

GEOTECHNICAL BEHAVIOUR OF A LIME TREATED EXPANSIVE SOIL

A Thesis

by

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**Submitted to the Department of Civil Engineering, Bangladesh University
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of the requirements for the degree
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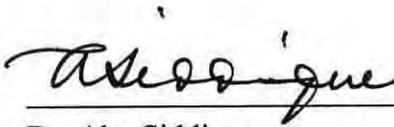
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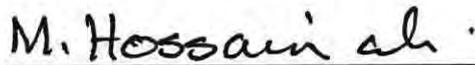
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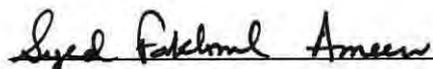
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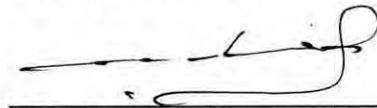
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ABSTRACT

In this research, geotechnical properties of a selected lime treated expansive soil have been investigated. For this purpose, initially, index and swelling properties of untreated samples collected from six different locations of Rajendrapur Cantonment, Gazipur were determined. The relative degree of expansion of the six soil samples has been evaluated in order to select the most expansive soil used for subsequent lime stabilisation. On the basis of the recommended criteria of expansive soil, the soil sample collected from POL store area of Rajendrapur Cantonment, Gazipur was found to be the most expansive soil. Samples of this soil (liquid limit = 56, plasticity index = 43) were stabilised with five different lime contents of 3%, 6%, 9%, 12% and 15%. Index, swelling and engineering properties of the lime treated expansive soil were evaluated in order to examine the effect of lime stabilisation on geotechnical properties of this expansive soil.

Compared with the untreated sample, liquid limit of the stabilised samples initially decreases with the increment of lime content up to 6% and then increased with increasing lime content. Plastic limit and shrinkage limit of the treated samples increased considerably with increasing lime content. Compared with the untreated sample, plasticity index, linear shrinkage, volumetric shrinkage and shrinkage ratio of lime treated expansive soil decrease markedly due to increase in lime content.

Swelling test results show that free swell and free swell index of the stabilised samples decrease significantly with increasing lime content. Swelling pressure and swelling potential of the treated samples also reduce markedly. Compared with the untreated sample, volume change of the stabilised samples from air-dry to saturated condition decreased considerably. The experimental results on the influence of lime stabilisation on swelling properties clearly demonstrate that lime can be considered a very effective additive to reduce various swelling properties of an expansive soil.

It has been found that with the increase in lime content, maximum dry density reduces while optimum moisture content increases. Compared with the untreated sample, unconfined compressive strength of the treated samples increases significantly, depending on the lime content and curing age. It was found that unconfined compressive strength of samples of this expansive soil treated with 6%, 9%, 12% and 15% lime fulfilled the requirements for its use for upgrading heavy clays to sub-base material quality type, as proposed by Ingles and Metcalf (1972). It was found that strength

development index increases with increasing curing age and lime content. It has been observed from the present investigation that long-term curing has profound influence on the gain in strength. The effect of long-term curing on the increase in unconfined compressive strength has been found to be more pronounced when samples were treated with higher lime contents. Unconfined compressive strength of sample treated with 15% lime and cured for 16 weeks was about 8.4 times higher than the strength of the untreated sample. It was also found that unconfined compressive strength of samples treated with different lime contents and cured for different ages increased with the increase in moulding water contents. Unconfined compressive strength was found to be the maximum and minimum at moulding moisture contents of wet side of optimum and dry side of optimum, respectively.

CBR tests performed on treated and untreated samples indicated that CBR-values of the treated samples at all levels of compaction increase considerably with increasing lime content. At any particular lime content, CBR increases significantly with the increase in compaction energy. Compared with the untreated sample, CBR-values of treated samples increased by 4.75 to 8.75 times due to increase in lime content from 3% to 15%.

The flexural stress versus deflection curves has been found to be approximately linear. Compared with the untreated sample, flexural strength and flexural modulus of the stabilised samples increase significantly, depending on the lime content and curing age. Flexural strength and flexural modulus of samples of the expansive soil cured for 8 weeks increased by about 66% to 209% and 37% to 76%, respectively, due to increase in lime content from 3% to 15%.

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CONTENTS

	Page
ABSTRACT	i
ACKNOWLEDGMENTS	ii
CONTENTS	iv
NOTATIONS	x
CHAPTER 1 INTRODUCTION	1
1.1 General	1
1.2 Statement of the Problem	5
1.3 Objectives of the Present Research	6
1.4 The Research Scheme	7
1.5 Geology of the Project Area	9
1.5.1 Madhupur Clay Residuum	9
1.6 Thesis Layout	11
CHAPTER 2 LITERATURE REVIEW	12
2.1 General	12
2.2 Occurrence of Expansive Soils	12
2.3 Mechanisms of Swelling	15
2.4 Factors Affecting Swelling	15
2.4.1 Compositional Factors	16
2.4.1.1 Amount and Type of Minerals	16
2.4.1.2 Structure of Clay Minerals	16
2.4.1.3 Exchangeable Cations	16
2.4.1.4 Shape and Size Distribution of Particles	16
2.4.1.5 Pore Water Composition	17

2.4.2 Environmental Factors	17
2.4.2.1 Moisture Content	17
2.4.2.2 Density	18
2.4.2.3 Confining Pressure	18
2.4.2.4 Temperature	18
2.4.2.5 Fabric	18
2.5 Engineering Properties of Expansive Soil	18
2.5.1 Swelling Potential	19
2.5.1.1 Plasticity Index and Swelling Potential	19
2.5.1.2 Clay Content and Swelling Potential	20
2.5.1.3 Activity and Swelling Potential	21
2.5.2 Free Swell	24
2.5.3 Differential Free Swell	24
2.5.4 Linear Shrinkage	25
2.5.5 Percentage Linear Free Swell	25
2.5.6 Swelling Pressure	26
2.5.6.1 Initial Density and Swelling Pressure	26
2.5.6.2 Initial Moisture Content and Swelling Pressure	28
2.5.6.3 Degree of Saturation and Swelling Pressure	31
2.5.6.4 Curing Time and Swelling Pressure	31
2.5.6.5 Surcharge Pressure and Swelling Pressure	31
2.5.6.6 Layer Thickness and Swelling Pressure	33
2.5.6.7 Relations of Swelling Pressure with Other Properties	33
2.6 Identification and Classification of Expansive Soils	36
2.6.1 USBR Classification System	36
2.6.2 Activity Method	36
2.6.3 Chen's Method of Classification	36
2.6.4 Classification Based on Linear Shrinkage	39
2.6.5 Indian Classification System	39

2.6.6 United States Army Engineers Waterways Experimental Station Classification System	39
2.6.7 Classification Based on Swelling Potential	42
2.7 Foundations on Expansive Soil	42
2.7.1 Drilled Pier Foundations	43
2.7.2 Footing Foundation	43
2.7.2.1 Continuous Footing	44
2.7.2.2 Pad Foundations	44
2.7.2.3 Footing on Selected Fill	44
2.7.2.4 Mat Foundation	44
2.7.3 Slabs on Expansive Soils	45
2.7.3.1 Slab-on-Ground	45
2.7.3.2 Structural Floor Slabs	46
2.7.3.3 Floating Slabs	46
2.8 Elimination of Swelling	46
2.8.1 Replacement by Non-Swelling Soil	46
2.8.2 Moisture Control	47
2.8.2.1 Horizontal Moisture Barriers	47
2.8.2.2 Vertical Moisture Barrier	49
2.8.2.3 Subsurface Drainage	49
2.8.2.4 Surface Drainage	50
2.8.3 Prewetting	51
2.8.4 Soil Stabilisation	52
2.9 Lime Stabilisation	54
2.10 Materials for Lime Stabilisation	54
2.10.1 Lime	54
2.10.2 Soil	55
2.10.3 Water	56

2.11 Mechanisms of Lime Stabilisation	56
2.11.1 Base Exchange and Flocculation	57
2.11.2 Cementation	57
2.11.3 Carbonation	58
2.12 Factors Affecting Lime Stabilisation	59
2.12.1 Soil Characteristics	59
2.12.1.1 Type of Soil	59
2.12.1.2 Organic Matter Present in the Soil	61
2.12.2 Lime Content	61
2.12.3 Mixing and Compaction Procedure	66
2.12.3.1 Compactive Effort	66
2.12.3.2 Compaction Delay Time	67
2.12.4 Curing Time and Curing Conditions	69
2.13 Properties of Lime Stabilised Soil	71
2.13.1 Plasticity and Shrinkage Properties	71
2.13.2 Moisture-Density Relations	73
2.13.3 Unconfined Compressive Strength	74
2.13.4 California Bearing Ratio (CBR)	82
2.13.5 Tension and Flexural Properties	82
2.13.6 Permeability	85
2.14 Applications of Lime Stabilisation	87
CHAPTER 3 LABORATORY INVESTIGATIONS	88
3.1 General	88
3.2 Sampling and Collection of Soil Samples	88
3.3 Laboratory Testing Programme	89
3.4 Procedures for Determining Physical and Index Properties of Untreated Soils	91
3.5 Index Property Tests on Stabilised Soil Samples	92

3.6 Swelling Tests on Untreated and Treated Soil Samples	92
3.6.1 Free Swell	92
3.6.2 Differential Free Swell	93
3.6.3 One-Dimensional Swelling Pressure Test	93
3.6.4 Swelling Potential Test	94
3.6.5 Volume Change Test	95
3.7 Compaction Test	95
3.8 Unconfined Compressive Strength Test	96
3.8.1 Preparation and Mixing of Soils	96
3.8.2 Mould for Compression Test	97
3.8.3 Compaction of Samples	97
3.8.4 Curing of Samples	99
3.8.5 Compression Test	99
3.9 California Bearing Ratio (CBR) Test on Compacted Untreated and Stabilised Sample	100
3.9.1 Preparation and Mixing of Soils	100
3.9.2 Compaction of Samples	100
3.9.3 Soaking of Sample	100
3.9.4 Bearing Test	102
3.10 Flexure Test Using Simple Beam with Third-Point Loading System	102
3.10.1 Preparation and Mixing of Soils	102
3.10.2 Mould for Flexure Test	104
3.10.3 Moulding and Curing of Sample	104
3.10.4 Flexural Strength Test	106

CHAPTER 4 RESULTS AND DISCUSSIONS	109
4.1 General	109
4.2 Index and Swelling Properties of Untreated Soils	109
4.3 Assessment of Degree of Expansion of Untreated Soil Samples and Selection of Expansive Soil for Lime Stabilisation	112
4.4 Effect of Lime Stabilisation on Plasticity and Shrinkage Characteristics	113
4.5 Effect of Lime on Swelling Properties of Expansive Soil	120
4.6 Moisture- Density Relations of Untreated and Treated Samples	126
4.7 Effect of Lime Stabilisation on Unconfined Compressive Strength	130
4.8 Effect of Moulding Water Content on Unconfined Compressive Strength	134
4.9 Effect of Lime Stabilisation on California Bearing Ratio (CBR)	143
4.10 Effect of Lime Stabilisation on Flexural Strength and Flexural Modulus	146
CHAPTER 5 CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDY	151
5.1 Conclusions	151
5.2 Recommendations for Future Study	154
REFERENCES	156
APPENDIX-A : GRAIN SIZE DISTRIBUTION CURVES FOR SAMPLES OF UNTREATED SOILS	163
APPENDIX-B : VOLUME CHANGE VERSUS SURCHARGE PRESSURE CURVES FOR SAMPLES OF UNTREATED SOILS	167
APPENDIX-C : FLEXURAL STRESS VERSUS DEFLECTION CURVES FOR SAMPLES OF LIME TREATED EXPANSIVE SOIL	171

NOTATIONS

A_c	=	Activity
b	=	Average width of sample
CBR	=	California Bearing Ratio
Cl	=	Percentage of clay size
C_c	=	Compression index
C_s	=	Swelling index
d	=	Average depth of soil-lime beam sample
E	=	Flexural modulus
FS	=	Free swell
I	=	Moment of inertia of the soil-lime beam section
I_w	=	Plasticity index
L	=	Span length of sample
LL	=	Liquid limit
P	=	Maximum applied load
PL	=	Plastic limit
q_u	=	Unconfined compressive strength
R	=	Modulus of rupture or flexural strength
SDI	=	Strength development index
S_p	=	Swelling potential
w_{opt}	=	Optimum moisture content
Δ	=	Deflection of the beam in the mid span
ϵ_f	=	Axial strain at failure
γ_{dmax}	=	Maximum dry density

CHAPTER 1

INTRODUCTION



1.1 GENERAL

Mechanical properties of soils in many areas do not fulfil the requirements of construction. So improvement of its properties may be essential to meet the required soil condition for construction. Stabilisation is one of the most economical and desirable method for improving the strength, durability and resistance to deformation of in situ soil. Soil stabilisation always involves certain mechanical treatment of the natural soil or remixing the natural soil with admixtures followed by compaction of the mixture. Winterkorn (1975) defined soil stabilisation as the collective term for any physical, chemical or biological methods, employed to improve certain properties of a natural soil to make it serve adequately an intended engineering purpose. The different uses of soil demand various requirements of mechanical strength and of resistance to environmental forces. Stabilisation is considered as a technique that is applied only when there is a particular and obvious deficiency in a material underestimates its potential. The usual deficiencies are mainly associated with inadequate strength or stiffness, excessive sensitivity to changes in moisture content, high permeability, poor workability and tendency to erode. Stabilisation is also a means by which an engineer can better command a situation by altering the properties of materials to optimise benefits. Hence the concept of stabilisation should extend beyond the remedial type treatment to be a general tool applied to design and construction.

There are a number of methods of soil stabilisation for improving the physical and engineering properties. Undoubtedly, the most widely applied methods involve the use of inorganic cementative bonds between the particles in the soil system (Khan, 1989). Full use of the potential of stabilisation requires an awareness of the various methods available, their preferred applications and limitations, their properties and means of evaluation and their construction requirements. NAASRA (1986) discussed the factors for each of the commonly used methods. These methods, their effects and applications are summarised in Table 1. One of the relevant factors affecting the selection of the most suitable method of stabilisation is the type of soil to be treated. Based on particle size distribution and plasticity of soils, NAASRA (1986) reported the usual range of suitability of the various types of stabilisation which have been presented in Fig. 1.1.

Table 1 Mechanics and applications of stabilisation (after NAASRA, 1986)

Type of Stabilisation	Process	Effects	Applicable Soil Type*
Granular	Mixing of two or more materials to achieve planned particle size distribution.	Changes to soil strength, permeability, volume stability.	Poorly graded soils, granular soils with a deficiency in some sizes.
Cement	Cementitious interparticle bonds are developed.	*Low additive contents: Decreases susceptibility to moisture changes, improves strength. *High additive contents: Increases modulus and tensile strength significantly. Possibility of reduced thickness requirements.	Not limited apart from deleterious components (organics, sulphates etc.) which retard cement reactions. Suitable for granular soils but inefficient in predominantly one sized materials. Expensive in cohesive soils.
Lime (Including Hydrated Lime and Quicklime)	Cementitious interparticle bonds are developed but the rate of development is slow, relative to cement.	Improves handling properties of cohesive material. *Low additive contents: decreases susceptibility to moisture changes, improves strength. *High additive contents: increases modulus and tensile strength.	Suitable for cohesive soils. Requires clay components in soil that will react with lime. Organic material will retard reactions.
Lime plus Fly Ash, Pulverised Blast Furnace Slag	Lime and pozzolan modifies particle size distribution and develops cementitious bonds	Generally similar to cement but rate of gain of strength similar to lime. Also improves workability.	As for cement stabilisation, can be used when soils are not reactive to lime.
Bitumen and Tar	Agglomeration of fine Particles	Waterproofs and improves cohesive strength.	Applicable to granular low cohesion, low plasticity materials.

* Use is always constrained by properties of untreated materials.

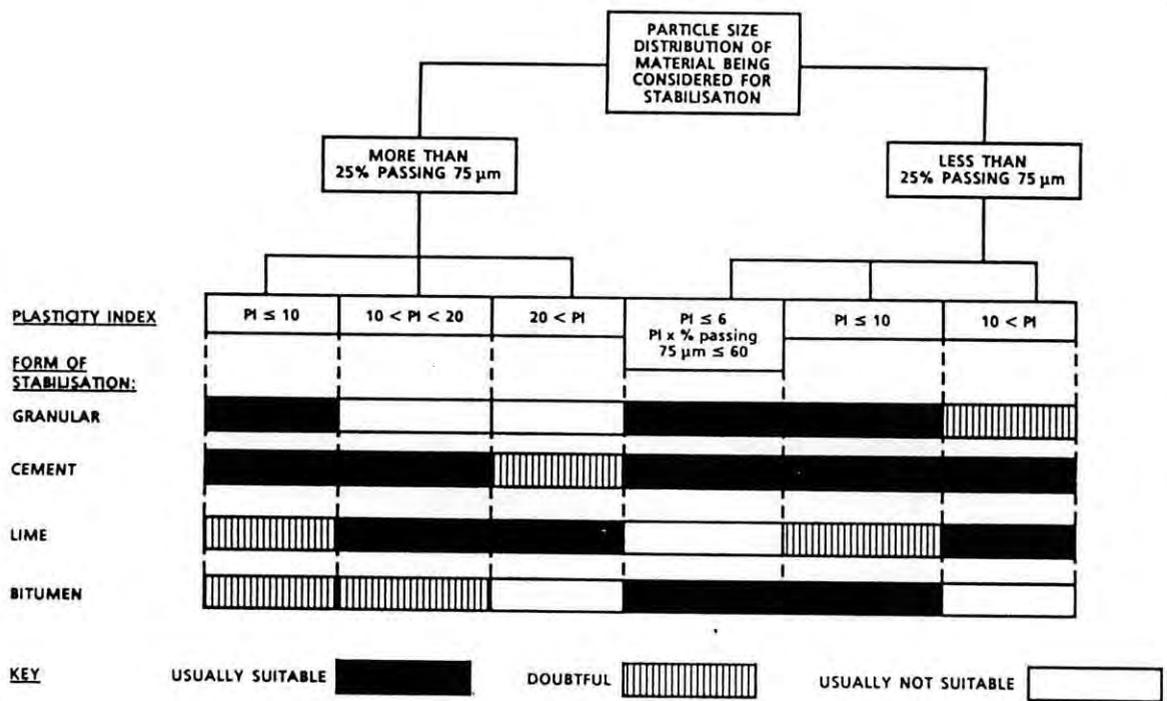


Fig. 1.1 Feasibility of stabilisation techniques for different types of soils
(after NAASRA, 1986)

The use of lime to stabilise soil has been known to engineers all over the world for a long time. For centuries, the Chinese have used lime as stabilising agent in foundation soils. Lime stabilisation are widely recommended for a number of construction works (Ingles and Metcalf, 1972; Mitchell, 1981; IRC, 1976; Kezdi, 1979; Broms and Boman, 1979; NAASRA, 1986; TRB, 1987; Hausmann, 1990; TRL, 1993; Bell, 1993). Ingles and Metcalf (1972) noted that soil stabilisation with lime alters the natural properties of unsuitable soil and thus provide sound geotechnical engineering requirements for design purpose. Thompson (1966) stated that the reaction between soil and lime involve interaction between soil silica (or alumina) and lime to form various types of cementing agents which are regarded as major sources of strength. The main engineering benefits of lime stabilisation are improved workability, increased strength and volume stability. Lime brings changes in the plasticity properties of a soil, it increases soil contact bonds and therefore the strength. Treatment with lime has been observed to decrease the soil plasticity, shrinkage properties (e.g., shrinkage limit and linear shrinkage) and swelling potential of soil. Lime-stabilisation has been successfully used to stabilise clayey soils. Lime is very effective in stabilising soft clays and is widely used for sub-grade, sub-base and base stabilisation as a construction expedient on the wet sites. The clay minerals carry a negative charge on the surface which adsorbed cations of sodium, magnesium, potassium or hydrogen, and to a large extent responsible for imparting plasticity to the soil sample.

Stabilisation of soil with lime has been successfully used for the construction of road foundation in USA, UK, Australia, South Africa, India and in many other countries. In United States and Europe, lime stabilisation is commonly used for improving remoulded strength, loading capacity of foundations of roads and embankments and for erosion control. Macham et al. (1977) and Markus et al. (1979) recommended that stabilised lime could be used for the construction of low-cost building in developing countries. Central Road Research Institute (CRRI), India has been advocating low cost soil stabilisation techniques for rural roads in India (Swaminathan et al., 1976). Lime has already been used as stabiliser for the sub-base layer construction of a segment of highway of length 24 km from Nagarbari to Sirajganj in the Northern district of Bangladesh.

1.2 STATEMENT OF THE PROBLEM

Expansive soils are those which swell considerably on absorption of water and shrink on the removal of water. Potentially expansive soils can be found almost anywhere in the world. Donaldson (1969) summarized the distribution of expansive soils. Expansive soils are more common in the semi-arid regions of the tropical and temperature climatic zones. In Bangladesh, expansive soils occur in the Barind Tract of Rajshahi, Lalmai Hill areas of Comilla, Joydevpur, Gazipur and some parts of Tangail. Hossain (1983) reported that the old alluvium of Modhupur Tract exhibit appreciable amount of volume change and high swelling potential. The recent alluvials are, however, reported to be non-expansive.

One would expect an enormous amount of damage to lightly loaded buildings on shallow foundation founded on expansive soils as the water content is changed. Heavy buildings, however, may not suffer much vertical movement. If otherwise structurally sound they may stand indefinitely with little or no damage, except the lightly loaded portions like floor slabs on the ground. Unless special precautions are taken to eliminate the effect of expansive soil, light buildings are often badly damaged by differential uplift as the volume of the soil changes. In the recent past, damage to buildings and other structures due to swelling of soils have been reported from different parts of Bangladesh. A number of instances were reported by Hossain (1983) and Khan (1995). More recently, in the region between Rajshahi and Nawabganj and the area surrounding Sreepur, especially the small structures as offices and staff quarters are severely and strangely cracked (BRTC, 1997). Foundation damages due to swelling of expansive soils can be avoided by providing proper treatment of expansive soils or special designing of foundation and structural members that will remain undamaged inspite of swelling.

Limited research has been carried out to investigate the behaviour of expansive soils of Bangladesh. Hossain (1983) investigated the swelling characteristics of fifteen Bangladeshi soils obtained from different geographic areas including seven samples from Modhupur Tract. Khan (1995) investigated the effects of sand layer on the swelling of the underlying expansive soil. A reconstituted expansive soil (natural soil from Savar area mixed with bentonite) was used.

Lime stabilisation has been recommended to reduce swelling of expansive soils. It is generally recognized that the addition of lime to expansive clay will reduce the plasticity of the soil and, hence its swelling potential. The successful use of mixing lime in

expansive soils for highway and airport construction is encouraging, although the depth of treatment required and the results of the treatment on a long term basis has not been evaluated. Mixing of lime in foundation soils to reduce swelling has not been seriously considered in the past. It appears that, with the knowledge gained from airport and highway construction using lime, treatment of under-slab with lime deserves more attention. The amount of lime required to stabilize the expansive soils ranges from 2 to 8 percent by weight. The pressure injection method of lime stabilization has been used in Jackson, Mississippi, in Calexico, California and in Tucson, Arizona (Chen, 1975). The method consists of injecting lime-water slurry under pressure into the soil through closely spaced drill holes.

Although a number of research works (Ahmed, 1984; Rajbongshi, 1997; Molla, 1997) were carried out in the past to investigate the geotechnical properties of lime stabilised local alluvial soils and soil from a coastal region, no attempt has yet been carried out to investigate the behaviour of lime stabilised expansive soils of Bangladesh. Investigation into properties of lime stabilised expansive soils would assess the suitability of using lime as stabiliser to reduce swelling of expansive soils. The present study, therefore, has been aimed to investigate the physical and engineering properties of a lime stabilised expansive soil. These properties can be used to assess the suitability of lime stabilisation for minimizing the degree of swelling of expansive soils.

1.3 OBJECTIVES OF THE PRESENT RESEARCH

This research has been intended to evaluate the behaviour and engineering properties (e.g., moisture-density relations, California Bearing Ratio, unconfined compressive strength, flexural strength and flexural stiffness) of a lime treated expansive soil collected from Rajendrapur Cantonment, Gazipur.

The major objectives of this research are as follows:

- (i) To determine the index and physical properties, and the swelling properties of the six soil samples collected from different locations of Rajendrapur Cantonment, Gazipur in order to select an appropriate expansive soil for further treatment with lime. Index and physical tests include Atterberg limit tests, specific gravity test, linear shrinkage test and grain size analysis. Swelling tests include free swell, free swell index, swelling pressure, swelling potential and volume change from air-dry to saturated condition.

- (ii) To investigate the index properties (e.g., liquid limit, plastic limit, shrinkage limit, linear shrinkage and volumetric shrinkage) of the selected expansive soil treated with five different percent of lime contents (3%, 6%, 9%, 12% and 15%).
- (iii) To investigate the swelling properties (e.g., free swell, free swell index, swelling pressure, swelling potential and volume change from air-dry to saturated condition) of a selected expansive soil treated with five different percentages of lime contents (3%, 6%, 9%, 12% and 15%).
- (iv) To investigate the behaviour and engineering properties (e.g., moisture-density relations, California Bearing Ratio, unconfined compressive strength, flexural strength and flexural stiffness) of the selected expansive soil treated with five different percent of lime contents (3%, 6%, 9%, 12% and 15%). The behaviour and engineering properties of the soil without any treatment were also investigated in order to examine the changes in behaviour and engineering properties between the treated and untreated soil.
- (v) To investigate the effect of the curing age on unconfined compressive strength and flexural properties of the lime treated soil. Special emphasis was given to assess the effect of long term curing on the unconfined compressive strength and flexural properties of the treated soil.
- (vi) To investigate the effect of moulding water content on unconfined compressive strength of the selected expansive soil treated with five different lime contents (3%, 6%, 9%, 12% and 15%).
- (vii) To investigate the influence of lime treatment on California Bearing Ratio (CBR) of the selected expansive soil treated with five different lime contents (3%, 6%, 9%, 12% and 15%) and the untreated soil.
- (vii) To examine the effect of compaction effort on California Bearing Ratio (CBR) of the selected expansive soil treated with five different lime contents (3%, 6%, 9%, 12% and 15%) and the untreated soil.

1.4 THE RESEARCH SCHEME

The whole research programme were carried out according to the following phases:

- (i) Firstly, index properties and swelling properties of the six soil samples collected from six different locations of Rajendrapur Cantonment, Gazipur were determined in order to select an appropriate expansive soil. Index tests include Atterberg limit

tests, specific gravity and grain size analysis. Swelling tests included free swell, free swell index, swelling pressure, swelling potential, volume change from air-dry to saturated condition.

(ii) Secondly, after the selection of the expansive soil, all the index property tests and swelling tests were carried out on the selected expansive soil treated with five different lime contents of 3%, 6%, 9%, 12% and 15%.

(iii) Thirdly, the following tests were carried out on the selected expansive soil without any treatment and also treated with three five different lime contents (3%, 6%, 9%, 12% and 15%):

- Standard compaction test
- Unconfined compressive strength test on compacted cylindrical samples of 2.8 inch diameter by 5.6 inch high
- California Bearing Ratio (CBR) test
- Flexural strength test using simple beam with third point loading system

Unconfined compressive strength tests were carried out on lime treated samples cured at different ages of 1 week, 2 weeks, 4 weeks, 8 weeks and 16 weeks while flexural strength tests using simple beam with third point loading were performed on lime treated samples cured at ages of 1 week, 2 weeks, 4 weeks and 8 weeks, in order to investigate the effect of curing age, particularly long-term curing on the measured unconfined compressive strength and flexural strength and stiffness.

In order to investigate CBR - Dry density relationships for the untreated and treated soils, laboratory CBR tests were carried out on the untreated samples and samples treated with various lime contents using three levels of compaction energies.

(iv) Finally, in order to investigate the effect of moulding water content on the compressive strength, unconfined compression strength tests were carried out on 2.8 inch diameter by 5.6 inch high stabilised samples of the soil treated with 3%, 6%, 9%, 12% and 15% lime which had been compacted according to the Standard Compaction test with two moulding water contents. The following water contents were used for compaction:

- (a) water content corresponding to 95% of maximum dry density on dry side of the optimum moisture content.
- (b) water content corresponding to 95% of maximum dry density on wet side of the optimum moisture content.

Unconfined compressive strength tests were carried out on lime treated samples cured at different ages of 1 week, 2 weeks, 4 weeks, 8 weeks and 16 weeks.

1.5 GEOLOGY OF THE PROJECT AREA

Bangladesh can be divided into three major physiographic units namely, (i) the tertiary hill formations, (ii) the Pleistocene terrace, and (iii) the recent flood plains. The generalized physiographic map of Bangladesh is shown in Fig. 1.2. According to the study of Morgan and McIntire (1959), there are two major areas of Pleistocene sediments, commonly known as the Modhupur tract and Barind tract. Gazipur district lies within the Modhupur tract. Modhupur tract is underlain by relatively homogeneous clay, known as Modhupur clay. These clay deposits are believed to have been laid down in a stable marine or deltaic environment in the late miocene. The clay is underlain by fine sand and the land systems are fault blocks which have since been uplifted and locally tilted.

The Madhupur block lies between the Jamuna and Old (18th century) Brahmaputra channels and 6-30 metres above mean sea level. Modhupur tract is bounded by faults; they appear to be uplifted and structurally complex; the Madhupur block has been tilted eastward (Morgan and McIntire, 1959). All or part of the clay is depositional. Most of the oxidized clay is now considered to be the product of weathering (the residuum) is, therefore, a relict paleosol. Residuum is defined as material derived by in-place chemical weathering of clastic sediment with no appreciable subsequent lateral transport. Patches of residuum also overlie gently dipping Tertiary units in the Fold Belt, including the Lalmai Hills, Comilla area.

1.5.1 Madhupur Clay Residuum

Madhupur clay residuum is composed of light-yellowish-gray, orange, light to brick-red, and greyish-white, micaceous silty clay to sandy clay. The clay is plastic and abundantly mottled in upper 8 m and contains small clusters of organic matter. Sand fraction consists dominantly of quartz; minor feldspar (orthoclase greater than plagioclase) and mica. Sand content increases with depth. Dominant clay minerals in this residuum are kaolinite and illite. Iron manganese oxide nodules are concentrated in zones while calcium carbonate nodules rare. Locally, a cohesive, 35-cm-thick iron oxide zone is preserved near the surface of residuum. Unoxidized clay coatings occur along root tubes, burrows, vugs, cavities, and fracture planes. There is gradational contact with underlying sand.

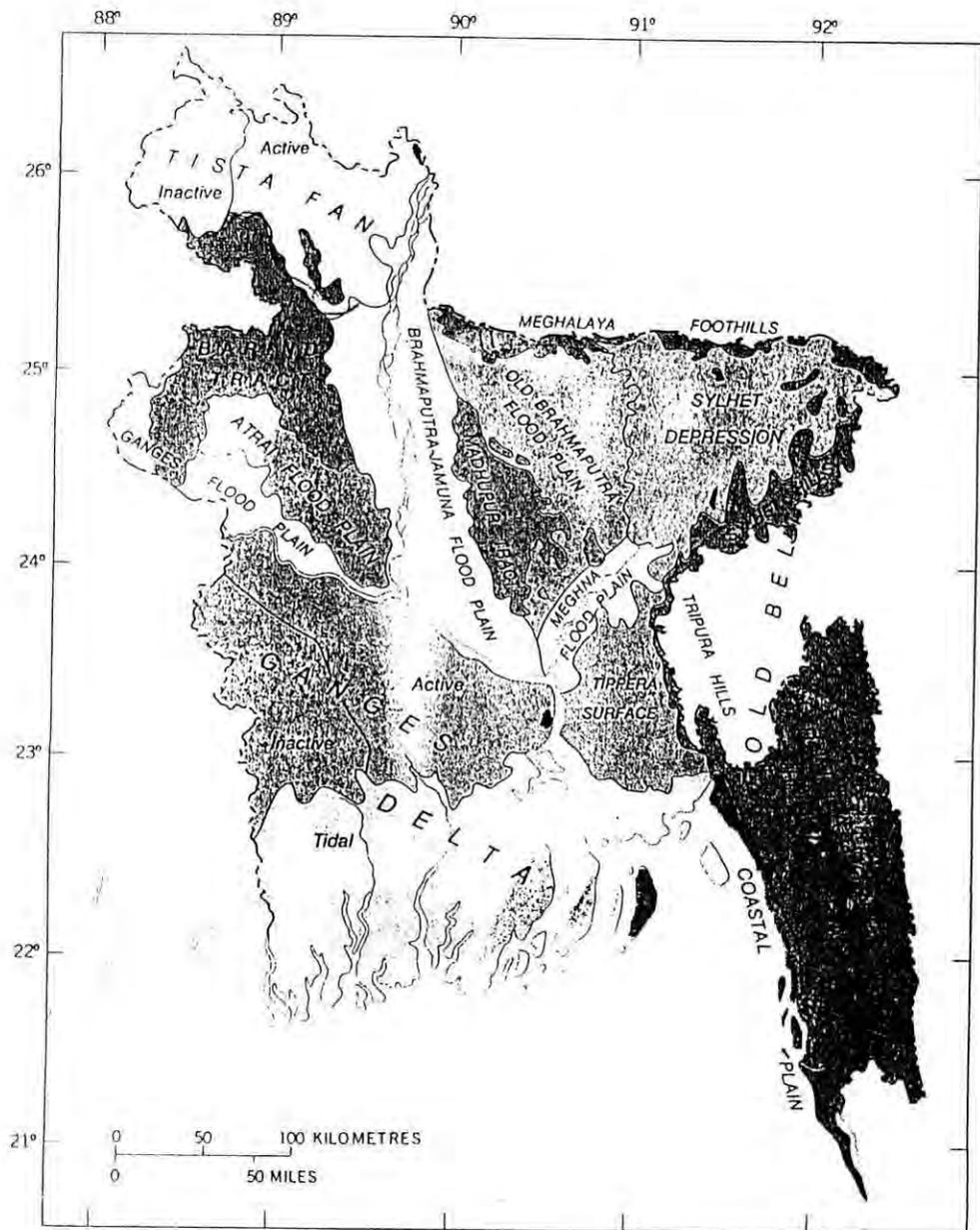


Fig. 1.2 Generalised physiographic map of Bangladesh

1.6 THESIS LAYOUT

A review on expansive soil and lime stabilisation is presented in Chapter 2. The review on expansive soil mainly includes occurrence of expansive soil, mechanics of swelling, engineering properties of expansive soil, identification of expansive soil and suitable foundation adopted for expansive soil. The review on lime stabilisation mainly includes the mechanisms of lime stabilisation, factors governing the properties of lime treated soils, the characteristics of lime stabilised soils and their applications.

Chapter 3 presents the details of laboratory testing procedures and equipment used for determining the swelling properties of expansive soil. Equipment and testing procedures to investigate the effects of lime stabilisation on the physical and engineering characteristics of the expansive soil studied have also been presented in Chapter 3.

Chapter 4 first presents the results of index, physical and swelling tests conducted on the six samples collected from various locations of Rajendrapur Cantonment, Gazipur in order to select the appropriate expansive soil used for lime stabilisation. Discussions regarding the degree of expansion of these six soil samples have been presented in this chapter. Finally, the index properties, swelling properties and engineering properties of lime stabilised samples have been presented and the effects of lime stabilisation on various index, swelling and mechanical properties have been discussed.

Chapter 6 presents the major findings and conclusions of the present investigation. Recommendations for further research in this field are also presented in this chapter.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

Expansive soils are those which swell considerably on absorption of water and shrink on the removal of water. In many parts of the world, the possibility of damage to structures due to swelling of soils constitutes a severe problem in design and construction. The solution to this problem cannot be achieved without an understanding of the fundamental characteristics of expansive soils and the variables involved that affect the swelling phenomenon. Lime stabilisation of clay soils, especially expansive clay soils, can minimise the amount of shrinkage and swelling they undergo. Hence, such treatment can be used to reduce the number and size of cracks developed by buildings founded on expansive soils.

A review on expansive soil and lime stabilisation is presented in the following sections. The review on expansive soil mainly includes occurrence of expansive soil, mechanics of swelling, engineering properties of expansive soil, identification of expansive soil and suitable foundation for expansive soil. The review on lime stabilisation mainly includes the mechanisms of lime stabilisation, factors governing the properties of lime treated soils, characteristics of lime stabilised soils and their applications.

2.2 OCCURRENCE OF EXPANSIVE SOILS

Potentially expansive soils can be found almost anywhere in the world. Donaldson (1969) summarised the distribution of expansive soils. It can be seen that expansive soils are more common in the semiarid regions of the tropical and temperature climatic zones. Expansive soils are in abundance where the annual evapotranspiration exceeds the precipitation. This follows the theory that in semiarid zones, the lack of leaching has aided the formation of montmorillonite type clay minerals.

In India, the black cotton soils which occupy about 16 percent of the total area of the country are highly expansive (Singh, 1975). This soil is characterised by its extreme hardness when dry and with high swelling during the process of wetting. In Bangladesh, expansive soils occur in the Barind Tract of Rajshahi, Lalmai Hill areas of Comilla,

Joydevpur, Gazipur and some parts of Tangail. Hossain (1983) reported that the old alluvium of Modhupur Tract exhibit appreciable amount of volume change and high swelling potential. The recent alluvials are, however, reported to be non-expansive.

2.3 MECHANISMS OF SWELLING

Swelling of soils is a complex physico-chemical process caused through absorption of water by the surface of clayey particles, as well as by osmotic and, in part by capillary action. According to Mielenz and King (1955), the following two mechanisms are involved in the swelling of soils:

- (i) Osmotic imbibation of water by clay minerals with an expanding lattice
- (ii) A relaxation of effective compressive strength related to enlargement of capillary films

The negative electric charges due to isomorphous substitution on the surface of the clay minerals attract cations from the solution of clay-water system. The high concentration of cations near the surface of the clay particle takes place which creates a repulsive force between the clay-water system. Thus the interlayer solution has a higher concentration of dissolved electrolyte than the external solution and the subsequent entry of water by osmosis. The resulting repulsive pressure is, therefore, the osmotic pressure. Bolt (1956) suggests that osmotic pressure indeed develops in the soil-water system and is responsible for the swelling mechanism. Bolt (1956) concluded that swelling of both illite and montmorillonite clays is caused by the excess osmotic pressure in the adsorbed layer of ions.

Based on the theory, osmotic pressure is the only internal pressure acting between particles. If the soil is subjected to external pressure, the distance between particles will decrease and water will be squeezed out. As a result, the ion concentration between the particles will increase and the osmotic pressure in turn increases. An equilibrium is finally reached when the osmotic pressure equals the external pressure. The reverse process involves the decrease of external pressure and the suction of liquid by osmotic pressure between the particles to dilute the concentration of ions. The distance between the particles would increase, resulting in volume increase and a reduction of osmotic pressure. The process continues until a new equilibrium is established. The imbibation of water is the most important cause of swelling.

Moisture heave sometimes takes place in heavily consolidated clays with high plasticity index. In many cases this is a matter of the reswelling of montmorillonite which has been reduced to a relatively low moisture content by consolidation pressures. However, there seem to be cases in which such moisture heave takes place in soils which do not have any expandable clay minerals. Such soils seem to be composed essentially of illite and chlorite with very little non-clay mineral material. Apparently such soils may have a texture which does not come to equilibrium under consolidation pressure so that it re-swells when the pressure is removed and additional water becomes available. The added moisture does not expand the lattice of the clay minerals but perhaps serves as a lubricant between the particles so that strains may be relieved.

From experience it is found that no swelling will take place if the environment of expansive soil is not changed. Environmental change can consist of volume increase due to increase of moisture, Pressure release due to excavation desiccation caused by temperature increase. The effect of water on expansive soils is the most important element and of most concern to the practising engineer, There must be a potential gradient to cause water migration and a continuous passage through which water transfer can take place. Volumetric expansion takes place when the moisture content of expansive soil increases. Swelling pressure is the pressure to be applied to prevent expansion and to maintain the initial volume.

The types and pattern of moisture migration depends on the geological formations, climatic conditions, topographic features, type of soil and ground water level. The most common method of moisture is by gravity. The seepage of surface water, precipitation, and snow melting into the soil are common examples. The moisture migration can occur in all directions. The flow can be upward under artesian conditions. Important differences in the moisture migration pattern between covered and natural areas have been studied extensively by the Commonwealth Scientific and Industrial Research Organisation in Australia, South Africa, and the United States in an attempt to stabilise pavements constructed in expansive soil areas.

The heaving of expansive soils may take place without the presence of free water to proving the means for the volume increase of expansive soils vapour transfer plays an important role. Water vapour at a temperature higher than its surroundings will migrate toward to cooler area to equalise the thermal energy of the two areas. When water

reaches the cooler area, generally the covered area beneath a structure, Condensation can take place and provide sufficient moisture to initiate swelling.

