(\frac{1}{a/t + b}\right) = \frac{1}{b} \tag{6.2}$$

Hence from a limited data taken over a short interval of time the swell at higher times and the maximum swell, can be predicted through the rectangular hyperbolic relation. Earlier attempts have been made to predict the swell response using this procedure (Daskhianamurthy, 1978; Rao and Kodandaramaswamy, 1981; Shridharan et al., 1986; Sridharan and Gurtug, 2004).

Fig. 6.5 shows good match between the predicted and the measured swell over 30 days, for the soils used in the present study. This establishes that the rectangular hyperbola method is valid for the present set of soils. Therefore for the soils wherein maximum swell has not been reached, because the tests were terminated at 30 days, the rectangular hyperbola method is now used to predict the same. The maximum swell (i.e. swell potential, S_p) of the soils along with their index properties are presented in Table 6.2.

It could be observed that the variation of the swell potential with the index properties (Table 6.2) is similar to that of free swell index (Table 6.1), indicating that the mechanical effects between the soil grains in the two cases is nearly same. The similarity in behaviour between the oedometer swell and free swell responses indicates that the swell potential (S_p) can be correlated with the free swell index (FSI) which has been attempted in Fig. 6.6. It can be noted that the variation between the two is nearly linear. The correlation equation may be written as

$$S_p = 4.45(FSI) \qquad 6.3$$

with a correlation coefficient (R^2) of 0.95.

Where, S_p is the swell potential in % and FSI is the free swell index in ml/gm. From Table 6.2 it can further be observed that the swell potential is larger for soils having higher plasticity characteristics. It was observed that swell potential increases very steeply with plastic limit and decreases with shrinkage limit. A higher value of swell potential is observed at low shrinkage limit.

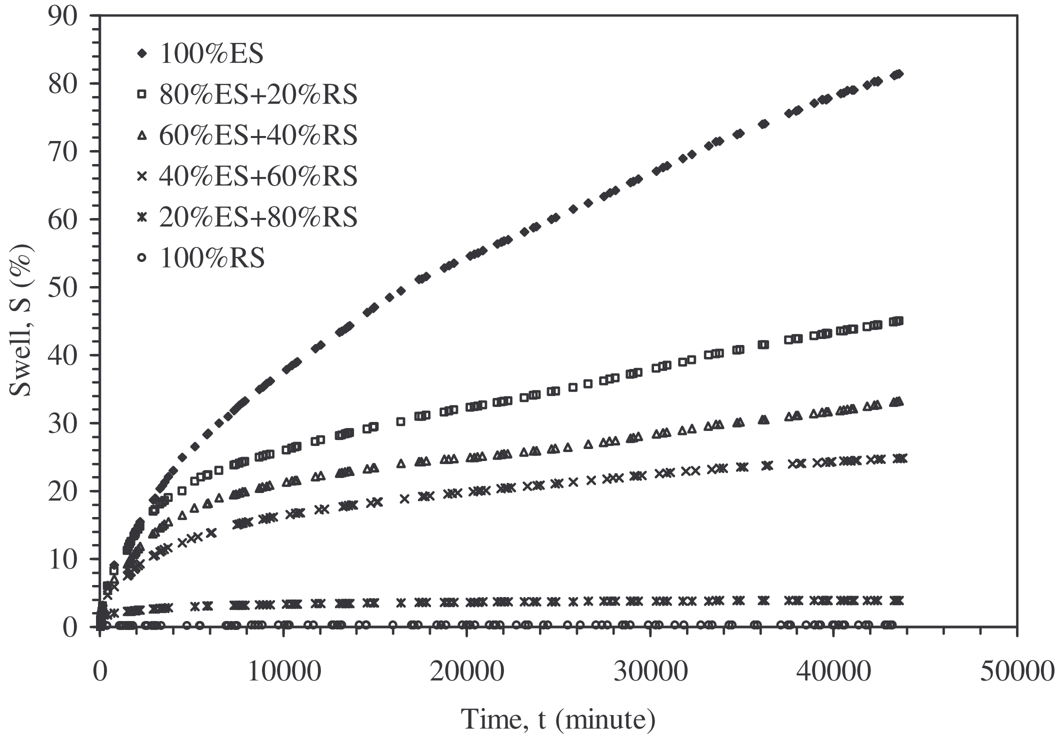

Fig. 6.1 Time vs. percent swell responses for soils

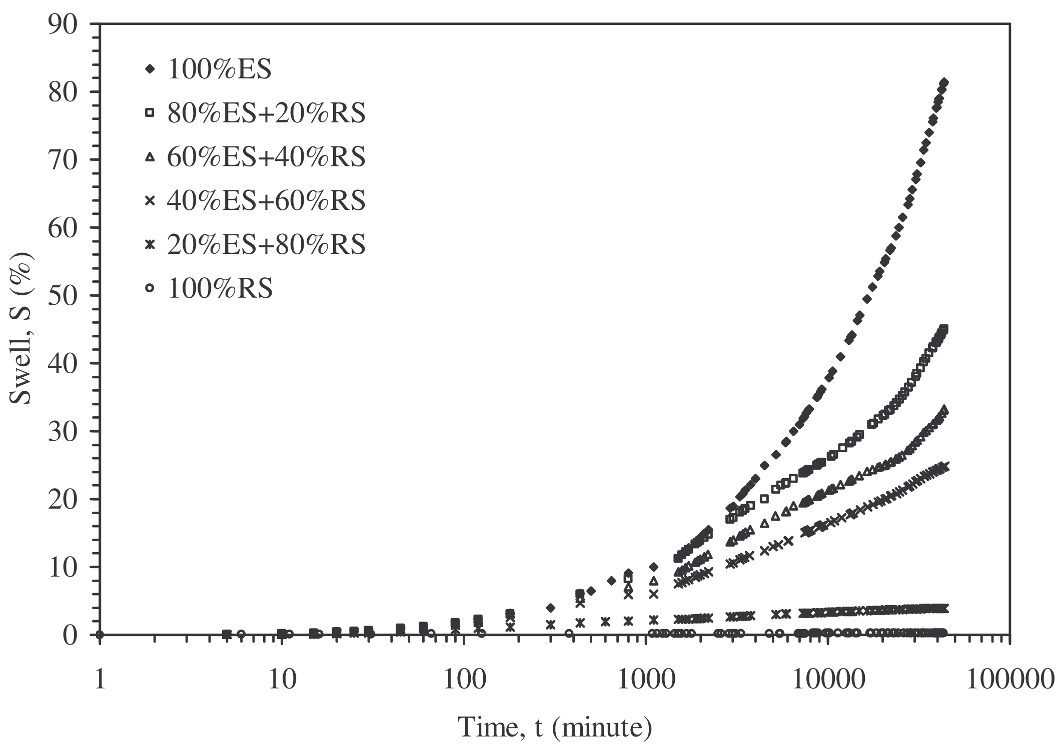

Fig. 6.2 Log time vs. percent swell relationship for soils

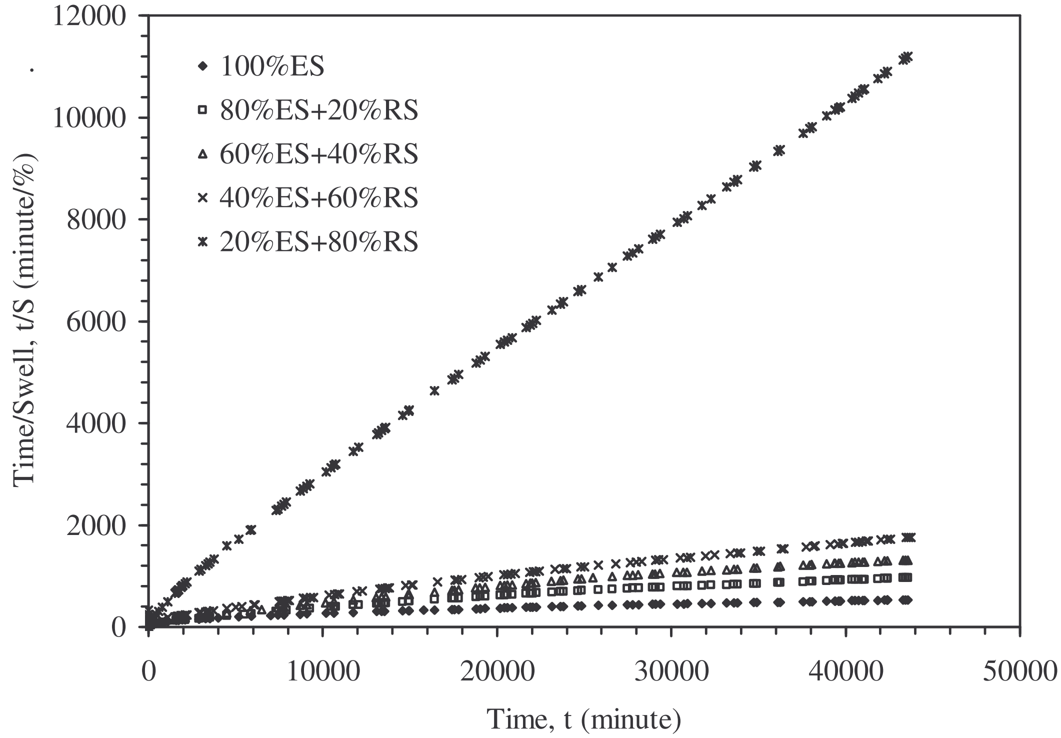

Fig. 6.3 Time vs. (time / swell) relationship for soils

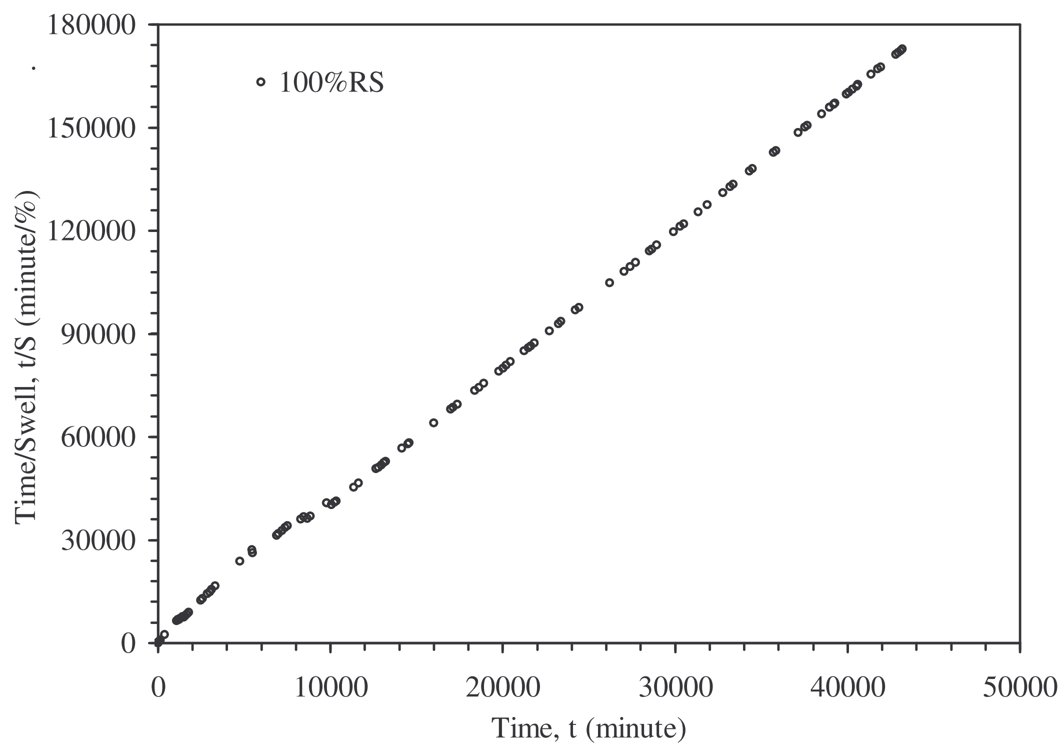

Fig. 6.4 Time vs. (time / swell) relationship for residual soil

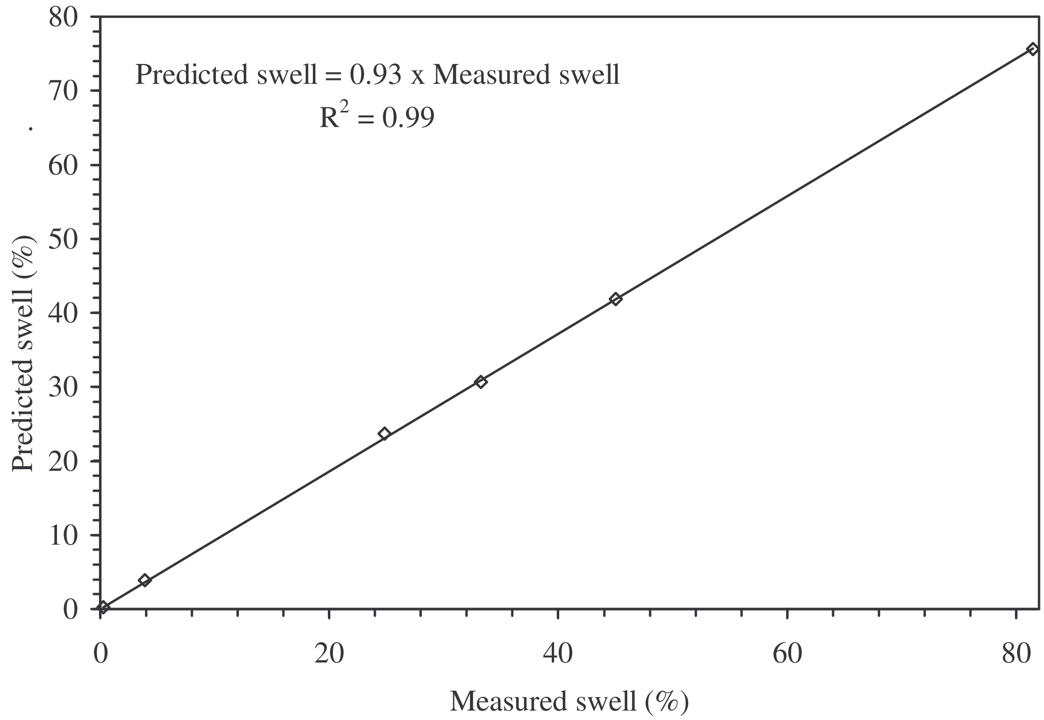

Fig. 6.5 Measured swell vs. predicted swell for soils (ES, ES+RS, RS)

Table 6.2 Swell potential variation with index properties

Soil	Liquid limit (%)	Plastic limit (%)	Shrinkage limit (%)	Specific surface area, m^2/gm	Swell potential (%)
100%ES	459.9	53.7	7	86.45	97.09
80%ES+20%RS	351.9	49.4	8.97	83.24	48.08
60%ES+40%RS	248.2	38.1	9.89	65.41	34.25
40%ES+60%RS	178.8	30.7	10.29	55.32	26.32
20%ES+80%RS	99.2	28.7	13.36	49.15	3.89
100%RS	45.3	25.9	25.45	38.76	0.25

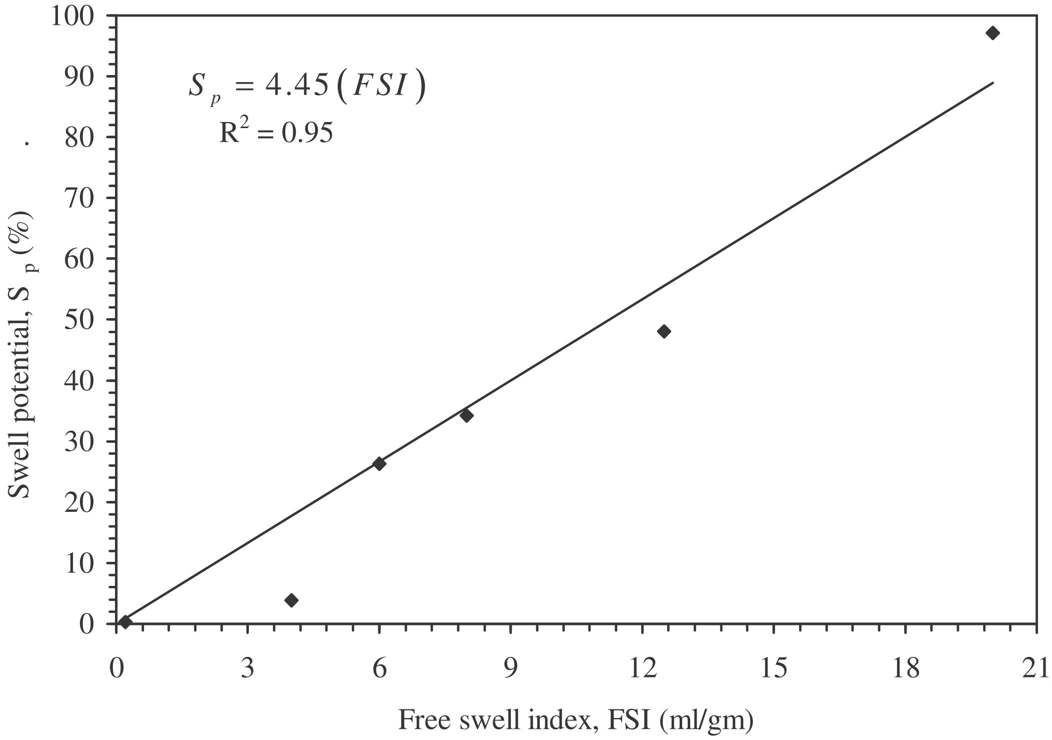

Fig. 6.6 Free swell index vs. swell potential for soils

Swell pressure is defined as the pressure required to compress a specimen, in the Oedometer, which has been swollen due to soaking under a given seating pressure, back to its original thickness (i.e. before swelling). Fig. 6.7 shows the change in sample height with applied pressure. It could be observed that for soils with higher plasticity (i.e ES ≥ 40%) initially the reduction in sample height with increased pressure is relatively small. However, after a certain stage there is a steep reduction in sample thickness with increased pressure. It is believed that initially the pore water sustains majority of applied pressure. Since it is viscous, due to the charge of the plastic clay soil, it takes some time to get drained out. Once drained out the surcharge pressure is transferred to the soil skeleton that it collapses leading to substantial reduction in sample thickness. The swell pressure is the one when the soil sample has reached its original height (i.e. change in sample height = 0).

Fig. 6.8 presents the percent swell versus swell pressure for all the soils. For high swelling soils there is a nearly linear relation between swell and swell pressure. This is because in such high plastic soils due to presence of large volume of diffused double layer and adsorbed water surrounding the soil particles, the soil grain to grain interaction under the applied pressure is marginal. Since the change in height is mostly due to expulsion of this water rather than soil grain rearrangement the response is linear. However for low swelling soils there is deviation from linearity. In such low plastic soils due to less quantity of adsorbed water, it is the rearrangement of grains that contributes substantially to the reduction in sample thickness. As this interaction is different for different soils the swell vs. swell pressure have shown non linear variation.

Overall the correlation equation between these two parameters can be written as

$$P_s = 12.56(S) \qquad 6.4$$

with a correlation coefficient (R^2) of 0.99. Where, swell pressure (P_s) is in kPa and swell (S) is in percent.

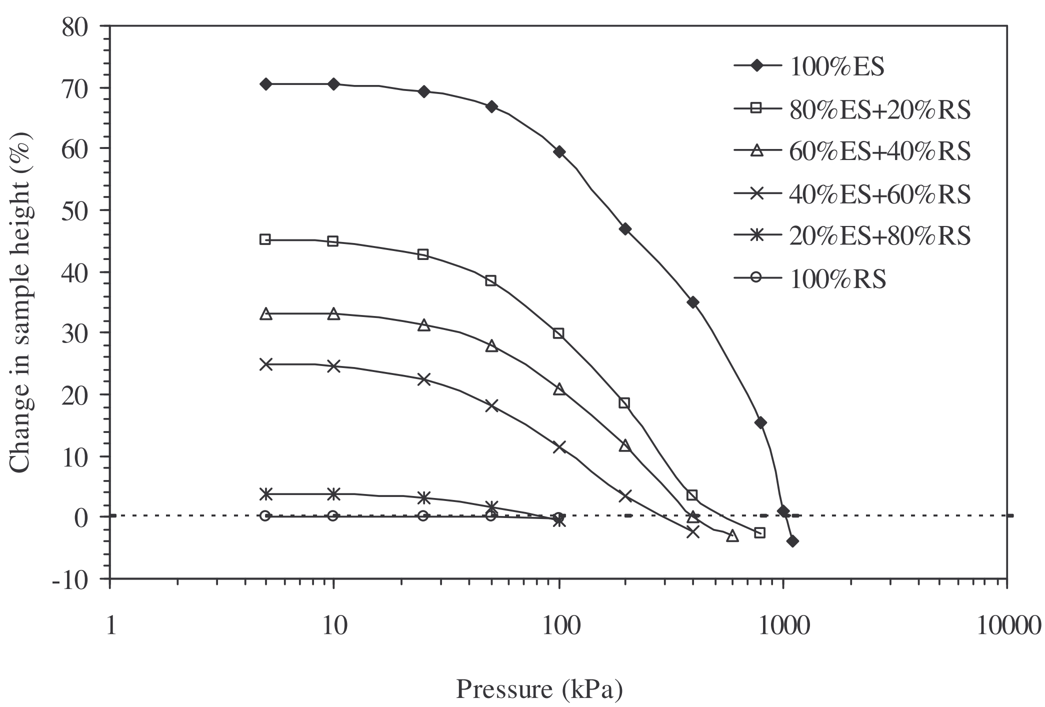

Fig. 6.7 Percent change in sample height versus applied pressure for soils.

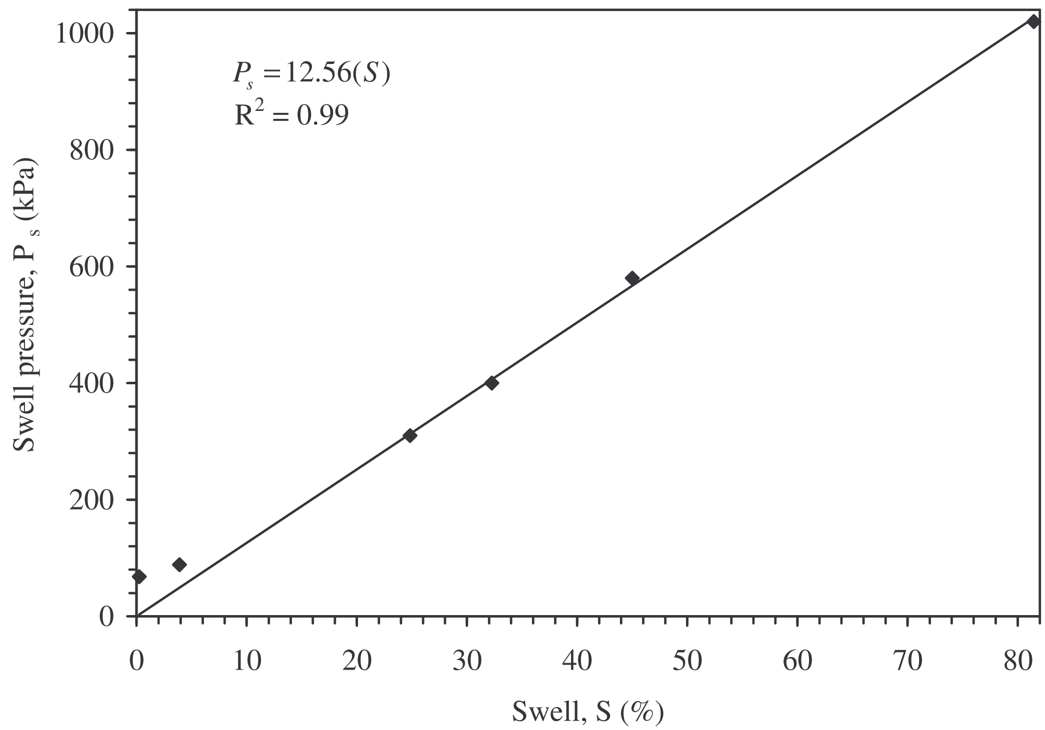

Fig. 6.8 Swell vs. swell pressure for soils

6.2.2 Swell Behaviour of ES-RS-Lime Mixes

6.2.2.1 Free Swell

Fig. 6.9 depicts the variation of free swell index (FSI) with percentage of lime added to different soils. It could be observed that the expansive soil (100%ES) originally had a free swell index of 20 ml/gm. With addition of lime it sharply reduces and comes to about 4 ml/gm at 3% of lime. Similar such behaviour is observed for the soil mixes (ES+RS) however the order of reduction in FSI has gradually decreased with increased content of residual soil. The reduction in free swell index value with increased lime content is attributed to the reduced thickness of diffuse double layer due to increased electrolytic concentration in the water brought about by lime (CaO). With increased percentage of residual soil in the mixes the clay content reduces. As a result of which the sum total diffuse double layer water reduces in volume. Therefore reduction in the free swell index due to addition of lime, being primarily due to depression of the diffuse double layer, shows lesser degree of reduction. At about 3% lime there takes place saturation of electrolytic concentration and therefore further addition of lime has not shown much change in the free swell index value. It is of interest to note that the pattern of variation in liquid limit of the soils with percentage of lime added, depicted in Fig. 6.10, is almost same as that of the free swell index. Hence it can be said that the diffuse double layer is the primary mechanism that influence, both free swell index test and liquid limit test of expansive soils.

Besides, it can also be concluded that lime treatment is more effective for soils having higher clay content. In case of residual soil the free swell index shows an increase with increase in lime content till about 3% lime, beyond which it remains nearly constant with further addition of lime. In the nonexpansive residual soil the water is mostly held within the flocculated structure of the soil matrix. With addition of lime the flocculation increases leading to increased free swell index. However the liquid limit values do not show any significant change with increased lime content. This is because unlike free swell test where the soil remains undisturbed, in the liquid limit test it is remoulded, as a result of which the flocculated structure breaks. In the absence of the flocculated structure the water holding capacity of the soil does not increase and so is the liquid limit.

Fig. 6.11 shows a very good correlation between free swell index and liquid limit (no curing) for lime treated soils. The equation can be written as

$$FSI = 0.03(LL) \qquad 6.5$$

with a corelation coefficient (R^2) of 0.92. Where, free swell index (FSI) is in ml/gm and liquid limit (LL) is in percentage.

Fig. 6.12 shows that the free swell index of lime treated soils increases with their plastic limit, however the corelation between the two is poor. There is almost no correlation between the free swell index and shrinkage limit (Fig. 6.13).

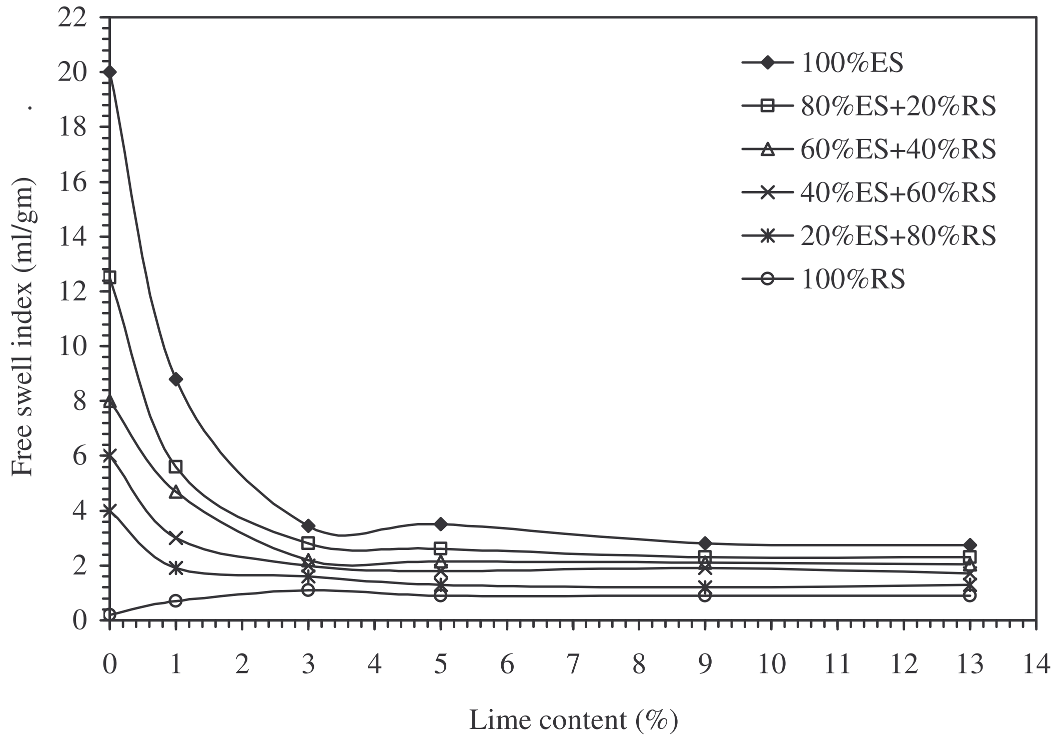

Fig. 6.9 Variation of free swell index with lime content of soils

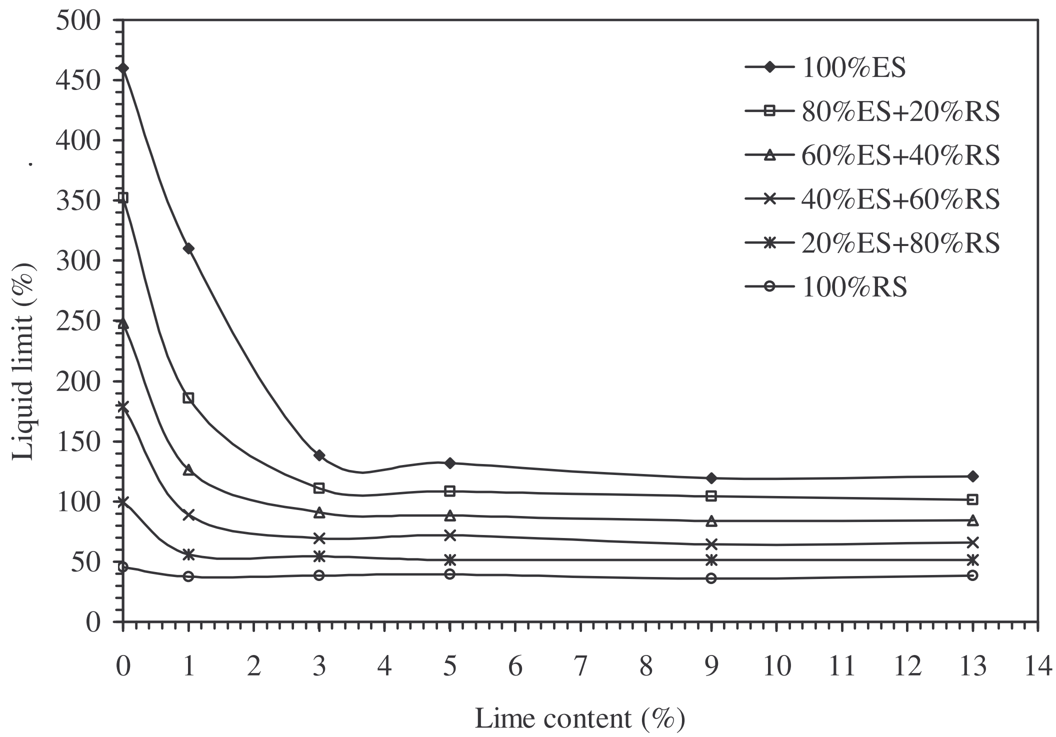

Fig. 6.10 Variation of liquid limit with lime content for lime treated soils without curing.

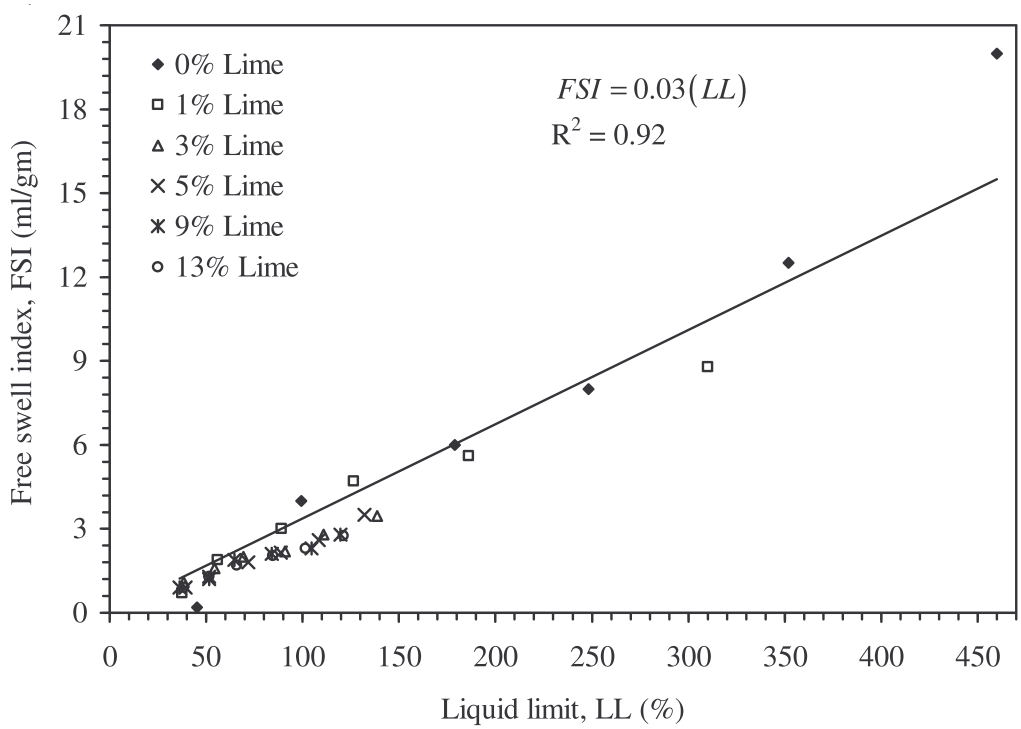

Fig. 6.11 Free swell index vs. liquid limit for lime treated soils.

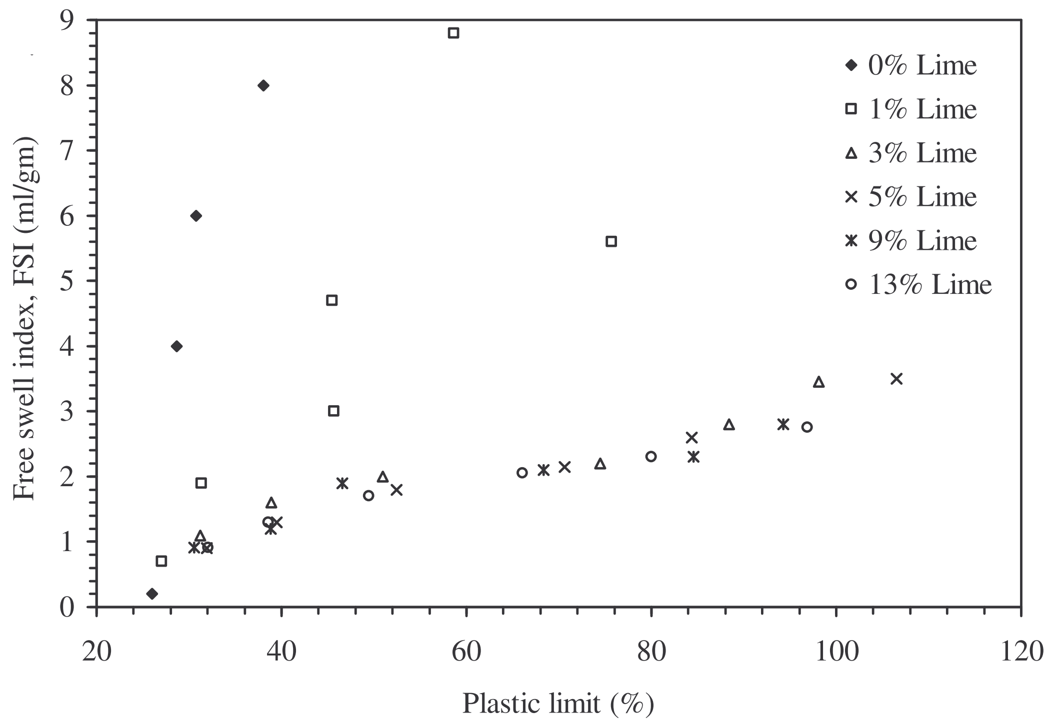

Fig. 6.12 Free swell index vs. plastic limit for lime treated soil.

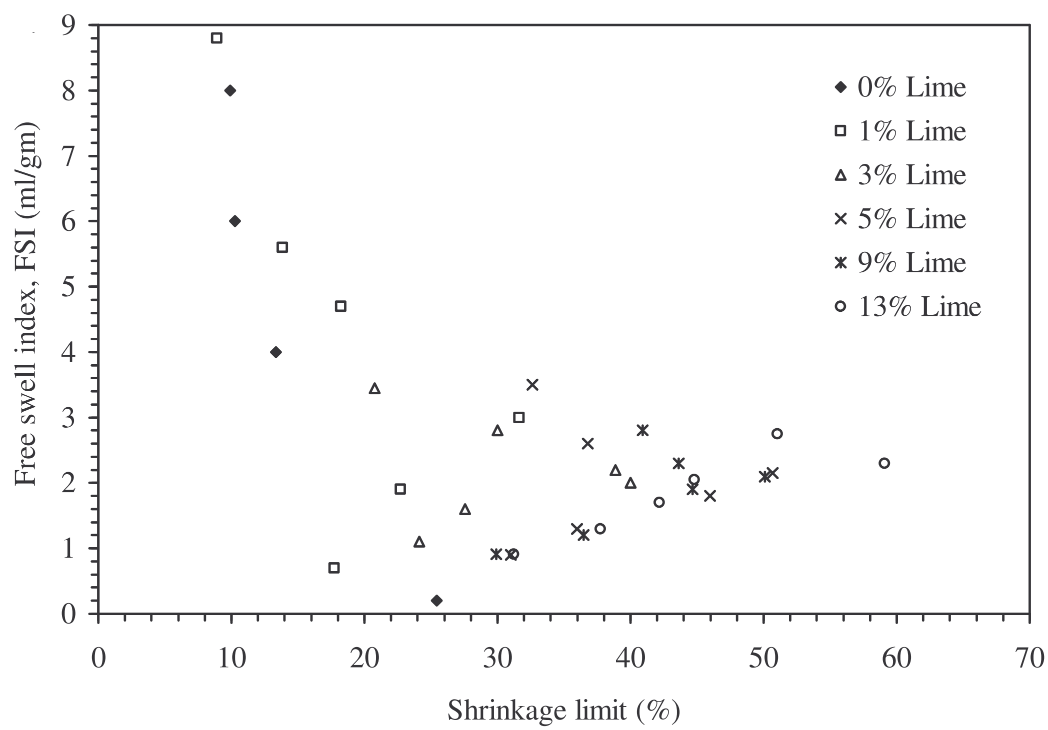

Fig. 6.13 Free swell index vs. shrinkage limit for lime treated soil.

6.2.2.2 Oedometer Swell

The time-swell responses with varied lime contents (0-13%) for soils; 100%ES, 80%ES+20%RS, 60%ES+40%RS, 40%ES+60%RS, 20%ES+80%RS and 100%RS are depicted in Fig. 6.14, 6.16, 6.18, 6.20, 6.22 and 6.24 respectively. In these plots the responses for untreated soils (i.e. 0% lime) are included for comparison purpose. It could be observed that even with addition of 1% lime the swelling has been substantially reduced. With further increase in lime content swelling continues to reduce to reach a practically negligible value. The corresponding log time versus percent swell responses depicted in Figs. 6.15, 6.17, 6.19, 6.21, 6.23 and 6.25 show that the slope of the responses have undergone reduction with increased percent of lime. This indicates that the rate of swelling too reduces with increase in lime content. The reduction in swelling and rate of swelling with lime is attributed to the increased electrolytic concentration giving rise to reduced thickness of the diffuse double layer. However it is of interest to note that after this initial reduction the swelling of soils has once again increased with further addition of lime. This phenomenon has manifested in greater prominence in case of soils having higher percentage of residual soil. This is partly attributed to fabric change, that the soil at higher lime content, with the pH value increasing, tends to be flocculated in structure thereby holds more water and hence induces higher swelling. The fabric aspect has been brought out in more detail in a later section. Other major reason is attributed to pozzolanic reaction that produces gelatinous cementitious products which holds substantial quantity of water onto it giving rise to increased swelling. As silica is a major source for such gel formation in the silica rich residual soils the swelling has increased high, so much that it has surpassed the untreated soils (Fig. 6.22, 6.24).

In most of the cases the swelling of lime treated soil after initial increase has become asymptotic. This is due to the formation of cemented bonds that holds the soil particles against swelling. However with 1% lime the swelling continues even after 28 days (i.e. 40000 minutes). In case of residual rich soils with higher lime content the time-swell response shows a rectangular shape, but when plotted as log time vs. swell (%) it shows a nearly rectangular hyperbolic shape.

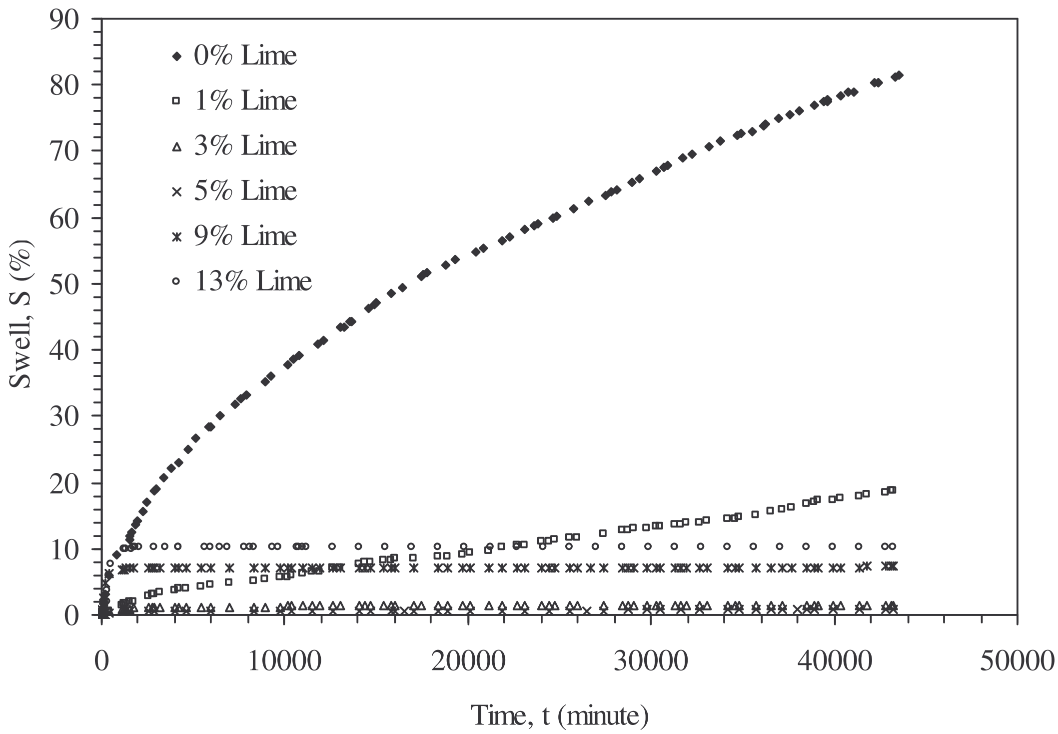

Fig. 6.14 Time-swell behaviour of lime treated soil (100%ES)

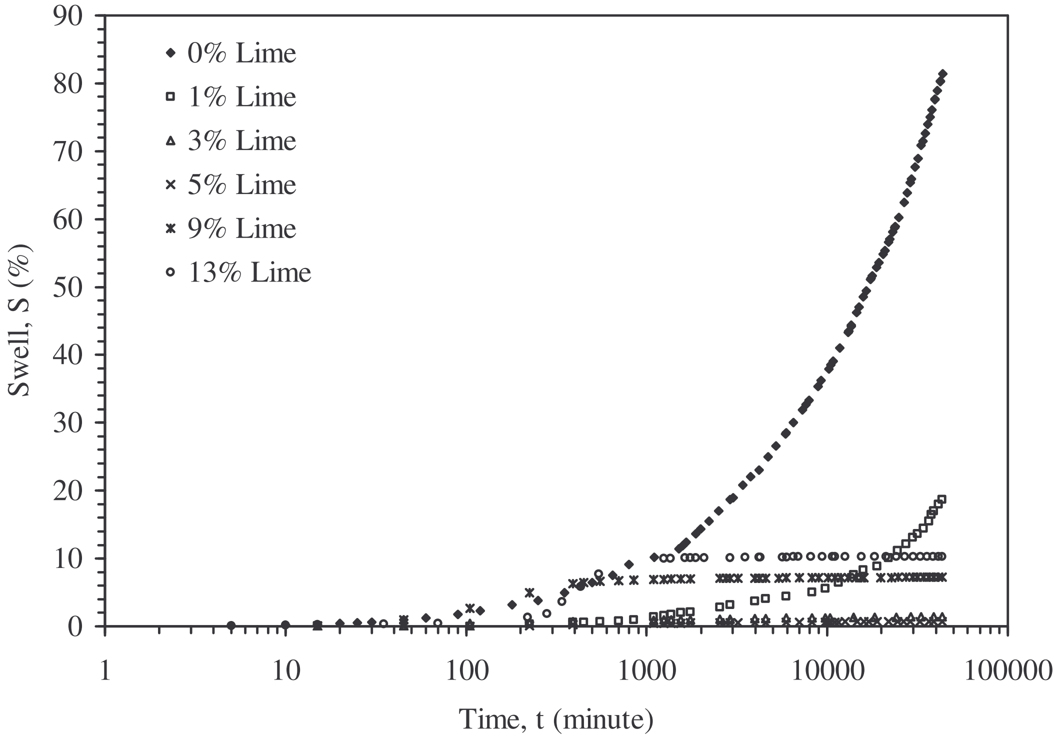

Fig. 6.15 Log time vs. percent swell relationship for lime treated soil (100%ES)

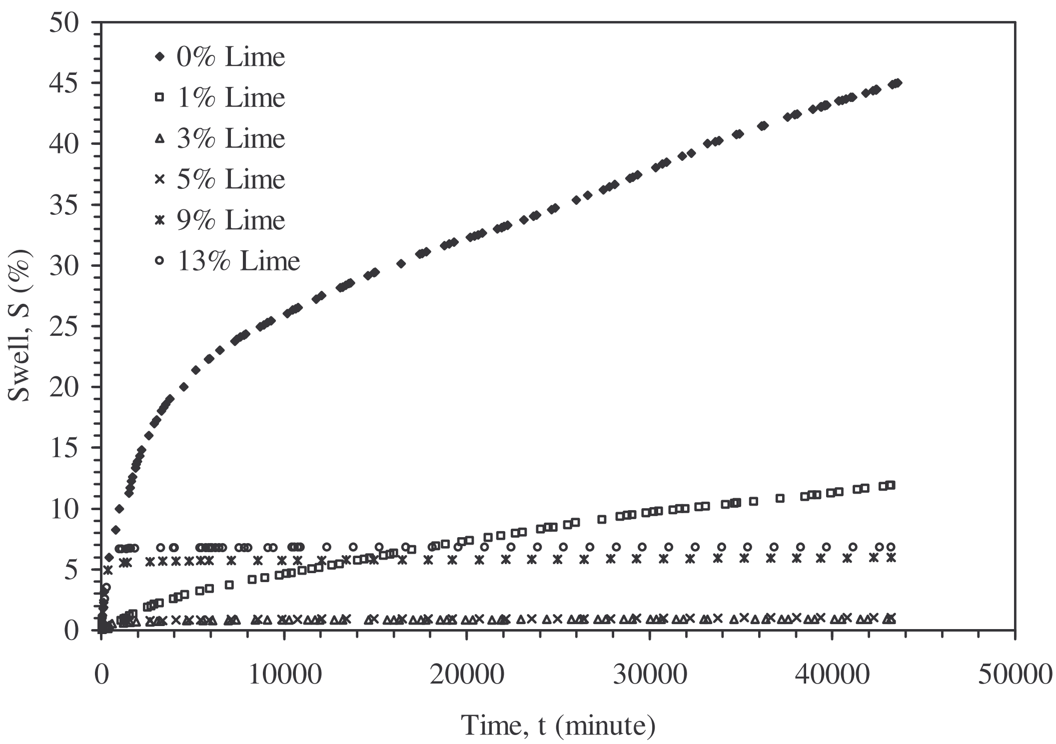

Fig. 6.16 Time-swell behaviour of lime treated soil (80%ES+20%RS)

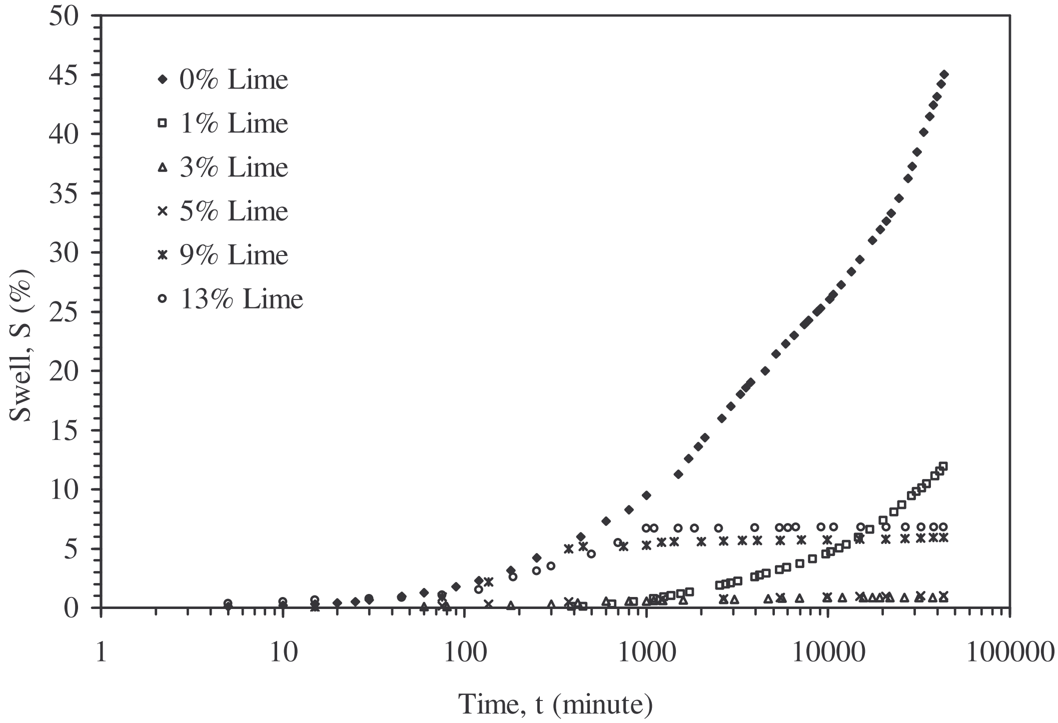

Fig. 6.17 Log time vs. percent swell relationship for soil (80%ES+20%RS)

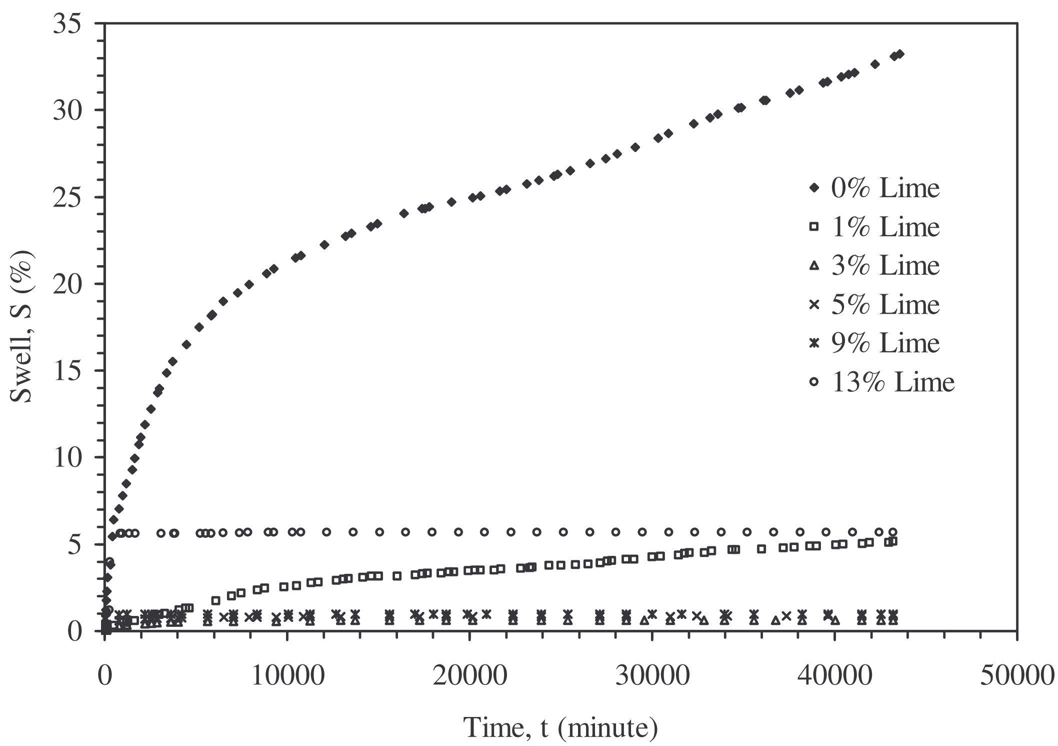

Fig. 6.18 Time-Swell behaviour of lime treated soil (60%ES+40%RS)

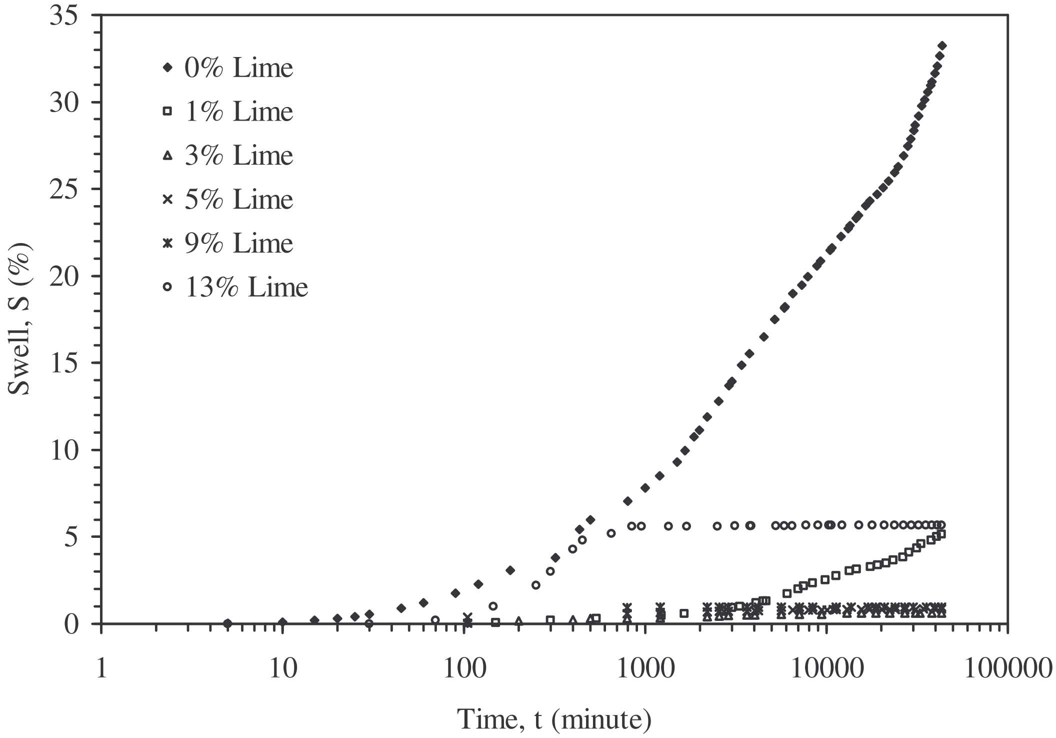

Fig. 6.19 Log time vs. percent swell relationship for lime treated soil (60%ES+40%RS)

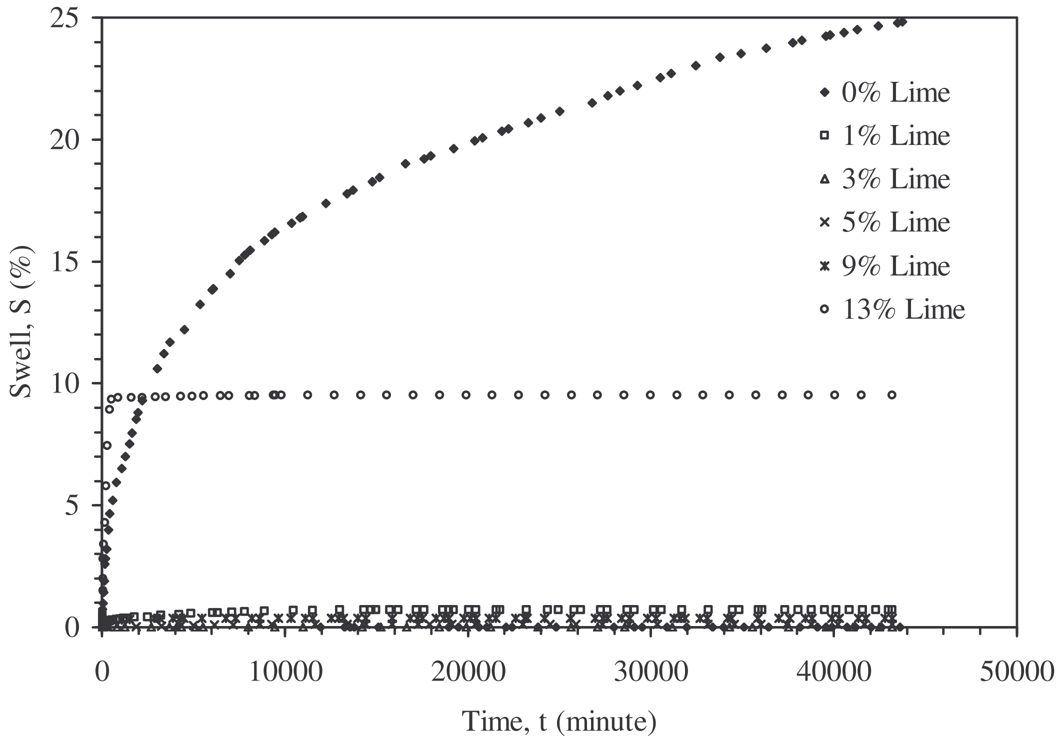

Fig. 6.20 Time-Swell behaviour of lime treated soil (40%ES+60%RS)

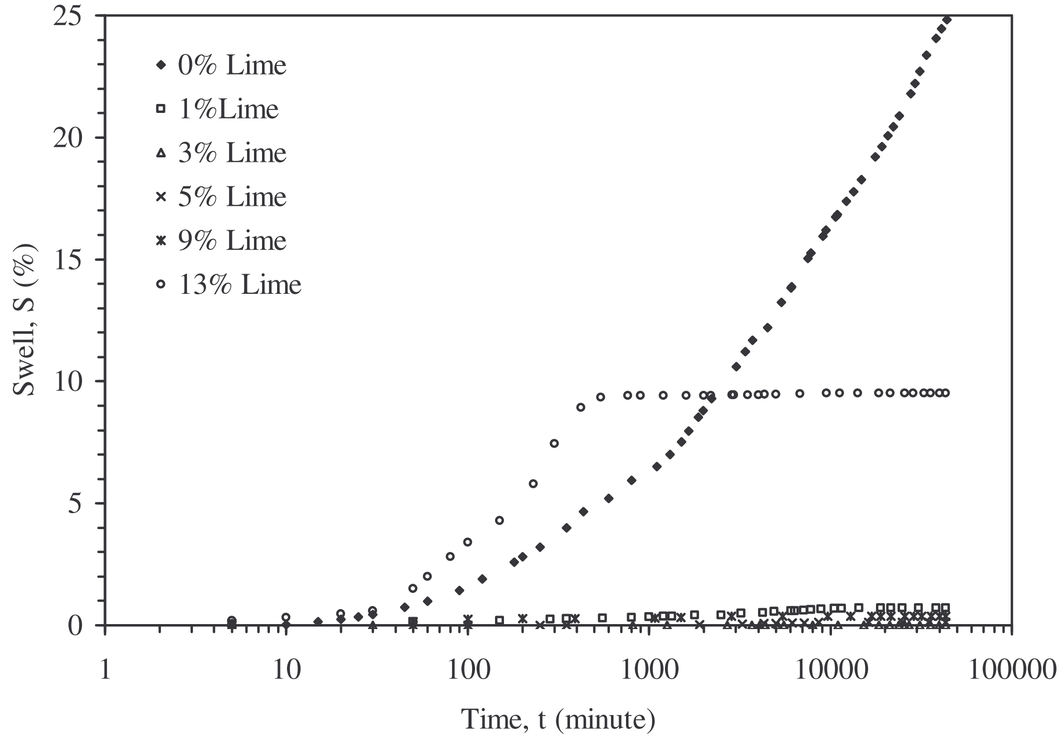

Fig. 6.21 Log time vs. percent swell relationship for lime treated soil (40%ES+60%RS)

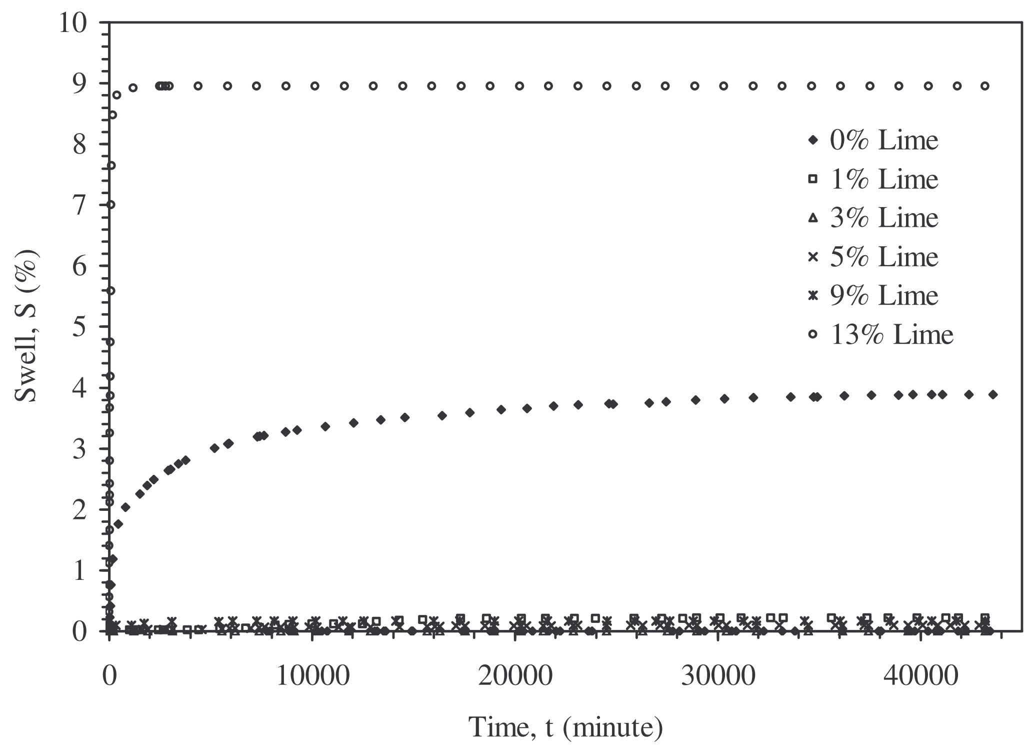

Fig. 6.22 Time-Swell behaviour of lime treated soil (20%ES+80%RS)

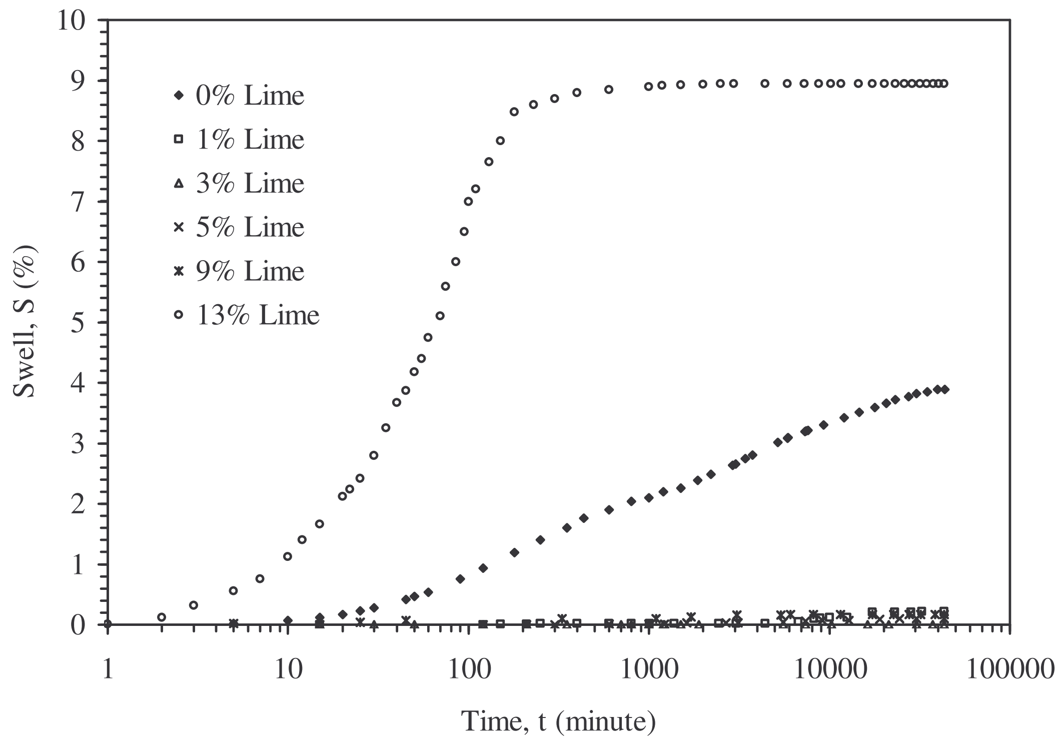

Fig. 6.23 Log time vs. percent swell relationship for lime treated soil (20%ES+80%RS)

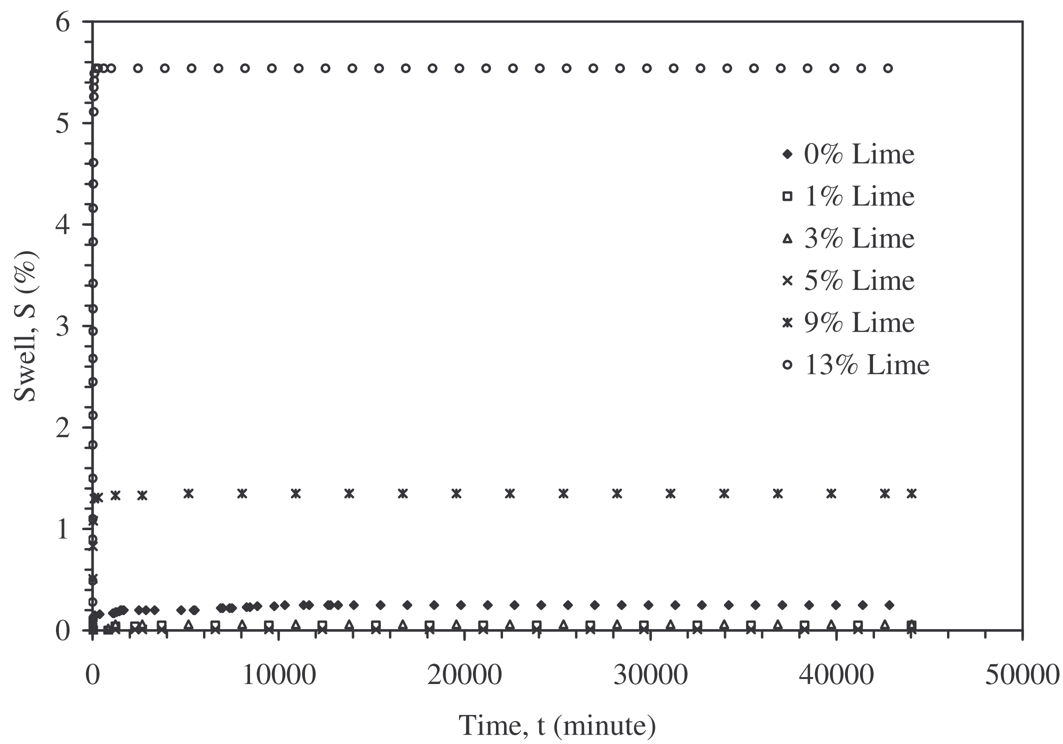

Fig. 6.24 Time-Swell behaviour of lime treated soil (100%RS)

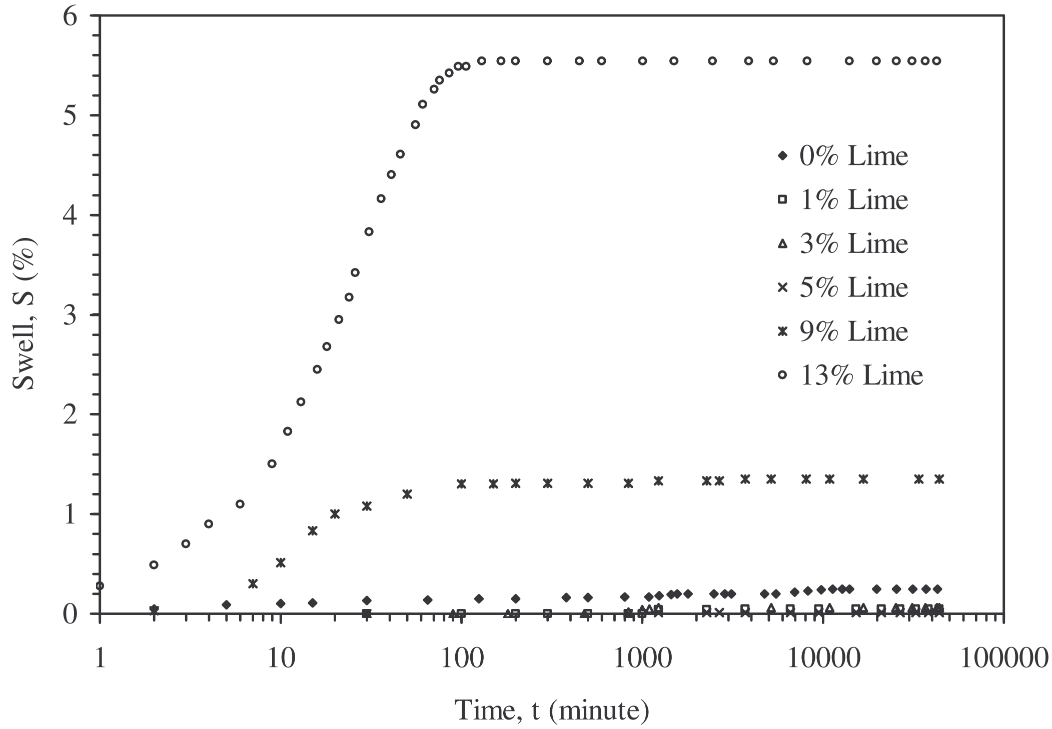

Fig. 6.25 Log time vs. percent swell relationship for lime treated soil (100%RS).