Three theories have been forwarded to explain the mechanism of swelling. These are Gouy-Chapman diffuse double layer theory, Terzaghi elastic theory and Schofield suction potential theory. Detailed descriptions of these theories have been provided by Hossain (1983) and Khan (1995).

2.4 FACTORS AFFECTING SWELLING

The swelling properties of a soil depend on the composite effects of several interacting and often interrelated factors. These factors may be divided into two groups: compositional and environmental.

Compositional factors determine the potential range of values for any property. Compositional factors controlling swelling are as follows:

- (i) Amount and type of minerals
- (ii) Structure of clay mineral
- (iii) Type of adsorbed cations
- (iv) Shape and size distribution of particles
- (v) Pore water composition

Environmental factors determine the actual value of any property. The following environmental factors control the degree of swelling:

- (i) Moisture content
- (ii) Density
- (iii) Confining pressure
- (iv) Temperature
- (v) Fabric

A brief discussion of the above factors is presented in the following sections.

2.4.1 COMPOSITIONAL FACTORS

2.4.1.1 Amount and Type of Minerals

The volume changes associated with expansive soils depend primarily upon the nature and amount of clay minerals. The three major minerals are montmorillonite, illite and kaolinite. These minerals are crystalline hydrous aluminosilicates. Montmorillonite is the clay mineral that creates most of the expansive soil problems. The montmorillonite clay minerals swell when comes in contact with water, whereas the other two clay minerals swell very little or not at all. The amount of swelling of various clay minerals decreases in the order. montmorillonite, illite, halloysite and kaolinite.

2.4.1.2 Structure of Clay Minerals

Swelling occurs in the crystal of montmorillonite because the units of this mineral are bound together loosely by mutual attractions for the exchangeable cations and the weak Vander Walls forces and may be imagined to be stacked one above the other like the leaves of a book. Attracted water molecules can easily insert themselves between the sheets cause them to expand and dissociate the crystal into its basic structural units (Grim, 1968)

2.4.1.3 Exchangeable Cations

In the soils containing expansive clay minerals, the type of exchangeable cations exerts a controlling influence over the amount of expansion that takes place in the presence of water. For example sodium and lithium montmorillonite can exhibit almost unrestricted interlayer swelling provided water is available, the confining pressure is small and the excess electrolyte concentration is low (Mielenz and King, 1955) with expanding lattice clays it is generally observed that in contact with liquid water. The interlayer absorption of water decreases in the order $L_i > N_a > K > R_b > C_s$.

2.4.1.4 Shape and Distribution of Size of Particles

Depending on mineralogy and exchangeable ions, smaller the particle size, larger the volume change takes place. Sands and silts do no swell. Clays containing expanding minerals have very small particle size ($0.1 \mu m$ for montmorillonite) and very thin ($10^0 A$) sheet like structure. These small particles exhibit very high swelling characteristics.

2.4.1.5 Pore Water Composition

In the case of a soil containing expansive clay mineral, any change in the pore solution composition that tends to depress double layers leads to reduction in swell or swelling pressure. For soils containing non-expansive clay minerals, the pore water composition has relatively little effect on swelling.

2.4.2 Environmental Factors

2.4.2.1 Moisture Content

When the moisture content of the clay changes, volume expansion, both in vertical and horizontal direction, takes place. Complete saturation is not necessary to accomplish swelling. Slight changes of moisture content, in the magnitude of only 1 to 2 percent, are sufficient to cause detrimental swelling. The initial moisture content of the expansive soils controls the amount of swelling. This is true both for soils in undisturbed and in remolded states. The relationship between initial moisture content and swelling has been studied by Holtz and Gibbs (1956) and Seed et al. (1962). Very dry clays with natural moisture content below 15 percent usually indicate danger. Such clay will easily absorb moisture to as high as 35 percent with resultant damaging expansion to structures. Conversely, clay with moisture contents above 30 percent indicates that most of the expansion has already taken place and further expansion will be small. However, moist clays may desiccate due to lowering of water table or other changes in physical conditions and upon subsequent wetting will again exhibit swelling.

In the laboratory clay samples swell in the consolidometer with slight increase of humidity. It is known that floor slab foundation on expansive soils cracked most severely when the moisture content increased slightly due to local wetting. As mentioned earlier, there must be a potential gradient which can cause water migration and a continuous passage through which water transfer can take place. Moisture migration depends on geological formations, climatic conditions, topographic features, soil types and ground water level. Gravity, capillary force, vapour and liquid moisture transfer under thermal gradient and soil suction are the important means of water transfer in expansive soils.

2.4.2.2 Density

The dry density of the clay is another index controlling the degree of expansion. Soils with dry densities in excess of 110 pcf generally exhibit high degree of swelling (Chen, 1975). Remarks made by excavators complaining that the soils are as hard as a rock is an indication that soils inevitable will present expansion problems. The dry density of the clays is also reflected by the standard penetration resistance test results. Clays with penetration resistance in excess of 15 usually possess some amount of swelling (Chen, 1975).

2.4.2.3 Confining Pressure

It is a well recognised fact that if sufficient pressure is applied on the expansive clay, the detrimental volume increase can be controlled. The volume change in an expansive clay layer would decrease with the increase in the surcharge pressure.

2.4.2.4 Temperature

When the temperature is decreased, differences in the volumetric shrinkage of the soil grains and water cause a tension in the pore water, which in turn causes soil to absorb water. In moisture-deficient soils, water vapor at a temperature higher than its surroundings will migrate toward the cooler area to equalize the thermal energy of the two areas and provide sufficient moisture to initiate swelling.

2.4.2.5 Fabric

Compacted expansive soils with flocculent structures are likely to be more expansive than with dispersed structure. At pressures less than the preconsolidation pressure, a soil with a flocculent structure is less compressible than the same soil with a dispersed structure. The reverse is true for pressures greater than the preconsolidation pressure.

2.5 ENGINEERING PROPERTIES OF EXPANSIVE SOIL

Holtz and Gibbs (1954) first reported the engineering properties of expansive soils. Early studies of expansive soils were mainly based on plasticity approaches using several index tests such as plasticity index, shrinkage limit, clay content. Correlations between these index properties in most cases, did not show satisfactory results for

various types of expansive soils (Seed et al., 1962). Recently more reliable techniques have been developed to explain swelling characteristics of expansive soils. These are free swell, free swell index, linear shrinkage, swelling potential, swelling pressure, volume change from air-dry to saturated condition. To know the swelling properties of soils studied, these engineering properties of expansive soils have been reviewed.

2.5.1 Swelling Potential

Although the swelling phenomenon has been fully recognized for many years, a definite method of measuring the swelling potential of clay has not been established. Seed et al. (1962) defined swelling potential as the percentage of swell of a laterally confined sample on under 1 psi (6.89 kPa) vertical stress, after being compacted to maximum dry soaking density corresponding to Standard Compaction test method (ASTM D698). Swelling potential is influenced by a number of factors. These are type and amount of clay in soil, structure of soil, dry density, water content at compaction, method of compaction and electrolyte concentration of water. In fact, these factors are determined by the placement condition and the environmental conditions for the soil and determine the extent to which the swelling potential of the comprising grains of the soil may be realized. Thus two soils having the same swelling potential may exhibit quite different amounts of swell after compaction either because they are placed at different compositions or because they are compacted by different climatic conditions. Alternatively, a soil exhibiting a higher swelling potential than another may swell much less because of differences in placement conditions. Different relationships between swelling potential and index properties have been developed by different researchers (Seed et al., 1962; Chen, 1975). Some of these are outlined in the following sections.

2.5.1.1 Plasticity Index and Swelling Potential

Seed et al. (1962) established the following simplified relationship between swelling potential (S_p) and plasticity index (I_w).

$$S_p = 60 K (I_w)^{2.44} \quad (2.1)$$

In which, $K = 3.6 \times 10^{-5}$ and is a constant

The above mentioned equation applies only to soils with clay content between 8 to 65 percent.

Chen (1975) proposed a relationship between swelling potential and plasticity index as follows :

$$S_p = B e^{A(I_p)} \quad (2.2)$$

in which $A = 0.0838$ and $B = 0.2558$

Fig. 2.1 shows the relationship between plasticity index and swelling potential. Fig. 2.1 shows that with the increase in plasticity index, the increase of swelling potential is much less than that predicted by Holtz and Gibbs (1956) or by Seed et al. (1962). All tests refer to a surcharge pressure of 1 psi with moisture content between 15 and 20 percent and dry density between 100 and 110 pcf. Hossain (1983) investigated the effect of plasticity index on swelling potential of seven expansive clays (old alluvium) of Bangladesh. Hossain (1983) reported that swelling potential increased with the increase in plasticity index.

2.5.1.2 Clay Content and Swelling Potential

Seed et al. (1962) found that for any given clay type, the relationships between swelling potential and percentage of clay sizes may be expressed by an equation of the form

$$S_p = B (CI)^A \quad (2.3)$$

Where,

S_p = Swelling potential, expressed as percentage of swell under 1 psi surcharge for a sample compacted at optimum moisture content to maximum dry density in Standard Compaction test

CI = percentage of clay sizes (finer than 0.002 mm)

B and A are constants depending on the type of clay

Fig. 2.2 shows the relationship between percent of clay and percent swell for various clay minerals such as commercial bentonite, 1:1 commercial illite-bentonite, 3:1 Commercial illite-bentonite, 1:1 illite-kaolinite and commercial kaolinite. Bentonite and illite bentonite mixtures exhibit greater swelling potential than the other mixtures of illite - kaolinite and kaolinite mineral. Seed et al. (1962) obtained the value of A as 3.44 for a wide range of clay types. Thus the coefficient B is the only factor differentiating one clay from another. The value of B varies from 0.28×10^{-5} to 152×10^{-5} for kaolinite and bentonite clay minerals, respectively. The intermediate values are for bentonite and illite mixtures. Hossain (1983) evaluated the values of B for seven expansive clays

(Modhupur soils) of Bangladesh. The values of B varied from 1.6×10^{-5} to 12×10^{-5} , with an average value of 3×10^{-5} .

It is to be noted that the prediction of swelling potential on the basis of plasticity index and clay content is a complex problem because these phenomena are affected not only by their inherent physical, chemical and mineralogical properties but also they are governed and altered by environmental factors such as moisture, surcharge pressure, weather and geological formation.

2.5.1.3 Activity and Swelling Potential

According to Skempton (1964), activity (A_c) is defined as follows:

$$A_c = \frac{\text{Plasticity Index}}{\text{Percentage of Clay Sizes}} = \frac{I_w}{CI} \quad (2.4)$$

Equation (2.4) defines a straight line passing through the origin. Seed et al. (1962) observed that the line does not always pass through the origin. Seed et al. (1962) suggested an equation of the following form:

$$A_c = \frac{I_w}{CI - L} \quad (2.5)$$

The value of L is 5 for natural soils.

Seed et al. (1962) reported the following relationship between the coefficient B and activity A_c .

$$B = (3.6 \times 10^{-5}) A_c^{2.44} \quad (2.6)$$

Substituting the value of B from Equation (2.6) into Equation (2.3), the expression for swelling potential becomes as follows:

$$S_p = (3.6 \times 10^{-5}) (A_c)^{2.44} (CI)^{3.44} \quad (2.7)$$

Thus using Equation (2.7), the swelling potential can be determined from the results of activity and clay content.

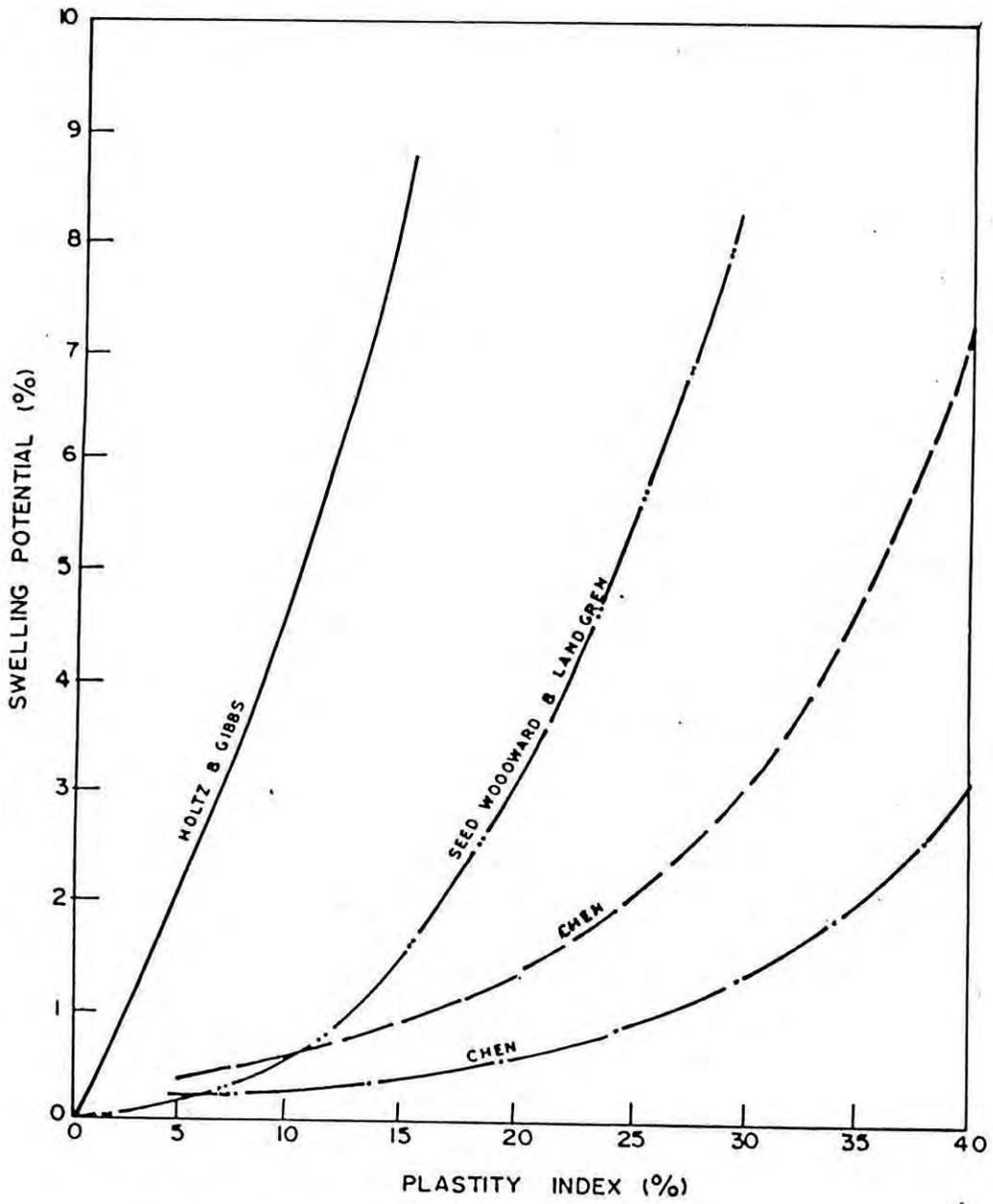


Fig. 2.1 Relationship between swelling potential and plasticity index as predicted by various researchers (after Chen, 1975)

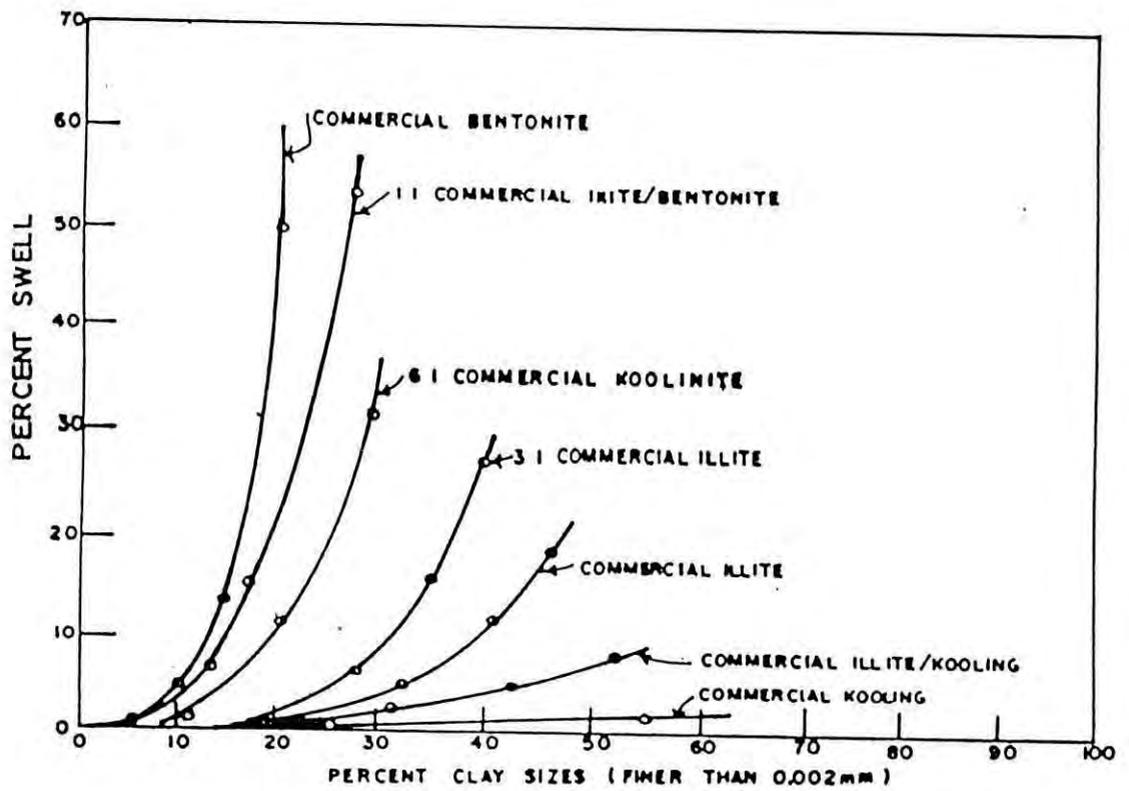


Fig. 2.2 Relationship between percentage of swell and percent of clay sizes for reconstituted soils (after Seed et al., 1962)

2.5.2 Free Swell

Holtz and Gibbs (1956) determined free swell value from the volume change expressed as the percentage of original volume when 10 cc of dry soil passing No. 40 sieve slowly poured in cylinder filled water and noted the swelled volume of the soil after 24 hours.

Experiments conducted by Holtz and Gibbs (1956) indicated that a good grade of high swelling commercial bentonite will have a free swell value of 1200 to 2000. Holtz and Gibbs (1956) suggested that soils having free swell value as low as 100 percent can cause considerable damage to lightly loaded structures and soils having free swell value below 50 percent seldom exhibit appreciable volume change even under very light loadings.

It has been reported that the free swell shows good correlation with the plasticity index of soils. Using test results of 82 calcareous clay soils (marl) and 96 clays. Komornik et al. (1977) obtained the following linear regression equations relating free swell (FS) to plasticity index (I_w) values:

$$FS = 6.7 + 2.4 I_w \quad \text{for marl} \quad (2.8)$$

$$FS = 0.9 + 2.1 I_w \quad \text{for clay} \quad (2.9)$$

2.5.3 Differential Free Swell

Indian Standard (IS: 2720, 1977) determined differential free swell value from the volume change between soil specimen poured in graduated cylinder containing distilled water and kerosene oil (a non-polar liquid) expressed as the percentage of original volume when 10 gm of dry soil passing No. 40 sieve slowly poured in cylinder filled with kerosene oil and noted the swelled volume of the soil after 24 hours.

The free swell index is calculated using the following expression:

$$\text{Free swell index, percent} = \frac{V_d - V_k}{V_k} \times 100 \quad (2.10)$$

where, V_d = the column of soil specimen read from the graduated cylinder containing distilled water

V_k = the volume of soil specimen read from the graduated cylinder containing kerosine

2.5.4 Linear Shrinkage

According to the British Standard (1975), the linear shrinkage of a soil is defined as the volume change in length expressed as a percentage of original length when a dry soil passing No. 40 sieve mixed uniformly with water nearly equal to its liquid limit is placed in a linear shrinkage mould and then dried to obtain its final length.

The swell potential is presumed to be related to the opposite property of linear shrinkage. Heidema (1957) correlated linear shrinkage between plasticity index as shown in Fig. 2.3. Fig. 2.3 shows that almost a straight line, passing through origin, is obtained from the plot of plasticity index and bar linear shrinkage values. Starting of the line from origin can be explained by the fact that non-plastic soil (silt, sand) does not undergo any volume change.

2.5.5 Percentage Linear Free Swell

Komronik et al. (1977) determined free swell test in a different manner and termed as percentage linear free swell. Laboratory consolidation cell was used for this test in which the sample of soil was first allowed to saturate distilled water under constant volume. Then the load of the scale beam which maintained the constant volume was released and the corresponding volume change was recorded. At the equilibrium condition when there was no volume change the final reading was noted and expressed as percentage of the original volume to get percentage linear free swell.

Analysis by McDowell et al. (1977) of the results of 370 percentage linear free soil tests conducted on natural undisturbed clays, as reported by Komornik et al. (1977) reveal that this value can be correlated with soil parameters such as liquid limit and initial water content. Linear multiple regression analysis of the 370 tests yielded the following equation of correlation.

$$S_L = 2.77 + 0.131 LL - 0.274 w \quad (2.11)$$

In which, S_L = Percentage linear free swell

w = Initial moisture expressed as percentage

LL = Liquid limit as percentage

McDowell et al. (1977) observed that the probable minimum initial moisture content, W_{min} for which swell will begin is given by the following expression:

$$w_{min} (\%) = 0.2 LL + 9 \quad (2.12)$$

Substituting this value of moisture content from equation (2.12) into equation (2.11) a simple linear relationship between percentage linear free swell and liquid limit is obtained for stated conditions of initial moisture content. This relationship is shown in Fig. 2.4. The relationship proposed by McDowell et al. (1977) is based on limited swell test data for selected Texas clays. Fig 2.4 demonstrates that the relationship proposed by McDowell et al. (1977) is generally applicable for clays of high plasticity (CH) but deviates significantly from the regression line for clays of low to medium plasticity (CL), particularly those with liquid limits of less than 35.

2.5.6 Swelling Pressure

When an expansive soil attracts and accumulates water, a pressure known as swelling pressure builds up in the soil and it is exerted on the overlying materials and structures, if there are any. The swelling pressure of an undisturbed or remoulded specimen is measured at a constant volume. Experiments showed that swelling pressure, of a clay is independent of the surcharge pressure, initial moisture content, degree of saturation and stratum thickness (Chen, 1975). The swelling pressure increases with dry density and curing time (Chen 1975; Kassif and Baker 1971).

2.5.6.1 Initial Density and Swelling Pressure

Initial density, whether undisturbed or remolded, is the only element that affects the swelling pressure. For constant moisture samples, the volume change increases with dry density, as does the swelling pressure. Chen (1975) establishes a straight line relationship between dry density and volume change which is shown in Fig. 2.5. Chen (1975) reported that the relationship between dry density and swelling pressure plotted on a semi-log scale is linear while on a rectangular scale, the relationship between swelling pressure and dry density is of exponential form, i.e., when dry density decreases, swelling pressure rapidly approaches zero and when dry density increases, swelling pressure rapidly increases and approaches infinity. The geotechnical engineers are interested only with a narrow range of dry density varying from 100 to 130 pcf (15.7 to 20.4 kN/m³).

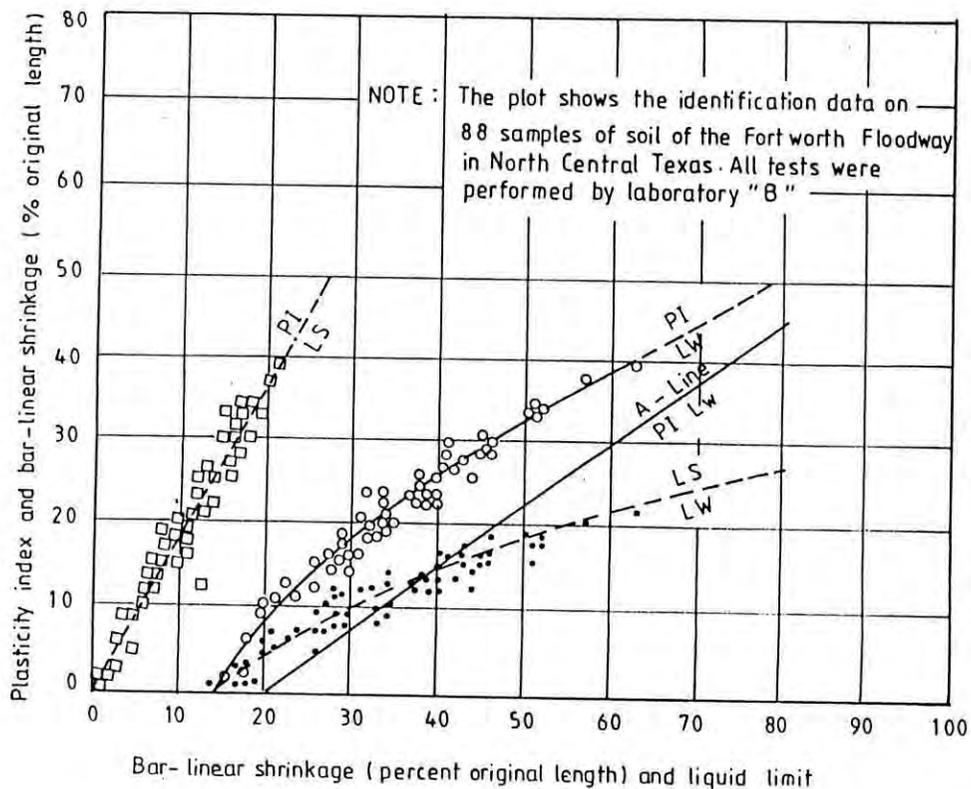


Fig. 2.3 Typical liquid limit, plasticity index and bar-linear shrinkage relationships (after Heidema, 1957)

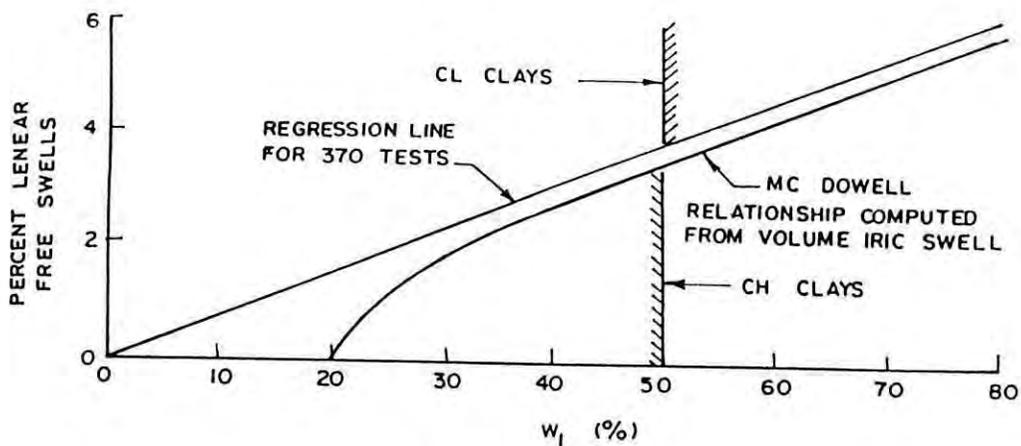


Fig. 2.4 Percentage linear free swell for soil with initial moisture content of $0.2 LL + 9$ (after McDowell et al., 1977)

Hossain (1983) investigated the effect of dry density on swelling pressure for six pleistocene clays (Modhupur soils). Hossain (1983) reported that swelling pressure increased rapidly with the increase in dry density and that swelling pressure rapidly approached zero when dry density decreased. Khan (1995) also investigated the effect of dry density on volume change and swelling pressure on reconstituted expansive clays. Khan (1995) found that both volume change for a given surcharge (seating) pressure and swelling pressure increased with the increase in initial dry density of expansive soil. The effect of dry density on swelling pressure of a sample is shown in Fig. 2.6. Khan (1995) also reported that volume change for samples of constant moisture content decreased with the increase in surcharge pressure.

2.5.6.2 Initial Moisture Content and Swelling Pressure

Expansive soils will not be subject to volume change unless there is a change in moisture content. A drier soil will swell more than a wet soil. In a series of test Chen (1975) made an attempt to determine the effect of increasing the initial moisture content on the volume change as well as swelling pressure. The results of these tests indicated that the amount of volume change of soil samples compacted at constant density but varying moisture contents decreased with the increase in moisture content. However, the swelling pressure required for zero volume change remained practically constant. Kassif and Baker (1971) stated that if clay is given enough time for ageing, then for the same dry density, the swelling pressure is not affected by moisture content. Hossain (1983), however, found that swelling pressure reduced with the increase in initial moisture content for six remoulded expansive clays of Bangladesh prepared at constant dry density. Khan (1995) found that for soils having the same dry density, the volume change decreased with an increase in moisture content. Swelling pressure, however, has been found to increase with the increase in moisture content.

Hossain (1983), however, found different relationship between swelling pressure and initial moisture content. Hossain (1983) found that swelling pressure reduced with the increase in initial moisture content for six remoulded expansive clays of Bangladesh prepared at constant dry density. Khan (1995) found that for soils having the same dry density, the volume change decreased with an increase in moisture content. Swelling pressure, however, has been found to increase with the increase in moisture content.

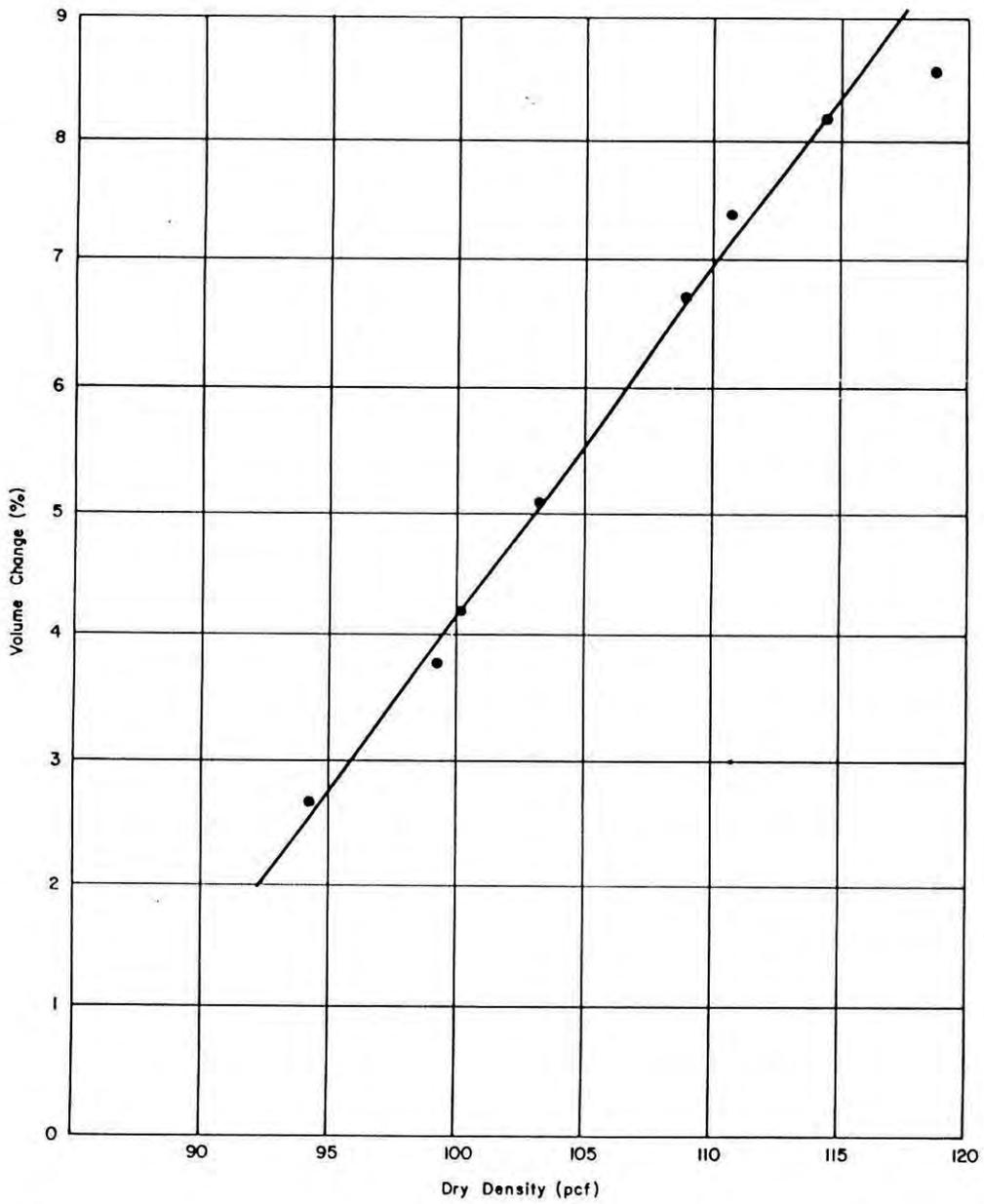


Fig. 2.5 Effect of varying density on volume change for constant moisture content samples (after Chen, 1975)

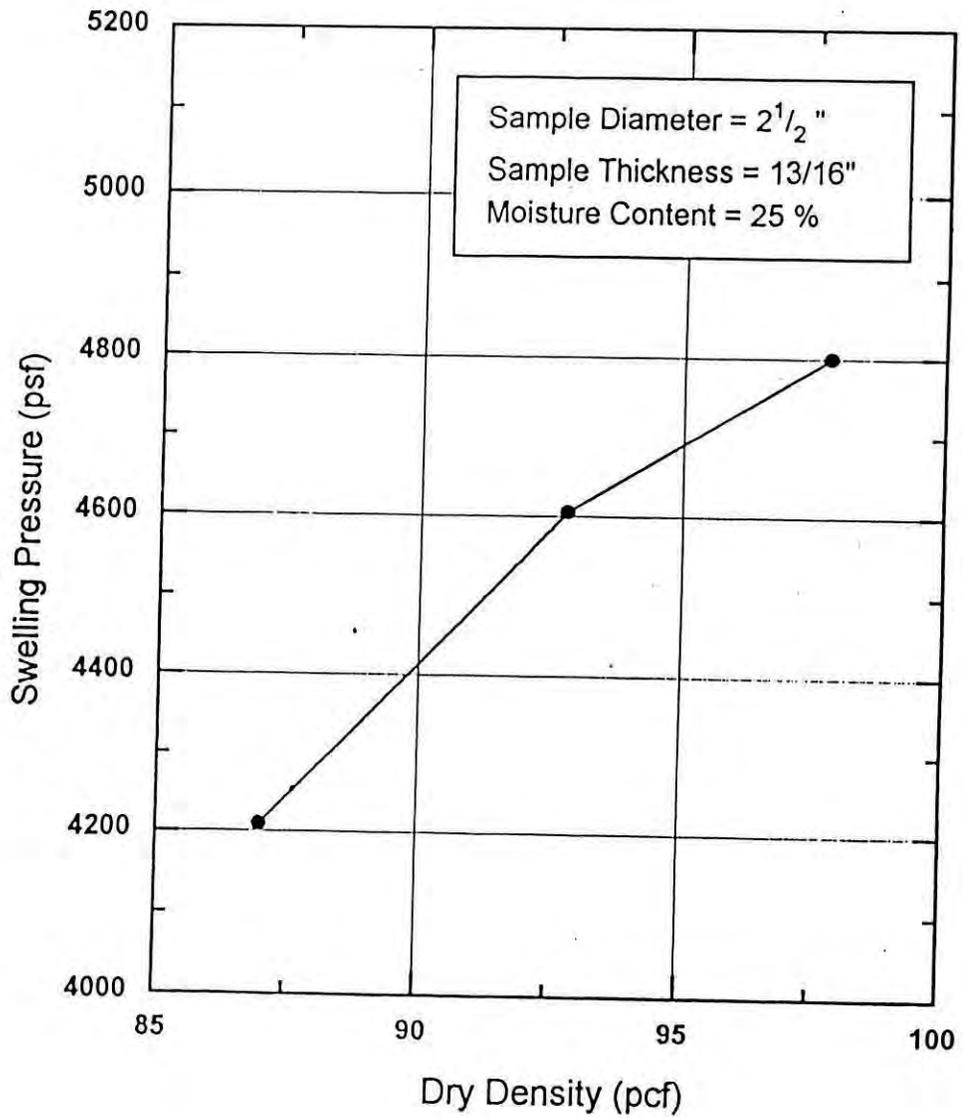


Fig. 2.6 Effect of dry density on swelling pressure for constant moisture content samples (after Khan, 1995)

2.5.6.3 Degree of Saturation and Swelling Pressure

Chen (1975) performed a series of tests to study the effect of duration of wetting on swelling characteristics. Since it difficult to control in a short sample height, the duration of wetting to achieve the same effect in the laboratory the degree of saturation on the sample was varied. The samples were compacted in the consolidometer with uniform density and moisture content, and a measured amount of water was then added to the sample. Sufficient time was allowed for all added water to soak into the sample. The amount of volume change and swelling pressure were recorded. Fig. 2.7 shows the effect of degree of saturation on volume change. It can be seen from Fig. 2.7 that the amount of volume change increases with the increase in degree of saturation at the end of test. The swelling pressure has been found to be constant or in other words the pressure required to maintain constant volume is independent of the duration of wetting or the degree of saturation.

2.5.6.4 Curing Time and Swelling Pressure

Kassif and Baker (1971) studied the swelling pressure of compacted clay as a function of curing time. Samples of clay soil were compacted to a predetermined density and water content and sealed in wax and their swelling pressure was determined in the triaxial apparatus. The reported results, shown in Fig. 2.8, indicate that the swelling pressure increases markedly with curing time, The maximum value reached at about 4 days of curing time. There is apparently a tendency to a decrease in swelling pressure after a certain curing time (about 20 days), indicating an increase of pore pressure with time. Hossain (1983) also investigated the effect of curing time on swelling pressure of a highly expansive clay of Bangladesh. The sample was allowed to cure up to 32 days. The results showed that swelling pressure increased markedly with curing time and reached its maximum value after about 4 days. After that it gradually decreased. These results are in agreement with those reported by Kassif and Baker (1971).

2.5.6.5 Surcharge Pressure and Swelling Pressure

It is a well recognised fact that if sufficient load is applied on an expansive clay, the detrimental volume increase can be controlled. The surcharge pressure applied to the soil sample in the consolidometer simulates the dead load pressure exerted on the footings or pier foundation. Chen (1975) reported that for constant density and moisture content samples, the volume change (swelling) reduces with the increase in surcharge pressure. The swelling pressure of samples, however, would remain unchanged.

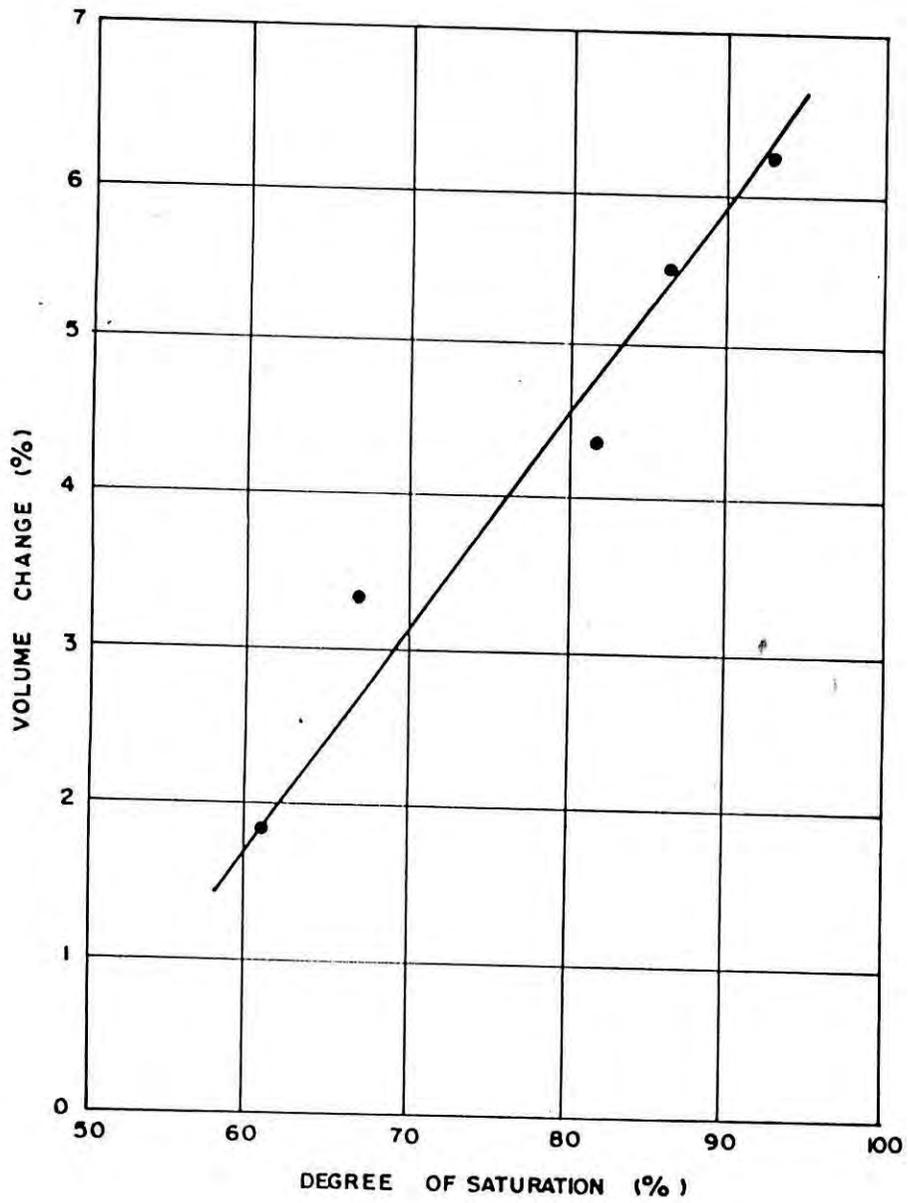


Fig. 2.7 Effect of varying degree of saturation on volume change for constant density and moisture content samples (after Chen, 1975)

2.5.6.6 Layer Thickness and Swelling Pressure

Chen (1975) investigated the effect of stratum thickness on the amount of volume change and swelling pressure. The magnitude of the volume change has been found to be proportional to the sample thickness and the percentage of volume increase remains constant. Chen (1975) also found that the swelling pressure of the samples was constant.