In order to examine if rectangular hyperbola concept can be applied to lime treated soils, the test data are plotted as time vs. time/swell (Fig. 6.26-6.31). The linear responses indicates that the rectangular hyperbola method can be applied to the lime treated soils as well. In few cases (Fig. 6.29 and 6.30) there has occurred an irregular pattern of swell in the initial stages and the linear response appears at a later stage. In these cases the soil has higher proportion of residual soil (i.e. 60%RS and 80%RS). The coarse residual soil grains with finer expansive soil in between are prone to slide past over each other. Due to cementation induced volume change and surcharge pressure induced deformation the coarse soil mass tends to get rearranged leading to change in fabric, which causes deviation of swell response from linearity. However at a later stage when this readjustment attains equilibrium the swell response has shown linearity. Using the intercept (a) and slope (b) of the straight lines in the plots depicted in Fig. 6.26-6.31 the swell at 30 days, for soils with varied lime contents, have been calculated using the hyperbolic relation, S = t / (a+bt). The predicted swells are compared with the measured ones in Fig. 6.32. It can be observed that the predicted swell and measured swell values are almost same ($R^2 = 0.99$), indicating that rectangular hyperbola method can be used for predicting the time-swell behaviour of lime treated soils.

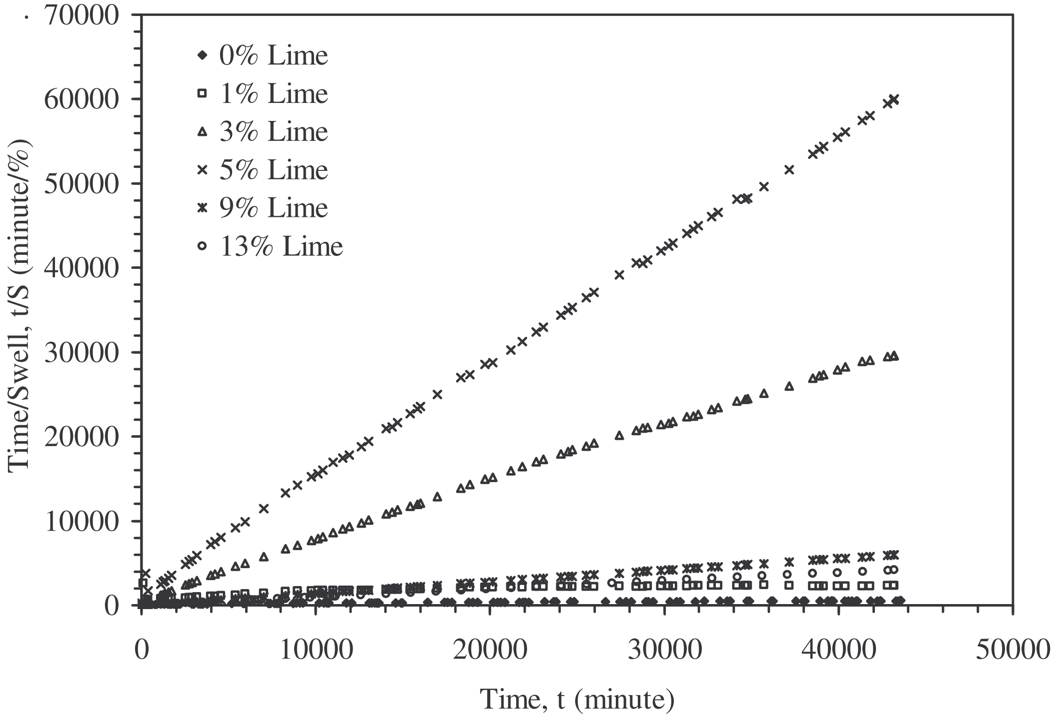

Fig. 6.26 Time vs. (time/swell) relationship for lime treated soil (100%ES).

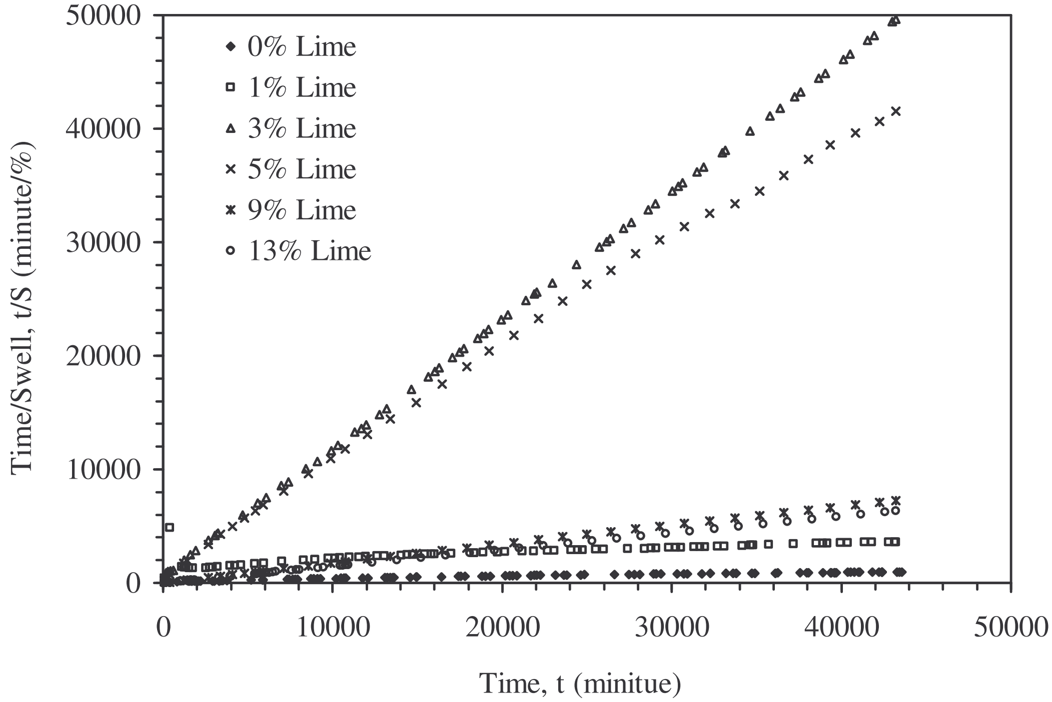

Fig. 6.27 Time vs. (time/swell) relationship for lime treated soil (80%ES+20%RS).

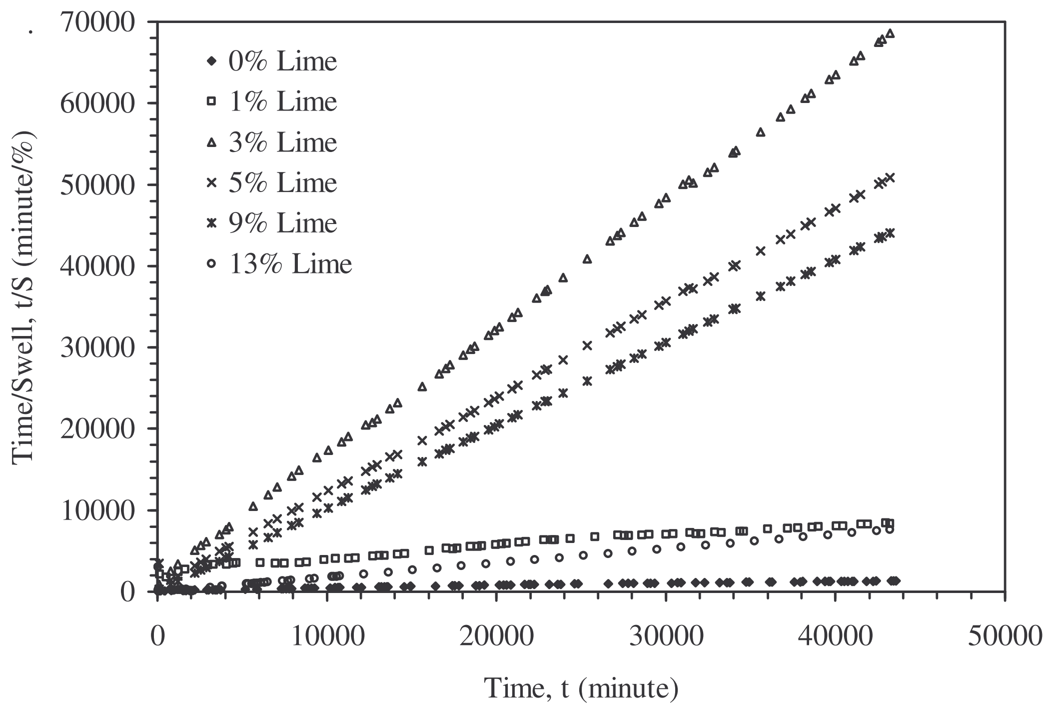

Fig. 6.28 Time vs. (time/swell) relationship for lime treated soil (60%ES+40%RS).

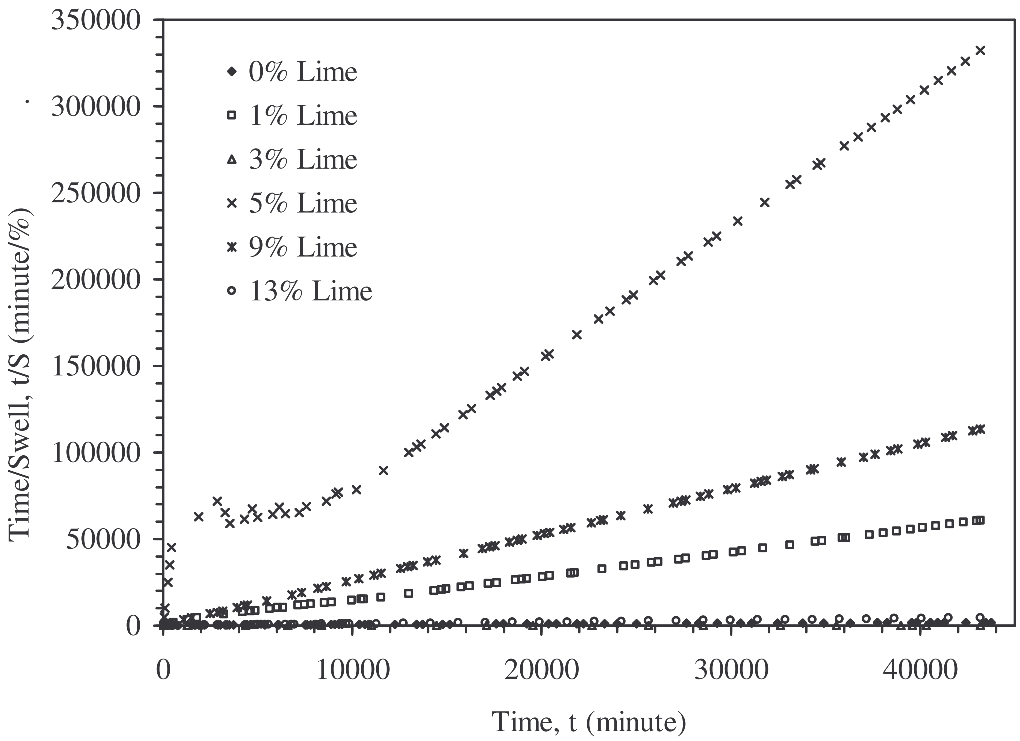

Fig. 6.29 Time vs. (time/swell) relationship for lime treated soil (40%ES+60%RS)

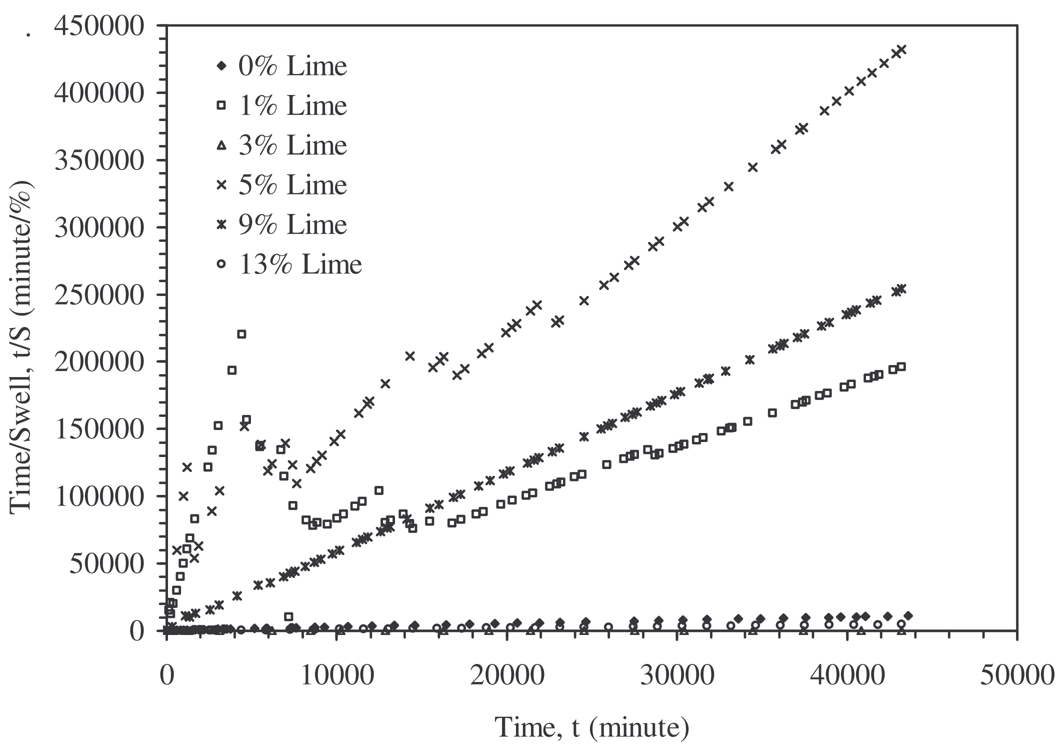

Fig. 6.30 Time vs. (time/swell) relationship for lime treated soil (20%ES+80%RS).

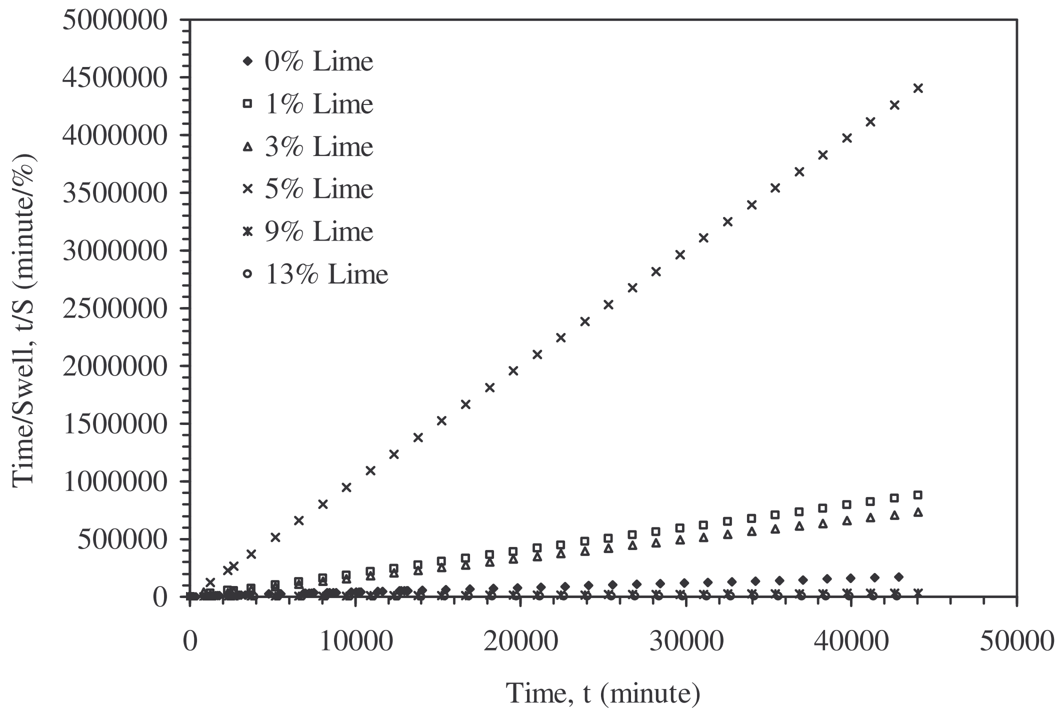

Fig. 6.31 Time vs. (time/swell) relationship for lime treated soil (100%RS).

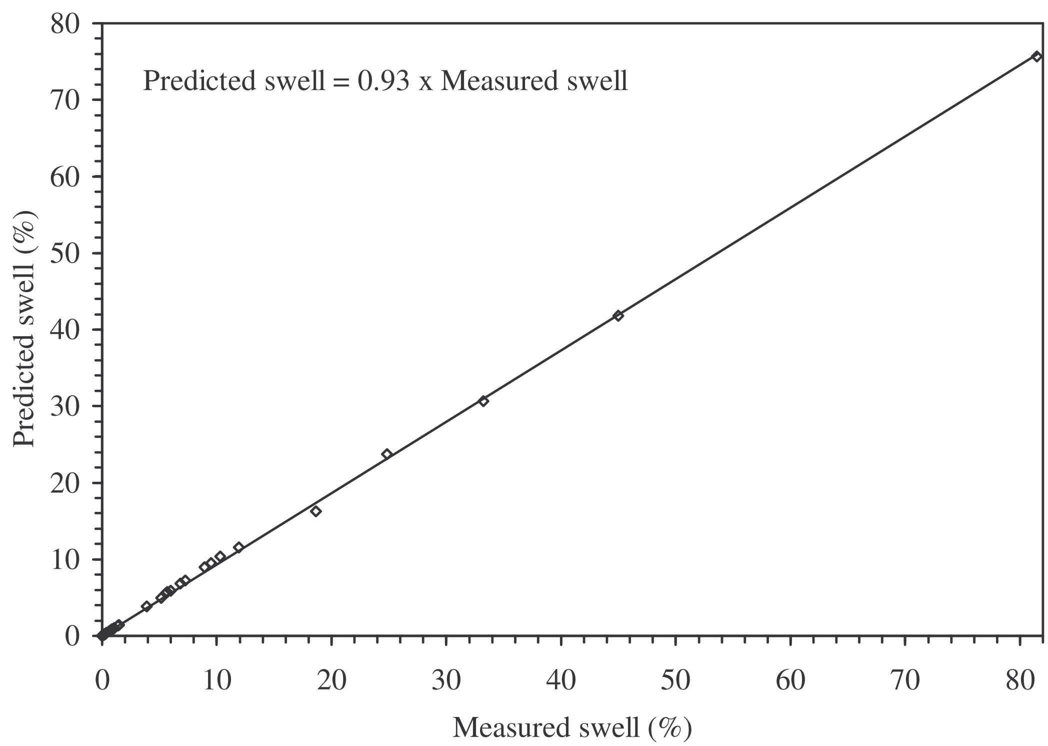

Fig. 6.32 Measured swell vs. predicted swell for lime treated soils.

Using the rectangular hyperbolic model the maximum swell (i.e. swell potential) for the lime treated soils have been obtained. The variation of swell potential with lime content for different soils is shown in Fig. 6.33. It could be observed that the swell potential of all soils initially reduces with increase in lime content to a practically negligible value, beyond which it once again increases with increase in lime content. The initial reduction in swell potential is due to depression of double layer water held onto the soils and the subsequent increase is due to the pozzolanic reaction induced gel formation as has been discussed earlier. It is of interest to note that the limit at which the soil starts to manifest increased swelling is 5% lime content for 100%ES and 80%ES+20%RS soils while it is 9% lime content for higher percentage of residual soil (i.e. 60%ES+40%RS, 40%ES+60%RS, 20%ES+80%RS, 100%RS). It should be mentioned here that the Oedometer swell tests were carried out on soil samples compacted at their maximum dry density-optimum moisture condition. With increased residual soil content their comes a stage when the coarser residual soil particles come in contact with each other forming inter clod voids. At higher lime content, as pozzolanic reaction induced gel formation takes place for expansive soil rich soils having large fines content, it starts expanding the volume of the sample leading to swelling. However for residual soil reach specimens the gel formed takes its place within the voids of the coarser nonswelling fractions. Therefore, only at large quantity of lime when the gel is more in volume (i.e higher than the intergranular voids) that it tends to manifest inform of external swelling. Hence for such soils only at higher lime content (i.e. >9%) the soil sample has shown increased swelling.

Fig. 6.34 shows the variation of swell potential with free swell index for soils with varied lime content. Though there is an increasing trend between the two but the correlation is poor. Same is the case with liquid limit and plastic limit as depicted in Fig. 6.35 and 6.36 respectively. Hence it can be said that unlike untreated soil for lime treated soil there takes place different mechanical phenomenon in the Oedometer swell tests and in the free swell test, liquid limit and plastic limit tests. The main reason is that in Oedometer swell test the cementation process is not disturbed much while in the index tests (LL and PL) they are disturbed a lot. In the free swell test the soil mass bang in a suspension state, in the water medium, the grain to grain contact is poor and hence cementation effect is weak. Therefore, while for untreated soils the Oedometer swell potential holds a good correlation with free swell index, for lime treated soil the correlation is poor. However, the swell potential shows no correlation with the shrinkage limit (Fig. 6.37).

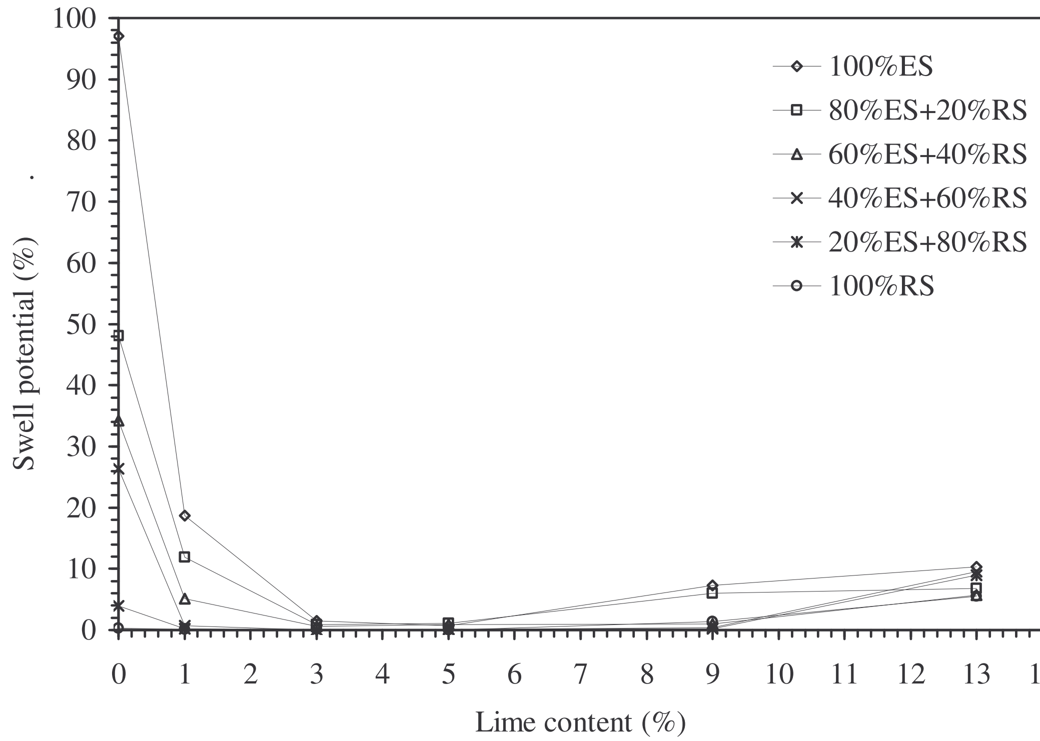

Fig. 6.33 Swell potential vs. percentage of lime for lime treated soil

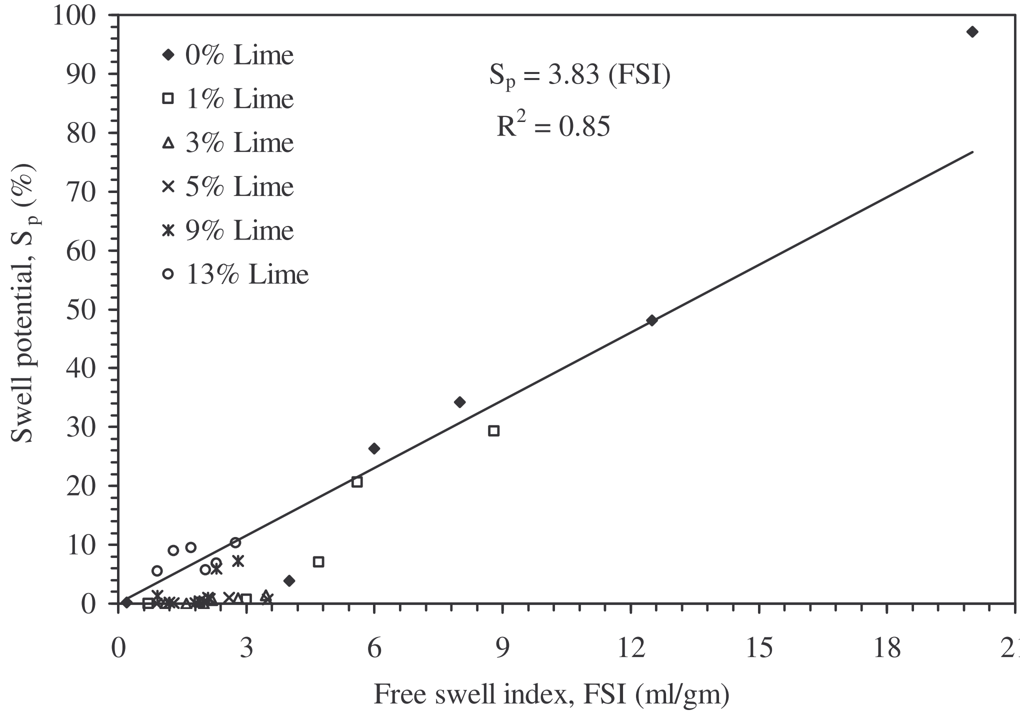

Fig. 6.34 Free swell index vs. swell potential for lime treated soils.

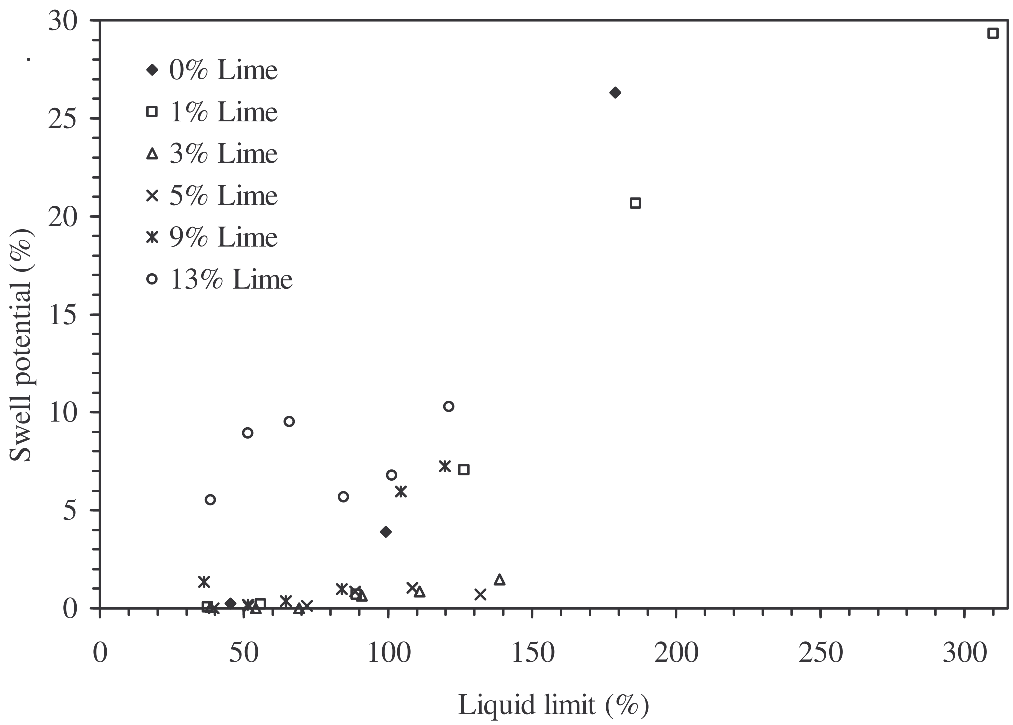

Fig. 6.35 Liquid limit vs. swell potential for lime treated soils.

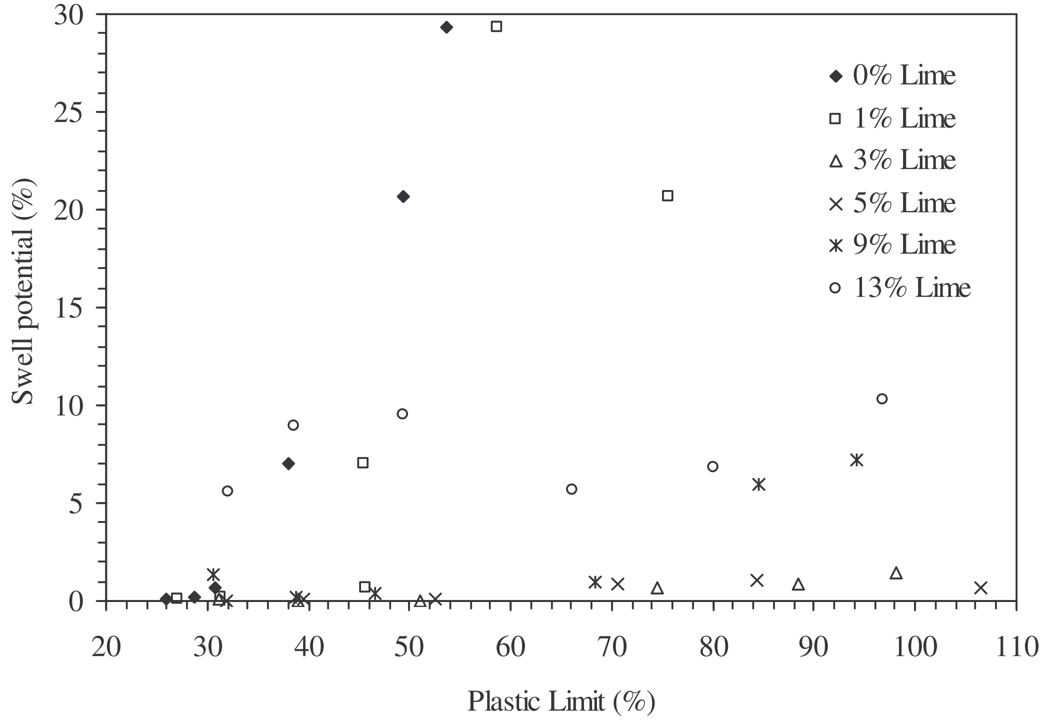

Fig. 6.36 Plastic limit vs. swell potential for lime treated soils

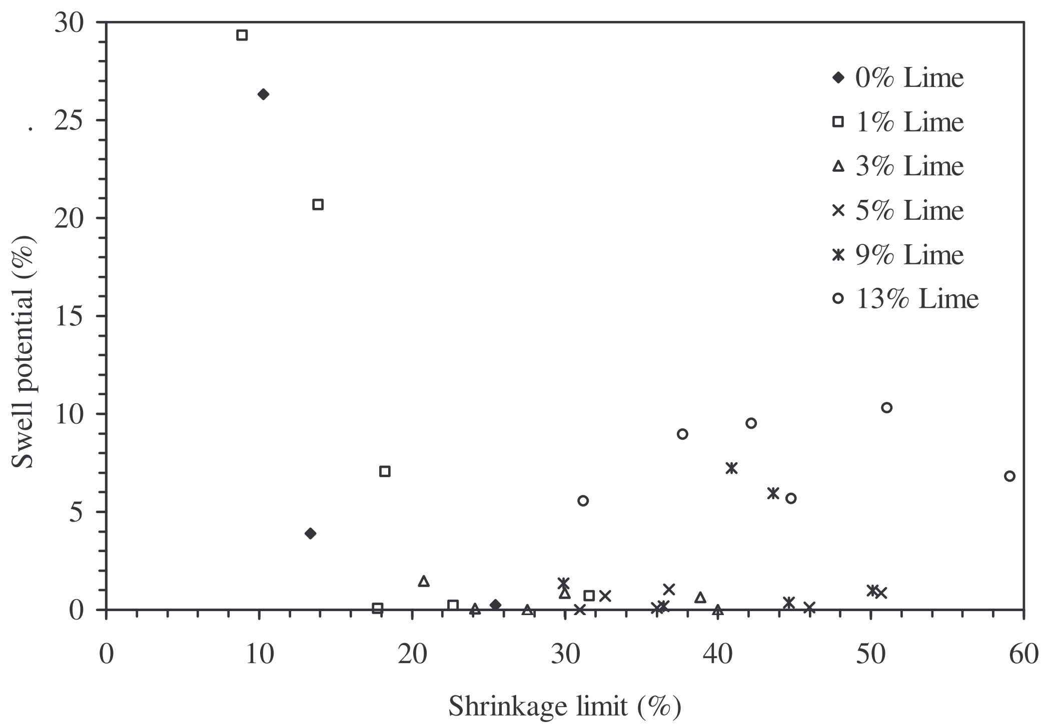

Fig. 6.37 Shrinkage limit vs. swell potential for lime treated soils.

After 30 days of swelling the soil specimens were reloaded in steps till the original height was reached. The responses depicting change in sample height as percentage of original height, with increased surcharge pressure, for different soils with varied lime content are depicted in Fig. 6.38-6.43. As defined, conventionally, the swell pressures in these cases were taken as the one where the soil sample has reached its original height (i.e. change in sample height = 0). The swell pressure thus obtained are plotted with the corresponding lime content in Fig. 6.44. It could be observed that after an initial decrease the swell pressure has increased sharply high with increased lime content. This is attributed to the cemented bonds formed due to lime and the prolonged curing period of 30 days. Therefore, to bring the lime treated soil sample to its original height it needs surcharge pressure of very high magnitude. As this pressure is used to break the cemented bonds it can not be called as the swell pressure. Hence the conventional swell pressure concept can not be applied to the lime treated soils.

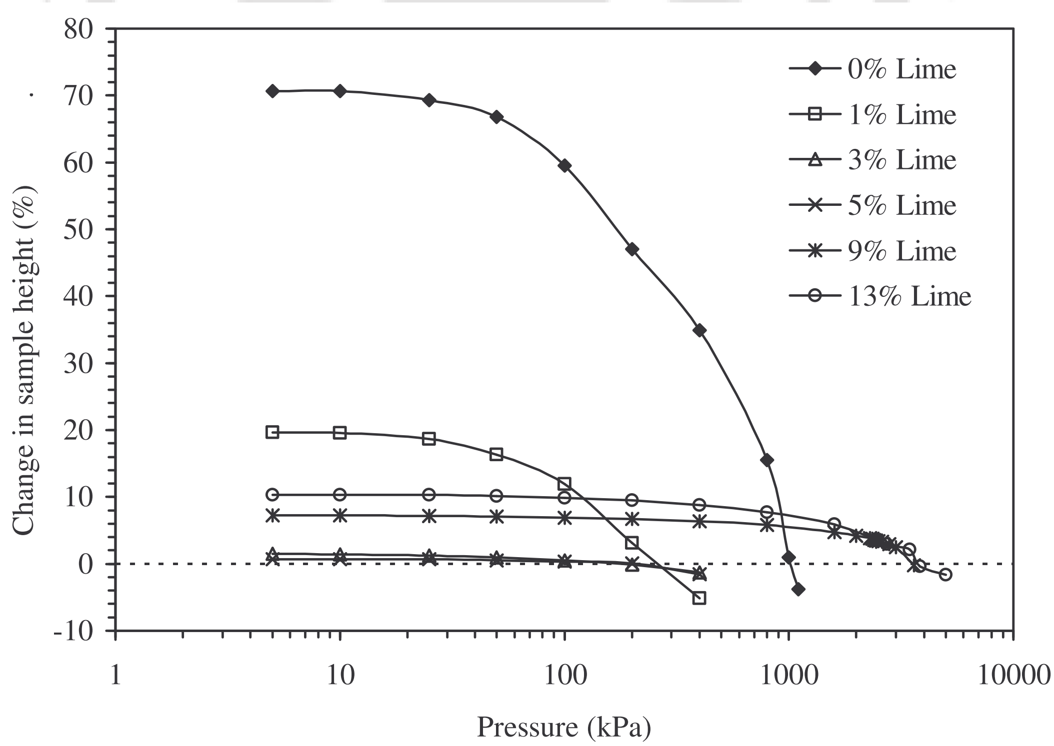

Fig. 6.38 Percent change in sample height versus loading applied pressure at different percent of lime for soil sample-100%ES

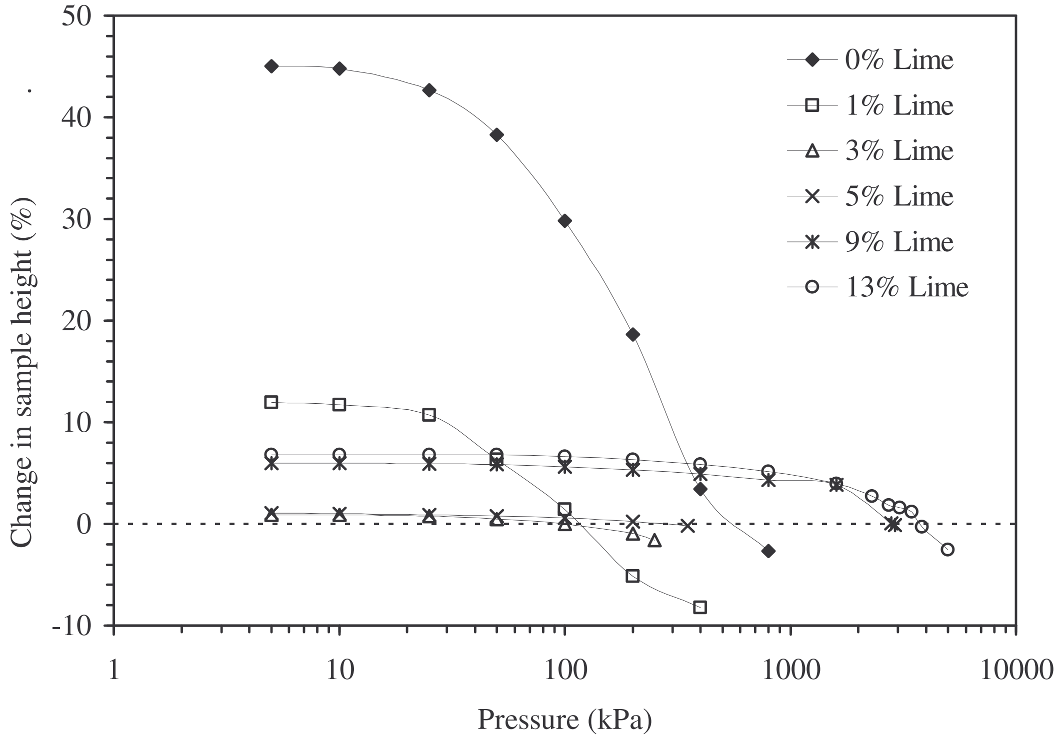

Fig. 6.39 Percent change in sample height versus loading applied pressure at different percent of lime for soil sample- 80%ES+20%RS

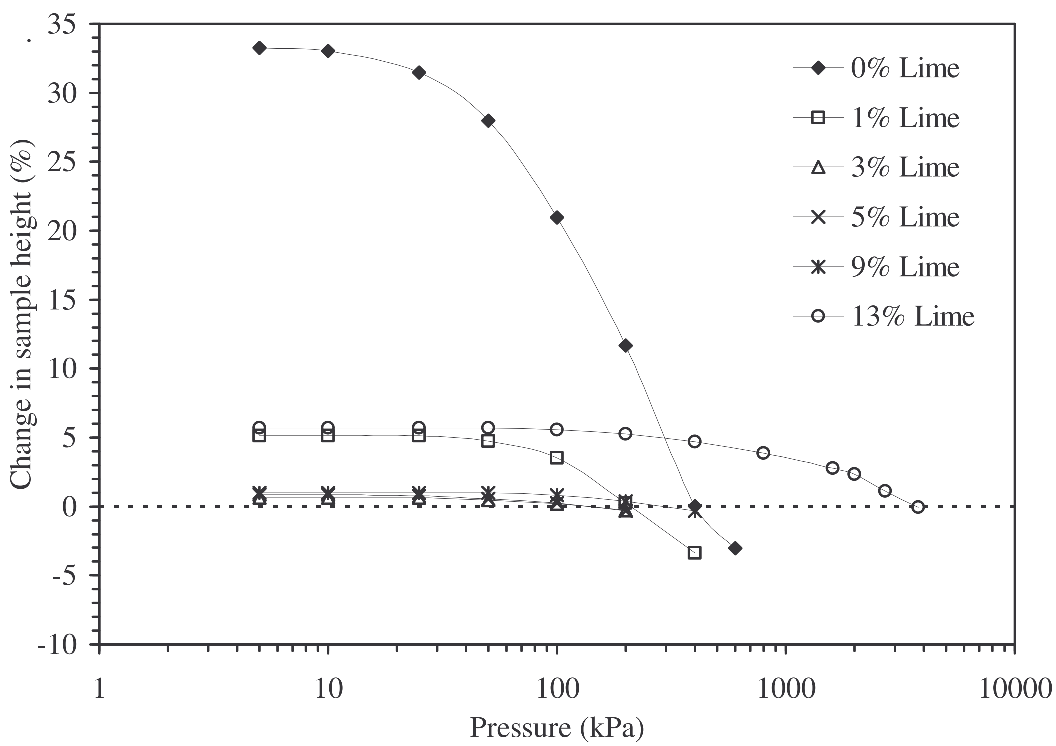

Fig. 6.40 Percent change in sample height versus loading applied pressure at different percent of lime for soil sample- 60%ES+40%RS

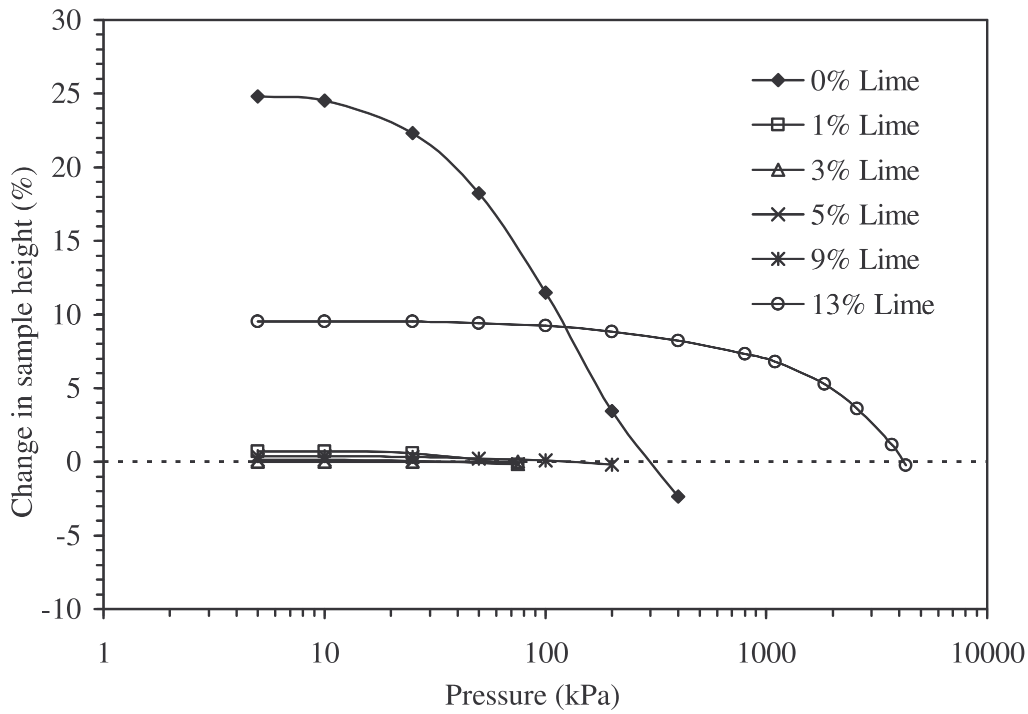

Fig. 6.41 Percent change in sample height versus loading applied pressure at different percent for soil sample- 40%ES+60%RS

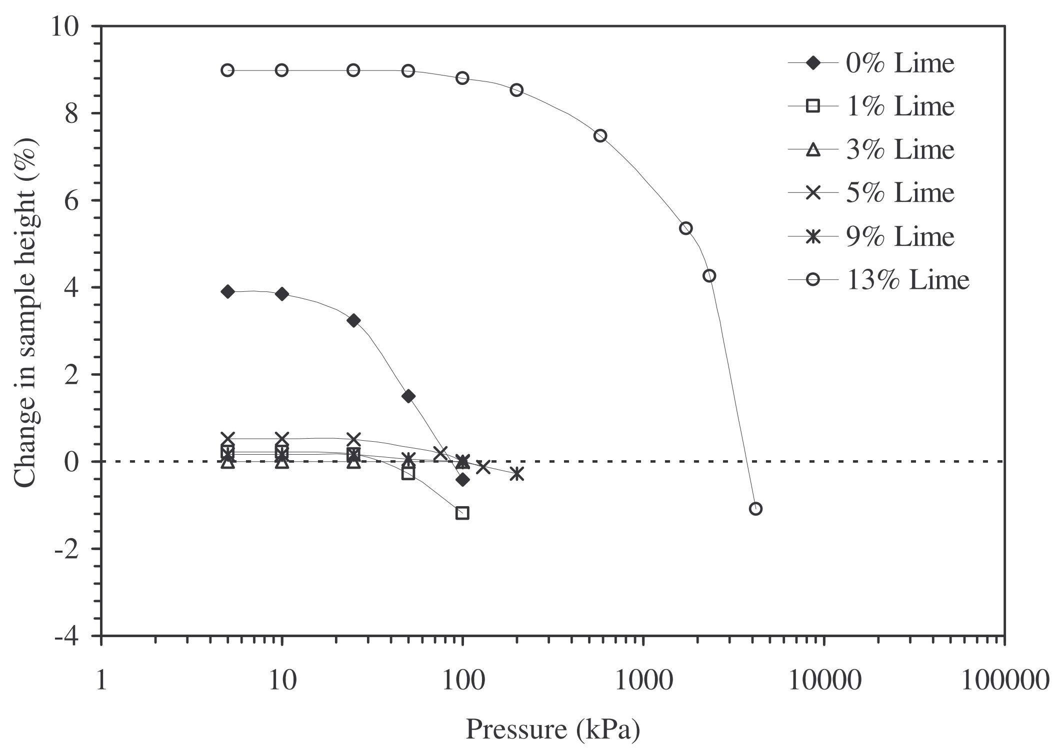

Fig. 6.42 Percent change in sample height versus loading applied pressure at different percent of lime for soil sample- 20%ES+80%RS

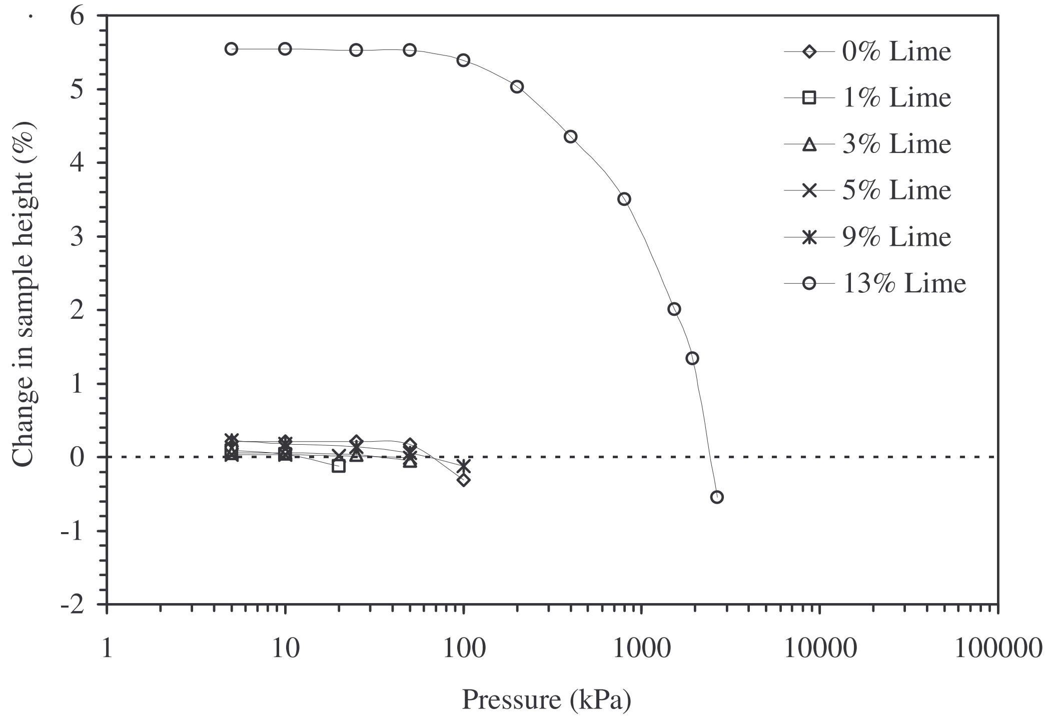

Fig. 6.43 Percent change in sample height versus loading applied pressure at different percent of lime for soil sample-100%RS

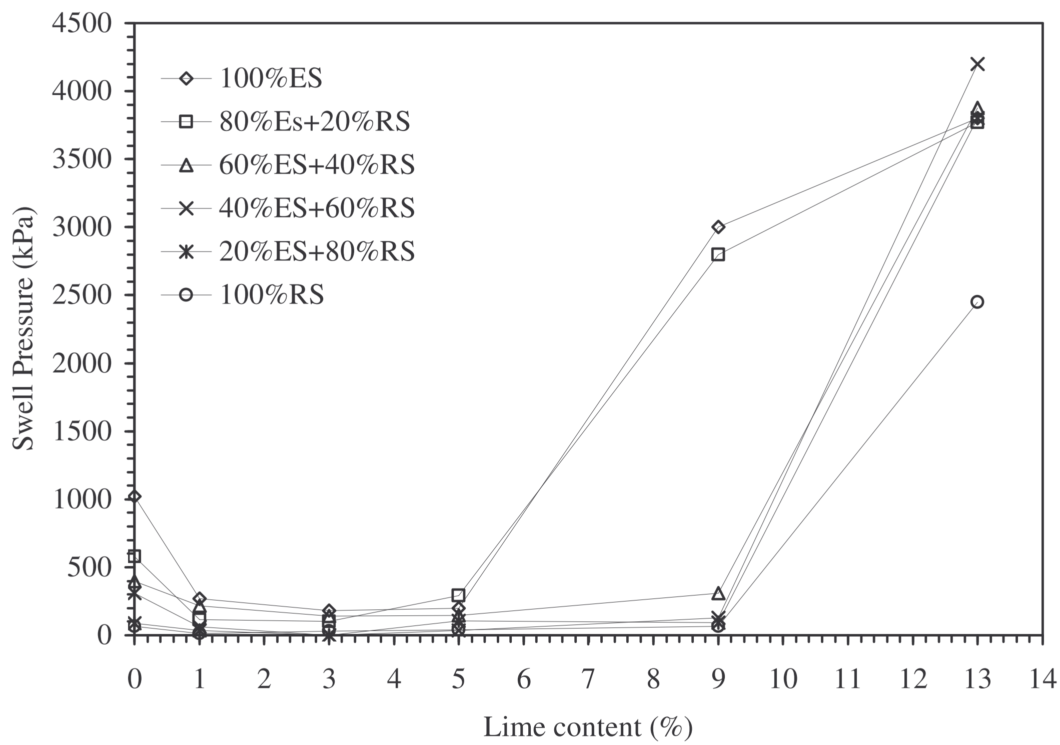

Fig. 6.44 Variation of swell pressure with percentage of lime content for treated and untreated soil.

6.3 SHRINKAGE BEHAVIOUR

The shrinkage response of expansive soil-residual soil mixes and the lime treated soils has been quantified, in the present study, through two different parameters i.e. shrinkage limit and volumetric shrinkage. The influence of various parameters such as soil type, lime content, curing period, fabric etc. on the shrinkage behaviour of soils has been brought out in the following sections.

6.3.1 Shrinkage Behaviour of ES-RS Mixes

6.3.1.1 Shrinkage Limit

In the process of drying there comes a stage when the soil particles come in contact with each other thereby further reduction in water content does not change the volume of the soil mass that it behaves as a solid. The water content at this stage is referred to as shrinkage limit which quantitatively deciphers the potential of the soil to shrink. Fig. 6.45 presents the variation of shrinkage limit and liquid limit with residual soil content in the soil matrix. It could be observed that while the liquid limit has gradually decreased with increase in the residual soil content the shrinkage limit variation has undergone two different phases. Initially it has increased at relatively slower rate till residual soil content of about 60%, beyond which it has undergone a sharp increase with further increase in residual soil content. This indicates that it is not plasticity of soil alone that dictates its shrinkage characteristics, particle size distribution too plays a major role in the shrinkage response of soils. Sridharan and Rao (1971) have reported that the interparticle shearing resistance influences substantially the shrinkage limit of the soils. For a fine grained expansive soil this resistance is primarily through the viscous resistance of the pore water while incase of the coarse grained non-expansive residual soil it is the intergranular frictional resistance. For relatively less percentage of residual soil the coarse grains remain suspended in the fine soil mass. However, beyond certain percentage the residual soil turns to be the major constituent that the coarse grains come in contact with each other leading to substantial increase in interparticle resistance and hence the shrinkage limit.

The results can further be analysed with respect to the fabric of the soil structure, which basically is the arrangement pattern of the soil particles. Primarily there happens three different fabrics in soils, such as; dispersed, aggregated and flocculated (Mitchell and Soga, 2005).

In dispersed structure the clay particles without having any face to face association, mostly are in parallel orientation in the soil-water system. This phenomenon takes place in case of plastic clays where the interparticle repulsion is dominant. In such soils the shrinkage is more and therefore the shrinkage limit is less. However, due to large thickness of diffuse double layer that creates repulsion, the liquid limit of these soils is high.

In aggregated fabric the clay particles are in face to face association that they form clusters. These clusters give rise to apparently higher particle size leading to increased void space and hence increased shrinkage limit. Such a phenomenon takes place when the interparticle repulsion is relatively less. Therefore aggregation can take place when the diffused double layer thickness is relatively less, which is the case for relatively low plastic soils. Hence the aggregated fabric can be indicated by low liquid limit but high shrinkage limit.

In flocculated structures the soils are in the edge to edge association forming a card house like structure giving rise to large inter granular voids and hence high shrinkage limit. Such edge to face arrangement takes place with fine grained soils in acidic environment.

The flocculated structure is therefore characterized by high shrinkage limit and high liquid limit (Sivapullaiah et al., 2000).

The variation of shrinkage limit with liquid limit for the expansive soil-residual soil system indicates that initially with 100%ES the soil is in dispersive state, characterized by high liquid limit and low shrinkage limit (Fig. 6.46). With increased residual soil content the fabric of the soil system has changed towards aggregation indicated through low liquid limit and high shrinkage limit. This indicates that soils having predominately fine clay particles have a dispersed fabric and the one dominated by coarse particles has an aggregated fabric.

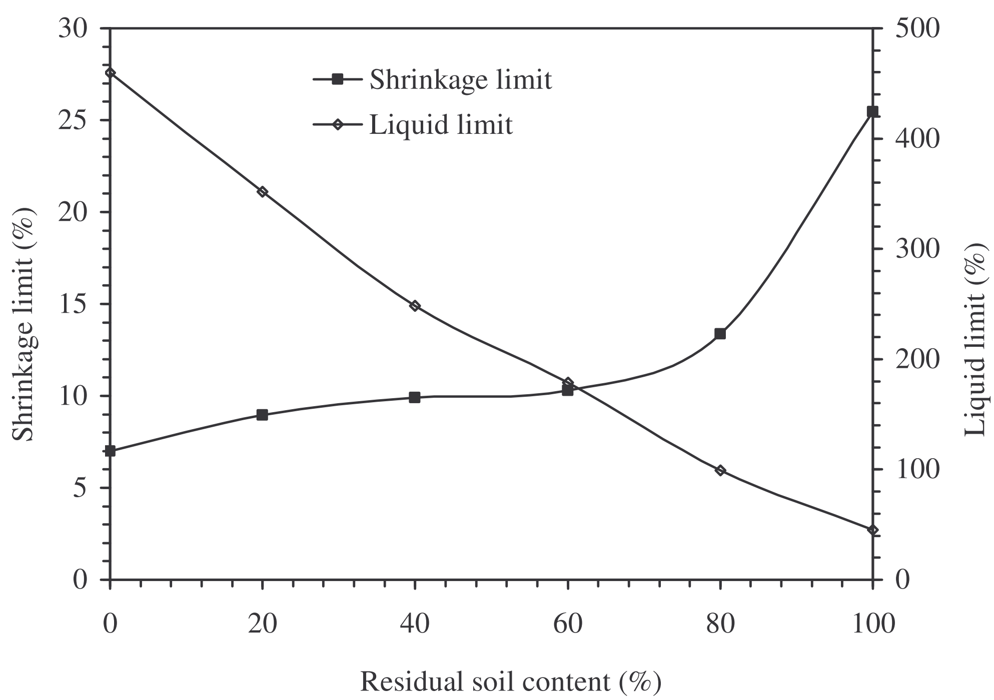

Fig. 6.45 Variation of liquid limit and shrinkage limit with residual soil content in expansive soil-residual soil system.

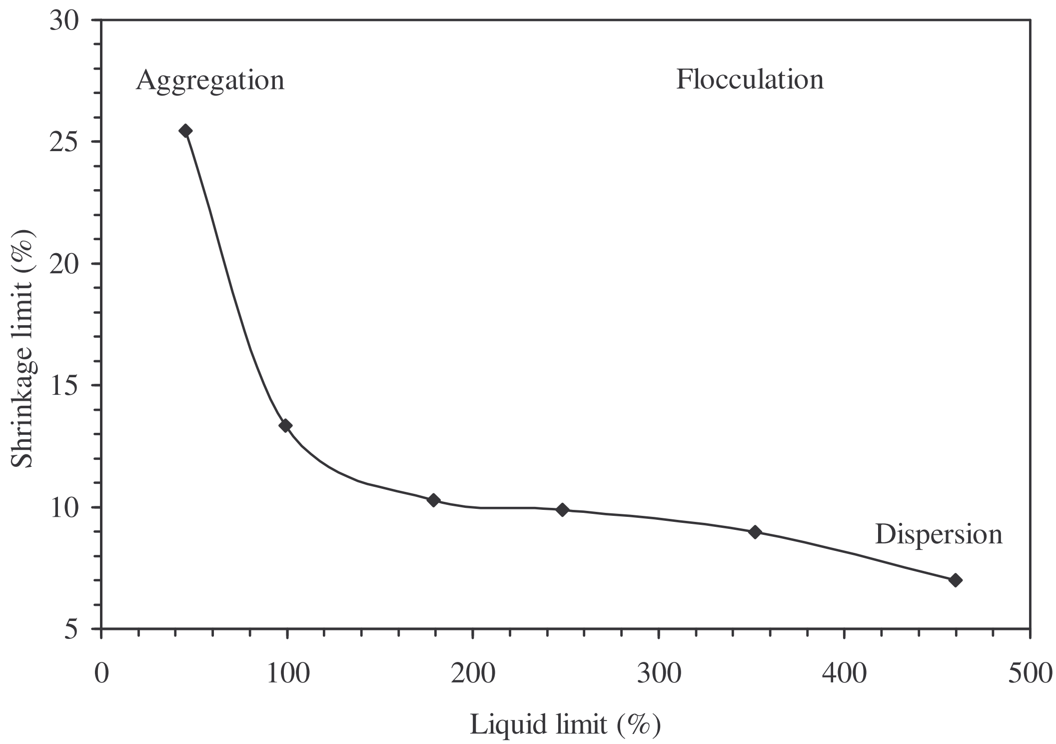

Fig. 6.46 Liquid limit vs. shrinkage limit response depicting the fabric in expansive soil-residual soil systems.

6.3.1.2 Volumetric Shrinkage

The volumetric shrinkage response resulting from progressive drying, as it happens in the field, were studied for the present soils (ES, ES+RS, RS). The soil specimens compacted at their optimum moisture content and maximum dry density condition were allowed to swell under a surcharge of 5 kPa. After the swelling nearly ceased, the specimens were air dried at a nearly constant temperature of about $30C^0$. The water content and void ratio of the specimens were continuously monitored during shrinkage. The process was terminated when weight of the specimen became nearly constant on drying indicating that shrinkage is almost over. More details of the test procedure has already been discussed in chapter 3. Figure 6.47 shows the void ratio vs. water content responses for the soil specimens during shrinkage. It could be observed that the shrinkage responses are generally of S-shape, indicating that there are three different stages of shrinkage. This can more clearly be observed from the typical response shown in Fig. 6.48. The schematic diagram showing these three phases i.e. initial shrinkage, primary shrinkage and residual shrinkage; are depicted in Fig. 6.49. However, as the non-expansive residual soil percentage in the soil specimen increases the shrinkage response grows flatter indicating that the rate of shrinkage has reduced with reduced fines content. It is the capillary pressure generated in the soil pores that brings the particles closer during shrinkage. With increased grain size the voids grow in size leading to reduced capillary pressure and hence reduced shrinkage. Besides the inter particle resistance acting against the capillary suction increases with increased coarse fractions in the soil mass. These two factors are believed to have retarded the rate of shrinkage. The equilibrium shrinkage void ratio has shown an increasing trend with increased residual soil content, indicating that the maximum shrinkage reduces with increased coarse fraction.

The shrinkage in the initial phase and residual phase are much lesser compared to the primary stage. In fact most of the shrinkage takes place in the primary phase. This is because the initial shrinkage is due to loss of surface water which is insignificant. Only when the capillary suction starts building up in the voids (Fredlund and Rahjardo, 1993) the soil grain start getting closer to each other reducing the void space substantially, giving rise to increased rate of shrinkage. However a stage reaches when the soil particles come closest that the interparticle resistance is high enough to stand against the capillary suction leading to reduced shrinkage with further reduction in water content. During this, the shrinkage is due to some fabric re-arrangement or bending of particles (Yong and Warkentin, 1975). This is called residual shrinkage.

It could be observed that while for expansive soil (100%ES) the transition from primary to residual shrinkage occurs over a wide range of water content (i.e. 84-16%), in case of residual soil it takes place in a narrow range (i.e. 14-7%). From the above observation it can be concluded that in soils having extremely high fines fraction the plasticity governs shrinkage while for moderate and low fines content the grain size distribution governs the shrinkage characteristics of the soil.

The magnitudes of initial, primary and residual shrinkage for different soil samples are calculated as follows.

Initial Shrinkage (S_I) = $$\frac{(e_s - e_{is})}{1+e_s} \times 100 \qquad 6.6$$

Primary Shrinkage (S_P) = $$\frac{(e_{is} - e_{ps})}{1+e_{is}} \times 100 \qquad 6.7$$

Residual Shrinkage (S_R) = $$\frac{(e_{ps} - e_{rs})}{1+e_{ps}} \times 100 \qquad 6.8$$

where, e_s is the void ratio at maximum swell. e_{is}, e_{ps} and e_{rs} are the void ratios at the end of initial, primary and residual shrinkage of the specimens as depicted in Fig. 6.49.

The magnitudes of initial, primary and residual shrinkage of the soils are presented in Table 6.3. The initial shrinkage ranges from 0.18 to 0.97% while, the primary shrinkage ranges from 3.29 to 54.88% and the residual shrinkage ranges from 0.06 to 1.42%. It could be observed that in general shrinkage is more for soil with higher fines content and plasticity. This is in agreement with the findings of Mishra et al. (2008). As suggested by Yong and Warkentin (1975) soil with higher swollen water content would shrink more which indeed has happened in the present case. It is of interest to note that the variation of primary shrinkage with plasticity parameter (i.e. LL, PI) is substantially high while that of initial shrinkage and residual shrinkage is marginal.

The void ratio (e) versus water content (w) relationships for the initial, primary and residual shrinkage of the different soils are presented in Table 6.3. The linear fits with correlation coeffient (R^2) above 0.9 indicates that irrespective of soil type, shrinkage in all phases (i.e. initial, primary, residual) follows linear trend of variation with water content.

For all these soils having wide range of plasticity (very low to very high) the slopes of the primary shrinkage responses (0.005–0.027) are higher than that of the initial shrinkage (0.0004–0.003) and residual shrinkage (0.0003–0.004) responses. This indicates that with increased coarse fraction the rate of shrinkage is relatively faster. The e-w relations will be of use in predicting the shrinkage behaviour of the soils (Tripathy et al., 2002).

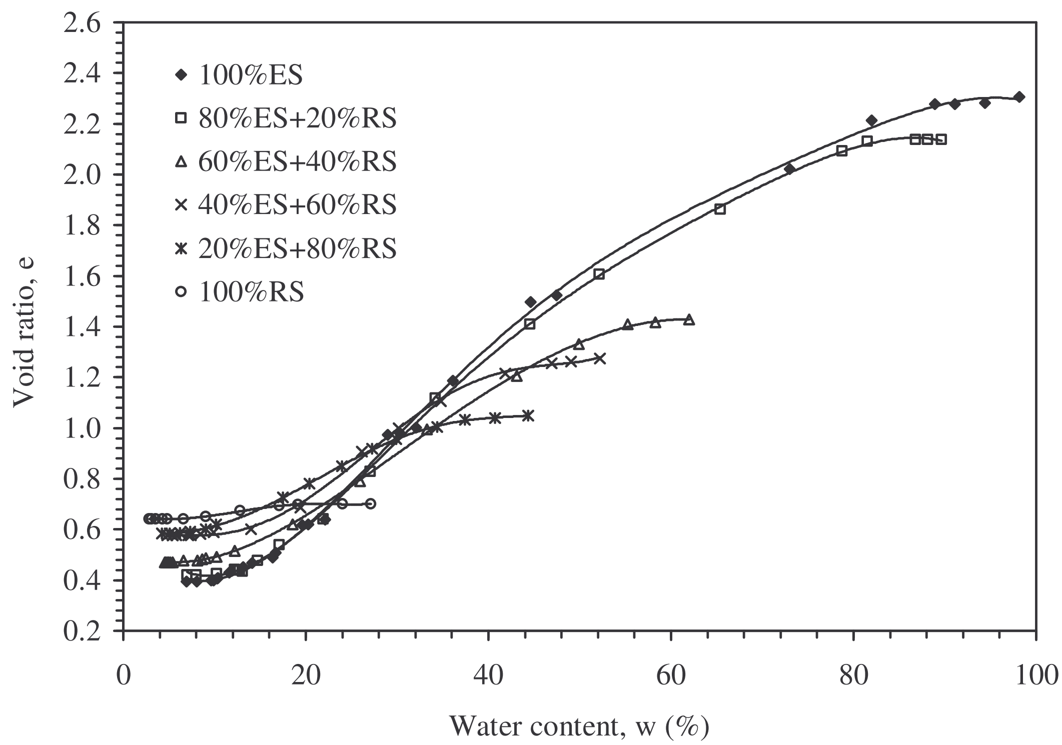

Fig. 6.47 Water content vs. void ratio responses for different soils.

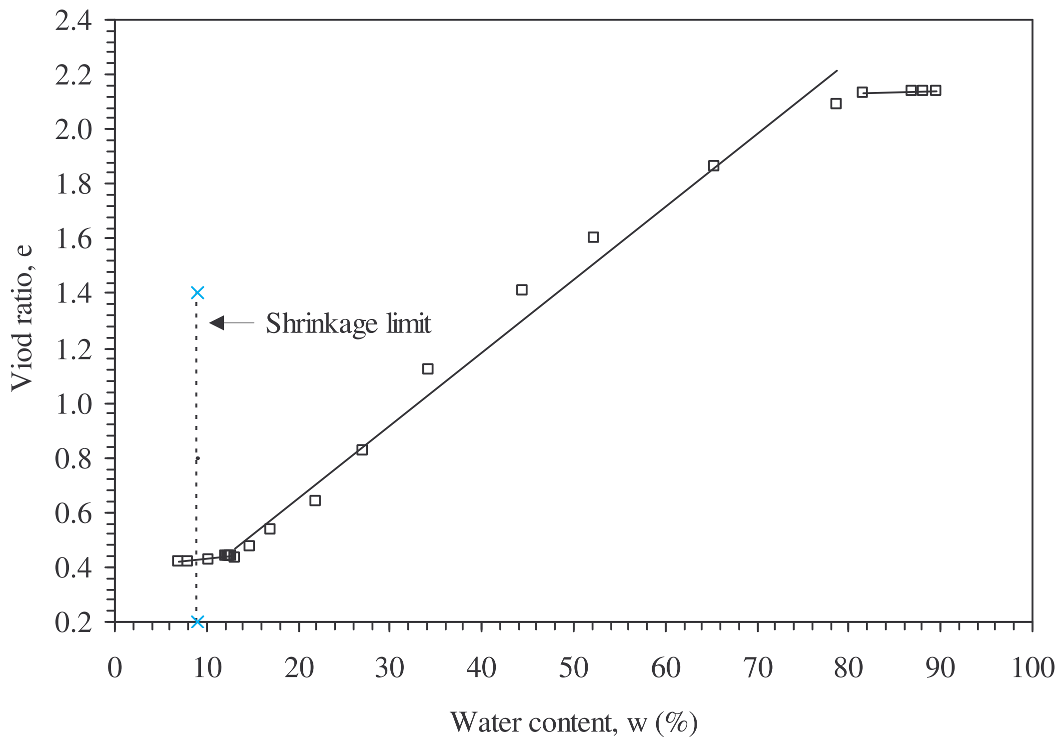

Fig. 6.48 Water content vs. void ratio for the soil (80%ES+20%RS).

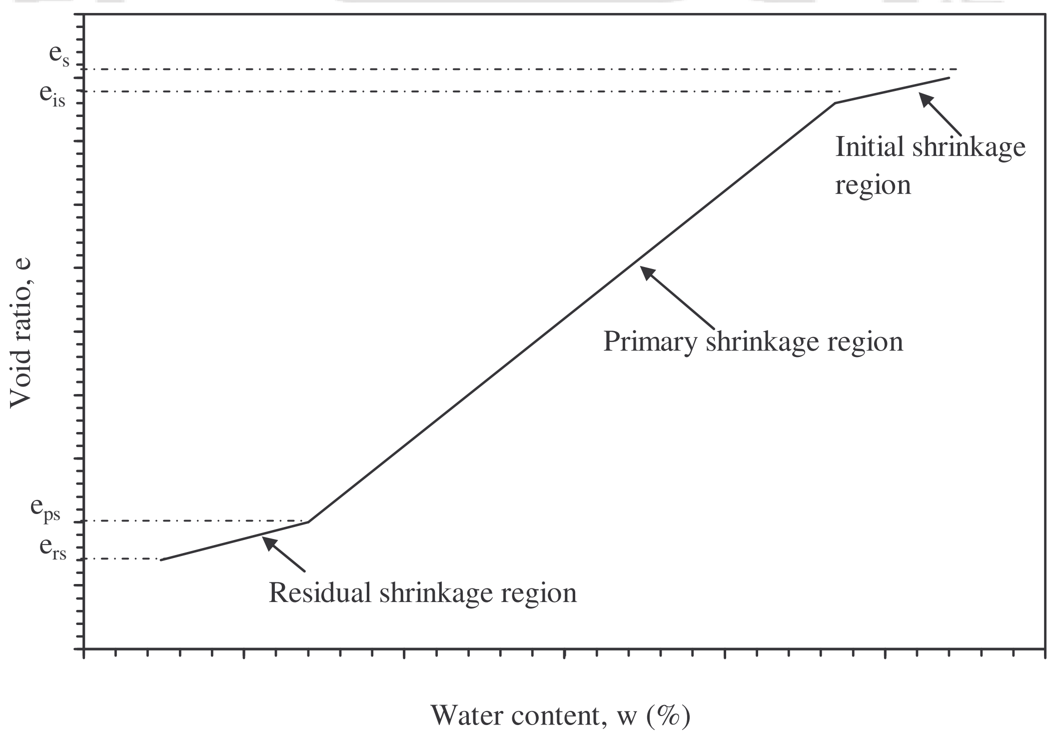

Fig. 6.49 Schematic diagram showing the three phases of shrinkage i.e. initial, primary and residual shrinkage

Table 6.3 Shrinkage magnitudes and e-w relations for soils.

Soil	Shrinkage magnitude			e-w relations		
	S_I (%)	S_P (%)	S_R (%)	Initial shrinkage	Primary shrinkage	Residual shrinkage
100%ES (LL = 459.94%, PI = 406.24)	0.61	54.88	1.42	e = 0.002w +2.12 R^2 = 0.9	e = 0.027w +0.12 R^2 = 0.98	e = 0.001w +0.39 R^2 = 0.62
80%ES+20%RS (LL = 351.9%, PI = 302.53%)	0.32	53.99	1.39	e = 0.001w +2.04 R^2 = 0.92	e = 0.027w +0.12 R^2 = 0.99	e = 0.004w +0.39 R^2 = 0.94
60%ES+40%RS (LL = 248.19%, PI = 210.13%)	0.82	38.17	1.34	e = 0.003w +1.25 R^2 = 1	e = 0.021w +0.26 R^2 = 0.99	e = 0.004w +0.45 R^2 = 0.93
40%ES+60%RS (LL = 178.84%, PI = 148.12%)	0.44	29.20	1.25	e = 0.003w +1.09 R^2 = 0.99	e = 0.021w +0.32 R^2 = 0.97	e = 0.003w +0.56 R^2 = 0.95
20%ES+80%RS (LL = 99.19%, PI = 70.51%)	0.97	21.67	0.63	e = 0.002w + 0.94 R^2 = 0.99	e = 0.016w +0.47 R^2 = 0.99	e = 0.003w + 0.57 R^2 = 0.92
100%RS (LL = 45.33%, PI = 19.33%)	0.18	3.29	0.06	e = 0.0004w + 0.69 R^2 = 0.99	e = 0.005w + 0.61 R^2 = 0.99	e = 0.0003w + 0.64 R^2 = 0.93

6.3.2 Shrinkage Behaviour of ES-RS-Lime Mixes

6.3.2.1 Shrinkage Limit

Variation of shrinkage limit with lime content at different periods for 100%ES, 80%ES+20%RS, 60%ES+40%RS, 40%ES+60%RS, 20%ES+80%RS and 100%RS are depicted in Fig. 6.50, 6.51, 6.52, 6.53, 6.54 and 6.55 respectively. The corresponding variations of shrinkage limit with liquid limit showing the change in fabric of these soils due to lime treatment are presented in Fig. 6.56, 6.57, 6.58, 6.59, 6.60 and 6.61 respectively. For 100%ES and 80%ES+20%RS, from Fig. 6.50 and 6.51 it could be observed that, initially the shrinkage limit has increased sharply till about 5% lime content beyond which the rate of increase has reduced with the reduction being more at less curing period. The initial increase is attributed to aggregation of particles due to addition of lime as observed in Fig. 6.56 and Fig. 6.57. With increased electrolyte concentration, in the pore fluid, the diffuse double layer thickness reduces and thereby the repulsion between the clay particles reduces. As a result of which the particles come closer and form aggregated clusters. These aggregated clusters offer increased resistance against capillary suction induced volumetric shrinkage leading to increased shrinkage void ratio and thereby the water content (i.e. shrinkage limit). With further increase in lime content the pH of the pore fluid increases that it becomes alkaline. Under alkaline environment the cation exchange capacity considerably increases leading to flocculated fabric. Indeed Fig. 6.56 and Fig. 6.57 indicate increased flocculation with increased curing period. The flocculated fabric due to edge-to-face card house structure posses high internal voids. Besides it manifests increased resistance against shrinkage. Therefore, with increased curing period as the flocculation increases, the shrinkage limit shows an increasing trend.

For soils with increased residual soil content (Fig. 6.52, 6.53, 6.54) the shrinkage limit shows similar trend that it increases with increase in lime content. Correspondingly Fig. 6.58, 6.59, 6.60 indicates that the fabric has visibly changed from dispersed to aggregation. However further addition of lime with low curing period does not bring much change in the shrinkage limit. For such cases it is observed that the fabric of the soils have mostly remained in the aggregation region. This is attributed to the reduced content of fines in the soil specimen, and reduced curing period that flocculation has not been formed substantially. However, with higher curing period (i.e. 21 days and 28 days) flocculation has increased giving rise to increased shrinkage limit.

For the residual soil (100%RS) there is an initial reduction in shrinkage limit with addition of 1% lime. The soil in the present case being predominantly non-swelling kaolinitic and there being less fines content, the influence of diffuse double layer depression with the small quantity of lime is marginal. Rather, the lime induced lubrication has facilitated the intergranular rearrangement during shrinkage leading to better packing and hence reduced shrinkage limit. With increased lime content the aggregation and flocculation takes place leading to increased shrinkage limit. A relatively higher rate of increase in shrinkage limit at higher lime content and increased curing period is attributed to the formation of cementitious gels that holds more water onto it leading to increased shrinkage limit.

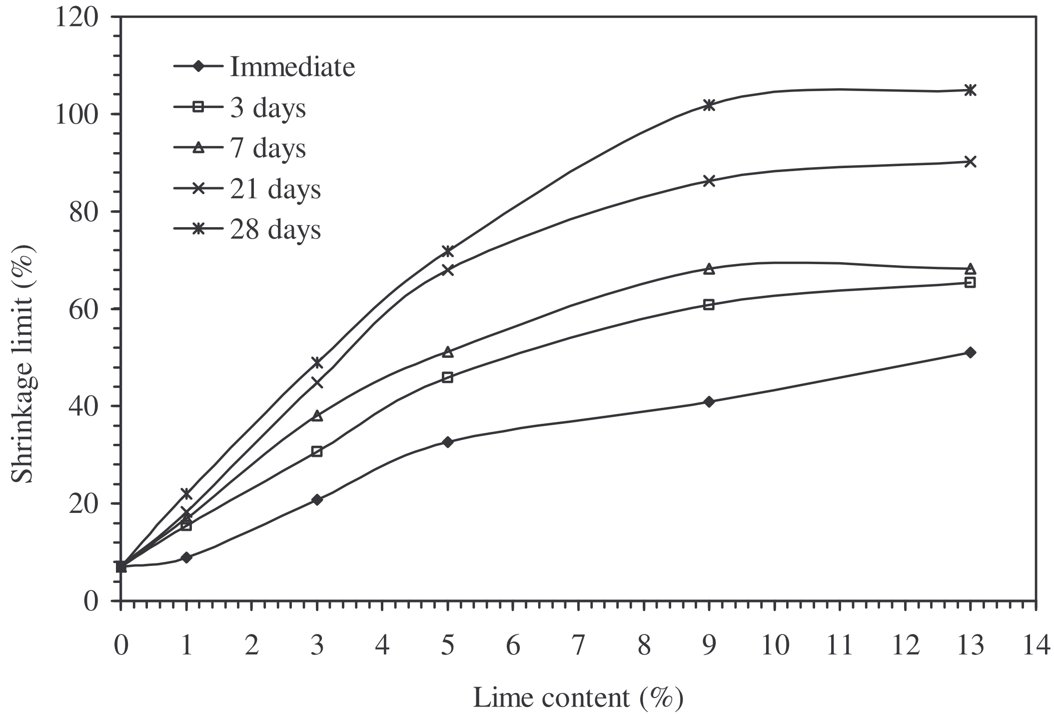

Fig. 6.50 Variation of shrinkage limit with lime content at different curing periods for (100%ES)

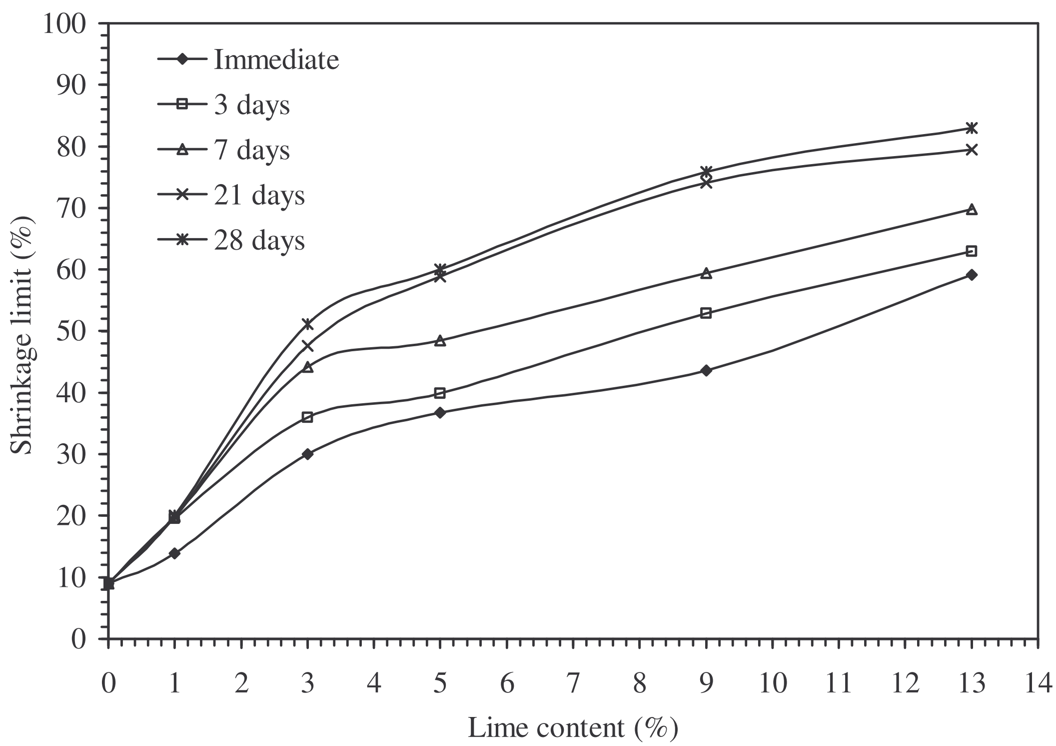

Fig. 6.51 Variation of shrinkage limit with lime content at different curing periods for (80%ES+20%RS)

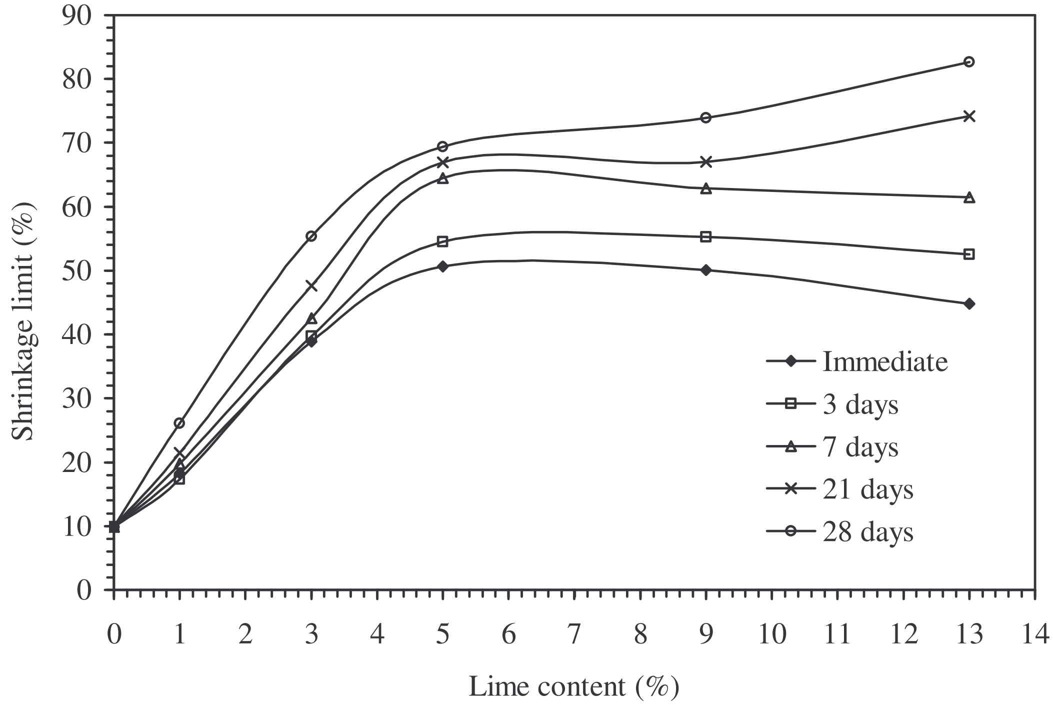

Fig. 6.52 Variation of shrinkage limit with lime content at different curing periods for (60%ES+40%RS)

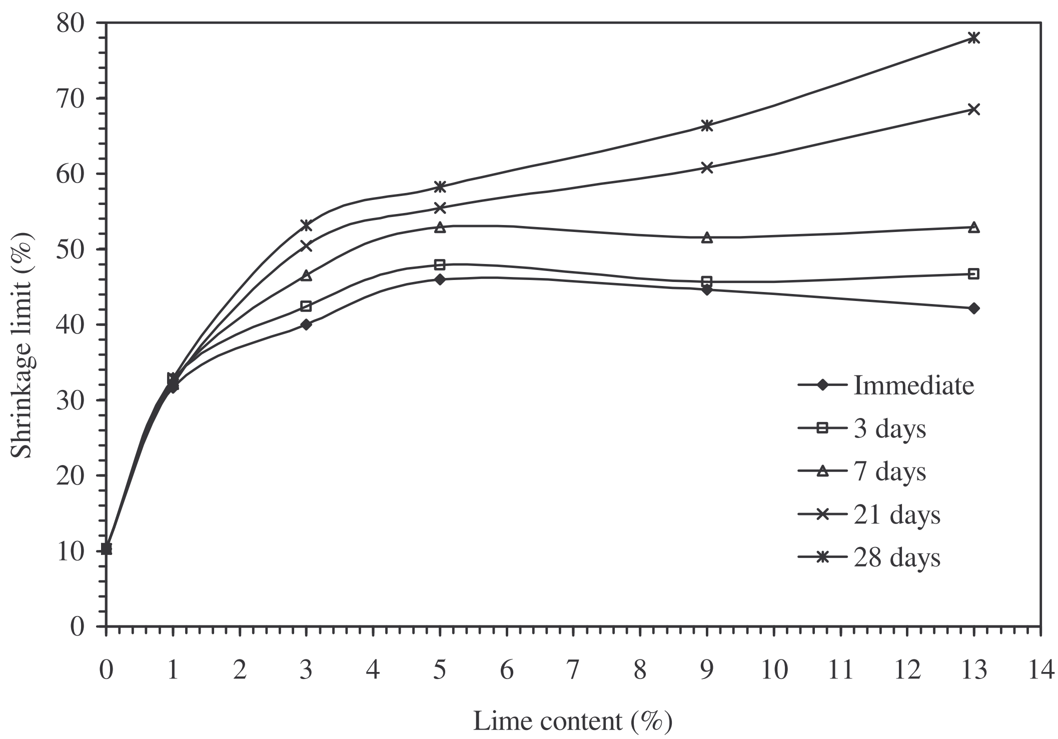

Fig. 6.53 Variation of shrinkage limit with lime content at different curing periods for (40%ES+60%RS)

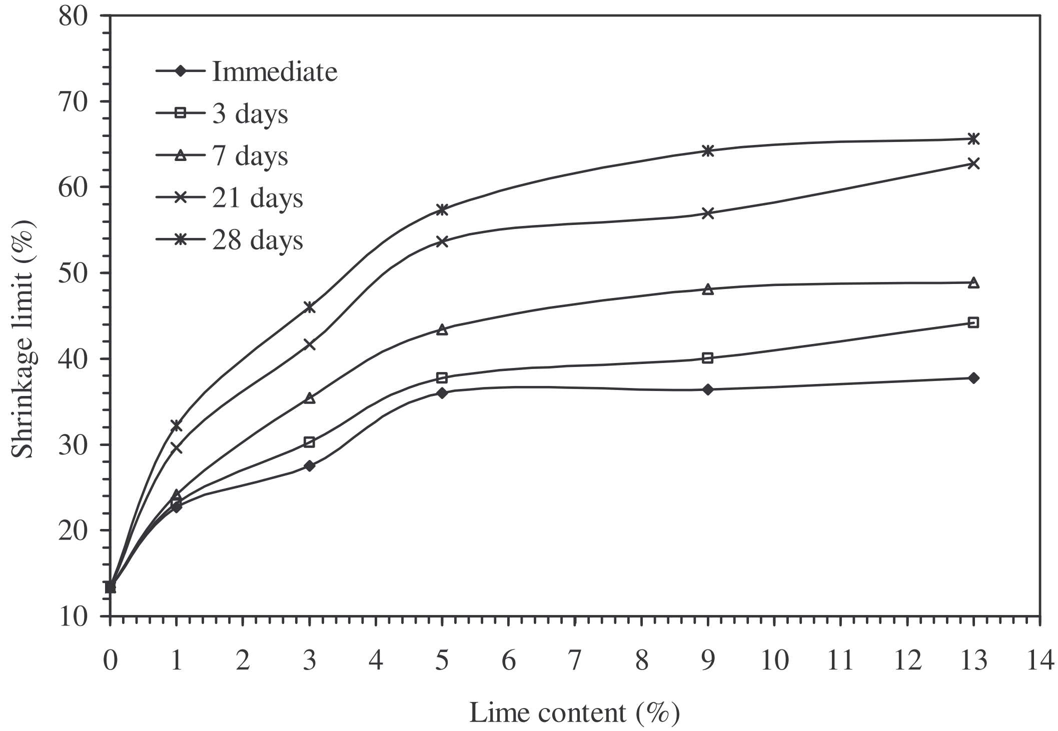

Fig. 6.54 Variation of shrinkage limit with lime content at different curing periods for (20%ES+80%RS)

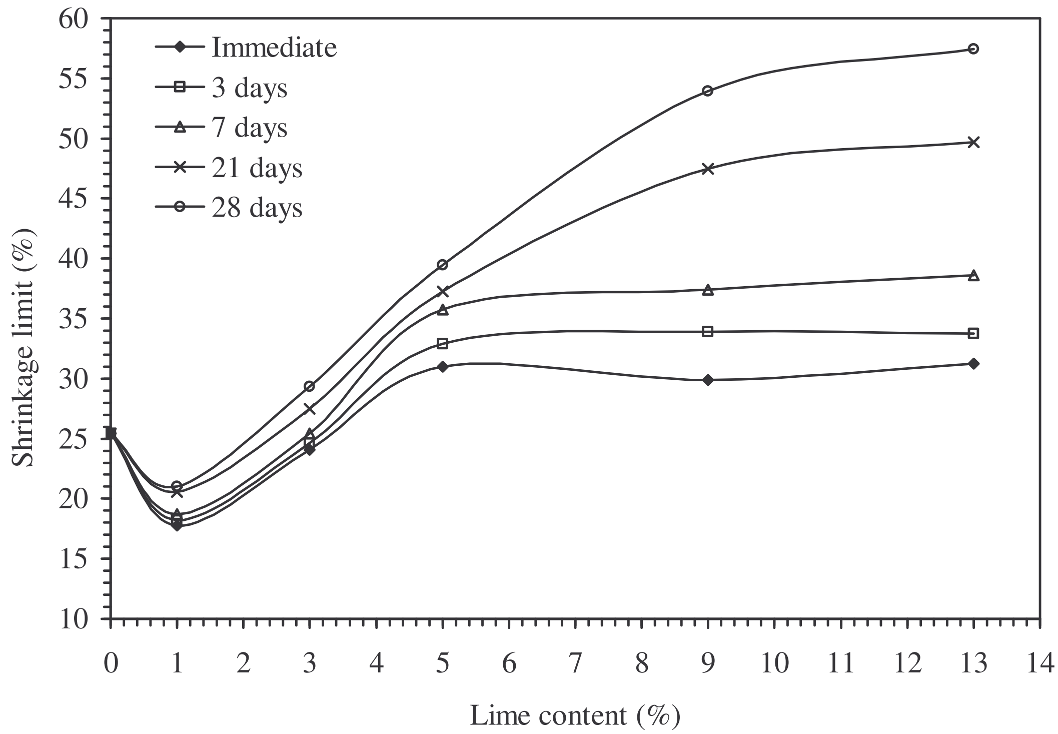

Fig. 6.55 Variation of shrinkage limit with lime content at different curing periods for (100%RS)

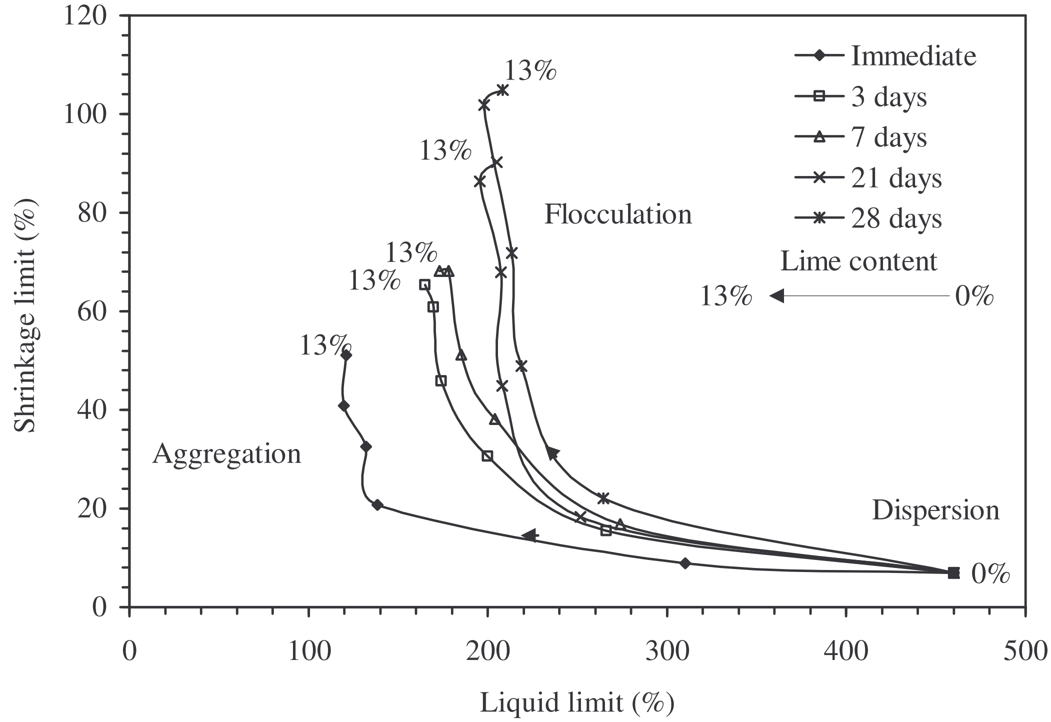

Fig. 6.56 Liquid limit vs. shrinkage limit responses depicting fabric change in 100%ES due to lime treatment.

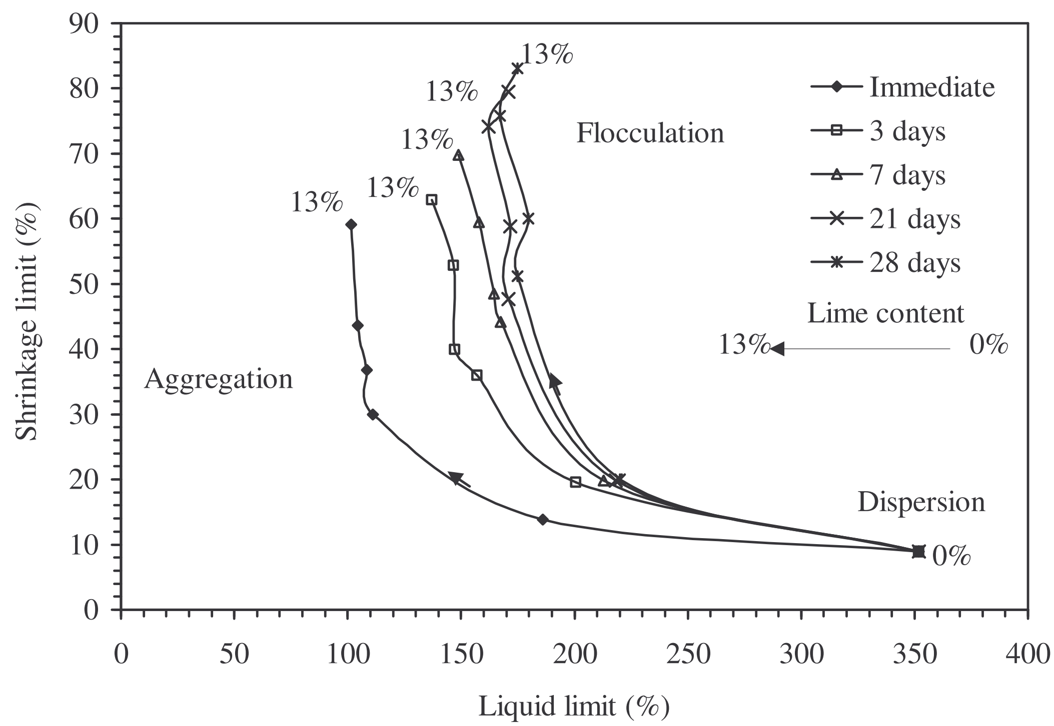

Fig. 6.57 Liquid limit vs. shrinkage limit responses depicting fabric change in 80%ES+20%RS due to lime treatment.

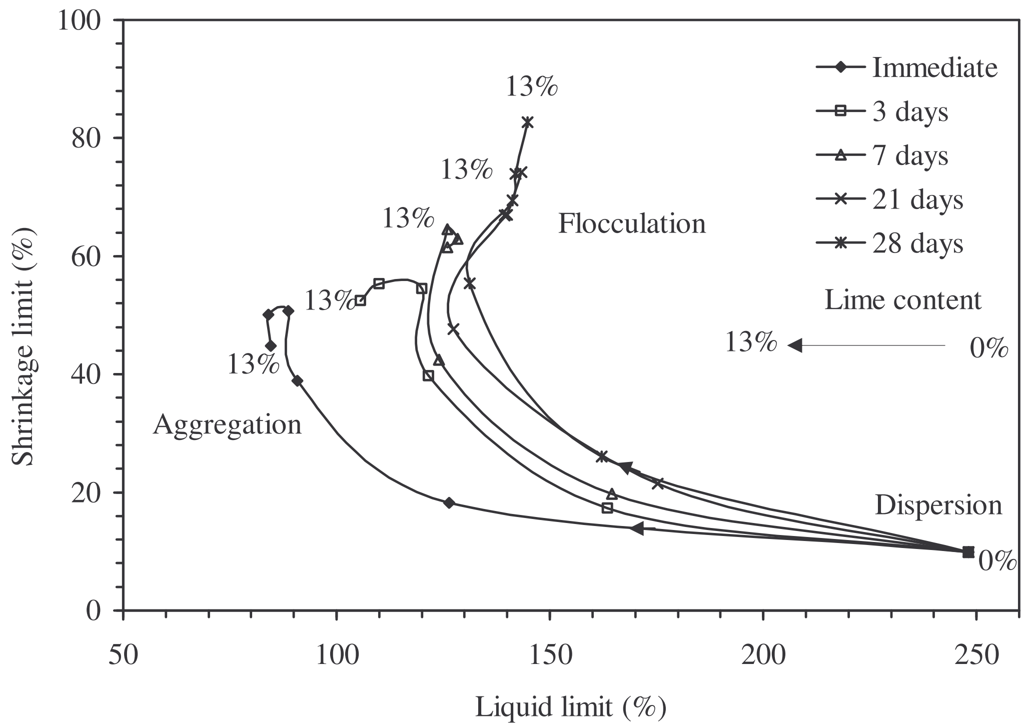

Fig. 6.58 Liquid limit vs. shrinkage limit responses depicting fabric change in 60%ES+40%RS due to lime treatment.

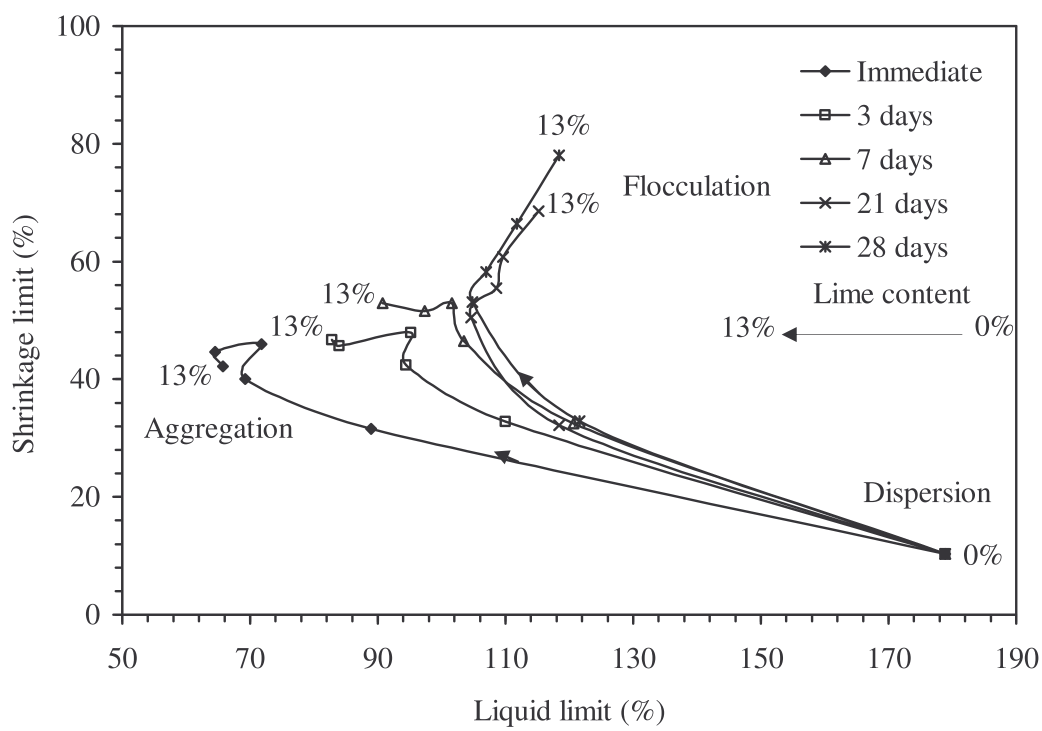

Fig. 6.59 Liquid limit vs. shrinkage limit responses depicting fabric change in 40%ES+60%RS due to lime treatment.

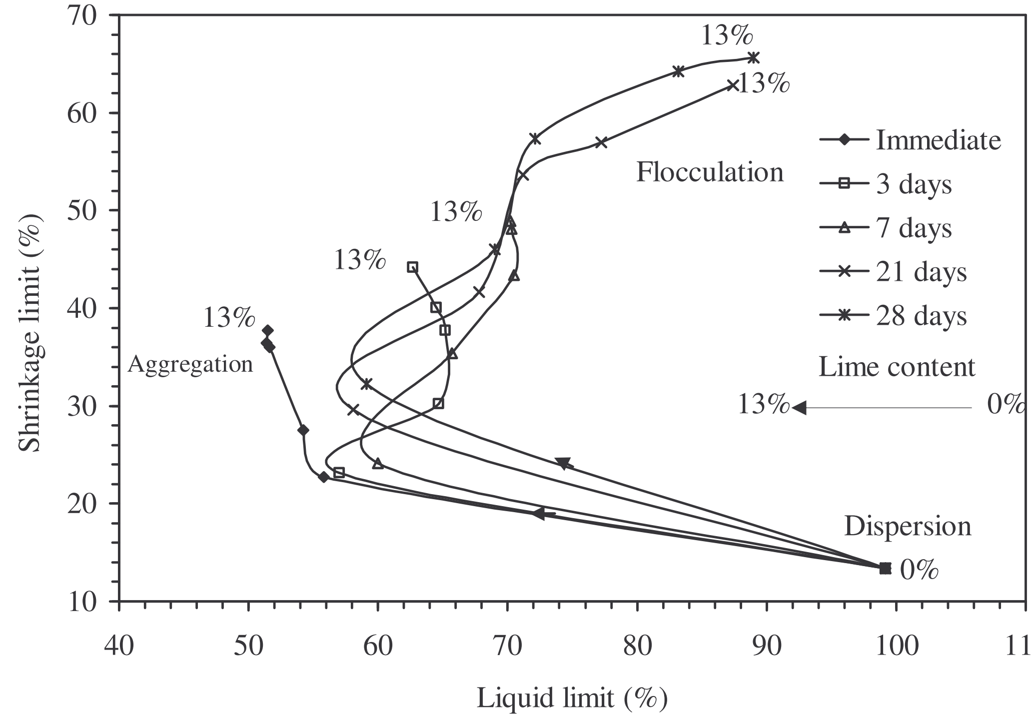

Fig. 6.60 Liquid limit vs. shrinkage limit responses depicting fabric change in 20%ES+80%RS due to lime treatment.

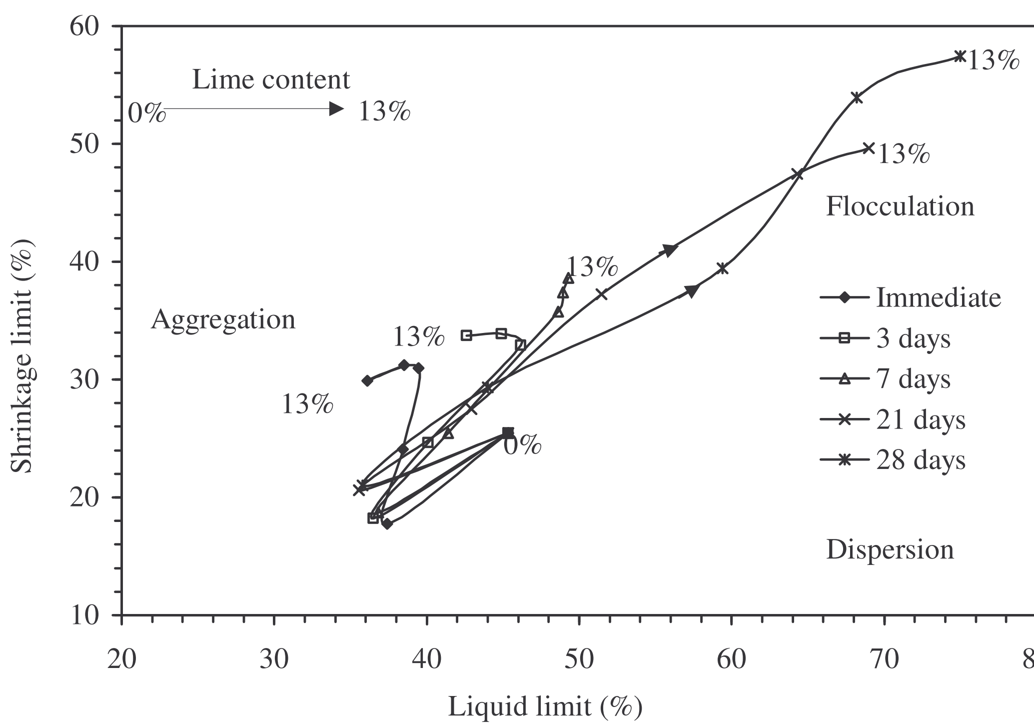

Fig. 6.61 Liquid limit vs. shrinkage limit response depicting fabric change in residual soil (100%RS) due to lime treatment.

Fig. 6.62 shows that irrespective of soil type the shrinkage has increased with lime content. Apart from fabric change the cementing action of lime induces increased shear resistance leading to reduced shrinkage giving rise to increased shrinkage void ratio and hence increased shrinkage limit. From the data presented in Fig. 6.63 it could be observed that the shrinkage limit reduces with increased residual soil content. With increased residual soil content the clay content has reduced and thereby the flocculation induced shrinkage reduction has reduced. Hence it can be said that lime is more effective for clayey soils in reducing the shrinkage potential.

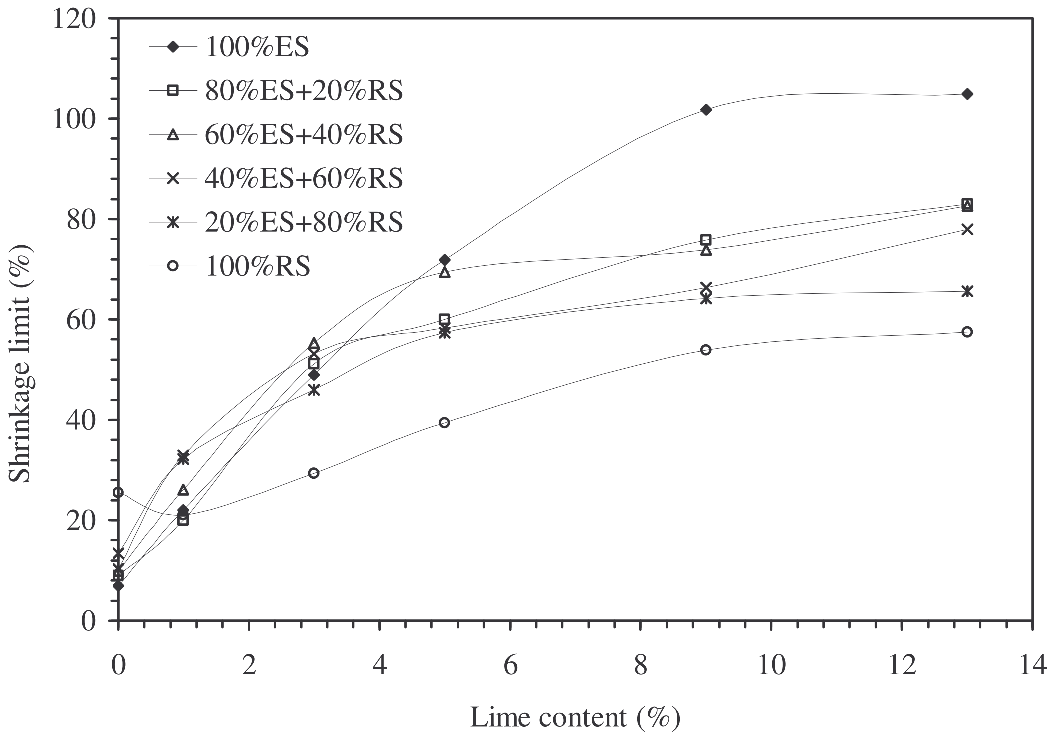

Fig. 6.62 Variation of shrinkage limit with lime contents for maximum curing period 28 days.

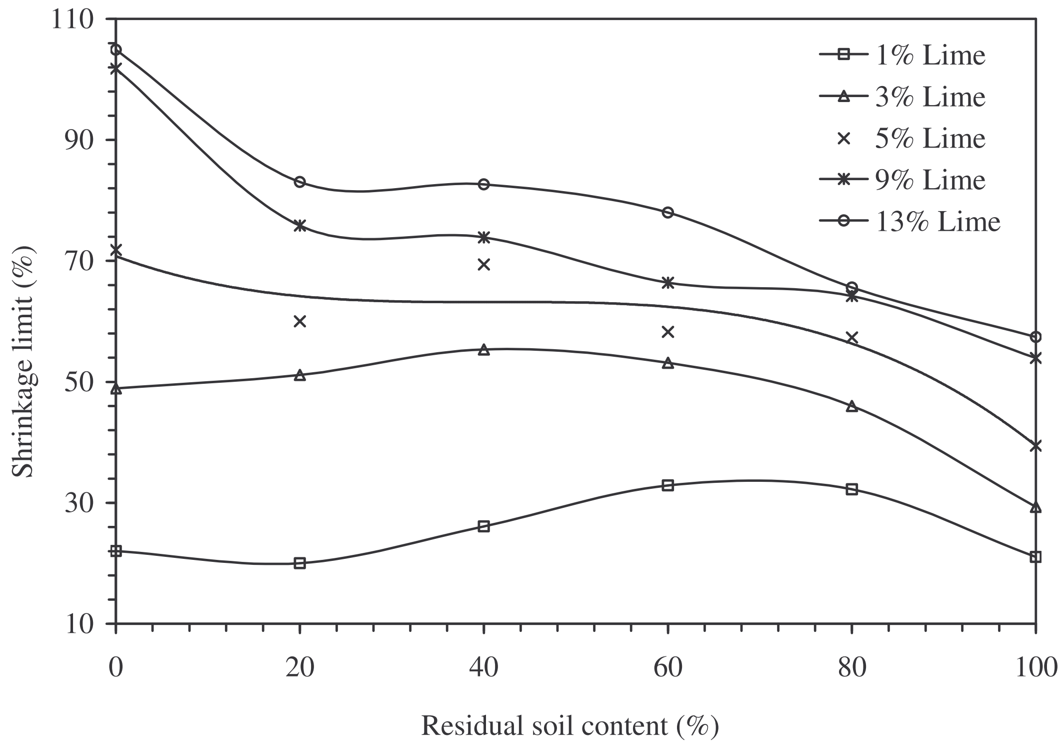

Fig. 6.63 Variation of shrinkage limit with residual soil contents for different lime contents at maximum curing period 28 days.

6.3.2.2 Volumetric Shrinkage

The shrinkage response (e-w) of a typical lime treated soil is shown in Fig. 6.64. It could be observed that similar to the untreated soil the lime treated soil too shows a three stage shrinkage (i.e. initial, primary and residual). Among the three, maximum shrinkage has taken place under primary shrinkage, indicating that the general mechanism of shrinkage too prevails in case of lime treated soils. Fig. 6.65-6.70 depicted the shrinkage responses of different soils with varied percentage of lime and curing period. The corresponding magnitudes of the three shrinkages (i.e. S_I, S_P, S_R) and e-w relations are presented in Tables 6.4-6.9. It could be observed that in general shrinkage magnitude has reduced with increase in lime content. This is attributed to the combined effect of aggregation, flocculation and cementation that has taken place due to lime addition. Fig. 6.71 depicting variation of primary shrinkage with lime content shows that shrinkage reduces substantially with increase in lime content till about 5% beyond which it nearly remains constant with further increase in lime content. This can be taken as the optimum lime content giving maximum percentage reduction in the shrinkage.

The variation in void ratio with reduction in water content, in general, shows a linear relationship. However for residual soil with more than 1% lime content shows nearly a straight line indicating that shrinkage is practically negligible. In the absence of swelling clay the non-swelling kaolinitic soil particles get effectively locked due to cementation brought about by the lime. Indeed Fig. 6.72 shows that shrinkage has reduced with increased residual soil content. The reduced slope of the shrinkage response with increased lime content (depicted in e-w relations) indicates that the rate of shrinkage has reduced with increased lime content. A reduced rate of shrinkage leads to reduced cracks in the soil. Indeed microcracks that developed in the untreated soil specimens were practically absent with lime treatment.

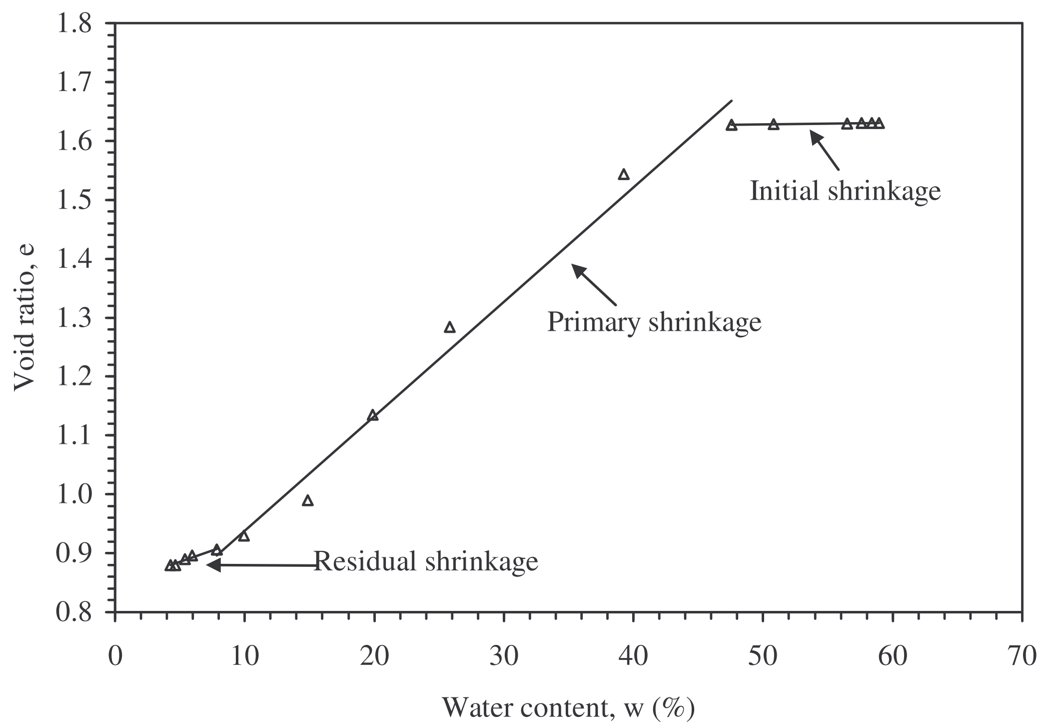

Fig. 6.64 Water content vs. void ratio responses of a typical lime treated soil (100%ES+3% Lime)

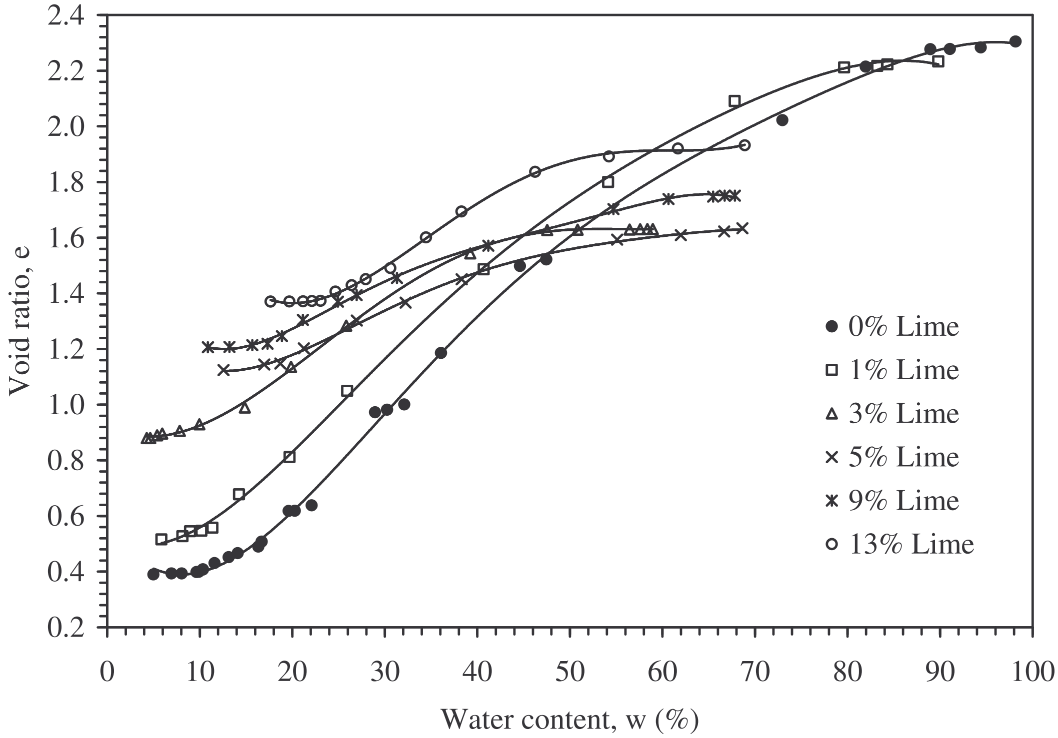

Fig. 6.65 Water content vs. void ratio responses of 100%ES with different percent of lime

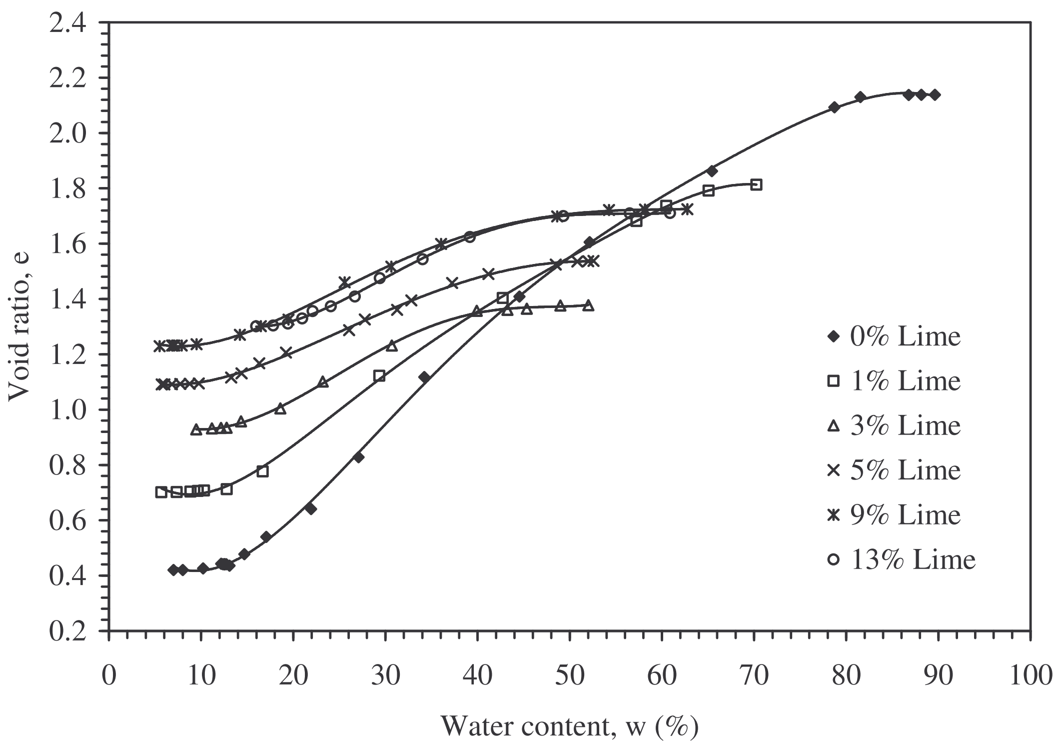

Fig. 6.66 Water content vs. void ratio responses of 80%ES+20%RS with different percent of lime.

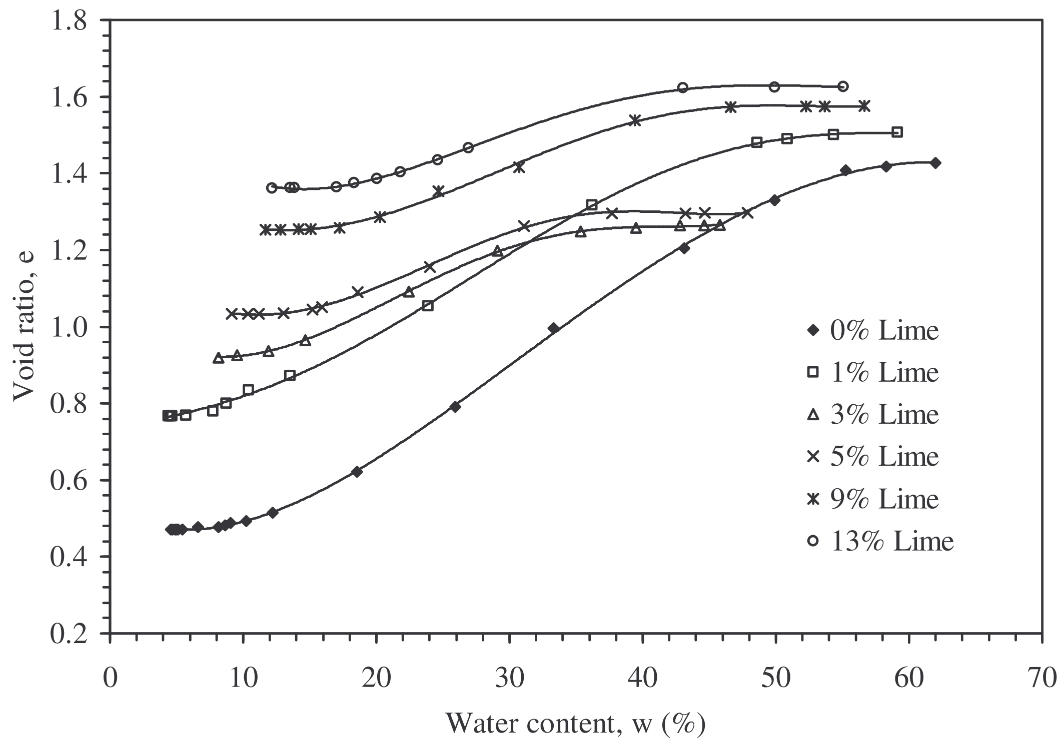

Fig. 6.67 Water content vs. void ratio responses of 60%ES+40%RS with different percent of lime.

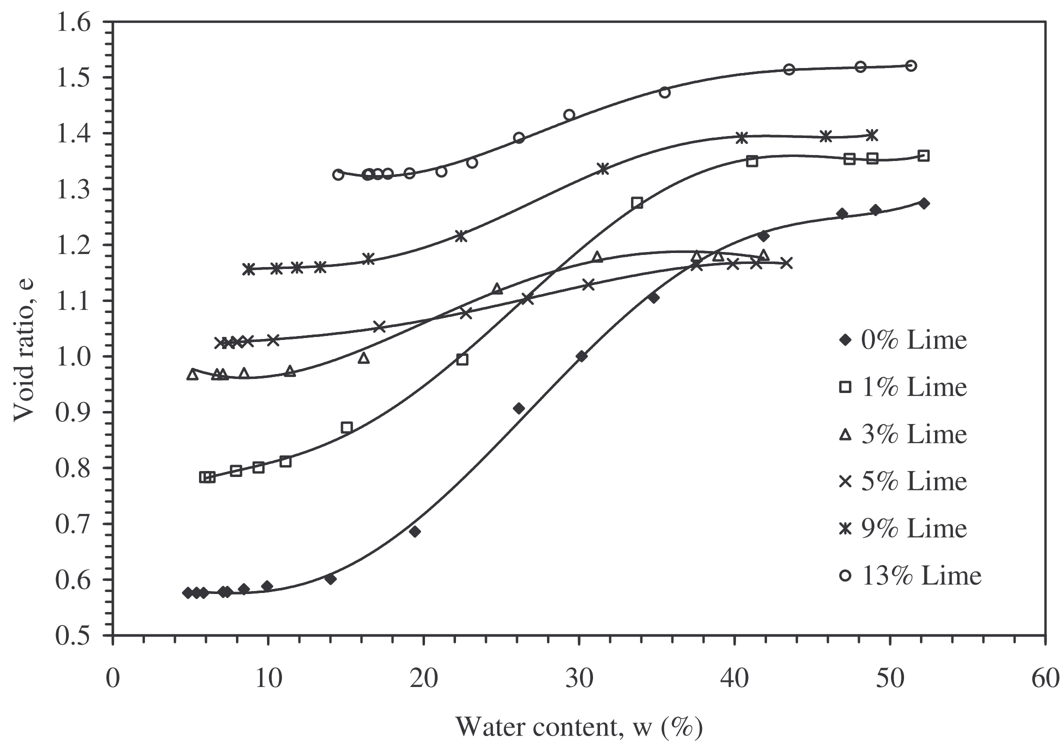

Fig. 6.68 Water content vs. void ratio responses for 40%ES+60%RS with different percent of lime.

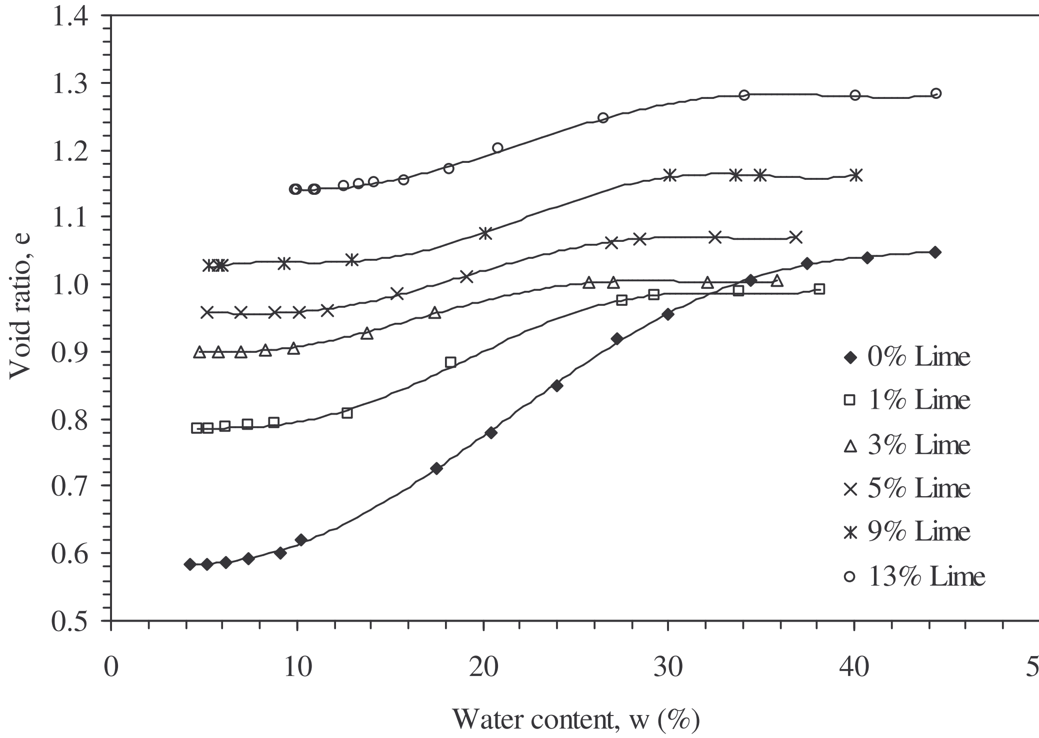

Fig. 6.69 Water content vs. void ratio responses of 20%ES+80%RS with different percent of lime.

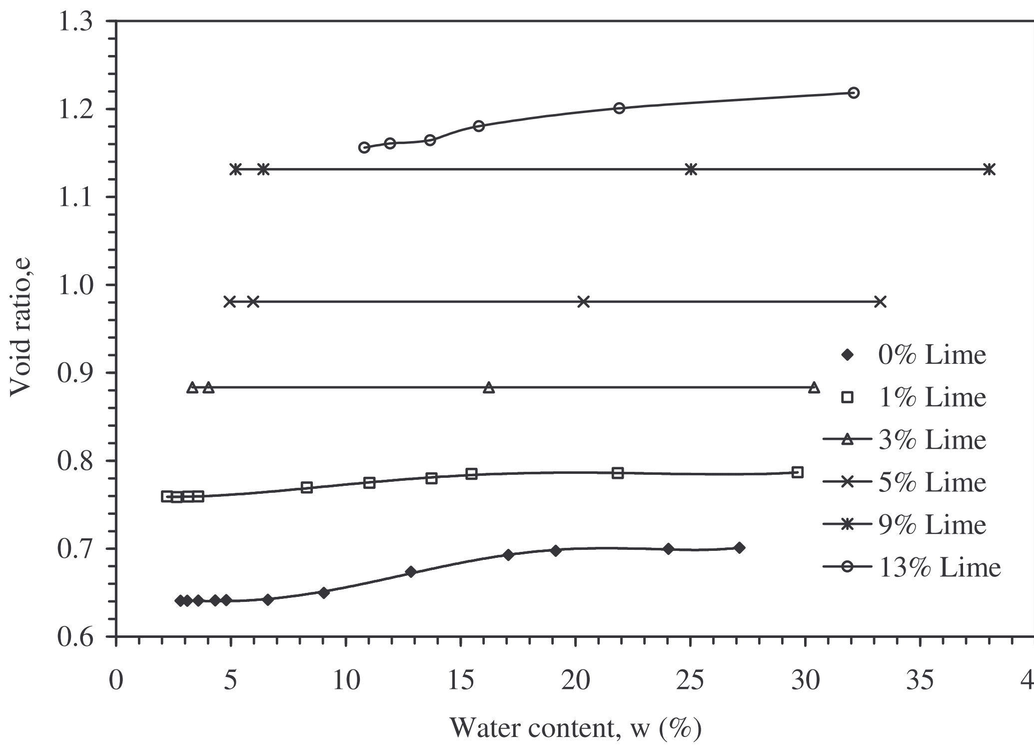

Fig. 6.70 Water content vs. void ratio responses of 100%RS with different percent of lime.

Table 6.4 Shrinkage magnitudes and e-w relations for lime-treated expansive soil (100%ES).