Khan (1995) also investigated the effect of layer thickness on volume change and swelling pressure. Khan (1995) reported that both the percentage volume change at a given surcharge pressure and swelling pressure are independent of thickness of expansive soil layer. However, for a particular thickness of expansive soil, the amount of percentage volume change decreases with the increase of surcharge pressure. It may be noted that although the percentage volume change does not, practically, change with the thickness of expansive soil layer, the total amount of vertical movement reduces due to the reduction of layer thickness. This indicates that partial removal of expansive soil layer will not affect either percentage volume change under a certain surcharge pressure or the swelling pressure but it may help in reducing the total amount of vertical movement. This finding is in agreement with the results reported by Chen (1975) and Kassiff and Baker (1971).

2.5.6.7 Relations of Swelling Pressure with Other Properties

Gupta et al. (1967) studied the physico-chemical properties of Indian Black cotton soils relation to their engineering behaviour. Swelling pressure, free swell, liquid and shrinkage limits and consolidation characteristics were determined. Gupta et al. (1967) found a linear relationship between free swell and swelling Pressure for Indian black cotton soil as shown in Fig. 2.9. It can be seen from Fig. 2.9 that swelling pressure increases linearly with the increase in free swell.

Swelling pressure plotted against the shrinkage and liquid limits is shown in Fig. 2.10 while Fig. 2.11 shows the plottings of swelling pressure against compression index (C_c) and swelling index (C_s). Fig. 2.10 shows that swelling pressure decreases with the increase in shrinkage limit while swelling pressure increases with the increase in liquid limit. It can be seen from Fig. 2.11 that both compression index and swelling index increase linearly with the increase in swelling pressure.

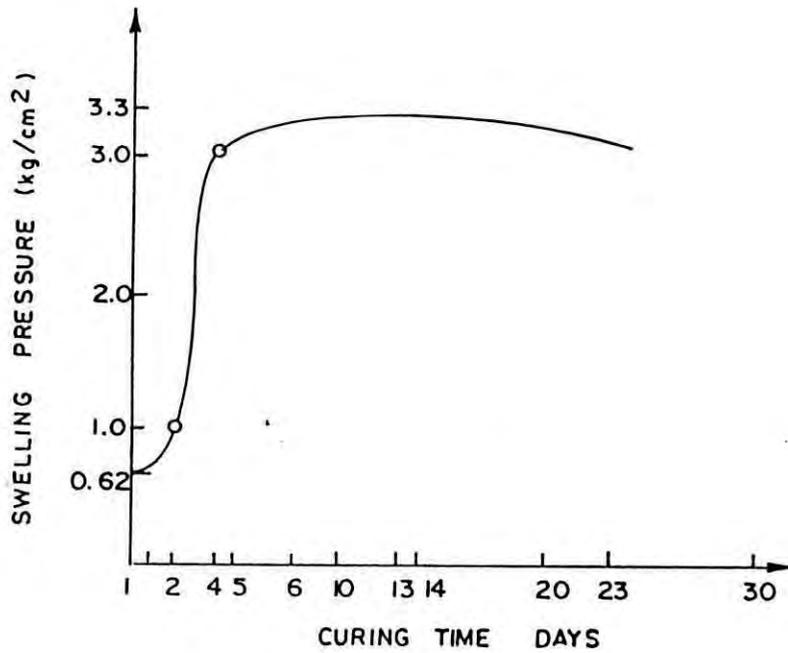


Fig. 2.8 Relationship between swelling pressure and curing time (after Kassiff and Baker, 1971)

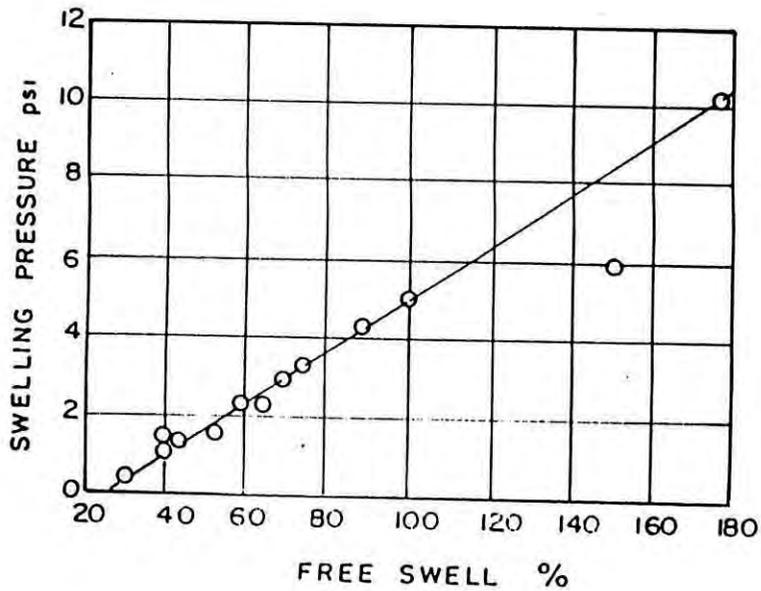


Fig. 2.9 Relationship between free swell and swelling pressure (after Gupta et al., 1967)

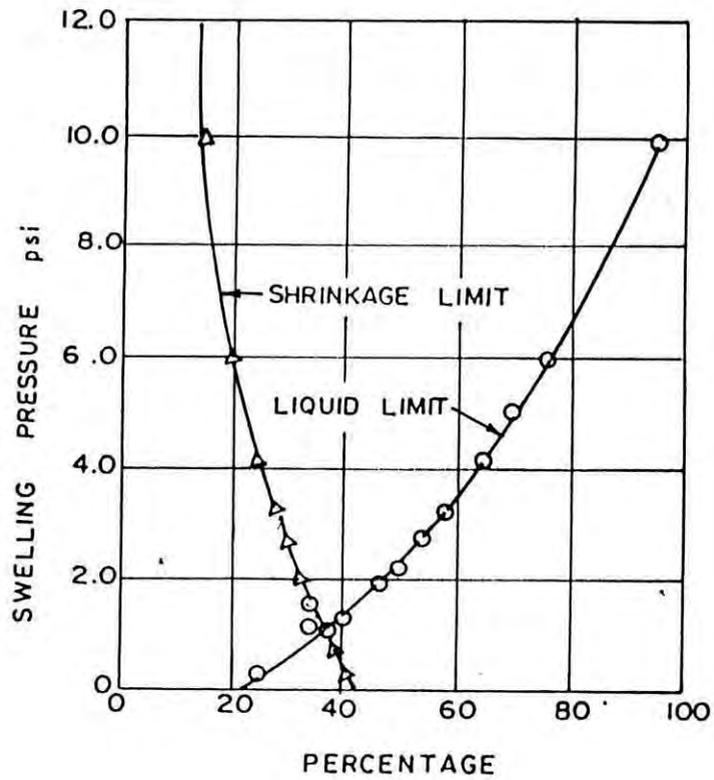


Fig. 2.10 Relationship of swelling pressure with shrinkage limit and liquid limit (after Gupta et al., 1967)

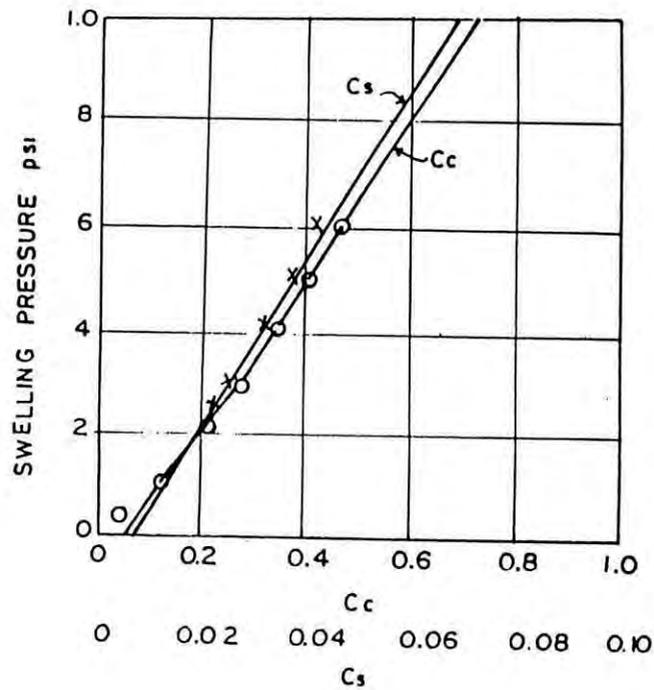


Fig. 2.11 Relationship between swelling pressure and compression index (after Gupta et al., 1967)

2.6 IDENTIFICATION AND CLASSIFICATION OF EXPANSIVE SOILS

There are three different methods of recognizing potentially expansive soils. These are mineralogical identification, indirect methods and direct measurement method. Methods of mineralogical identification are important for exploring the basic properties of expansive clays. Mineralogical identification and direct measurement method for identification of expansive clays have been discussed by Chen (1975). Indirect method of identification, based on index properties, soil suction and activity are generally used extensively. A number of criteria for identifying expansive soils are reviewed in the following sections.

2.6.1 USBR Classification System

Developed by Holtz and Gibbs (1956), United States Bureau of Reclamation (USBR) classification system is based on the simultaneous consideration of several soil properties, e.g., colloid content, plasticity index and shrinkage limit. The typical relationships of these properties with volume change (air-dry to saturated condition under a load of 1 psi) are shown in Fig. 2.12. Based on the curves presented in Fig. 2.12 Holtz (1959) proposed the identification criteria of expansive clays as shown in Table 2.1.

2.6.2 Activity Method

The activity method proposed by Seed et al. (1962) is based on remolded, artificially prepared soils composed of 23 mixtures of bentonite, illite, kaolinite and fine sand. The expansion was measured as swelling potential. The activity for the artificially prepared sample was defined as :

$$\text{Activity} = \frac{I_w}{CI - 10} \quad (2.13)$$

The Proposed classification chart is shown on Fig. 2. 13.

2.6.3 Chen's Method of Classification

Chen (1975) accumulated years of test data on expansive soils and found that it is more convenient to correlate expansive properties with the percentage of silt and clay material finer than No. 200 sieve, liquid limit and field penetration resistance (N-value).

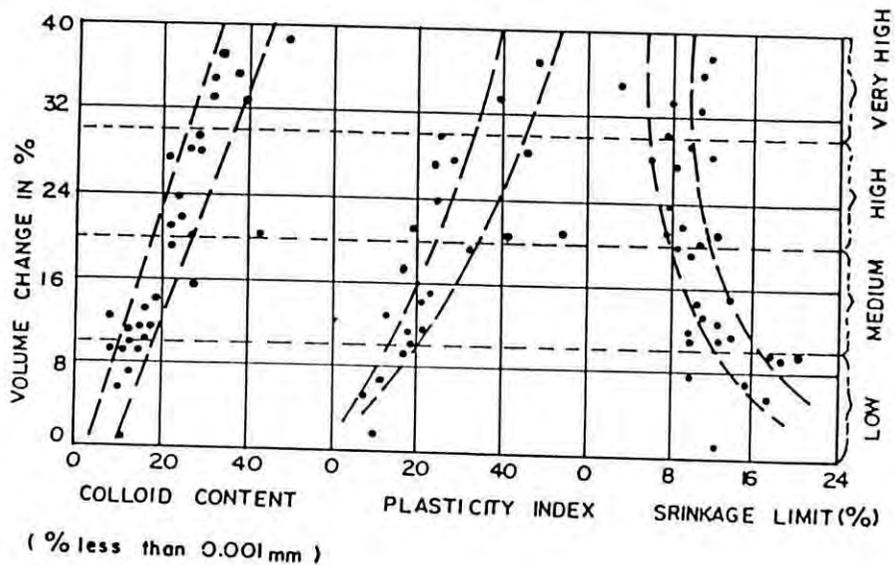


Fig. 2.12 Relation of volume change (air dry to saturated condition under a load of 1 psi) to colloid content, plasticity index and shrinkage limit (after Holtz and Gibbs, 1956)

Table 2.1 Data for making estimates of probable volume changes for expansive soils (after Holtz, 1959)

Data from index tests*			Probable Expansion, (Percent total Volume Change)	Degree of Expansion
Colloid Content (Percent finer 0.001 mm)	Plasticity Index	Shrinkage Limit		
> 28	>35	<11	>30	Very high
20-31	25-41	7-12	20-30	High
13-23	15-28	10-16	10-20	Medium
<15	<18	>15	<10	Low

* Data based on vertical loading of 1.0 psi (6.89 kPa)

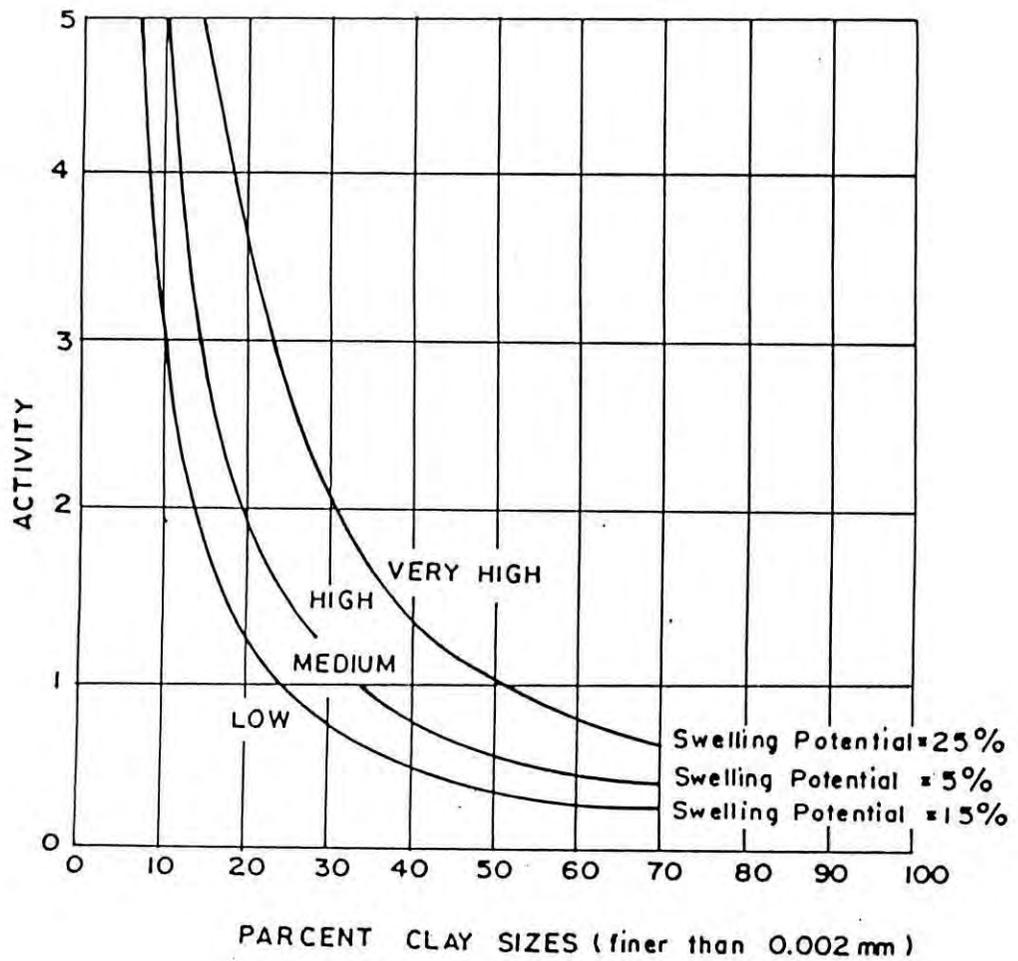


Fig. 2.13 Classification chart for determination of swelling potential (after Seed et al., 1962)

Since most lightly loaded structures will exert a maximum dead load pressure of about 1,000 psf on the footings, it is realistic to use a vertical load of 1000 psf to gauge the swelling potential. A guide for estimating the probable volume changes of expansive soils is presented in Table 2.2.

2.6.4 Classification Based on Linear Shrinkage

Theoretically, it appeared that the shrinkage characteristics of clays is a reliable index to determine their degrees of expansion (Chen, 1975). The swelling potential is presumed to be opposite property of linear shrinkage. It was suggested by Altmeyr (1955) as a guide to the determination of potential expansiveness for various values of shrinkage limits and linear shrinkage as shown in Table 2.3. Recent research, however failed to show conclusive evidence of the correlation between swelling potential and shrinkage limit (Chen, 1975).

On the basis of previous data for linear shrinkage of Bangladesh soils, criteria for the degree of expansion proposed by Hossain (1983) is shown in Table 2.4

2.6.5 Indian Classification System

On the basis of the values of free swell, Indian standard (IS: 1948, 1970) recommends criteria of expansion. These criteria are shown in Table 2.5.

Based on the value of free swell index, Indian Standard (IS: 2911, Part III, 1980) suggests criteria for the degree of expansion of soils which are shown in Table 2.6.

Based on the values of liquid limit, plasticity index and shrinkage limit, Indian Standard (IS: 2911, Part III, 1980) suggests criteria for the degree of expansiveness of soils which is shown in Table 2.7.

2.6.6 United States Army Engineers Waterways Experimental Station Classification System

Based on values of liquid limit, plasticity index and soil suction, United States Army Engineers Waterways Experimental Station (USAEWES) reported a criteria for potential expansion for highway sub-grades. These criteria are shown in Table 2.8.

Table 2.2 Data for prediction of degree of expansion (after Chen, 1975)

Laboratory and Field Data			Probable Expansion, (Percent Total Volume Change)	Swelling Pressure (ksf)	Degree of Expansion
Percentage Passing No. 200 Sieve	Liquid Limit	Standard Penetration Resistance, N-value Blows/ft			
>95	>60	>30	>10	>20	Very High
60-95	40-60	20-30	3-10	5-20	High
30-60	30-40	10-20	1-5	3-5	Medium
<30	<30	<10	<1	1	Low

Note : Data based on vertical loading of 1000 psf (48 kPa)

Table 2.3 Classification of expansive soil (after Altmeyr, 1955)

Shrinkage Limit (%)	Linear Shrinkage (%)	Degree of Expansion
<10	> 8	Critical
10-12	5-8	Marginal
> 12	0-5	Non- critical

Table 2.4 Degree of expansion based on linear shrinkage of Bangladesh soils (after Hossain, 1983)

Linear Shrinkage (%)	Degree of Expansion
> 14	High
10-14	Medium
0-10	Low

Table 2.5 Criteria for degree of expansion based on free swell (after IS: 1948, 1970)

Free Swell (%)	Degree of Expansion	Danger of Severity
<50	Low	Non-critical
50-100	Medium	Marginal
100-200	High	Critical
>200	Very High	Severe

Table 2.6 Criteria for degree of expansion based on free swell index
(after IS: 2911, Part III, 1980)

Free Swell Index(%)	Degree of Expansion	Danger of Severity
Less than 20	Low	Non-critical
20 to 50	Medium	Marginal
35 to 50	High	Critical
Greater than 50	Very high	Severe

Table 2.7 Criteria of degree of expansion based on liquid limit and plasticity index
(after IS: 2911, Part III, 1980)

Liquid Limit (%)	Plasticity Index	Degree of Expansion	Danger of Severity
20-35	<12	Low	Non-critical
35-50	12-23	Medium	Marginal
50-70	23-32	High	Critical
70-90	>32	Very high	Severe

Table 2.8 USAEWES classification system (after Snethen, 1979)

Liquid Limit (%)	Plasticity Index	Initial Suction (kPa)	Potential Expansion (%)	Potential Expansion Classification
<50	<25	<144	<0.5	Low
50-60	25-35	144-383	0.5-1.5	Marginal
>60	>35	>383	>1.5	High

2.6.7 Classification Based on Swelling Potential

Based on the values of swelling potential. Seed et al. (1962) proposed the four categories of expansion characteristics. These are presented in Table 2.9.

Table 2.9 Degree of expansion based on swelling potential (after Seed et al., 1962)

Swelling Potential (%)	Degree of Expansion
0-1.5	Low
1.5-5	Medium
5-25	High
>25	Very high

2.7 FOUNDATIONS ON EXPANSIVE SOIL

One would expect an enormous amount of damage to lightly loaded buildings on shallow foundations founded on expansive soils as the water content is changed. Unless special precautions are taken to eliminate the effect of expansive soil. Lightly loaded buildings are often badly damaged by differential uplift as the volume of the soil changes. In some areas, where the water table is normally high and the clay is saturated, sever damage may be caused by shrinkage of the clay when exposed to an unusual drought which has occurred for the first time since construction of the building.

It is necessary to understand that all parts of a building foundation will not equally be affected by the swelling of the soil. Beneath the center of a building where the soil is protected from sun and rain, the moisture changes are small and the soil movements are the least. Beneath outside walls the movements are the greatest. This causes a differential uplift of the building and inevitably causes severe damage to the outside walls of the buildings.

The general types of foundations those are considered in expansive soils are:

- (i) Drilled pier foundation: Structures that can be kept isolated from the swelling effects of the soils.
- (ii) Footing foundations: Designing of foundation and structural members that will remain undamaged inspite of swelling.

2.7.1 Drilled Pier Foundations

For highly swelling soil with swelling pressure in excess of 5,000 psf (2400 kPa), a pier foundation will be required to concentrate the dead load pressure on a small area. It is not difficult to assign a dead load pressure in excess of 20,000 psf in a small diameter pier. However, attention must be given to the additional swelling effect on the shaft of the pier embedded in swelling soil. The swelling pressure exerted at the bottom of the pier. Dead load pressure alone generally is not sufficient to prevent the uplifting of the pier. Anchorage of the pier in a zone not affected by moisture change should be used to assist the dead load pressure requirement. The drilled pier foundation is a rational solution to combat the problem of expansive soils, however the design and construction must be closely controlled. The principle of the use of drilled piers is to transfer the structural loads down to an earth material or to a stable zone where moisture changes are improbable. There should be no direct contact between soil and the structure with the exception of the soils supporting the piers. Detailed description on drilled pier foundation has been provided by Chen (1975).

2.7.2 Footing Foundation

If the swelling pressure is not excessive (on the order of 5,000 psf), a spread footing foundation can be used. The requirement will be to assign a minimum dead load pressure of 5000 psf so that the volume change of the soil will not be allowed even in excessive wetting condition, In the design of the footing foundation, it may be possible to allow certain amounts of uplift movement so as to minimize the required dead load pressure. Uplift movement can be tolerated in certain structures in the same manner as some settlement can be tolerated in some structures. A differential uplift of three fourths of an inch is considered to be tolerable (Chen, 1975). With $\frac{3}{4}$ inch allowable differential uplift, the required dead load pressure can be drastically reduced. From the pressure versus volume change curve and the tolerable uplift, a working swelling pressure can be established.

Footing foundations can be successfully placed in expansive soil provided the one or more of the following criteria are met:

- (i) Sufficient dead load pressure is exerted on the foundation.
- (ii) The structure is rigid enough so that differential heaving will not cause cracking
- (iii) The swelling potential of the foundation soil can be eliminated or reduced.

2.7.2.1 Continuous Footing

The most common type of foundation for lightly loaded structure is the continuous footing. It should be noted that continuous spread footing cannot be expected to function well in highly expansive soil areas. The use of this system should be limited to soil, with a low degree of expansion, those having a swelling potential of less than 1 percent and a swelling pressure of less than 3000 psf (144 kPa). It will be necessary to use very narrow footing ($B \approx 12"$) for single storey buildings.

2.7.2.2 Pad Foundations

The pad foundation system consists essentially of a series of individual footing pads placed on the upper soil and spanned by grade beams. The principle of the pad foundation system is similar to that of drilled pier foundation in that the load of the structure is concentrated at several points, the difference being that pads bear on the upper soils and skin friction is not involved.

By using an individual pad foundation system, it is theoretically possible to exert any desirable dead load pressure. Actually, the capacity of the pad is limited by the allowable bearing capacity of the foundation soils. To allow for the concentration of dead load pressure on the individual pads, a void space is required beneath the grade beam and should be constructed in the same manner as grade beams and pier system.

2.7.2.3 Footing on Selected Fill

The removal of natural expansive soils and their replacement with nonexpansive soil is the most obvious method of preventing structural damage due to soil heaving. In a few cases, it may be possible to completely remove the expansive strata, thus eliminating heaving problem. In most cases, the expansive material extends to too great depth to allow complete removal and backfilling.

2.7.2.4 Mat Foundation

The mat foundation should be designed to resist both positive and negative moment. Positive moment includes that induced by both dead and live load pressure exerted on the slab. Negative moment consists mainly of those pressures caused by swelling of the under-slab soil. Since the swelling pressure in an expansive soil area can be high negative moment consideration generally controls the design of mat foundation. If all

structural elements are placed on a stiffened slab, then slabs movement will not affect the stability of the structure. However, there could be tilting of the mat, but the performance of the building would not be structurally affected. Such conception has been studied by the "Building Research Advisory Board" [Ref.: Chen (1975)].

2.7.3 Slabs on Expansive Soils

Slab-on-ground construction, when on expansive soils, is a very difficult aspect to control. In the category of slabs are interior floor slabs, exterior sidewalks or apron. Generally, floor slabs do not support any appreciable live load, and the dead load actually exerted on the slab is small. consequently, movement of the slab is to be expected when the underslab moisture content increases, and it should be designed accordingly.

2.7.3.1 Slab-on-Ground

Concrete slabs, placed directly on the ground, are much less expensive than structural floor slab type construction. Most of the residential houses, warehouse structures call for the use of slab-on-ground construction. It was not until the discovery of the expansive soil problem that engineers began to question the wisdom of using slab-on-ground construction.

Slab-on-ground, sometimes referred to as slab-on-grade, are concrete slabs placed directly on the ground with little consideration given to their structural requirement. These slabs are constructed both with or without reinforcement. The unreinforced slabs are generally constructed in residential houses or where light floor load is expected. A lightly reinforced slab is normally reinforced with temperature control as a prime design factor. The Portland Cement Association [Ref.: Chen (1975)] recommended the use of a 4-inch-thick slab reinforced with No. 3 bar at 24 inches on center each way for slabs placed in moderately swelling soil areas. For high swelling soil areas, the Association recommended the use of No. 3 bars at 18 inches on center each way. Slab-on-ground construction on expansive soil will always pose a cracking and heaving problem unless the subgrade soils are treated or replaced. Minor floor cracking of slab-on-ground construction is difficult to prevent.

2.7.3.2 Structural Floor Slabs

The best method to prevent floor movement is to construct a structural slab supported on each side by grade beams and provide a void beneath the slab to prevent contact between the soil and the slab. But the system is very expensive.

2.7.3.3 Floating Slabs

A floating slab refers to a slab-on-ground construction in which the slabs are totally separated from the grade beam and building structure. Theoretically, the slab is capable of moving independently without being in contact with the surroundings structure. Interior floor slabs should be totally separated from the grade beams and interior columns to allow for free slab movement.

2.8 ELIMINATION OF SWELLING

Swelling can be eliminated using one of the following methods:

- (i) Replacement by non-swelling soil
- (ii) Moisture control
- (iii) Prewetting
- (iv) Soil stabilisation

2.8.1 Replacement by Non-Swelling Soil

A simple and easy solution for slabs and footings on expansive soils is to replace the foundation soil with non-swelling soils. Experience indicates that there is no danger of foundation movement of the subsoil, if it consists of more than about 1.5 m of non-swelling soil underlain by highly expansive soils (Chen 1975). According to Murthy (1992), excavation may be carried out up to a depth greater than the depth of the foundation by about 20 to 30 cm. Freely draining soil, such as a mixture of sand and gravel is filled up and compacted up to the base level of the foundation. Reinforced concrete footing is constructed at this level and over this brick wall may be constructed. Mixture of sand and gravel is filled up loosely over the footing. A reinforced concrete apron of about 2 m wide is provided around the building to prevent moisture from directly entering the foundation.

A cushion of granular bed below the foundation absorbs the effect of swelling and thereby its effect on the foundation will be considerably reduced. A foundation of this

type has to be constructed only during the dry season when the soil has shrunk to its lowest level. The main requirement for the replacement soils is that it should be non-expansive. All granular soils ranging from GW to SC may fulfil this requirement. However, granular soils with fines are preferable. The degree of compaction should be 90 percent of Standard Proctor density for supporting slabs and 95 to 100 percent for supporting footings. A minimum thickness of the fill beneath the bottom of footing and floor slabs should be 1 m, although 1.5 m is preferable (Singh, 1992). The replacement should also extend laterally for more than about 2 m beyond the building lines (Chen 1975).

Khan (1995) investigated the effect of sand layer on the swelling of the underlying expansive soil. The investigation reveals that the effect of sand layer on swelling of underlying expansive soil layer is significant. Even a small thickness of sand bed on expansive soil reduces both the percentage volume change and swelling pressure appreciably. It has been found that under a given surcharge pressure, the amount of percentage volume change reduces with the increase of H/h ratio (h = thickness of sand layer and H = total thickness of soil layer). Swelling pressure has also been found to decrease considerably with the increase of H/h ratio.

2.8.2 Moisture Control

Ever since the acknowledgment of expansive soil problems, engineers have been attempting to isolate water from the foundation structure. It is a relatively simple undertaking to remove free water which may seep into a building foundation by providing adequate surface drainage and properly installed subdrainage systems. However, it is difficult to isolate the migration of moisture from an exterior location to a covered area. Vapour barriers, both horizontal and vertical, have been used with only a limited degree of success in impeding moisture migration. Further research is necessary in both the field and laboratory to establish a practical and economical method of controlling moisture migration.

2.8.2.1 Horizontal Moisture Barriers

Horizontal moisture barriers can be installed around a building in the form of membranes, rigid paving or flexible paving. The purpose of the horizontal barriers is to prevent excessive intake of surface moisture. The use and effectiveness of these moisture barriers are discussed below.

Membranes

Horizontal impervious membrane along the exterior walls may be provided. Before the installation of membrane lime-water slurry may be sprinkled on the soil along the exterior wall, preferably during the dry season. This horizontal moisture barrier (2 m in length) is a combination of polythene membrane (10 mm thick and glued to the exterior wall) extending beyond the limits of backfill and loose gravel (50 mm thick) placed on top of membrane. A timber plank (25 mm by 100 mm cross-section) may be installed along the edge of the membrane. The slope of the moisture barrier should be 20 : 1 (horizontal : vertical). The purpose of the impervious membrane is to prevent surface water from seeping through the backfill into the building and to prevent the growth of weeds.

Concrete Aprons

The installation of concrete aprons or sidewalks has been found effective in controlling moisture fluctuation. The advantage of using concrete aprons rather than plastic membrane is that the former offers a positive barrier to water. Obviously, the wider the concrete apron, the more protection it offers to the building.

The function of the apron is to move the marginal moisture variation away from the building. While the use of concrete aprons around the exterior of the building may prove beneficial, care should be exercised in obtaining an effective seal between the aprons and the foundation walls. Swelling soils can heave an apron so that surface drainage is toward the building rather than away. With poorly constructed joints, water will enter the joint and seep into the foundation soil. Thus an apron can be damaging as well. In those areas where concrete aprons are used, constant care and maintenance is required.

Asphalt Membranes

Asphalt membranes can be used to cover the surface of expansive soils so that nonexpansive fill can be placed on top of the membrane. This will minimize the infiltration of surface water into the underslab soils. Where slab-on-ground constructions required, such treatment can be very advantageous. The amount of asphalt cement required to construct a membrane, according to the Asphalt Institute, is about 1.3 gallons per square yard. Research conducted by the Asphalt Institute advocates that asphalt membranes constructed from catalytically blown asphalt can be effective in preventing moisture from intruding into subgrade soils. Van London (1953) used the 50-60

penetration asphalt membrane to completely envelop the highway embankment. The purpose of the membrane was to maintain a constant moisture content in the embankment soil, thus preventing volumetric change of the fill material. Another type of asphalt membrane consisted of prefabricated asphalt sheets. Less than one-half inch thick, 3 to 4 feet wide, and up to 20 feet long. Such material can be conveniently handled and easily placed.

2.8.2.2 Vertical Moisture Barrier

Vertical moisture barriers have been used around the perimeter of the building to cut off the source of water that may enter the underslab soils. Theoretically, vertical barriers should be more effective than horizontal barriers in minimizing seasonal drying and shrinking of the perimeter foundation soils, as well as maintaining long term uniform moisture conditions beneath the covered area. The installation of a vertical barrier prevents edge wetting due to lateral moisture migration within the depth to which the membrane extends. By installing a moisture barrier, the potential for damage would be less because of the slower rate of moisture migration and more uniform moisture content of the soil at any particular time.

Vertical moisture barriers should be installed to a depth equal to or greater than the depth of seasonal moisture change. Vertical moisture barriers should be installed 2 to 3 feet from the perimeter foundation to permit machine excavation of the trench for the membrane. The vertical barrier is sometimes attached to a horizontal barrier to prevent wetting between the vertical barrier and the building.

Theoretically, vertical moisture barriers have a distinct advantage over horizontal moisture barriers. However, in view of the high cost involved in the installation of a vertical moisture barrier, especially where great depth is required, it is doubtful that such an installation is of sufficient merit to warrant the expense (Chen, 1975)

2.8.2.3 Subsurface Drainage

The purposes of a subsurface drainage system are as follows :

- (i) Intercept the gravity flow of free water.
- (ii) Lower the ground water or perched water and
- (iii) Arrest the capillary moisture movement and movement of moisture in the vapour state.

Intercepting Drains

Intercepting drains are effective in minimising the wetting of the foundation soils where the wetting is due to gravity flow of free water in a subsurface pervious layer such as a layer of gravel or fissured clay. Intercepting drains are most effective when located along the toe of a slope where ground water leaves the deep strata and where it may emerge to the surface. When a structure is located near an irrigation ditch or canal with a leakage problem, the installation of an intercepting drain will protect against the infiltration of seepage water. Intercepting drains are also widely used for improving slope stability and preventing landslides.

Peripheral Drains

Peripheral drains can be installed around either the interior or exterior of the building. The sub-drainage system is effective in minimizing general wetting of the foundation soils, which occur not only because of gravitational flow of free water but also because of moisture migration. Moisture migration includes capillary moisture movement in the liquid state and movement of moisture in the vapour state due to temperature differential. Where the water table is deep, capillary action and vapor transfer are probably the major causes of wetting of the moisture-deficient soils in a covered area. The gravel used to fill the sub-drain trench should have a gradation between $\frac{3}{4}$ and 3 inches in size with percent of fines less than 5.

2.8.2.4 Surface Drainage

The ground surface around a building should be graded so that surface water will drain away from the structure in all directions. Moisture change at the perimeter of the building appears to be the most significant contributor to damage. Therefore, by improving the drainage, a beneficial effect is inevitable.

Sprinkling System

Lawn sprinkling systems often create foundation soil problems. Lawn sprinkling systems should be installed at least 10 feet from the building. Nozzles of the sprinkling system should never be directed toward a building. An automatic timing device should be provided for all sprinkling systems so that excessive watering is avoided.

Vegetation

Many studies have shown that large bushes and trees can cause differential drying of the foundation soils and results in damage to the building from shrinkage (Hammer and Thompson, 1966). Most of the damage caused by shrinkage takes place in non-swelling or low-swelling soil areas. It is doubtful whether large trees will pose a problem in high swelling soil areas. Nevertheless, it is good practice to plant trees and shrubs at least 10 feet from a structure.

Roof Drain

Roof downspouts must be directed away from a structure so that water will not seep into the foundation soils. The downspouts should extend well beyond the perimeter of the foundation and should discharge to an area where the surface drainage is adequate to carry off the water rapidly and prevent any possible ponding of water

2.8.3 Prewetting

The prewetting theory is based on the assumption that if soil is allowed to swell by wetting prior to construction and if the high soil moisture content is maintained the soil volume will remain essentially constant. Prewetting leads to achieve a no-heave state and therefore structural damage will not occur.

The present prewetting practice usually involves direct flooding or ponding of the building area. The foundation and floor area is flooded by constructing a small earth berm around the outside of the foundation trenches to impound the water. Another practice includes first prewetting the foundation trenches then placing the foundation which is used as a dike to flood the floor area. In some cases where the moisture content at footing depth is stable, it is possible to place concrete footings to utilize them as dikes so that only floor area is prewetted.

Experience in southern California reported by Portland Cement Association indicates that prewetting moderately expansive soils to a condition of 85 percent saturation at a depth of 2½ feet is often satisfactory. In the case of highly expansive soils, prewetting to as much as 3 feet may not be sufficient. For slab-on-ground construction, after completing of the prewetting treatment, the ground surface must be kept moist until the slab is placed. A gravel or sand bed 4 to 6 inches thick should be placed over the

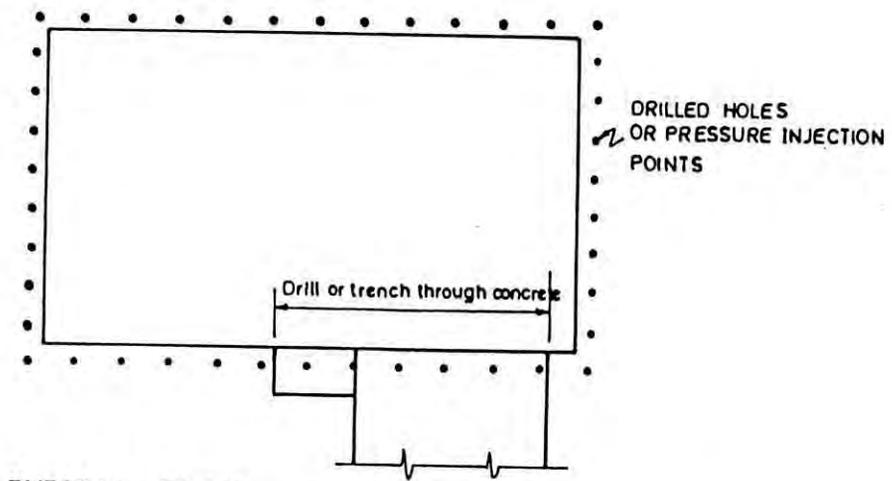
subgrade prior to the prewetting period. The gravel layer prevents the clay from drying and shrinking.

2.8.4 Soil Stabilisation

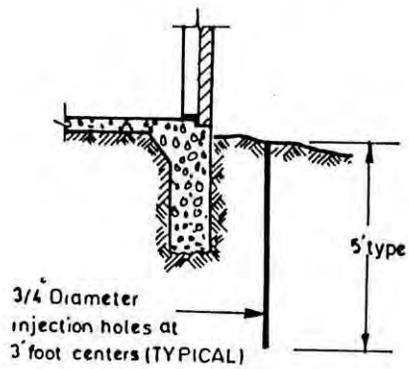
The use of lime to stabilise subgrade soil has been known to engineers all over the world for a long time. It is generally recognised that the addition of lime to expansive clays will reduce the plasticity of the soil and, hence, its swelling potential. The amount of lime required to stabilise the expansive soils ranges from 2 to 8 percent by weight. The pressure injection method of lime stabilisation has been used in Jackson, Mississippi, in Calexico, California, and in Tucson, Arizona (Chen, 1975). The method consists of pressure injecting lime-water slurry into the soil through closely spaced drill holes as shown in Fig. 2.14. The drilled holes are 5 feet deep. This is normally carried out in dry season. The method consists of pressure injecting lime-water slurry (2500 pounds, i.e., 1134 kg of hydrated lime in 900 US gallons, i.e., 3.40 m³ of water) into the soil through closely spaced drill holes. The drilled holes should be 1.5 m deep, located adjacent to the building, and on 1 m centres. Lime and water slurry should be mixed in a blending tank prior to injection through a two-pipe system. The outer pipe is 19 mm diameter, pointed at bottom and perforated in lower foot with 3 mm diameter holes. The inner pipe is 6 mm diameter. The pipes are jetted in the ground. Injection of slurry should be continued in each hole until slurry comes out of ground around the pipe. The injection pressure at nozzle should be in the range of 1500 kPa to 2500 kPa.

If lime slurry pressure injection method is not possible, all backfill around the building can be removed and lime-water slurry should be sprayed at the foundation bottom level. The backfill may be replaced with non-expansive soil and the backfill may be stabilised using lime. For stabilisation 6 to 7 percent lime would be adequate. This technique is shown in Fig. 2.15. After one year of lime-water slurry treatment impervious horizontal membrane along the exterior walls of the buildings may be provided.

Spangler and Patel (1949) reported results on laboratory treatment of an expansive soil with Portland cement. The addition of 2% and 4% of cement significantly reduced the potential volume change of the soil. Jones (1958) reported that addition of 2% to 6% of Portland cement to the expansive Porterville clay of California resulted in considerable reduction of volume change characteristics. Both cement and lime have been used in highway construction for modifying the swelling property of the subgrade soil.



TYPICAL PLAN



DETAIL

Fig. 2.14 Pressure injection method of lime stabilisation (after Chen, 1975)

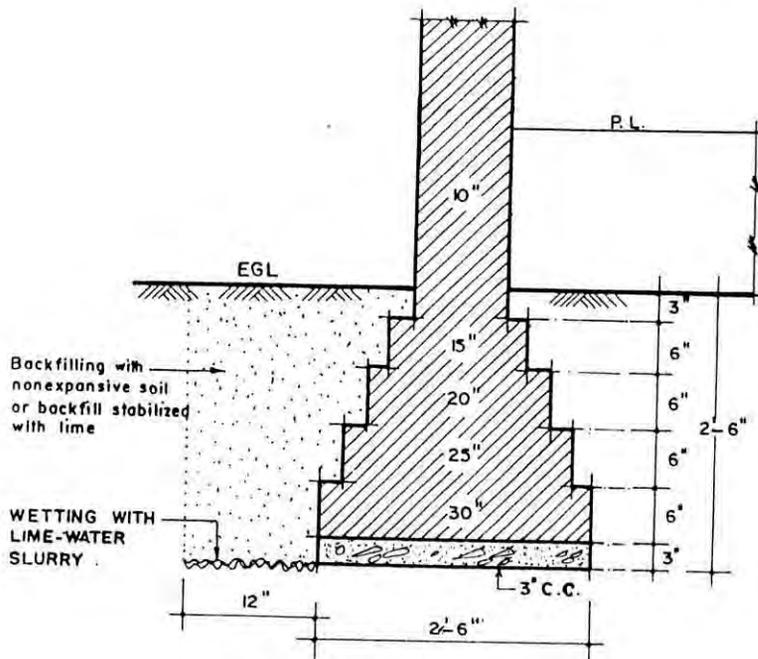


Fig. 2.15 Lime-water slurry treatment at the foundation bottom level (after BRTC, 1997)

2.9 LIME STABILISATION

The objectives of mixing additives with soil are to improve volume stability strength and stress strain permeability and durability. The development of high strength and stiffness is achieved by reduction of void space, by bonding particles and aggregates together, by maintenance of flocculent structures, and by prevention of swelling. Good mixing of stabiliser with soil is the most important factor affecting the quality of results. Feasibility of stabilisation techniques for different types of soils the most commonly used stabilisers for improving the physical and engineering properties of soils is lime. The use of lime to stabilise subgrade soil has been known to engineers all over the world for a long time. Lime stabilisation is widely recommended for construction of roads (Ingles and Met calf, 1972 ; NASARA , 1986, Hausmann, 1990). The pressure injection method of lime stabilisation has been used in Jackson, Mississippi, in Calexico, California and in Tucson, Arizona (Chen 1975).

Lime is an effective additive for clayey soils for improving workability, strength and volume stability. Lime stabilisation is suitable for more plastic clayey soils and is less suitable for granular materials. It is used more widely as a construction expedient, that is to prepare a soil for further treatment or to render a sufficient improvement to support construction traffic. A number of research works (Ahmed, 1984; Rajbongshi, 1997; Molla, 1997) was carried out in the past to investigate the geotechnical properties of lime stabilised local alluvial soils and soils from coastal regions.

This review looks into the fundamental concepts, mechanisms of lime treatment factors influencing the properties of lime stabilised soils, characteristics of lime treated soil, and applications of lime stabilisation.

2.10 MATERIALS FOR LIME STABILISATION

The materials to be considered in lime stabilisation are lime, soil and water, and it is important that the type of lime to be used is clearly defined.

2.10.1 Lime

Lime, refers to hydrated or slaked lime (calcium hydroxide), quicklime (calcium oxide), or dolomitic limes (calcium/magnesium oxide), that is, the highly alkaline (pH > 12.3) lime products. Agricultural lime (calcium carbonate) is not suitable for stabilisation. Dolomitic

lime is usually not as effective as calcium lime (i.e., hydrated or slaked lime and quicklime). In order to give a common quantitative base, lime contents are expressed as equivalent 100 per cent pure hydrated lime. On a mass basis pure quicklime is equivalent to 1.32 units of hydrated lime. All commercial lime products are likely to have impurities (carbonates, silica, alumina, etc.) which dilute the active additive but are not harmful to the stabilisation reaction.

Hydrated lime comes in the form of a dry, very fine powder or as slurry. Quicklime and dolomitic limes are commonly much more granular than the hydrated products and are available only as a dry product. These limes rapidly react with any available water producing hydrated lime, releasing considerable amounts of heat. The water content of common slurry limes can range from 80 to 200 per cent. Table 2.5 summarises the properties of hydrated, quick and slurry lime.

The efficiency of lime stabilisation depends in part on the type of lime material used. Quicklime is generally more effective than hydrated lime (Kezdi, 1979), but generally it needs care in handling for soils with high moisture contents. Unslaked lime or quicklime is more effective since water will be absorbed from the soil and more importantly, the hydration will cause an increase in temperature which is favourable to strength gain (Broms, 1986).

Table 2.10 Properties of lime (after NAASRA, 1986)

Parameters	Hydrated Lime	Quick Lime	Slurry Lime
Composition	Ca(OH) ₂	CaO	Ca(OH) ₂
Form	Fine Powder	Granular	Slurry
Equivalent Ca(OH) ₂ /Unit Mass	1.00	1.32	0.56 to 0.33
Bulk Density (kg/m ³)	450 to 560	1050	1250

2.10.2 Soil

The addition of lime has little effect on soils that contain either a small clay content or none at all. Lime has also little effect in highly organic soils and also in soils with little or no clay content. Lime usually reacts with most soils with a plasticity index ranging from 10% to

50%. Those soils with a plasticity index of less than 10% require a pozzolan for the necessary reaction with lime to take place, fly ash being commonly used. Lime is particularly suited to stabilise highly plastic clay soils. In such soils the lime will immediately create a more friable structure, which is easier to work and compact, although a lower maximum density will be achieved, and lime may be used solely for this reason as a pre-treatment to further additions of lime. Lime reacts more quickly with montmorillonitic clays than with kaolinitic clays. In montmorillonitic clays the plasticity is reduced, but this may not happen with kaolinitic clays.

The effect of soil moisture content is important only where it affects the operation of compacting or pulverising equipment by being either too low or too high. In wet clays the use of lime to effect rapid changes in plasticity is the basis of the application of lime stabilisation as a construction expedient.

2.10.3 Water

Potable water is preferred for lime stabilisation. Acidic (organic) water should be avoided. Sea water can be used but should be avoided where a bituminous seal is to be placed, as crystallisation of salts may lift the seal. The amount of water used in lime stabilisation is governed by the requirements of compaction. However, if quicklime is used then extra water may be required in soils having less than 50 per cent moisture content to provide for the very rapid hydration process. However, the moisture content of the soil at the pulverisation and mixing stage is less important than in the case of cement stabilisation.

2.11 MECHANISMS OF LIME STABILISATION

It is recognised that lime has an immediate effect on clay soils, improving its granulation and handling properties. The effect varies with the actual clay mineral present, being large with montmorillonite group clays and low to non-existent with kaolinite group clays. Lime has longer term effects on strength, causing continuing strength improvements with time.

The basic mechanisms of soil-lime interactions have been described by Eades and Grim (1960), Compendium (1987), IRC (1973a) and Hausmann (1990). The basic mechanisms that have been identified in soil-lime interaction are base exchange (ion exchange), flocculation, cementation and carbonation. These mechanisms are briefly presented in the following sections.

2.11.1 Base Exchange and Flocculation

Clay particles are usually negatively charged and they contain adsorbed exchangeable cations of sodium, magnesium, potassium or hydrogen on the surface. The strong positively charged cations of calcium present in lime replace the weaker ions of sodium, magnesium, potassium or hydrogen present on the clay surface and this base-exchange results in a predominance of positively charged calcium ions on the surface of clay particles. This reaction is usually completed within a few days of the mixing.

This change in the cation exchange complex affects the way the structural components of the clay minerals are connected together. Lime causes clay to coagulate, aggregate, or flocculate. The plasticity of clay (measured in terms of Atterberg limits) is reduced, making it more easily workable and potentially increasing its strength and stiffness.

Eades and Grims (1960) indicated to the formation of new crystalline phases in the soil lime electrolyte system due to the addition of lime to the soil in presence of water which are tentatively identified as calcium silicate hydrate. The reaction of lime with three layers material, which are montmorillonite, kaolinite, and illite begin by the replacement of existing cations between the silicate sheet with Ca^{++} . Following the saturation of inter layer positions with Ca^{++} , the whole clay minerals deteriorate without the formation of substantial new crystalline phases.

2.11.2 Cementation

Cementation is the main contributor to the strength of the stabilised soil. The higher the surface area of the soil, the more effective is this process. If lime is added in excess of the lime fixation point, complex chemical reactions similar to pozzolanic reactions are known to take place between lime and the clay minerals in the soil. These reaction products are cementitious. The aluminous and siliceous materials in clayey soil have no cementitious value by themselves but react with calcium hydroxide in the presence of water to form cementitious compounds according to the following equations:



In equations 2.14 and 2.15, CSH and CAH are cementitious products. The above reactions represented by Equations 2.14 and 2.15 are slow and long-term in nature. Long term chemical reaction of lime with certain clay minerals (silicate and aluminate) of soil in presence of water is referred to pozzolanic reaction in lime stabilisation. Moreover, these reactions are more effective when the soil-lime mixture is adequately compacted. Cementation is, however, limited by the amount of a available silica. Increasing the quantity of lime added will increase strength only up to the point where all the silica of the clay is used up; adding too much lime can actually be counterproductive. This contrasts with cement stabilisation, where strength continues to improve with the amount of admixture. Cementation on the surface of clay lumps causes a rapid initial strength gain, but further diffusion of the lime in the soil will bring about continued improvement in the longer term, measured in weeks or months.

Herzog and Mitchell (1963) indicated that soil lime pozzolanic reaction usually does not appear until after long curing period and than only in cases where a high percentage of lime was added. Pozzolanic materials (silicious or Aluminous) possess little or no cementitious value, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties. Asserson et al. (1974) worked with red tropical soils suggested that after the initial 7 days of curing, strength increases are the result of hydration and increase in crystallizing of reaction products rather than from the continued formation of additional pozzolanic compounds.

Ramie (1987) indicated that surface chemical reaction can occur and new phase may nucleate directly on the surface of clay particles while conducting research concerning the adsorption of lime by kaolinite and montmorillonite. They mentioned that it is also possible that the reactions may occur by a combination of through solution (solution-precipitation) and surface chemical (hydration-crystallization) process. Kezdi (1979) stated the dissociation of hydrated lime into Ca^{++} and OH^- causes loss of its crystalline structure and assume an amorphous form and flocculation of clay particles occurs, causing improvement of soil texture, rendering the soil more workable.

2.11.3 Carbonation

As lime absorbs carbon dioxide from the air, calcium carbonate (CaCO_3) is formed. These carbonates are relatively weak cementing agent (Hausmann, 1990). This reaction is the slowest of all the reactions involved in a soil-lime system and as in pozzolanic reaction,

requires that the mixture must be thoroughly compacted. Carbonation may be beneficial where lime is plentiful; the CaCO_3 formed will not react any further with the soil.

Eades et al. (1962) demonstrated that although carbonation does take place, the strength gain is said to occur by virtue of cementation of soil grains with calcium carbonate is negligible. Yu Kuen (1975) stated that carbonation is normally confined to the surface exposed to the air and involves the conversion of lime to the Calcium carbonate by carbon dioxide absorbed from the air.

2.12 FACTORS AFFECTING LIME STABILISATION

Properties of lime-treated soils are influenced by several factors. These factors are broadly classified as material factors and production factors. Material factors deal with the composition of the untreated soil and its response to lime. The production factors include the quality of water, lime, the uniformity of mixing and curing. The factors influencing the properties of lime-treated soil are described in the following sections.

2.12.1 Soil Characteristics

2.12.1.1 Type of Soil

For lime to be effective, there must be within the soil, clay particles or other pozzolanic materials that are reactive with the lime. Thompson (1966a) stated that the extent of improvement of the engineering characteristics of soil depends largely upon the soil type. The gain in strength of a soil lime system is mainly due to the pozzalonic reaction i.e the long term reaction between lime and certain clay minerals (silicate and aluminates) in the presence of water. He also noted that soils having larger amount of clay fraction and less amount of organic matter are very effective to lime stabilisation.

In general the more plastic the clay fines and the higher the clay content, the larger will be the lime content to produce a specific strength gain or other effect. On the other hand, the amount of bonding achievable with lime can be limited by the amount of reactive material. For lime stabilisation to be successful, the clay content of the soil should not be less than 20% and the sum of the silt and clay fractions should preferably exceed 35%, which is normally the case when the plasticity index of the soil is greater than 10 (Broms, 1986). Ingles and Metcalf (1972) did not recommend crushed rock and sands for use in lime stabilisation.

NASSRA (1970) stated that highly plastic soils are more effective to gain strength. NASSRA (1970) pointed out that soil having plasticity index in the range of 10 to over 50 are suitable for lime stabilisation. Soils with plasticity index lower than 10 do not react readily with lime, although there are some few exceptions. Ingles and Metcalf (1972) studied the effect of the unconfined compressive strength on different types of soil stabilised using lime. It was found that the strength of lime stabilised silty clay is higher than the other types of soil.

Yu Kuen (1975) stated that in general, highly plastic soils are more effective than other types of soil when stabilised with lime.

Compendium (1987) stated that lime is very effective in stabilizing the clay soils with a substantial portion of the coarse grained soil.

Rodriguez et al. (1988) noted that the maximum effect of lime is on clayey gravel soil. Sometimes, the strength increase due to lime stabilisation on these types of soil is such that the stabilised soil becomes stronger than those that would be obtained with cement. Rodriguez et al. (1988) also reported that lime has been more frequently used with plastic clays, which become more workable and easy to compact. Lime also provides volumetric stability of the soil in the presence of changing water.

Locat et al. (1990) studied the effect of four types of soil of Canada stabilised with lime. Locat et al. (1990) observed that the unconfined compressive strength of the silty clay soil is higher than the other types of soil. Fig. 2.16 shows the variation of unconfined compressive strength with lime content for four types of soil. It has been found that the maximum strength is gained by the soil with higher clay content.

Serajuddin (1992) reported the results of three types of lime treated soil of the South-West region of Bangladesh. Silt and clay types of soil were used in the investigation. The results of the investigation are shown in Fig. 2.17. It has been found that silty soil has much lower unconfined compressive strength than the clay types of soil.

The pH value of the soil which indicates its acidity or alkalinity is of great importance to lime-stabilisation. Ho and Handy (1963) have shown that for montmorillonite clays that no lime reaction occurs at pH less than 11.0. The presence of significant amounts of sulphates diminishes the effectiveness of lime. The Indian Road Congress, IRC (1976) specifications also requires that where the sulphate content is in excess of 0.2 percent, special studies would be needed to determine the efficacy of lime-treatment.

2.12.1.2 Organic Matter Present in the Soil

One of the important factors that inhibits lime-soil reaction is the organic content. One of the possible reason is that organic matter has a high base exchange capacity and when lime is added to such soils, some of the Ca^{++} ions are used to satisfy the exchange capacity of the organic matter, thus depriving the clay minerals of calcium ions for pozzolanic reactions. Ingles and Metcalf (1972) reported that organic soils should not be used in lime stabilisation. However, IRC (1973a) recommended a maximum limit of 2% organic content for lime stabilisation.

NASSARA (1970) stated that the presence of organic matter in the soil reduces the strength of the stabilised soil. He pointed that soil containing more than 3% of organic matter is very harmful to the strength development of the stabilised soil.

Arman and Muhfakh (1972) studied the effect of the percent of organic matter on the unconfined compressive strength of the lime stabilised soil. It has been found that the presence of organic matter in the soil reduce the strength of the stabilised soil to a large extent. As the organic content on the soil increase, unconfined compressive strength continues to decrease as shown in Fig. 2.18.

Holm et al. (1983) also stated that the effect of lime decreases with increasing organic content. The strength increase of lime stabilised organic soil is very low. According to them, one of the possible reasons is that organic matter has high base exchange capacity. When lime is added to organic soils me of the Ca^{++} ions are used to satisfy the exchange capacity of organic matter, thus depriving the clay minerals of calcium ions for pozzalanic tenons. Even a small amount of organic content can have a large effect on strength.

2.12.2 Lime Content

The strength of soil-lime mix is determined to a great extent by the quantity of lime added. Small quantities of lime, 1 to 2 percent, help in the immediate effects caused by the base exchange and flocculation. The tangible effect of soil-lime stabilisation in increasing the strength of the mixture begins to be felt as the lime content is further increased and this is due to pozzolanic reactions resulting in the production of cementitious compounds. It is also observed that this strength gain is time-dependent and efficiencies in strength gain due to varying lime percentages are more marked for longer curing periods.

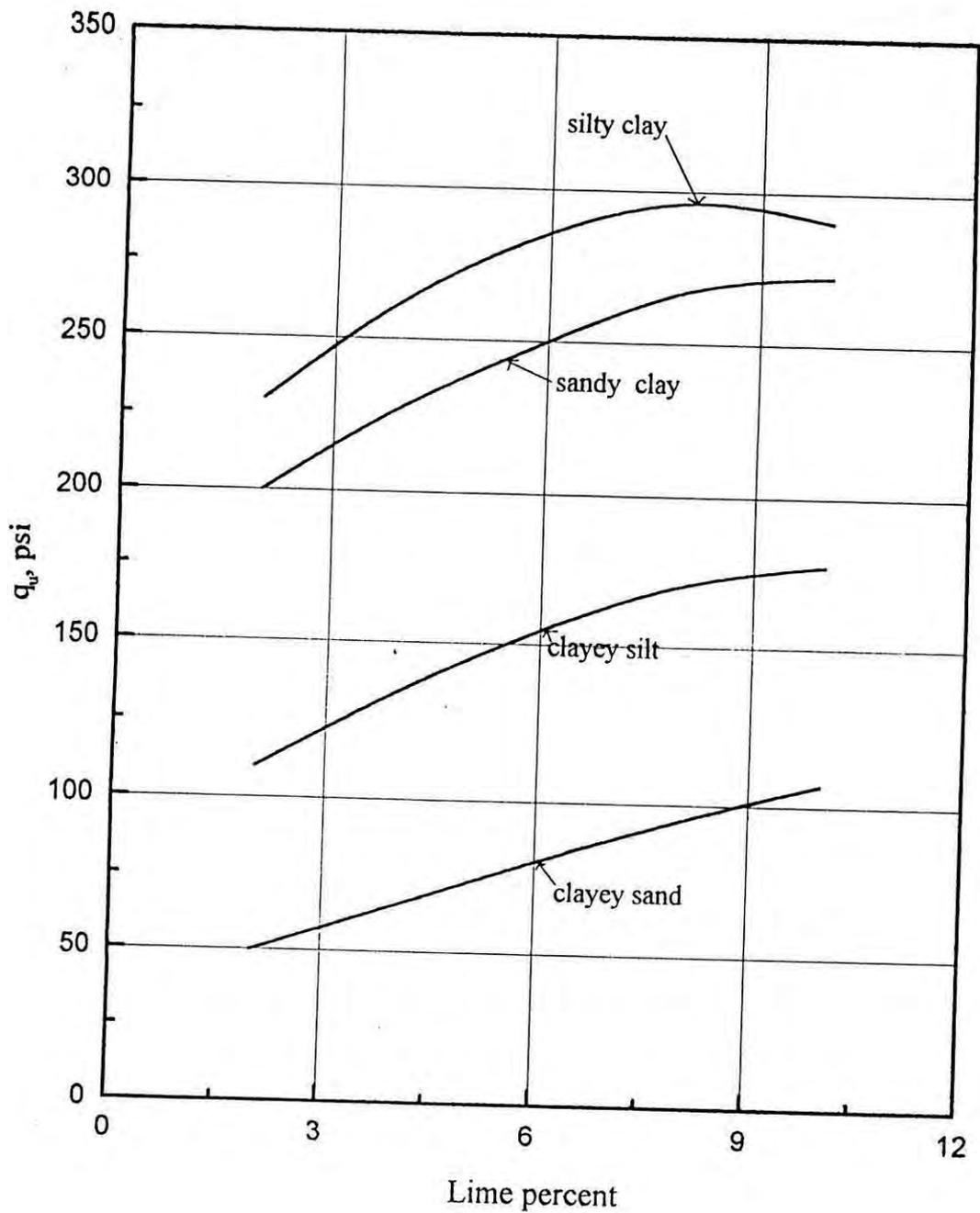


Fig. 2.16 Variation of unconfined compressive strength (q_u) with lime content for various types of soil (after Locat et al., 1990)

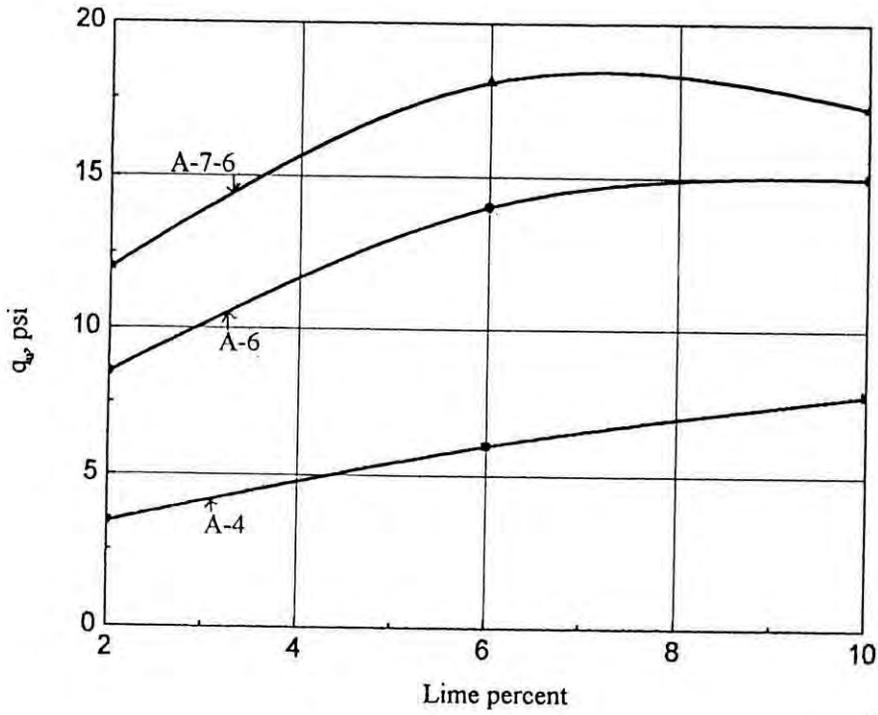


Fig. 2.17 Variation of unconfined compressive strength with lime content for different types of soil (after Serajuddin, 1992)

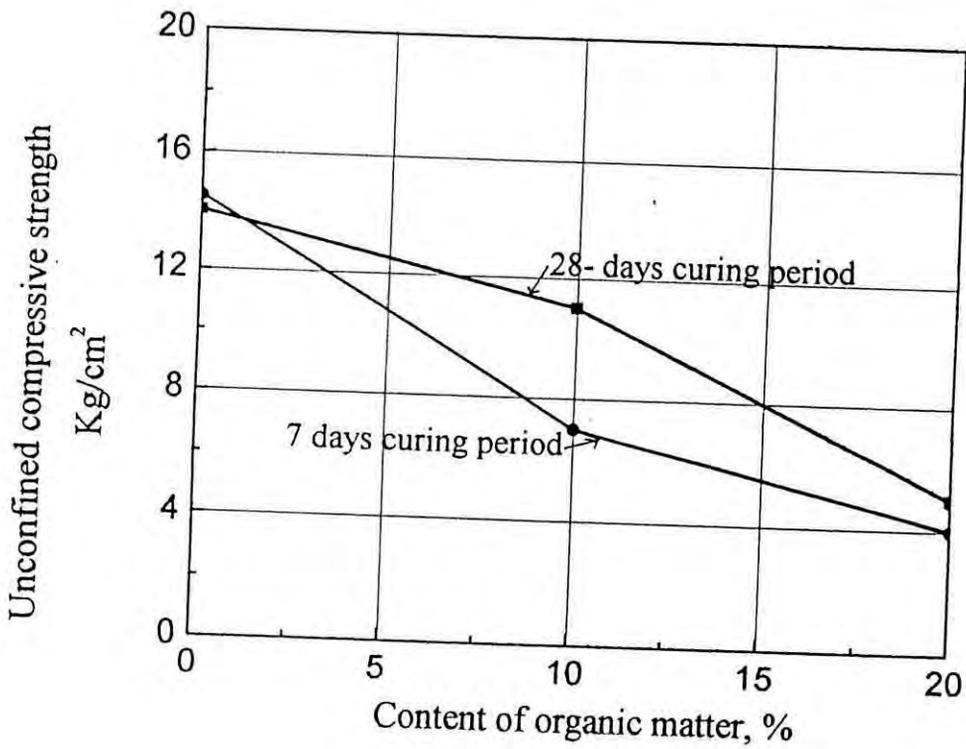


Fig. 2.18 Effect of organic matter on unconfined compressive strength of lime treated soil (after Arman and Muhfakh, 1972)

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Ingles and Metcalf (1972) suggested that the addition of up to 3% of lime would modify well graded clay gravels, while 2% to 4% was required for the stabilisation of silty clay, and 3% to 8% was proposed for stabilisation of heavy and very heavy clays. Ingles and Metcalf (1972) further suggested that a useful guide is to allow 1% of lime (by weight of dry soil) for each 10% of clay in the soil. Hausmann (1990) stated that the practical lime content for lime stabilisation varies from 2% to 8%. Variation of the unconfined compressive strength of the lime stabilised soil due to the variation of the lime content as found by Molla (1997) is shown in Fig. 2.19 for three regional soils of Bangladesh. It can be seen from Fig. 2.19 that the unconfined compressive strength of the lime stabilised soil increase with the increase of lime content for all the three soil types.

Optimum lime content is the lime content by which the maximum strength of the lime stabilised soil can be achieved. Researchers stated different criteria for optimum lime content. Herrin and Mitchell (1961) pointed that there appears to be no optimum lime content in the lime stabilised soil which will produce a maximum strength of the soil under all conditions. However, it can be stated that for a particular condition of soil type and curing time, there is a corresponding lime content which will produce maximum strength.

Based on intensive investigation at the Iowa State University, Diamond and Kinter (1965) defined optimum lime content as one at which the percentage of lime is such that additional increments of lime will produce no appreciable increase in the plastic limit. According to them, lime content above the lime fixation point for a soil will generally contribute to the improvement of soil workability, but may not result in sufficient strength increase. Hilt and Davidson (1960) suggested that the plastic limit is the indicative only of the optimum lime content in clayey soil and it is necessary to use additional amount of lime to permit the formation of cementing materials within clay soil for strength increase.

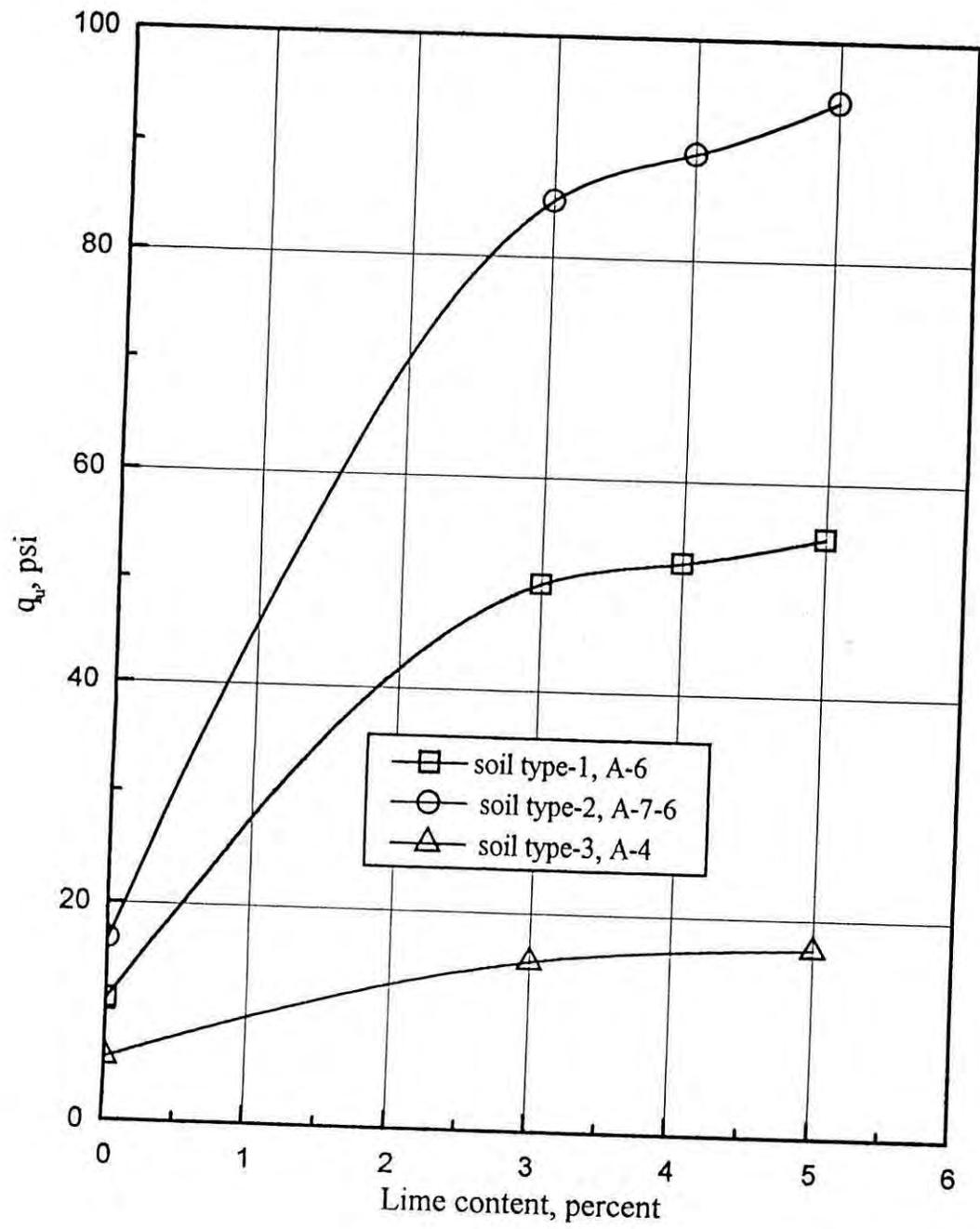


Fig. 2.19 Variation of the unconfined compressive strength of lime stabilised soil due to variation of lime content (after Molla, 1997)

2.12.3 Mixing and Compaction Procedure

2.12.3.1 Compactive Effort

The success of lime-soil stabilisation technique depends to a great extent on adequate compaction of the mixture. Compaction is considered to be necessary for bringing the clay minerals into close and intimate contact with the lime particles so that the inter-growth of crystalline reaction products is facilitated (Croft, 1964). With soil-lime mixture, the greater the compactive effort, the more is the strength attained. Taking typical data from Remus and Davidson (1961), a calcitic lime (6 percent) used with glacial till soil yielded an unconfined compressive strength (7 days cure and 24 hours immersion) of 250 psi at Standard AASHTO compaction. For the same conditions, but with modified AASHTO compaction, the strength increased to 525 psi.

Compendium (1987) stated that the maximum dry density normally continues to decrease as the lime content is increased. In addition, the optimum moisture content increases with increasing lime content.

Hausmann (1990) pointed that flocculation and cementation will make the soil more difficult to compact, therefore, the maximum dry density achieved with a particular compactive effort is reduced. Faisal et al. (1992) noted that the addition of lime leads to decrease in the dry density of the soil and an increase in optimum moisture content, for the same compactive effort. The decrease in maximum dry density of the treated soil is the reflection of the increased resistance offered by the flocculated soil structure to that compactive effort. Faisal et al. (1992) also noted that the increase in optimum moisture content is probably a consequence of additional water held within the flocculated soil structure resulting from lime interaction with soil.

Dunlop (1977) observed that unconfined compressive strength of the lime stabilised soil is increased about 15% percent for Modified Proctor test method than the Standard Proctor test method, about 25% reduction of strength at about half of the Standard Proctor compactive effort. Dunlop (1977) also stated that strength of the stabilised soil is also dependent upon the uniformity of the compaction. He showed that increasing the number of blows per layer from the standard compactive effort but keeping the weight less than the standard compactive effort and reducing the falling height gives as much as 10% increase in strength.

Serajuddin (1992) reported lime stabilised soil attains higher strength and density in Modified Proctor test method than the Standard Proctor test method. Serajuddin (1992) also observed that the compactive effort has a large effect on the CBR value of the lime stabilised soil. Serajuddin (1992) found that the CBR value of the stabilised soil is as twice in the Modified Proctor test method than the Standard Proctor test method. It has also been reported that unconfined compressive strength of the lime stabilised soil increase about 25% percent in the modified proctor test method than the standard proctor test method and about 40% in reduction of strength at about half of the compactive effort in the standard proctor test method.

Molla (1997) investigated the effect of the amount of compaction energy on unconfined compressive strength of three regional soils ($LL = 34 - 47$, $I_w = 9 - 26$) of Bangladesh. Molla (1997) reported that unconfined compressive strength increases with the increase in compaction energy as shown in Fig. 2.20.

2.12.3.2 Compaction Delay Time

Compaction delay time is the time interval between mixing of lime with soil and compaction. Mitchell and Hooper (1961) from their experiments on an expansive clay reported that a delay between mixing and compaction is definitely detrimental in terms of density, swell and strength for samples under the same compactive effort. Croft (1964) also concluded that compaction should proceed immediately. The sooner the particles are brought into contact with one another, the greater will be the final strength achieved and prolonged delays will certainly be detrimental. The IRC (1973b) stipulates a maximum time lag of 3 hours between mixing and compaction for the construction of roads and runways.

NAASRA (1986) suggests that if high strengths are required, then this can best be obtained by early compaction as these results in high densities. Delayed compaction lowers density but the rate of reduction in maximum density is nowhere near as rapid as with cement. If soils are wet, a delay can be used to improve handling and compactability. Conversely, with dry soils a delay in compaction, will increase the moisture requirements.

Townsend et al. (1970) observed that the compaction delay time of 24 hours can reduce the strength of the specimen upto 30% as compared to the specimen prepared by compacting immediately after mixing.

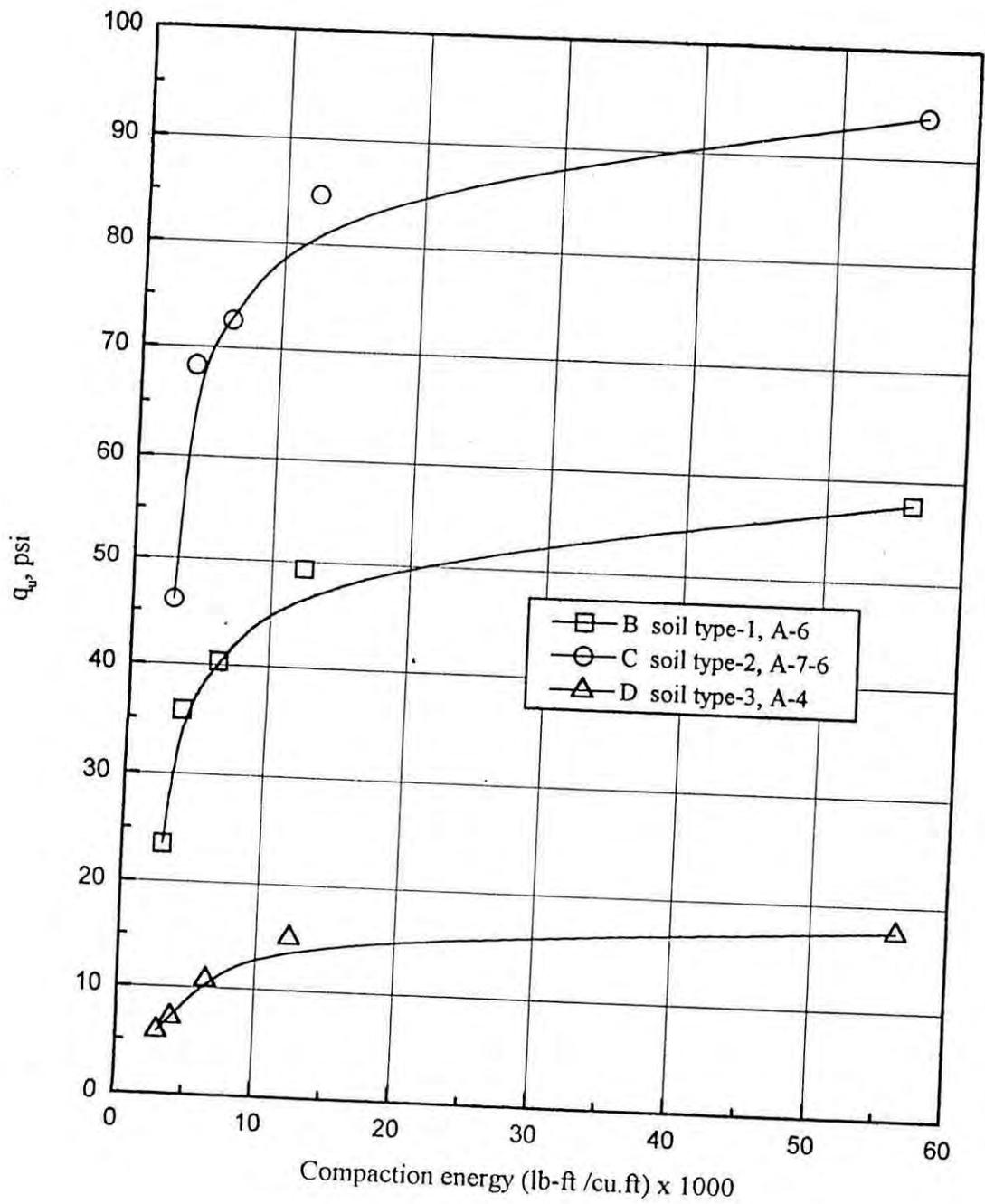


Fig. 2.20 Variation of unconfined compressive strength (q_u) at different compactive effort for stabilised soils using 3% lime (after Molla, 1997)

Sastry et al. (1987) observed that for a delay period of time for two hours between mixing and compaction, there is practically no reduction in strength. But for further delay the strength of soil lime mixture continues to fall. By an independent study Sastry et al. (1987) observed the delay for 96 hours between mixing and compaction, strength of the soil lime mixture continues to fall in the same trend.

Compendium (1987) stated that granular soil-lime mixture should be compacted as soon as possible after mixing, although delays up to two days are not detrimental, especially if the soil is not allowed to dry out. Fine grain soils can also be compacted, soon after final mixing, although delays of up to 4 days are not detrimental.