Soil	Lime (%)	Shrinkage magnitude			e-w relations		
		S_I (%)	S_P (%)	S_R (%)	Initial shrinkage	Primary shrinkage	Residual shrinkage
100%ES	0	0.61	54.88	1.42	e = 0.002w + 2.12 R^2 = 0.9	e = 0.027w + 0.12 R^2 = 0.98	e = 0.001w + 0.39 R^2 = 0.62
	1	0.65	51.84	2.00	e = 0.002w + 2.04 R^2 = 0.98	e = 0.026w + 0.3 R^2 = 0.98	e = 0.007w + 0.47 R^2 = 0.94
	3	0.15	27.45	1.36	e = 0.0003w + 1.61 R^2 = 0.98	e = 0.019w + 0.74 R^2 = 0.99	e = 0.007w + 0.85 R^2 = 0.96
	5	1.56	17.16	1.12	e = 0.003w + 1.43 R^2 = 0.97	e = 0.012w + 0.955 R^2 = 0.97	e = 0.004w + 1.07 R^2 = 0.98
	9	0.40	18.98	0.54	e = 0.002w + 1.63 R^2 = 0.98	e = 0.012w + 1.05 R^2 = 0.98	e = 0.002w + 1.18 R^2 = 0.92
	13	1.36	17.94	0.13	e = 0.003w + 1.75 R^2 = 0.95	e = 0.018w + 0.96 R^2 = 0.98	e = 0.0005w + 1.36 R^2 = 0.97

Table 6.5 Shrinkage magnitudes and e-w relations for lime-treated expansive soil-residual soil mix (80%ES+20%RS)

Soil	Lime (%)	Shrinkage magnitude			e-w relations		
		S_I (%)	S_P (%)	S_R (%)	Initial shrinkage	Primary shrinkage	Residual shrinkage
80%ES + 20%RS	0	0.32	53.99	1.39	e = 0.001w +2.04 R^2 = 0.92	e = 0.027w +0.12 R^2 = 0.98	e = 0.004w +0.39 R^2 = 0.93
	1	0.78	38.66	0.58	e = 0.004w + 1.52 R^2 = 1	e = 0.021w + 0.46 R^2 = 0.99	e = 0.002w + 0.69 R^2 = 0.93
	3	0.88	17.90	0.31	e = 0.002w + 1.28 R^2 = 0.95	e = 0.016w + 0.73 R^2 = 0.99	e = 0.002w + 0.91 R^2 = 0.99
	5	0.55	17.04	0.14	e = 0.003w + 1.36 R^2 = 0.93	e = 0.012w + 0.97 R^2 = 0.98	e = 0.001w + 1.09 R^2 = 0.98
	9	0.15	17.82	0.31	e = 0.0004w + 1.70 R^2 = 0.94	e = 0.012w + 1.12 R^2 = 0.97	e = 0.002w + 1.22 R^2 = 0.92
	13	0.37	14.44	0.43	e = 0.001w + 1.65 R^2 = 0.92	e = 0.014w + 1.05 R^2 = 0.98	e = 0.003w + 1.25 R^2 = 0.93

Table 6.6 Shrinkage magnitudes and e-w relations for lime-treated expansive soil-residual soil mix (60%ES+40%RS)

Soil	Lime (%)	Shrinkage magnitude			e-w relations		
		S_I (%)	S_P (%)	S_R (%)	Initial shrinkage	Primary shrinkage	Residual shrinkage
60%ES + 40%RS	0	0.82	38.17	1.34	e = 0.003w +1.25 $R^2 = 1$	e = 0.021w +0.26 $R^2 = 0.99$	e = 0.004w +0.45 $R^2 = 0.93$
	1	1.08	28.22	0.67	e = 0.003w + 1.36 $R^2 = 0.93$	e = 0.018w + 0.65 $R^2 = 0.99$	e = 0.004w + 0.75 $R^2 = 0.92$
	3	0.75	13.83	0.88	e = 0.002w + 1.19 $R^2 = 0.93$	e = 0.014w + 0.77 $R^2 = 0.99$	e = 0.004w + 0.88 $R^2 = 1$
	5	0.04	11.37	0.098	e = 0.0001w + 1.29 $R^2 = 0.95$	e = 0.012w + 0.88 $R^2 = 0.98$	e = 0.001w + 1.03 $R^2 = 0.93$
	9	0.08	12.28	0.22	e = 0.0002w + 1.56 $R^2 = 0.95$	e = 0.011w + 1.07 $R^2 = 0.98$	e = 0.001w + 1.24 $R^2 = 0.92$
	13	0.15	9.87	0.085	e = 0.0003w + 1.6 1 $R^2 = 0.96$	e = 0.01w + 1.19 $R^2 = 0.99$	e = 0.001w + 1.36 $R^2 = 0.95$

Table 6.7 Shrinkage magnitudes and e-w relations for lime-treated expansive soil-residual soil mix (40%ES+60%RS)

Soil	Lime (%)	Shrinkage magnitude			e-w relations		
		S_I (%)	S_P (%)	S_R (%)	Initial shrinkage	Primary shrinkage	Residual shrinkage
40%ES + 60%RS	0	0.44	29.20	1.25	e = 0.003w +1.09 R^2 = 0.99	e = 0.021w +0.32 R^2 = 0.97	e = 0.003w +0.56 R^2 = 0.95
	1	0.42	22.89	1.54	e = 0.001w + 1.32 R^2 = 0.94	e = 0.019w + 0.59 R^2 = 0.98	e = 0.006w + 0.75 R^2 = 0.99
	3	0.14	9.41	0.30	e = 0.0003w + 1.17 R^2 = 0.98	e = 0.011w + 0.84 R^2 = 0.98	e = 0.001w + 0.96 R^2 = 0.89
	5	0.18	6.24	0.25	e = 0.001w + 1.17 R^2 = 0.95	e = 0.005w + 0.97 R^2 = 0.99	e = 0.002w + 1.01 R^2 = 0.94
	9	0.21	9.69	0.19	e = 0.001w + 1.37 R^2 = 0.99	e = 0.009w + 1.03 R^2 = 0.98	e = 0.001w + 1.15 R^2 = 0.92
	13	0.28	7.27	0.21	e = 0.001w + 1.47 R^2 = 0.98	e = 0.008w + 1.17 R^2 = 0.95	e = 0.001w + 1.31 R^2 = 0.91

Table 6.8 Shrinkage magnitudes and e-w relations for lime-treated expansive soil-residual soil mix (20%ES+80%RS)

Soil	Lime (%)	Shrinkage magnitude			e-w relations		
		S_I (%)	S_P (%)	S_R (%)	Initial shrinkage	Primary shrinkage	Residual shrinkage
20%ES + 80%RS	0	0.97	21.67	0.63	$e = 0.003w + 0.94$ $R^2 = 0.99$	$e = 0.012w + 0.47$ $R^2 = 0.99$	$e = 0.004w + 0.57$ $R^2 = 0.92$
	1	0.45	9.57	0.50	$e = 0.001w + 0.95$ $R^2 = 0.99$	$e = 0.01w + 0.70$ $R^2 = 0.99$	$e = 0.002w + 0.77$ $R^2 = 0.99$
	3	0.10	4.84	0.32	$e = 0.0002w + 0.99$ $R^2 = 0.98$	$e = 0.006w + 0.85$ $R^2 = 0.99$	$e = 0.001w + 0.89$ $R^2 = 0.92$
	5	0.05	5.27	0.15	$e = 0.0001w + 1.06$ $R^2 = 0.99$	$e = 0.007w + 0.89$ $R^2 = 0.99$	$e = 0.0004w + 0.96$ $R^2 = 0.92$
	9	0.09	5.78	0.44	$e = 0.0002w + 1.16$ $R^2 = 0.97$	$e = 0.007w + 0.94$ $R^2 = 0.98$	$e = 0.001w + 1.02$ $R^2 = 0.99$
	13	0.09	5.66	0.42	$e = 0.0002w + 1.27$ $R^2 = 0.99$	$e = 0.007w + 1.05$ $R^2 = 0.97$	$e = 0.003w + 1.11$ $R^2 = 0.93$

Table 6.9 Shrinkage magnitudes and e-w relations for lime-treated residual soil mix (100%RS)

Soil	Lime (%)	Shrinkage magnitude			e-w relations		
		S_I (%)	S_P (%)	S_R (%)	Initial shrinkage	Primary shrinkage	Residual shrinkage
100%RS	0	0.18	3.29	0.06	e = 0.0004w + 0.69 R^2 = 0.99	e = 0.005w + 0.61 R^2 = 0.99	e = 0.0003w + 0.64 R^2 = 0.93
	1	0.09	1.45	0.05	e = 0.0001w + 0.78 R^2 = 0.99	e = 0.002w + 0.75 R^2 = 0.99	e = 0.001w + 0.75 R^2 = 0.97
	3	0	0	0	----	----	----
	5	0	0	0	----	----	----
	9	0	0	0	----	----	----
	13	0.77	1.64	0.42	e = 0.002w + 1.16 R^2 = 1	e = 0.004w + 1.11 R^2 = 0.96	e = 0.003w + 1.13 R^2 = 0.97

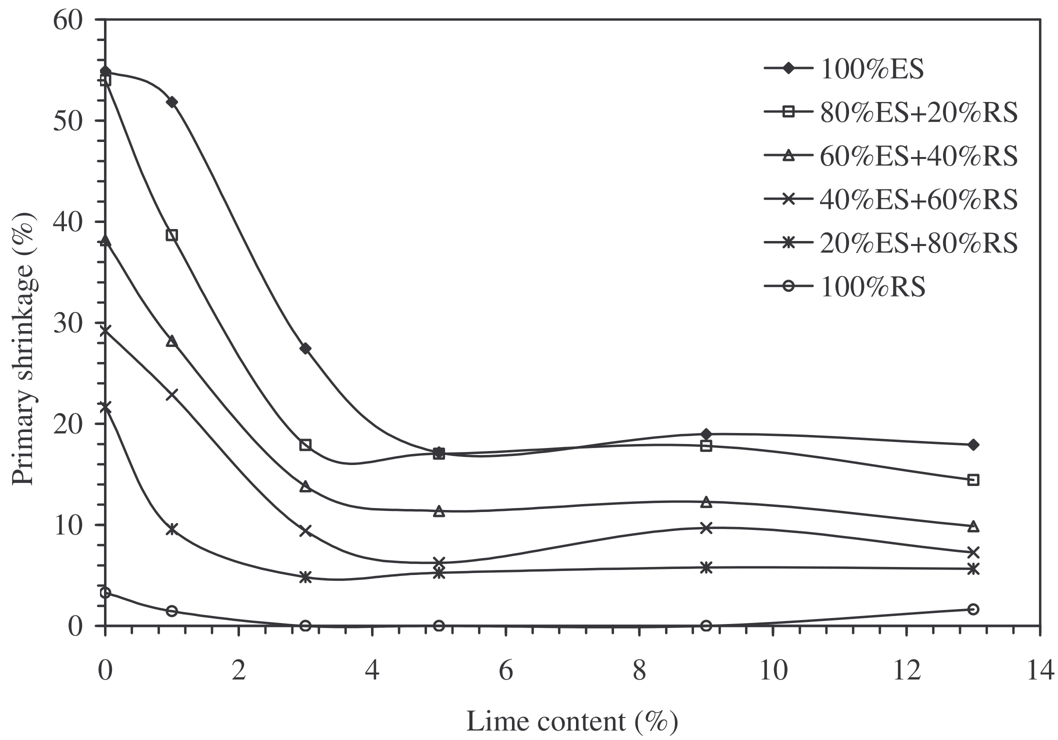

Fig. 6.71 Effect of lime on primary shrinkage of soils

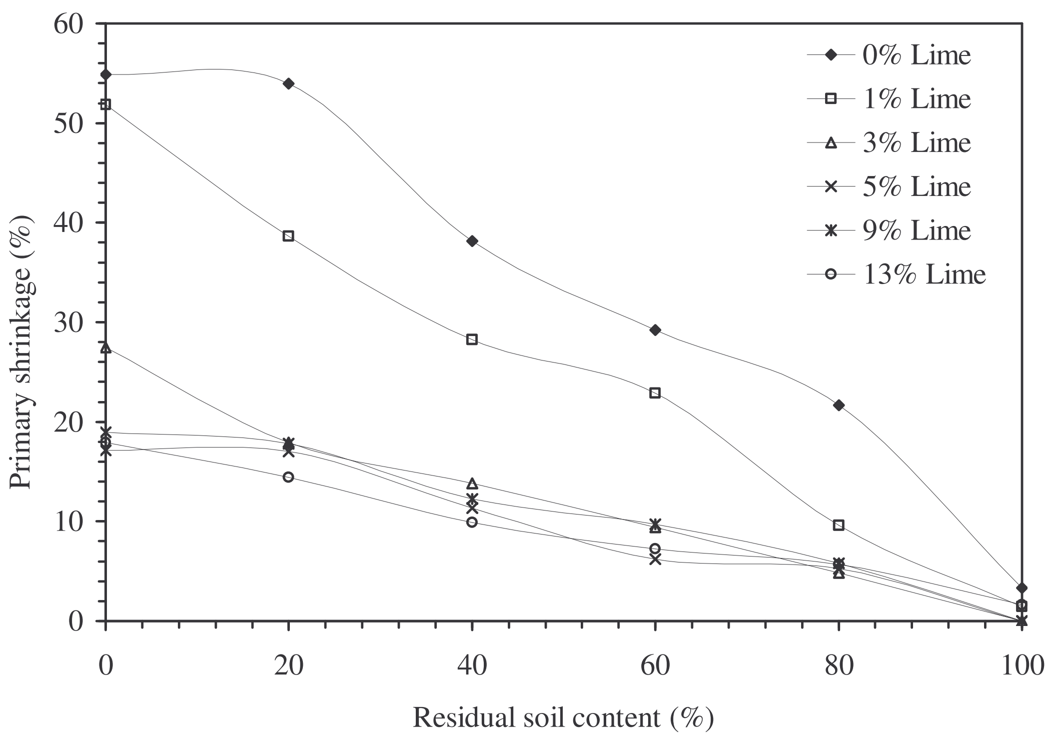

Fig. 6.72 Effect of residual soil content on shrinkage of lime treated soils.

Variation of primary shrinkage magnitude with liquid limit and plasticity index are depicted in Fig. 6.73 and Fig.6.74 respectively. It could be observed that irrespective of soil type and lime content plasticity plays a dominant role on the shrinkage potential of the lime treated soils.

Fig. 6.75 shows that the void ratio of the soil specimens at residual shrinkage (e_{rs}) shows an increasing trend with increased lime content. However, beyond about 5% lime content the rate of increase in e_{rs} has reduced. This indicates that after certain stage the process of lime induced modification in soil fabric tends to get stabilized.

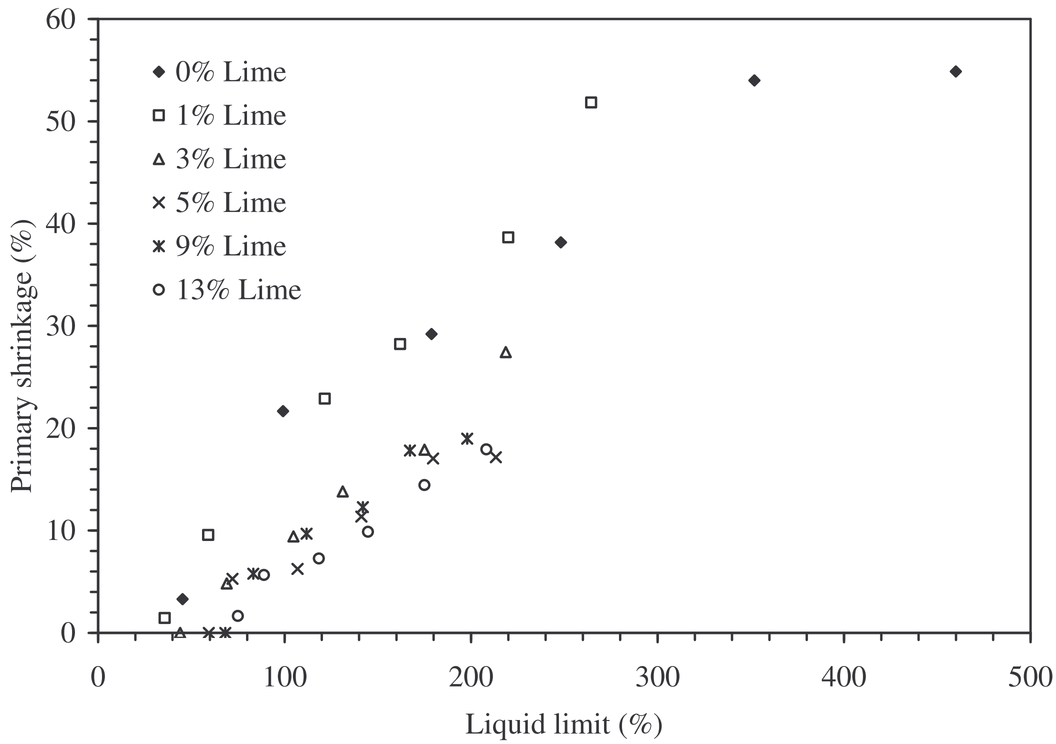

Fig. 6.73 Primary shrinkage vs. liquid for lime treated soils.

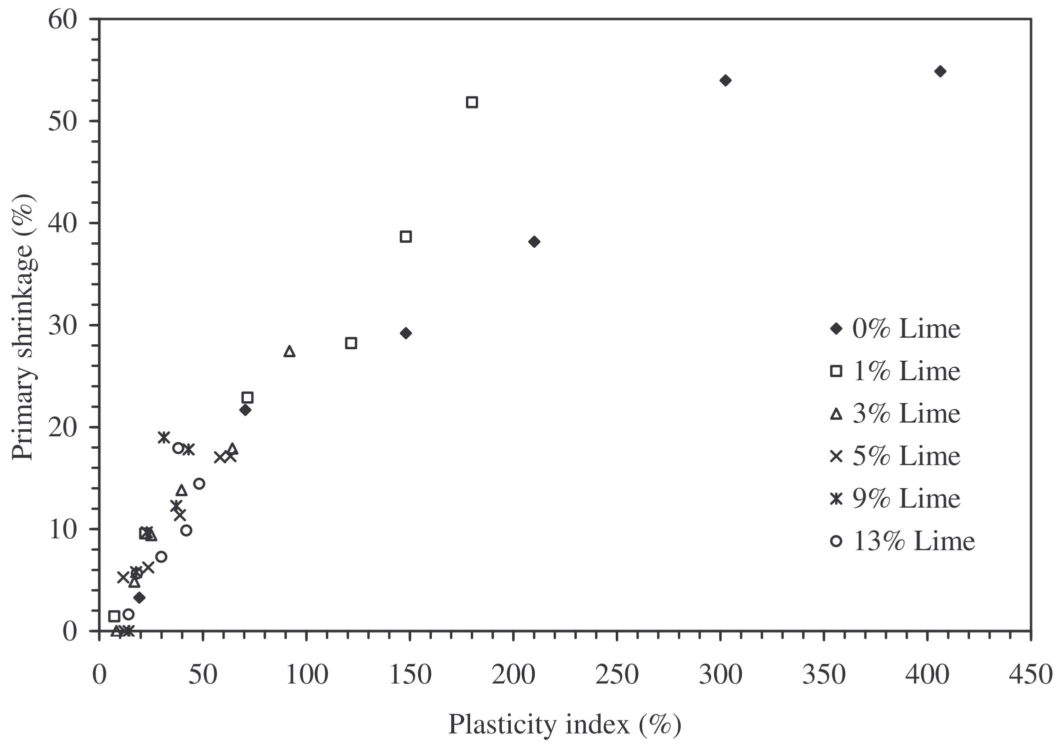

Fig. 6.74 Primary shrinkage vs. plasticity index for lime treated soils.

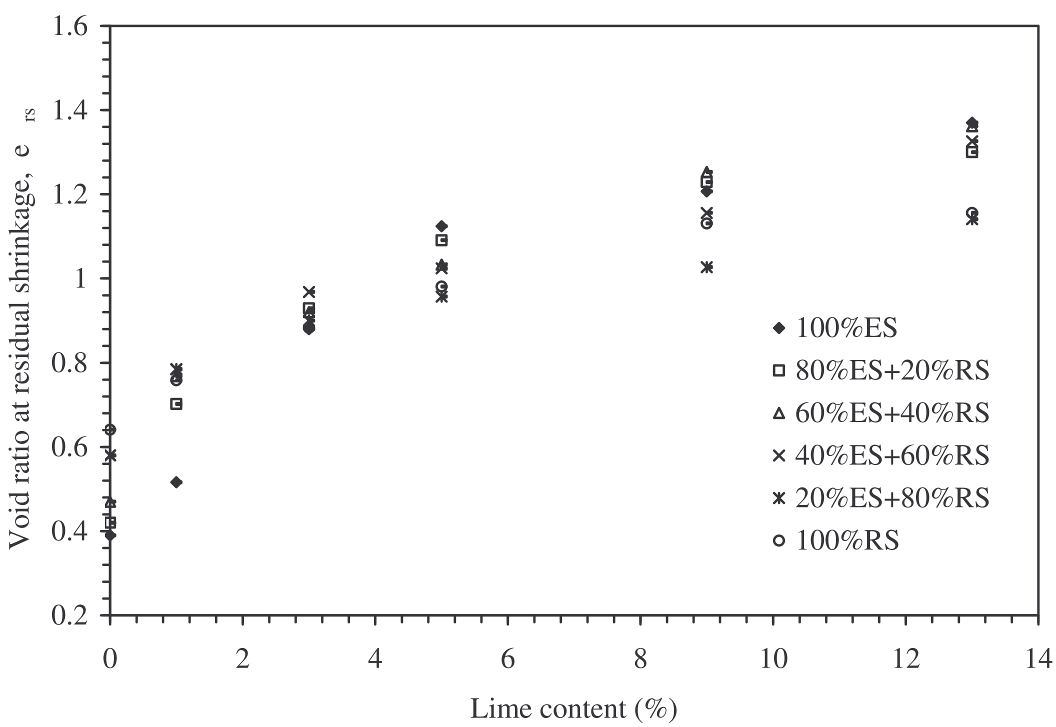

Fig. 6.75 Void ratio at residual shrinkage vs. lime content for lime treated soils

6.4 SUMMARY

In this chapter the influence of lime on the swell and shrinkage behaviour of expansive soil with varied content of residual soils has been studied. These soil samples having wide range of plasticities, running from very low to extremely high, typically represent the soils with varied plasticity characteristics encountered in nature. The lime content was varied from 0% to 13%. The curing period was varied from immediate to 28 days. The test results indicate that lime can effectively reduce swelling and shrinkage. However for silica rich soils swelling tends to increase at relatively higher lime content (i.e. > 9%). Similar to the untreated soils the free swell index of lime treated soils correlates well with liquid limit and Oedometer swell potential. The shrinkage of lime treated soil shows similar responses (i.e. initial, primary and residual shrinkage) as the untreated soils. The linear variation of void ratio with water content during shrinkage has been made use of to obtain equations for predicting the shrinkage responses.

CHAPTER 7

STRENGTH BEHAVIOUR

7.1 INTRODUCTION

Civil engineering structures founded on weak soils are prone to damage as these soils owing to their poor strength and high compressibility undergo bearing capacity failure and large settlement. The problem is more severe in case of expansive soils which undergo further reduction in shear strength upon increased void ratio due to swelling. In view of this, compressive strength of expansive soil is an important parameter in evaluating its suitability for use as a construction material.

In the literature review it has been observed that the strength of expansive soils can be improved through addition of non swelling soils and lime. In the present research work it has been attempt to study the combined use of both these, on the strength behaviour of the expansive soil. The non-swelling residual soil has been added to the expansive soil in varied proportion. Subsequently these soil mixes were treated with lime in varied quantity and subjected to different curing periods. As the expansive soil-residual soil mixes cover an extreme range of plasticity characteristics (i.e. very low to very high) the test data also bring out the behaviour of lime treated soils over a wide range of plasticity of soils as encountered in practice.

Sivapullaiah et al. (1998) suggest that the increase in strength of soils due to cementation can better be studied thought unconfined compression test rather than by triaxial test where additional strength is contributed by confinement pressure. Besides many researchers (Hilt and Davidson, 1960; Mateos, 1964; Mitchell and Hooper, 1961; Croft, 1967; Al-Rawi, 1981; Bell,1988; Narasimha Rao et al., 1990; Rajasekaran and Narasimha Rao, 2000) have studied the compressive strength of stabilized soils using unconfined compressive strength test and have found this method to be quick and convenient. Therefore in the present investigation, unconfined compressive strength test has been carried out for studying the strength behaviour of untreated and lime-treated expansive soil-residual soil samples. The effect of quantity of lime, curing period and fines content etc. on the stress-strain response, failure pattern, stiffness, ductility and peak strength of the soils are studied. The results are presented and analysed in the following sections.

7.2 STRENGTH BEHAVIOUR OF ES-RS MIXES

The stress-strain responses of different soils (ES-RS mixes) are presented in Fig. 7.1. It could be observed that the residual soil (100%RS) in its stress-strain response has a well defined peak and large drop in post peak compression resistance (i.e. axial stress). This indicates that the compacted residual soil has a brittle behaviour. Indeed the post test picture of the soil specimen depicted in Fig. 7.2 (f) indicates the soil has undergone cracking under loading which is typical of brittle material characteristics. With addition of expansive soil the peak stress has reduced substantially. However there is no pronounced peak that the soil mass continues absorb the strain energy which is typical of ductile behaviour. Indeed the post test soil samples (Fig. 7.2 a-e) have not shown any clear slip plane and have undergone visible bulging.

The variation of unconfined compressive strength (peak stress) with residual soil content in the soil sample is shown in Fig. 7.3. It could be observed that the compressive strength of the soil mass at relatively less percentage of residual soil (i.e. < 40%) has nearly remained unchanged for varied content of residual soil. However with further increase in the residual soil content the compressive strength has shown a reducing tendency but once again peaks up at 100% residual soil. These results can better be explained with respect to the fines content in the soil mass as depicted in Fig. 7.4. It could be observed that initially the residual soil matrix has strength of about 380 kPa. With increased fines content it reduces to about 170 kPa. With further increase in fines content it once again begins to increase reaching a value of about 250 kPa at 93% fines content. Beyond this the compressive strength nearly remained unchanged with further increase in fines content. It should be mentioned here that the residual soil itself has 83% fines content out of which only about 11% is clay and rest are silt. While the expansive soil has most of fines as clay. The increased fines content in the ES-RS mixes being due to added expansive soil (ES) is therefore increased clay content in the system. With relatively less quantity of this clay, it mostly remains within the voids formed by the coarse particle skeleton. As a result of which it gets protected against the compaction stress and hence remains loose. This loose clay along with compaction water forms a slushy mass. The slushy mass serves as lubricant that reduces the friction at the interface of the coarse particles leading to substantial reduction in strength. With further increase in fines content it tends to fill the void that it shares the compaction energy and gets compacted that it effectively sustains the compression load leading to increased strength. A stage reaches when the fines content over flow the coarse grain void that the coarse grains are in floating condition in the fine soil mass thereby the overall behaviour is mostly due to response of the fine soil mass. With further increase in percentage of fines there is not much change in the behaviour, therefore, the compressive strength remains nearly unchanged.

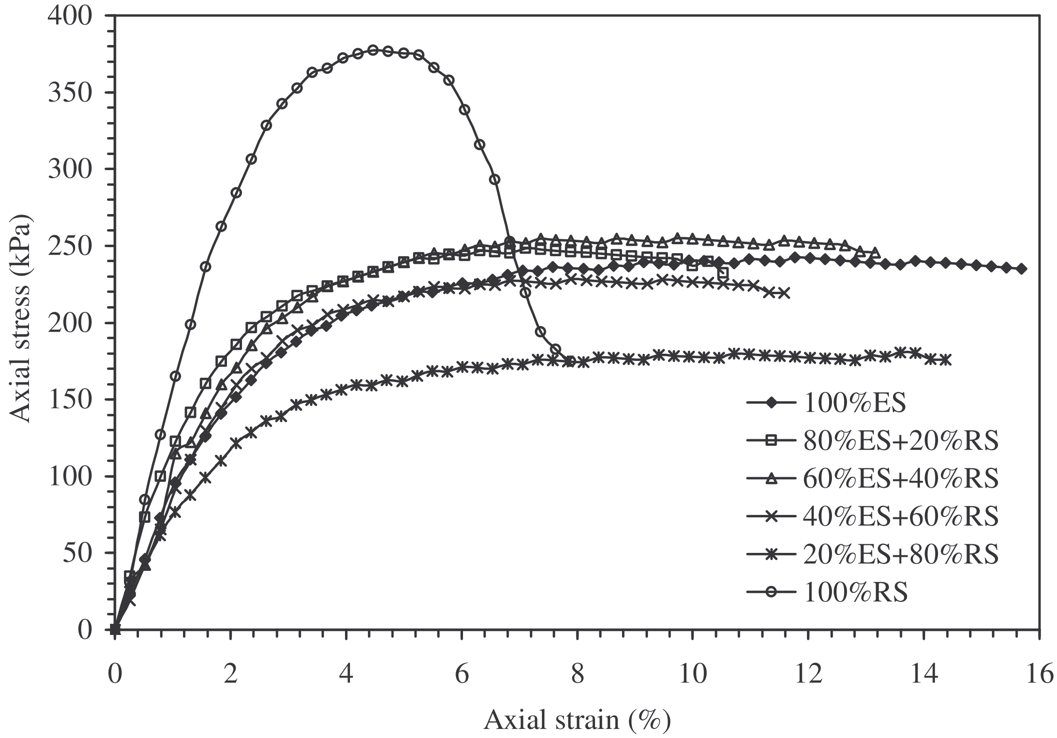

Fig. 7.1 Stress-strain responses of soils.

Fig. 7.2 Failure characteristics of soils (a) 100%ES (b) 80%ES+20%RS (c) 60%ES+40%RS (d) 40%ES+60%RS (e) 20%ES+80%RS (f) 100%RS

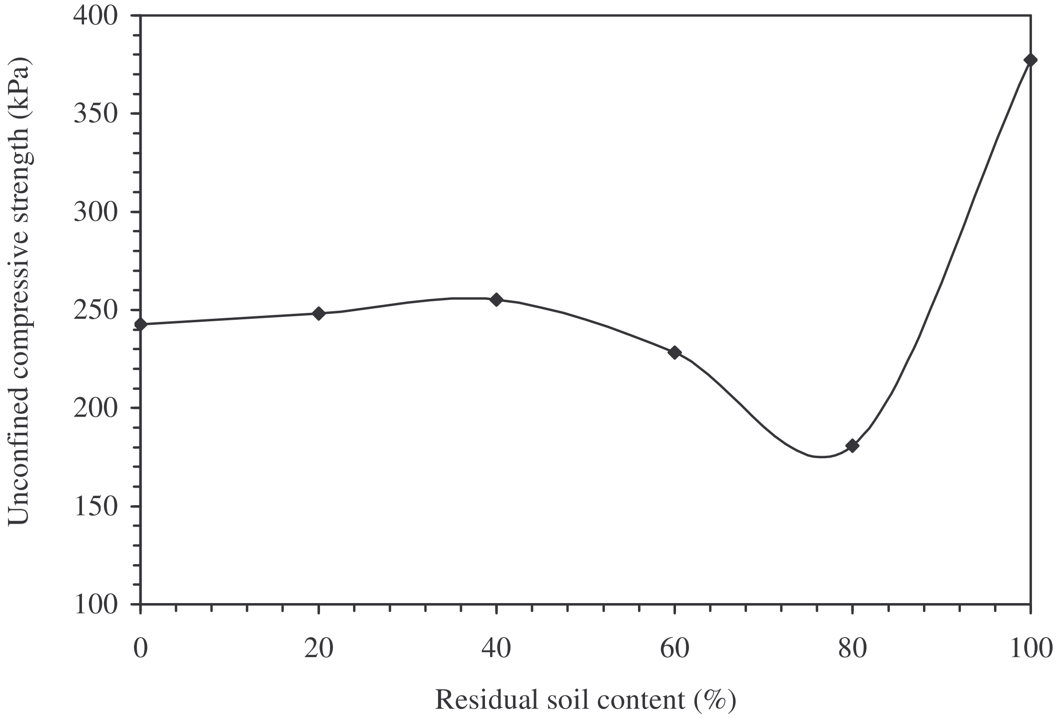

Fig. 7.3 Variation of unconfined compressive strength with residual soil content in expansive soil-residual soil mixes.

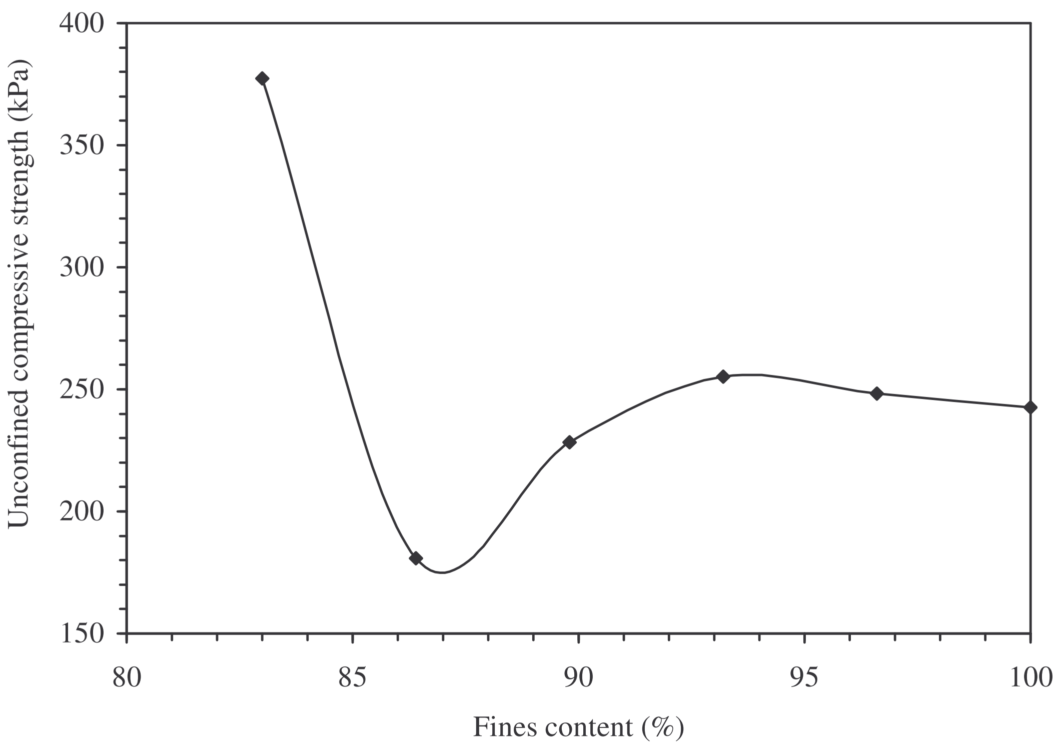

Fig. 7.4 Variation of unconfined compressive strength with fines content in expansive soil-residual soil mixes

7.3 STRENGTH BEHAVIOUR OF ES-RS-LIME MIXES

The stress-stain responses of lime treated expansive soils (100%ES) obtained from unconfined compression tests at 3 days curing period are presented in Fig. 7.5. The response of untreated soil (i.e. 0% lime) is included for comparison purpose. It could be observed that the curves exhibit increasingly pronounced peak with increased percentage of lime which is attributed due to cementation. The increased slope of the responses indicates an increase in stiffness of the soil as well. The untreated soil which has shown very large strain, typical of a ductile response; with lime treatment shows a sudden drop in stress after the peak indicating a brittle failure. Indeed the post test observed failure patterns depicted in Fig. 7.6 show visible cracks within akin to brittle failure, contrary to bulging in case of untreated soil (Fig. 7.2). It is of interest to note that with relatively low percentage of lime (i.e. 1% and 3%) the failure plane is inclined at about 45^0 indicating that the failure is due to shear. With increased lime content the cracks are nearly vertical or in wedge form which indicates that the specimen has behaved as vertically loaded column.

The influence of lime on stress-stain responses of the soil (100%ES) at curing period of 7 days, 21 days and 28 days are depicted in Fig. 7.7, 7.9 and 7.11 respectively. The corresponding failure patterns are shown in Fig. 7.8, 7.10 and 7.12 respectively. In some cases the soil samples collapsed under the load test, therefore, the failure pattern could not be photographed and hence are not reported here. It could be observed that the stress-strain responses indicate increased performance improvement with increased curing period and the failure pattern indicate an increased column like behaviour. With increased lime content and curing period the soil mass gets substantially cemented that it behaves as a coherent vertical member under the applied loading.

Fig. 7.13 shows the variation of unconfined compressive strength of the expansive soil with varied curing period. It should be mentioned here that the peak compressive stress at which failure took place is reported as the unconfined compressive strength of the soil sample. It could be observed that with 1% lime the strength increase is marginal. A small quantity of lime is mostly used for the initial requirement of the expansive soil in altering its diffuse double layer. With further increase in lime content the pozzolanic reaction starts taking place forming cementitious compound leading to increased strength. This is supported by the fact that strength increase with curing period is practically insignificant with 1% lime while it is substantially high with increased lime content. At 9% lime content and 28 days curing period the strength has gone upto 3000 kPa against 250 kPa for untreated soil, indicating a 12 fold increase. With adequate percent of lime and prolonged curing period there is increased formation of the cementitious compounds leading to increased strength. In other words, in the initial stage, due to less cementation the bridging between soil particles is not mechanically felt (Locat et al., 1990). With prolonged period of pozzolanic reaction, effective bridging between soil particles takes place leading to increased strength. It is of interest to note that beyond 9% lime, there takes place reduction in strength with further increase in lime content. This effect is more pronounced at increased curing period. It has been suggested by earlier researchers that since lime itself has neither appreciable friction nor cohesion (Bell, 1996) an excess amount of lime serves as a lubricant to the soil particles and thereby reduces strength Kumar et al. (2007) attributed the strength reduction due to the platy shapes of the unreacted lime particles present in the soil mass. However under the present study it has been observed that at higher lime content the liquid limit increases due to formation of cementitious gels having substantial volume of pores that hold the water within. Hence it can be said that with higher lime content the soil structure tends to be more porous that reduces the strength which counteracts the strength gain due to cementation. At very high lime, content due to formation of large quantity of gel material, there occurs an overall decrease in strength. Fig. 7.14 shows the influence of lime content on the failure strain. It could be observed that with addition of lime the failure strain has substantially reduced from about 12% to about 2%. Further addition of lime has not brought any more change in the failure strain.

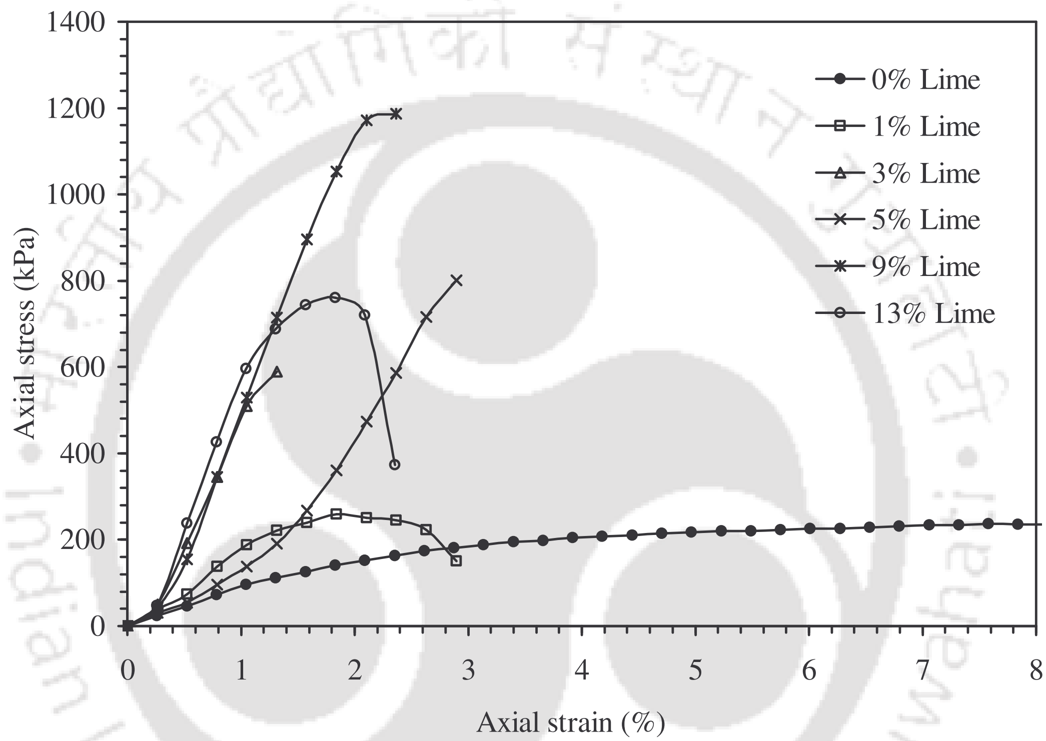

Fig. 7.5 Stress-strain responses of lime treated expansive soil (100%ES) at 3 days curing period

a b c d e

Fig. 7.6 Failure characteristics of lime treated expansive soil (100%ES) at 3 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime.

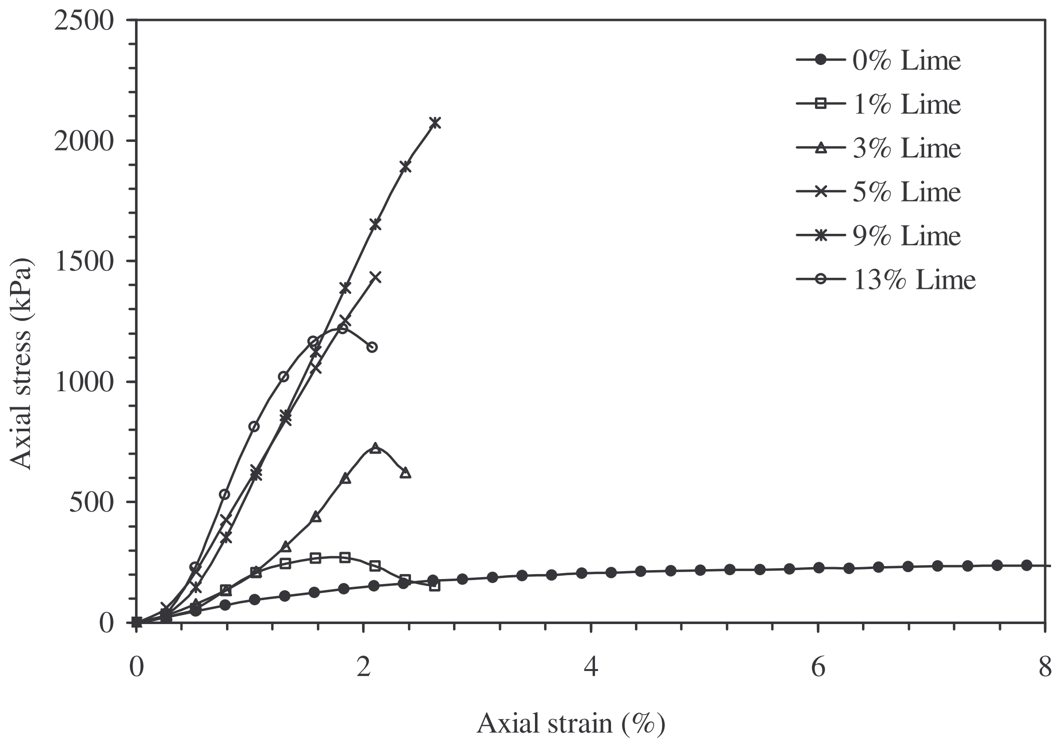

Fig. 7.7 Stress-strain responses of lime treated expansive soil (100%ES) at 7 days curing period.

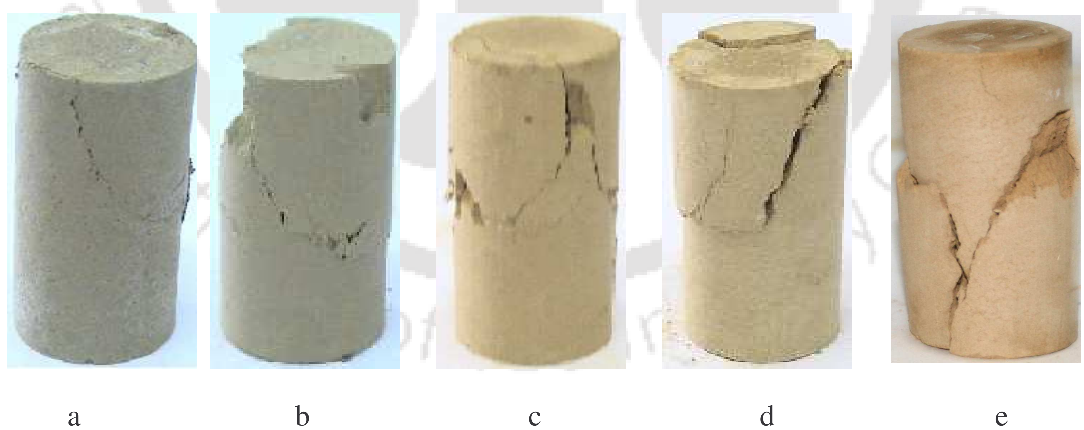

Fig. 7.8 Failure characteristics of lime treated expansive soil (100%ES) at 7 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime

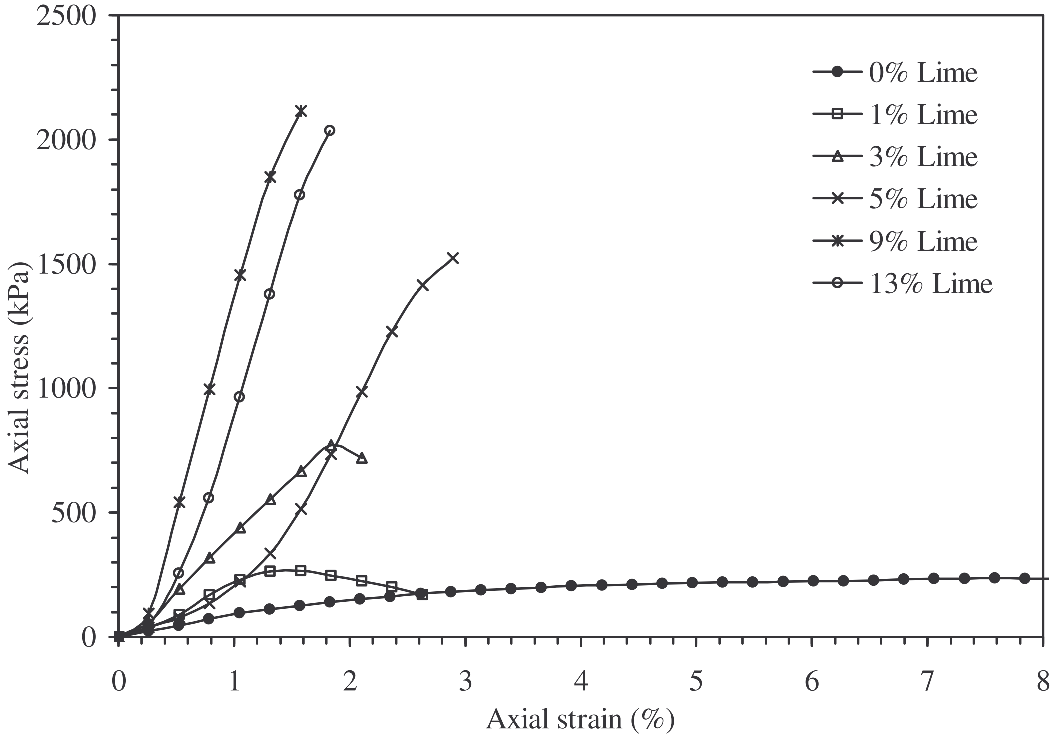

Fig. 7.9 Stress-strain responses of lime treated expansive soil (100%ES) at 21 days curing period

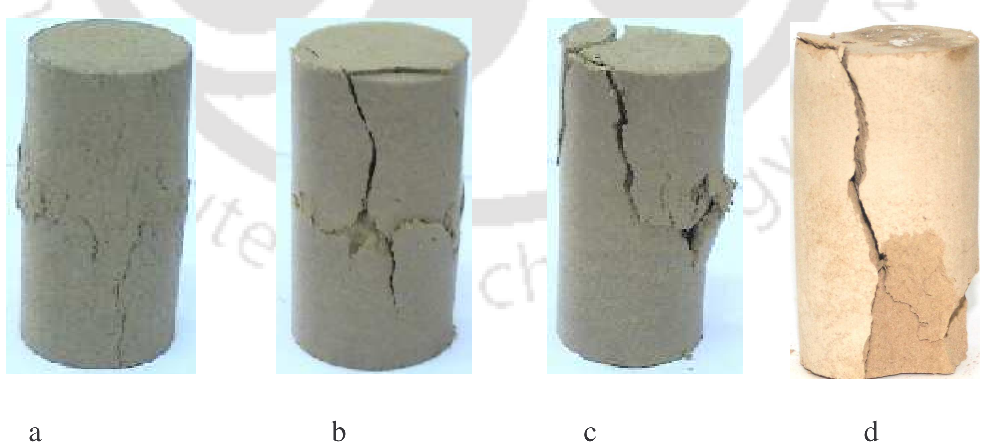

Fig. 7.10 Failure characteristics of lime treated expansive soil (100%ES) at 21 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 13% Lime

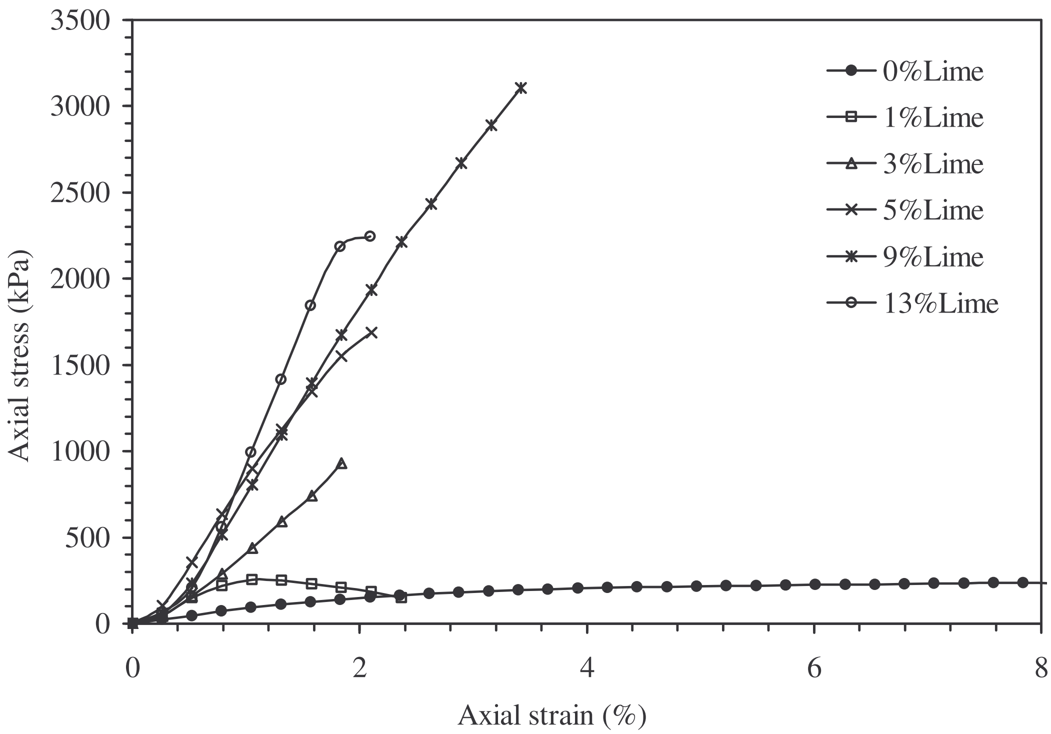

Fig. 7.11 Stress-strain responses of lime treated expansive soil (100%ES) at 28 days curing period

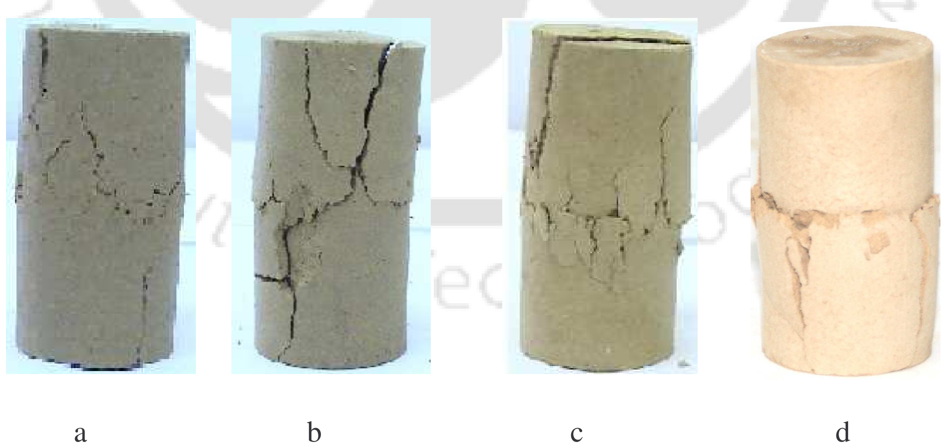

Fig. 7.12 Failure characteristics of lime treated expansive soil (100%ES) at 28 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 13% Lime

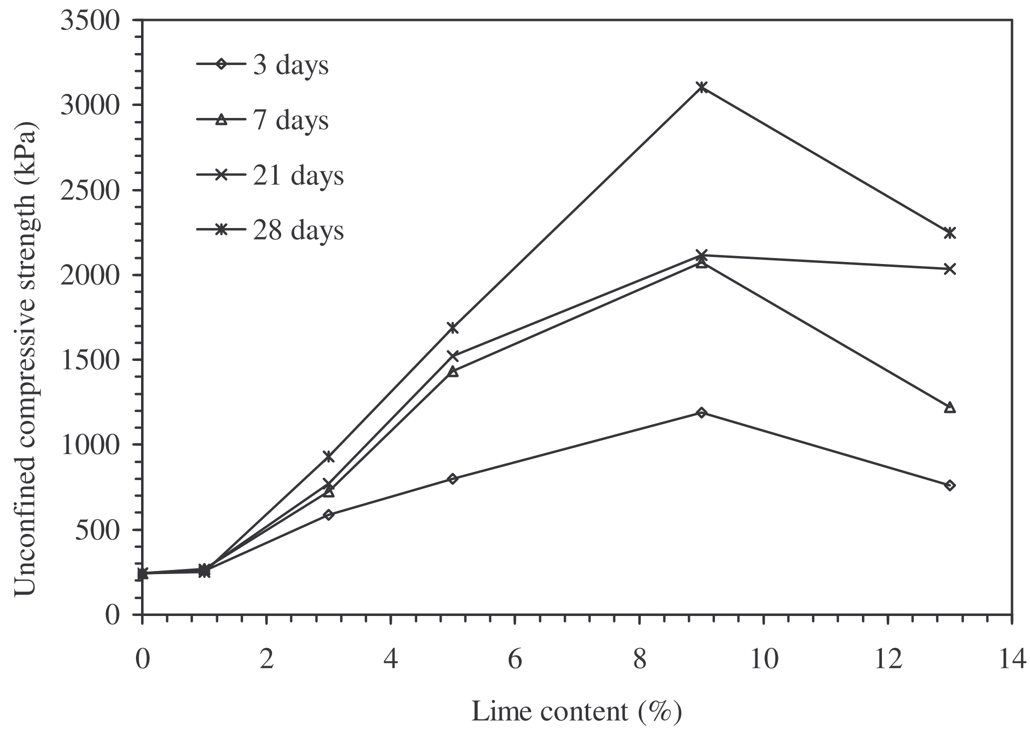

Fig. 7.13 Unconfined compressive strength vs. lime content for expansive soil (100%ES)

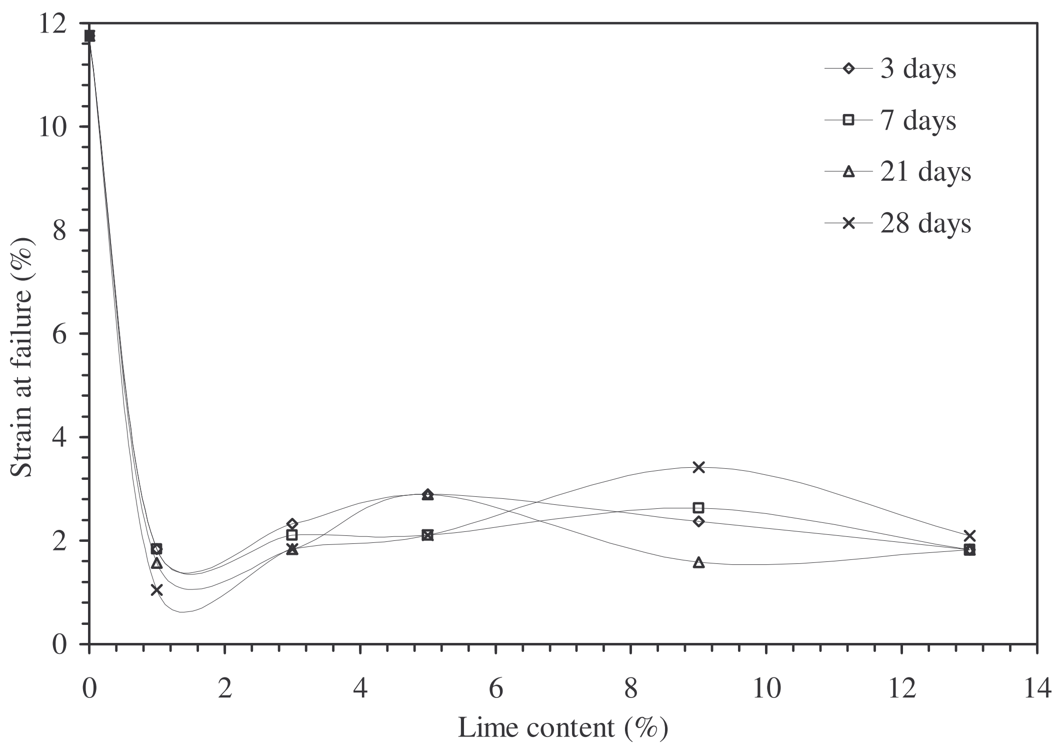

Fig. 7.14 Failure strain vs. lime content for expansive soil (100%ES).

The stress-strain responses of the lime treated soil mix having 80%ES and 20%RS with varied lime content and curing periods are presented in Figs. 7.15, 7.17, 7.19 and 7.21. It could be observed that in most of cases the strength after reaching peak, suddenly drops to zero. This trend is more prominent for 3%, 5% and 9% lime content. Correspondingly the failure patterns depicted in Fig. 7.16, 7.18, 7.20 and 7.22 show that in these cases the sample has undergone large scale cracking. While the soil specimen with 1% and 13% lime have shown relatively less cracking. Correspondingly the stress-strain responses in these cases have shown a relatively softer response. Fig. 7.23 shows the variation of unconfined compression strength with lime content for this soil (80%ES+20%RS), at different curing periods. It could be observed that the maximum strength in the present case is about 3600 kPa (Fig. 7.23; 9% lime, 28 days curing period) while in case of 100%ES it was about 3000 kPa (Fig. 7.13). The increase in strength is attributed to the increased content of sand present in the residual soil which is a stronger material compared to the expansive clay. The failure strains depicted in Fig. 7.24 shows that initially there is substantial reduction in failure strain due to addition of lime. However at higher lime content and prolonged period of curing the soil specimen have undergone relatively higher strain before failure. The soil presently has higher percentage of silica. Therefore with increased lime and curing period more of pozzolanic gelatinous material is formed. Besides in an acute alkaline environment (pH>12) the silica gets dissolved to form gel. As this gel has large volume of pores, it imparts a relatively porous structure that leads to increased strain. However these pores being extremely fine (called micropores) do not affect the coherency of the soil sample, that it behaves as a nearly intact mass giving rise to well defined failure planes.

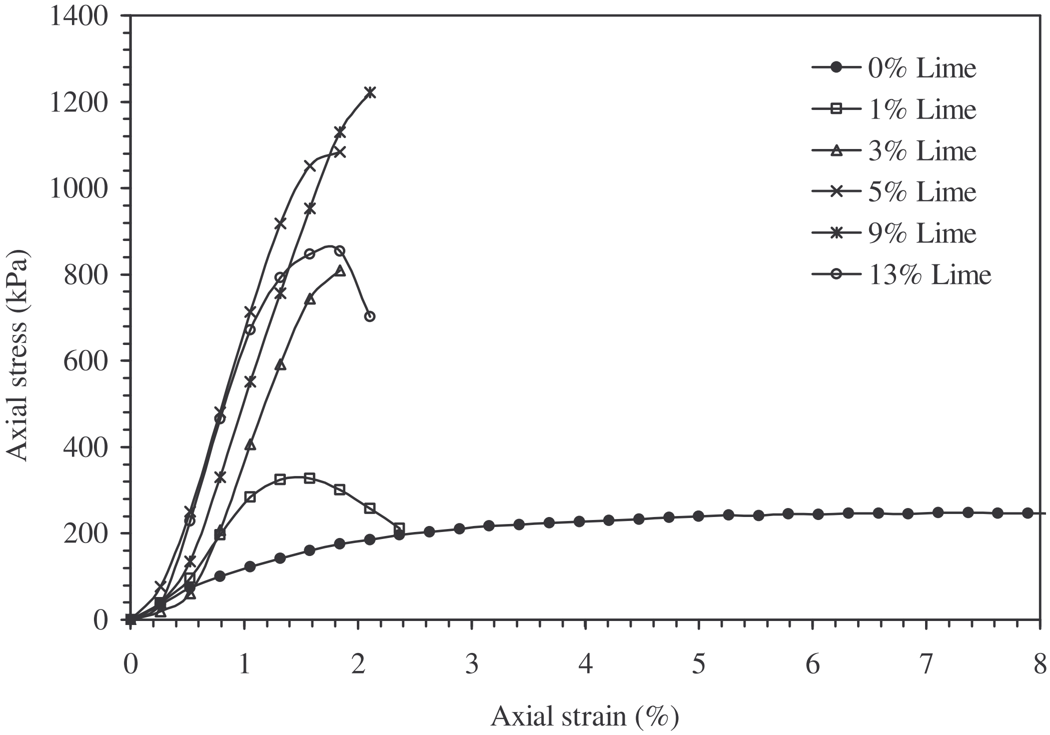

Fig. 7.15 Stress-strain responses of lime treated expansive soil-residual soil mix (80%ES+20%RS) at 3 days curing period.

a b c d e

Fig. 7.16 Failure characteristics of lime treated expansive soil-residual soil mix (80%ES+20%RS) at 3 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime

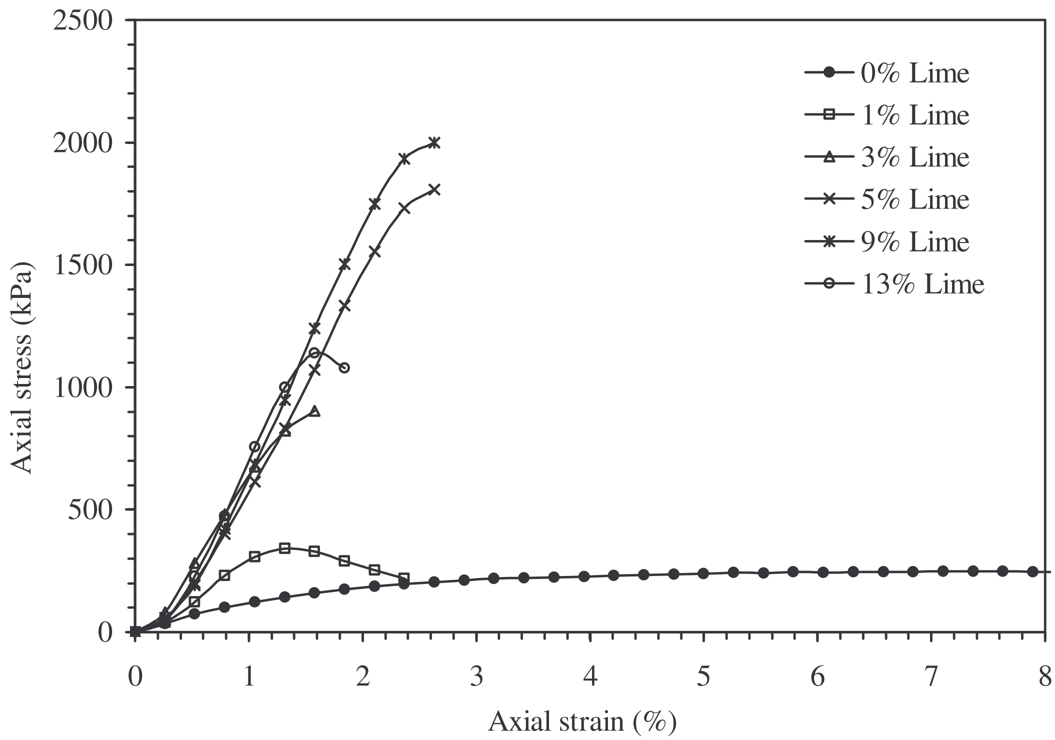

Fig. 7.17 Stress-strain responses of lime treated expansive soil-residual soil mix (80%ES+20%RS) at 7 days curing period.

a b c d

Fig. 7.18 Failure characteristics of lime treated expansive soil-residual soil mix (80%ES+20%RS) at 7 days curing period (a) 1% Lime (b) 5% Lime (c) 9% Lime (d) 13% Lime

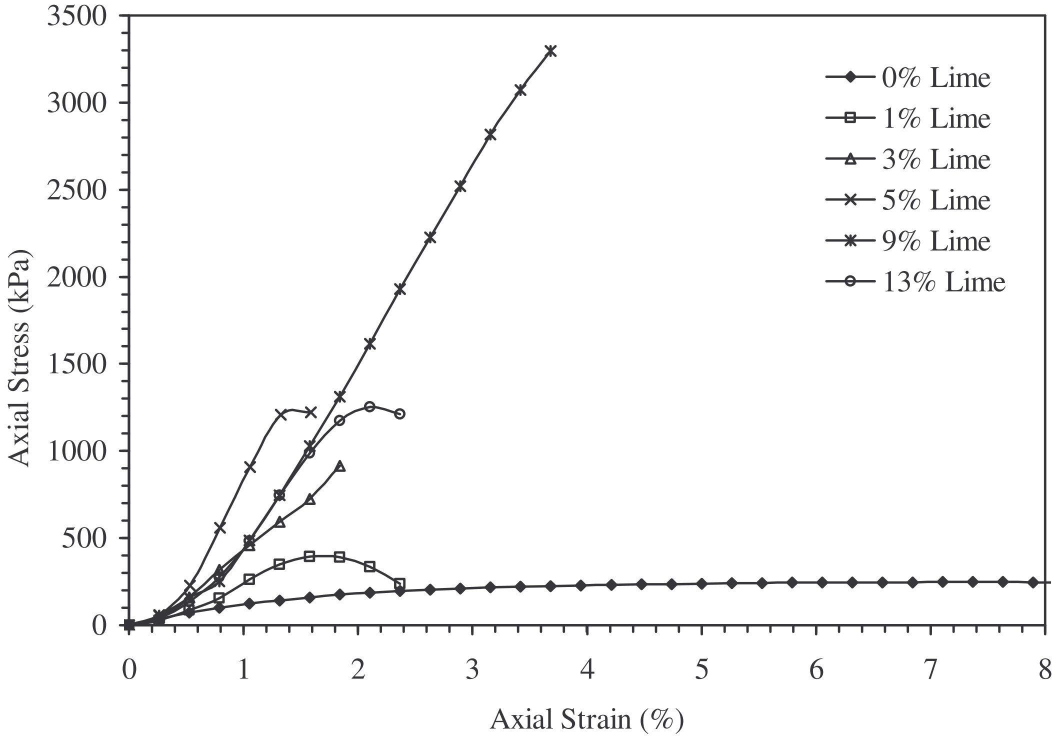

Fig. 7.19 Stress-strain responses of lime treated expansive soil-residual soil mix (80%ES+20%RS) at 21 days curing period.

a b c d

Fig. 7.20 Failure characteristics of lime treated expansive soil-residual soil mix (80%ES+20%RS) at 21 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 13% Lime

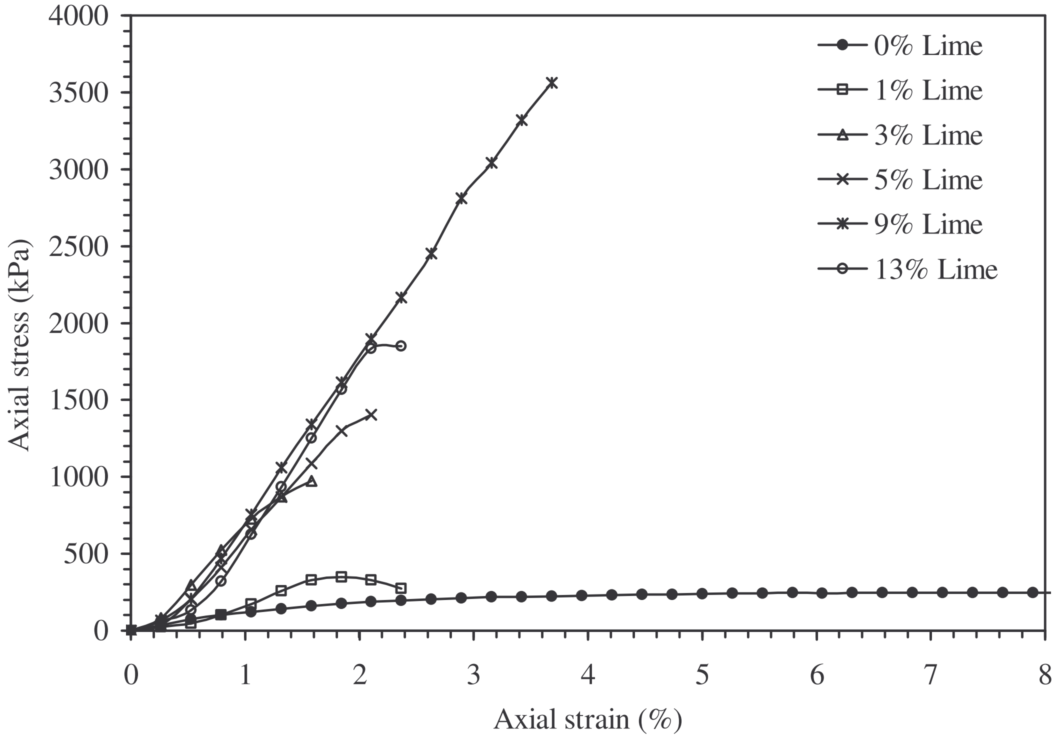

Fig. 7.21 Stress-strain responses of lime treated expansive soil-residual soil mix (80%ES+20%RS) at 28 days curing period.

a b c d

Fig. 7.22 Failure characteristics of lime treated expansive soil-residual soil mix (80%ES+20%RS) at 28 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime

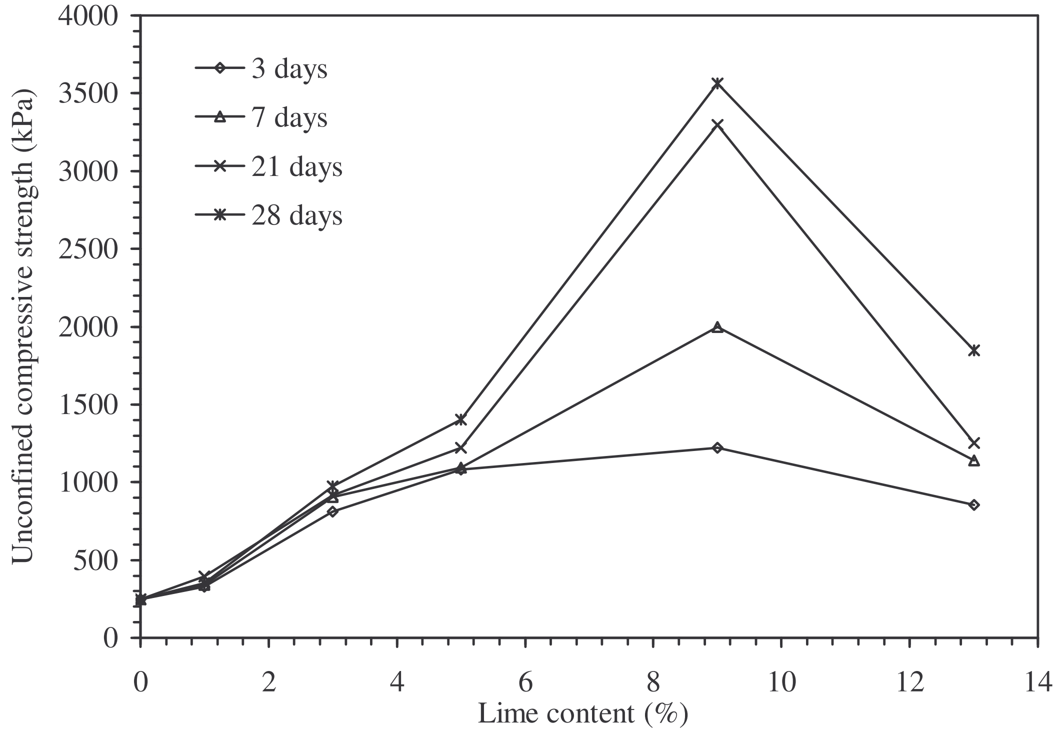

Fig. 7.23 Unconfined compressive strength vs. lime content for expansive soil-residual soil mix (80%ES+20%RS).