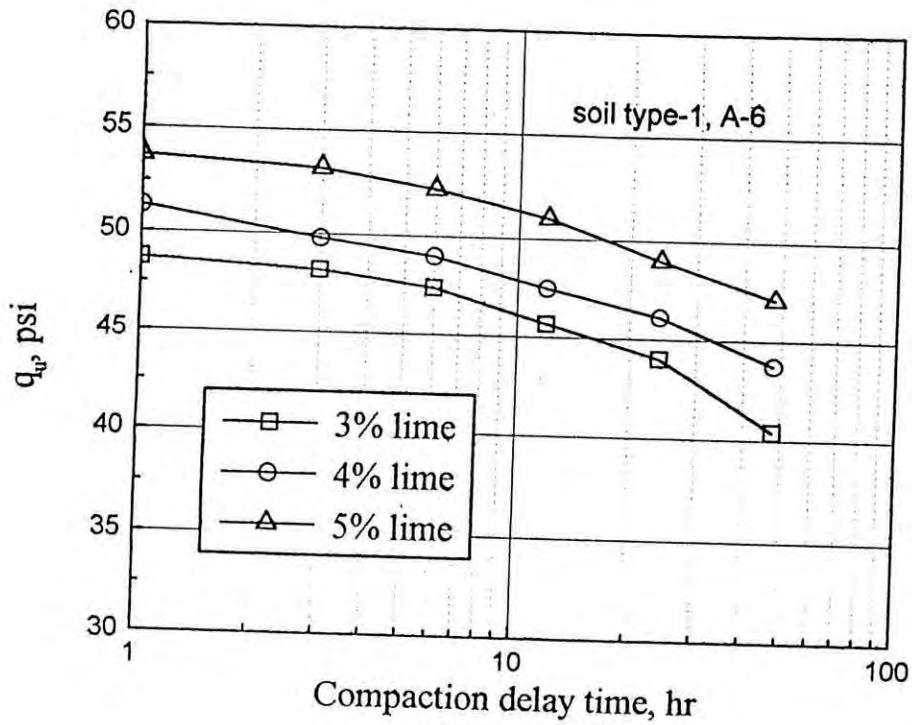
Boominathan and Prasad (1992) stated that compaction delay of 24 hours can decrease the strength from 30% to 70%. Boominathan and Prasad (1992) reported that the reduction in strength and density are attributed to granulation of loose soil particles by weak cementation, as the soil mellows.

Molla (1997) investigated the effect of compaction delay time on unconfined compressive strength of three regional soils of Bangladesh. Molla (1997) reported that unconfined compressive strength decreases with the increase in compaction delay time. This trend is presented in Fig. 2.21 for two soils.

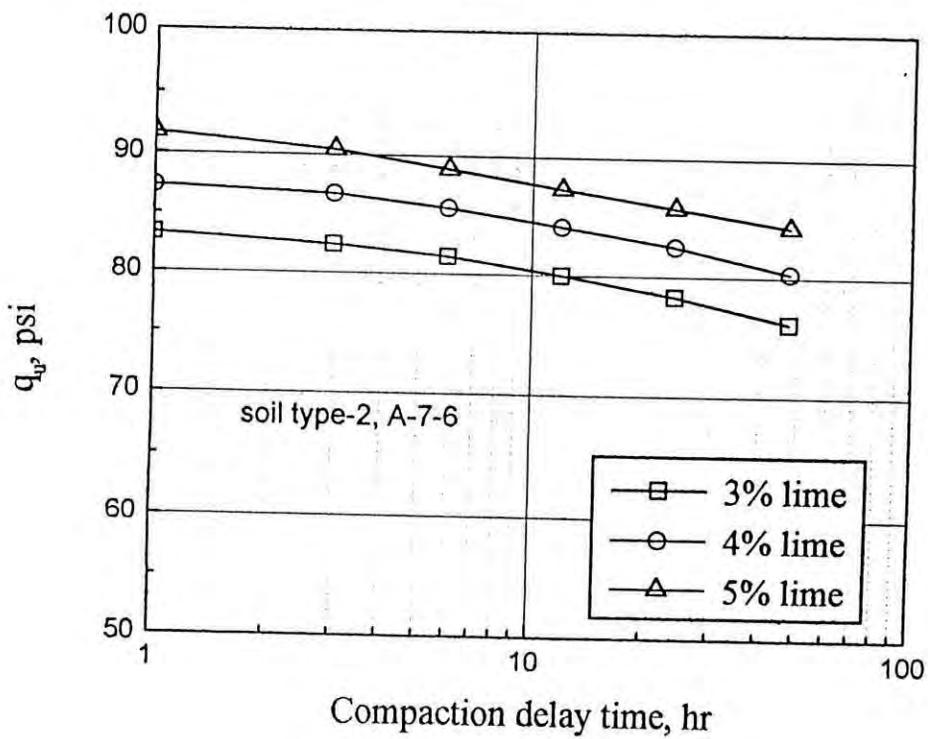
2.12.4 Curing Time and Curing Conditions

The shear strength of lime-treated soils increase with time in a manner similar to concrete or soil-cement mix. The rate of increase is generally rapid at the early stage of curing time and thereafter the rate of increase in strength reduces with time. Though strength gains do occur even after prolonged curing, the soil-lime mixtures are normally designed for a curing period of 7 to 28 days (IRC, 1976). Broms (1986) reported that shear strength of stabilised clays will normally be higher than that of untreated clay after mixing.

Hilt and Davidson (1960) conducted unconfined compressive strength test on lime stabilised silty clays and found that the rate of strength gain is relatively constant upto 150 days, after which the rate slowed.



(a)



(b)

Fig. 2.21 Variation of unconfined compressive strength (q_u) with compaction delay time
 (a) Soil type A-6 (b) Soil type A-7-6 (after Molla, 1997)

Ingles and Metcalf (1972) also studied the effect of time on the unconfined compressive strength. The variation of strength for the different curing age as found by Ingles and Metcalf (1972) is presented in Fig. 2.22. From Fig. 2.22, it can be seen that strength gain of the lime stabilised soil is highly dependent upon the soil type. For some soil the rate of increase in strength with curing time is high but for some soil the rate is slow.

The temperature at which soil-lime mixtures are cured has a profound effect on the strength characteristics (IRC, 1976; Broms, 1986). Low temperatures are not suitable for the chemical reactions that are necessary for the cementitious action. The chemical reactions in the soil are favoured by a high temperature. In fact, one of the limitations of soil-lime stabilisation is the climatic factor. It is found that reactions are not effective at temperatures below 50°F and therefore under such circumstances, soil-lime stabilisation is not desirable (IRC, 1976). The rate of strength gain is temperature sensitive and there is some evidence that the physical form of the cementitious products is sensitive to curing temperatures (Ingles and Metcalf, 1972; Bell, 1993). The effect of curing temperature and time on unconfined compressive strength on a plastic clayey soil stabilised with 5% lime is shown in Fig. 2.23. It can be seen from Fig. 2.23 that for a particular curing age unconfined compressive strength increases considerably with curing temperature and that at a particular temperature strength increases with increasing curing age.

2.13 PROPERTIES OF LIME STABILISED SOIL

The main benefits of lime stabilisation of clays are imported workability, increased strength, and volume stability. The properties of soil-lime mix have been summarised by a number of investigators (Ingles and Metcalf, 1972; IRC, 1976; Mitchell, 1981; Kezdi, 1979; NAASRA, 1986; TRB, 1987; Bell, 1993). In the following sections the various physical and engineering properties of lime stabilised soils are reviewed.

2.13.1 Plasticity and Shrinkage Properties

Substantial changes in the plasticity properties are produced by lime treatment. The liquid limit generally reduces with increasing quantity of lime. This observation is by and large true for clayey soils. In general, liquid limit decreases in the more plastic soils, and increases in the less plastic soils (IRC, 1976).

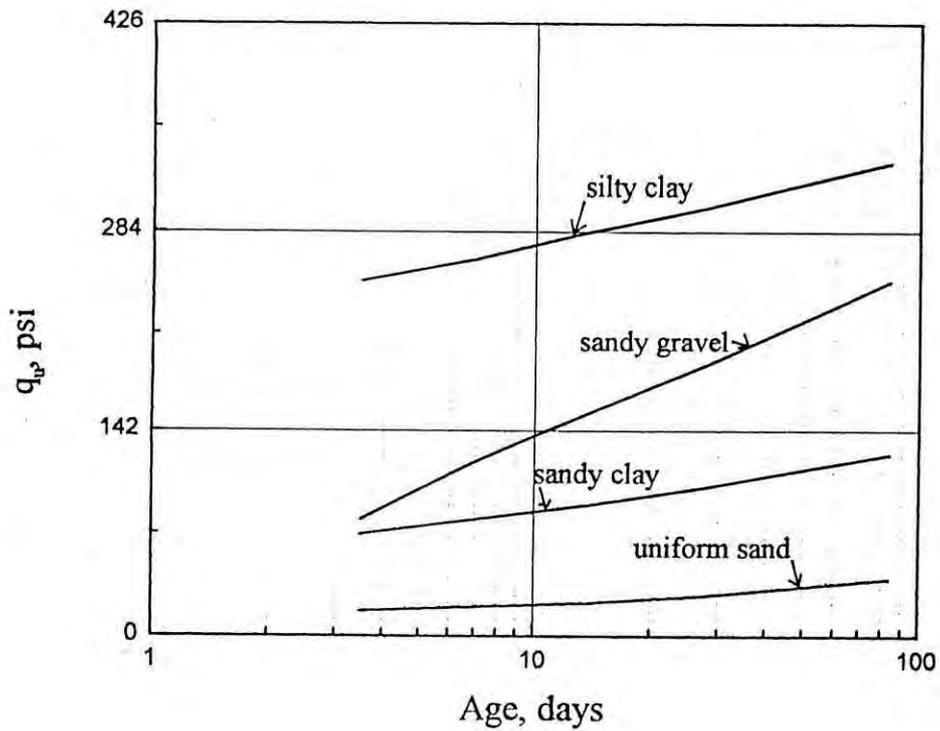


Fig. 2.22 Effect of curing age on unconfined compressive strength (q_u) for various types of soils stabilised with 5% lime (after Ingles and Metcalf, 1972)

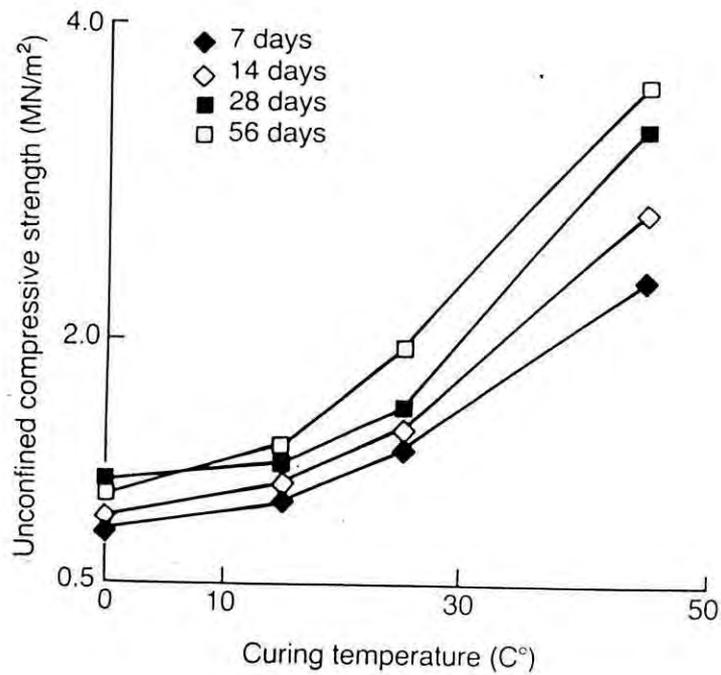


Fig. 2.23 Effect of curing temperature and curing age on unconfined compressive strength of a clay of high plasticity stabilised with 5% lime (after Bell, 1988)

Irrespective of the reduction or increase in the liquid limit of the mixture, the plastic limit increases with the addition of greater percentages of lime, whether the specimens are tested immediately or after a lapse of time. The plastic limit increases with the addition of lime up to some limiting lime content and any increase thereafter causes insignificant or no increase (Mateous, 1964). As a result of the general decrease in liquid limit and a good rise in the plastic limit, the plasticity index drops down very considerably and in many cases the soil may become nonplastic (Mateous, 1964; Rodroquez et al., 1988). Generally, soils with a high clay content or soils exhibiting a high initial plasticity index require greater quantities of lime for achieving the nonplastic condition, if it can be achieved at all. The amount of reduction in the plasticity index varies with the quantity and type of lime and also type of soil (IRC, 1976).

Holtz (1969) reported the effects of lime on plastic characteristics of four expansive montmorillonitic clays. These results are presented in Fig. 2.24. Holtz (1969) found that lime drastically reduces liquid limit and plasticity index and drastically raises the shrinkage limit of montmorillonitic clays, as shown in Fig. 2.24.

Ahmed (1984) investigated the effect of increasing lime content on the liquid limit, plastic limit and plasticity index of regional soils of Bangladesh. Ahmed (1984) found an increase in plastic limit while liquid limit and the plasticity index reduced with increasing addition of lime. Hossain (1991), however, found an increase in liquid limit and plastic limit while plasticity index reduced (became nonplastic) with increasing addition of lime for two regional soils ($LL = 25$ and 42 , $I_w = 12$ and 20) of Bangladesh. Rajbongshi (1997) also investigated the effect of increasing lime content on the liquid limit, plastic limit, plasticity index and shrinkage limit of a coastal soil ($LL = 44$, $I_w = 19$) of Bangladesh. Rajbongshi (1997) found an increase in plastic limit and shrinkage limit while liquid limit and the plasticity index reduced with increasing addition of lime, as shown in Fig. 2.25. The linear shrinkage of a clayey soil is also affected by addition of lime. Linear shrinkage reduces as the lime content increases (IRC, 1976). Typical results showing the influence of linear shrinkage are presented in Fig. 2.26. It can be seen from Fig. 2.26 that compared with the silty clay soil, the reduction in linear shrinkage with the increase in lime content in the heavy clay is much higher.

2.13.2 Moisture-Density Relations

The addition of lime to clayey soils increases the optimum moisture content and reduces the maximum dry density for the same compactive effort. This effect is shown in Fig. 2.27. The significance of these changes depends upon the amount of lime added and the amount of

clay minerals present. Flocculation and cementation make the soil more difficult to compact and therefore, the maximum dry density achieved with a particular compactive effort is reduced. As lime treatment flattens the compaction curve, a given percentage of the prescribed density can be achieved over a much wider range of moisture contents so that relaxed moisture control specifications are possible. Due to increase in optimum moisture content, lime stabilisation provides additional advantage when dealing with wet soils. NAASRA (1986), TRB (1987), Hausmann (1990) and Bell (1993) also reported reduction in maximum dry density due to lime stabilisation.

Ahmed (1984), Rajbongshi (1997) and Molla (1997) reported the effect of lime treatment on the maximum dry density and optimum moisture content of regional and coastal soils of Bangladesh. It has been reported by Ahmed (1984) that maximum dry density of two sandy silt and silty clay soils reduced as lime content increased. Rajbongshi (1997) and Molla (1997) reported that increment of lime content increases the optimum moisture content and reduces the maximum dry density. The reduction of maximum dry density with lime content for a coastal soil is shown in Fig. 2.28. Serajuddin and Azmal (1991) also found that compared with untreated sample, the maximum dry density of lime-treated samples of two fine-grained regional soils reduced while optimum moisture content slightly increased.

2.13.3 Unconfined Compressive Strength

The unconfined compressive strength of soil-lime mix increases with increasing lime content. The rate of gain of compressive strength of soil-lime mix in the initial stages (first few days) is considerably less than that for cement stabilised materials. Lime stabilised materials continue to gain strength with time provided curing is sustained.

Ahmed (1984) reported the effect of lime content and curing age on unconfined compression strength for sandy silt and silty clay samples (1.4 in. diameter by 2.8 in. high) treated with various lime contents (0.5% to 5%). A typical result for the silty clay sample is shown in Fig. 2.29 which shows that unconfined compressive strength increases with the increase in lime content and curing age. Serajuddin and Azmal (1991) and Serajuddin (1992) also reported the effect of lime content and curing age on unconfined compressive strength of samples (50 mm diameter and 100 mm high) of regional alluvial soils of Bangladesh. Samples were treated with 5%, 7.5% and 10% slaked lime. Typical results showed that unconfined compressive strength of lime-treated samples increase with the increase in curing age and lime content. Hossain (1991) also found an increase in unconfined compressive strength with the increase in lime content and curing age lime for two regional soils of Bangladesh.

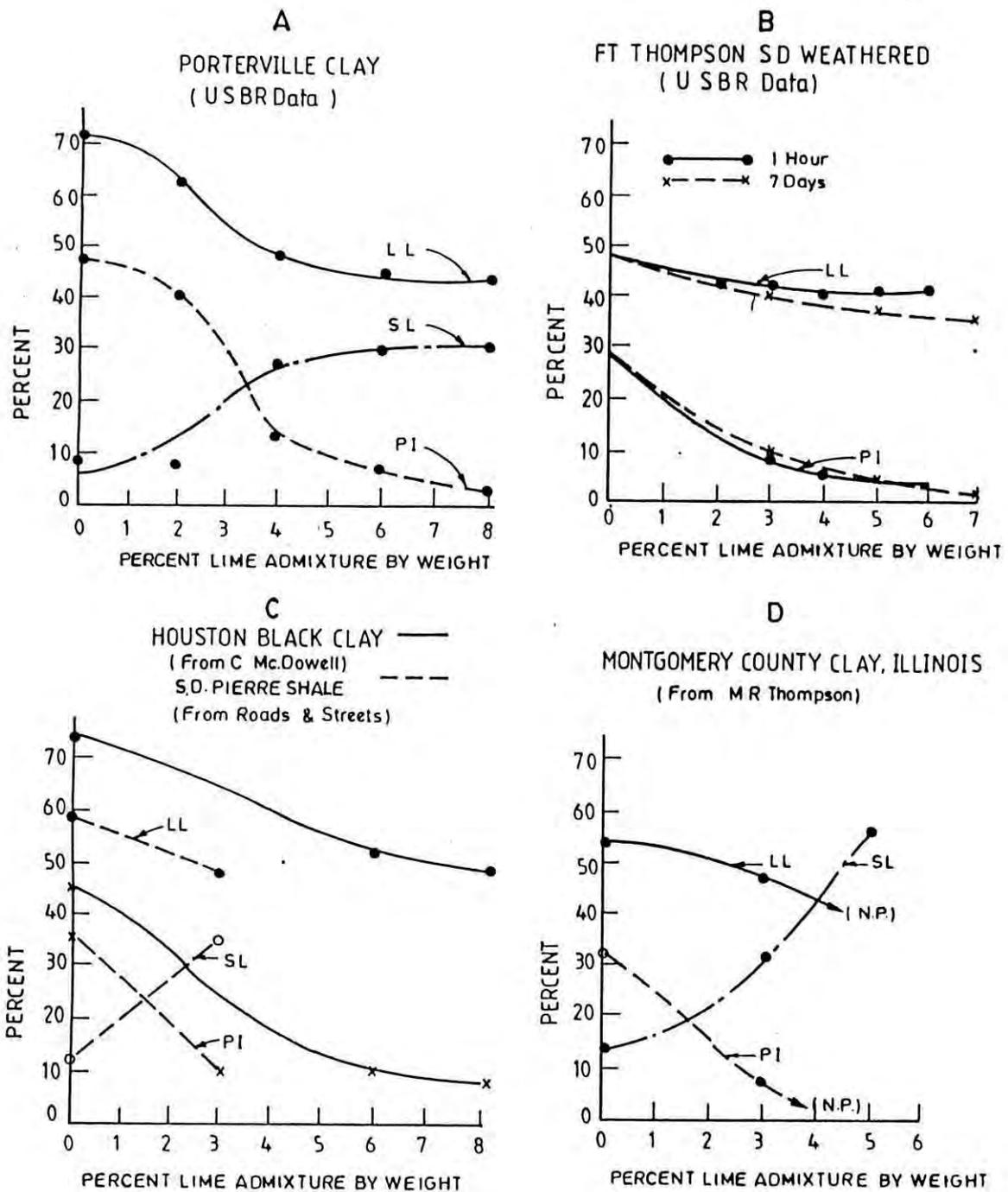


Fig. 2.24 Effect of lime on plastic characteristics of expansive montmorillonite clays (after Holtz, 1969)

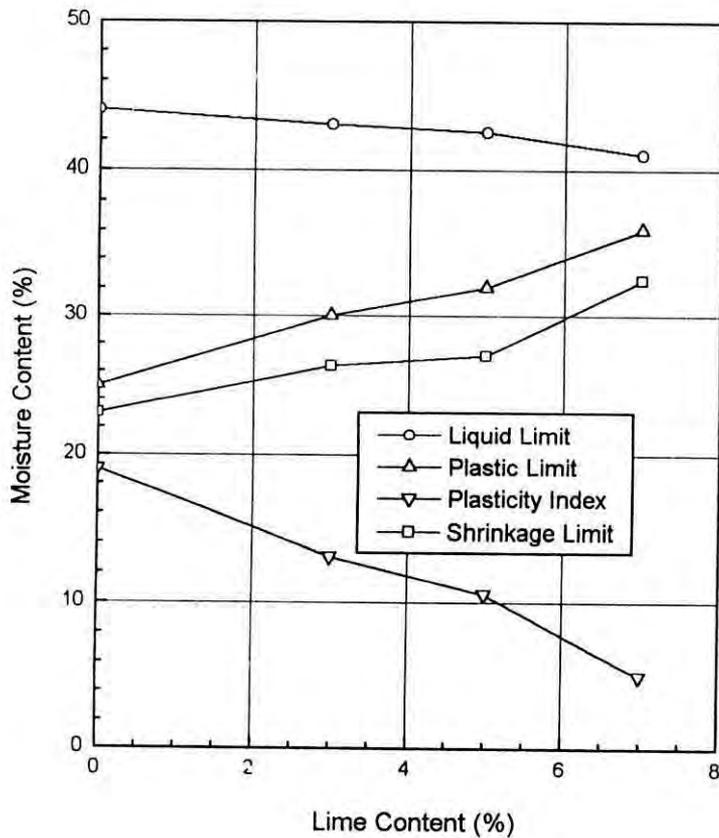


Fig. 2.25 Effect of lime content on Atterberg limits and shrinkage limit of a coastal soil (after Rajbongshi, 1997)

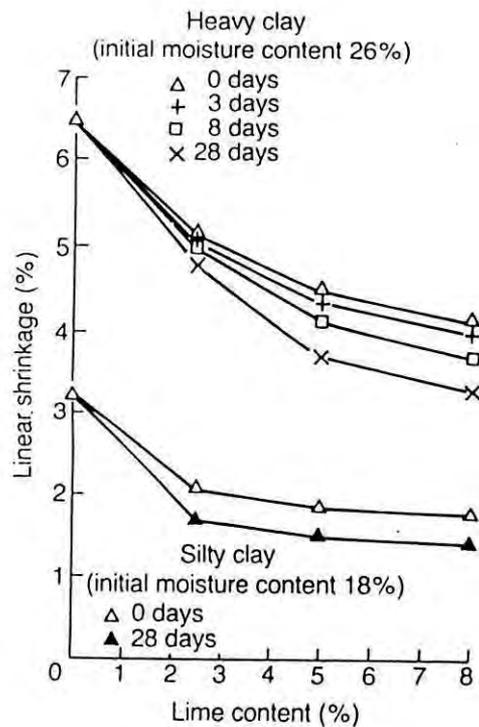


Fig. 2.26 Effect of lime content on linear shrinkage of clays (after Bell, 1988)

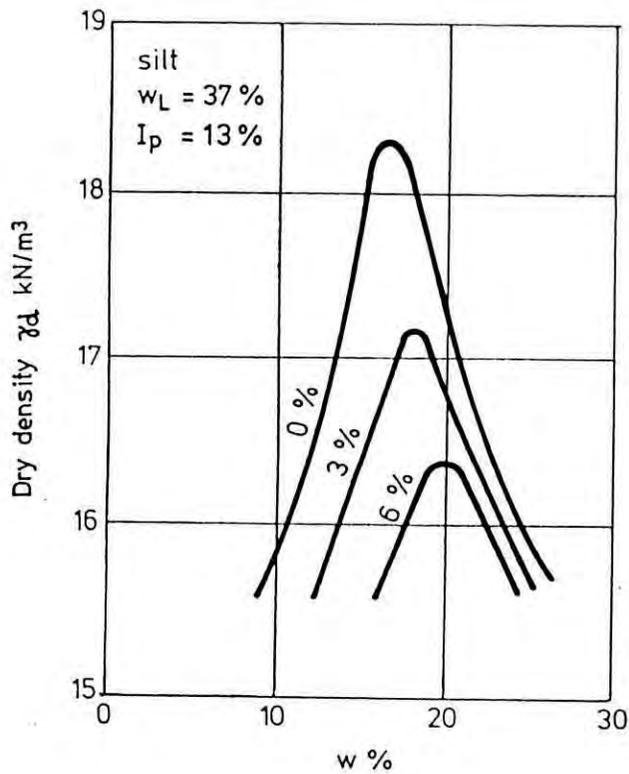


Fig. 2.27 Effect of lime content on maximum dry density and optimum moisture content of a lime-treated silt (after Kezdi, 1979)

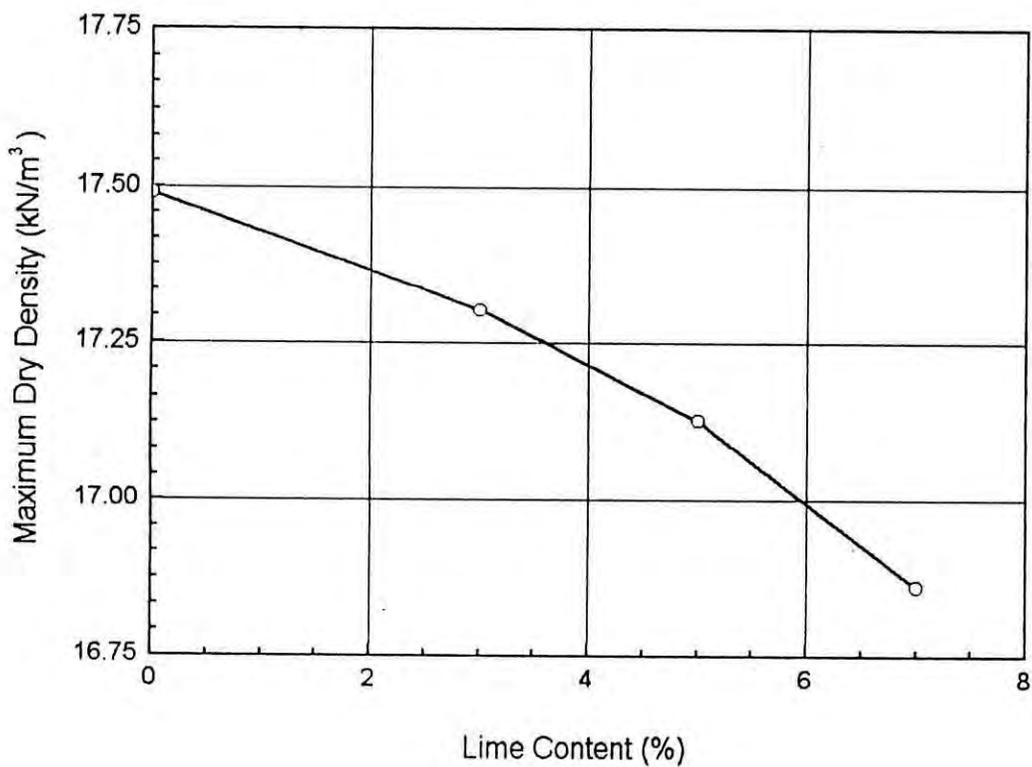


Fig. 2.28 Effect of lime content on maximum dry density of a lime-treated coastal soil (after Rajbongshi, 1997)

Rajbongshi (1997) also investigated the effect of lime content and curing age on unconfined compressive strength of large diameter samples (2.8 in. diameter by 5.6 in. high) of a coastal soil. Rajbongshi (1997) reported that unconfined compressive strength of lime-treated samples increase with the increase in lime content and curing age as shown in Fig. 2.30. Molla (1997) also found that unconfined compressive strength of lime-treated samples increased with the increase in lime content and curing age for three regional soils of Bangladesh.

Rajbongshi (1997) investigated the rate of strength gain with curing time in terms of the parameter termed as strength development index (SDI) as proposed by Uddin (1995). SDI is defined by the following expression (Uddin, 1995):

$$SDI = \frac{\text{Strength of stabilised sample} - \text{Strength of untreated sample}}{\text{Strength of untreated sample}} \quad (2.16)$$

Plottings of SDI with curing age of samples of a lime treated coastal soil is shown in Fig. 2.31. Fig. 2.31 shows that the values of SDI increases with increasing curing time and lime content as well. Fig. 2.31 clearly shows the relative degree of strength gain resulted due to increasing lime content and curing age. As can be seen from Fig. 2.31 that the strength gain for samples treated with 7% lime are relatively much higher than those of samples treated with 3% and 5% lime.

Rajbongshi (1997) and Molla (1997) investigated the effect of moulding moisture content on unconfined compressive strength of lime-treated samples. Unconfined compressive strength of samples was found to increase with increasing moulding moisture content as shown in Fig. 2.32. Rajbongshi (1997) reported that at a particular curing age the values of unconfined compressive strength of samples compacted at wet side are higher than the values of unconfined compressive strength of samples compacted at optimum or dry side of optimum moisture content as shown in Fig. 2.33. The values of unconfined compressive strength of samples compacted at dry side of optimum moisture content has been found to the least.

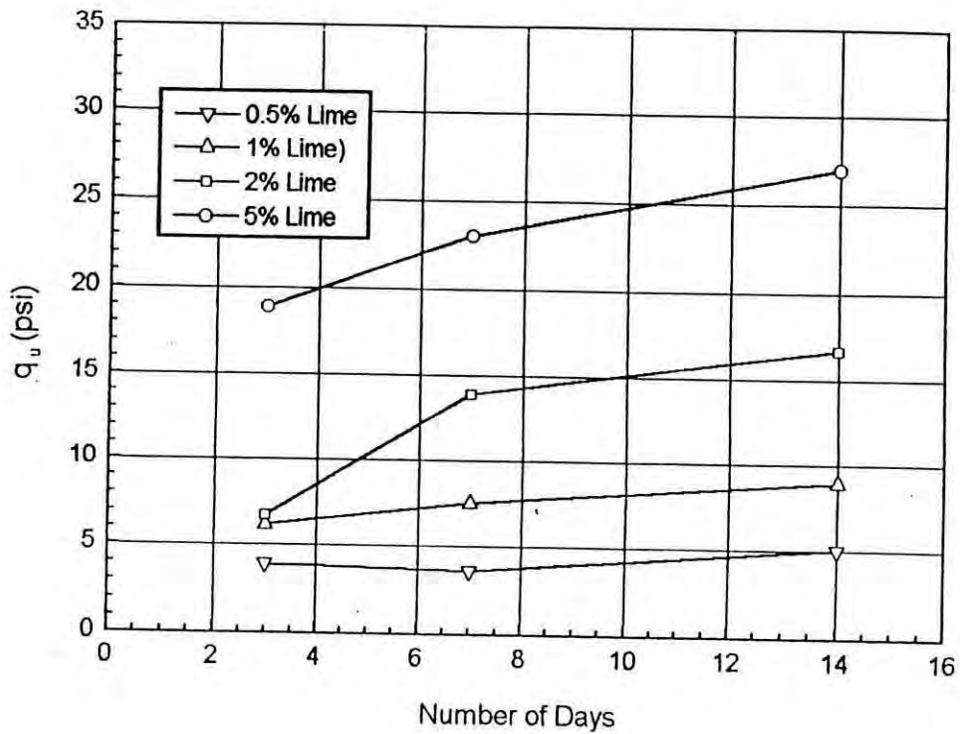


Fig. 2.29 Effect of curing age on unconfined compressive strength (q_u) of a silty clay at different lime contents (reproduced after Ahmed, 1984)

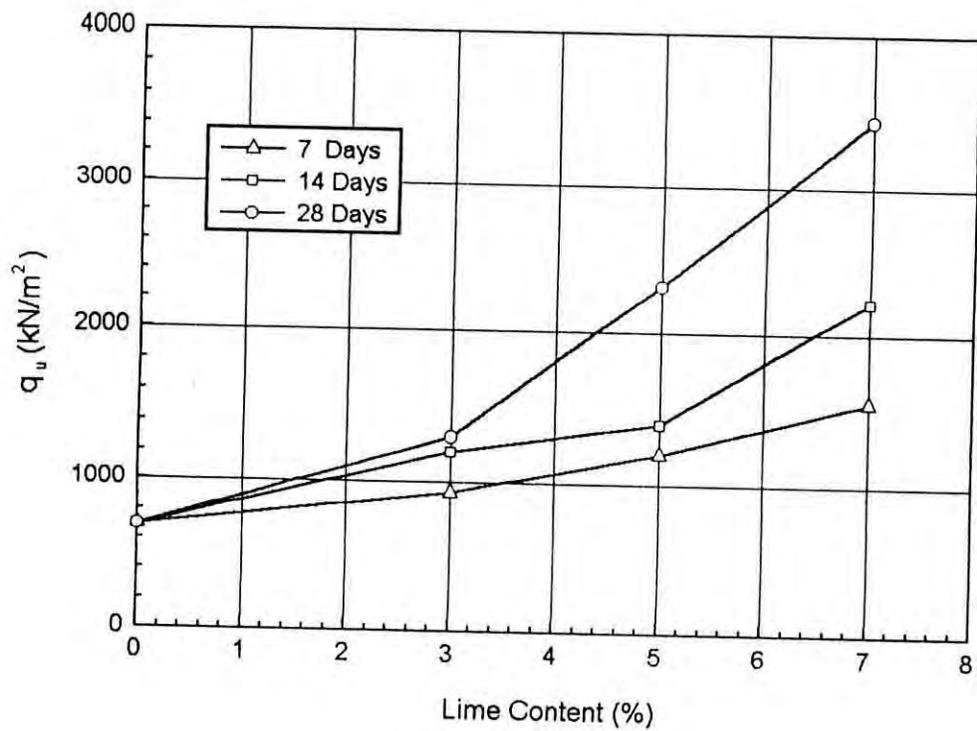


Fig. 2.30 Effect of lime content on unconfined compressive strength (q_u) of a coastal soil at different curing age (after Rajbongshi, 1997)

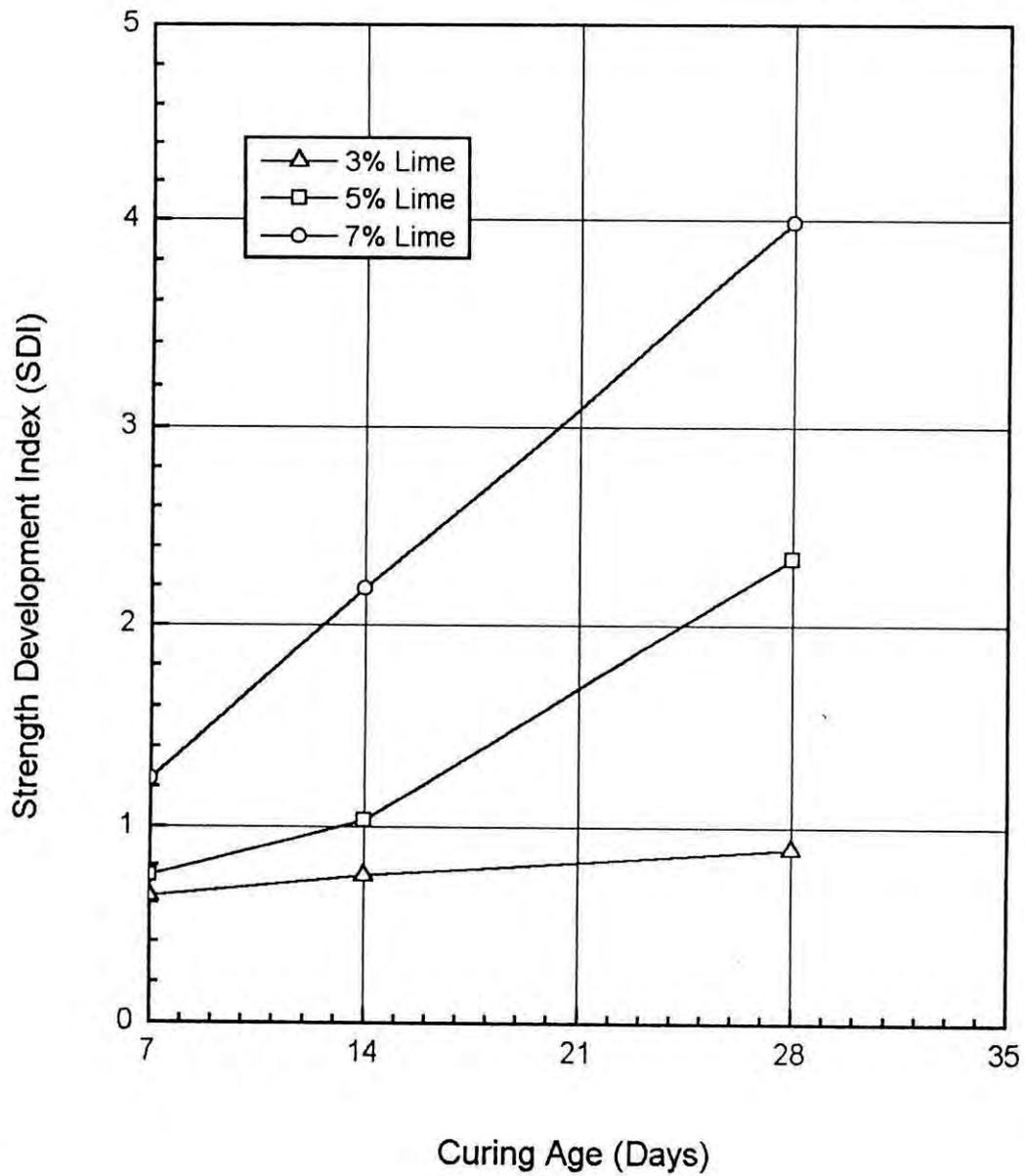


Fig. 2.31 SDI versus curing age curves for samples of a lime-treated coastal soil (after Rajbongshi, 1997)

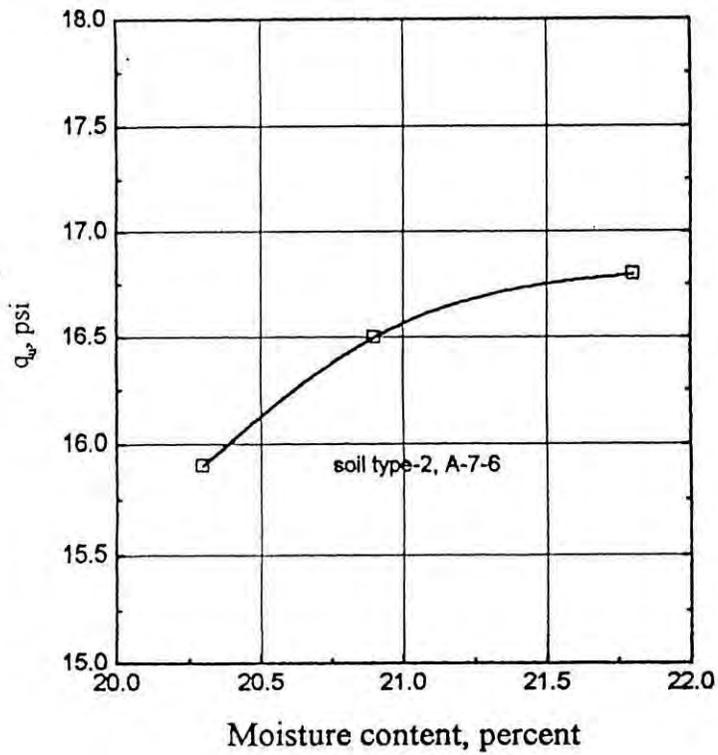


Fig. 2.32 Variation of unconfined compressive strength (q_u) with moulding moisture content for a lime-treated silty clay soil (after Molla, 1997)

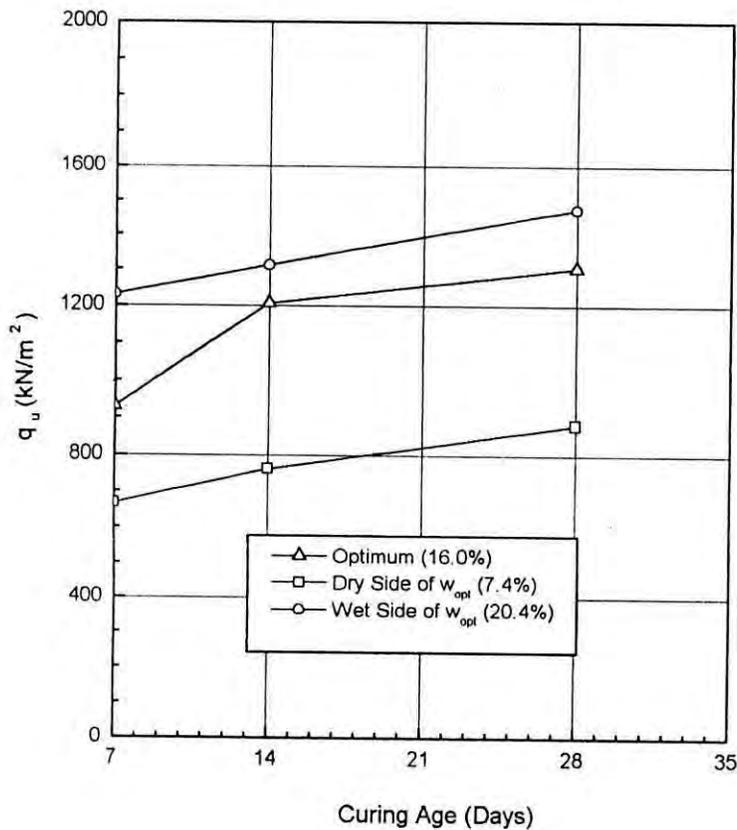


Fig. 2.33 Variation of q_u with curing age for a coastal soil treated with 3% lime and compacted at different moulding water contents (after Rajbongshi, 1997)

2.13.4 California Bearing Ratio (CBR)

The CBR test has been extensively used to evaluate the strength of lime stabilised soils. TRB (1987) reported the immediate effect of lime treatment on CBR-values for three plastic clays ($LL = 35$ to 59 , $PI = 15$ to 30). It has been found that for all the soils CBR increase markedly with increasing lime content.

Hossain (1991) investigated the effect of lime on CBR-values of two subgrade soils of Bangladesh stabilised with 2%, 4%, 6%, 8% and 10% lime. Hossain (1991) found that CBR-value increased due to increase in lime content. Molla (1997) and Rajbongshi (1997) also investigated the effect of lime on CBR-values of three regional and soils and a coastal soil of Bangladesh, respectively. The variation of CBR value due to increase in lime content is shown in Fig. 2.34 for three soils of different plasticity. From Fig. 2.34, it can be seen that CBR value of stabilised samples increases with increasing lime content. Rajbongshi (1997) performed CBR tests on samples of a coastal soil compacted according to Modified Compaction test using three levels of compaction energies, e.g., low compaction ($10,000 \text{ ft-lb/ft}^3$), medium compaction ($25,000 \text{ ft-lb/ft}^3$) and high compaction ($56,000 \text{ ft-lb/ft}^3$). The variation of CBR with lime content for samples of the coastal soil is shown in Fig. 2.35 while Fig. 2.36 presents the CBR-dry density relationships for the same samples. It can be seen from Fig. 2.35 that at all levels of compaction, CBR increases markedly with increasing lime content while Fig. 2.36 shows that at any particular lime content, CBR increases significantly with the increase in dry density.

2.13.5 Tension and Flexural Properties

Tensile strength properties of soil-lime mixtures are of concern in pavement design because of the slab action that is afforded by a material possessing substantial tensile strength (TRB, 1987). The flexural strength of soil-lime mixtures is important to use in sub-base and base courses. Two test methods, indirect tensile and flexure, have been used for evaluating the tensile strength of soil-lime mixtures. The indirect tensile test is essentially a diametral compression test in which the material fails in tension along the loaded diameter of the cylindrical test specimen.

Typical results indicate that the mixtures can possess substantial tensile strength (TRB, 1987). The ratio of indirect tensile strength to unconfined compressive strength in one study (Thompson, 1966b) was found to be approximately 0.13, while in another study (Tulloch et al., 1970), it was found to be much lower as indicated by the following regression equation:

$$S_T = 6.89 + 50.6 q_u \quad (2.17)$$

where S_T is the tensile strength in pounds per square inch and q_u is the unconfined compressive strength in kips per square inch.

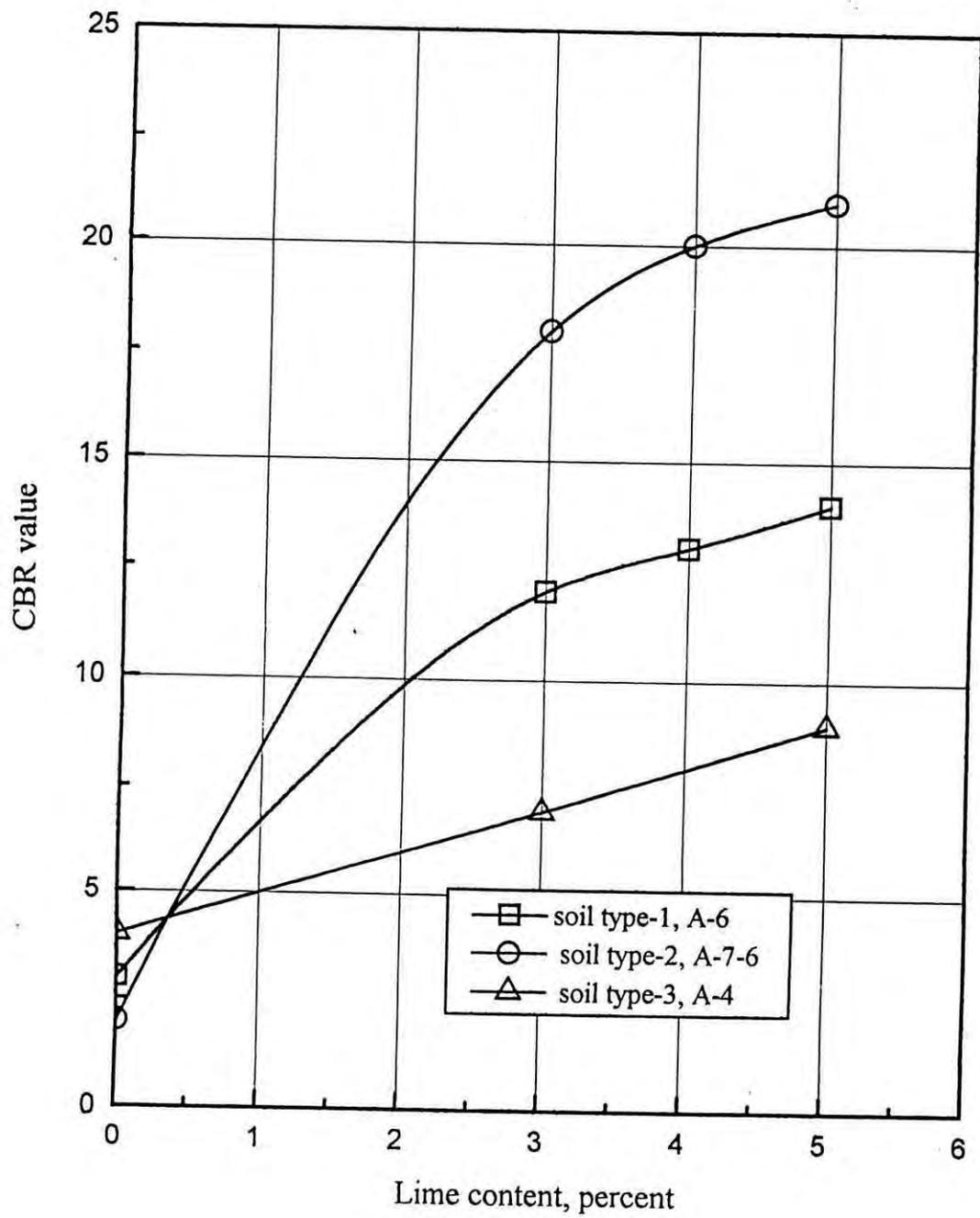


Fig. 2.34 Variation of CBR value with lime content for three regional soils (after Molla, 1997)

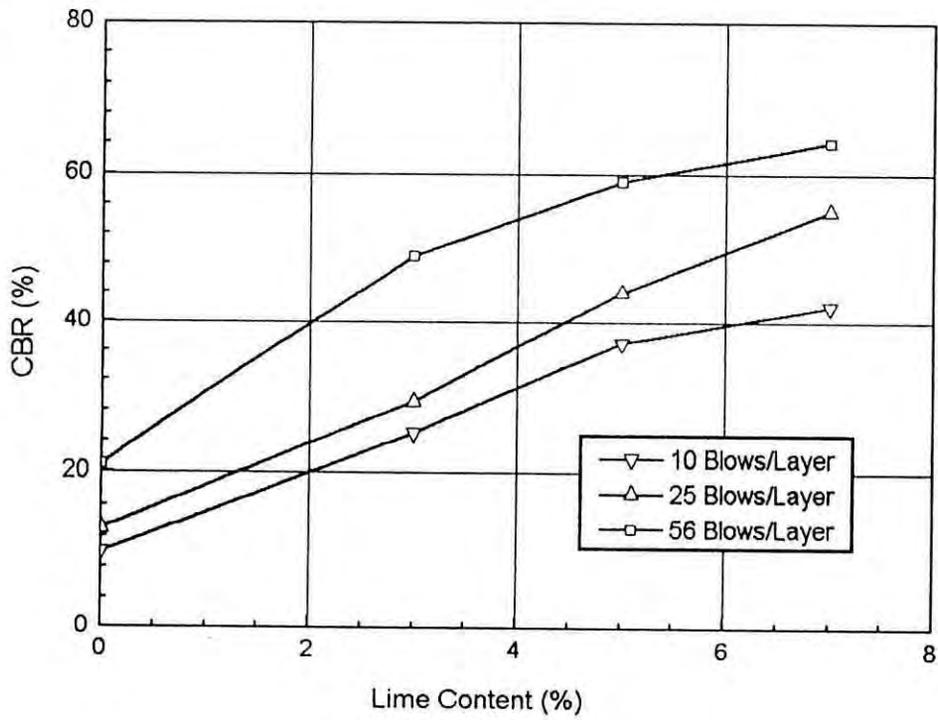


Fig. 2.35 Effect of lime content on CBR values of a coastal soil (after Rajbongshi, 1997)

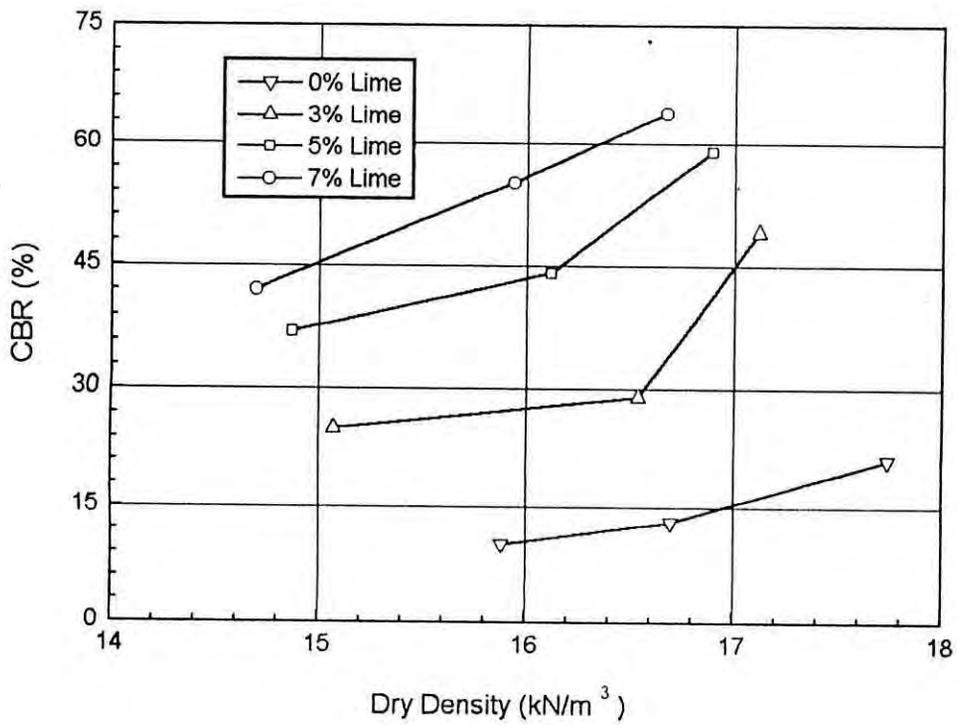


Fig. 2.36 CBR versus dry density curves of a lime-treated coastal soil (after Rajbongshi, 1997)

The most common method used for evaluating the tensile strengths of highway materials has been the flexural test. It has been found that the ratio of flexural strength to indirect tensile strength is approximately 2 (Thompson, 1969). Soil-lime mixtures continue to gain strength with time, and the ultimate strength of the mixture is a function of curing period and temperature. The magnitudes of the stress repetitions applied to the mixture are relatively constant throughout its design life. Therefore, as the ultimate strength of the material increases due to curing the stress level, as a per cent of ultimate strength, will decrease and the fatigue life of the mixture will increase.

The flexural properties of untreated and stabilised samples of a coastal soil have been investigated by Rajbongshi (1997). It has been found that compared with the untreated sample, flexural strength and modulus of the treated samples cured at 7 and 28 days increased significantly. Compared with the untreated sample, the flexural strength and modulus of samples treated with 7% lime and cured at 28 days are respectively about 2 times and 2.25 times higher than those of the untreated samples. The effect of lime content on flexural strength is shown in Fig. 2.37 while Fig. 2.38 presents the effect of lime content on flexural modulus. Figs. 2.37 and 2.38 show that flexural strength and modulus increases with increasing lime content. It is evident from Figs. 2.37 and 2.38 that curing age has got insignificant effect on increase in flexural strength and modulus.

2.13.6 Permeability

Townsend and Klyn (1970) stated that the permeability of the soil increase due to the addition of lime to the soil. While conducting the experiment with heavy clay, Townsend and Klyn (1970) observed a marked increase in permeability but for silty clay soil, erratic or no change of permeability was observed.

Broms and Boman (1977) and Brandl (1981) stated that the addition of lime usually increases the permeability of soft clay. The increase in permeability is associated with flocculation, where larger pore between the flocks enable the fluid to flow more readily in between the clay and corresponding change in grain size distribution.

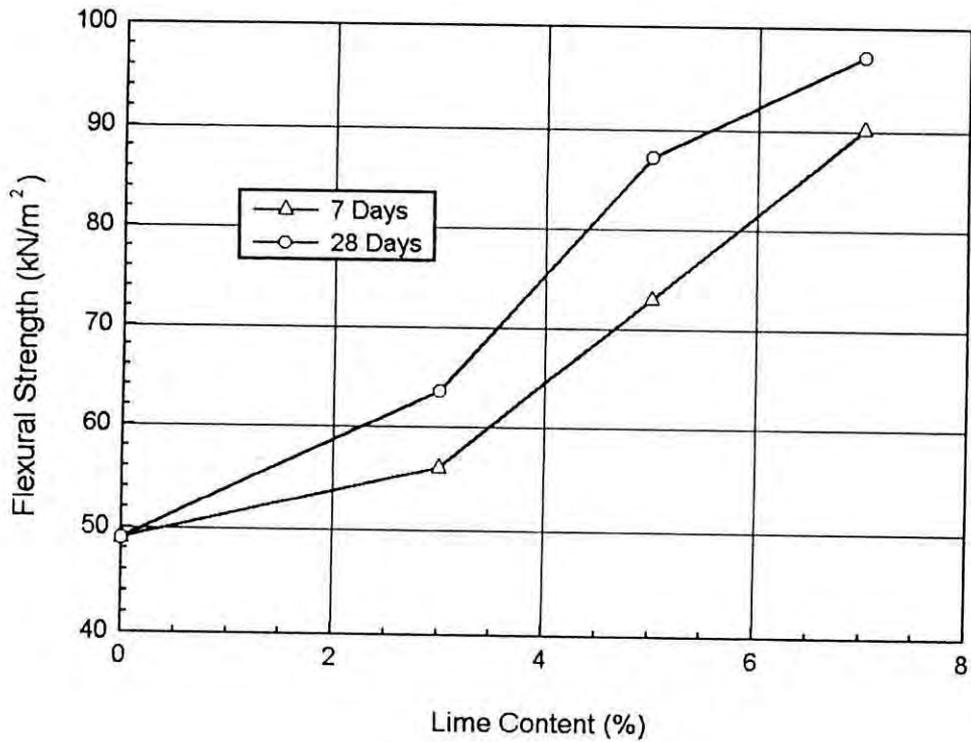


Fig. 2.37 Effect of lime content on flexural strength of a coastal soil (after Rajbongshi, 1997)

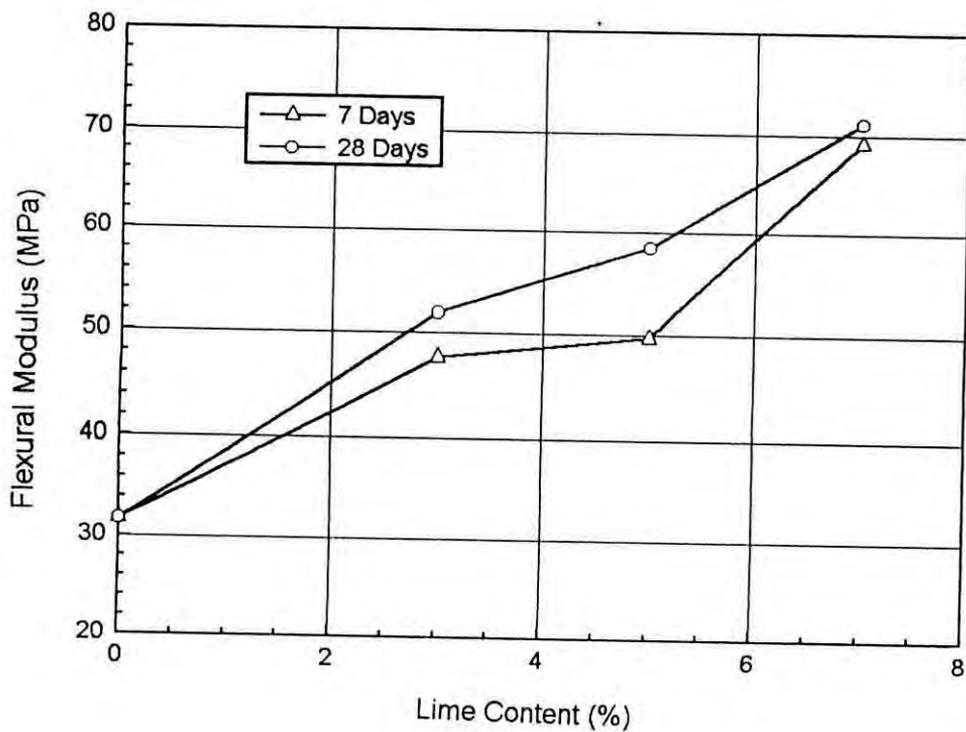


Fig. 2.38 Effect of lime content on flexural modulus of a coastal soil (after Rajbongshi, 1997)

2.14 APPLICATIONS OF LIME STABILISATION

The principal use of the addition of lime to soil is for subgrade and sub-base stabilisation and as a construction expedient on wet sites where lime is used to dry out the soil. As far as lime stabilisation for roadways is concerned, stabilisation is brought about by the addition of between 3 and 6% lime (by dry weight of soil). When lime stabilisation has been used to upgrade heavy clay soils to sub-base material quality or to upgrade plastic gravels to base course quality, an unconfined compressive strength of 250 psi at seven days, and a CBR of at least 80 are required, although values of unconfined compressive strength of 150 psi to 450 psi at seven days are also proposed (Ingles and Metcalf, 1972).

Lime is effective in modifying excessive plastic properties of sub-base and base course materials. Those that have plasticity indices and/or fines contents above the normally accepted level for the desired usage can usually be modified with lime. Such modification of base courses is a widely accepted and successful practice. At low lime contents (less than 2 to 3 percent) the risk of undesirable shrinkage cracking is low, and it would rarely be necessary to take special measures to combat reflective cracking. Lime is usually used to modify rather than bind soils. While high tensile strengths can easily be obtained with appropriate materials, careful control has to be exercised over the field construction techniques, particularly adequate moisture, early rolling and effective curing, for the assured production of a bound material (NAASRA, 1986).

Lime has no application in cohesionless sands and gravels regardless of particle size distribution. Fine and clayey gravels, clayey sands and silty sands may remain excessively friable and unsuitable for base course usage when stabilised with lime. The range of materials for subgrade, sub-base and base course that can be treated with lime or cement are fairly similar. Lime stabilisation is used in embankment construction for roads, railways, earth dams and levees to enhance the shear strength of the soil. In retaining structures it is used primarily to increase the resistance to water, either external or internal. For example, lime has been used to stabilise small earth dams constructed of dispersive soil and so avoid piping failure. Lime has also been used to stabilise low-angled slopes, a surface layer of soil about 150 mm thick being mixed in place.

Lime stabilisation of clay soils, especially expansive clay soils, can minimise the amount of shrinkage and swelling they undergo. Hence, such treatment can be used to reduce the number and size of cracks developed by buildings founded on suspect clay soils. Lime stabilisation may be applied immediately beneath strip footings for light structures. The treatment can be better applied as a layer below a raft in order to overcome differential movement.

CHAPTER 3

LABORATORY INVESTIGATIONS

3.1 GENERAL

Details of laboratory testing procedures and equipment used for determining the swelling properties of expansive soil have been presented in this chapter. A number of instruments and equipment have been used to investigate the effects of lime stabilisation on the physical and engineering characteristics of the expansive soil studied. This chapter describes the principal equipment and instruments used for this research. The various laboratory test procedures for conducting tests on untreated and lime stabilised samples are also presented in this chapter.

3.2 SAMPLING AND COLLECTION OF SOIL SAMPLES

Disturbed soils from six different sites, namely POL store, Air condition store office, AMO store, Main and RD office, COD store and Married ORS quarter area of Rajendrapur Cantonment, Gazipur. Soil sampling was carried out according to the procedure outlined in ASTM D420 (ASTM, 1989). For each location, approximately 2m by 2m area was excavated to a depth 2m to 3m using hand shovels. Proper care was taken to remove any loose material, debris coarse aggregates and vegetation from the bottom of the excavated pit. Disturbed samples were collected from the bottom of the borrow pit through excavation by hand shovels. All disturbed samples were packed in large polythene bags covered by gunny bags. Undisturbed samples were also collected from the bottom of the borrow pit by penetrating 3 inch diameter Shelby tubes. All disturbed samples and undisturbed tube samples were eventually transported to the geotechnical Engineering laboratory of Bangladesh University of Engineering and Technology, Dhaka. The soil samples were designated as follows :

- Soil-A : Collected from POL store, area
- Soil-B : Collected from Air condition store office area
- Soil-C : Collected from AMO Store area
- Soil-D : Collected from Main & RD office area
- Soil-E : Collected from COD store area
- Soil-F : Collected from Married ORS quarter area

3.3 LABORATORY TESTING PROGRAMME

Initially, a comprehensive laboratory investigation programme was undertaken in order to examine the physical, index and swelling properties of the six samples collected from different sites of Rajendrapur Cantonment, Gazipur in order to select an appropriate expansive soil. The following tests were conducted to determine the index and physical properties of the six samples:

- (i) Specific gravity test
- (ii) Liquid limit test
- (iii) Plastic limit and plasticity index test
- (iv) Shrinkage limit test
- (v) Linear shrinkage test
- (vi) Grain size distribution test

Swelling tests included determination of the following swelling properties:

- (i) Free swell
- (ii) Free swell index
- (iii) Swelling pressure
- (iv) Swelling potential
- (v) Volume change from air-dry to saturated condition

On the basis of index and swelling properties, a suitable expansive soil to be treated with lime stabilisation was selected. Details of laboratory tests were carried out on the selected expansive soil with different lime contents are presented in Table 3.1. The following testing programme was undertaken on the selected expansive soil:

- (i) Index properties (e.g., liquid limit, plastic limit, shrinkage limit, linear shrinkage, volumetric shrinkage) of the soil treated with five different lime contents of 3%, 6%, 9%, 12% and 15% were determined.
- (ii) Swelling properties (e.g., free swell, free swell index, swelling pressure, swelling potential, volume change from air-dry to saturated condition) of the soil treated with five different lime contents of 3%, 6%, 9%, 12% and 15% were determined.
- (iii) Following tests were carried out on the selected expansive soil without any treatment and also treated with five different lime contents (3%, 6%, 9%, 12% and

15%):

- Standard compaction test
- Unconfined compressive strength test on moulded cylindrical samples of 2.8 inch diameter by 5.6 inch high
- California Bearing Ratio (CBR) test
- Flexural strength test using simple beam with third point loading system

Compressive strength tests were carried out on lime treated samples cured at different ages of 1 week, 2 weeks, 4 weeks, 8 weeks and 16 weeks while flexural strength tests using simple beam with third point loading were performed on lime treated samples cured at ages of 1 week, 2 weeks, 4 weeks and 8 weeks, in order to investigate the effect of curing age on the measured compressive strength and flexural strength and stiffness.

In order to investigate CBR - Dry density relationships for the untreated and treated soils, laboratory CBR tests were carried out on the untreated samples and samples treated with various lime contents using three levels of compaction energies.

(iv) In order to investigate the effect of moulding water content on the compressive strength, unconfined compression strength tests were also carried out on 2.8 inch diameter by 5.6 inch high stabilised samples of the soil treated with 3%, 6%, 9%, 12% and 15% lime which were compacted according to the Standard Compaction test with two moulding water contents. The following water contents were used for compaction:

- (a) water content corresponding to 95% of maximum dry density on dry side of the optimum moisture content.
- (b) water content corresponding to 95% of maximum dry density on wet side of the optimum moisture content.

Unconfined compressive strength tests were carried out on lime treated samples cured at different ages of 1 week, 2 weeks, 4 weeks, 8 weeks and 16 weeks.

3.4 PROCEDURES FOR DETERMINING PHYSICAL AND INDEX PROPERTIES OF UNTREATED SOILS

The samples collected from the six different locations of Rajendrapur Cantonment, Gazipur were disturbed samples. These samples were then air-dried and the soil lumps were broken carefully with a wooden hammer so as to avoid breakage of soil particle. The required quantity of soil was then sieved through Sieve No. 40 (0.425 mm). The standard test procedures followed in determining the physical and index properties of the untreated soils are shown in Table 3.2

Table 3.1 Details of laboratory tests performed on the expansive soil with different lime contents

Type of Test	Sample	No. of Test
Liquid limit and plastic limit	Untreated Soil	1
	Soil-Lime mixture	5
Shrinkage limit linear shrinkage	Untreated Soil	1
	Soil-Lime mixture	5
Free swell and free swell Index	Untreated	1
	Soil-Lime mixture	5
Swelling potential	Untreated	1
	Soil-Lime mixture	5
Swelling pressure	Untreated Disturbed	1
	Untreated Undisturbed	1
	Soil-Lime mixture	5
Volume change from air-dry to saturated condition	Untreated undisturbed	5
	Untreated Disturbed	5
Standard Compaction test	Untreated	1
	Soil-Lime mixture	5
Unconfined Compression test	Untreated	1
	Soil-Lime mixture	35
CBR Test at three levels of compaction	Untreated	1
	Soil-Lime mixture	5
Flexural strength test using simple Beam with third point loading system	Untreated	1
	Soil-Lime mixture	20

Table 3.2 Test procedures followed for determining the physical and index properties of the untreated soils

Name of Test	Test Procedure Followed
Specific Gravity	ASTM D854
Liquid Limit	BS 1377
Plastic limit and plasticity Index	ASTM D4318
Shrinkage Limit	ASTM D427
Linear Shrinkage	BS 1377
% of material in soils finer than No. 200 sieve	ASTM D1140
Grain size distribution	ASTM D422

3.5 INDEX PROPERTY TESTS ON STABILISED SOIL SAMPLES

Liquid limit, plastic limit, plasticity index and shrinkage characteristics (shrinkage limit, linear shrinkage and volumetric shrinkage) of the selected expansive soil stabilised with lime were determined. Hydrated lime (i.e., slaked lime) in percentages of 3, 6, 9, 12 and 15 was used as additive..

Liquid limit test and plastic limit test on the stabilised samples were carried out on air-dried pulverised samples. The required quantity of pulverised soil was sieved through sieve No. 40 (0.425 mm). The lime treated soils were compacted following ASTM D558 method. The compacted samples were cured in moist environment for 7 days and air-dried. The air-dried samples were pulverised to pass through No. 40 sieve. Liquid limit test and plastic limit tests of the stabilised samples were determined following the standard procedure outlined in BS 1377 (BS, 1975) and ASTM D424 (ASTM, 1989), respectively. The shrinkage factors were determined in accordance with the procedure specified in ASTM D427 (ASTM, 1989). Linear shrinkage of the lime treated samples were determined following the procedure outlined in BS 1377 (BS, 1975).

3.6 SWELLING TESTS ON UNTREATED AND TREATED SOIL SAMPLES

3.6.1 Free Swell

Free swell tests on treated and untreated samples were carried out according to the procedure suggested by Holtz and Gibbs (1956). The test is performed by pouring slowly 10 cc of dry soil, passing through 425 micron sieve (No. 40 sieve), into a 100 cc

graduated cylinder filled with water. The volume of the settled and swelled soil is read after 24 hours, from the graduations of the cylinder. The difference between the final and the initial value (10 cc) expressed as a percentage of initial volume (10 cc) gives the free swell value.

3.6.2 Differential Free Swell

Differential free swell test was also carried out on treated and untreated samples following the procedure outlined in Indian Standard (IS: 2720, 1977). In this test, two samples of oven-dried soil weighting 10 g each and passing through 425 micron sieve were taken. One was put in a 100 cc graduated glass cylinder containing kerosene oil (a non-polar liquid) while the other sample was put in a similar cylinder containing distilled water. Both the samples were left undisturbed for 24 hours and then their volumes were noted. The free swell index is calculated using the following expression:

$$\text{Free Swell Index (\%)} = \frac{V_d - V_k}{V_k} \times 100 \quad (3.1)$$

where, V_d = the column of soil specimen read from the graduated cylinder containing distilled water

V_k = the volume of soil specimen read from the graduated cylinder containing kerosine

3.6.3 One-Dimensional Swelling Pressure Test

Swelling pressure of treated and untreated samples of soil were determined following the procedure outlined in ASTM D4546 (ASTM, 1989). The undisturbed specimens (2.5 inch diameter by 7/8 inch thick) were prepared from tube specimens by hand trimming and pushing 2.5 inch diameter by 7/8 inch high consolidometer ring into the tube specimens. The remoulded specimens were prepared by compacting soil in Standard Proctor Compaction mould (4 inch diameter by 4.584 inch high) following the procedure specified in ASTM D698 (ASTM, 1989). Specimens of 2.5 inch diameter by 7/8 inch high were then prepared by hand trimming and penetrating consolidometer ring into the laboratory compacted specimens. The water contents of remoulded specimens were equal to the respective optimum moisture content as obtained from Standard Compaction test. Each specimen was inundated and allowed to swell vertically at a seating pressure

of 1 kPa for 72 hours. Each specimen was then loaded incrementally until its initial volume is reached.

Swelling pressures of the undisturbed and compacted specimens were estimated from the volume change versus surcharge pressure curves.

3.6.4 Swelling Potential Test

Seed et al. (1962) defined swelling potential as the percentage of swell of a laterally confined sample on soaking under 1 psi (6.89 kPa) surcharge, after being compacted to maximum dry density at optimum moisture content according to Standard Compaction test method (ASTM D698). A total of four tests, two tests for each site, were carried out.

For this test, firstly, the maximum dry density and optimum moisture content of the soil was determined according to the procedure outlined in ASTM D698 (ASTM, 1989). Then, an amount of water equal to the optimum moisture content was added to about 6 lbs. of oven dried sample passing No. 40 sieve. It was then compacted in the same mould with the same energy for which the optimum moisture content was determined. After compaction, the compacted soil was extruded from the mould by a manual jack. Consolidometer ring of 2.5 inch diameter and 1 inch height is then pushed into the solid cylinder of the compacted soil. The wall of the ring was adequately greased so that friction between the soil and wall is negligible or totally eliminated.

Excess soil on two ends of the ring was trimmed with knife and piano wire to obtain a plane surface for loading. The specimen was then placed in the consolidation cell with two air-dry porous stones, one at the top and the other at the bottom and 1 psi vertical load was applied on the soil specimen and initial dial reading was noted. The specimen was then inundated and the change in thickness of the specimen was noted at increasing intervals of time. Readings at $\frac{1}{4}$, $\frac{1}{2}$, 1, 2, 4, 8, 15 and 30 minutes and 1, 2, 4, 8, 24, 48, 72 and 96 hours. Between 72 to 96 hours, usually, an equilibrium condition was reached when no further deflection of the dial was observed. This final dial reading minus the initial dial reading provided the total volume change of the specimen under 1 psi load when saturated from optimum moisture content. This change in volume expressed as a percentage of the original volume provided the swelling potential of the soil specimen.

3.6.5 Volume Change Test

The procedure developed by Holtz and Gibbs (1956) and adopted by the USBR (United States Bureau of Reclamation) was followed for carrying out test to determine volume change of undisturbed and compacted samples of treated and untreated soil.

Two identical specimens of 2.5 inch in diameter and 7/8 inch high were prepared from undisturbed sample. Each specimen was then placed in a consolidometer ring of 2.5 inch in diameter and 1 inch high. After determining the initial volume and moisture content of the specimen, they were allowed to dry in air to at least the shrinkage limit. The volume change at the air-dry condition was determined on the first specimen by mercury displacement method. The second specimen was placed in the consolidometer with two porous stones at the top and bottom of the specimen. A vertical loading of 1 psi was applied on the soil specimen and initial dial reading was noted. The specimen was then saturated and at the same time vertical movement of the dial was observed. Between 72 and 96 hours, an equilibrium condition was reached when there was no vertical movement of the dial. This final dial reading minus the initial reading gave the volume change of the specimen under saturation and 1 psi load. From this test, the volume change and vertical movements from initial to air-dry moisture conditions and from initial to saturated moisture conditions were determined. The combined results for both specimens gave the total volume change from air-dry to saturated moisture conditions of a specimen.

The specimens for volume change test for the lime treated samples were prepared by compaction of soil-lime mix in the consolidometer to a density and moisture content approximately equal to those of the respective undisturbed specimen. The volume change of the specimens was determined in a manner similar to that followed for undisturbed sample as described above.

3.7 COMPACTION TEST

The moisture content versus dry density relationship of the treated and untreated expansive soil was determined by carrying out Standard Compaction test. These tests were performed according to the standard procedure outlined by ASTM D698 (ASTM, 1989). Air-dried

samples passing through No. 4 sieve was used for compaction. For compaction of the moist samples, a cylindrical mould of 4 inch (102 mm) inside diameter and 4.584 inch height was used. The volume of the mould was 0.033 ft³. A series of moist samples of varying moisture contents were compacted in three layers of approximately equal height. Each layer was compacted by 25 blows from a hammer of weight 5.5 lb (2.49 kg) and falling from a free height of 12 inch (305 mm). The amount of material used was such that the third compacted layer was slightly above the top of the mould but not exceeding 6 mm. During compaction the mould was placed on a uniform rigid foundation. Finally, moisture content and dry density determinations were made on each compacted sample.

For lime treated samples of the selected expansive soil, samples for moulding specimens were prepared according to the procedure outlined in ASTM D558 (ASTM, 1989). A series of soil-lime samples of varying moisture contents were prepared. These samples were subsequently compacted in a cylindrical mould of 4 inch (102 mm) inside diameter, 4.584 inch height and of volume 0.033 ft³ in accordance with the above procedure as outlined in ASTM D698 (ASTM, 1989). The different lime contents used for preparing samples were 3%, 6%, 9%, 12% and 15%. Finally, moisture content and dry density determinations were made on each of the compacted stabilised sample.

3.8 UNCONFINED COMPRESSIVE STRENGTH TEST

3.8.1 Preparation and Mixing of Soils

The selected untreated expansive soil was first air-dried. Then the soil aggregates were broken carefully with a wooden hammer in order to avoid reducing the natural size of the individual particles. The required quantity of pulverised soil was then sieved through sieve No. 4 (4.76 mm). Soil retained on this sieve was discarded. Representative soil sample was used to prepare test specimen of desired density, i.e., the maximum dry density corresponding to Standard Compaction. Moisture content of air-dry soil sample was determined. Lime was used in percentages of 3, 6, 9, 12 and 15. The percentages of the additives were calculated on the basis of air-dry weight of the soil samples. Soils were mixed with lime in a laboratory mixer in several batches. This mixing was carried out in a steel pan. Required quantity of water was added into the soil mass until it was thoroughly blended. In order to attain the required moisture content for compaction, the water required

in addition to air-dry state was calculated and with this an additional water required for hydration was added to the soil and additives. For the hydration of lime water required was 47 per cent by weight of the lime (Kulkarni, 1977). The design moisture content of the mixes of the untreated and treated soils were equivalent to the respective optimum moisture contents corresponding to Standard Compaction tests for the untreated soils and soils treated with different lime contents.

3.8.2 Mould for Compression Test

The mould used for compacting untreated soil and soil-lime mix was fabricated using seamless mild steel pipe. The mould comply with the requirements of standard steel cylindrical mould with necessary accessories as outlined in ASTM D1632. The mould was fabricated for the preparation of compression test specimens of soil-lime in the laboratory under controlled test conditions. The design and dimensions of the mould are shown in Fig. 3.1. Mould having inside diameter of 2.8 ± 0.01 in. (71 ± 0.25 mm) and height of 9 in. (229 mm) for moulding test specimens 2.8 in. (71 mm) in diameter and 5.6 in. (142 mm) high, machined steel top and bottom pistons having a diameter 0.005 in. (0.13 mm) less than the mould, a 6 in. (152 mm) long mould extension, and a spacer clip were fabricated. Two moulds with necessary accessories were fabricated and four moulds which were earlier fabricated by Rajbongshi (1997) were used for this research work.

3.8.3 Compaction of Samples

Compression test samples of untreated and treated soils were prepared with the cylinder of size 2.8 inch (71.1 mm) in diameter by 5.6 inch (142.2 mm) in height. As soon as the mixing was complete, the inside surface of the mould was coated with oil. The cylindrical mould was held in place with the spacer clip over the bottom piston so that the spacer clip is extended about 25 mm into the cylinder. A separating disk was placed on top of the bottom piston and an extension sleeve was placed on top of the mould. The quantity of the uniformly mixed sample was placed in the mould. The sample was then compacted initially from the bottom up steadily and firmly with a square end cut $\frac{1}{2}$ -in. (13 mm) diameter smooth steel rod repeatedly through the mixture from the top down. The compaction was done uniformly over the cross section of the mould. The process was repeated until the sample was compacted to a height of approximately 6 inch (150 mm).

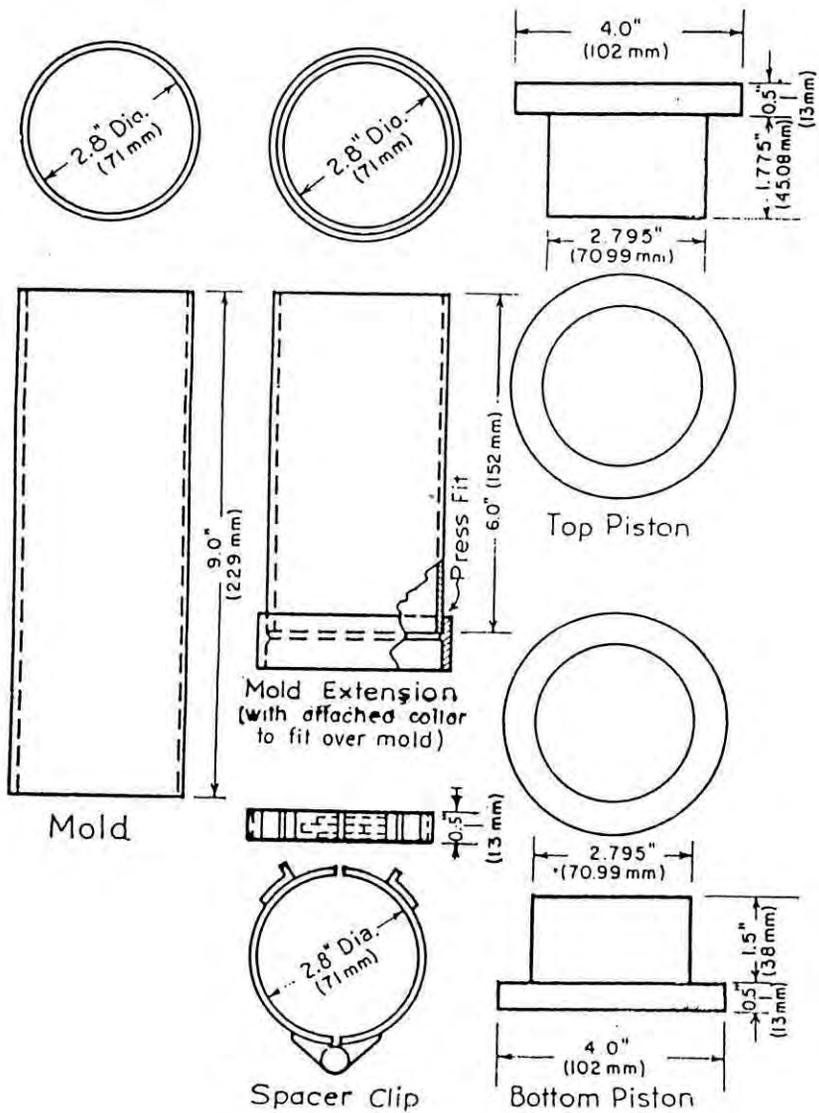


Fig. 3.1 Soil-lime mould for compressive strength test (after ASTM, 1989)

A separating disk was placed on the surface of the sample after removal of the extension sleeve. Spacer clip was then removed from the bottom of the piston. The top piston was placed in contact with the top surface of the sample and a static load was applied by a hydraulic compression machine until the sample became 5.6 inch (142 mm) high. The sample was then ejected from the mould using a hydraulic ejector. The compacted dry density of the samples were approximately equal to their respective maximum dry density achieved in the Standard Compaction test performed according to the standard procedures outlined in ASTM D698 (ASTM, 1989).