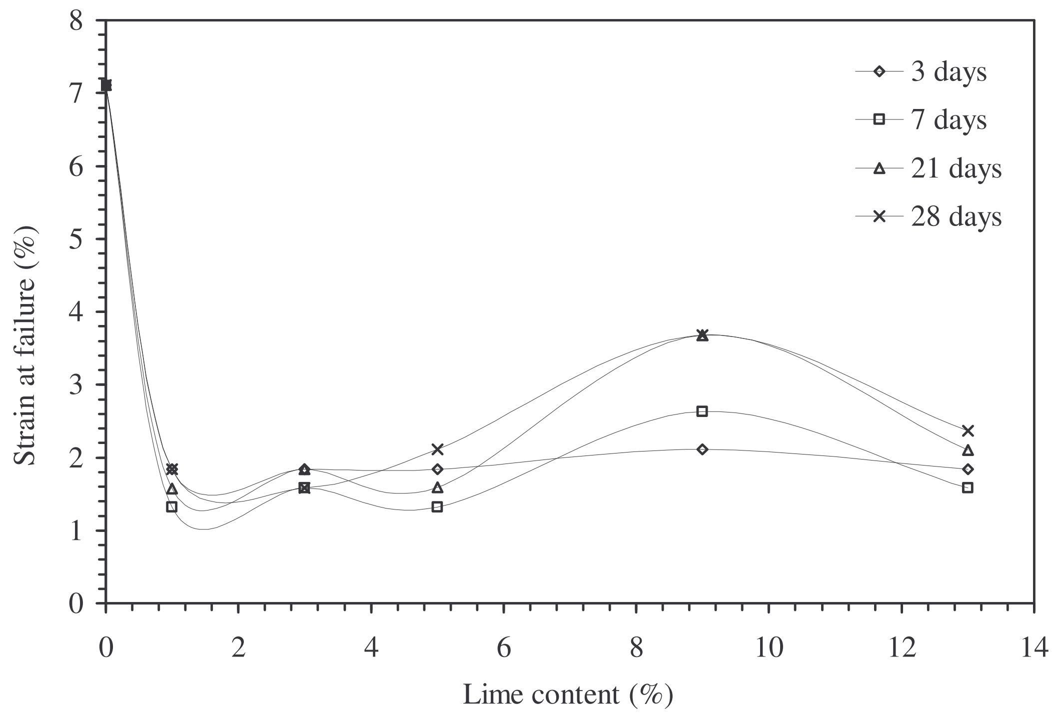

Fig. 7.24 Failure strain vs. lime content for expansive soil-residual soil mix (80%ES+20%RS).

The stress-strain responses and failure patterns of the specimens, 60%ES+40%RS and 40%ES+60%RS are presented in Fig. 7.25 to 7.40. It could be observed that the responses have shown increased stiffness and brittle behaviour. Besides the maximum strength has increased to about 4300 kPa (Fig. 7.39). It is attributed to the increased silica content that has enhanced the pozzolanic reaction and hence cementation giving rise to enhanced strength and stiffness. However with further increase in residual soil content (i.e. 20%ES+80%RS) the stress-strain responses (Fig. 7.41, 7.43, 7.45, 7.47) have shown relatively softer behaviour and the maximum strength has reduced to about 2700 kPa (28 days curing period, Fig. 7.47). However the failure patterns have shown more of a column like behaviour (vertical cracks). The reduction in strength and stiffness is attributed to the relatively high gel pores brought about by the pozzolanic relations due to increased silica content. This effect is more prominent in the case of residual soil (100%RS). The stress-strain responses have undergone visible reduction in stiffness (Fig. 7.49, 7.51, 7.53, 7.55). It is of interest to note that both at low percent of lime (i.e. 1%) and very high percent of lime (i.e. 13%) the lime treated soil has underperformed than the untreated soil. A very less quantity of lime does not raise the pH value of pore fluid to alkaline level to dissolve silica. Besides the residual soil having very less clay content, pozzolanic reactions are weak. Clare and Cruchley (1975) have reported that lime content of 1% and less produces a metastable state in soil. As a result of which the lime remains free thereby serves as a lubricant and hence reduces the strength of soil. A very large quantity of lime increases the pH value above 12 (Fig. 4.8) that the pore water turns highly alkaline, that it dissolves the silica. In a highly silica rich residual soil a large quantity of silica gel is formed that makes the soil specimen porous. Given the large volume of such pores the strength gain due to cementation and mechanical interaction is substantially counteracted by the strength reduction due to gel pores. Indeed the data presented in Fig. 7.57-7.60 indicate that the reduction in strength at high lime content increases with increase in residual soil content. Correspondingly the failure strain too has shown an increased magnitude (Figs. 7.61-7.64).

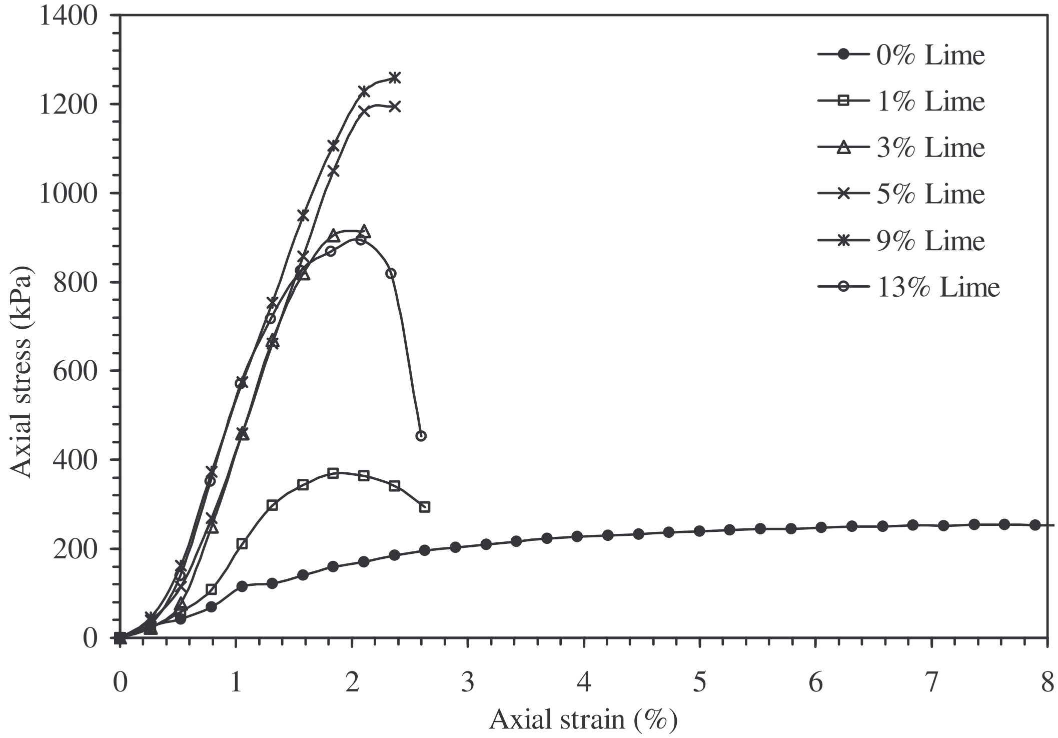

Fig. 7.25 Stress-strain responses of lime treated expansive soil-residual soil mix (60%ES+40%RS) at 3 days curing period.

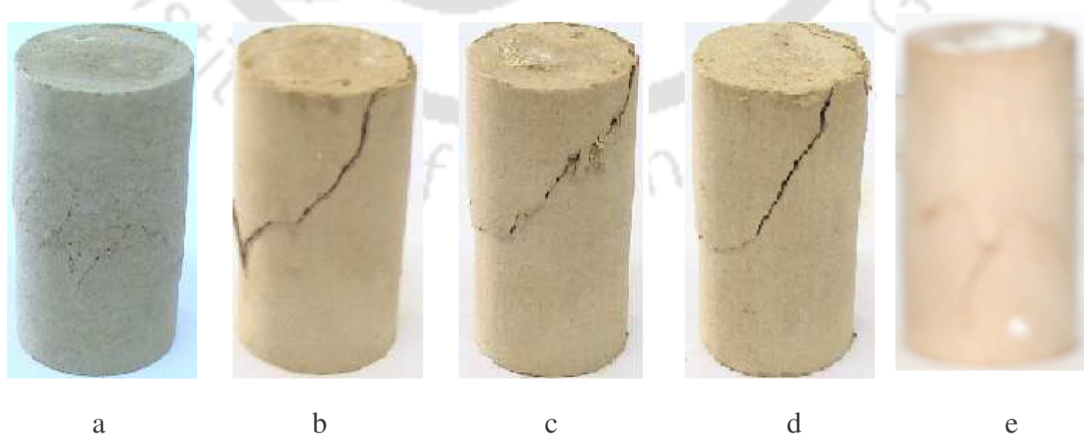

Fig. 7.26 Failure characteristics of lime treated expansive soil-residual soil mix (60%ES+40%RS) at 3 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime

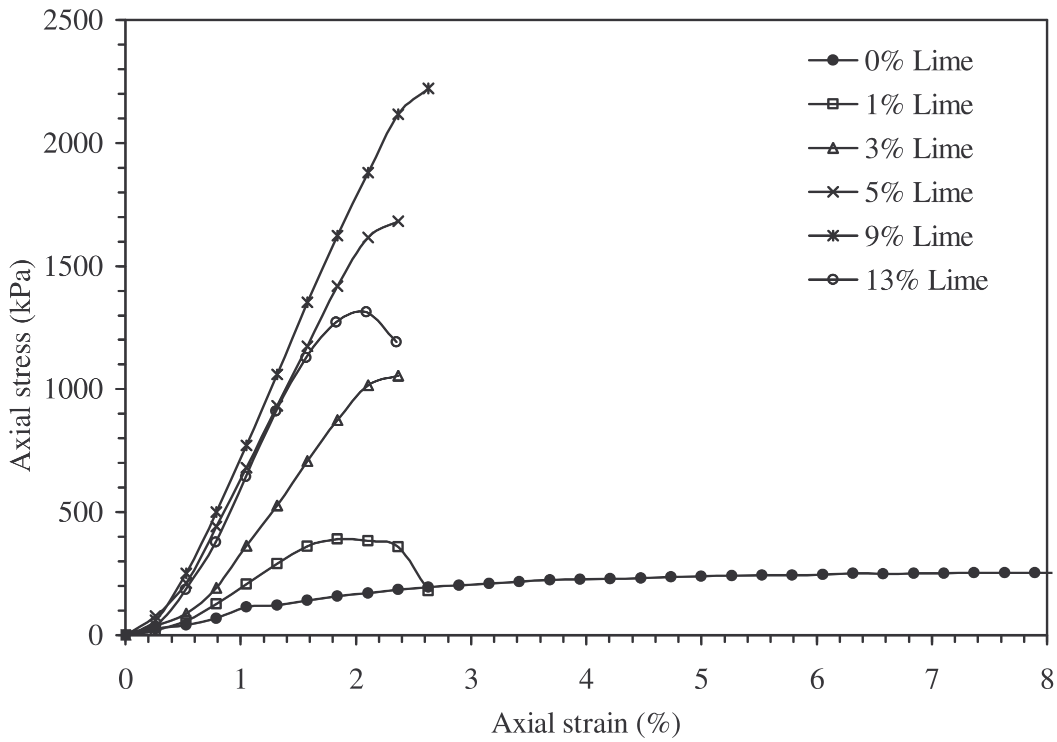

Fig. 7.27 Stress-strain responses of lime treated expansive soil-residual soil mix (60%ES+40%RS) at 7 days curing period.

a b c d e

Fig. 7.28 Failure characteristics of lime treated expansive soil-residual soil mix (60%ES+40%RS) at 7 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime

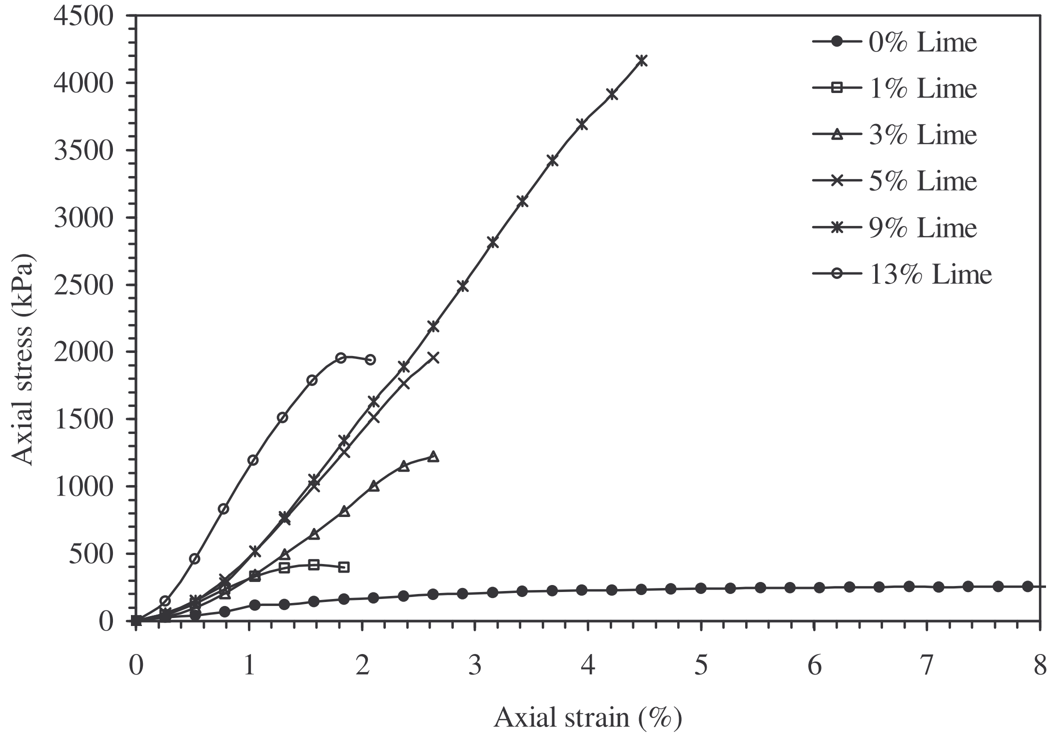

Fig. 7.29 Stress-strain responses of lime treated expansive soil-residual soil mix (60%ES+40%RS) at 21 days curing period.

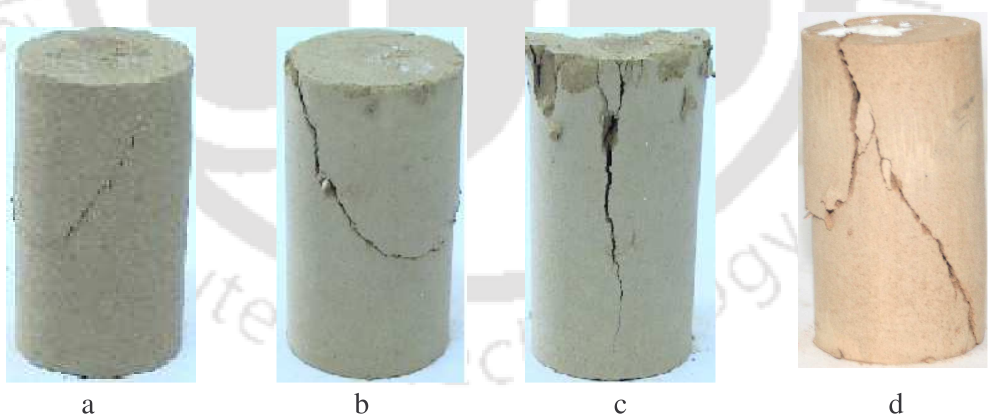

Fig. 7.30 Failure characteristics of lime treated expansive soil-residual soil mix (60%ES+40%RS) at 21 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 13% Lime

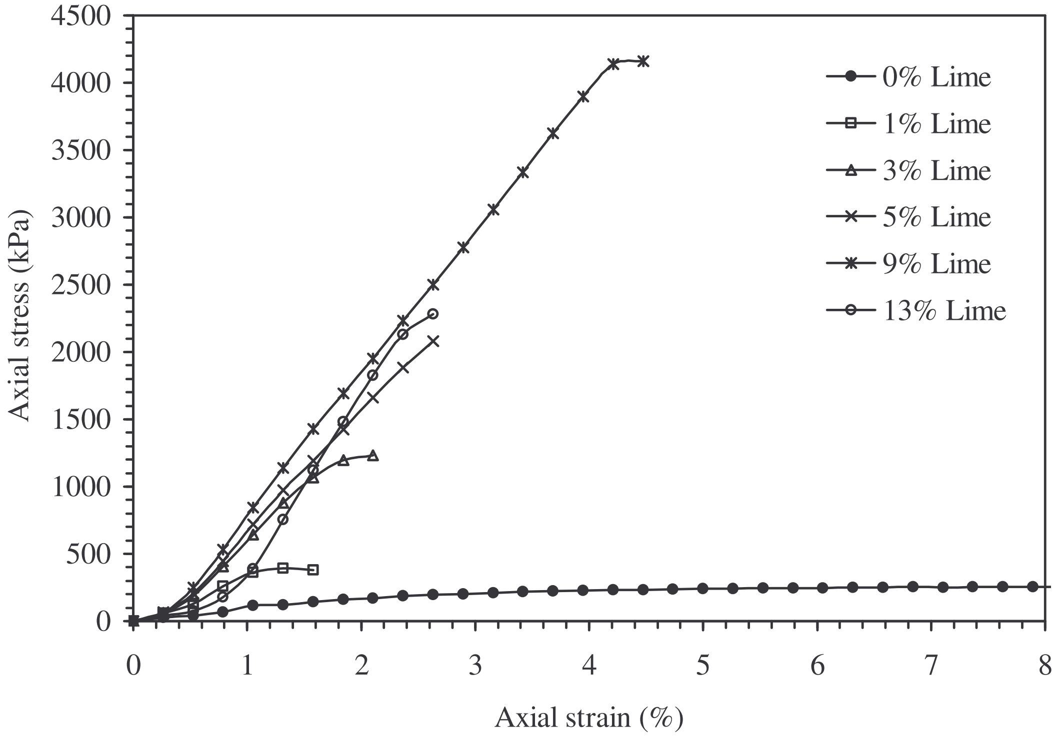

Fig. 7.31 Stress-strain responses of lime treated expansive soil-residual soil mix (60%ES+40%RS) at 28 days curing period.

Fig. 7.32 Failure characteristics of lime treated expansive soil-residual soil mix (60%ES+40%RS) at 28 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 13% Lime

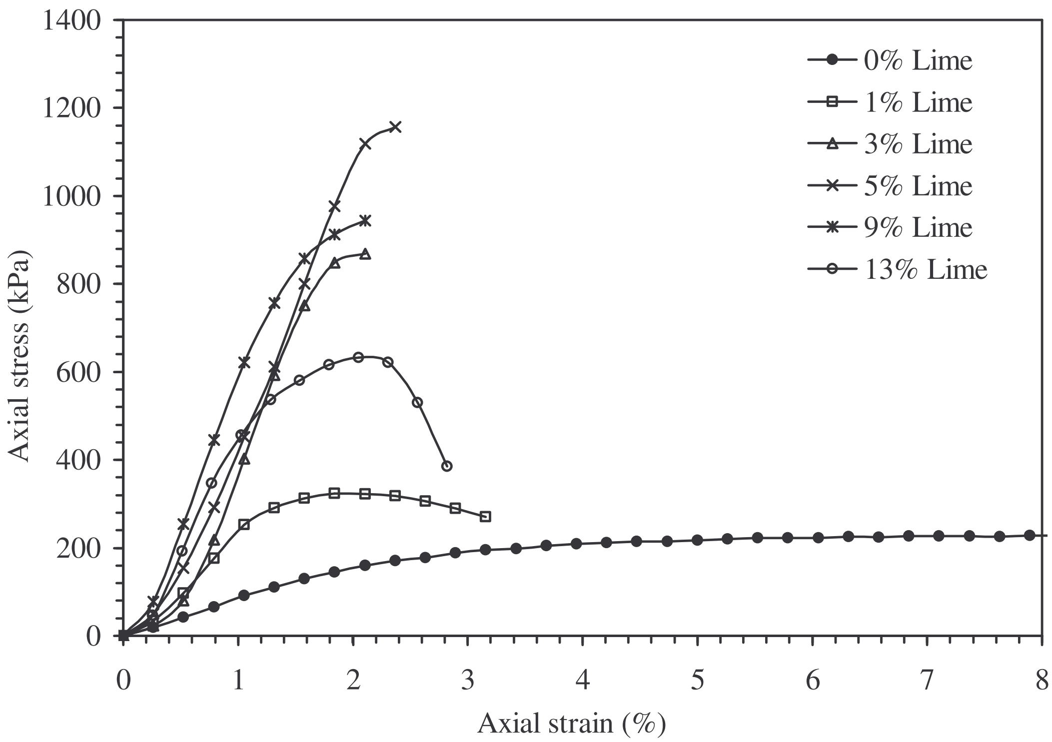

Fig. 7.33 Stress-strain responses of lime treated expansive soil-residual soil mix (40%ES+60%RS) at 3 days curing period.

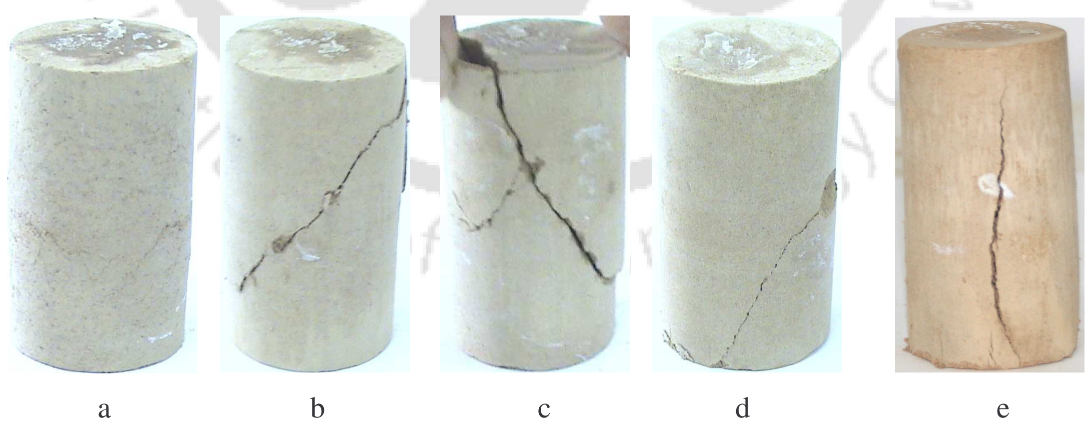

Fig. 7.34 Failure characteristics of lime treated expansive soil-residual soil mix (40%ES+60%RS) at 3 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime

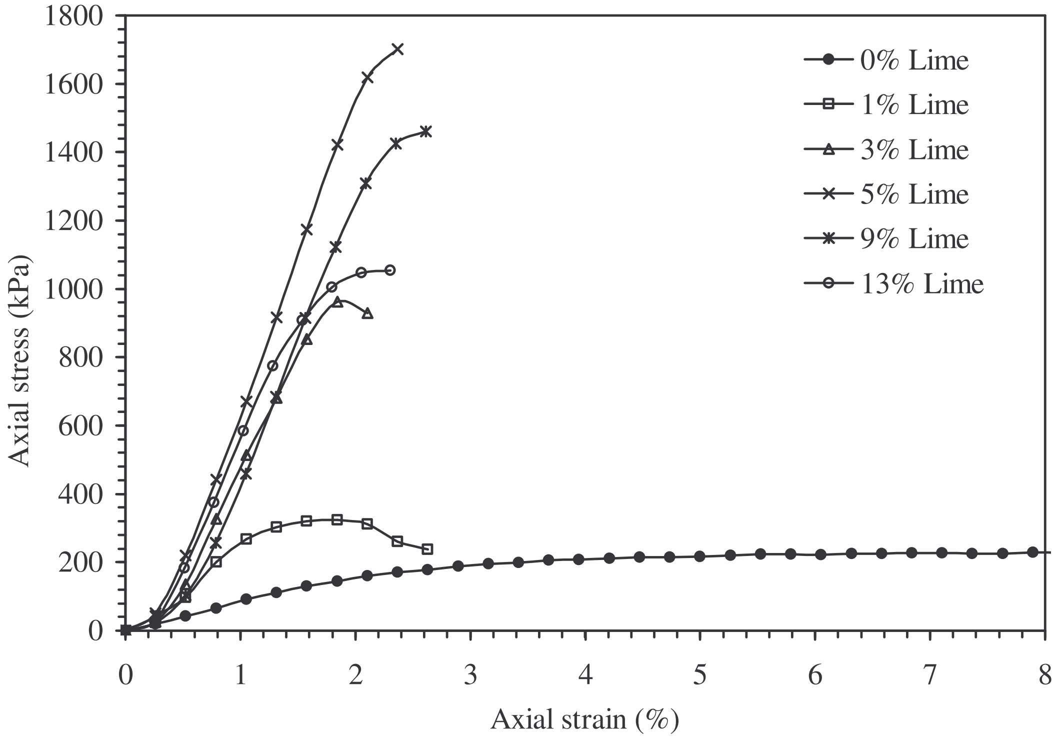

Fig. 7.35 Stress-strain responses of lime treated expansive soil-residual soil mix (40%ES+60%RS) at 7 days curing period.

a b c d e

Fig. 7.36 Failure characteristics of lime treated expansive soil-residual soil mix (40%ES+60%RS) at 7 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime

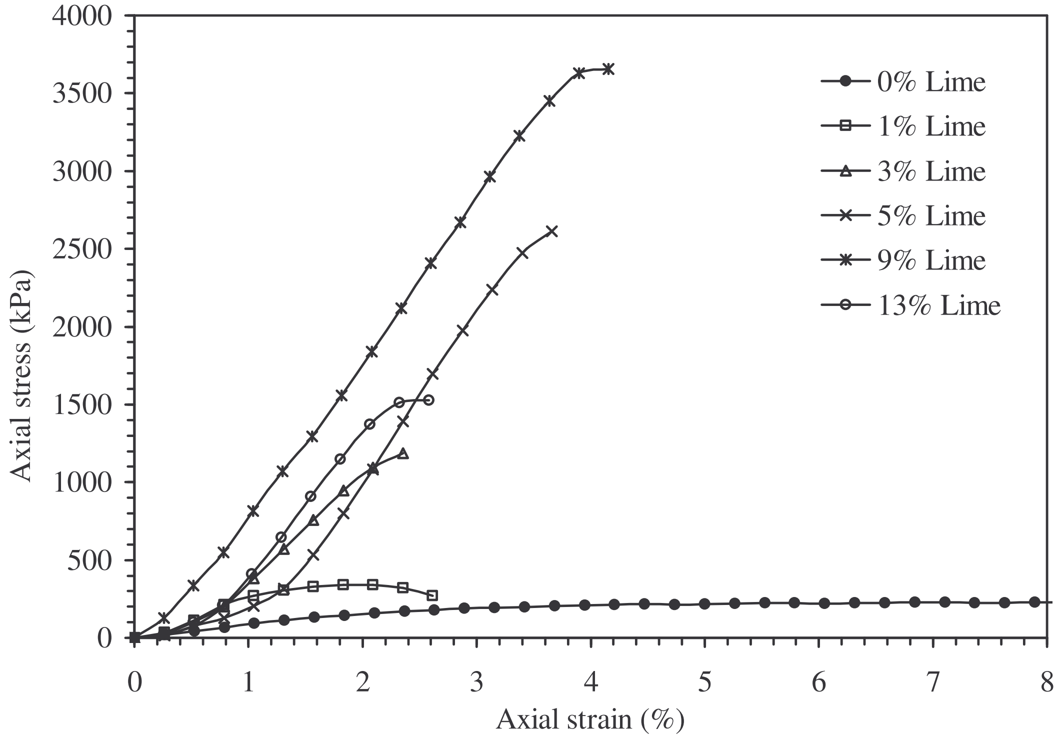

Fig. 7.37 Stress-strain responses of lime treated expansive soil-residual soil mix (40%ES+60%RS) at 21 days curing period.

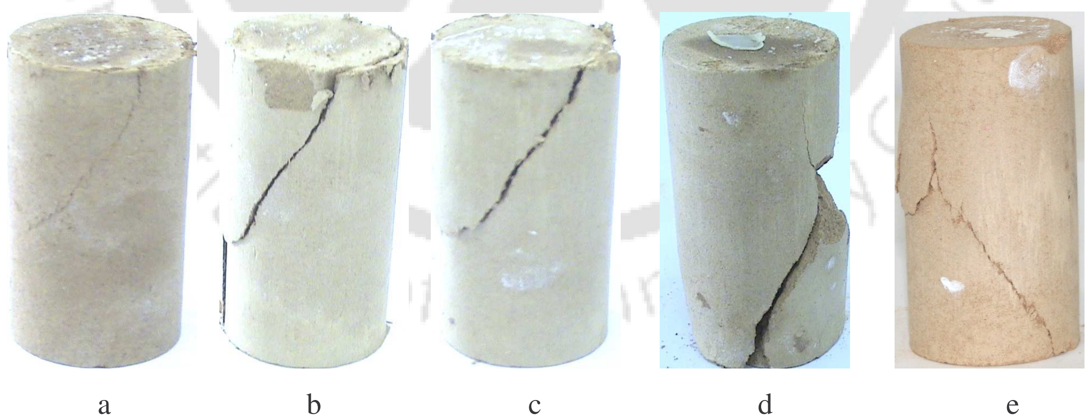

Fig. 7.38 Failure characteristics of lime treated expansive soil-residual soil mix (40%ES+60%RS) at 21 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime

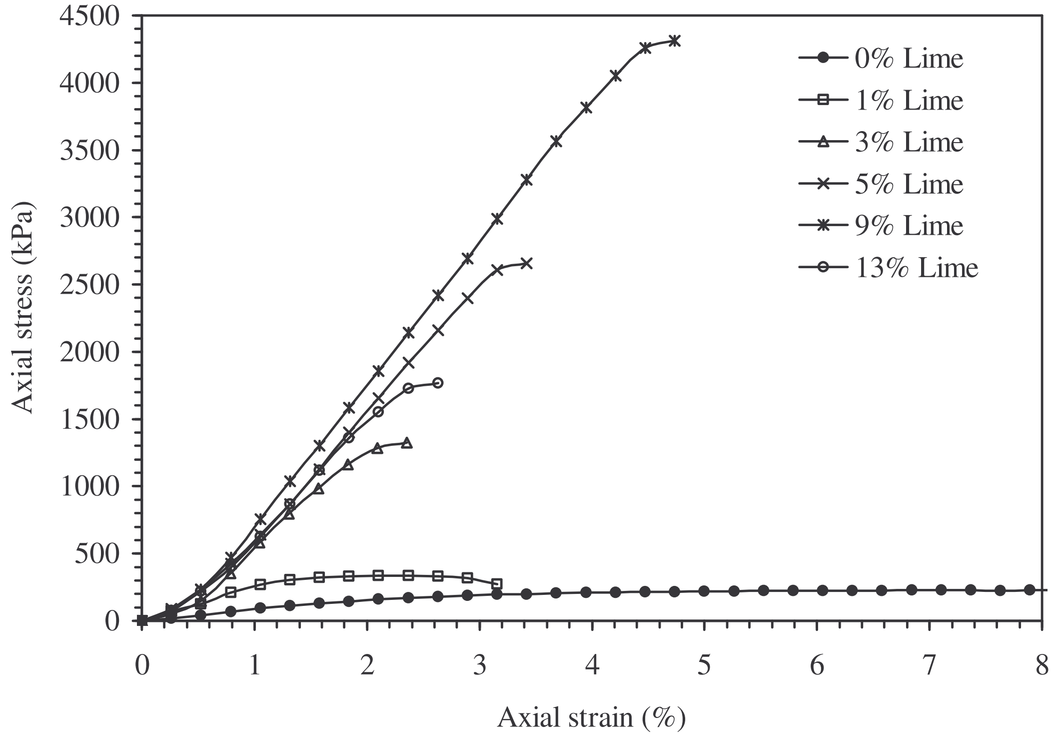

Fig. 7.39 Stress-strain responses of lime treated expansive soil-residual soil mix (40%ES+60%RS) at 28 days curing period.

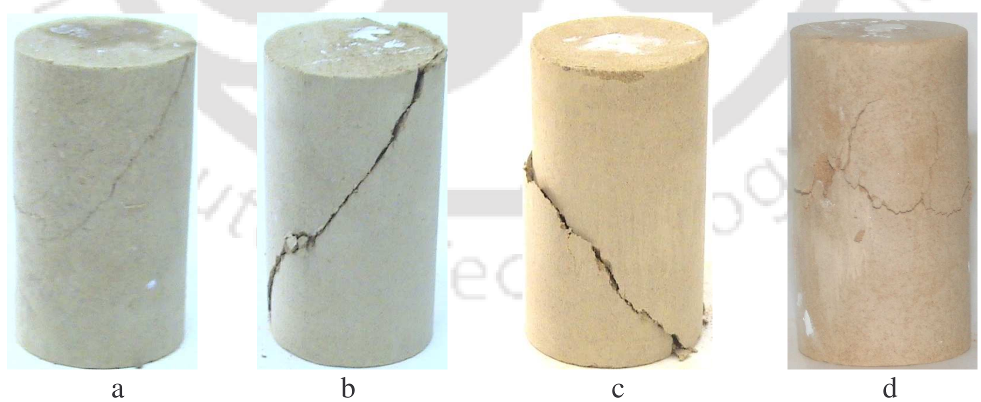

Fig. 7.40 Failure characteristics of lime treated expansive soil-residual soil mix (40%ES+60%RS) at 28 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 13% Lime

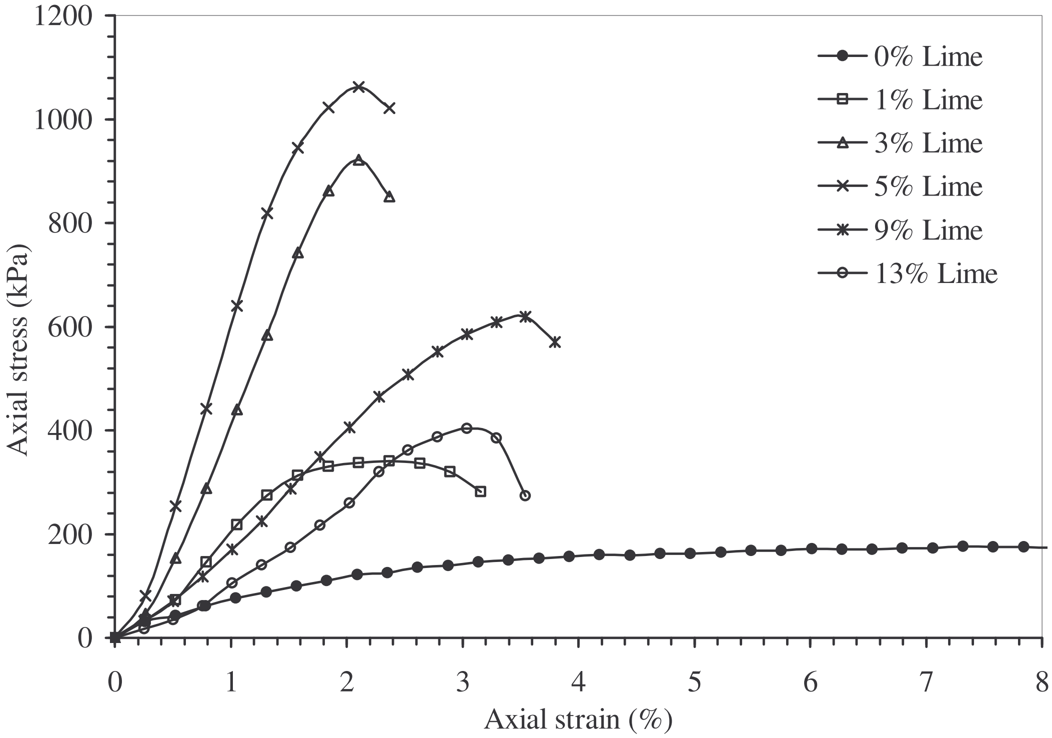

Fig. 7.41 Stress-strain responses of lime treated expansive soil-residual soil mix (20%ES+80%RS) at 3 days curing period.

a b c d e

Fig. 7.42 Failure characteristics of lime treated expansive soil-residual soil mix (20%ES+80%RS) at 3 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime

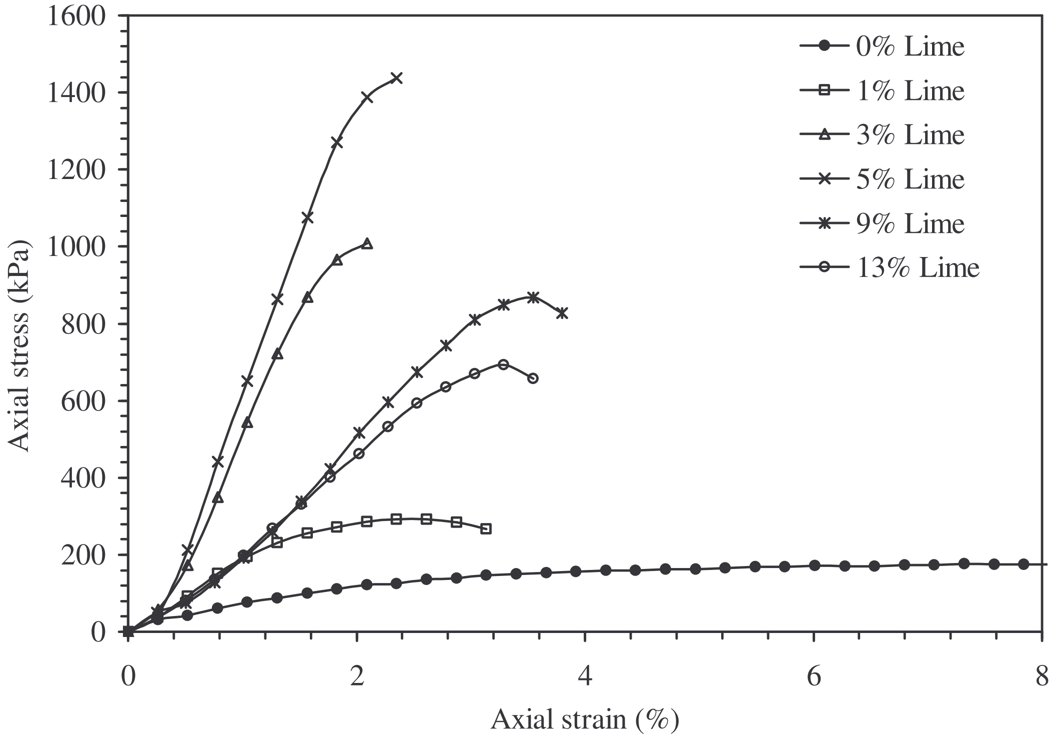

Fig. 7.43 Stress-strain responses of lime treated expansive soil-residual soil mix (20%ES+80%RS) at 7 days curing period.

a b c d e

Fig. 7.44 Failure characteristics of lime treated expansive soil-residual soil mix (20%ES+80%RS) at 7 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime

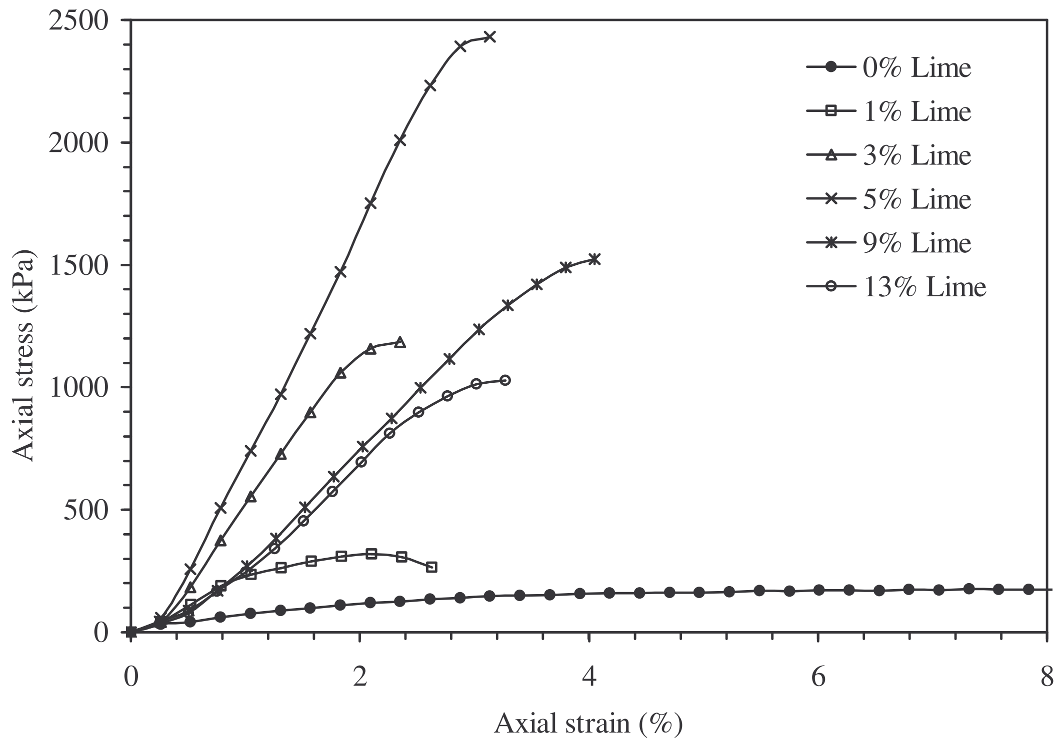

Fig. 7.45 Stress-strain responses of lime treated expansive soil-residual soil mix (20%ES+80%RS) at 21 days curing period.

a b c d e

Fig. 7.46 Failure characteristics of lime treated expansive soil-residual soil mix (20%ES+80%RS) at 21 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime

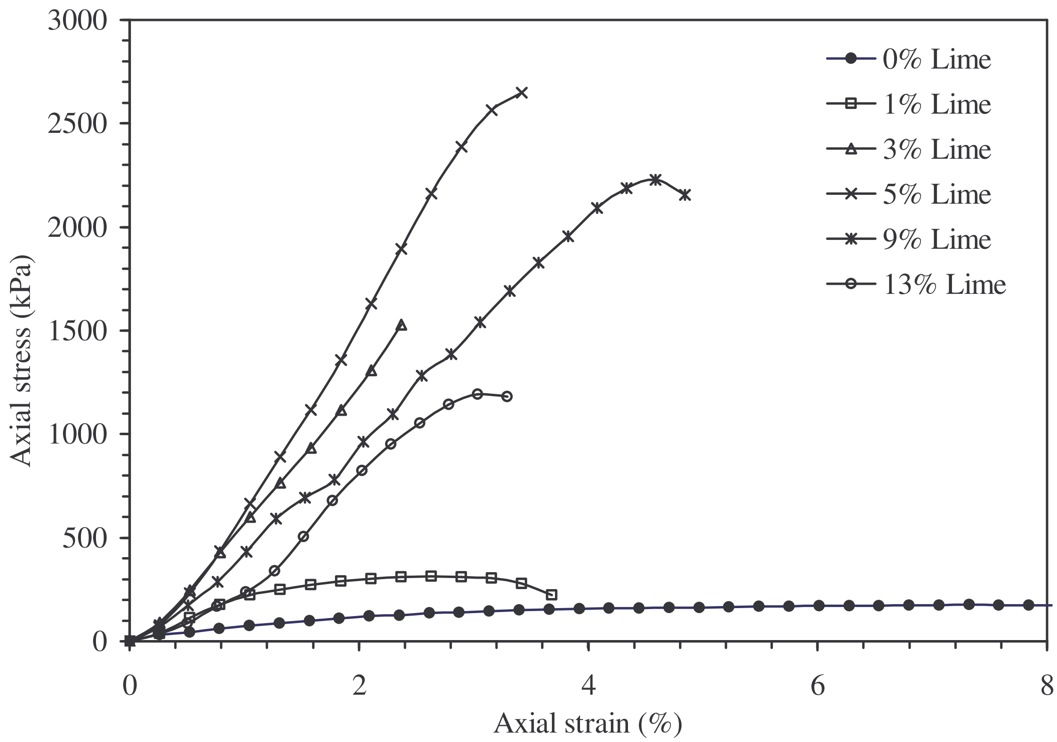

Fig. 7.47 Stress-strain responses of lime treated expansive soil-residual soil mix (20%ES+80%RS) at 28 days curing period.

a b c d e

Fig. 7.48 Failure characteristics of lime treated expansive soil-residual soil mix (20%ES+80%RS) at 28 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime

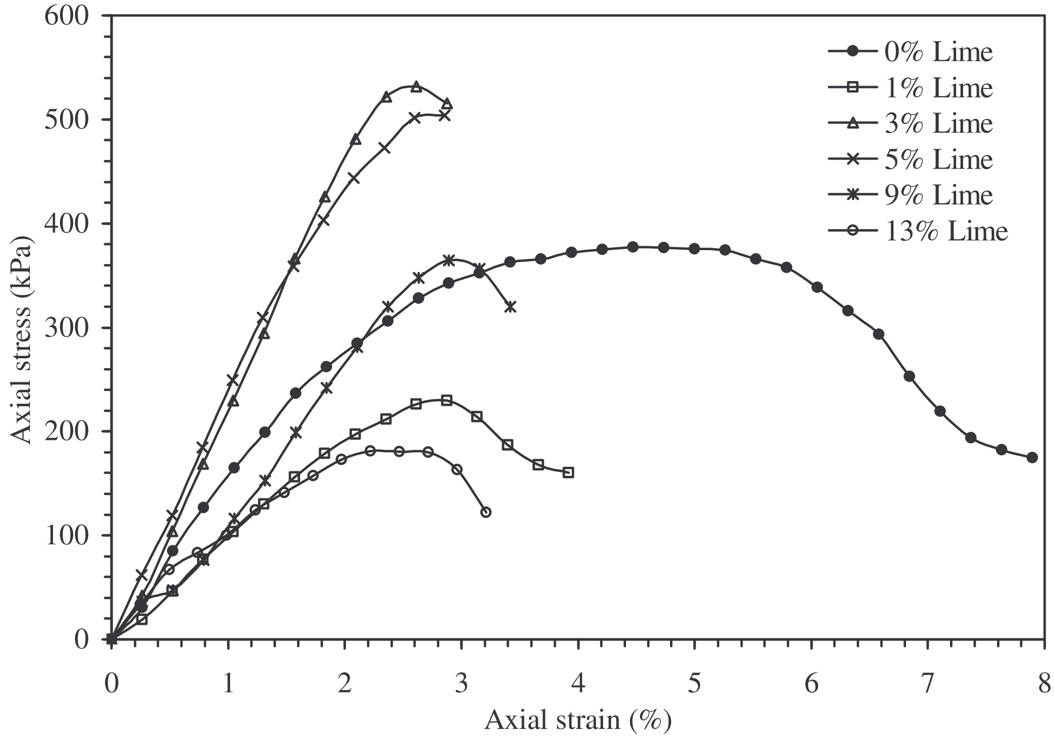

Fig. 7.49 Stress-strain responses of lime treated residual soil (100%RS) at 3 days curing period.

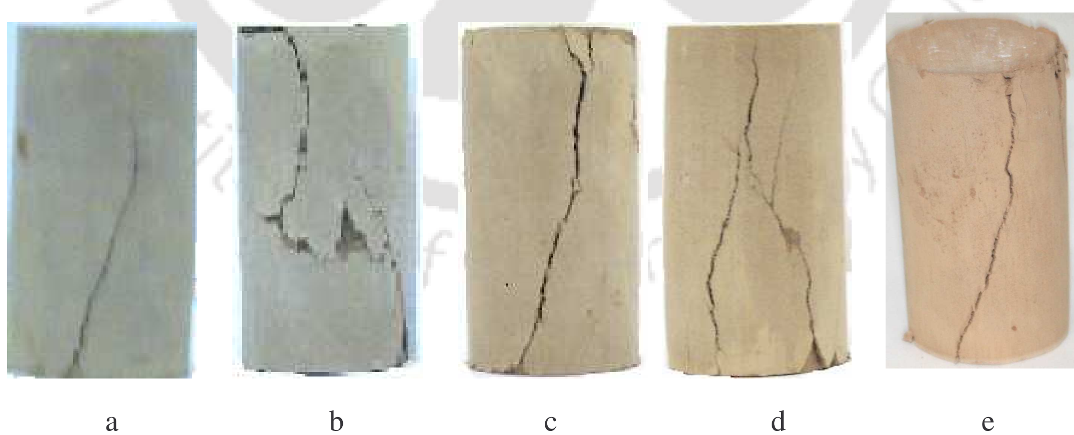

Fig. 7.50 Failure characteristics of lime treated 100%RS at 3 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime

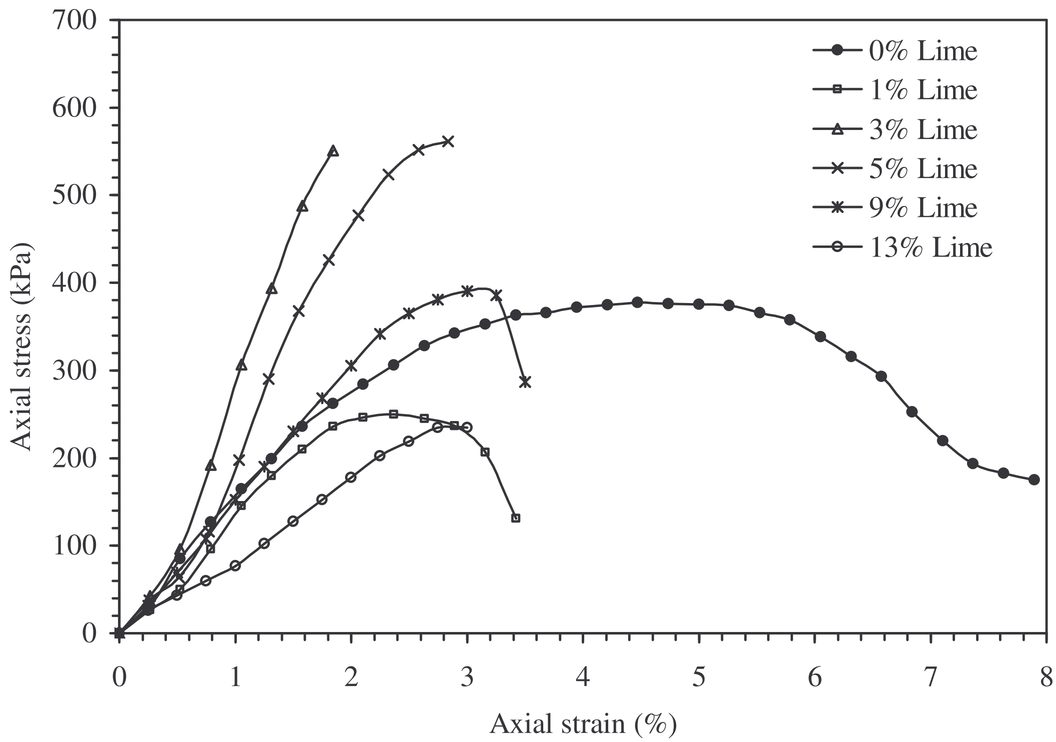

Fig. 7.51 Stress-strain responses of lime treated residual soil (100%RS) at 7 days curing period.

a b c d

Fig. 7.52 Failure characteristics of lime treated 100%RS at 7 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime

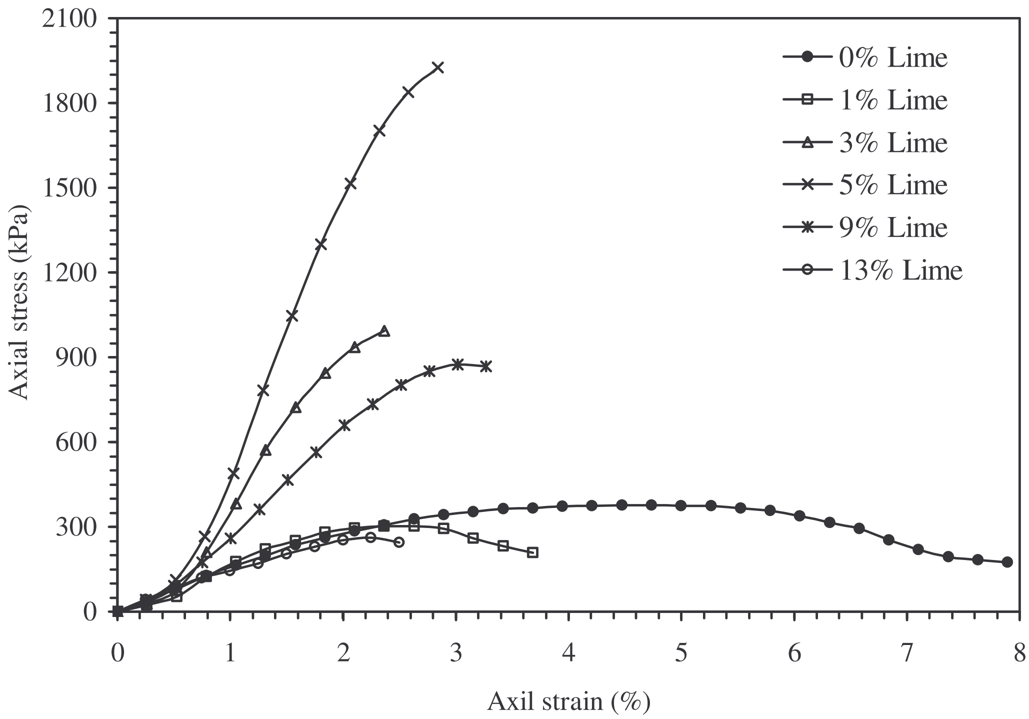

Fig. 7.53 Stress-strain responses of lime treated residual soil 100%RS at 21 days curing period.

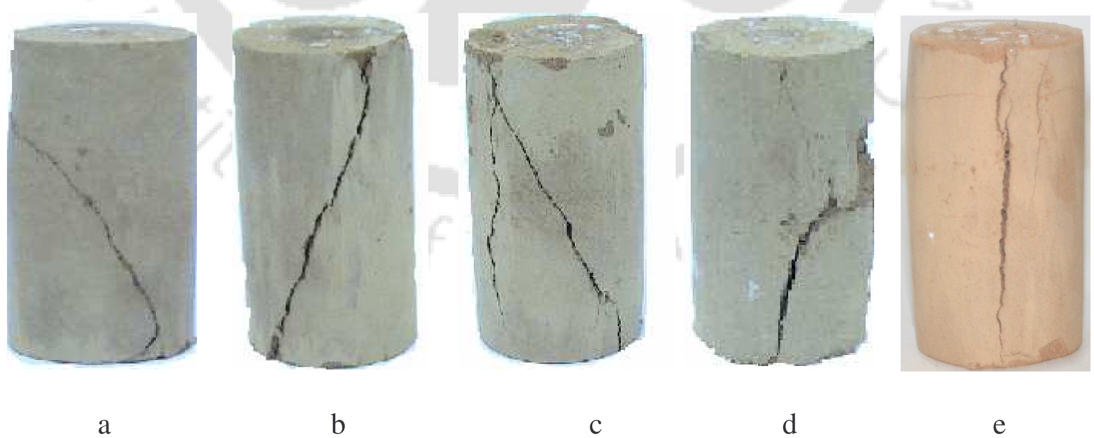

Fig. 7.54 Failure characteristics of lime treated 100%RS at 21 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime

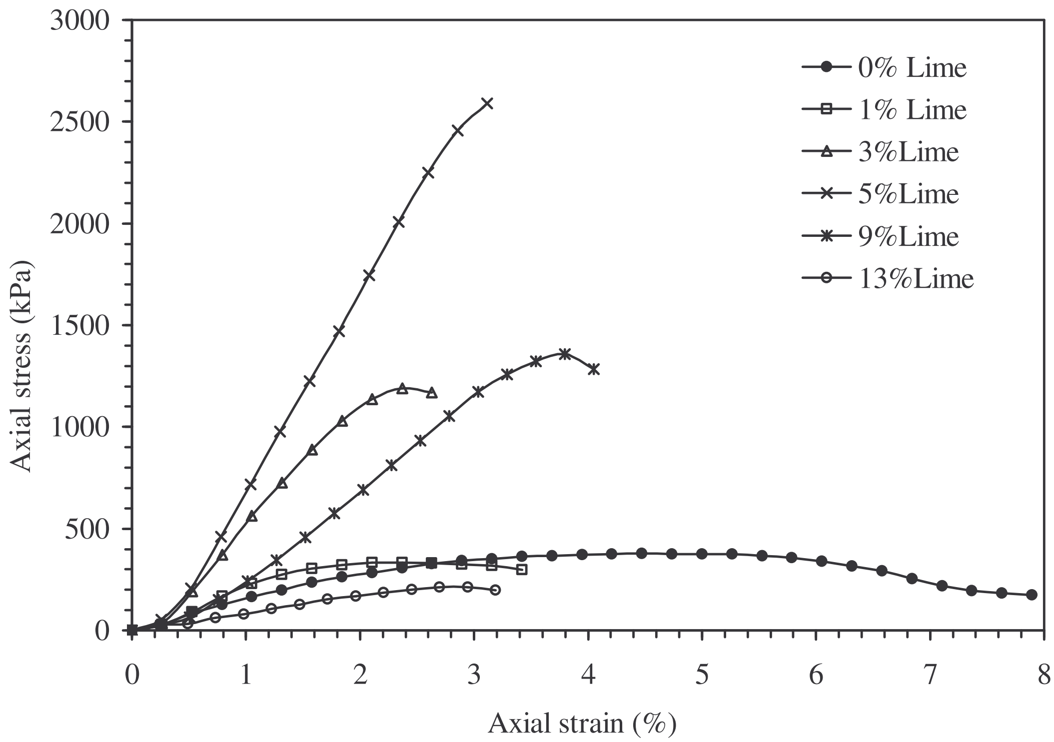

Fig. 7.55 Stress-strain responses of lime treated residual soil 100%RS at 28 days curing period.

a b c d e

Fig. 7.56 Failure characteristics of lime treated 100%RS at 28 days curing period (a) 1% Lime (b) 3% Lime (c) 5% Lime (d) 9% Lime (e) 13% Lime

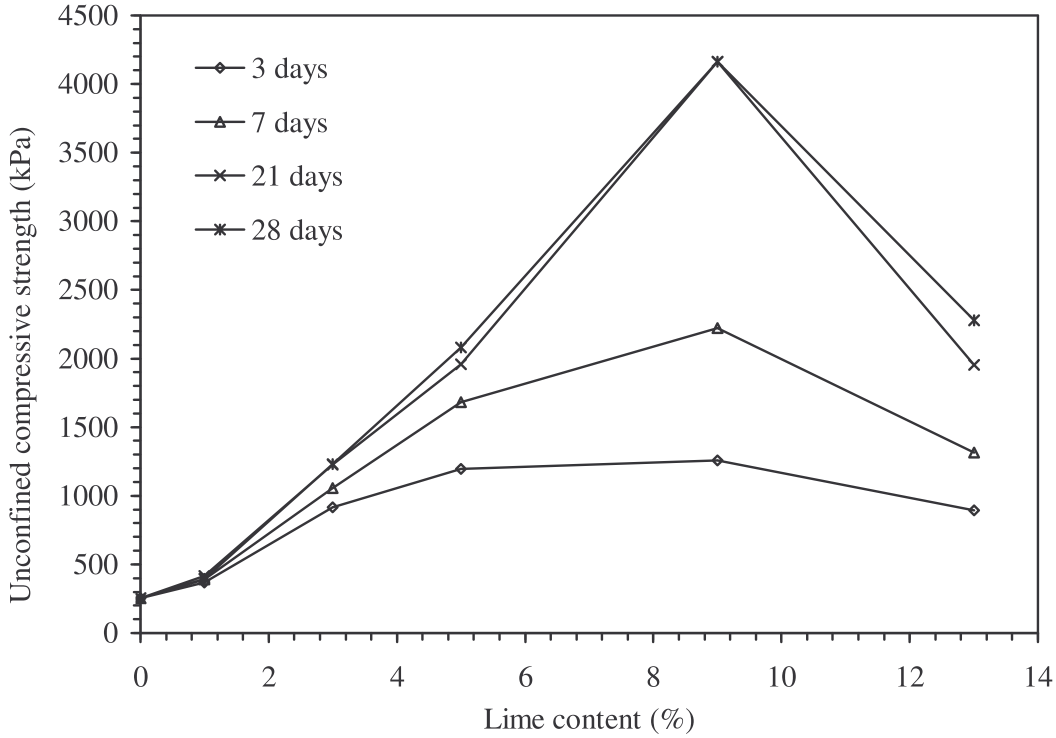

Fig. 7.57 Unconfined compressive strength vs. lime content for expansive soil-residual soil mix (60%ES+40%RS).

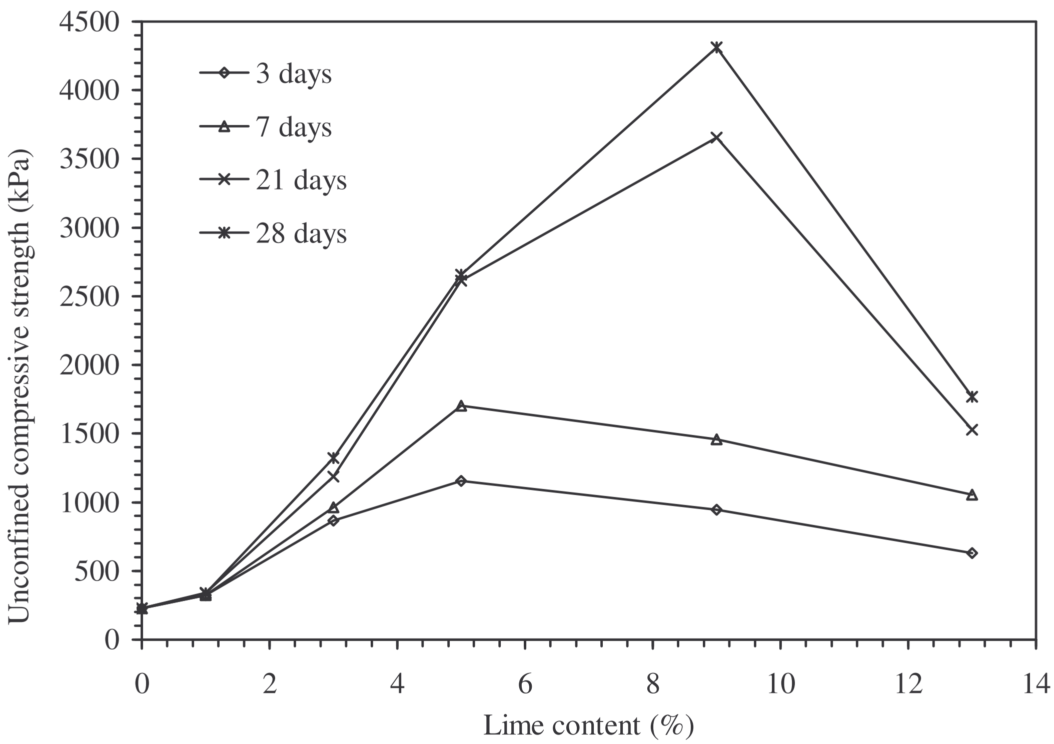

Fig. 7.58 Unconfined compressive strength vs. lime content for expansive soil-residual soil mix (40%ES+60%RS).

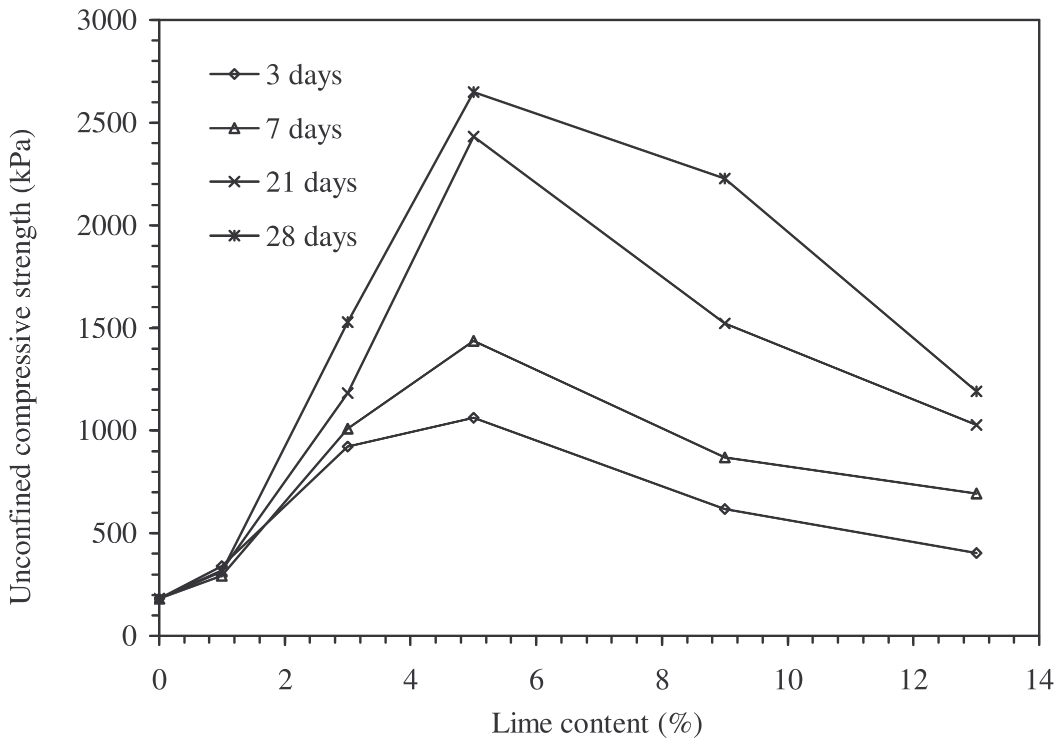

Fig. 7.59 Unconfined compressive strength vs. lime content for expansive soil-residual soil mix (20%ES+80%RS).

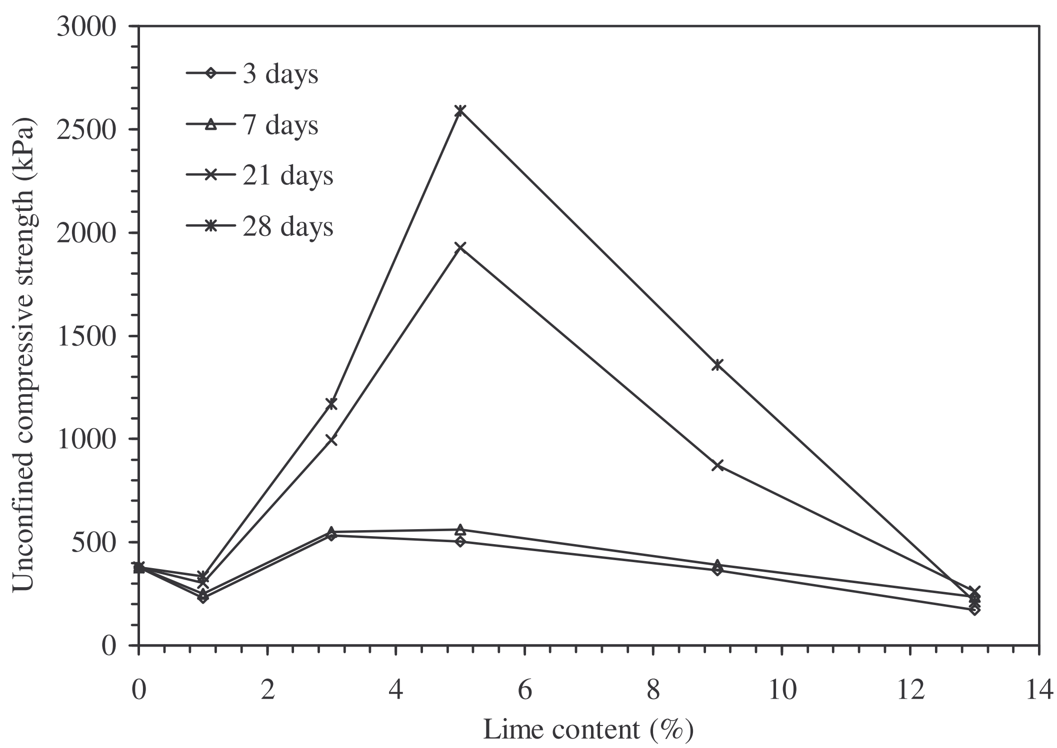

Fig. 7.60 Unconfined compressive strength vs. lime content for 100%RS.