3.8.4 Curing of Samples

As soon as the samples were ejected from the mould, the samples prepared for unconfined compressive strength were then kept on a level table covered with wetted jute hessian cloth to maintain moist condition. The samples were never cured with direct water spray or under submerged condition. The samples were always protected from free water for the specified moist curing periods of 1 week, 2 weeks, 4 weeks, 8 weeks and 16 weeks. It may be mentioned that the soil samples prepared without adding lime, i.e., the untreated samples were not cured. For long term curing of the stabilised samples, the jute hessian cloth was wetted continuously.

3.8.5 Compression Test

The stabilised samples were placed on the compression testing machine immediately after removal from the moist curing condition at different ages. A strain gauge attachment of perspex was used to monitor deformation during the application of load. Each sample was tested under strain controlled condition. During the progress of test, load was applied continuously and without shock at a deformation rate of approximately 0.05 in. (1 mm) per minute. The total load and the corresponding deformation at failure were recorded. The untreated samples were tested in compression immediately after preparation. Fig. 3.2 presents photograph of the compression test apparatus showing a sample being tested.

During testing, the axial deformation of the sample and the corresponding load applied were recorded at frequent intervals in order to plot the compressive stress versus axial strain curve. The maximum load causing failure of the sample was taken as unconfined compressive strength of the sample.

3.9 CALIFORNIA BEARING RATIO (CBR) TEST ON COMPACTED UNTREATED AND STABILISED SAMPLE

3.9.1 Preparation and Mixing of Soils

The untreated soil and soils treated with various lime contents were prepared and mixed in accordance with the procedure outlined in section 3.8.1 For the stabilised samples, lime was used in percentages of 3, 6, 9, 12 and 15. The design moisture content of the untreated samples and samples stabilised with lime were equivalent to the respective values of optimum moisture contents as obtained from the Standard Compaction tests (ASTM D698) for the untreated soils and soils stabilised with different lime contents.

3.9.2 Compaction of Samples

For compaction of the moist untreated and treated samples, a cylindrical mould of 6 inch (152.4 mm) inside diameter and of volume 0.075 ft^3 was used. Each sample was compacted in five layers of approximately equal height. Each layer was compacted by 56 blows from a rammer of weight 10 lb (4.54 kg) and dropping from a free height of 18 inch (457 mm). In order to investigate CBR - dry density relationships for the untreated and stabilised soils, laboratory CBR tests were carried out on the untreated samples and samples treated with lime using another two levels of compaction energies equivalent to 10 and 25 blows in five approximately equal layers with a hammer of weight 10 lb and 18 inches free fall and compacted in a mould of volume 0.075 ft^3 . After completion of compaction, extension collar was removed and the compacted soil was trimmed by means of a straight edge. Perforated base plate and spacer disk were removed and finally, moisture content and dry density determinations were made on each of the compacted sample. All these tests were performed following standard procedure outlined in ASTM D1883 (ASTM, 1989).

3.9.3 Soaking of Sample

A disk of coarse filter paper was placed on perforated base plate. The mould and compacted sample were inverted and the perforated base plate was clamped to the mould with compacted sample in contact with the filter paper. A surcharge weight of 10 lb (4.54 kg) was placed on the perforated plate and adjustable stem assembly placed onto the compacted sample in the mould. The mould and weights were immersed in water allowing free access of water to the top and bottom of the sample. Initial measurements were taken for swell and the sample was allowed to soak for 96 hours (4 days). A constant level of water was maintained during this period. At the end of 96 hours, final swell measurement was taken.

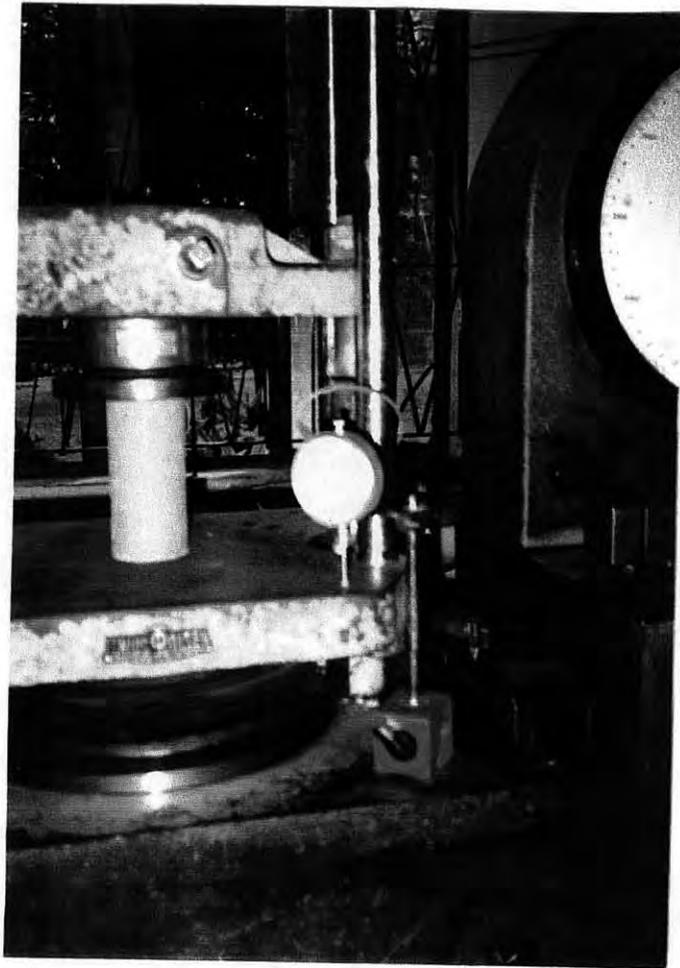


Fig. 3.2 Photograph showing the set-up for unconfined compression test

3.9.4 Bearing Test

The free water from the sample was removed and the sample was allowed to drain for 15 min. Care was taken not to disturb the sample during removal of water. A surcharge weight equivalent to that used during soaking period was placed on the sample. In order to prevent upheaval of the sample into the hole of the surcharge weights, a 2.27 kg annular weight was placed on the sample surface prior to seating the penetration piston, after which the remainder of the surcharge weights were placed. The penetration piston was seated with the smallest possible load (not more than 44 N). Load was applied on the penetration piston so that the rate of penetration was approximately 0.05 in. (1.27 mm) per min. The load readings were monitored at specified values of penetrations. All these tests were performed following the standard procedure outlined in ASTM D1883 (ASTM, 1989). Fig. 3.3 presents a photograph of the bearing test apparatus.

Load versus penetration curve was plotted for each sample and necessary corrections were applied to the load-penetration curve. CBR value was determined from the corrected load versus penetration curve.

3.10 FLEXURE TEST USING SIMPLE BEAM WITH THIRD-POINT LOADING SYSTEM

3.10.1 Preparation and Mixing of Soils

The untreated and soils treated with various lime contents were prepared and mixed in accordance with the procedure outlined in section 3.8.1 For the stabilised samples, Lime was used in percentages of 3, 6, 9, 12 and 15. The moisture content of the untreated samples and samples stabilised with lime were equivalent to the respective values of optimum moisture contents as obtained from the Standard Compaction tests (ASTM D698) for the untreated soils and soils stabilised with different lime contents.

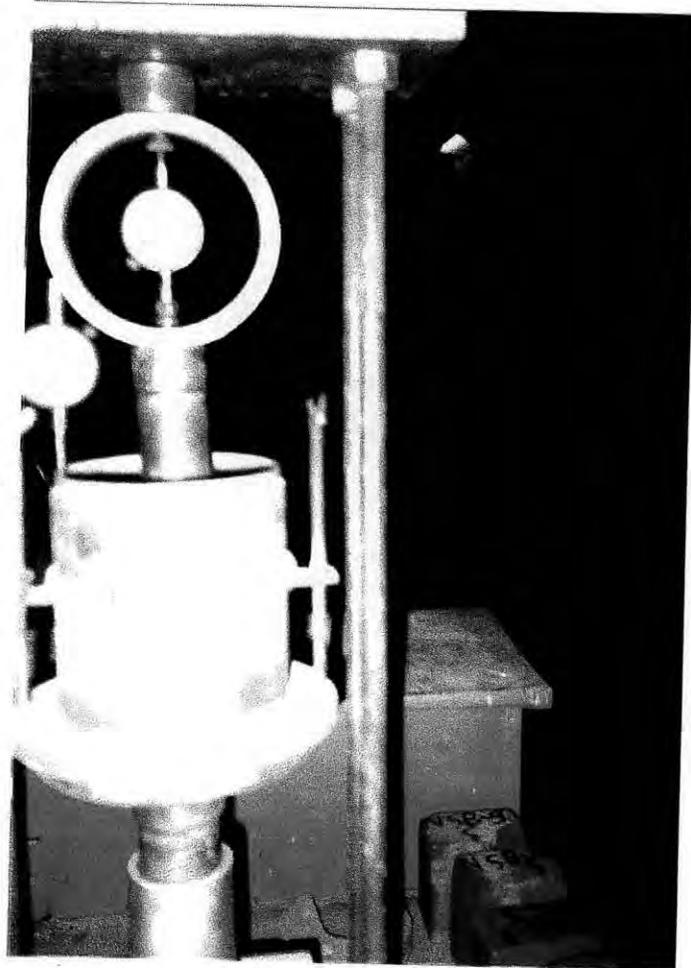


Fig. 3.3 Photograph showing the arrangements of CBR test

3.10.2 Mould for Flexure Test

The mould used for compacting untreated soil and soil-lime mixtures were fabricated using mild steel plates comply with the requirements of ASTM D1632 (ASTM, 1989). The fabrication procedure of this mould was rather difficult as compared with that for compression cylindrical mould. The mould consists of one piece of top plate, one piece of bottom plate, two pieces of side plates and two pieces of end plates. The top and bottom plates and side and end plates of the mould were made first by mild steel casting. After casting, the mould was shaped in proper dimensions through machining work. The detail design and dimensions of the mould for flexure test are shown in Fig. 3.4. This mould has inside dimensions of 3 in. by 3 in. by 11¼ in. (76.2 mm by 76.2 mm by 285.8 mm) for moulding specimens of the same size. The mould was manufactured in such a way the sample could be moulded with its longitudinal axis in a horizontal position. The parts of the mould were made to be tight-fitting and held together. The sides of the mould were sufficiently rigid to prevent spreading or warping. The interior faces of the mould were machined to plane surfaces within a variation, in any 3 in. (76.2 mm) line on a surface, of 0.002 in (0.051 mm). The distance between opposite sides was within 3 ± 0.01 in. (76.20 ± 0.25 mm). The height of the mould was made 3 in. (76.20 mm) within the variation of -0.01 in. (-0.25 mm). Four 3/8 in. (9.52 mm) spacer bars and top and bottom machined steel plates were provided. The plates fit the mould with a 0.005 in. (0.13 mm) clearance on all sides.

3.10.3 Moulding and Curing of Sample

The test samples were prepared with the longitudinal axis horizontal. The inside parts of the mould were first lightly oiled. Then the mould was assembled with the sides and ends separated from the base plate by the 3/8 in. (9.52 mm) spacer bars, one placed at each corner of the mould. Representative soil sample of required quantity was taken to prepare test sample of desired density, i.e., the maximum dry density obtained in the Standard Compaction test. Moisture content of air-dry soil sample was determined. The uniformly mixed sample was divided into three equal batches to make a beam of the designed density. One batch of the material was placed in the mould and levelled by hand. The sample was compacted initially from the bottom up by steadily and firmly, with impact a square-end cut ½-inch (13 mm) diameter smooth steel rod repeatedly through the mixture from the top down to the point of refusal.

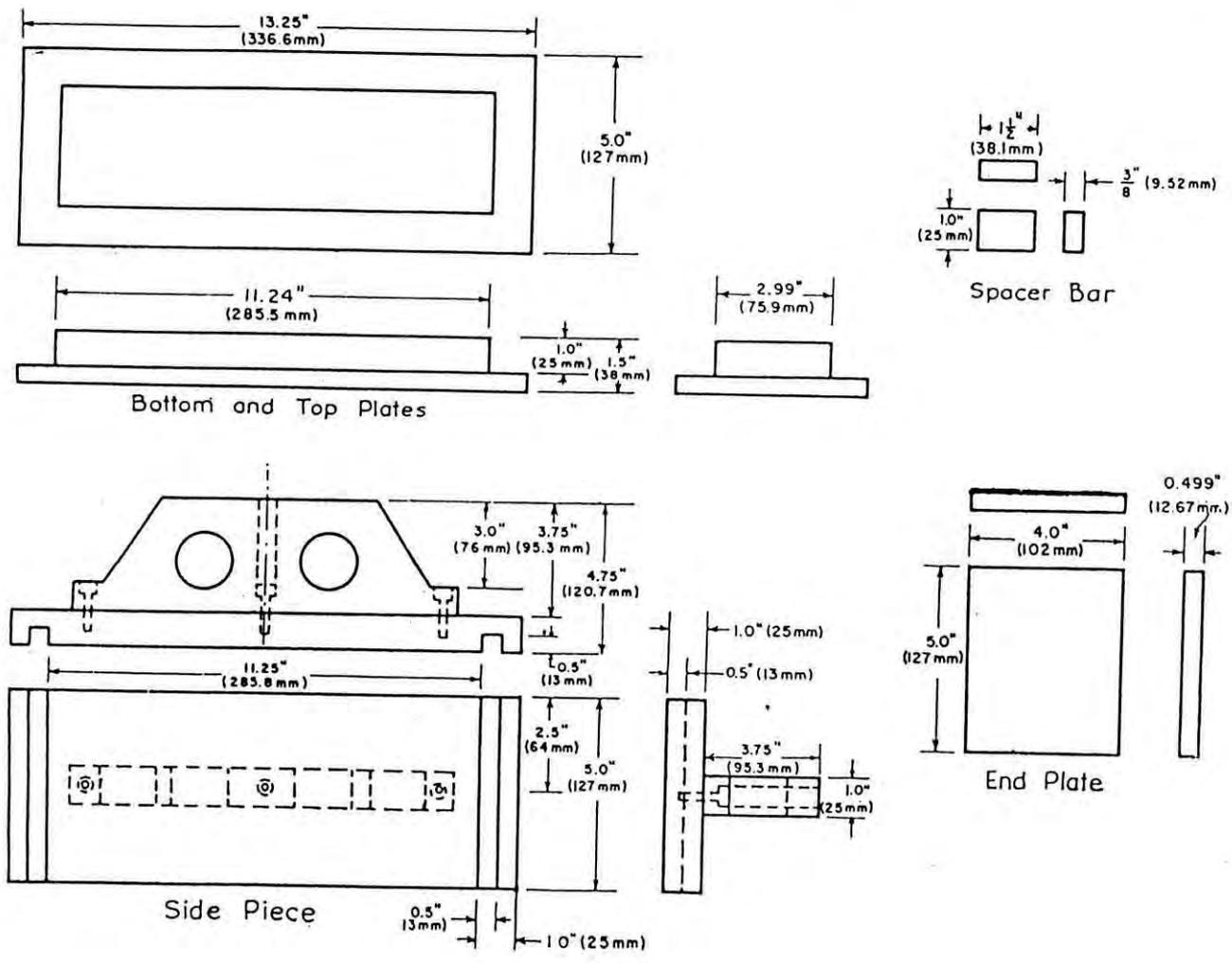


Fig. 3.4 Schematic diagram of soil-lime beam mould for flexural test with third point loading system (after ASTM, 1989)

Approximately 90 roddings were distributed uniformly over the cross section of the mould. This layer of compacted sample was levelled by hand and the layer two and three were compacted in the similar way. The sample at this time was made approximately $3\frac{3}{4}$ inch high. The top plate of the mould was then placed in position and spacer bars were removed. The final compaction was done with a static load applied by the hydraulic compression machine until the design height of 3 inch was reached. Immediately after the compaction, the mould was carefully dismantled and the sample was removed onto a smooth, rigid wooden pallet.

As soon as the soil-lime samples were removed from the mould, they were kept on a table covered with wetted jute hessian cloth. The samples were never cured with direct water spray or under submerged condition. The samples were always protected from free water for the specified moist curing periods of 1, 2, 4, 8 and 16 weeks. The soil samples prepared without adding lime were not cured. The treated samples were taken for testing purpose directly from the moist curing environment. For long term curing of the stabilised samples the jute hessian cloth was wetted continuously.

3.10.4 Flexural Strength Test

The flexure tests of untreated soil and soil-lime beam samples were performed in order to determine the flexural strength and flexural modulus of the samples by the use of a simple beam with third point loading system. The standard test samples were made 3 inch by 3 inch by $11\frac{1}{4}$ in. The sample was turned on its side with respect to its moulded position and centered it on the lower half-round steel supports, which was spaced apart a distance of three times the depth of the beam (i.e., 9 inch). The load applying assembly block was placed in contact with the upper surface of the beam at the third points between the supports. The centre of the beam was aligned with the centre of the thrust of the spherically seated head block of the machine. The movable parts of this head block were rotated as needed by hand until uniform seating was obtained. The load was applied continuously without any shock on the beam through the third point loading system. A hand operated compression machine was used with a proving ring of load capacity 10 kN. Load was applied at a deformation rate of approximately 0.05 in./min. (0.02 mm/s). Two dial gauges were fitted under the beam specimen to record the deflection of the beam. The total load until failure of the specimen was recorded. A schematic diagram of the apparatus for flexure test of soil and soil-lime samples by third point loading is shown in Fig. 3.5 while Fig. 3.6 shows the a photograph of a test set-up.

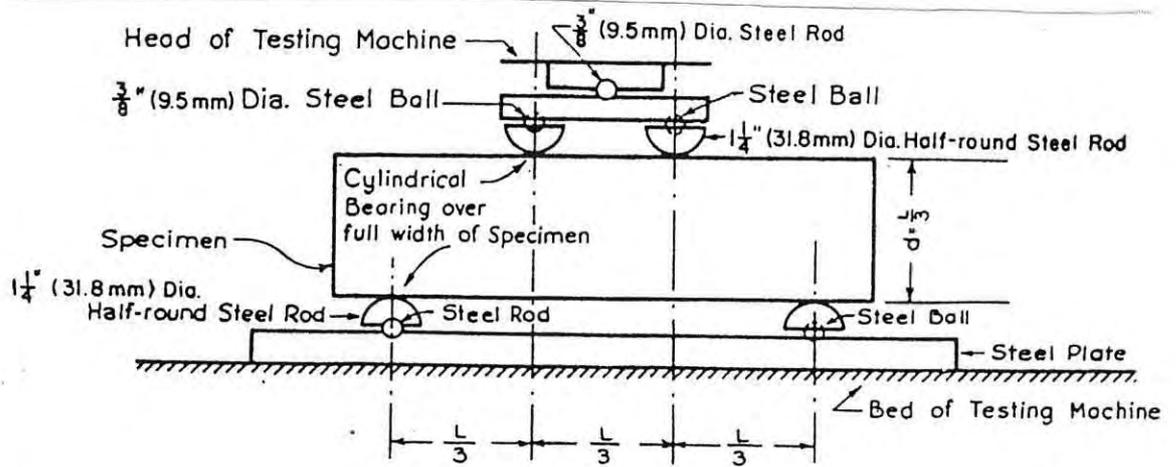


Fig. 3.5 Schematic diagram of the set-up for flexural test with third point loading system (after ASTM, 1989)

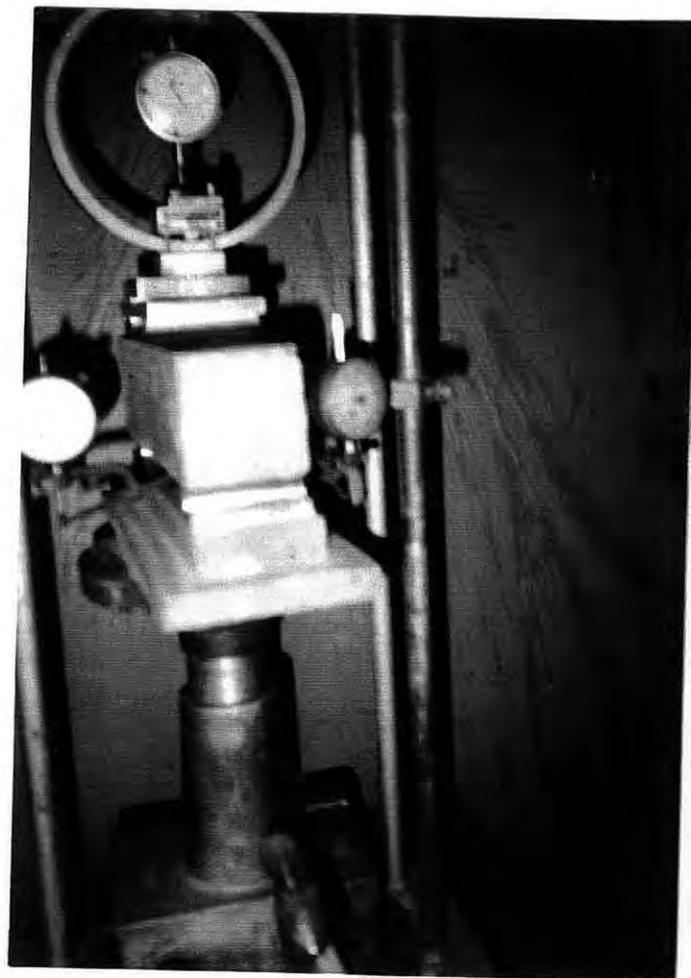


Fig. 3.6 Photograph showing the set-up for flexural test with third point loading system

The fracture location after the test was observed. When the fracture occurred within the middle third of the span length, the modulus of rupture (Flexural strength) has been calculated using the following expression:

$$R = \frac{PL}{bd^2} \quad (3.2)$$

where,

R = modulus of rupture or flexural strength

P = maximum applied load

L = span length of sample

b = average width of sample

d = average depth of sample

When the fracture occurred outside the middle third of the span length by not more than 5% of the span length, the modulus of rupture has been calculated using the following equation:

$$R = \frac{3Pa}{bd^2} \quad (3.3)$$

where:

a = distance between line of fracture and the nearest support measured along the centre line of the bottom surface of the beam.

The flexural modulus (E) of the untreated soil and soil-lime beam samples, as found from flexural strength tests were calculated using the following expression of simple beam theory:

$$E = \frac{23 PL^3}{1296 I \Delta} \quad (3.4)$$

where,

P = maximum applied load

L = span length of sample

I = moment of inertia of the beam section

Δ = deflection of the beam in the mid span

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 GENEAL

The findings of the laboratory investigations on the index and swelling properties of untreated samples collected from different locations of Rajendrapur Cantonment, Gazipur are presented. The relative degree of expansion of the untreated soil samples has been assessed in order to select the most expansive soil used for subsequent lime stabilisation. Finally, the physical and engineering properties of the lime stabilised expansive soil have been presented and discussed.

4.2 INDEX AND SWELLING PROPERTIES OF UNTREATED SOILS

Disturbed samples were collected from six different locations of Rajendrapur Cantonment, Gazipur. These samples were then air-dried and the soil lumps were broken carefully with a wooden hammer so as to avoid breakage of soil particle. The required quantity of soil was then sieved through sieve No. 40 (0.425 mm). Physical and index tests were performed on the sieved sample.

The grain size distribution curves of the six samples are presented in Appendix-A. The different fractions of sand, silt and clay of the samples were found from the grain size distribution curves following the MIT Textural Classification System (1931). The soils were classified according to Unified Soil Classification System (ASTM D2487). Table 4.1 presents the values of index and shrinkage properties, grain size distribution and classifications of samples of the six soils. The liquid limit and plasticity index of the six soil samples varied from 45 to 56 and 28 to 43, respectively, while the shrinkage limit and linear shrinkage varied from 11 to 21 and 13 to 20, respectively. The percentages of clay, silt and sand fractions of the samples varied from 22 to 46, 44 to 55 and 4 to 23, respectively while the percentage of material finer than No. 200 sieve varied between 74 and 96. Except Soil-A which is a clay of high plasticity (CH), the rest soil samples are clays of low to medium plasticity (CL).

Table 4.1 Index properties and classification of six soil samples collected from different locations of Rajendrapur Cantonment, Gazipur

Index Properties and Classification	Soil-A	Soil-B	Soil-C	Soil-D	Soil-E	Soil-F
Specific gravity	2.57	2.49	2.52	2.52	2.53	2.50
Liquid limit	56	48	45	50	47	46
Plastic limit	13	20	17	20	18	17
Plasticity index	43	28	28	30	29	29
Shrinkage limit	11	16	12	16	14	13
Linear shrinkage (%)	20	13	14	15	14	14
% Sand (>0.06 mm)	4	12	16	18	16	23
% Silt (0.06 mm to 0.002 mm)	50	48	44	49	44	55
% Clay (<0.002 mm)	46	40	40	33	40	22
% of material finer than No-200 sieve	96	84	86	84	86	74
Unified Soil Classification	CH	CL	CL	CL	CL	CL

A number of swelling properties of all the six soil samples were determined in order to assess the swelling characteristics of the samples and to select the most expansive soil for subsequent stabilisation with lime. The swelling properties investigated include free swell, free swell index, swelling pressure, swelling potential and volume change from air-dry to saturated condition. A summary of the swelling properties of the six untreated samples is presented in Table 4.2. Swelling pressures of the untreated expansive samples were determined from the plots of volume change versus surcharge pressure curve of the respective sample. The plots of volume change versus surcharge pressure curves for laboratory compacted samples of six soils are presented in Appendix-B. From Table 4.2, it can be seen that free swell and free swell index of the six soil samples vary from 44% to 75% and 20% to 40%, respectively while swelling pressure and swelling potential of laboratory compacted samples vary from 24 kPa to 53 kPa and 1.55% to 2.5 %, respectively. It can be seen from Table 4.2 that volume change from air-dry to saturated condition of the samples varied between 17% and 30%.

Table 4.2 Swelling properties of six soil samples collected from different locations of Rajendrapur Cantonment, Gazipur

Swelling Properties	Soil-A	Soil-B	Soil-C	Soil-D	Soil-E	Soil-F
Free swell (%)	75	45	46	48	47	44
Free swell Index (%)	40	22	21	22	23	20
Swelling pressure of laboratory compacted sample (kPa)	53	27.50	26.0	25.0	25.0	24.0
Swelling potential of laboratory compacted sample (%)	2.5	1.6	1.65	1.7	1.6	1.55
Volume change from air dry to saturated condition (%)	30	17	18	19	18	17

4.3 ASSESSMENT OF DEGREE OF EXPANSION OF UNTREATED SOIL SAMPLES AND SELECTION OF EXPANSIVE SOIL FOR LIME STABILISATION

Different index properties and swelling properties of the six samples as shown in Tables 4.1 and 4.2, respectively, have been compared with the corresponding values for expansive soils of varying degree of expansion. Based on the values of index and swelling properties, the various recommended criteria for expansive soils have already been presented in Article 2.6 of Chapter 2. By comparing the laboratory measured values of index and swelling properties of the six samples with the various recommended criteria outlined in Article 2.6 of Chapter 2, the relative degree of expansion of the six soil samples has been evaluated. A summary of the relative degree of expansion of the six soil samples has been presented in Table 4.3. Table 4.3 shows the following findings:

- (i) Based on the values of plasticity index and shrinkage limit as proposed by Holtz (1959), the degree of expansion of Soil-A is high while the degree of expansion of the rest of the soil samples is low to medium
- (ii) Based on the values of liquid limit and plasticity index as recommended by IS : 2911 (1980), the degree of expansion of Soil-A and Soil-D is high to very high and high, respectively, while the degree of expansion of the rest of the soil samples is medium to high.
- (iii) Based on the values of liquid limit and plasticity index as recommended by United States Army Engineers Waterways Experimental Station (Snethen, 1959), the degree of expansion of Soil-A and Soil-D is marginal to high and marginal, respectively, while the degree of expansion of the rest of the soil samples is low to marginal.
- (iv) Based on the values of linear shrinkage data of Bangladesh soils as recommended by Hossain(1983), the degree of expansion of Soil-A and Soil-D is high while the degree of expansion of the rest of the soil samples is medium.
- (v) Based on the values of free swell as proposed by IS : 1948 (1970), the degree of expansion of Soil-A is medium while the degree of expansion of the rest of the soil samples is low.
- (vi) Based on the values of free free swell index as proposed by IS : 2911 (1980), the degree of expansion of Soil-A is high while the degree of expansion of the rest of the soil samples is medium.

- (vii) Based on the values of swelling pressure as proposed by Chen (1975), the degree of expansion of Soil-A is low to medium while the degree of expansion of the rest of the soil samples is low.
- (viii) Based on the values of swelling potential as recommended by Seed et al. (1962), the degree of expansion of all the soil samples is medium.
- (ix) Based on the values of volume change from air dry to saturated condition as proposed by Holtz (1959), the degree of expansion of Soil-A is high while the degree of expansion of the rest of the soil samples is medium.

On the basis of the above mentioned findings, it has been concluded that the overall degree of expansion of Soil-A is high while the overall degree of expansion of Soil-D is medium. The overall degree of expansion of the rest four soils, i.e., Soil-B, Soil-C, Soil-E and Soil-F is low to medium. Therefore, Soil-A, collected from POL Store Area of Rajendrapur Cantonment, Gazipur has been selected for lime stabilisation. Soil-A was stabilised with five different lime contents of 3%, 6%, 9%, 12% and 15% in order to study the influence of lime stabilisation on geotechnical properties of this expansive soil. In the following sections the physical and engineering properties of the untreated expansive Soil-A and stabilised samples of the selected expansive Soil-A have been presented and discussed.

4.4 EFFECT OF LIME STABILISATION ON PLASTICITY AND SHRINKAGE CHARACTERISTICS

The values of plasticity and shrinkage properties of the untreated and lime stabilised samples of the expansive soil are shown in Table 4.4. Table 4.4 shows the following effects of lime stabilisation on the plasticity and shrinkage properties of the expansive soil:

- Compared with the untreated sample of the soil, liquid limit of the stabilised samples initially decreased with the addition of lime content and then increased
- Compared with the untreated sample, plastic limit and shrinkage limit, increased
- Compared with the untreated sample, plasticity index, volumetric shrinkage, shrinkage ratio and linear shrinkage decreased.

Table 5.3 Assessment of degree of expansion for different soils investigated

Soil	Degree of expansion based on different parameters proposed by various researchers								
	Plasticity Index & Shrinkage Limit (Holtz, 1959)	Liquid Limit & Plasticity Index (IS: 2911, 1980)	Liquid Limit & Plasticity Index (Snethen, 1959)	Linear Shrinkage (Hossain, 1983)	Free Swell (IS :1948, 1970)	Free Swell Index (IS : 2911, 1980)	Swelling Pressure (Chen, 1975)	Swelling Potential (Seed et al., 1962)	Volume Change from Air-Dry to Saturated Condition (Holtz, 1959)
Soil-A	High	High to very high	Marginal to high	High	Medium	High	Low to medium	Medium	High
Soil-B	Low to medium	Medium to high	Low to marginal	Medium	Low	Medium	Low	Medium	Medium
Soil-C	Low to medium	Medium to high	Low to marginal	Medium	Low	Medium	Low	Medium	Medium
Soil-D	Low to medium	High	Marginal	High	Low	Medium	Low	Medium	Medium
Soil-E	Low to medium	Medium to high	Low to marginal	Medium	Low	Medium	Low	Medium	Medium
Soil-F	Low to medium	Medium to high	Low to marginal	Medium	Low	Medium	Low	Medium	Medium

Fig 4.1 shows the variation of liquid limit and plastic limit with the increase in lime content while Fig. 4.2 presents the change in plasticity index with increasing lime content. Fig. 4.1 shows that liquid limit initially decreases with the increment of lime content up to 6% and then increases with increasing lime content. Fig, 4.1, however, shows that plastic limit increases drastically with increasing lime content. Compared with the untreated sample, plastic limits of samples have been found to increase by 1.8 to 3 times due to stabilisation with 3% to 15% lime content. It can be seen from Fig. 4.2 that plasticity index of lime treated expansive soil decreases markedly due to increase in lime content from 3% to 15%. Compared with the untreated sample, plasticity indices of the lime treated samples were found to reduce by 30% to 56% because of stabilisation with 3% to 15% lime content. Ahmed (1984) and Rajbongshi (1997) investigated the effect of increasing lime content on the liquid limit, plastic limit and plasticity index of soils of Bangladesh. Ahmed (1984) found an increase in plastic limit while liquid limit and the plasticity index decreased with increasing addition of lime. Rajbongshi (1997) found an increase in plastic limit while liquid limit and the plasticity index reduced with increasing addition lime content. The results of the present investigation agrees favourably with those reported by Ahmed (1984) and Rajbongshi (1997) for a number of soils of Bangladesh. Holtz (1969) reported the effects of lime on plastic characteristics of four expansive montmorillonitic clays. Holtz (1969) found that lime drastically reduces liquid limit and plasticity index of montmorillonitic clays.

Figs. 4.3, 4.4, 4.5 and 4.6 show the variation of linear shrinkage, shrinkage limit, volumetric shrinkage and shrinkage ratio, respectively, with the increment of lime addition. It can be seen from Fig. 4.3 that linear shrinkage decreases significantly with the increase in lime content, while Fig. 4.4 shows that shrinkage limit increases drastically with increasing lime content. Compared with the untreated sample, shrinkage limit has been found to increase by 2 to 3.36 times due to treatment with 3% to 15% lime while linear shrinkage was found to reduce by 35% to 75% due to stabilisation with 3% to 15% lime. Rajbongshi (1997) also found an increase in shrinkage limit while linear shrinkage reduced with increasing addition of lime for samples of a. coastal soil of Bangladesh. IRC (1976) and Bell (1993) also reported reduction in linear shrinkage with increasing lime content. Holtz (1969) found that lime drastically raises the shrinkage limit of expansive montmorillonitic clays.

Fig. 4.5 shows that volumetric shrinkage decreases considerably with increasing lime content while Fig. 4.6 shows that shrinkage ratio also reduces with the increase in lime content. Compared with the untreated sample, volumetric shrinkage and shrinkage ratio were found to reduce by 48% to 82% and 31% to 46%, respectively, due to stabilisation with 3% to 15% lime.