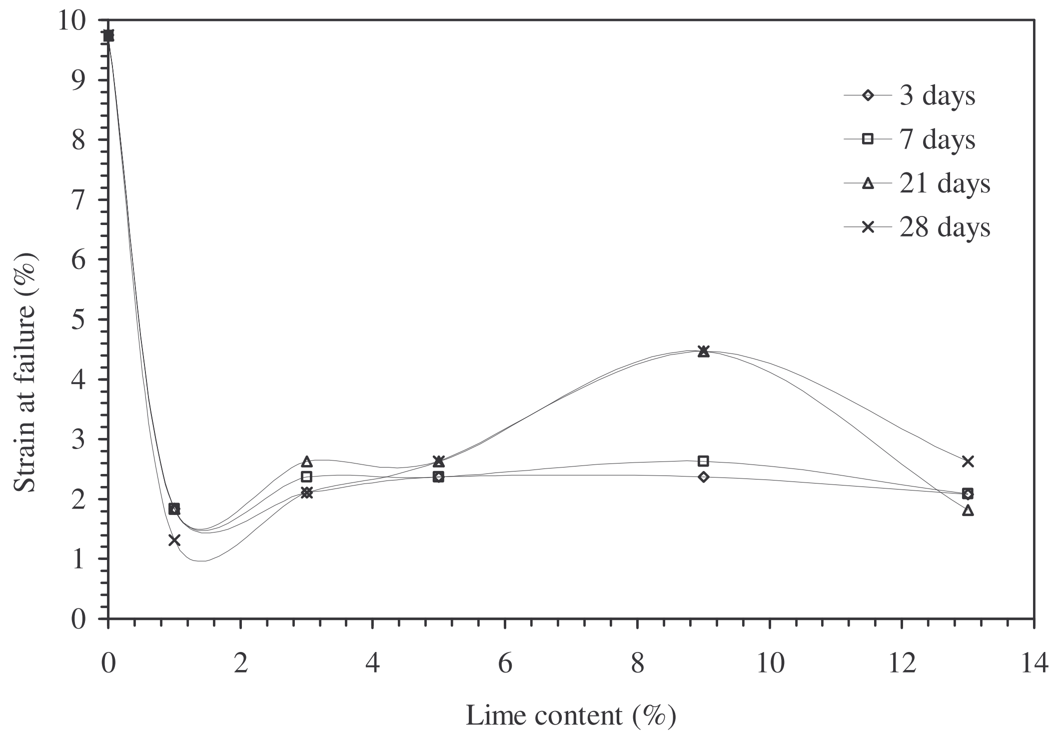

Fig. 7.61 Failure strain vs. lime content for expansive soil-residual soil mix (60%ES+40%RS).

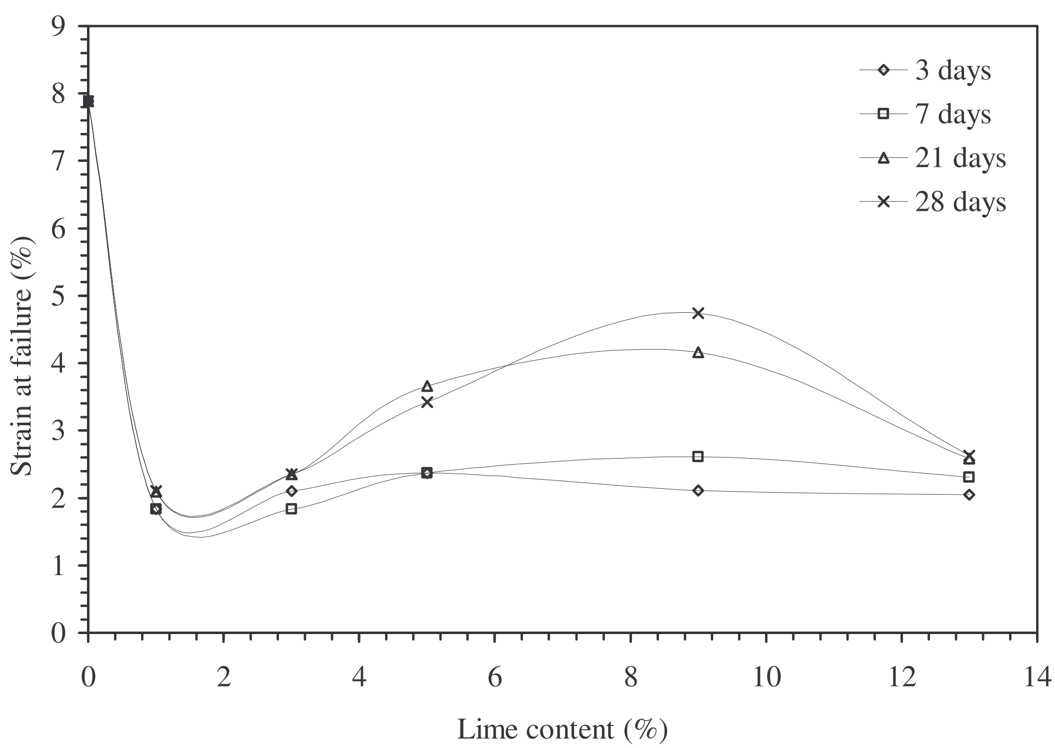

Fig. 7.62 Failure strain vs. lime content for expansive soil-residual soil mix (40%ES+60%RS).

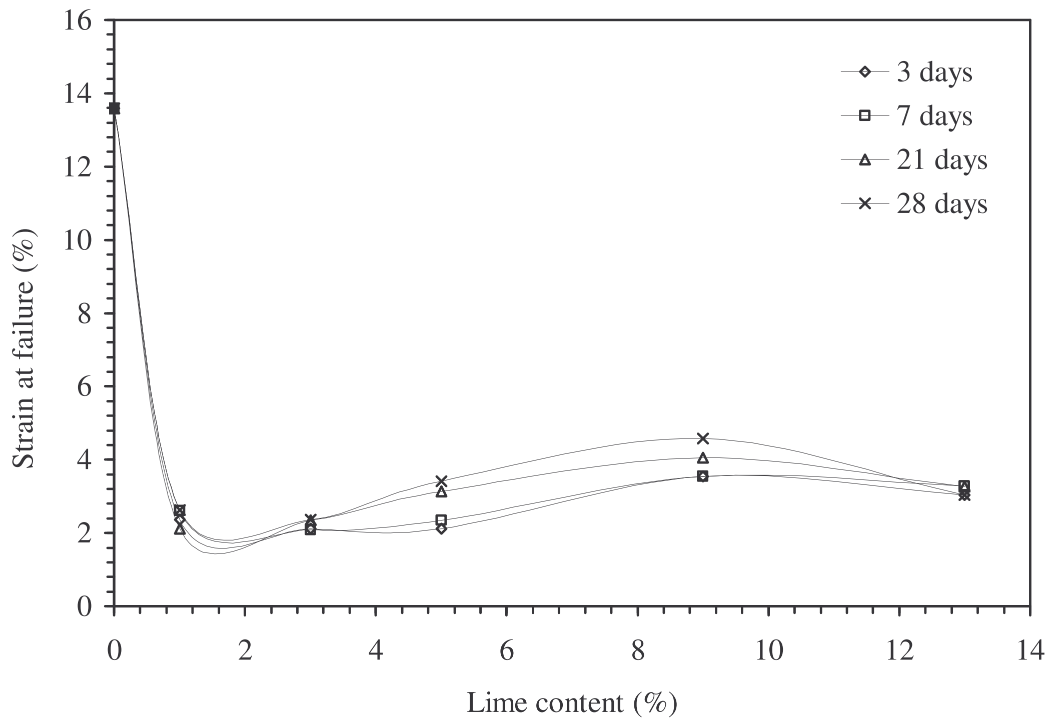

Fig. 7.63 Failure strain vs. lime content for expansive soil-residual soil mix (20%ES+80%RS).

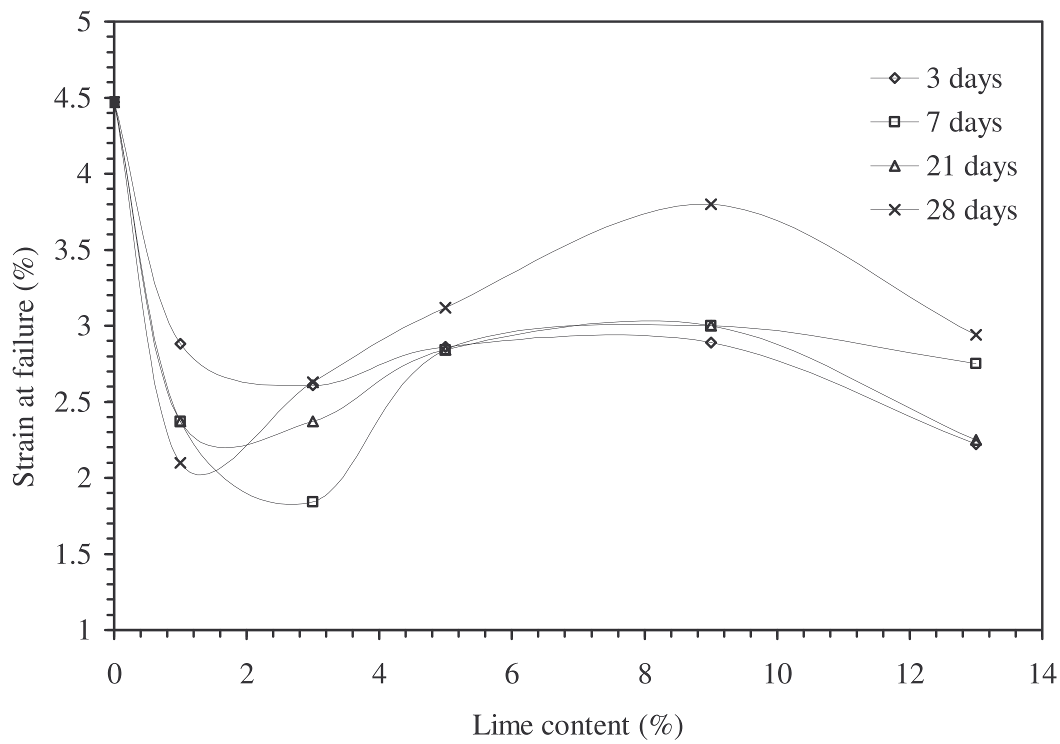

Fig. 7.64 Failure strain vs. lime content for residual soil (100%RS).

The optimum lime content giving maximum strength at varying curing periods for different soils are summarized in Table 7.1. It could be observed that the optimum lime content is 9% for expansive soil dominant samples (i.e. 100%ES, 80%ES+20%RS, 60%ES+40%RS) while with residual soil rich specimens (i.e. 20%ES+80%RS, 100%RS) it has reduced to 5%. The expansive soil dominants having higher clay frictions need higher content of lime for changing the cation exchange capacity and diffuse double layer characteristics leading to flocculation. It is only after this initial requirement is met then excess lime is available for pozzolanic reaction. Therefore such soils are in need of more lime for strength gain. On the other hand the residual soil rich specimens having low clay content need less quantity of lime for their initial requirement that relatively large amount of lime gets available for pozzolanic reaction. As a result of which maximum strength gain is achieved at a relatively lesser percentage of lime. Besides with relatively coarse particle structure the cementation bonding is more effective in residual soils leading to effective strength gain with relatively less lime content.

Table 7.1 Optimum lime content for different soils at varying curing period

Soil	Optimum lime content (%)			
	3 days	7 days	21 days	28 days
100%ES	9	9	9	9
80%ES+20%RS	9	9	9	9
60%ES+40%RS	9	9	9	9
40%ES+60%RS	5	5	9	9
20%ES+80%RS	5	5	5	5
100%RS	3	3	5	5

A comparative analysis of the data presented in Fig. 7.65-7.68 shows that at 3 days, 7 days and 21 days curing periods, maximum strength is observed in case of 40% residual soil with 9% lime. In case of 28 days curing period there is marginal strength gain when the residual soil content has increased from 40% to 60%. Hence it can be said that the soil mix having 60% expansive soil and 40% residual soil added with lime of 9% of total weight can give maximum improvement in strength. It is attributed to the reason that in this combination of both the soils the fine particles are effectively packed in voids formed by the coarser particles giving rise to a compact structure that provides increased resistance against compression. Besides in a compact structure as there is less voids the lime induced cementation effectively bonds the soil particles together giving rise to an enhanced strength gain.

Typical variation of unconfined compressive strength with dry density for lime treated soils depicted in Fig. 7.69 and Fig. 7.70 indicate that the two parameters do not show any well defined correlation between them. This observation establishes that it is primarily the pozzolanic reaction that controls the strength of lime treated soils.

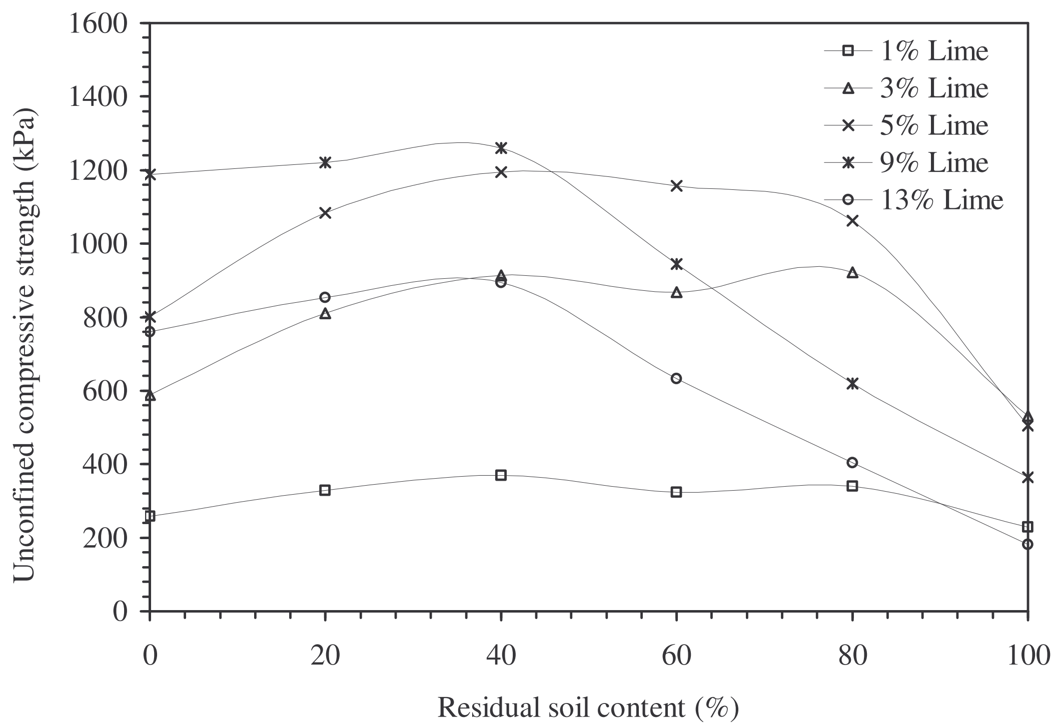

Fig. 7.65 Variation of unconfined compressive strength with residual soil content for different percent of lime at 3 days curing period

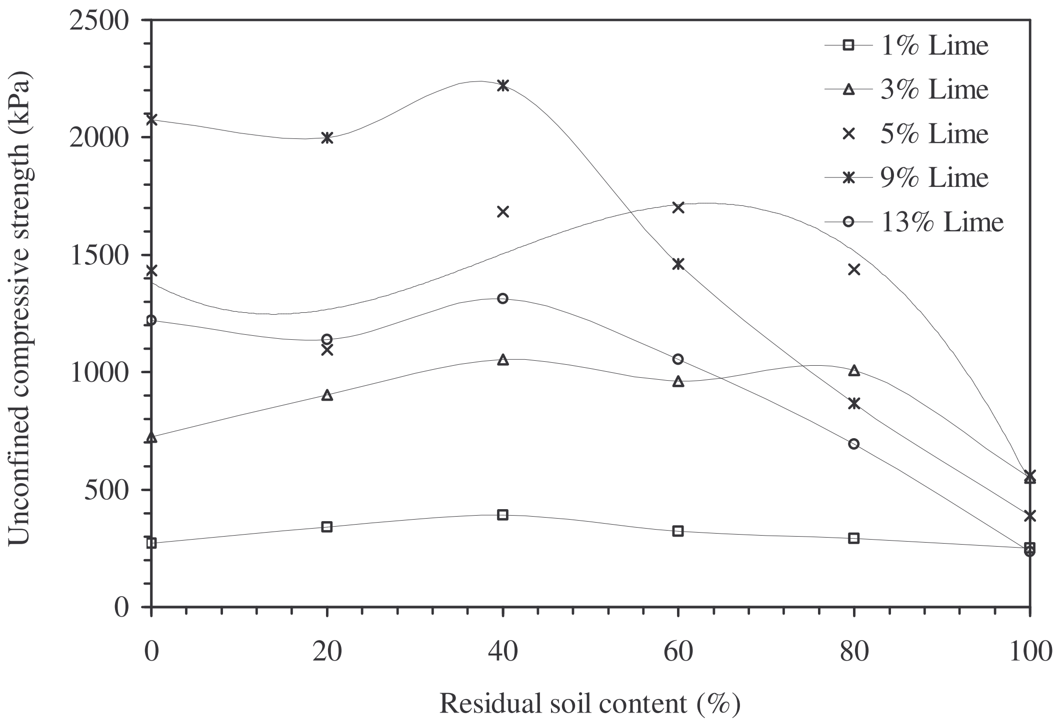

Fig. 7.66 Variation of unconfined compressive strength with residual soil content for different percent of lime at 7 days curing period

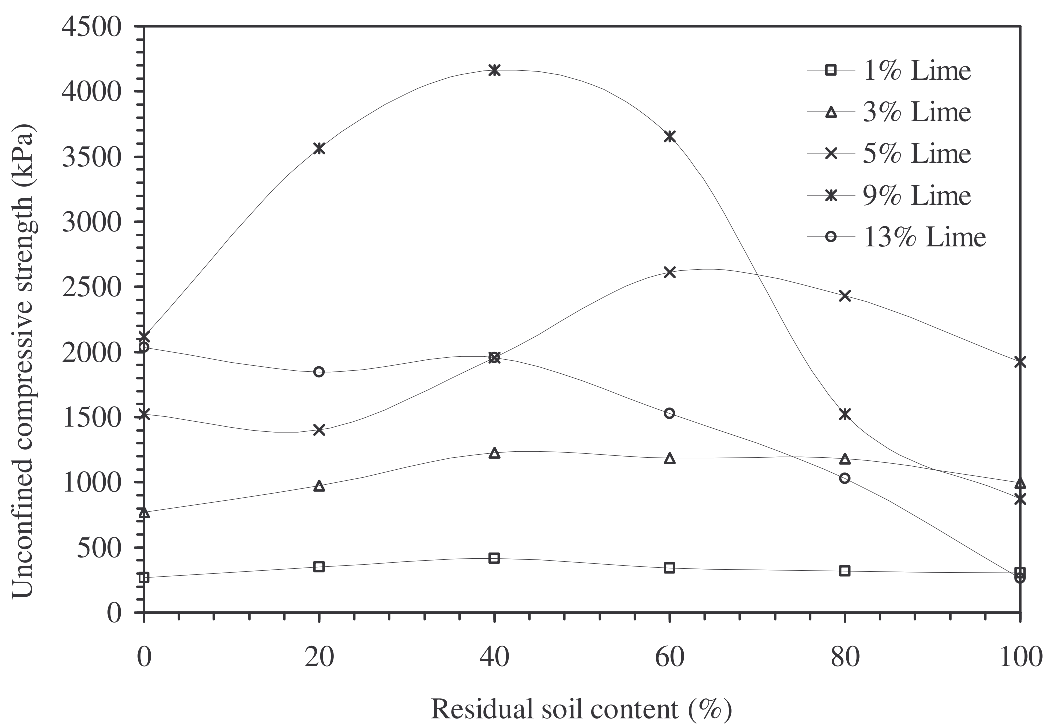

Fig. 7.67 Variation of unconfined compressive strength with residual soil content for different percent of lime at 21 days curing period

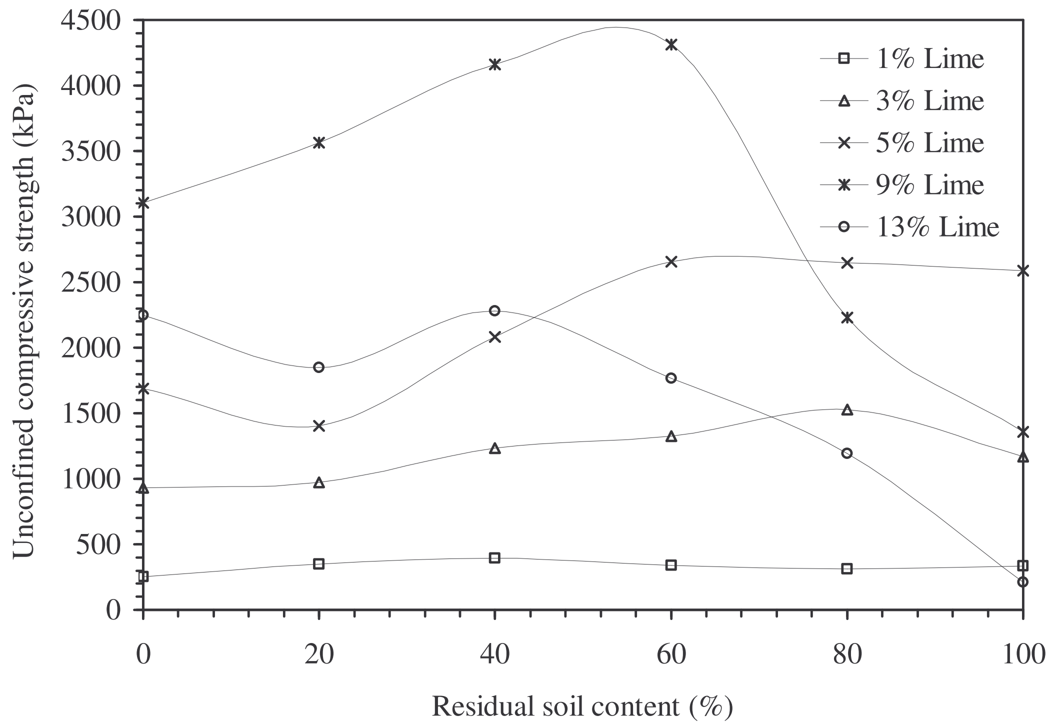

Fig. 7.68 Variation of unconfined compressive strength with residual soil content for different percent of lime at 28 days curing period.

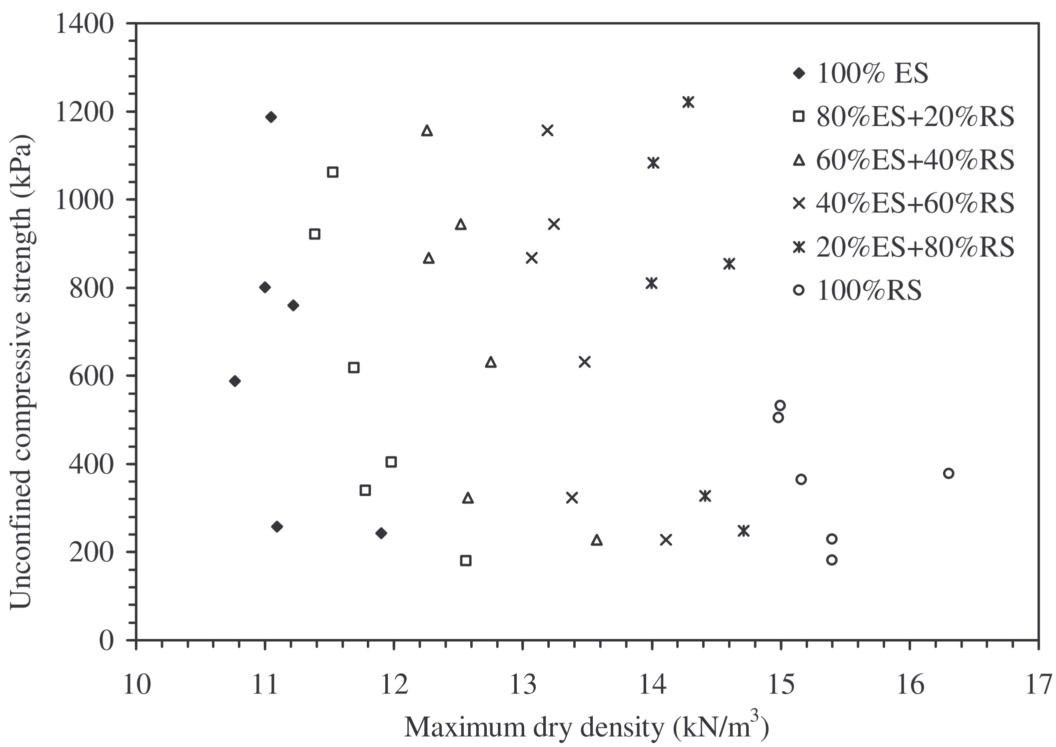

Fig. 7.69 Variation of unconfined compressive strength with maximum dry density for lime treated soils at 3 days curing period

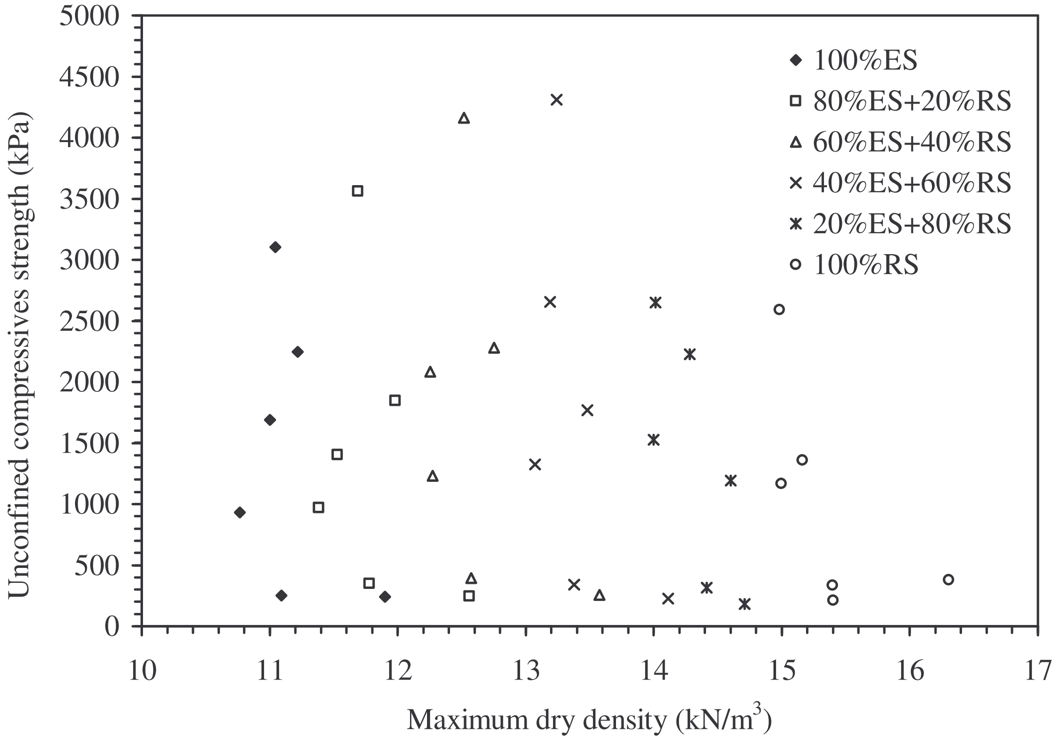

Fig. 7.70 Variation of unconfined compressive strength with maximum dry density for lime treated soils at 28 days curing period.

7.4 SUMMARY

This chapter has brought out the influence of residual soil and lime in improving the strength and stiffness of expansive soils. A series of unconfined compression tests have bean carried out by varying the residual soil content, lime content and curing period. The test results indicate that there is an optimum combination of expansive soil, residual soil and lime that gives maximum strength gain. The failure patterns indicate that the soil that has under gone bulging initially, with lime has shown inclined slip plane typical of a shear failure. With further increase in lime content the soil specimen behaves like a column marked by deep vertical cracks.

CHAPTER 8

MICROSTRUCTURAL BEHAVIOUR

8.1 INTRODUCTION

The behaviour of a soil is essentially the cumulative effect of its behaviour at microstructural level. Correspondingly the microstructural behaviour is the combined effect of the fabric and mineralogical composition that dictates the interparticle forces. Therefore the changes in the physical properties of the soils due to lime treatment can be attributed to the change in fabric and formation of new reaction products. In view of this a series of tests have been carried out to investigate the microstructural characteristics of the lime treated soil systerms using scanning electron micrograph (SEM) and X-ray diffraction (XRD) techniques, the details of which are presented in chapter 3 (Table 3.2, section 3.4.11). The results are presented and analysed in the following sections.

8.2 ENERGY DISPERSIVE X-RAY (EDX) AND X-RAY DIFFRACTION (XRD) ANALYSIS

The EDX and XRD pattern for the soil sample with100%ES are depicted in Fig. 8.1 and Fig. 8.2 respectively. From Fig. 8.1 it could be observed that the main elements present in 100%ES are O, Na, Mg, Al, Si, K, Ca, Ti and Fe. The atomic (%) quantity of different elements present in the soil sample is shown in Table 8.1. The EDX data shows that among all the elements present Si percentage is the maximum. From the XRD pattern shown in Fig. 8.2 it can be observed that Quartz (SiO_2) has the most prominent peak. Therefore the EDX and XRD are in agreement with each other. The presence of Na, Mg and Al in the EDX is attributed to the minerals montmorillonite [NaO_3 (Al Mg)$_2$ $SiO_{10}OH_{2}.6H_2O$] and Aluminum Oxide (Al_2O_3) as confirmed from the XRD response.

One can see from the EDX that there are other elements present in the soil. However, their percentage is very less and hence would have marginal influence on the over all behavoiur.

Fig. 8.3, 8.5, 8.7, 8.9 and 8.11 present the EDX pattern for expansive soil (100%ES) stabilized with different percent of lime (i.e. 1%, 3%, 5%, 9%, and 13%). The main elements present are O, Na, Mg, Al, Ca, Ti, Fe, K, C and Cl. The corresponding XRD pattern for these soil samples are shown in Figs. 8.4, 8.6, 8.8, 8.10 and 8.12 respectively. The atomic quantities of these elements are shown in Table 8.1. It can be observed that Si quantity is the highest in percentage. This is attributed to the presence of quartz (SiO_2) which is the dominating mineral present in the soil sample (i.e. maximum peak length in the XRD response). The presence of Ca and Al element is in conformation to the cementitious compounds i.e. gyrolite [$2Ca.3SiO_2.H_2O$], calcium silicate hydrate (CSH) [$5Ca_2SiO_4.6H_2O$], calcium aluminum silicate hydroxide hydrate (CASHH) [$Ca_5Si_5Al(OH)O_{17}.5H_2O$] present in the soil sample.

The X-ray diffractograms show that compared to the untreated soil there has appeared several new peaks with low to moderate intensity due to lime treatment, which clearly indicates that new compounds are formed due to soil-lime reaction. It is confirmed that with addition of lime to the expansive soil, there is formation of cementitious compounds like gyrolite, CSH, CAH, CASHH and CAOH. Apart from this the other compounds present are quartz and montmorillonite, which were originally present in the untreated soil. However the peaks of these elements have shown a clear reduction which establishes the action of lime on these elements.

Table 8.2 shows that the d-spacing for the cementitious compound gyrolite for 1%, 3%, 5%, 9% and 13% lime are 4.28 A^0, 4.26 A^0, 4.26A^0, 4.26 A^0 and 4.25 A^0 respectively. Similarly for compound CASHH the d-spacing are 3.06A^0, 3.05 A^0 , 3.05 A^0, 3.03 A^0, 3.04 A^0 and for CSH the d-spacing are NF, NF, 4.18 A^0, 3.24 A^0, 4.12 A^0, 4.21 A^0 respectively. The d-spacing of the cementitious compounds do not show much variation with increasing lime content.

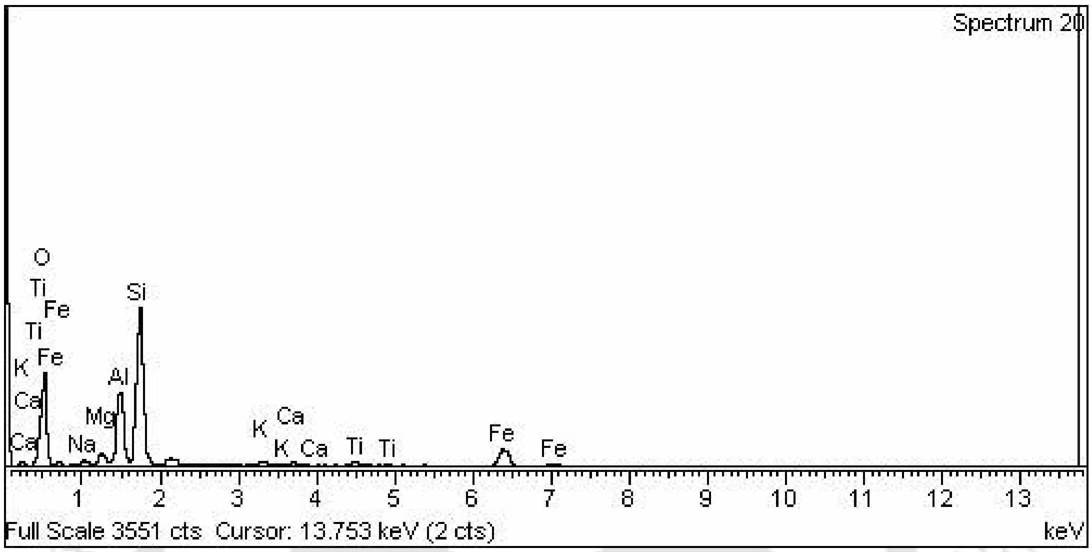

Fig. 8.1 EDX spectrum of sample: 100%ES

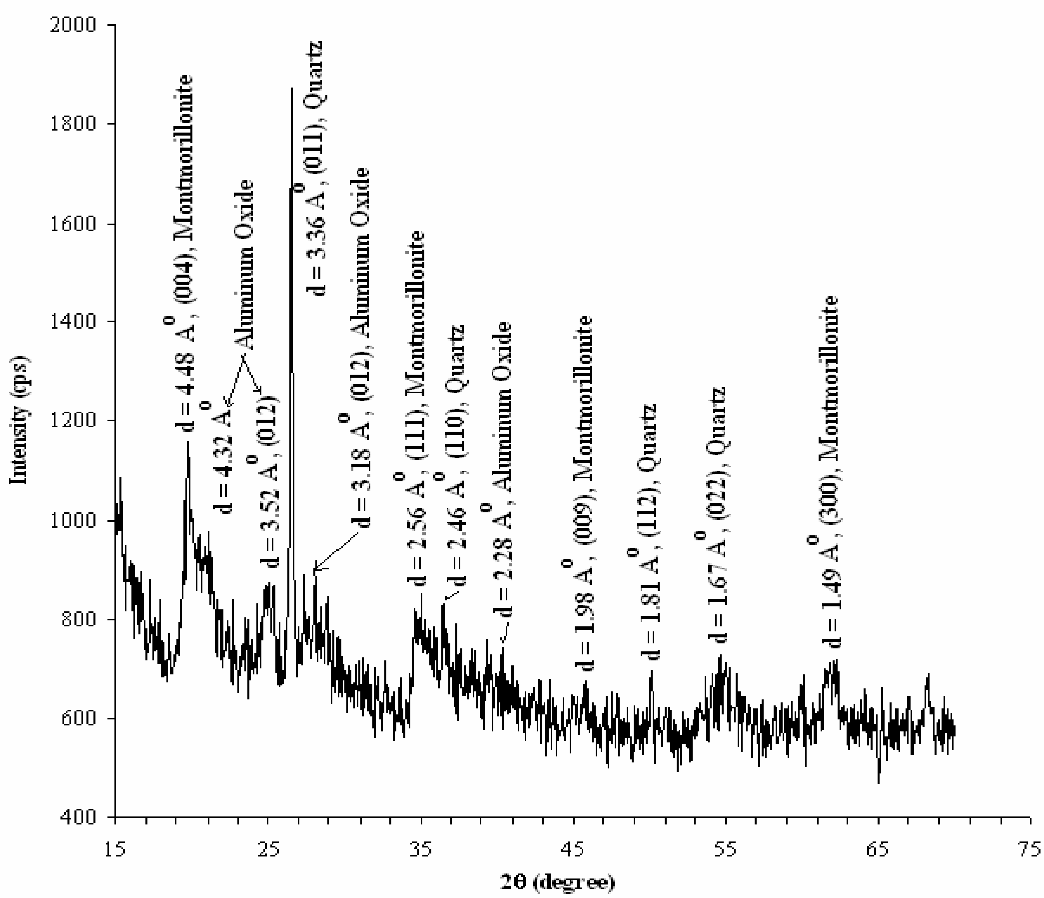

Fig. 8.2 XRD pattern of sample: 100%ES

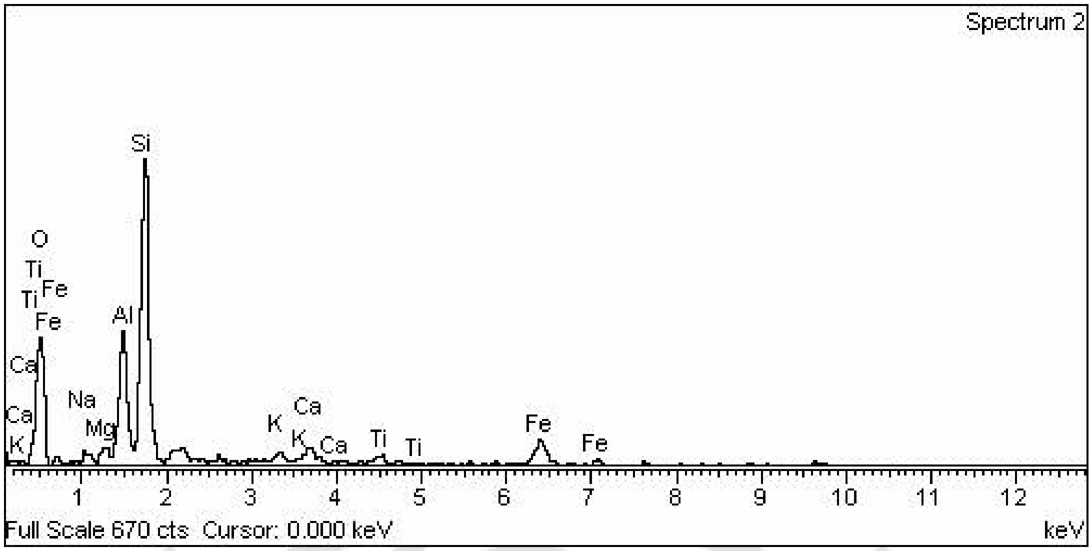

Fig. 8.3 EDX spectrum of sample: 100%ES + 1% Lime

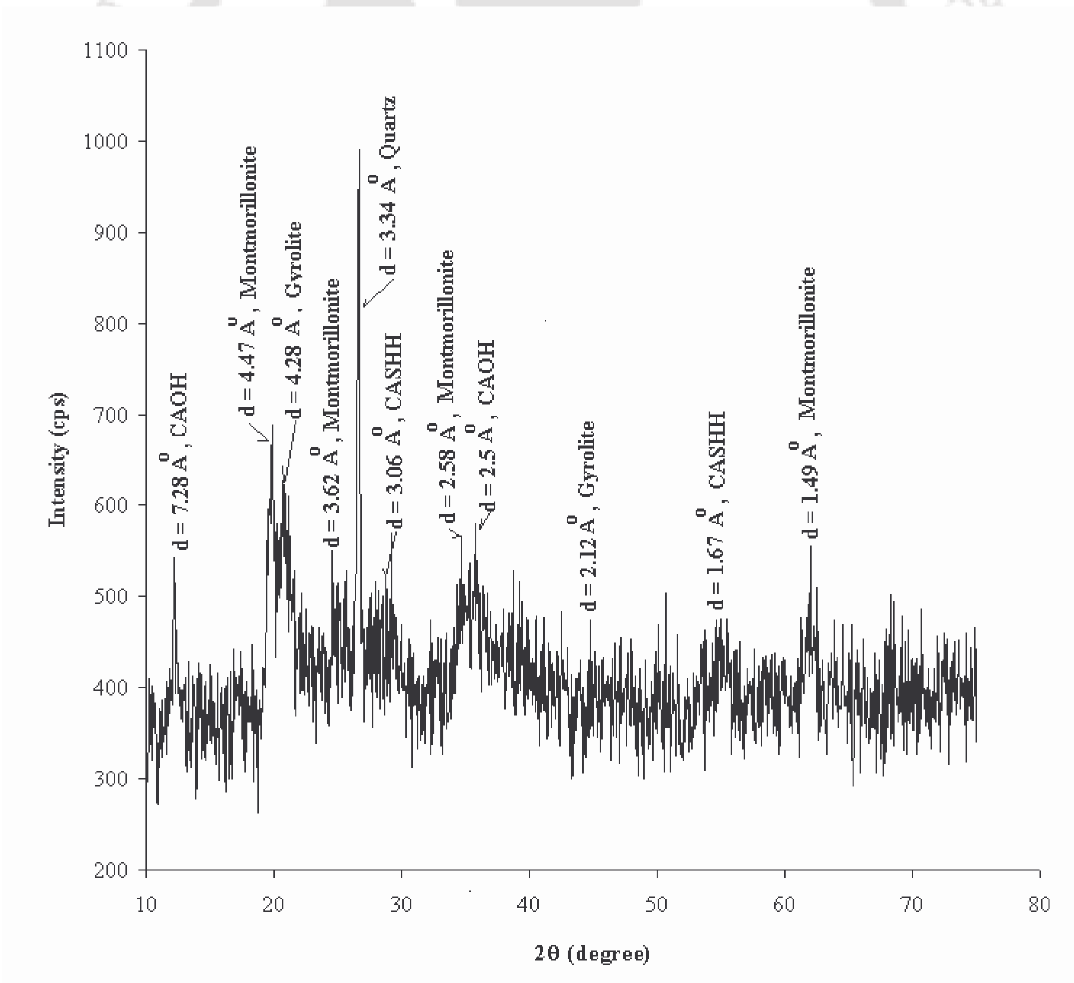

Fig. 8.4 XRD pattern of sample: 100%ES + 1% Lime

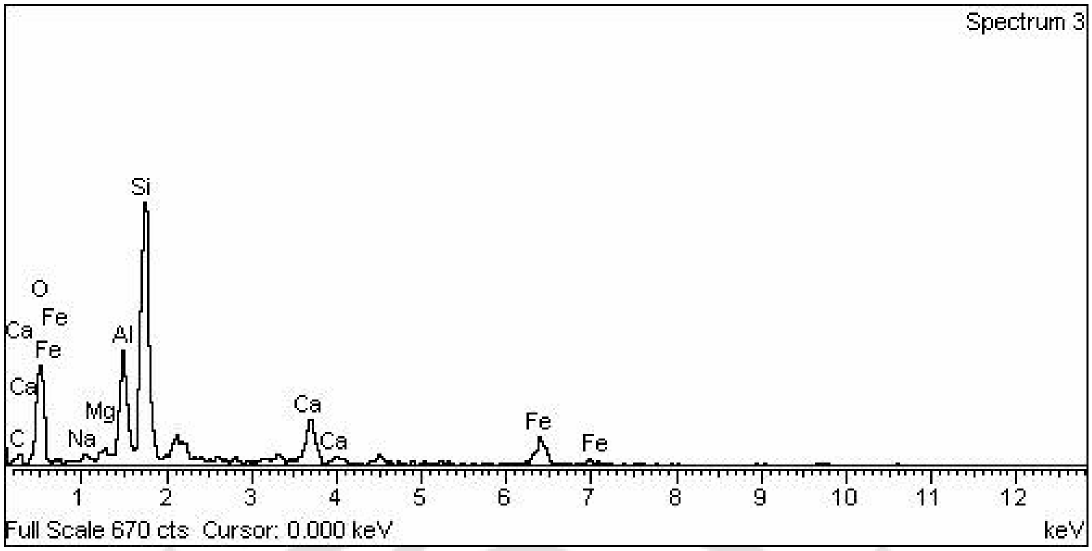

Fig. 8.5 EDX spectrum of sample: 100%ES + 3% Lime

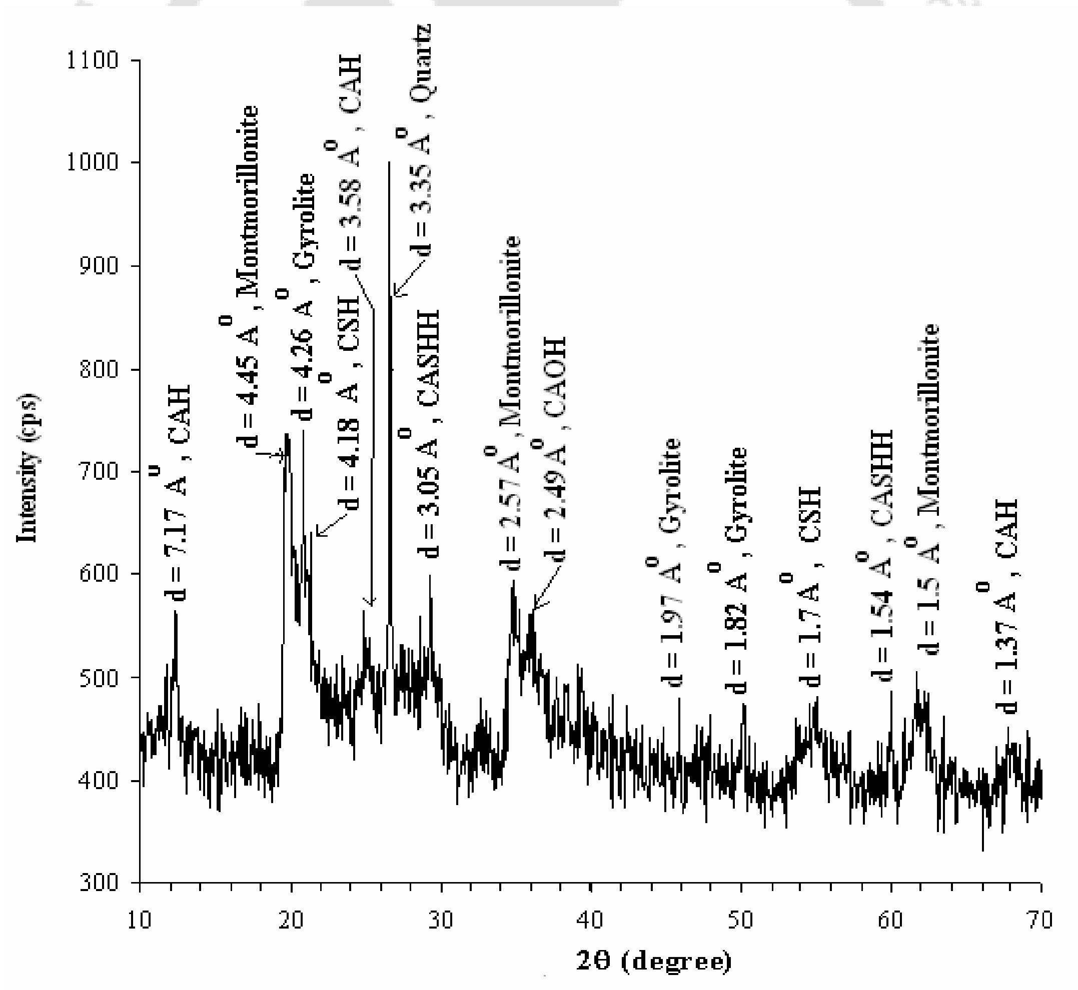

Fig. 8.6 XRD pattern of sample: 100%ES + 3% Lime

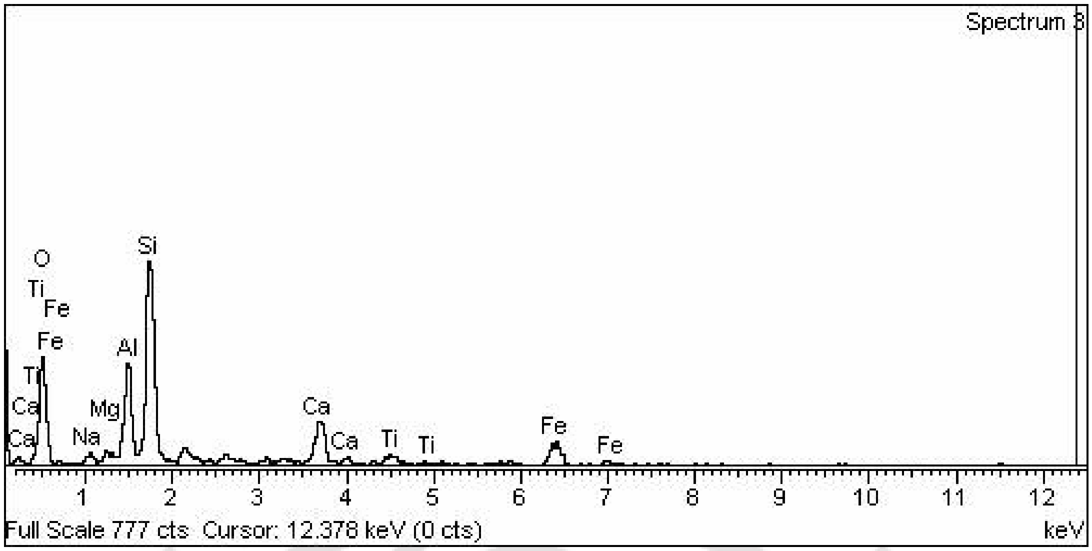

Fig. 8.7 EDX spectrum of sample: 100%ES + 5% Lime

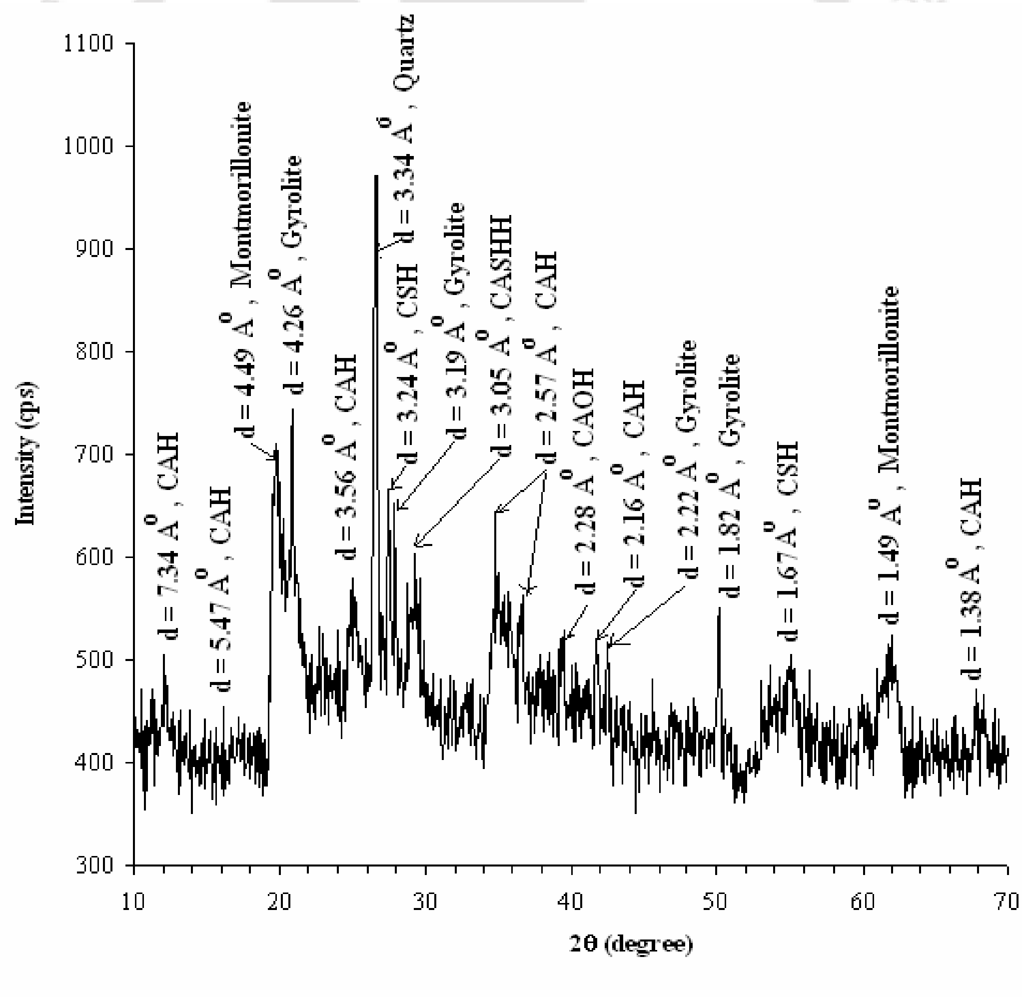

Fig. 8.8 XRD pattern of sample: 100%ES + 5% Lime

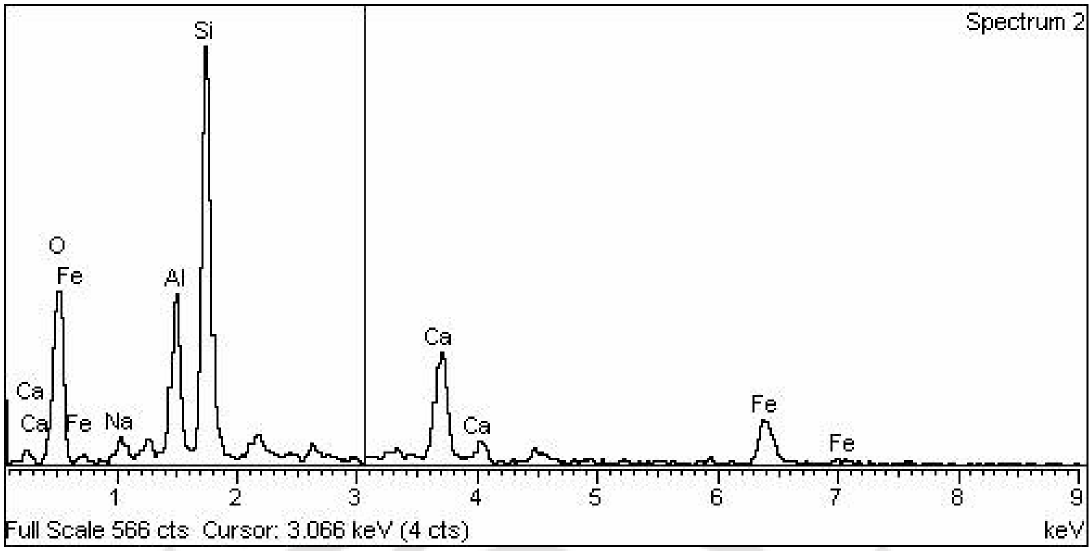

Fig. 8.9 EDX spectrum of sample: 100%ES + 9% Lime

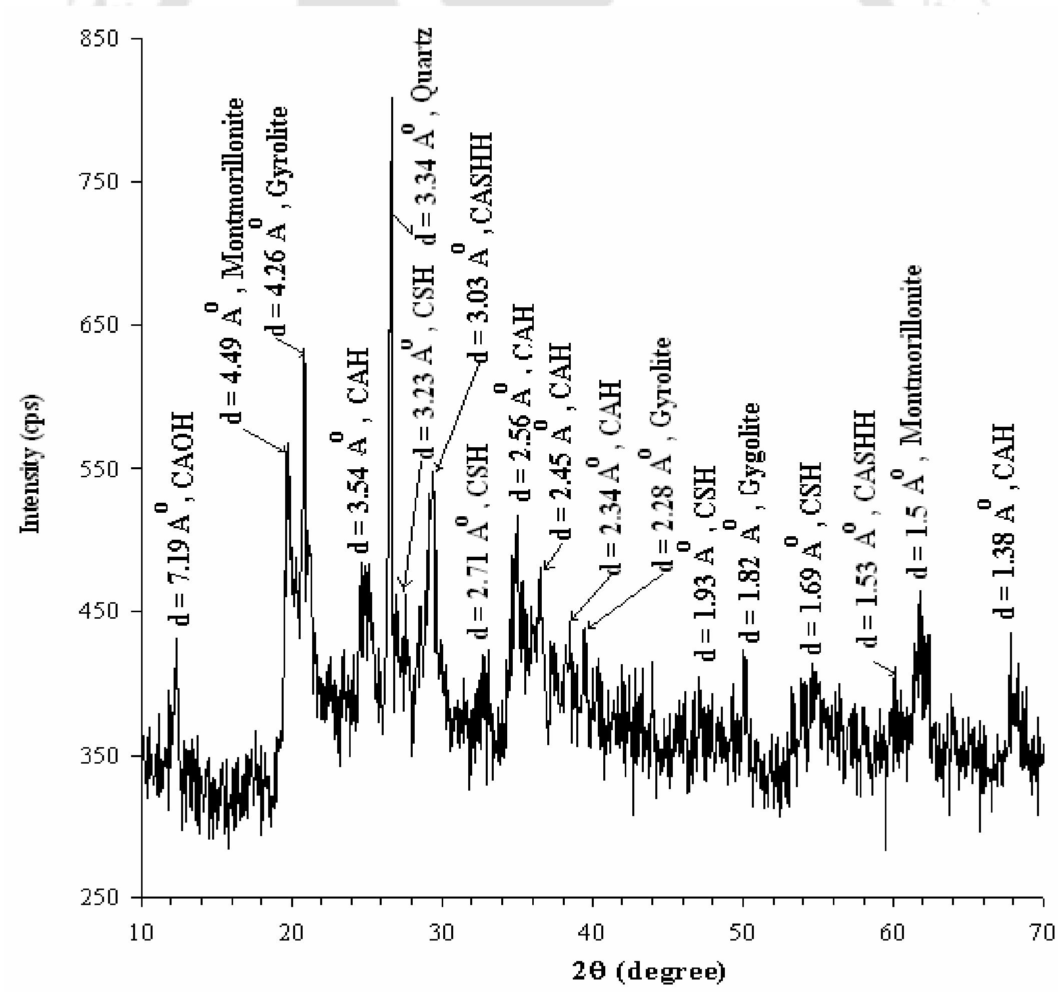

Fig. 8.10 XRD pattern of sample: 100%ES + 9% Lime

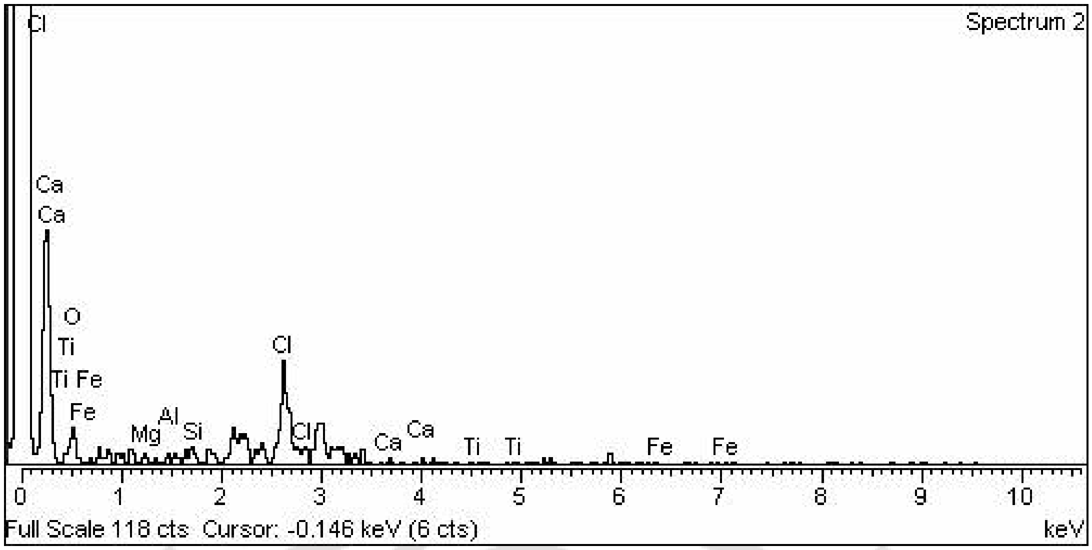

Fig. 8.11 EDX Spectrum of sample: 100%ES +13% Lime

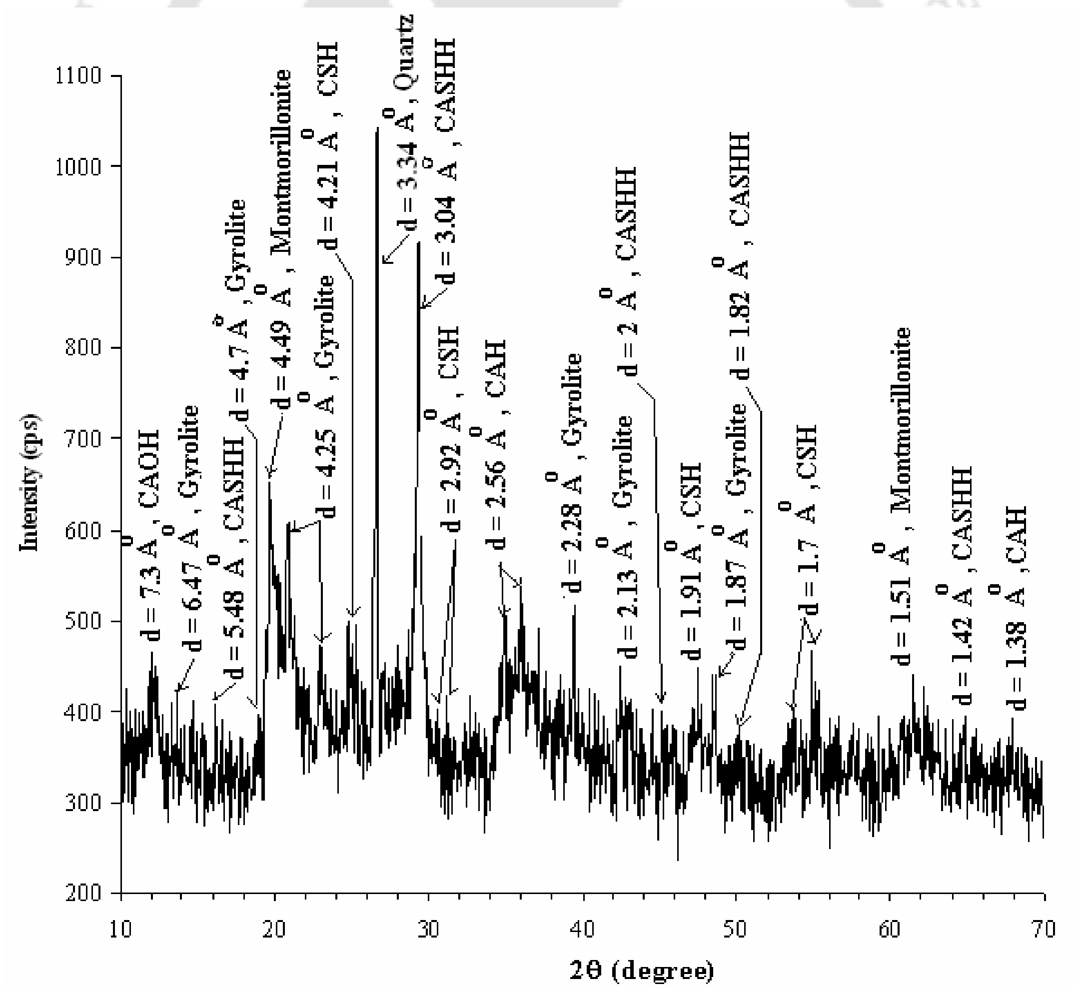

Fig. 8.12 XRD pattern of sample: 100%ES + 13% Lime

The EDX and XRD pattern for the soil sample with 60%ES+40%RS are depicted in Fig. 8.13 and Fig. 8.14 respectively. From the Fig. 8.13 it could be observe that the main elements present in the soil mix (60%ES + 40%RS) are O, Na, Mg, Al, Si, Cl, Ti and Fe. The atomic (%) quantity of different elements presents in the soil sample is shown in Table 8.1. The EDX data shows that among all the elements present Si percentage is the maximum. Fig. 8.14 that depict the XRD pattern it is observed that the element quartz (SiO_2) has the most prominent peak among all, which is in agreement with the EDX data that shows that Si has highest percentage. The presence of Na, Mg and Al in the EDX is attributed to the mineral montmorillonite [$NaO_3(Al\ Mg)_2\ SiO_{10}OH_2.6H_2O$] and Aluminum Oxide ($Al_2O_3$) respectively as depicted in the XRD response. From the EDX response one can observe that there are other elements present in the sample. However, their percentage is very less.

Figs 8.15, 8.17, 8.19, 8.21 and 8.23 show the EDX pattern for the soil mix (60%ES+40%RS) stabilized with different percent of lime (i.e. 1%, 3%, 5%, 9%, and 13%). The main elements present are O, Na, Mg, Al, Si, Cl, Ti, Fe, K, Ca and C. The atomic quantities of these elements are shown in Table 8.1. It can be observed that Si quantity is the highest in percentage. This is attributed to the presence of quartz (SiO_2). The presence of Ca and Al element is in conformation to the cementitious compounds such as; gyrolite [$2Ca.3SiO_2.H_2O$], calcium silicate hydrate (CSH) [$5Ca_2SiO_4.6H_2O$], calcium aluminum silicate hydroxide hydrate (CASHH) [$Ca_5Si_5Al(OH)O_{17}.5H_2O$] formed due to lime induced reactions as observed in the XRD responses depicted in Fig. 8.16, 8.18, 8.20, 8.22 and 8.24.

From the data presented in Table 8.2 (soil: 60%ES+40%RS) it could be observed that the d-spacing for the cementitious compound Gyrolite with 1%, 3%, 5%, 9% and 13% lime are 4.25 A^0, 4.25 A^0, 4.26A^0, 4.26 A^0 and 4.26 A^0 respectively. Similarly for the compound CASHH the d-spacing are 2.95A^0, 3.03 A^0, 3.03 A^0, 3.04 A^0 and 3.04 A^0 and for CSH the d-spacings are 3.51A^0, 2.79 A^0, 2.7 A^0, 2.72 A^0 and 2.59 A^0 respectively. The d-spacing of Gyrolite and CASHH show an increasing trend with increased lime content.

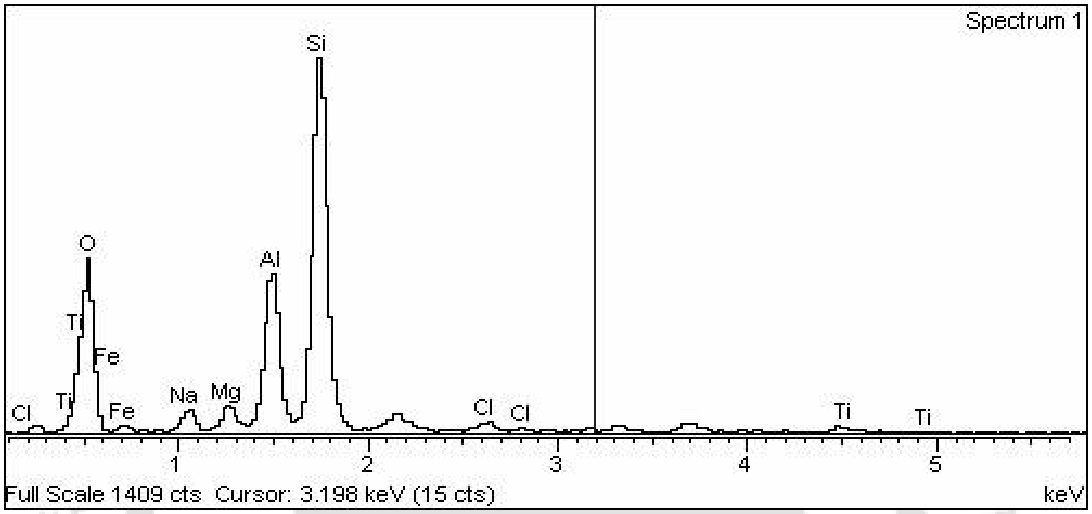

Fig. 8.13 EDX spectrum of sample: 60%ES+40%RS

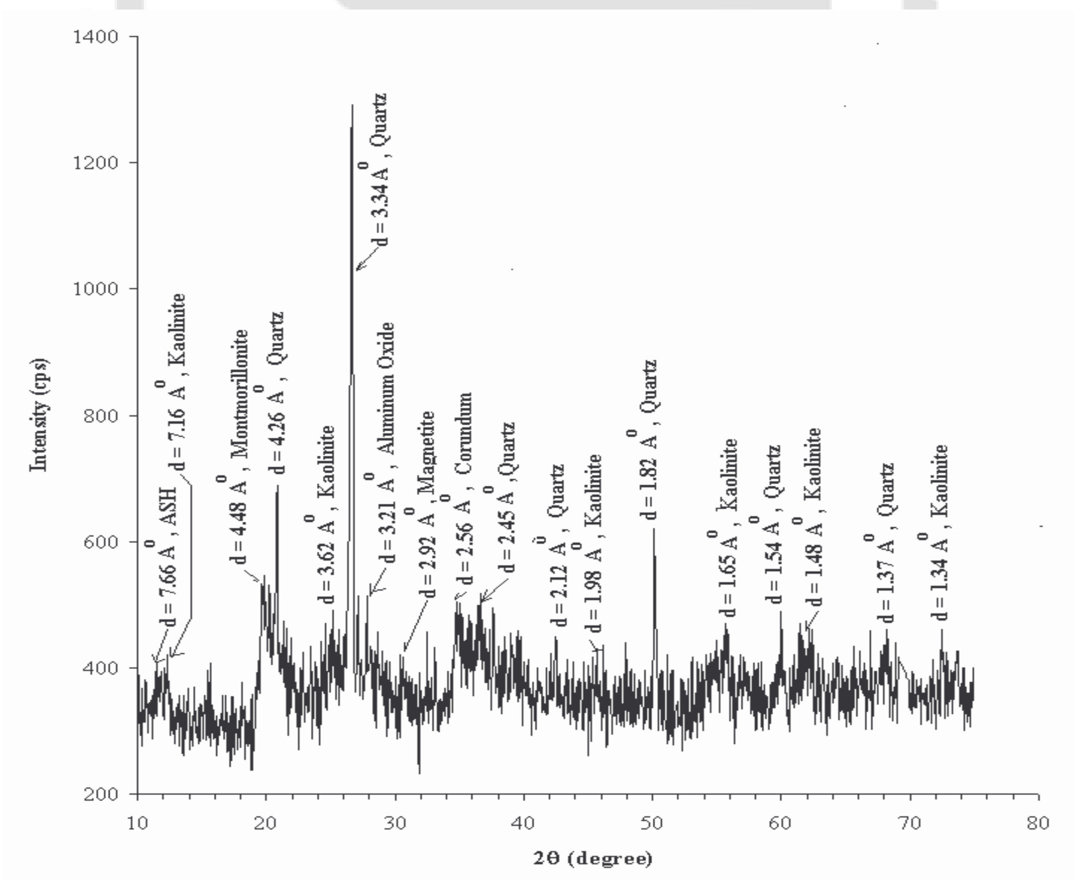

Fig. 8.14 XRD pattern of sample: 60%ES+40%RS

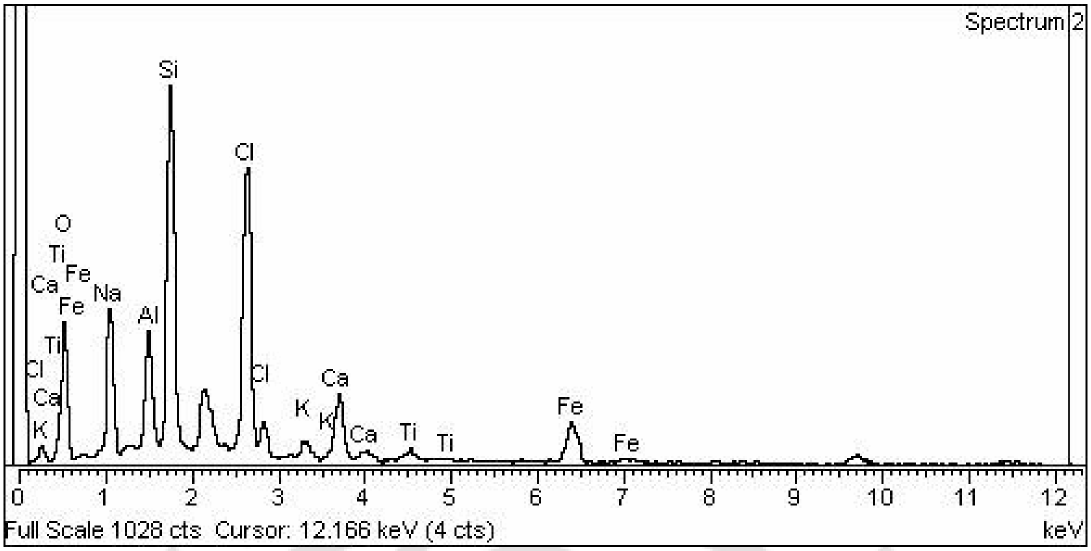

Fig. 8.15 EDX spectrum of sample: 60%ES+40%RS+1% Lime

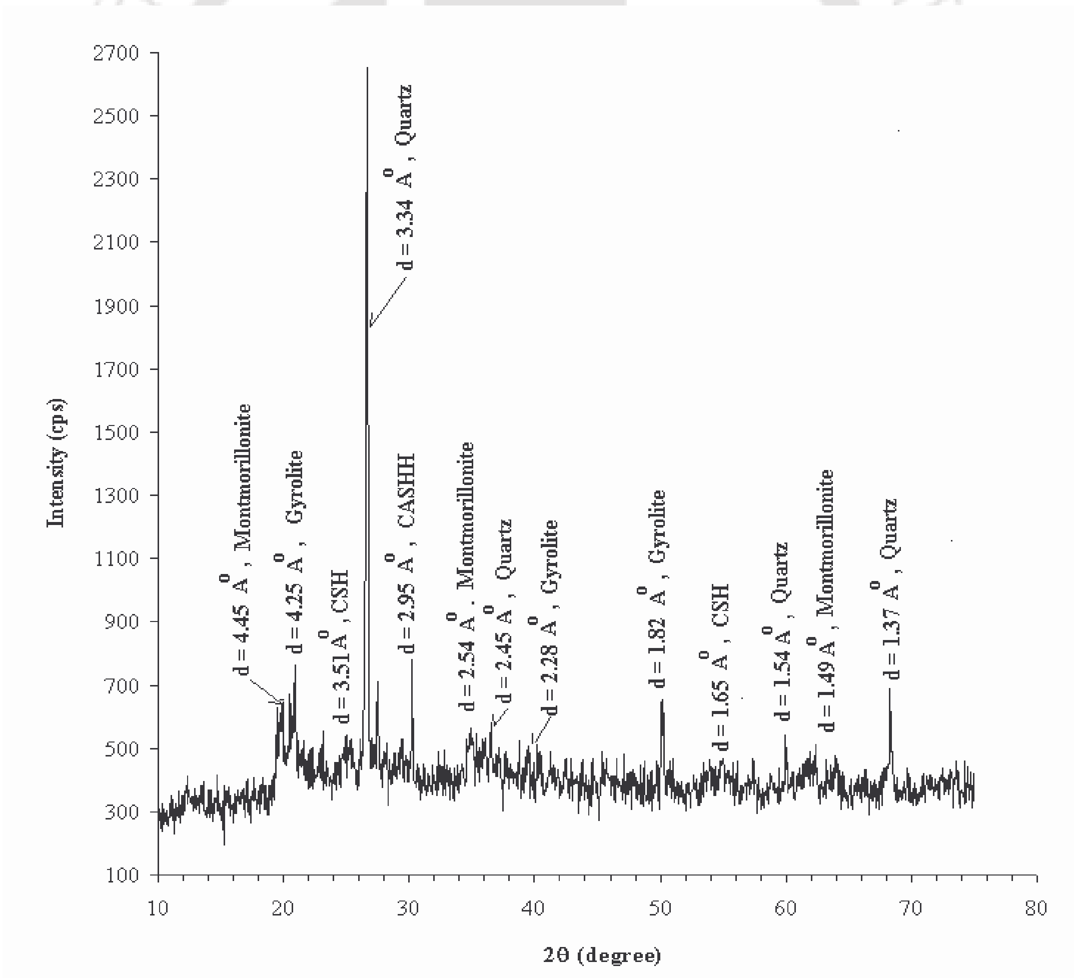

Fig. 8.16 XRD pattern of sample: 60%ES + 40%RS+1% Lime

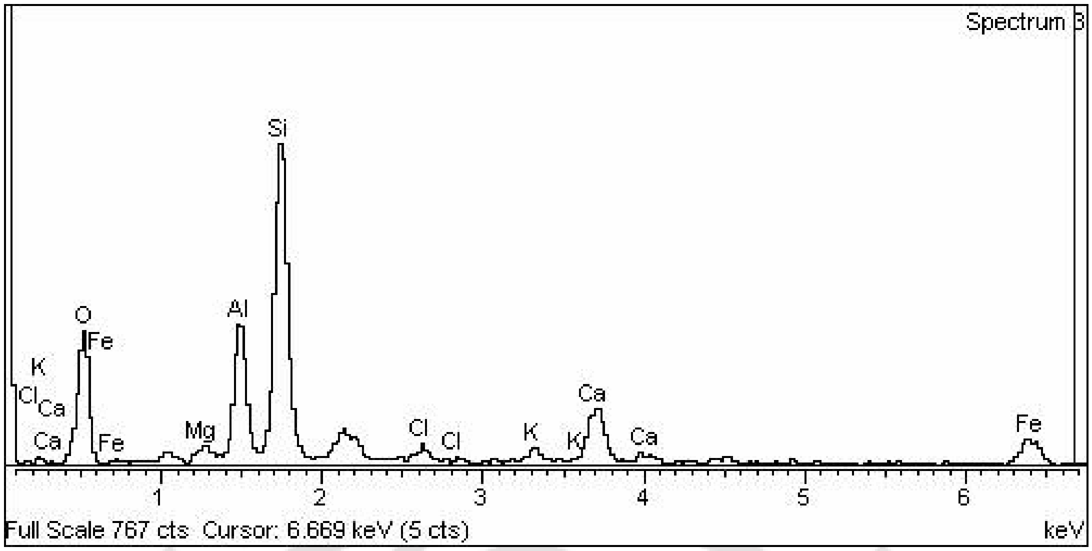

Fig. 8.17 EDX spectrum of sample: 60%ES + 40%RS + 3% Lime

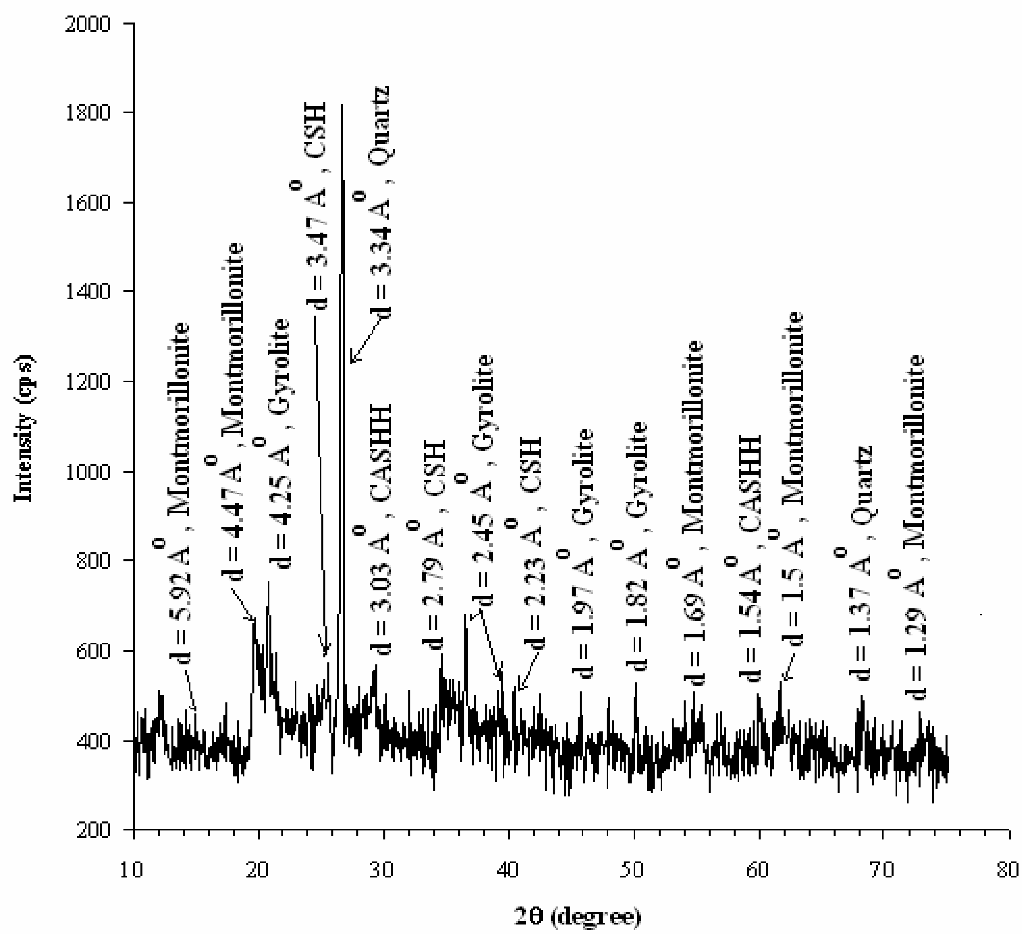

Fig. 8.18 XRD pattern of sample: 60%ES + 40%RS + 3% Lime

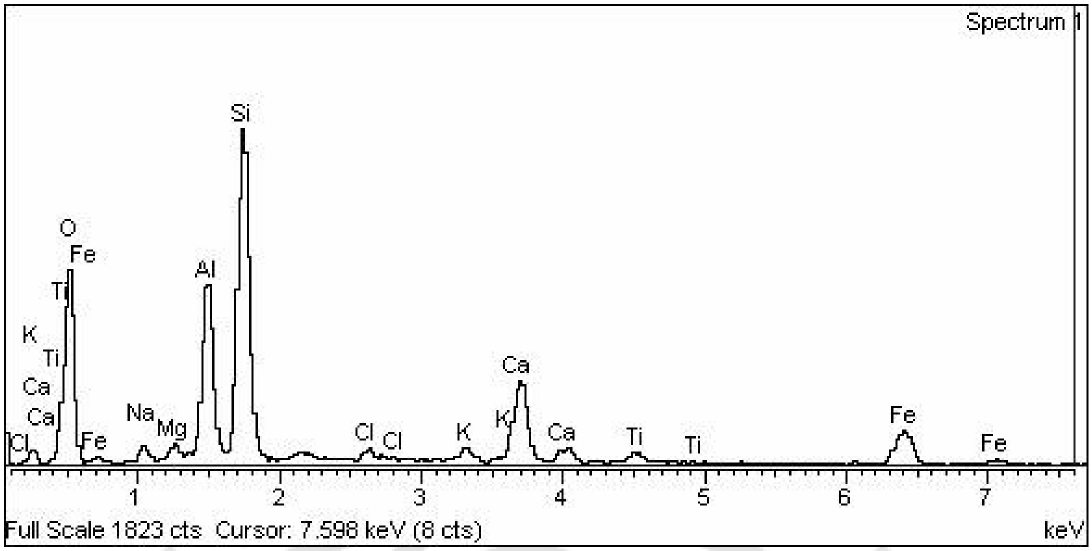

Fig. 8.19 EDX spectrum of sample: 60%ES + 40%RS + 5% Lime

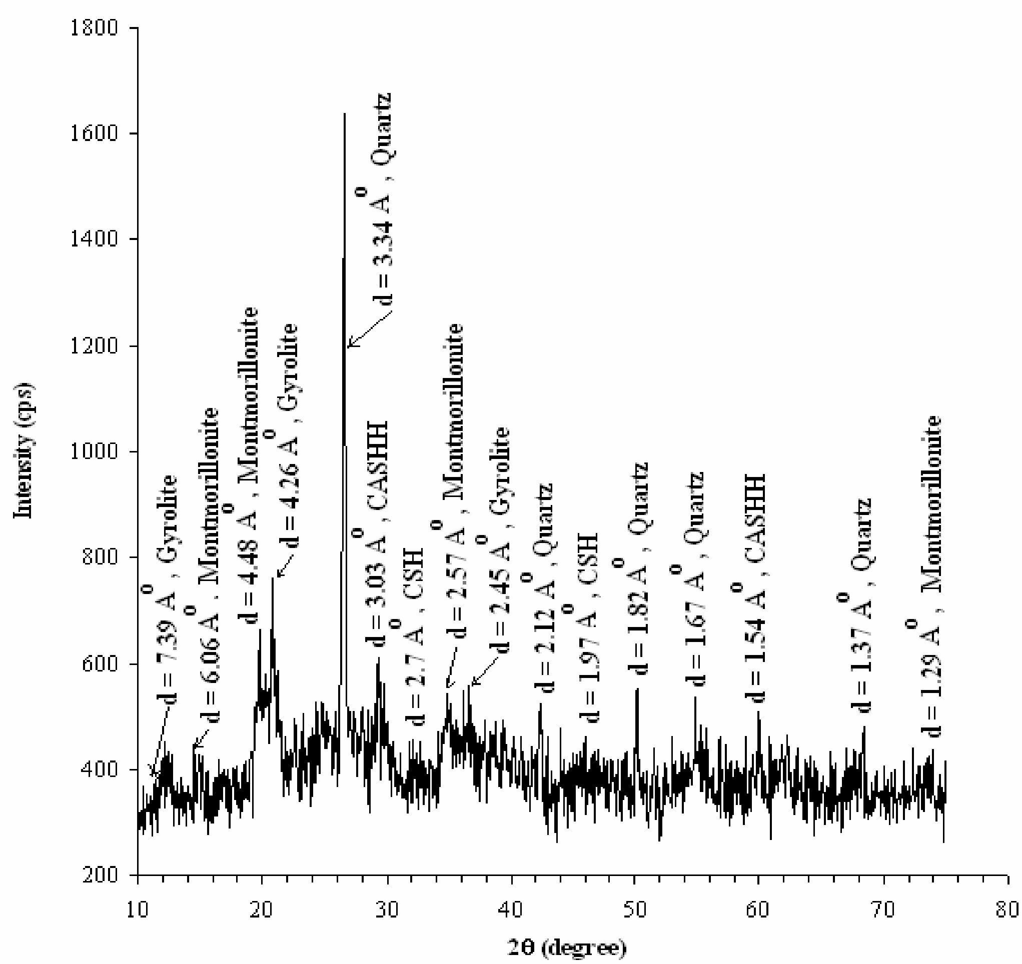

Fig. 8.20 XRD pattern of sample: 60%ES + 40%RS + 5% Lime

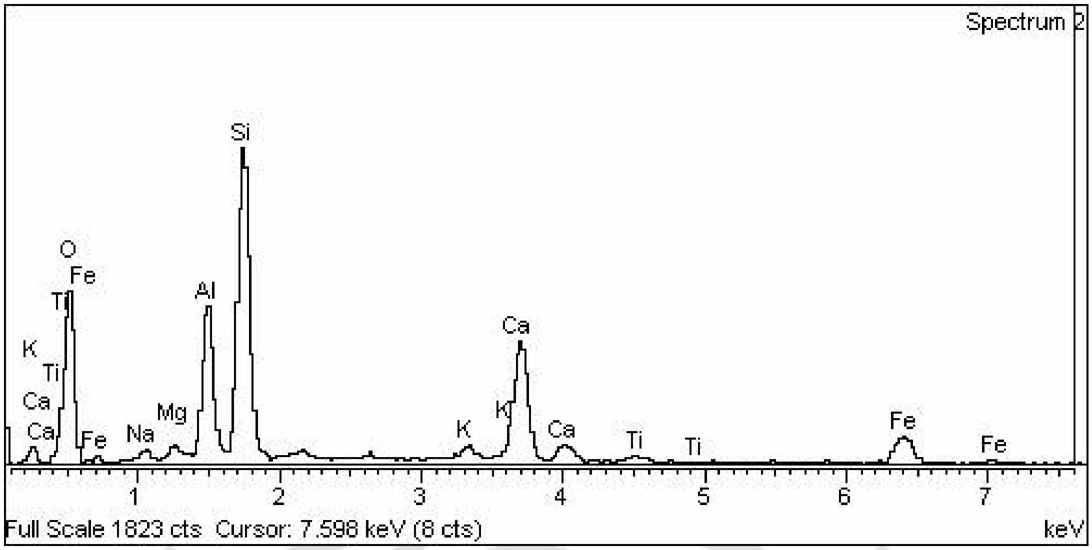

Fig. 8.21 EDX spectrum of sample: 60%ES + 40%RS + 9% Lime

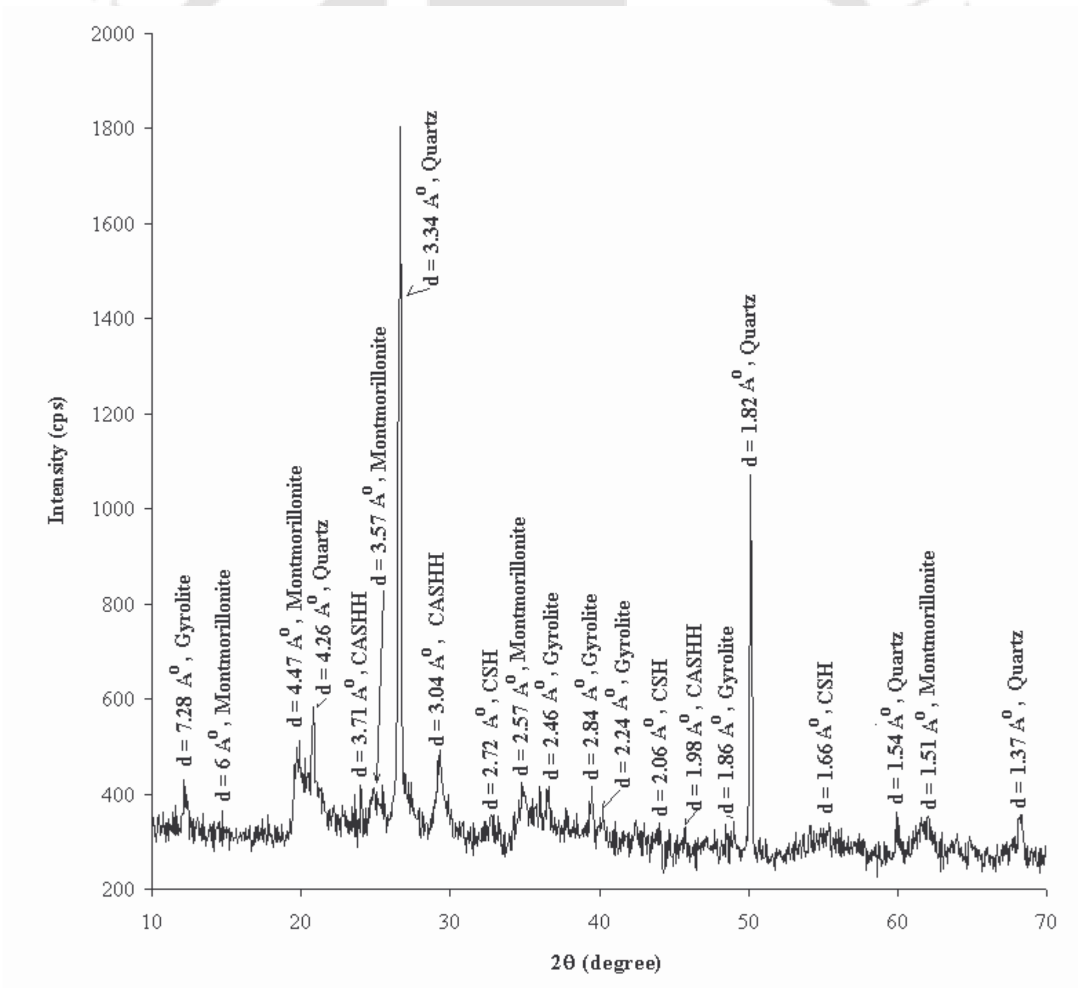

Fig. 8.22 XRD pattern of sample: 60%ES + 40%RS + 9% Lime

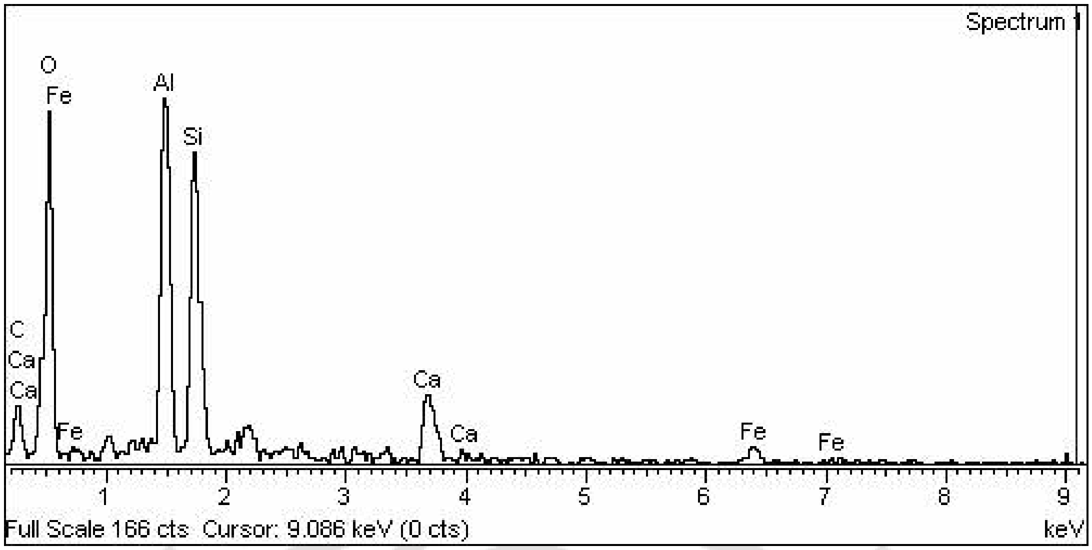

Fig. 8.23 EDX pattern of sample: 60%ES + 40%RS+ 13% Lime

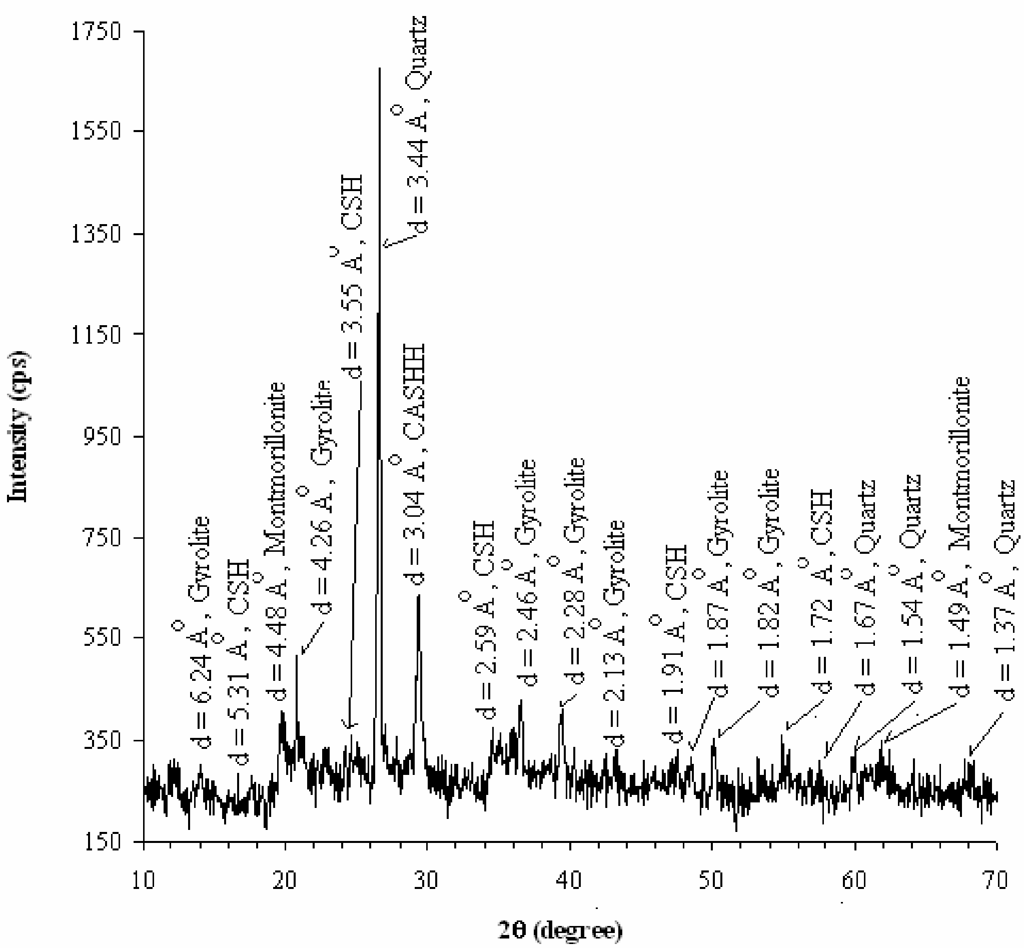

Fig. 8.24 XRD pattern of sample: 60%ES + 40%RS + 13% Lime

The EDX and XRD pattern for the soil sample with 100%RS are depicted in Fig. 8.25 and Fig. 8.26 respectively. From Fig. 8.25 it could be observed that the main elements present in 100%RS are Si, Al, Fe, O and K. The atomic (%) quantities of different elements present in the soil sample are shown in Table 8.1. The EDX data shows that among all the elements present Si percentage is the maximum. From Fig. 8.26 that depicts the XRD pattern it can be observed that for the element quartz (SiO_2) the peak length is the maximum indicating that it has highest percentage which is in agreement with the EDX data that shows that Si has highest percentage. The presence of Al and Fe in the EDX is attributed to the elements such as, Aluminum Silicate Hydroxide [$Al_3Si_2O_7(OH)_3$] and Magnetite (Fe_3O_4) respectively as conformed from the XRD response. From Fig. 8.26 it could be observed that the prominent minerals present are Kaolinite (7.19A^0), Magnetite (2.45A^0), Quartz (3.34A^0), Aluminum Silicate Hydroxide (2.46A^0), and Corundom (2.55A^0). Among all, Quartz is the predominant mineral.

Figs. 8.27, 8.29, 8.31, 8.33 and 8.35 show the EDX pattern for residual soil (100%RS) stabilized with different percent of lime (i.e. 1%, 3%, 5%, 9% and 13%). The main element present are O, Al, Si, K, Fe, and Ca. The atomic quantities of these elements are shown in Table 8.1. It can be observed that Si quantity is the highest in percentage. The presence of Ca and Al elements is in conformation to the cementitious compounds formed such as Gyrolite [$2CaO.3SiO_2.H_2O$] and Calcium Aluminum Silicate Hydroxide Hydrade (CASHH) [$Ca_5Si_5Al(OH)O_{17}.5H_2O$] present in the soil sample. This is established from the XRD patterns depicted in the Figs. 8.28, 8.30, 8.32, 8.34 and 8.36 respectively. Apart from this the other compounds present are quartz, Kaolinite, Magnetite and corundum, which were originally present in the untreated soil. The Quartz peak which was originally about 3700 cps in the untreated soil (Fig. 8.26) has reduced to about 2800 cps due to lime treatment with 13% lime added to it (Fig. 8.36). This indicates that Quartz has been substantially attacked by lime. This is clearly noted in terms of formation of silica gel as discussed in the next section, through SEM micrographs.

From the data presented in Table 8.2 it can be noted that the d-spacing for the compound Gyrollite for the cases with 1%, 3%, 5%, 9%, 13% lime are 4.19A^0, 4.18A^0, 4.19A^0, 4.26A^0 and 4.26A^0 respectively. Similarly for the compound CASHH the d-spacing is 2.89A^0, 2.99A^0, 2.98A^0, 3.04A^0 and 3.04A^0 respectively. This clearly shows that the d-spacing value of cementitious compounds increases prominently with lime, for lime contents greater than 5%, indicating that there is expansion of cementitious compounds at higher percent of lime treatment. From the test data presented in chapter 7 it was observed that the unconfined compressive strength of the residual soil reduces with lime, for lime content greater than 5%. Besides, in chapter 6 it was reported that the swelling of this soil increases with lime content greater than 5%. Since an expanded structure is relatively weak because of large opening spaces formed within, it gives rise to reduced strength and higher swelling. The visible reduction of peaks at higher lime content indicates the formation of amorphous compounds. This aspect is once again analysed through the SEM micrographs presented in the next section.

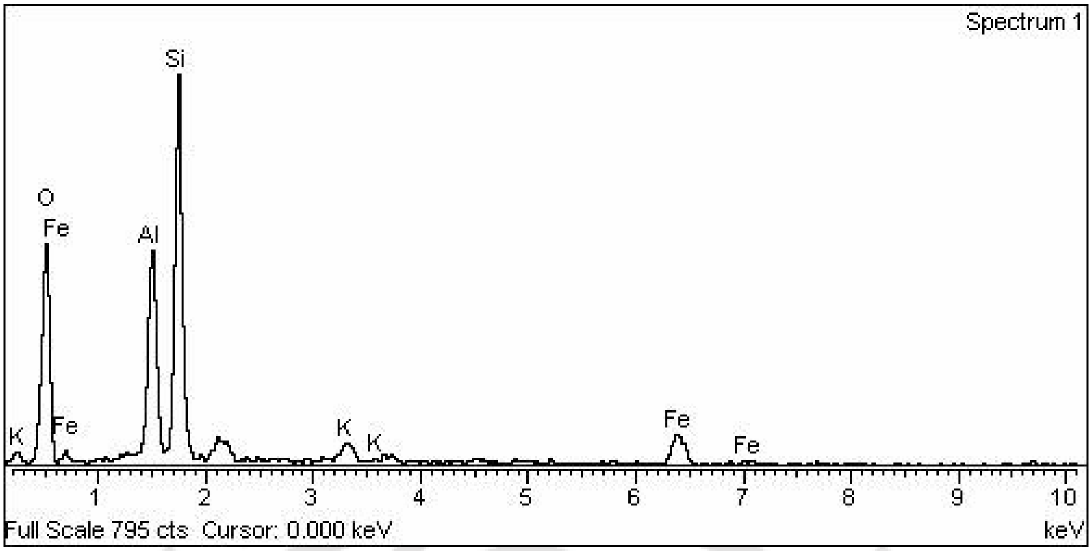

Fig. 8.25 EDX spectrum of sample: 100%RS

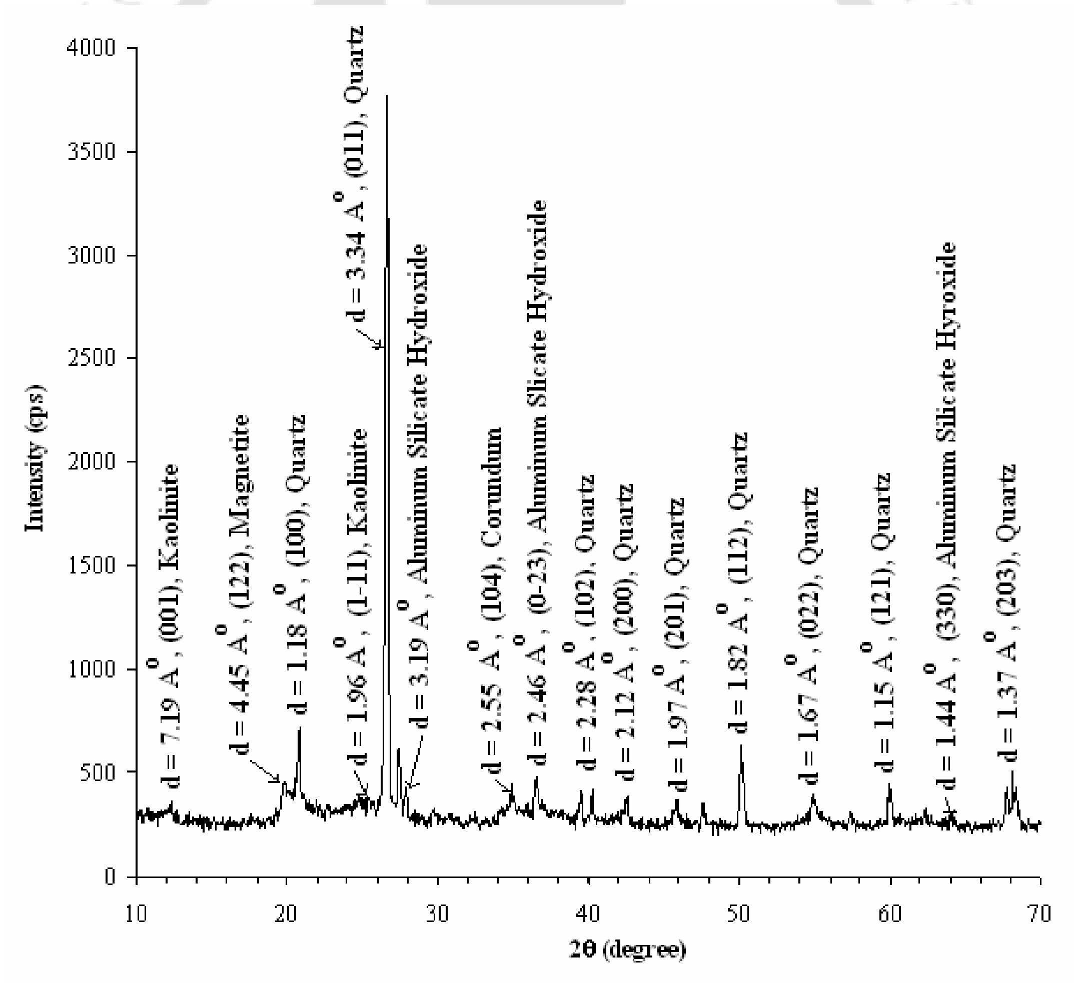

Fig. 8.26 XRD pattern of sample: 100%RS

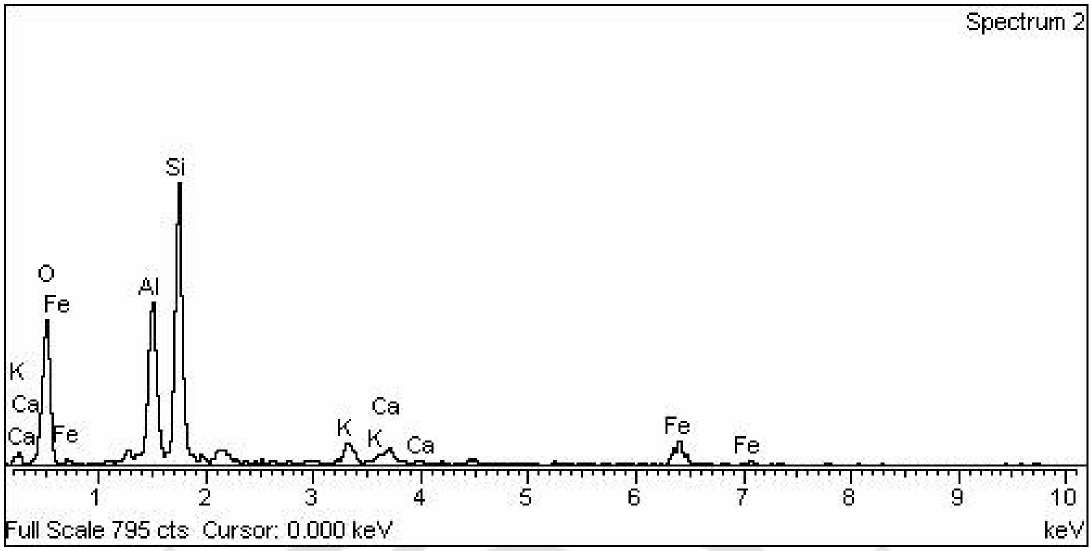

Fig. 8.27 EDX spectrum of sample: 100%RS+1%Lime

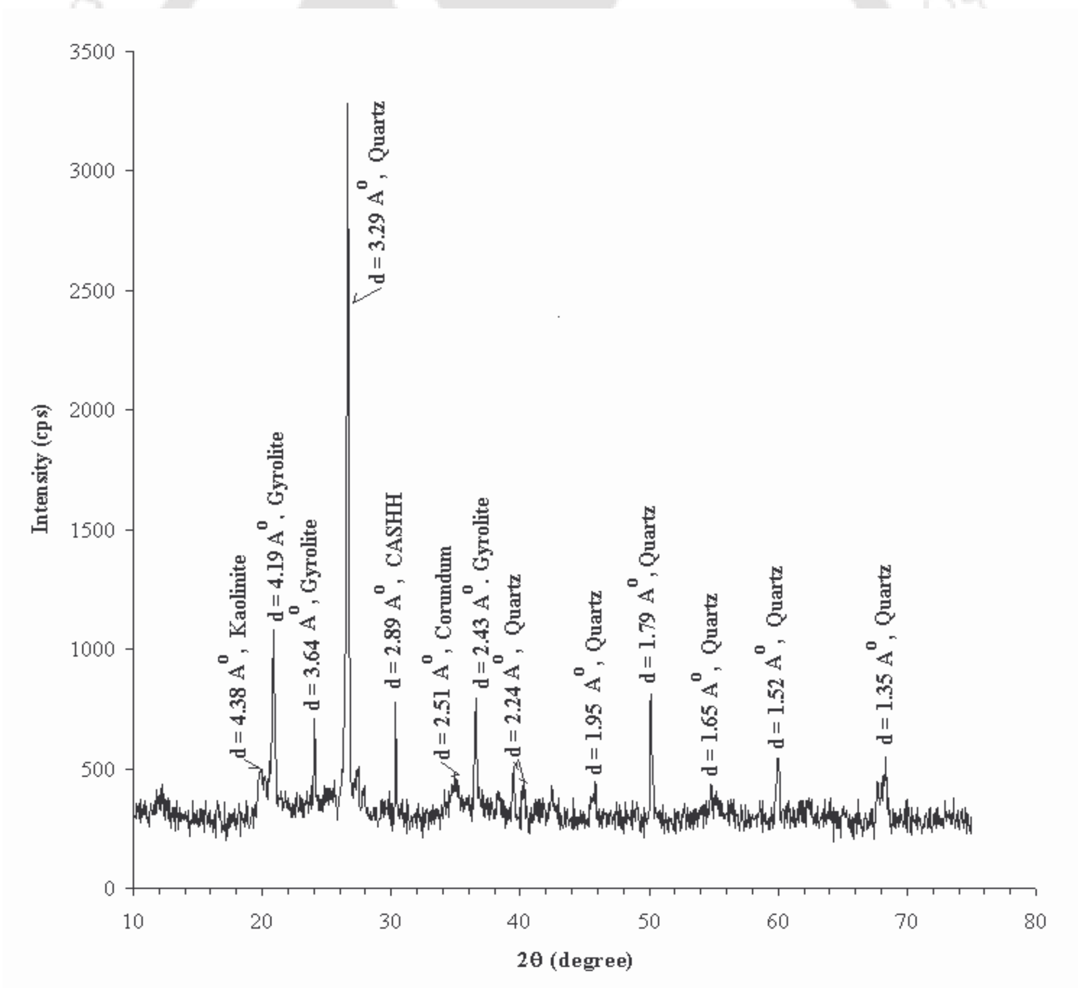

Fig. 8.28 XRD pattern of sample: 100%RS+1%Lime

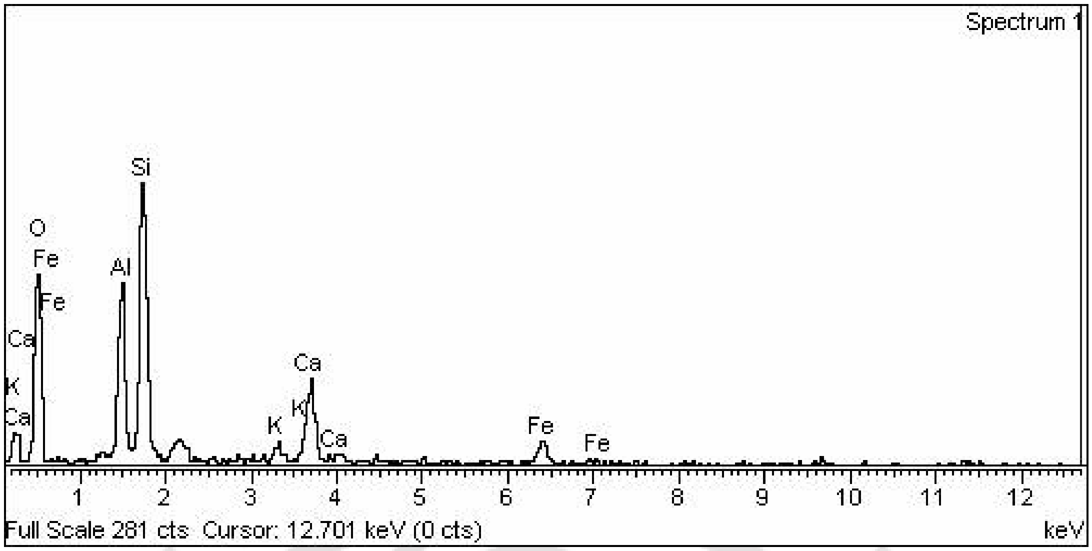

Fig. 8.29 EDX spectrum of sample: 100%RS+3%Lime

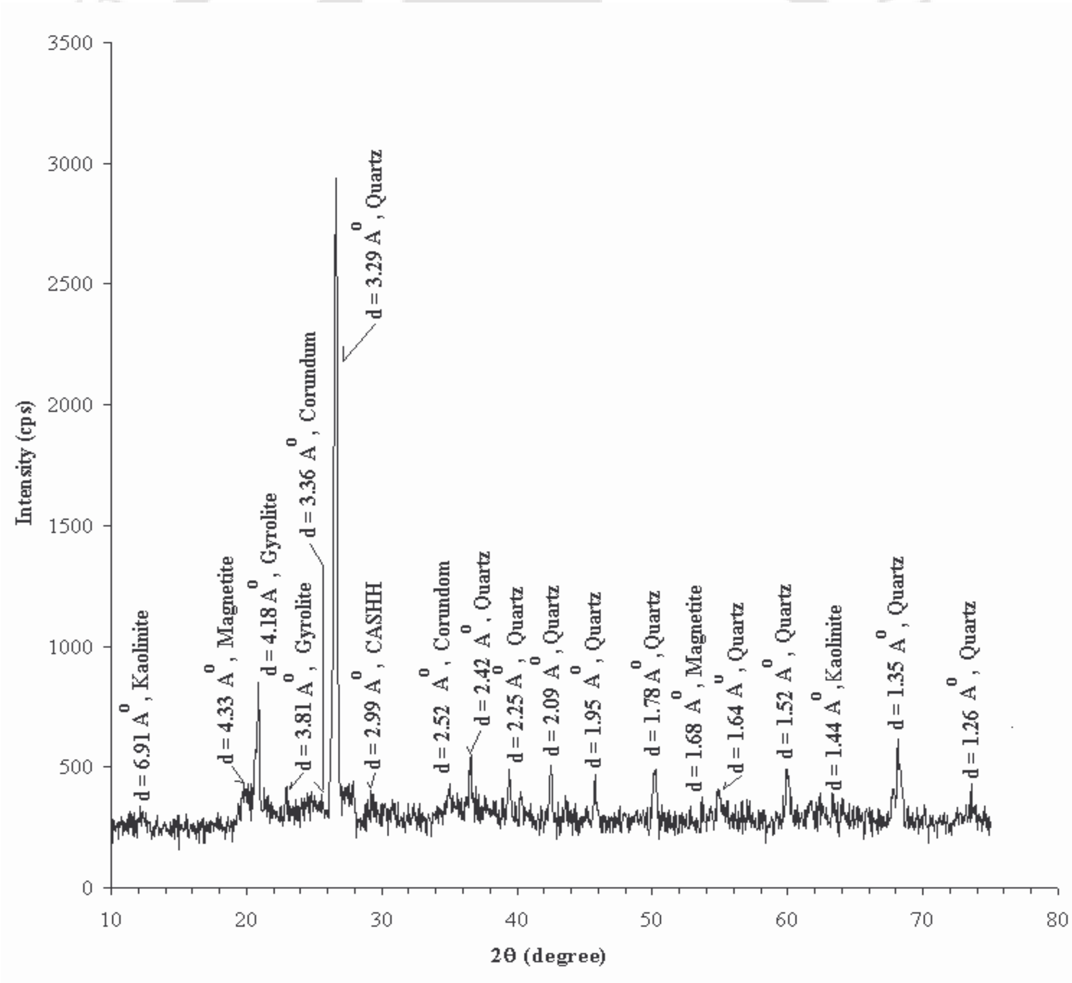

Fig. 8.30 XRD pattern of sample: 100%RS+3%Lime

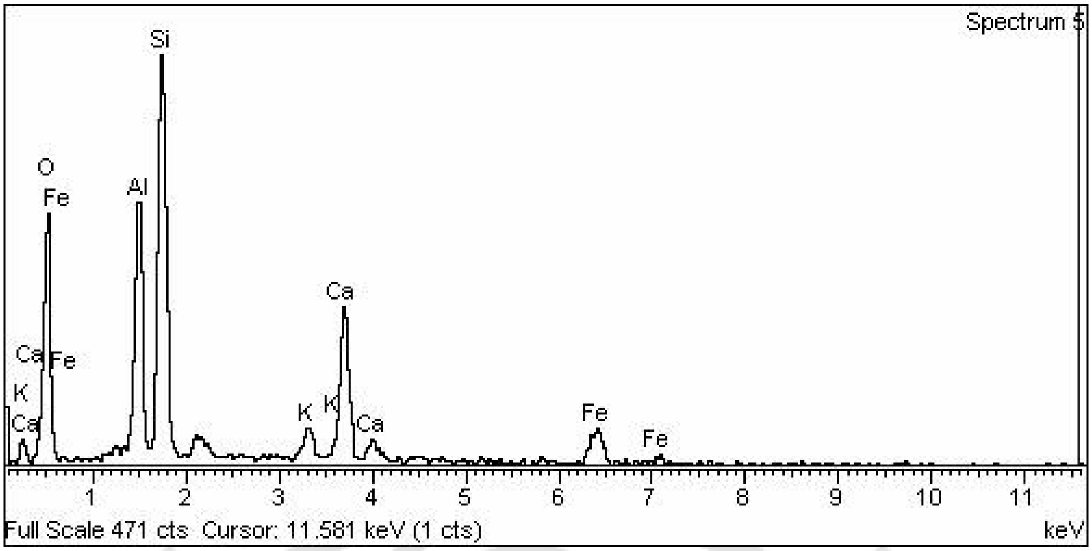

Fig. 8.31 EDX spectrum of sample: 100%RS + 5% Lime

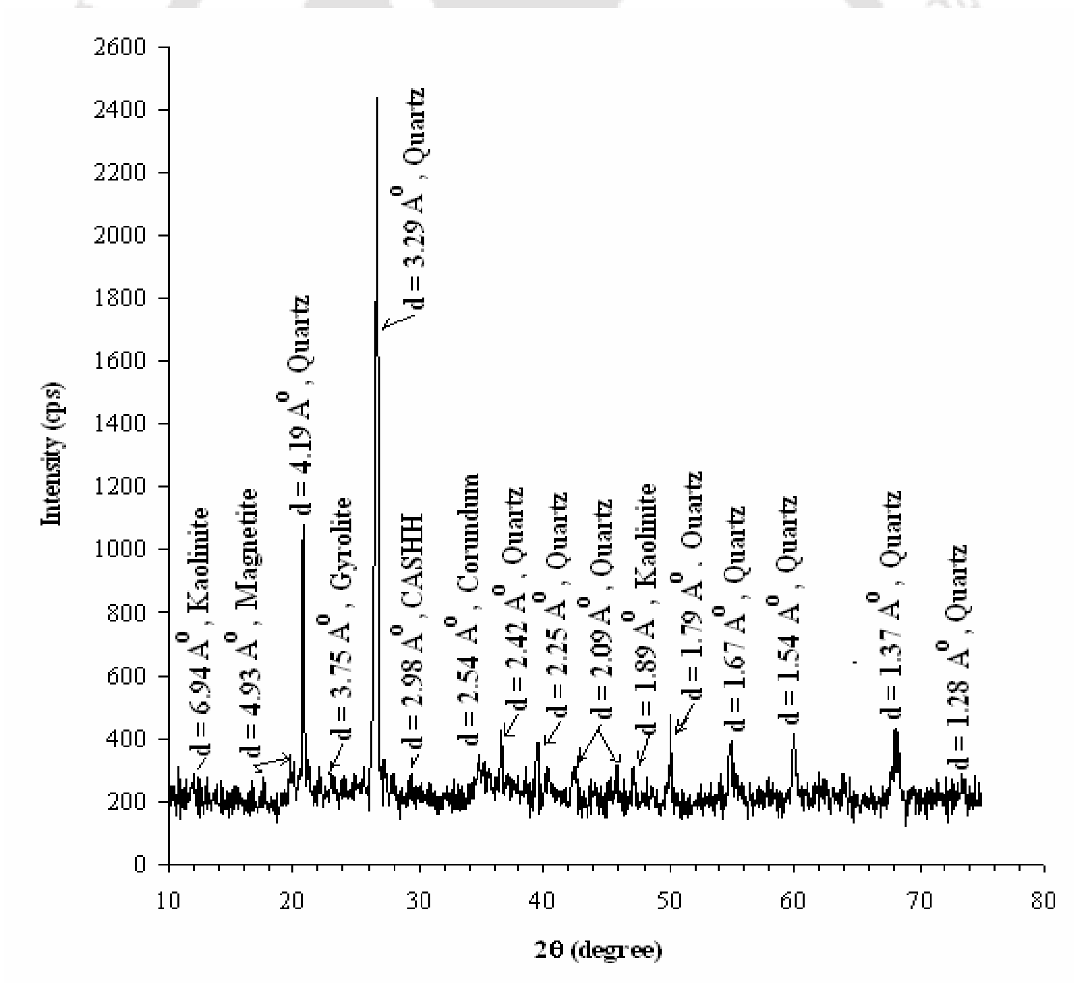

Fig. 8.32 XRD pattern of sample: 100%RS+5%Lime

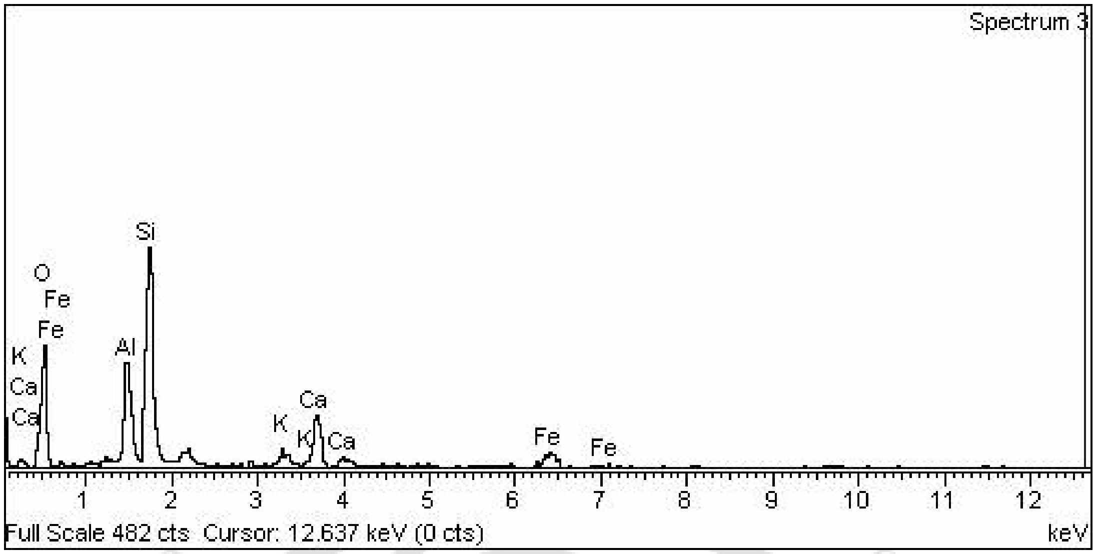

Fig. 8.33 EDX spectrum of sample: 100%RS + 9% Lime

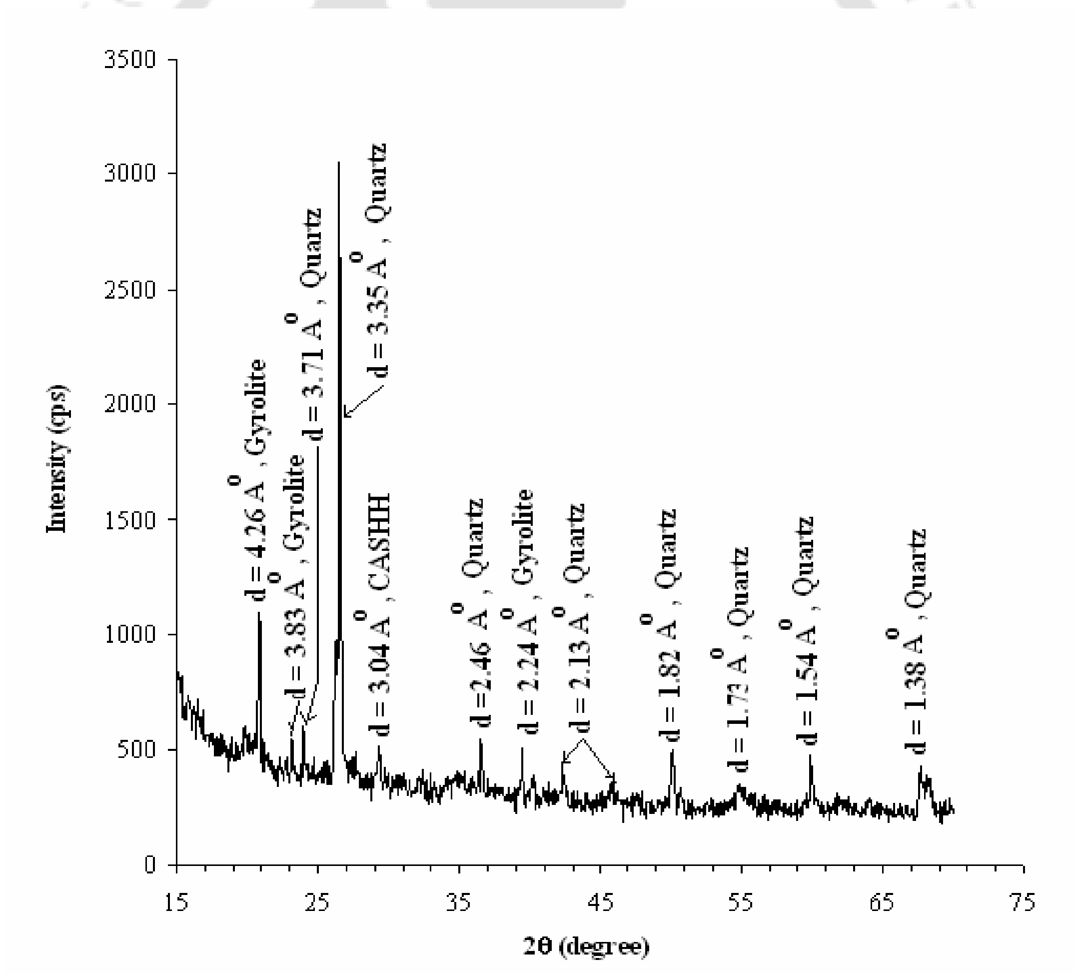

Fig. 8.34 XRD pattern of sample: 100%RS + 9% Lime

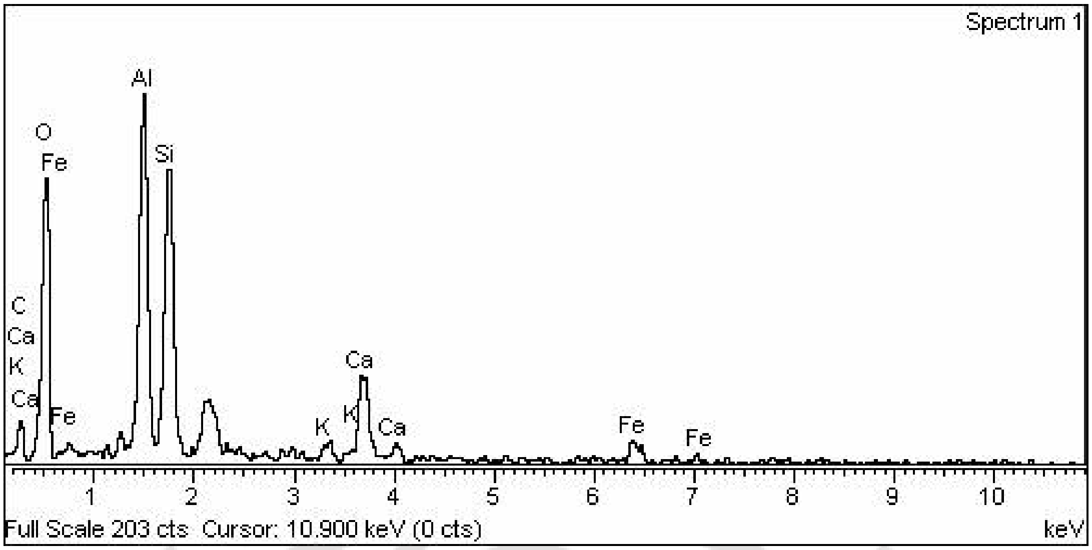

Fig. 8.35 EDX spectrum of sample: 100%RS + 13% Lime

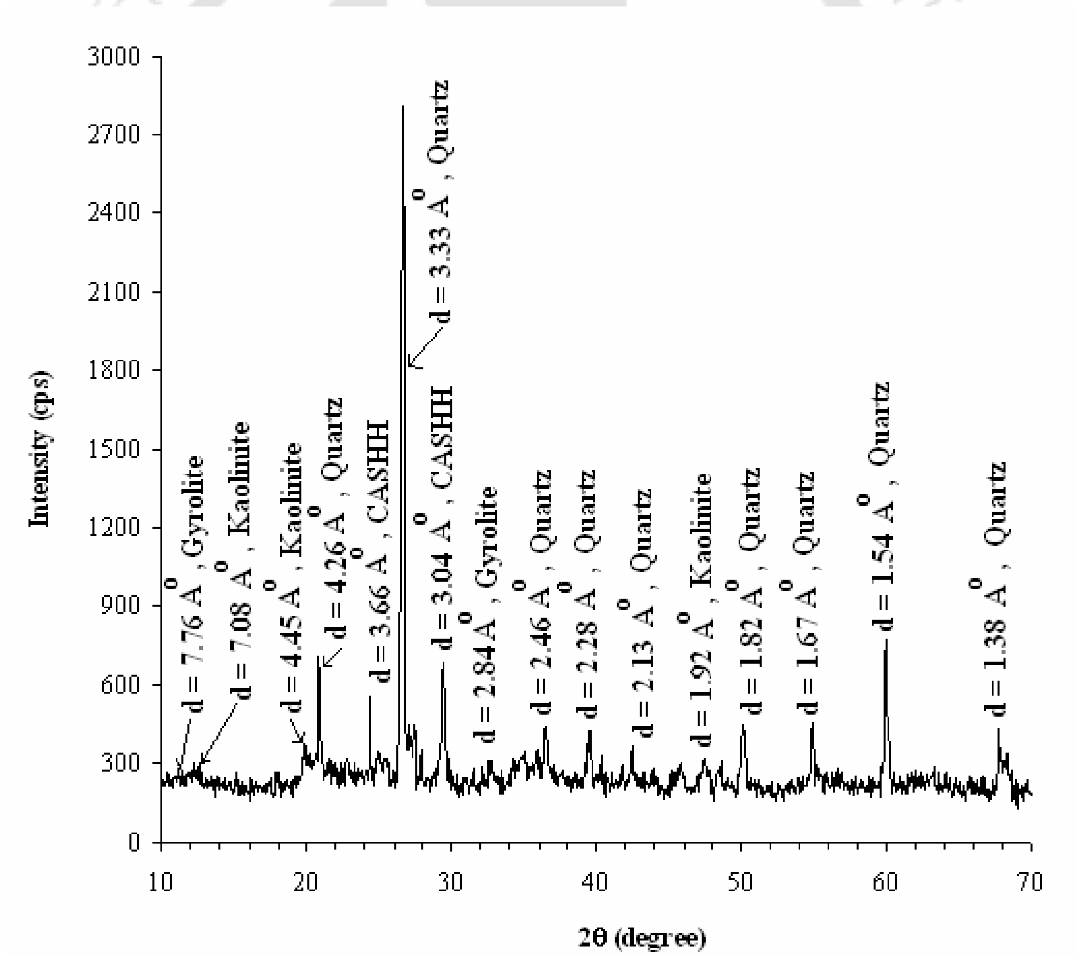

Fig. 8.36 XRD pattern of sample: 100%RS + 13% Lime

Table 8.1 Atomic quantity (%) of elements for lime treated soils obtain from EDX

Soil	Atomic quantity (%)						
	Elements	0% lime	1%lime	3% lime	5% lime	9% lime	13% lime
100%ES	O	61.68	61.05	50.16	61.96	66.95	59.93
	Na	1.12	1.31	NF	1.87	NF	1.67
	Mg	1.6	1.14	0.65	1.09	NF	1.34
	Al	8.76	8.89	6.03	8.3	5.4	9.56
	Si	21.14	21.99	15.72	18.53	11.94	22.42
	K	0.47	0.81	NF	NF	NF	NF
	Ca	0.31	0.9	2.6	3.6	12.88	2.07
	Ti	0.54	0.58	NF	0.78	NF	NF
	Fe	4.39	3.33	2.71	3.87	2.19	3
	C	NF	NF	21.13	NF	NF	NF
	Cl	NF	NF	NF	NF	0.64	NF
60%ES+ 40%RS	O	60.28	48.85	61.05	63.65	65.07	49.95
	Na	2.4	11.77	NF	1.48	0.99	NF
	Mg	1.86	NF	1.23	0.8	0.76	NF
	Al	8.82	5.35	8.34	8.32	7.32	10.41
	Si	23.77	14.71	20.64	16.81	16.14	9.84
	Cl	0.57	12.22	0.87	0.48	NF	NF
	Ti	0.38	0.45	NF	0.57	0.31	NF
	Fe	1.92	3.22	3.29	3.15	2.67	2.02
	K	NF	0.69	0.75	0.6	0.53	NF
	Ca	NF	2.74	3.84	4.15	6.21	2.68
	C	NF	NF	NF	NF	NF	25.11
100%RS	O	63.98	62.4	68.96	66.39	67.15	62.71
	Al	10.07	10.42	8.97	9.16	8.22	13.87
	Si	21.88	21.8	14.86	15.54	17.32	13.86
	K	0.9	1.52	0.91	1	0.91	1.09
	Fe	3.17	2.99	2.17	2.52	2.11	3.3
	Ca	NF	0.86	4.13	5.39	4.29	5.17

NF: Not found

Table 8.2 d-spacing value of compounds formed in lime treated soils

Soil	Lime (%)	d-spacing (A^0)						
		Cementitious compounds					Mineral	
		Gyrolite	CASHH	CSH	CAH	CAOH	Quartz	Montmorillonite
100%ES	0	NF	NF	NF	NF	NF	2.36	4.48
	1	4.28	3.06	NF	NF	2.5	3.34	4.47
	3	4.26	3.05	4.18	7.17	2.49	3.35	4.45
	5	4.26	3.05	3.24	2.57	2.28	3.34	4.49
	9	4.26	3.03	4.21	2.56	7.19	3.34	4.49
	13	4.25	3.04	4.21	2.49	7.3	3.34	4.49
60%ES+40%RS	0	NF	NF	NF	NF	NF	3.34	4.48
	1	4.25	2.95	3.51	NF	NF	3.34	4.45
	3	4.25	3.03	2.79	NF	NF	3.34	4.47
	5	4.26	3.03	2.7	NF	NF	3.34	4.48
	9	4.26	3.04	2.72	NF	NF	3.34	4.47
	13	4.26	3.04	2.59	NF	NF	3.44	4.48
100%RS	0	NF	NF	NF	NF	NF	3.34	NF
	1	4.19	2.89	NF	NF	NF	3.29	NF
	3	4.18	2.99	NF	NF	NF	3.29	NF
	5	4.19	2.98	NF	NF	NF	3.29	NF
	9	4.26	3.04	NF	NF	NF	3.35	NF
	13	4.26	3.04	NF	NF	NF	3.33	NF

NF: Not found

8.3 SCANNING ELECTRON MICROSCOPIC (SEM) ANALYSIS

Fig. 8.37 and 8.38 show the scanning electron micrograph of the dry powder bentonite used as the expansive soil in the present study. The wavy shaped scaling appearance on the soil surface indicates a layered structure within, which is typical of montmorillonite type. The extremely rough texture indicates large volume of micropores that can hold substantial amount of water into it. Indeed the micrograph of the compacted specimen of the wet soil (at OMC) depicted in Fig. 8.39 shows relatively plain surface indicating that the voids are filled up due to volume increase of the expansive soil, through absorption of water. From this micrograph it can be observed that there are aggregated clay packets, with intermittent pores. This is because in the presence of water the clay particles adhere to each other to form clusters which when compacted forms large clay packets. However the compactive effort applied is not high enough to completely close the inter cluster voids. Finer details of the soil particles under further high magnifications could not be obtained. This is attributed to the extremely fine size of the soil particles. Similar such difficulties in case of montmorillonite soils was experienced by Keller (1985).

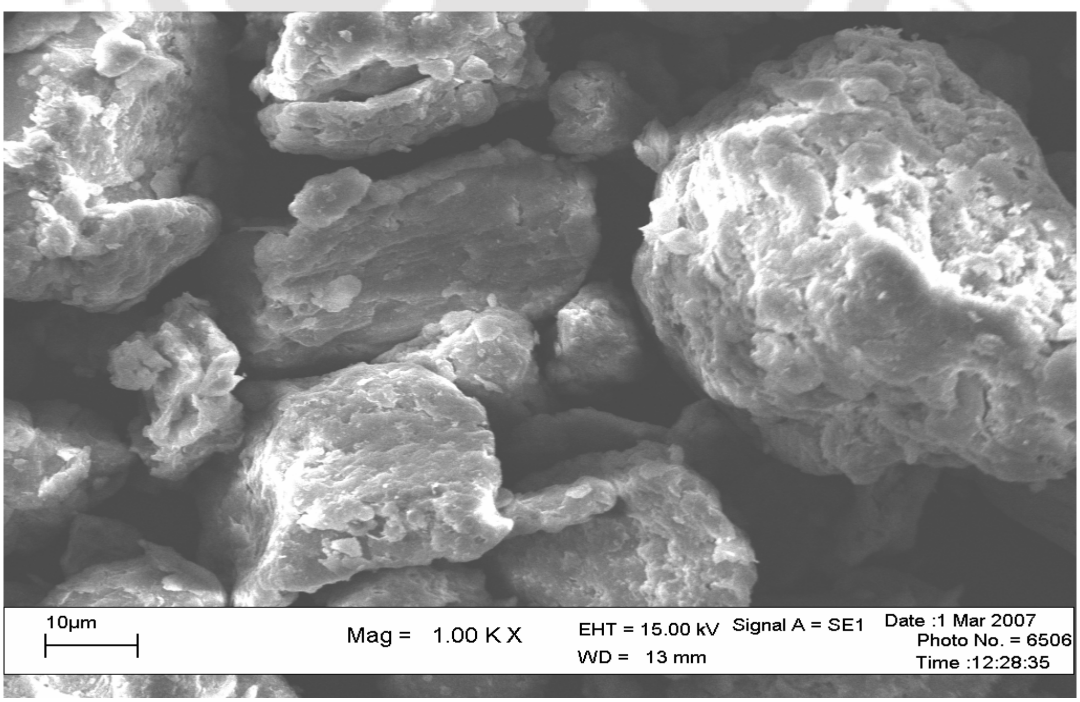

Fig. 8.37 SEM micrograph of expansive soil (powder sample, M = 1.00KX)

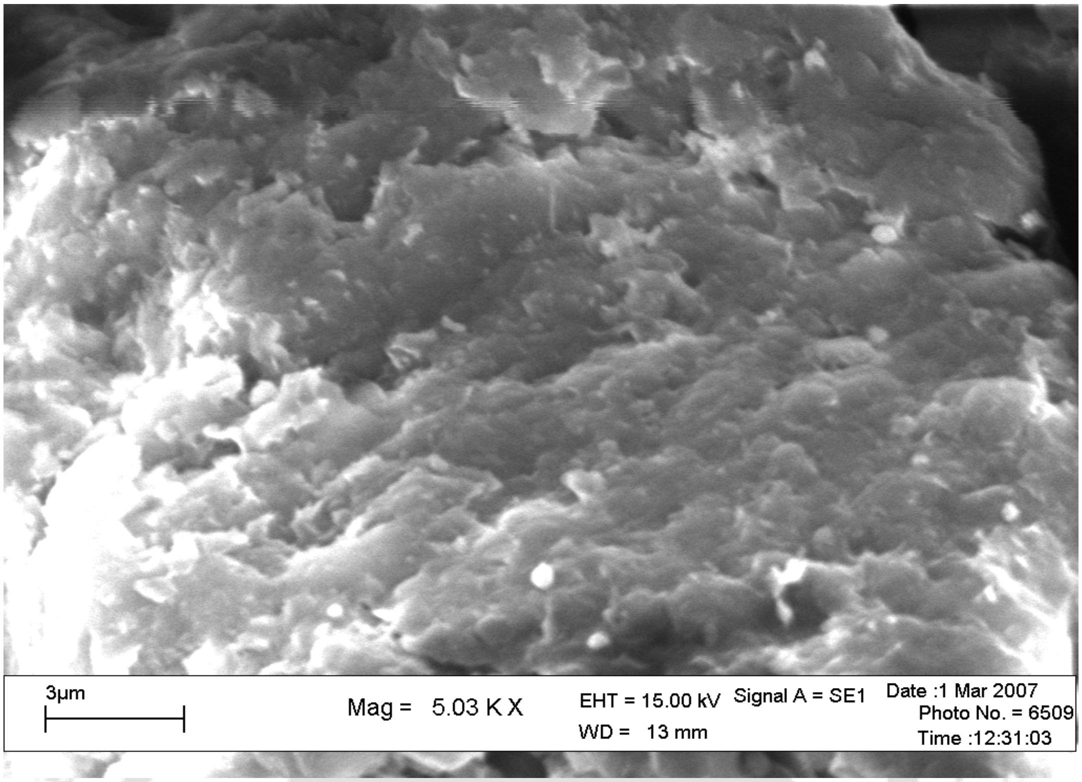

Fig. 8.38 SEM micrograph of expansive soil (powder sample, M = 5.03KX)

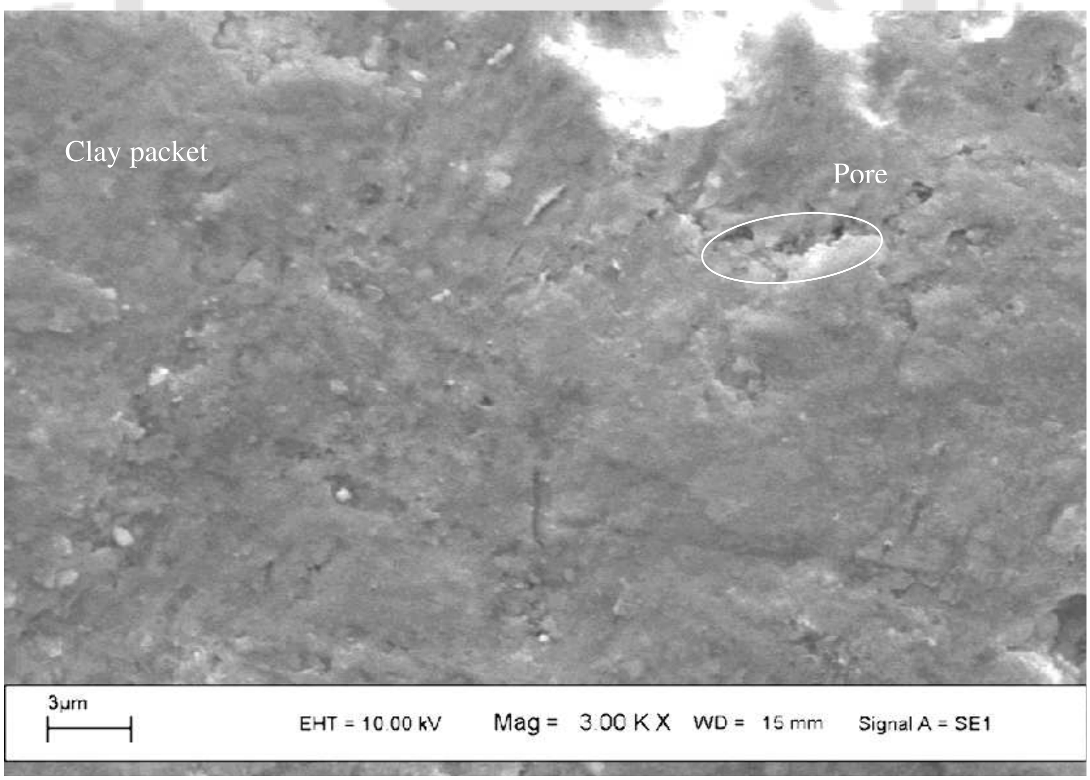

Fig. 8.39 SEM micrograph of expansive soil (compacted sample, M = 3 KX)

The micrograph of the lime treated expansive soils with 1%, 3%, 5% and 9% lime are depicted in Figs. 8.40, 8.41, 8.42, and 8.43 respectively. In comparison to the untreated soil (Fig. 8.39) the micrographs of the lime treated soils show an increase in grain size. This is attributed to flocculation and aggregation of clay particles caused due to lime treatment. The white patches are the cementitious gels formed due to lime induced pozzolanic reactions. These cementitious gels apart from bonding the clay particles together fill in the voids between the aggregated particles leading to a denser fabric. These microchanges are believed to have enhanced the performance of the expansive soil interms of enhanced strength and reduced swell-shrink etc. The cementation products formed are identified by XRD to be calcium aluminum silicate hydroxide hydrate (CASHH), calcium aluminum hydrate (CAH), calcium silicate hydrate (CSH) and Gyrolite [$2CaO.3SiO_2.H_2O$]. These products contribute to the long-term strength development in the lime treated soil.

The micrograph of expansive soil with 13% lime depicted in Fig. 8.44 shows less number of islands and hence a more uniform structure. The voids are relatively small in size and more uniformly distributed. This is believed to be due to more dissolution of soil particles and formation of gels under increased alkaline environment due to increased lime content (13%). The gels having large volume of micropores makes the entire soil mass porous as inferred from the spongy appearance of the micrograph. Besides the reduced crystalline texture indicate an amorphous structure. Theses factors are believed to have contributed to the reduced strength of the soil at increased lime content (i.e. 13%).

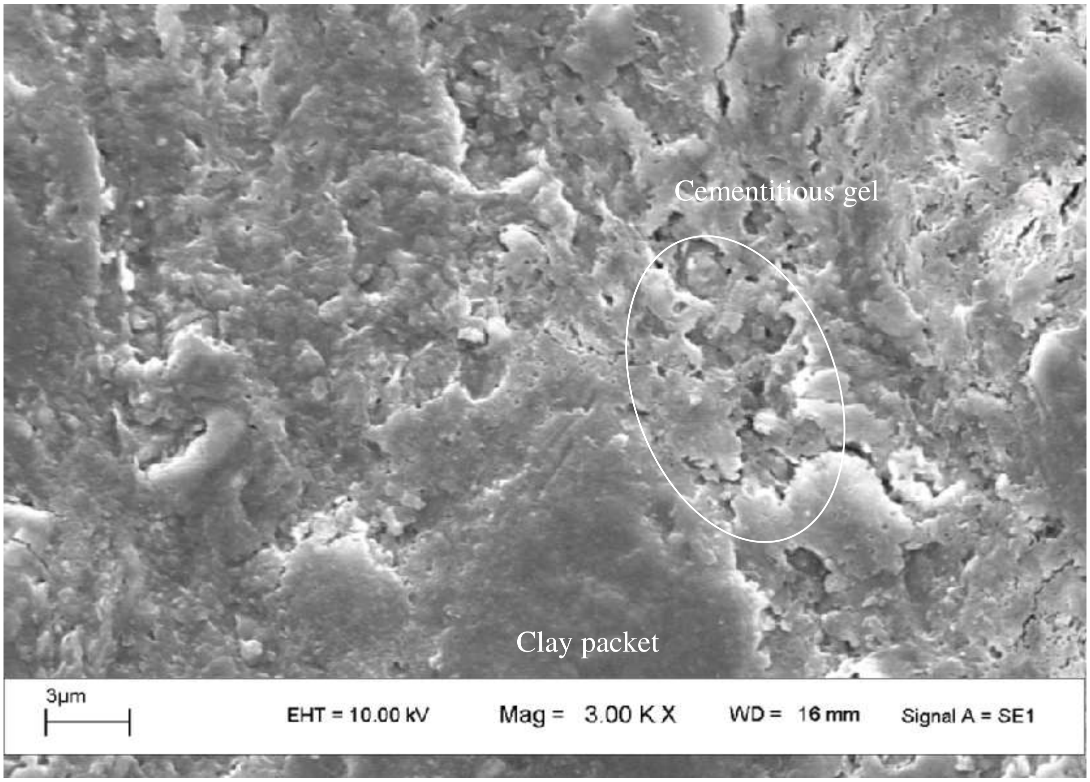

Fig. 8.40 SEM micrograph of 100%ES + 1%Lime (Compacted sample, M = 3 KX)

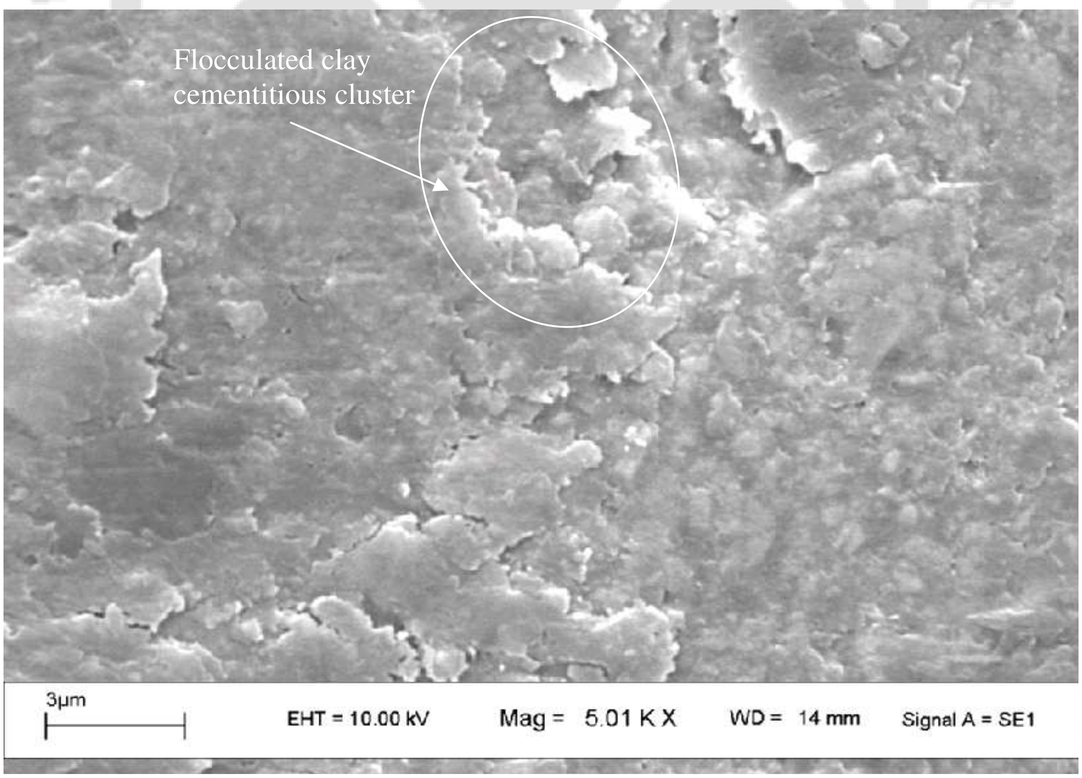

Fig. 8.41 SEM micrograph of 100%ES + 3% Lime (Compacted sample, M = 5.01 KX)

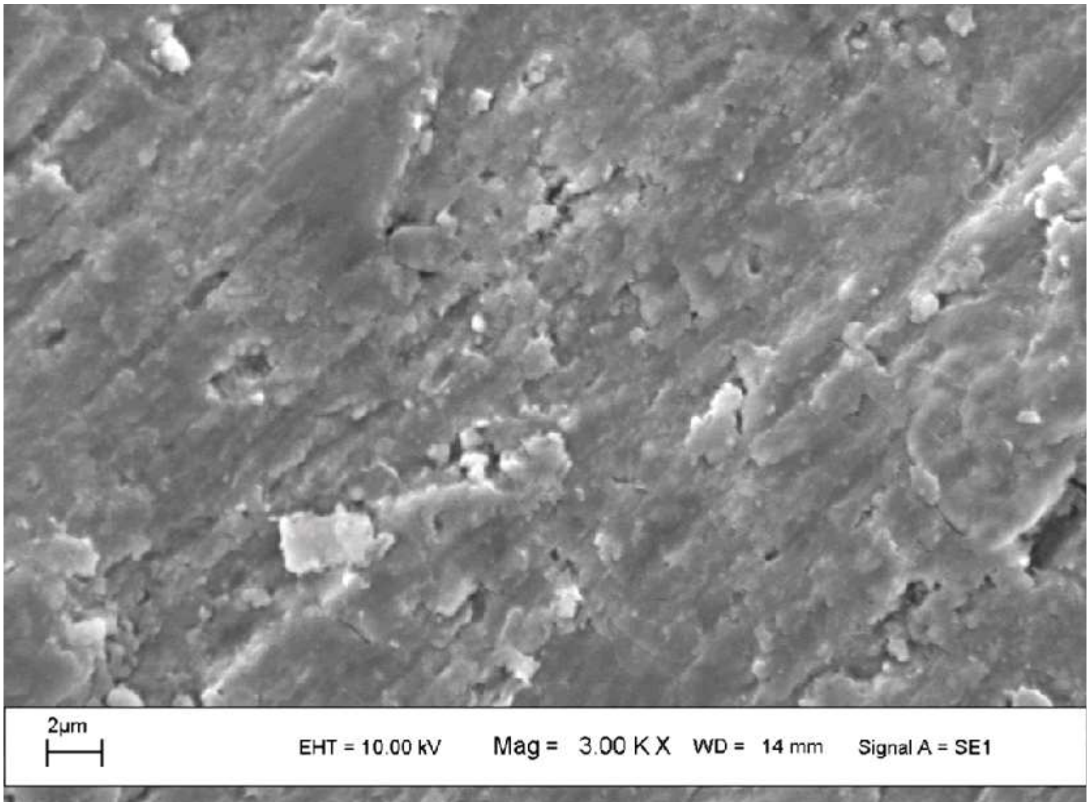

Fig. 8.42 SEM micrograph of 100%ES + 5% Lime (Compacted sample, M = 3 KX)

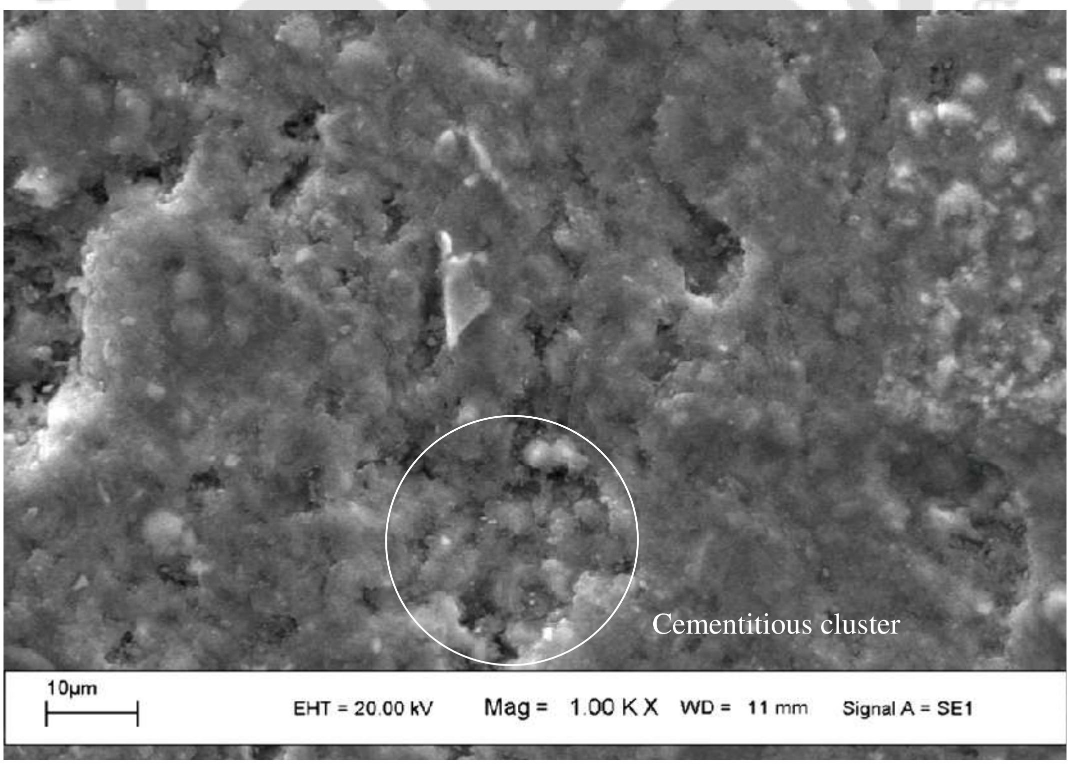

Fig. 8.43 SEM micrograph of 100%ES + 9% Lime (Compacted sample, M = 1 KX)

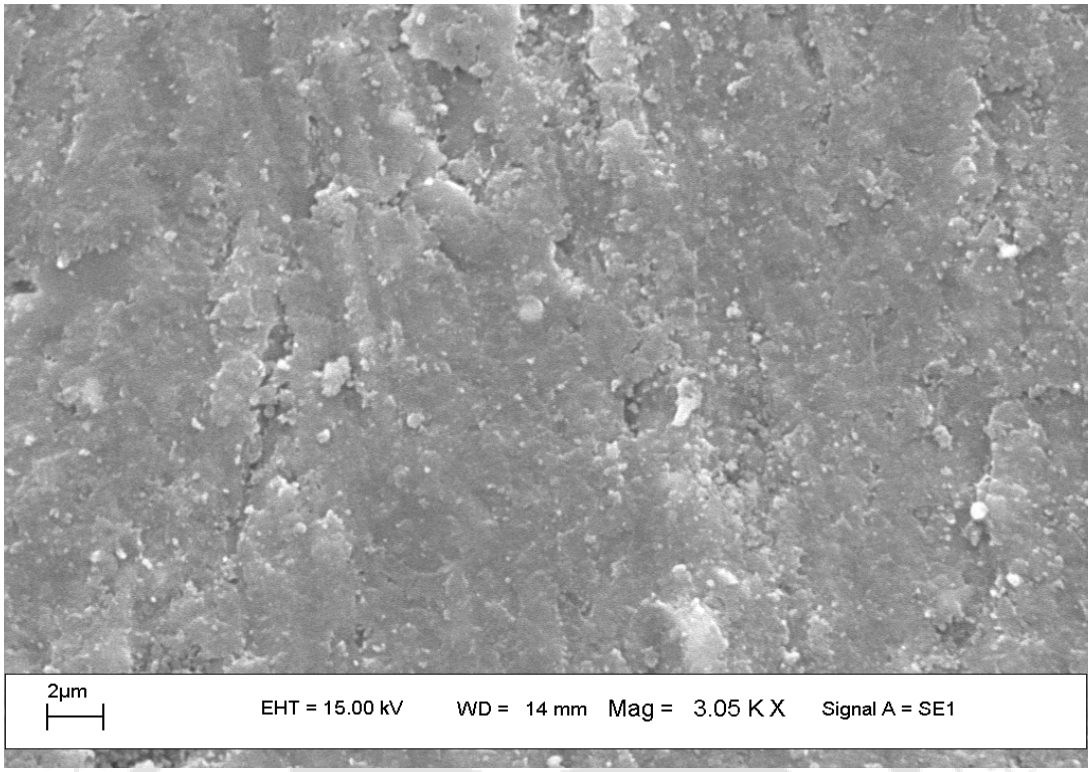

Fig. 8.44 SEM micrograph of 100%ES + 13% Lime (Compacted sample, M = 3 KX)

Fig. 8.45 shows the micrograph of the expansive soil-residual soil mix having 60%ES and 40%RS. The compacted soil shows large clay clods with intermittent voids. The micrographs of this soil treated with varied lime contents are depicted in Figs. 8.46-8.50. Nearly similar microcharacteristics as that in the previous case (i.e. 100%ES) are observed herein, however, the features have a greater prominence. This is attributed to the increased silica content (due to addition of residual soil) that facilitates pozzolanic reaction and hence better cementation. Besides the relatively coarse grains in the residual soil give rise to better cementation. Indeed the micrograph of the soil with 9% lime depicted in Fig. 8.49 shows very large size particles effectively held by wide spread cementation. With relatively less quantity of lime (i.e. 1%) the micrograph (Fig. 8.46) shows a layered structure indicating that there has taken place flocculation and aggregation in the soil mass. With very large quantity of lime (i.e. 13%) the micrograph (Fig. 8.50) shows an amorphous texture. The absence of large size crystals is attributed to the dissolution of soil particles under alkaline environment in the pore fluid leading to formation of gels.

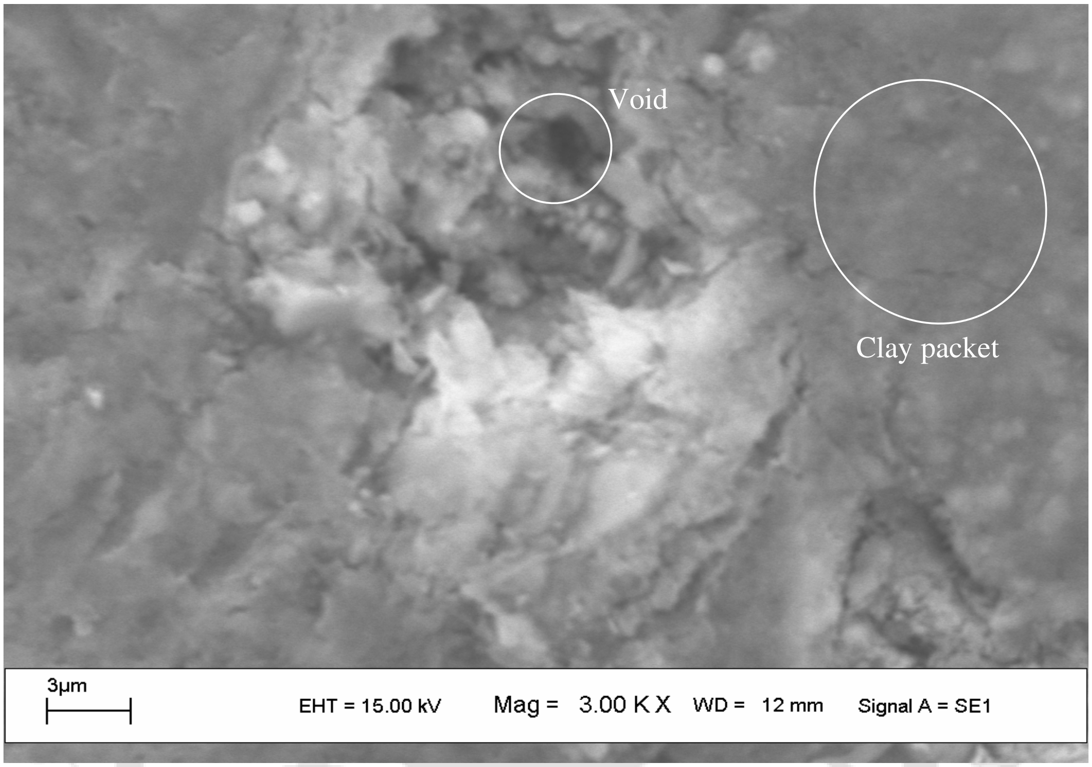

Fig. 8.45 SEM micrograph of 60%ES+40%RS (Compacted sample, M = 3.00 KX)

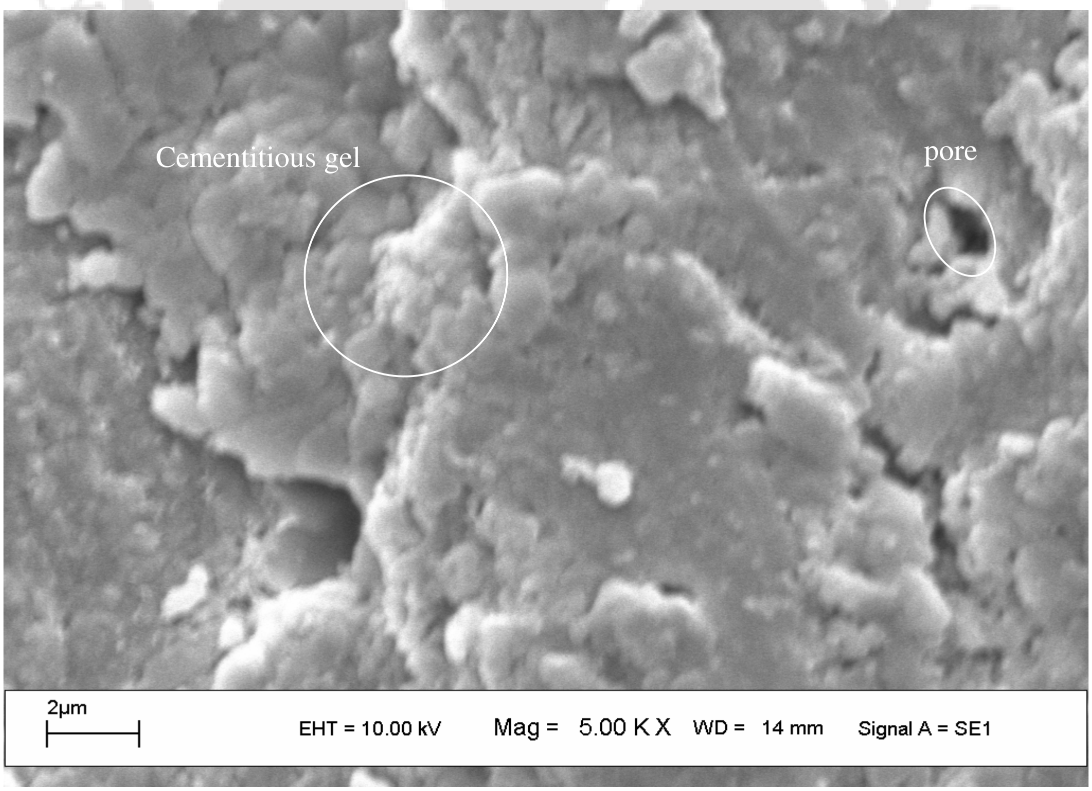

Fig. 8.46 SEM micrograph of 60%ES+40%RS+ 1% Lime (compacted sample, M = 5KX)

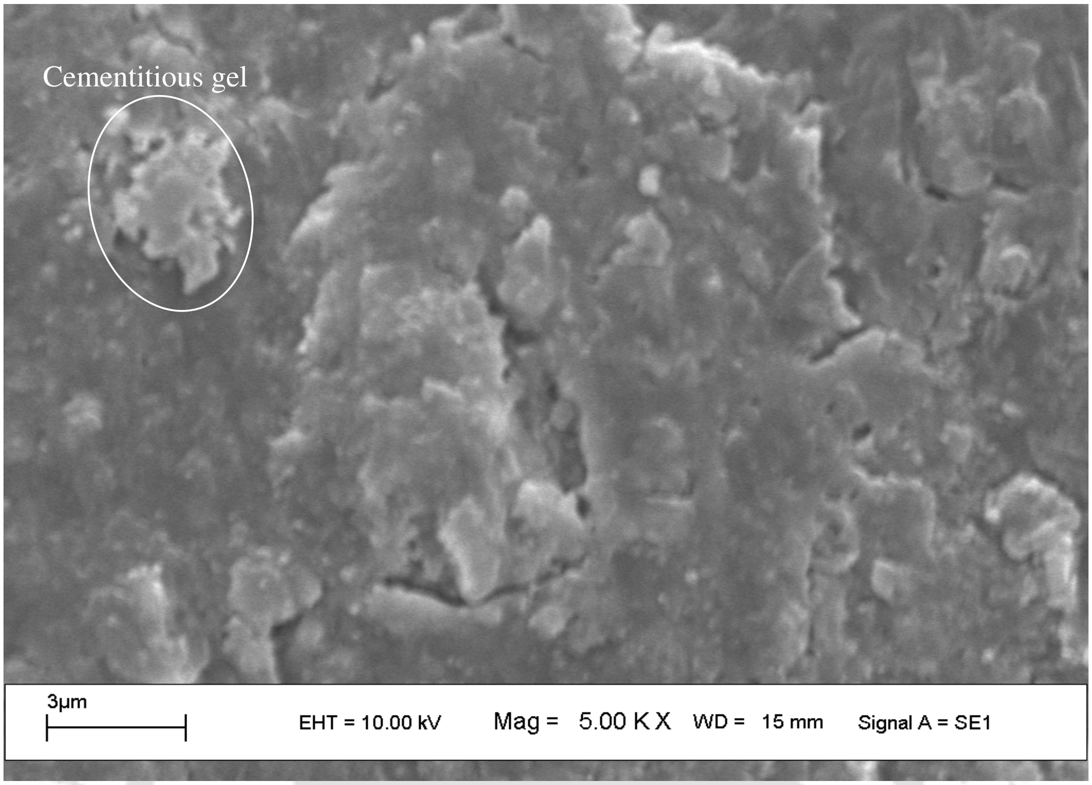

Fig. 8.47 SEM micrograph of 60%ES + 40%RS+3% Lime (Compacted sample, M = 5KX)

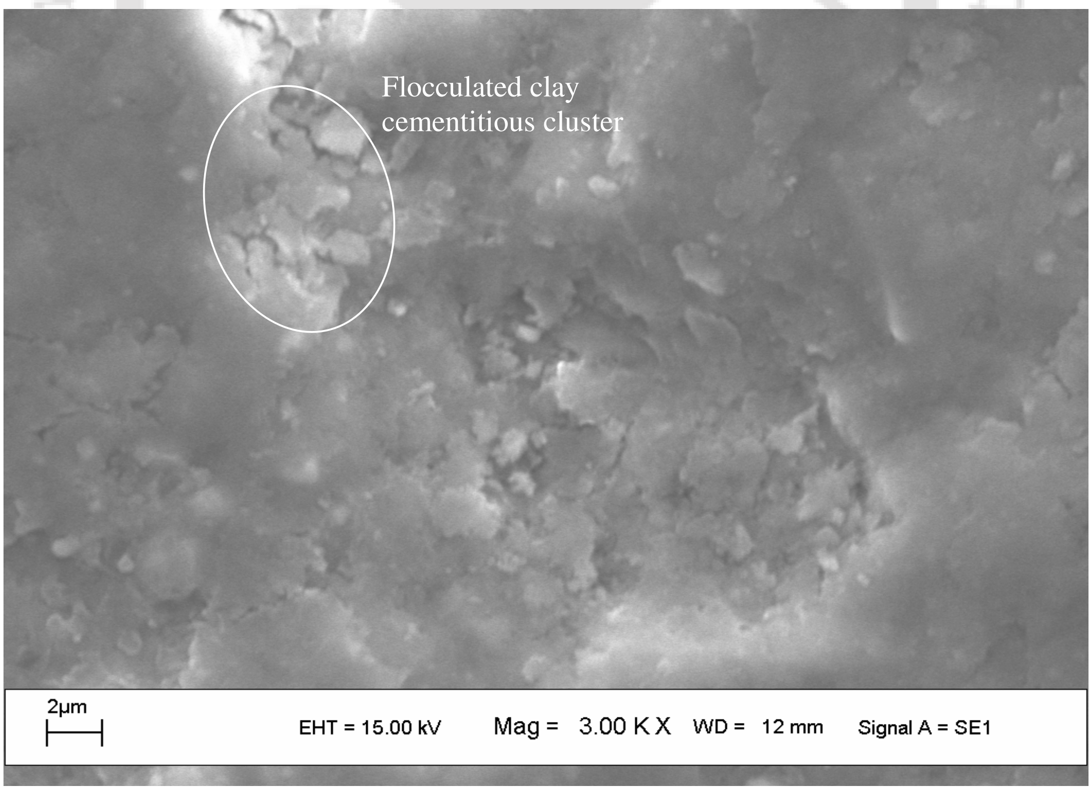

Fig. 8.48 SEM micrograph of 60%ES + 40%RS + 5% Lime (Compacted sample, M = 3.00KX)

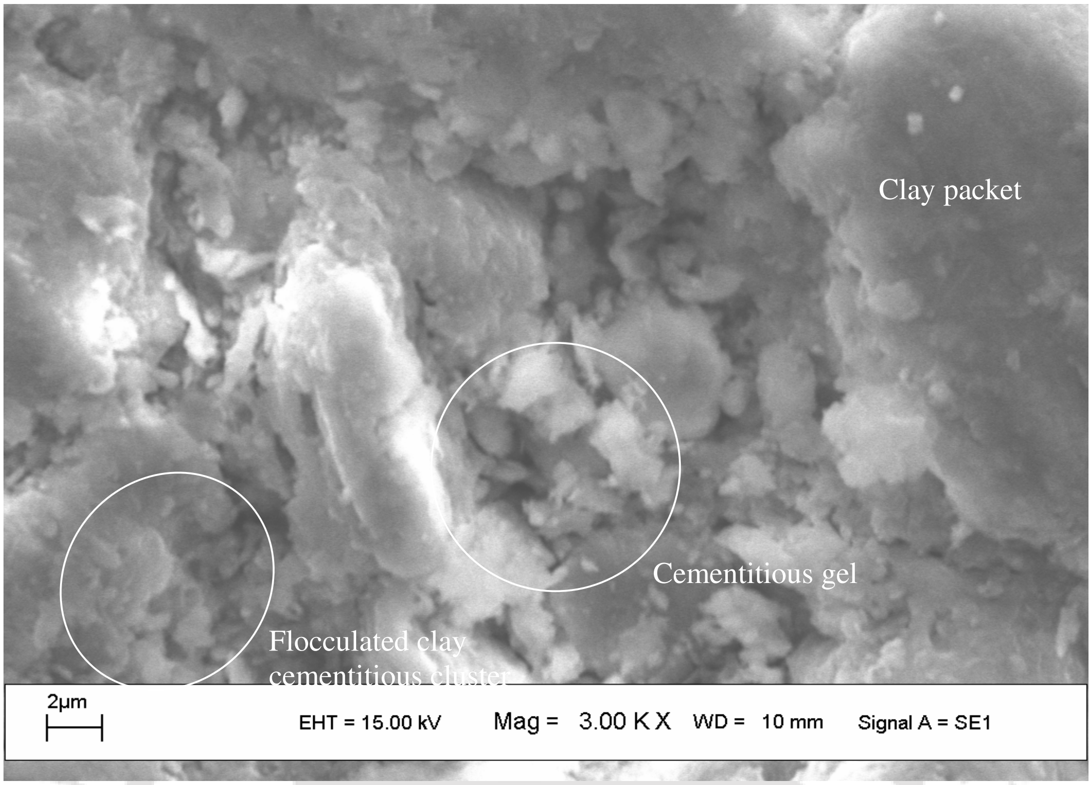

Fig. 8.49 SEM micrograph of 60%ES + 40%RS + 9% Lime (Compacted sample, M =3.00KX)

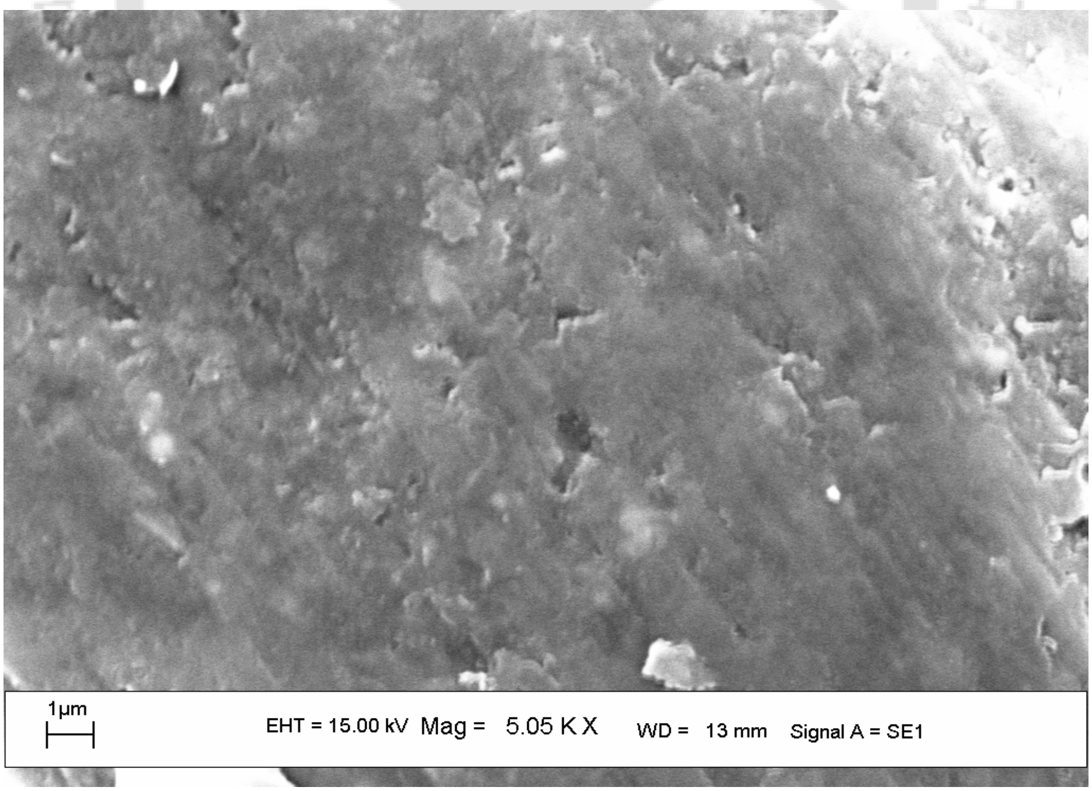

Fig. 8.50 SEM micrograph of 60%ES + 40%RS + 13% Lime (Compacted sample, M = 5.05KX)

The micrograph of dry powder residual soil depicted in Fig. 8.51 shows a relatively coarse and plain texture as compared to the expansive soil. The micrograph of the soil compacted at OMC, however, shows formation of clay packets (Fig. 8.52). The ragged appearance observed in the micrograph of lime treated residual soil (1%, 3%, 5% lime; Figs. 8.53, 8.54, 8.55) indicates that the soil particles are attacked by lime at their edges. Similar such behaviour was also noted by Bell (1996). At 5% lime the micrograph shows well cemented structures with crystalline nature. This is in agreement with maximum strength gain due to lime treatment with 5% lime content in residual soil as observed from UCS tests reported in chapter 7.

The UCS tests have shown that with further increase in lime content the strength of the residual starts reducing. Indeed Fig. 8.56 depicting the micrograph for 9% lime content shows formation of patches of reaction products and the crystalline structure has diminished. This phenomenon has further increased with 13% lime (Fig. 8.57). This is attributed to the increased silica gel formation through dissolution of the predominantly silica rich residual soil in an increased alkaline environment due to increased lime content.

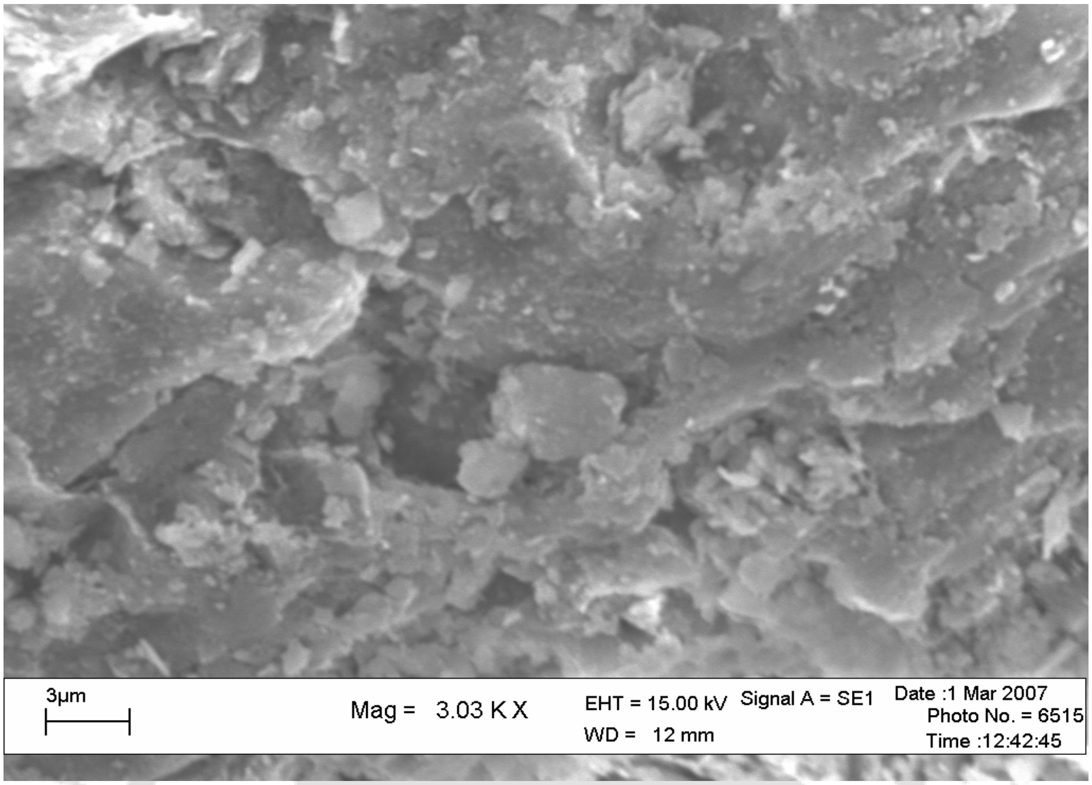

Fig. 8.51 SEM micrograph of residual soil (Powder sample, M = 3.03KX)

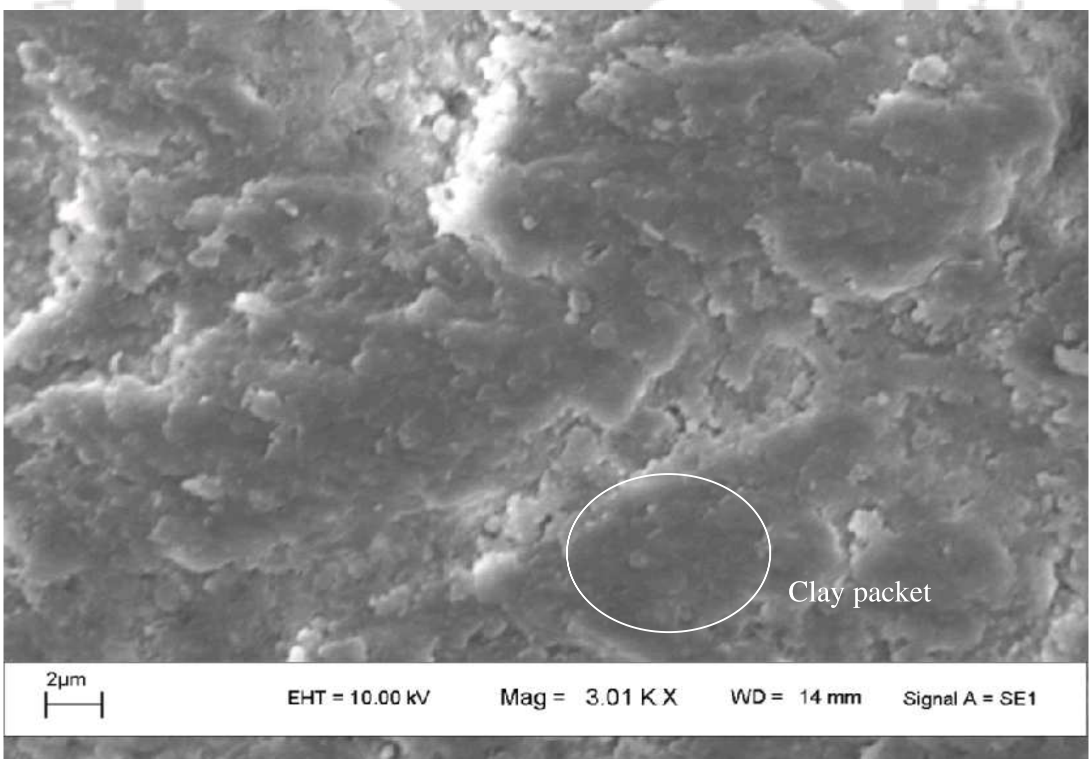

Fig. 8.52 SEM micrograph of residual soil (compacted sample, M = 3.01 KX)

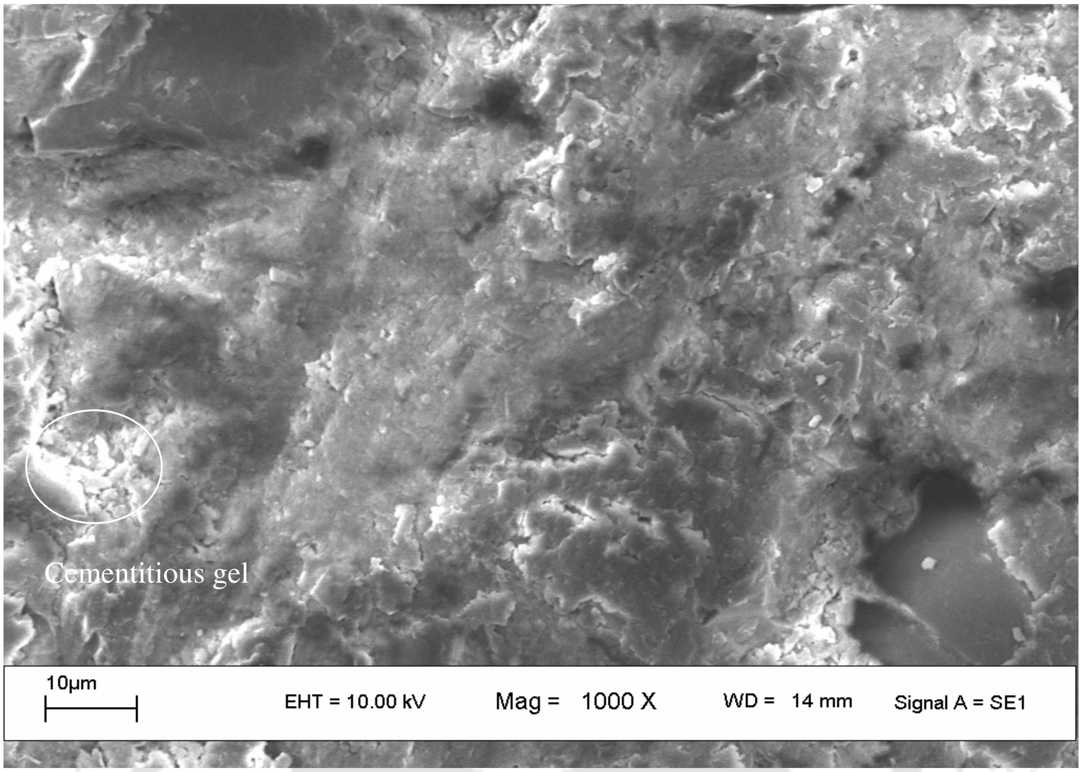

Fig. 8.53 SEM micrograph of 100%RS + 1% Lime (Compacted sample, M = 1000 X)

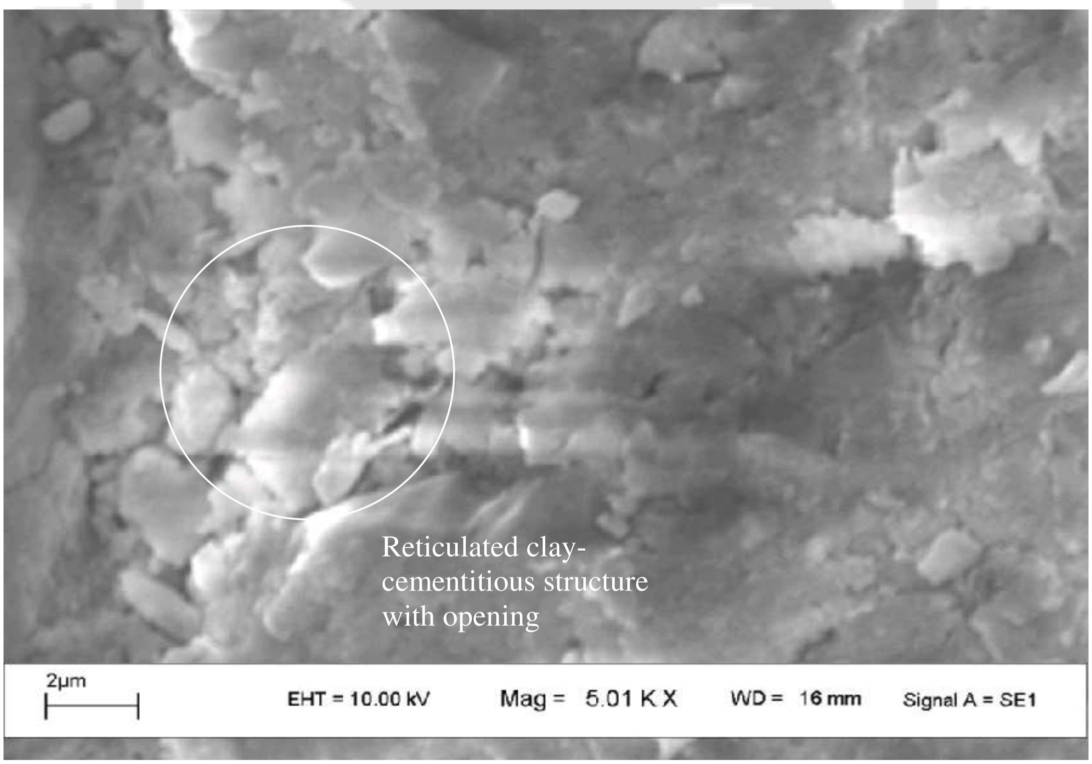

Fig. 8.54 SEM micrograph of 100%RS + 3% Lime (compacted sample, M = 5.01KX)

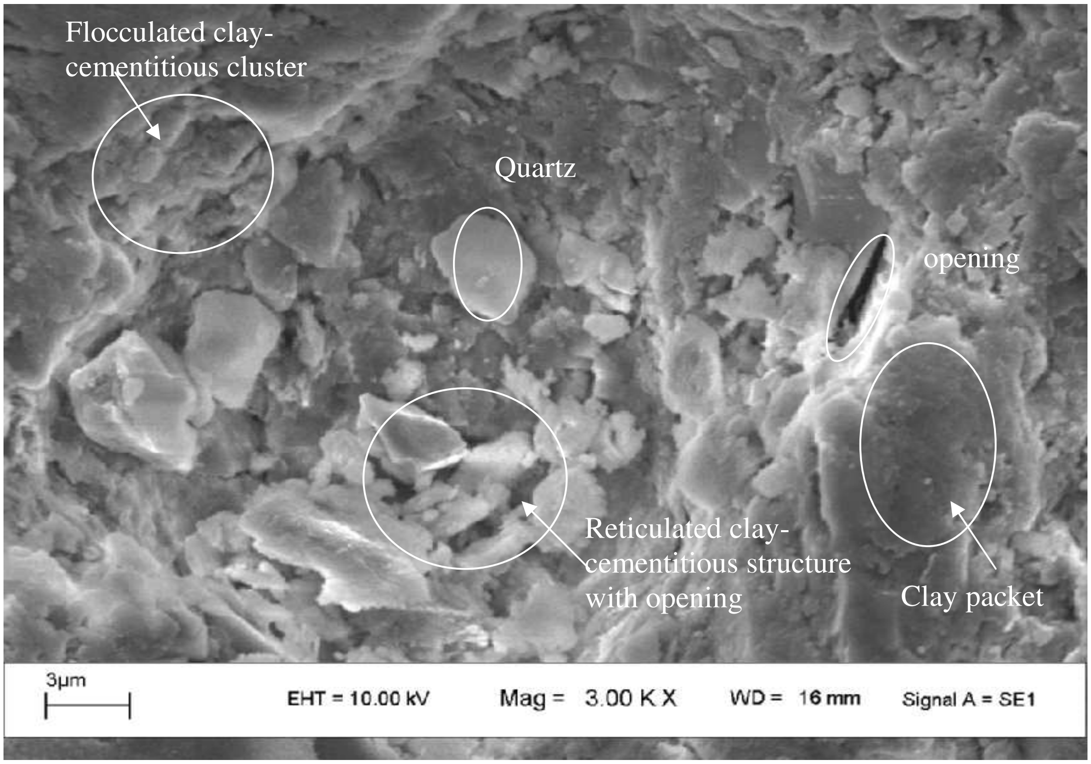

Fig. 8.55 SEM micrograph of 100%RS + 5% Lime (Compacted sample, M = 3KX)

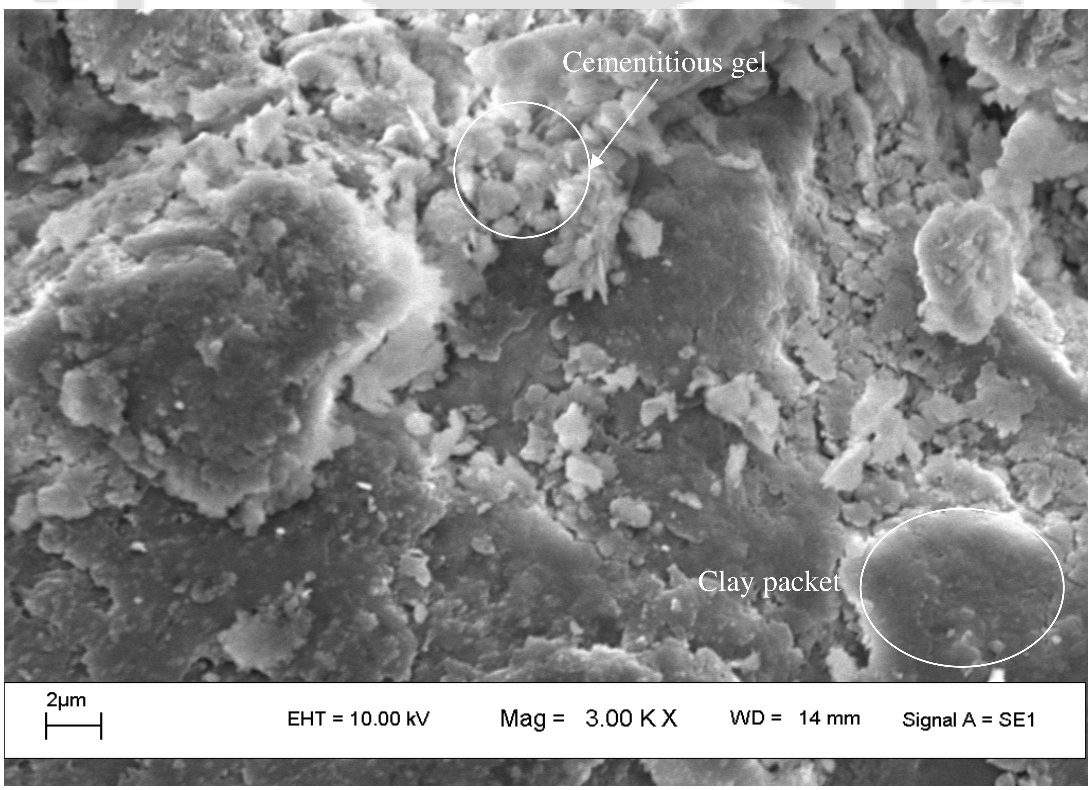

Fig. 8.56 SEM micrograph of 100%RS + 9% Lime (compacted sample, M = 3KX)

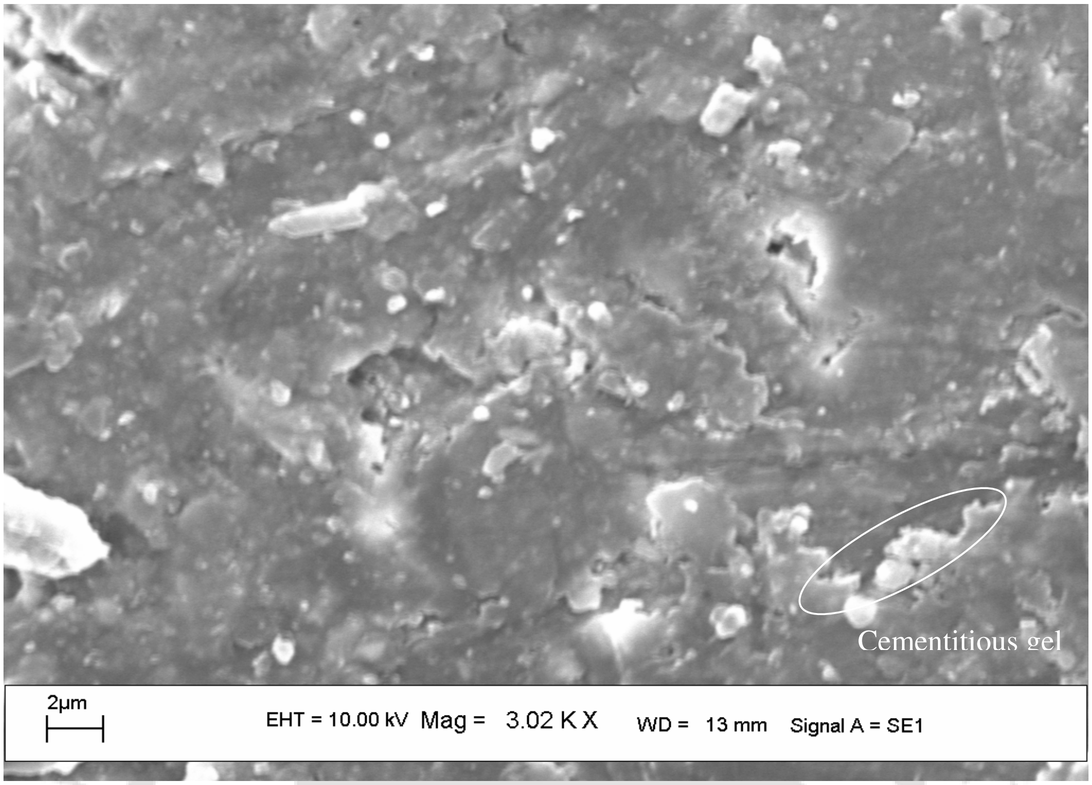

Fig. 8.57 SEM micrograph of 100%RS + 13%Lime (Compacted sample, M = 3.02KX)

8.4 SUMMARY

In this chapter the formation of new products and change in fabric of the soils due to lime treatment are studied through EDX, XRD and SEM. The results indicate formation of cementitious compounds such as; CAH, CSH, Gyrolite, CASH, CAOH etc. The SEM micrographs bring out the microstructural changes due to lime treatment with varied lime content and curing period.

CHAPTER 9

SUMMARY AND CONCLUSIONS

9.1 SUMMARY

Natural expansive soils have been found in several countries across the world. 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 various treatment methods to stabilize the expansive soils. Among 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 these two techniques i.e. mechanical stabilization and lime stabilization, is 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.

In the present investigation six different series of experiments under a systematically planned scheme, have been carried out, to study the physico-chemical and engineering behaviour of residual soil-lime treated expansive soil. The details of these test series are presented in Chapter 3 (Table 3.2). In each test series expansive soil was admixtured with residual soil of different percentage (i.e. 0%, 20%, 40%, 60%, 80% and 100%). All these six soil samples were added with different percent of lime (0, 1, 3, 5, 9 and 13% 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, 21 and 28 days) before the tests were carried out. Tests are carried out to study the plasticity characteristics, compaction behaviour, swell-shrink response, strength behaviour and micro-structural characteristics. Based on the results obtained, the following conclusions can be made on the behaviour of residual soil-lime treated expansive soils.

9.2 CONCLUSIONS

1. The liquid limit, irrespective of the clay soils, initially reduces with increase in lime content. Beyond about 3% of lime the liquid limit for expansive soils practically remains unchanged. But for soils having larger fraction of silica rich residual soils the liquid limit has once again shown increasing trend at higher lime content. This trend is more prominent at higher curing period. The initial reduction is attributed to the reduction in the thickness of double layer due to increased electrolyte concentration in the pore fluid. For silica rich soils with increased lime content there takes place pozzolanic reactions forming gelatinous materials. These gel materials hold a very large amount of water onto themselves leading to increased liquid limit.

2. Plastic limit increases with increase in lime content. With reduced thickness of diffused double layer the associated water grows more viscous due to increased charge concentration. This leads to increase in inter particle shear resistance leading to increased plastic limit. However beyond 5% lime content, it does not change much, except incase of silica rich residual soils which is attributed to the gelatinous products formed. Therefore 5% lime content can be considered as lime fixation point that can provide substantial increase in workability of the soil.

3. In general the plasticity index reduces till 5% lime content except for residual soil where it shows an increasing trend beyond 3% lime. The soils, which are high plastic clay (CH), just with 3% lime have changed to silt (CM).

4. Lime treatment flattens the compaction curve indicating that the targeted density can be achieved over a wider range of moisture content thereby bringing about an increase in the workability of the soils.

5. With increase in percentage of lime added, the maximum dry density continues to reduce till 3% lime content, beyond which it again increases with increase in lime content. Based on the present test data it can be said that to obtain better compaction density relatively higher percentage of lime (more than 5%) should be added to clay soils.

6. The present study shows that for lime treated soils, among the different index properties, plastic limit is the one that correlates well with the compaction parameters (maximum dry density and optimum moisture content). The following correlation equations are proposed to predict the compaction parameters

$$\gamma_{d\max} = 35.2(PL)^{-0.25}$$

$$OMC = 6(PL)^{0.4}$$

7. The time-swell response of both untreated soils and lime treated soils follow rectuangular hyperbola relationship. Therefore for high plastic soils with relatively less quantity of lime, wherein swelling continues for a long period of time, the maximum swelling can be predicted from the initaial readings taken over a short interval of time, using the rectangular hyperbolic model.

8. While the free swell index of lime treated soils correlates well with the corresponding liquid limit (FSI = 0.03 × LL , R^2 = 0.92) its correlation with oedeometer swell potential is relatively poor (S_p = 3.83 × FSI, R^2 = 0.85). the reason is both in liquid limit test and free swell test there being large quantity of water, the intergranular contact is poor. Therefore the lime induced cementation

effect is week. As nearly similar mechanical condition prevails in both the tests the test parameters (i.e. FSI and LL) correlate well. However, in Oedometer swell test the soil matrix being compact the lime induced cementation is well formed that tends to hold the particles, which is in contrast to the mechanism prevailing in the free swell test. Therefore, the Oedometer swell potential does not correlate well with the free swell index. Nevertheless there are some mechanisms such as depression of diffuse double layer and flocculation, which are common to both the tests.

9. Irrespective the plasticity of clay soils, the swell potential initially reduces, with increased percentage of lime added, to a practically negligible value beyond which it once again increases with increase in lime content. The initial reduction in swell potential is due to the depression of the diffuse double layer caused by increased electrolytic concentration in pore water through addition of lime. The increase in swell at higher lime content is due to formation of cementitious gel through pozzolanic reactions.

10. The lime content at which swelling starts increasing is about 5% for soils having higher fines fraction however for soils having higher percentage of coarse fraction it is 9%. With coarse particles the intergranular voids are larger. Therefore the cementatitious gel formed, initially, is contained within the void space that it does not contribute to swelling. Only with large quantity of lime when the gel formed is large in quantity that it fills the intergranular voids the external swelling begins. Hence it can be said that apart from plasticity characteristics the grain size distribution too plays a significant role on the swelling behaviour of lime treated soils.

11. The swell pressure after an initial decrease continues to increase high with increased percentage of lime. This contradicting behaviour is attributed to the cemented bonds formed during swell period. To bring the soil specimen to its original height the cemented bonds are to break which needs surcharge pressure of very high magnitude that apparently gives increased swell pressure. Hence the concept of swell pressure obtained through the swell-reload method can not be applied to the lime treated soils.

12. Effect of lime increases the shrinkage limit irrespective of swelling characteristics of the soil. However the percentage of increase is greater in case of high swelling expansive soil than the low swelling residual soil. The shrinkage limit of lime treated expansive soil is as high as 100%.

13. Initially the shrinkage limit increases sharply till about 5% lime content beyond which the rate of increase has reduced, particularly at lower curing period. The initial increase is attributed to aggregation of particles. Subsequent increase is attributed to flocculation of the particles due to increased electrolytic concentration in the pore fluid. Indeed the relationship between shrinkage limit and liquid limit clearly establishes such fabric change in the soil due to lime.

14. The shrinkage limit of lime treated soils reduces with increased percentage of residual soil. With increased percentage of coarse grained residual soil the clay content in the soil specimen reduces. As clay fraction is the major source for flocculation with its reduction the flocculation induced shrinkage reduction has reduced. Hence it can be said that lime is more effective in reducing shrinkage potential for clayey soils.

15. Similar to the untreated soils the lime treated soils too undergo three stage shrinkage i.e. initial, primary and residual, maximum shrinkage taking place under primary shrinkage. This indicates that the general mechanism of shrinkage for untreated soil too prevails in case of lime treated soils.

16. The void ratio versus water content relationship of lime treated soils during shrinkage is generally linear.

17. The magnitude of primary shrinkage reduces substantially till 5% lime content beyond which further reduction is marginal. Hence it can be said that the optimum lime content giving maximum reduction in shrinkage is about 5%.

18. The shrinkage responses grow flatter with increased lime content indicating that lime reduces rate of shrinkage and thereby reduces the possibility of cracking of the soil.

19. The strength of a soil is dependent on the percentage of fines present in it. With increase in fines content the strength of the soil initially reduces however with further increase in fines content it starts increasing again to be nearly constant at very high fines content. A relatively less quantity of fines remains content within the voids formed by coarse grain skeleton that it remains under compacted therefore in presence of water behaves as a lubricant leading to reduced strength of the soil mass. With increased fines content it fills the voids forming a compact structure that gets effectively compacted leading to increased strength.

20. The strength and stiffness of expansive soil can be increased by adding residual soil in combination with lime. With increase in percentage of residual soil and lime the strength continues to increase and reaches a maximum value and thereafter starts decreasing. It is observed that the soil mix having 60% expansive soil and 40%

residual soil with 9% lime by weight of total soil, gives maximum improvement in strength. Further increase in percentage of residual soil increases silica content that forms silica gel in an increased alkaline environment due to increased lime content in the pore water. The silica gel consists of large volume of micro pores i.e. 28% of volume of gel (Neville and Brooks, 2004) that reduces the strength of soil. When formed in large quantity the strength gain due to cementation is substantially counteracted by the strength reduction due to gel pores leading to overall reduction in strength of the soil.

21. The X-ray diffractograms show that several new peaks are formed due to lime treatment indicating that new compounds are formed due to soil-lime reaction. Besides peaks of original elements have shown visible reduction which establishes the action of lime on these elements. The major cementitious compounds formed are; gyrolite, calcium silicate hydrate, calcium aluminum silicate hydroxide hydrate etc. The increase in d-spacing of these cementitious compounds beyond a certain lime content indicates formation of a relatively porous structure at higher lime content that gives rise to reduced strength and increased swelling, as has been observed in the UCS tests and oedometer swell tests.

22. The SEM micrographs indicate an increase in particle size of the soil due to lime treatment. This is attributed to flocculation and aggregation of clay particles due to increased ion concentration in the pore water caused by the lime. Cementitious gels are observed to have been formed in patches in the expansive soil while in residual soil they are observed in ragged pattern. This indicates that the fine expansive soils behave as ingredients in the lime induced pozzolanic reactions while the coarse residual soils are attacked by lime at their edges.

23. At higher lime content (i.e. >9% for expansive soil, >5% for residual soil) the micrographs show relatively uniform structure with voids of small size distributed over. This is attributed to increased volume of gel formed under alkaline environment. Besides, the micrographs for these cases indicate an increased amorphous structure. The increased voids and reduced crystalline structure are believed to have contributed to the reduced strength of soil observed at higher percentage lime content.

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