Table 4.4 Comparison of index and shrinkage properties of untreated and lime treated expansive soil samples

Index and Shrinkage Properties	Lime Content (%)					
	0	3	6	9	12	15
Liquid limit (%)	56	54	53	54	57	59
Plastic limit (%)	13	24	28	32	37	40
Plasticity index (%)	43	30	25	22	20	19
Shrinkage limit (%)	11	22	25	29	33	37
Volumetric shrinkage (%)	90	47	36	23	18	16
Shrinkage ratio	2.02	1.39	1.30	1.20	1.15	1.10
Linear shrinkage (%)	20	13	11	9	7	5

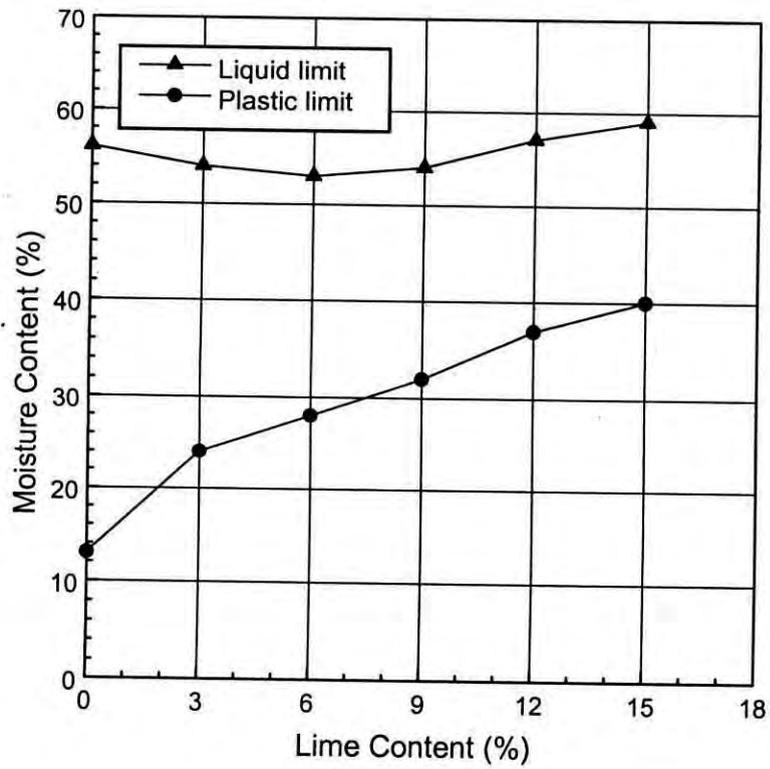


Fig 4.1 Effect of lime content on liquid limit and plastic limit

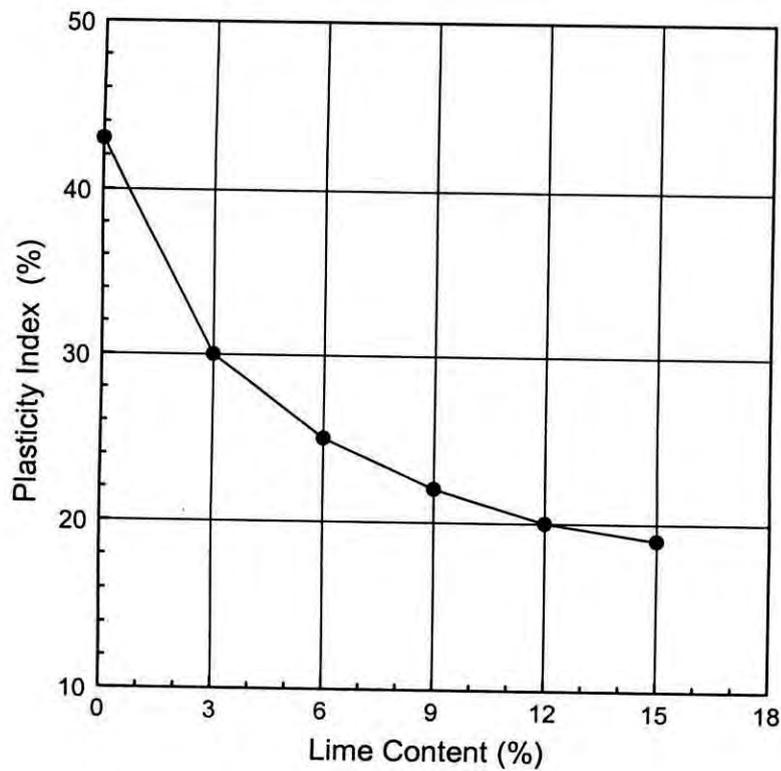


Fig 4.2 Effect of lime content on plasticity index

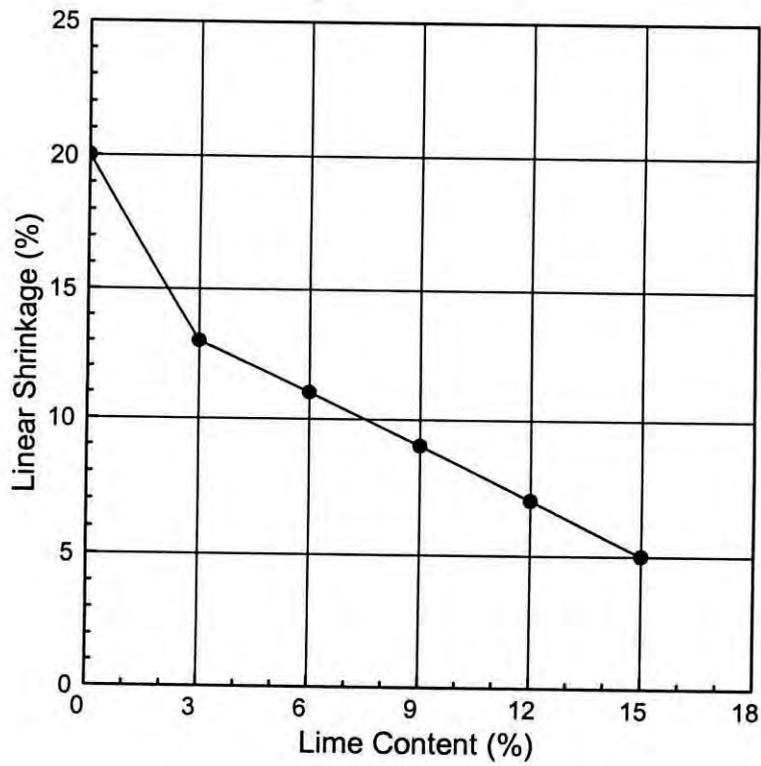


Fig 4.3 Effect of lime content on linear shrinkage

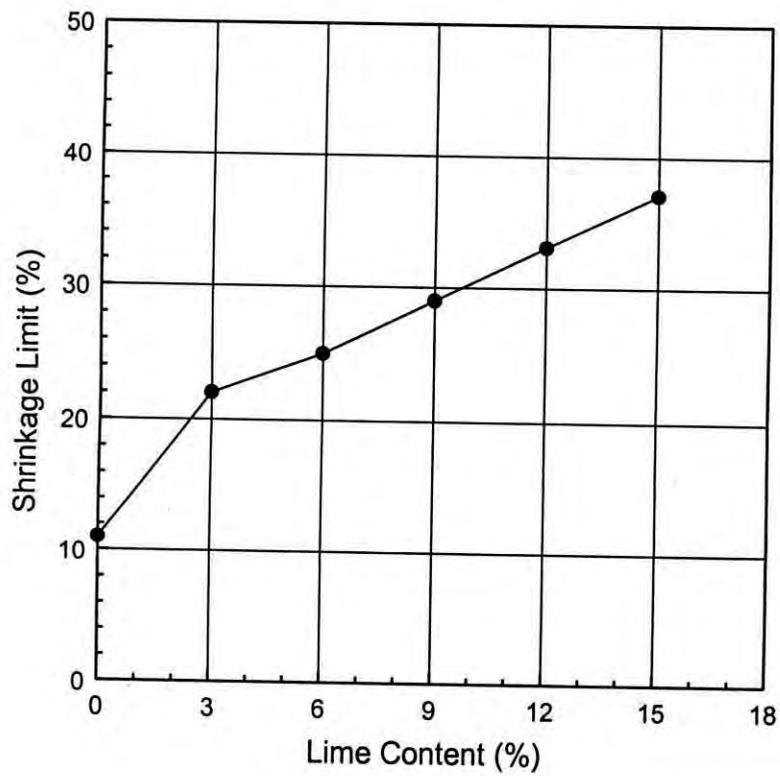


Fig 4.4 Effect of lime content on shrinkage limit

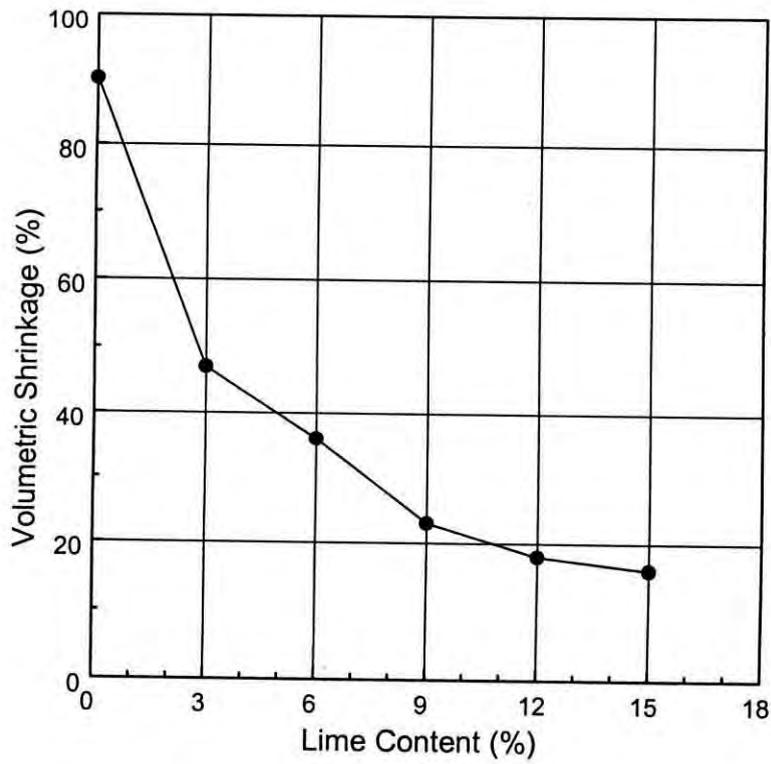


Fig 4.5 Effect of lime content on volumetric shrinkage

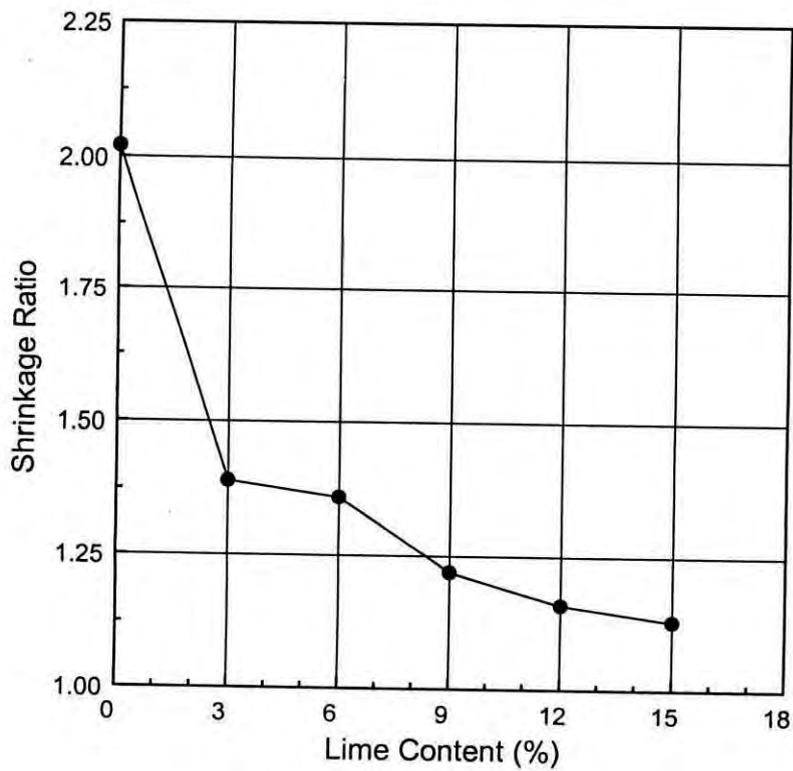


Fig 4.6 Effect of lime content on shrinkage ratio

4.5 EFFECT OF LIME ON SWELLING PROPERTIES OF EXPANSIVE SOIL

Attempt has been made to investigate the influence of lime stabilisation on swelling properties of the expansive soil. Table 4.5 presents a comparison of the swelling properties of untreated and lime stabilised samples of the expansive soil. Swelling pressures of the lime treated expansive samples were determined from the plots of volume change versus surcharge pressure curve of the respective sample. Fig. 4.7, Fig. 4.8 and Fig. 4.9 presents the plots of volume change versus surcharge pressure curve for expansive samples stabilised with 3%, 6% and 9% lime, respectively. Table 4.5 shows the following effects of lime stabilisation on swelling properties of the expansive soil:

- Compared with the untreated sample of the soil, free swell and free swell index of the stabilised samples decrease.
- Compared with the untreated sample, swelling pressure of the treated samples reduced.
- Compared with the untreated sample, swelling potential of the stabilised samples decreases.
- Compared with the untreated sample, volume change of the stabilised samples from air-dry to saturated condition decreases considerably.
- Swelling pressure and swelling potential become zero when the soil has been stabilised with 9% and 12% lime.

The variations of free swell and free swell index of the treated samples with the increase in lime content are shown in Figs. 4.10 and 4.11, respectively. Fig. 4.10 shows that free swell decreases significantly with increasing lime content. Fig. 4.11 shows that free swell index reduces markedly with the increase in lime content. Compared with the untreated sample, free swell and free swell index were found to reduce by 17% to 87% and 38% to 95%, respectively, due to stabilisation with 3% to 15% lime.

The variations of swelling pressure and swelling potential with the increase in lime content have been presented in Figs. 4.12 and 4.13, respectively. It can be seen from Fig. 4.12 that swelling pressure of the stabilised samples reduces drastically with the increase in lime content. Fig. 4.13 shows that swelling potential of the treated samples decreases sharply with increasing lime content. Swelling potential and swelling pressures are usually decreased significantly by treating clay with lime. These reduced swell characteristics are generally attributed to decreased affinity for water of the calcium

saturated clay and the formation of a cementitious matrix that resists volumetric expansion. It can also be seen from Table 4.5 that volume change of the stabilised samples from air-dry to saturated condition decreased considerably with the increase in lime content. Compared with the untreated sample, volume change of the stabilised samples from air-dry to saturated condition were found to decrease by 55% to 83% due to stabilisation with 3% to 15% lime.

The above mentioned results on the influence of lime stabilisation on swelling properties of the expansive soil clearly demonstrate that lime is an effective additive in reducing the various swelling properties of an expansive soil.

Table 4.5 Comparison of swelling properties of untreated and lime stabilised expansive soil samples

Swelling Properties	Lime Content (%)					
	0	3	6	9	12	15
Free swell (%)	75	62	52	38	15	10
Free swell index (%)	40	25	17	10	4	2
Swelling pressure of laboratory compacted sample (kPa)	53	15	7	2	0	0
Swelling potential of laboratory compacted sample (%)	1.8	0.40	0.20	0.075	0	0
Volume change from air-dry to saturated condition (%)	29.0	13.0	10.8	7.5	6.0	5.0

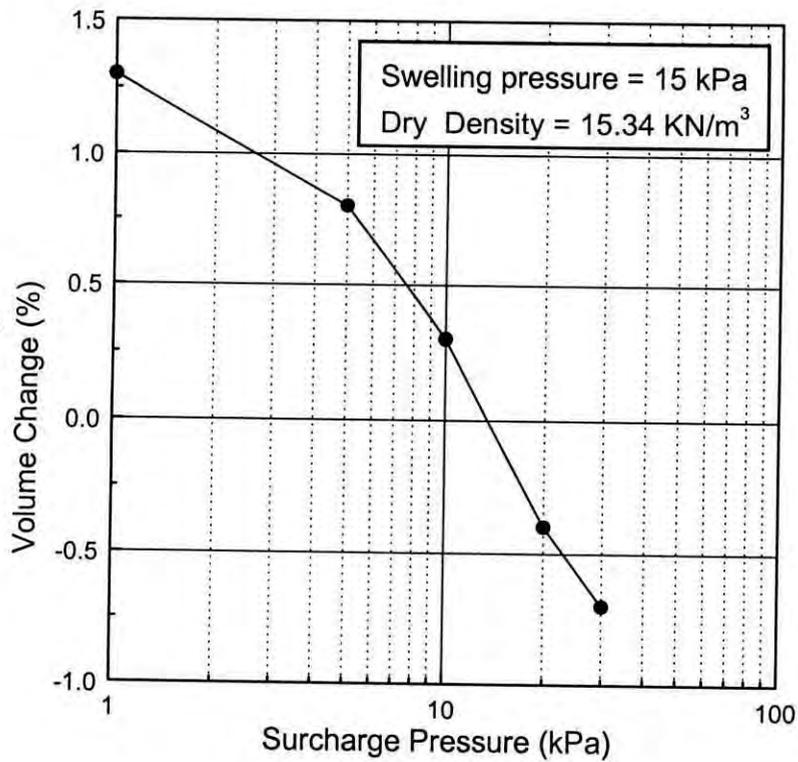


Fig. 4.7 Volume change versus surcharge pressure curve for 3% lime treated laboratory compacted soil

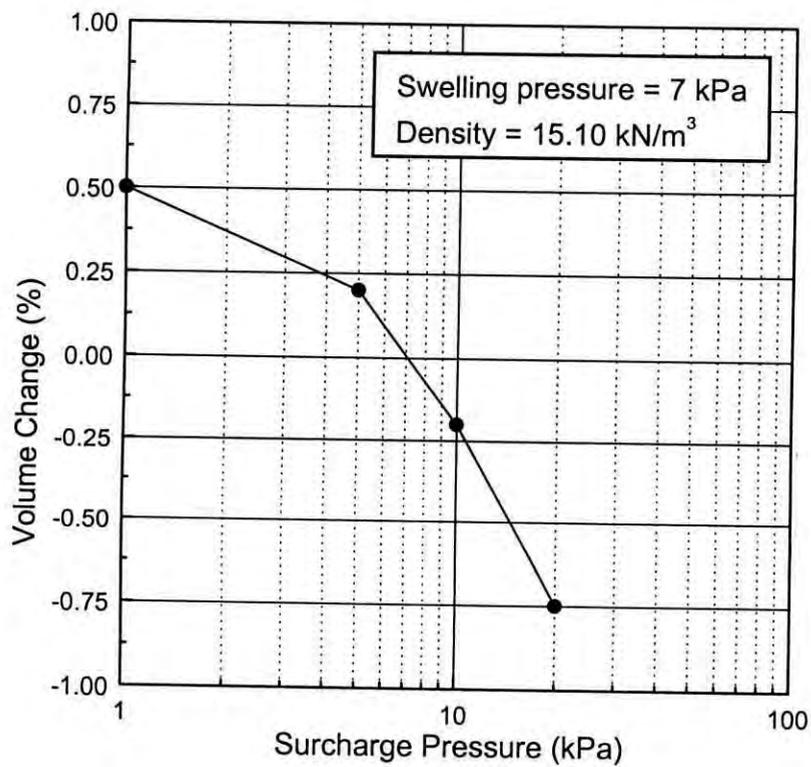


Fig. 4.8 Volume change versus surcharge pressure curve for 6% lime treated laboratory compacted soil

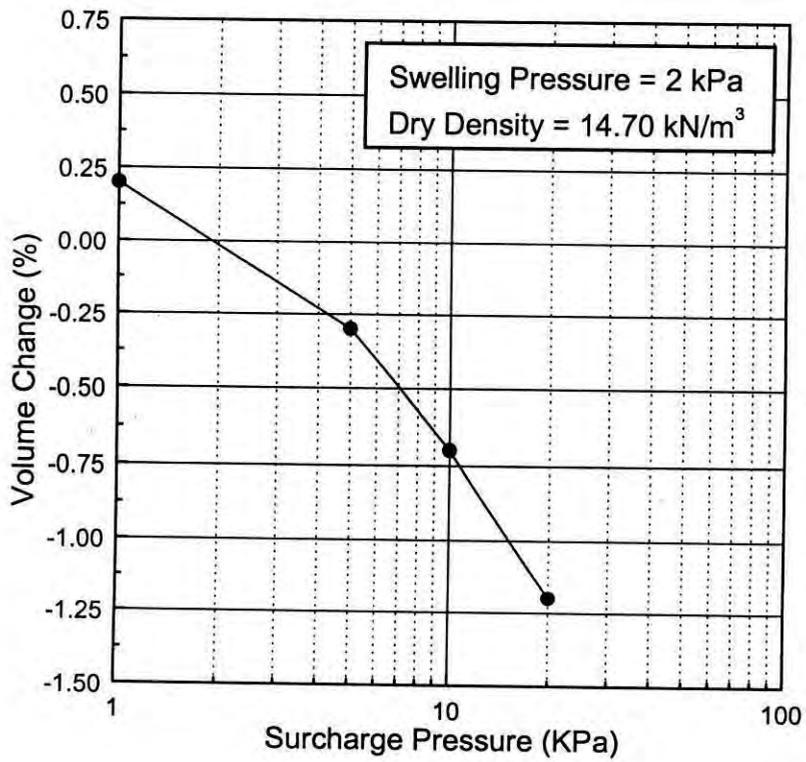


Fig. 4.9 Volume change versus surcharge pressure curve for 9% Lime treated laboratory compacted soil

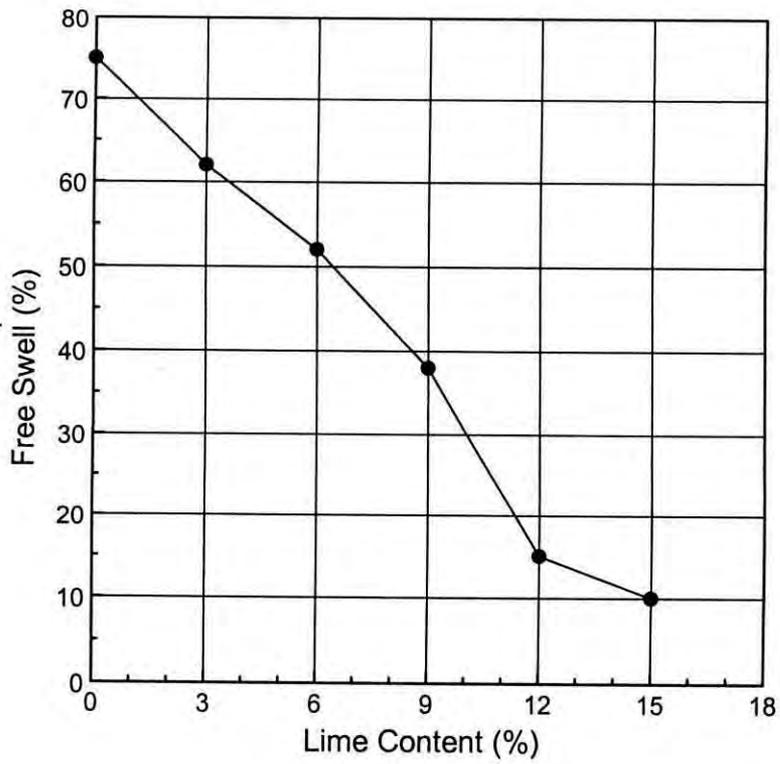


Fig. 4.10 Effect of lime content on free swell

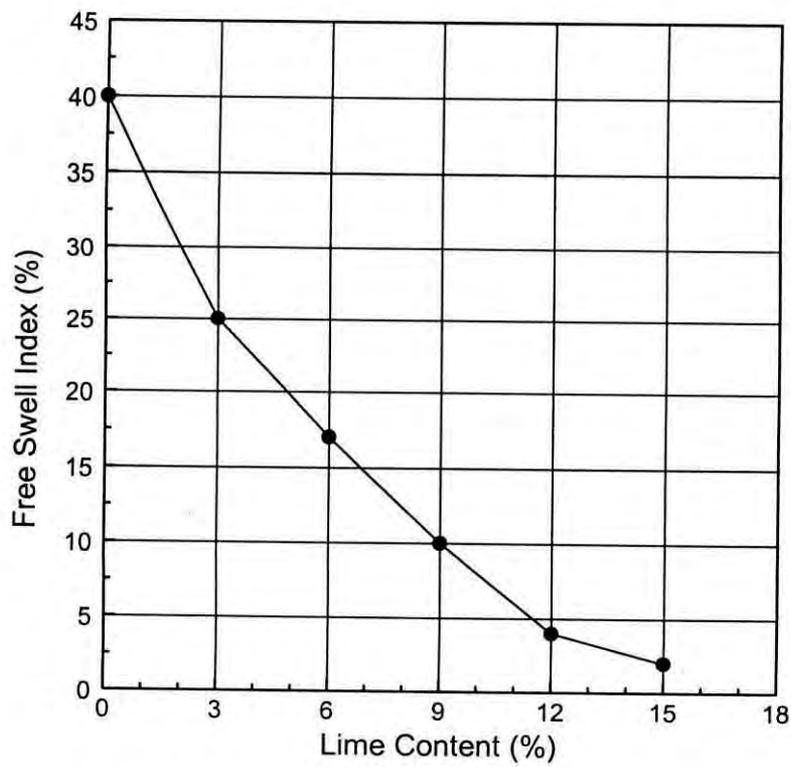


Fig. 4.11 Effect of lime content on free swell index

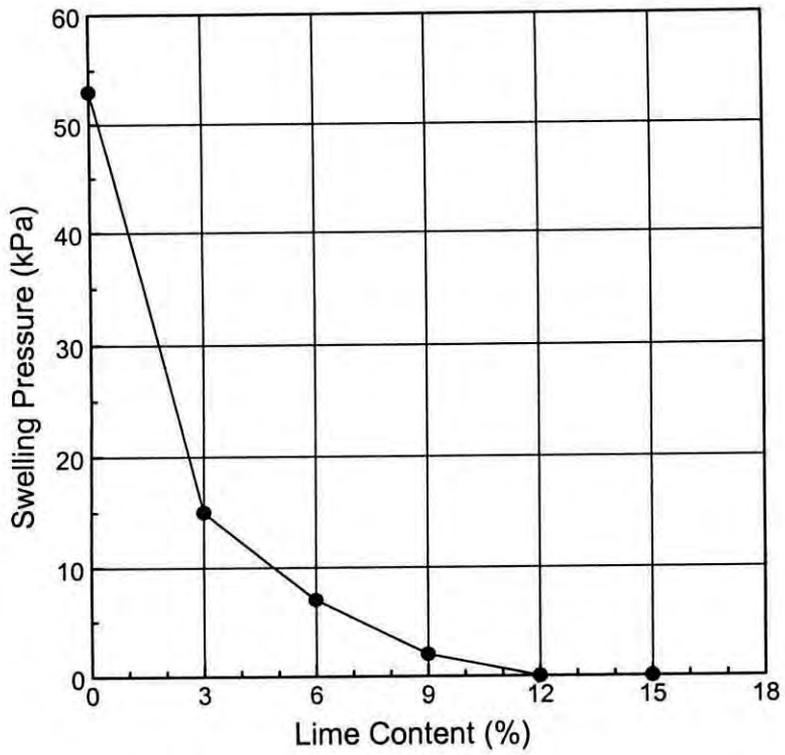


Fig. 4.12 Effect of lime content on swelling pressure of laboratory compacted soil

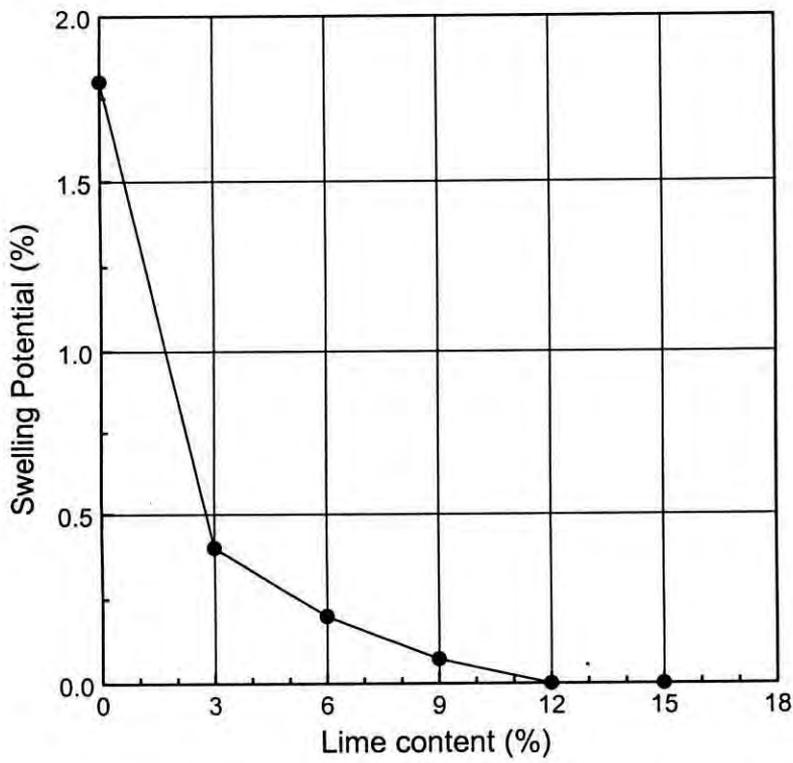


Fig.4.13 Effect of lime content on swelling potential of laboratory compacted soil

4.6 MOISTURE- DENSITY RELATIONS OF UNTREATED AND TREATED SAMPLES

Moisture - density relations of untreated and lime treated samples of the expansive soil are shown in Fig 4.14. From the relations presented in Fig 4.14, the maximum dry density (γ_{dmax}) and optimum moisture content (w_{opt}) of samples of the expansive soil have been determined which are presented in Table 4.6.

It can be seen from Table 4.6 that with the increase in lime content values of γ_{dmax} decreased while the values of w_{opt} increased. The reduction in γ_{dmax} with the increase in lime content for the stabilised samples is shown in Fig. 4.15 while Fig. 4.16 shows the increase in optimum moisture content with the increase in lime content. It has been found that compared with the untreated sample, the values of γ_{dmax} decreased by 10% to 17% for an increase in lime content from 3% to 15%. The values of w_{opt} have been found to increase by 10% to 65% due to stabilisation with 3% to 15% lime.

Ahmed (1984) found that for sandy silt and silty clay soils of Bangladesh, the maximum dry density reduced with the increase in lime content. Serajuddin and Azmal (1991) also found that compared with untreated samples the maximum dry density of lime treated samples of two fine-grained regional soils decreased while optimum moisture content slightly increased. For a Chittagong coastal soil, Rajbongshi (1997) reported an increase in w_{opt} and a reduction in γ_{dmax} . Molla (1997) also found an increase in w_{opt} and a reduction in γ_{dmax} for three regional soils (Liquid limit = 34 to 47, Plasticity index = 9 to 26) of Bangladesh stabilised with lime. Reductions in the values of γ_{dmax} with increasing lime content have also been reported by a number of other researchers (Kezdi, 1979; TRB, 1987; Hausman, 1990; Bell 1993).

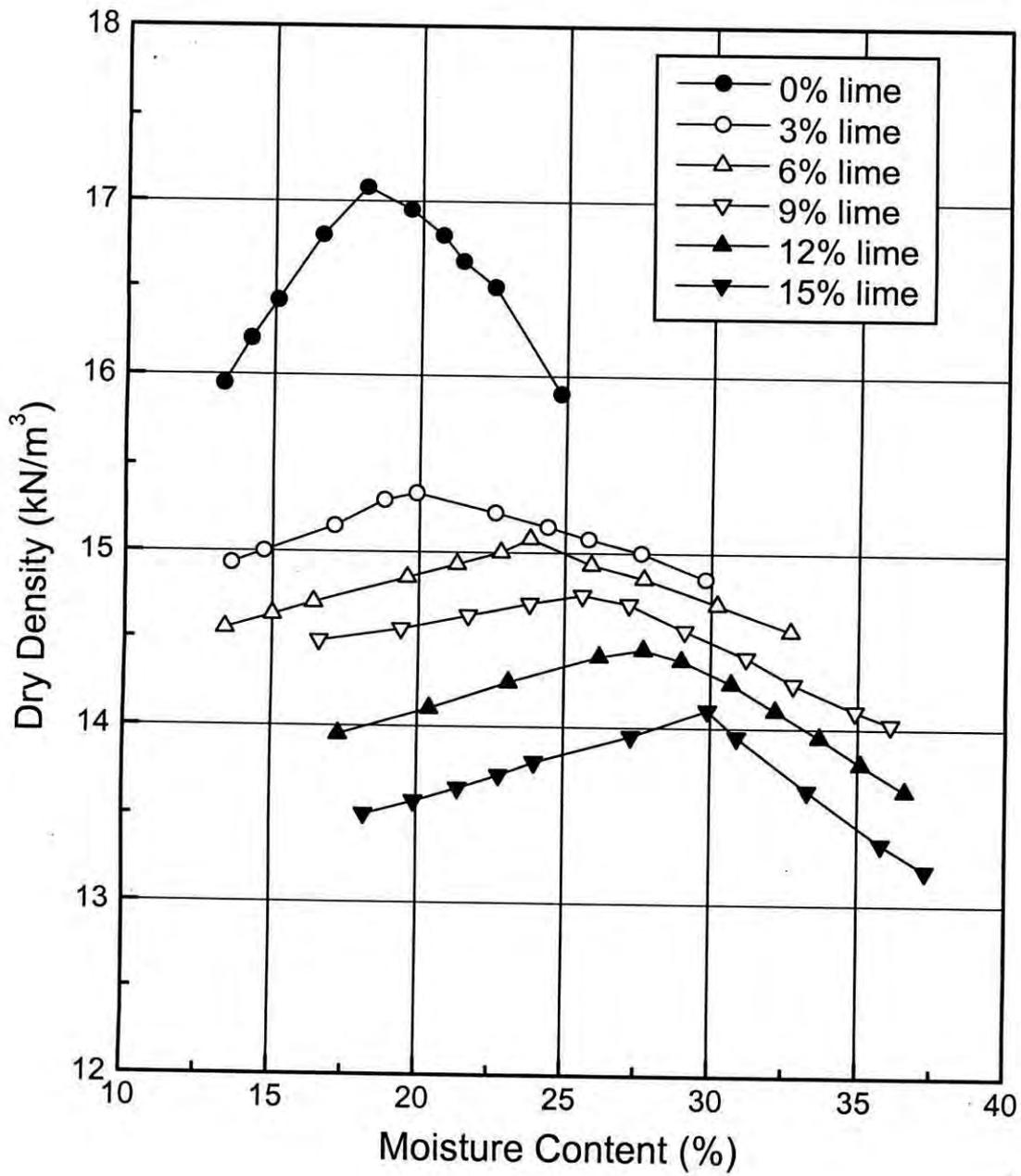


Fig 4.14 Moisture - density relations of untreated and lime treated samples of expansive soil

Table 4.6 Values of maximum dry-density and optimum moisture content of untreated and lime treated expensive soil

Lime Content (%)	Maximum Dry Density, γ_{dmax} (kN/m³)	Optimum Moisture Content, w_{opt} (%)
0	17.08	18.1
3	15.34	19.9
6	15.08	23.8
9	14.75	25.6
12	14.44	27.7
15	14.10	29.9

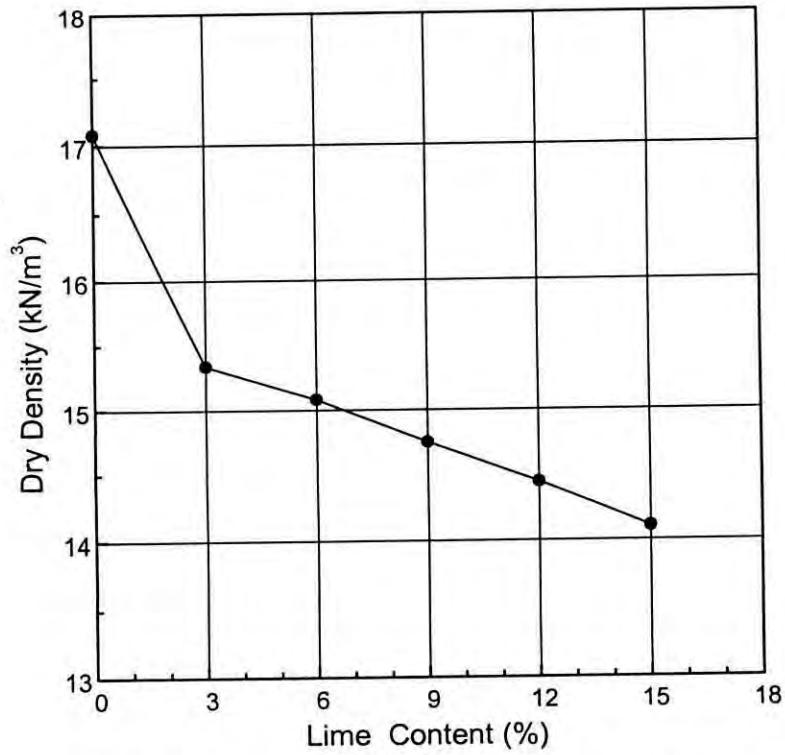


Fig 4.15 Effect of lime content on maximum dry density of lime treated expansive soil

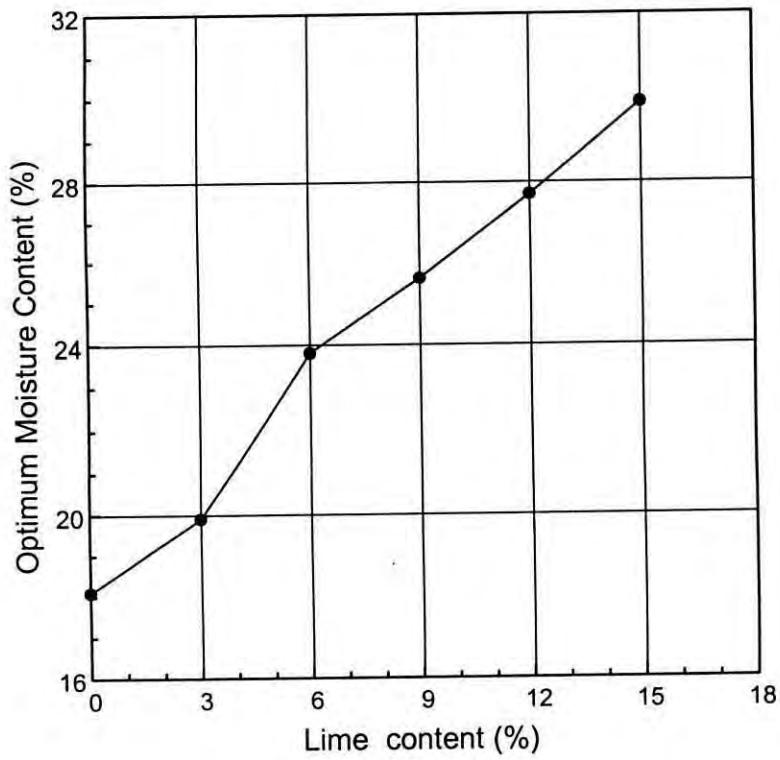


Fig 4.16 Effect of lime content on optimum moisture content of lime treated expansive soil

4.7 EFFECT OF LIME STABILISATION ON UNCONFINED COMPRESSIVE STRENGTH

Table 4.7 shows a summary of the unconfined compression test results of the untreated and treated samples of the expansive soil. The values of unconfined compressive strength (q_u) and axial strain at failure (ϵ_f) for the untreated samples and samples treated with different lime contents of 3%, 6%, 9%, 12% and 15% and cured for 1 week, 2 weeks, 4 weeks, 8 weeks and 16 weeks are presented in Table 4.7. Table 4.7 shows that compared with the untreated sample, the values of q_u of the treated samples increased significantly, depending on the lime content and curing age. Earlier studies (Ahmed, 1984; Serajuddin and Azmal, 1991; Serajuddin, 1992; Rajbongshi, 1997; Mollah, 1997) on lime stabilisation investigated the influence of curing age of up to 4 weeks on unconfined compressive strength. It has been observed from the present investigation that long-term curing (up to 16 weeks) has marked influence on the gain in strength. It has been found that at any particular lime content, unconfined compressive strength continued to increase with the increase in curing age. The effect of long-term curing on the increase in unconfined compressive strength has been found to be more pronounced when samples were stabilised with higher lime contents (more than 3%). It was found that the value of q_u of sample treated with 15% lime and cured at 16 weeks was about 8.4 times higher than the strength of the untreated sample. Ahmed (1984) found that unconfined compression strength for sandy silt and silty clay samples treated with various lime contents (0.5% to 5%) increased with the increase in lime content and curing age. Serajuddin and Azmal (1991) and Serajuddin (1992) also reported that the unconfined compressive strength of regional alluvial soils of Bangladesh treated with 5%, 7.5% and 10% slaked lime increased with the increase in lime content and curing age. Rajbongshi (1997) investigated the effect of lime content and curing age on unconfined compressive strength of large diameter samples (2.8 in. diameter by 5.6 in. high) of a coastal soil. Rajbongshi (1997) reported that unconfined compressive strength of lime treated samples increased with the increase in lime content and curing age. Molla (1997) also found that unconfined compressive strength of lime treated samples increased with the increase in lime content and curing age for three regional soils of Bangladesh.

Table 4.7 Summary of unconfined compressive strength test results of untreated and lime treated expansive soil samples

Lime Content	Curing Age (week)	q_u (kN/m ²)	ϵ_r (%)
0	-	550	4.28
3	1	750	2.0
	2	850	1.8
	4	1100	1.7
	8	1230	1.60
	16	1350	1.65
6	1	1010	1.78
	2	1120	1.50
	4	1820	1.4
	8	2450	1.58
	16	3100	1.60
9	1	1100	2.04
	2	1220	1.60
	4	1930	1.55
	8	2640	1.60
	16	3450	1.95
12	1	1180	1.70
	2	1350	1.65
	4	2300	1.60
	8	2840	1.73
	16	3980	1.80
15	1	1440	1.80
	2	1620	1.70
	4	2650	1.50
	8	3150	2.10
	16	4600	2.20

Ingles and Metcalf (1972) recommended that when lime stabilisation has to be used in order to upgrade heavy clays to sub-base material quality type, a q_u -value of 250 psi (1722 kN/m²) or 150 psi (1033 kN/m²) to 450 psi (3100 kN/m²) at seven days is required. Table 4.7 shows that the unconfined compressive strength of samples treated with 6%, 9%, 12% and 15% lime contents fulfilled the requirements as proposed by Ingles and Metcalf (1972). Table 4.7 also shows that compared with the untreated samples, the values ϵ_f of the stabilised samples reduced. This finding evidently indicates that the treated samples became more brittle due to lime stabilisation. Similar results were also reported by Rajbongshi (1997) for samples of a coastal soil.

The relationship between q_u for different lime contents and for samples cured at different ages are shown in Fig 4.17 and Fig 4.18, respectively. Figs 4.17 shows that the values of q_u of treated samples cured at any particular age increased with increasing lime content while Fig. 4.18 shows that values of q_u of samples treated with a particular lime content increased with the increase in curing age.

The rate of strength gain with curing time has been evaluated in terms of the parameter strength development index (SDI) as proposed by Uddin (1990) and defined by equation 2.16. Table 4.8 presents the values of SDI of the lime stabilised samples cured at various ages. Plots of SDI versus curing age of treated samples are shown in Fig 4.19 while Fig. 4.20 presents the plots of SDI versus lime content. Fig. 4.19 shows that for samples stabilised with any particular lime content, the values of SDI increases with increasing curing age while Fig. 4.20 shows that for samples cured at any particular age, SDI increases with the increase in lime content. Figs. 4.19 and 4.20 clearly show the relative degree of strength gain resulted due to increasing curing age and lime content. As can be seen from Fig 4.19 that long-term curing has got significant effect on strength development. It is also evident from the plots of Fig. 4.20 that the strength gain for samples treated with 15% lime are relatively much higher than those of samples treated with less lime contents. Rajbongshi (1997) also investigated the rate of strength gain with curing age for lime stabilised samples of a coastal soil. Values of SDI have been found to increase with increasing curing time and lime content, as obtained in the present investigation.

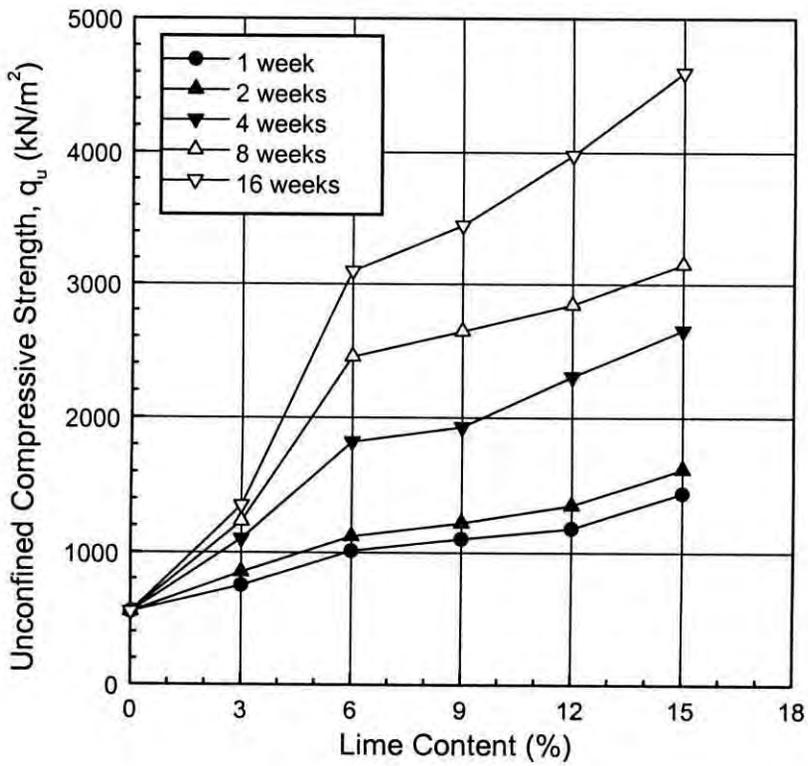


Fig 4.17 Effect of lime content on unconfined compressive strength

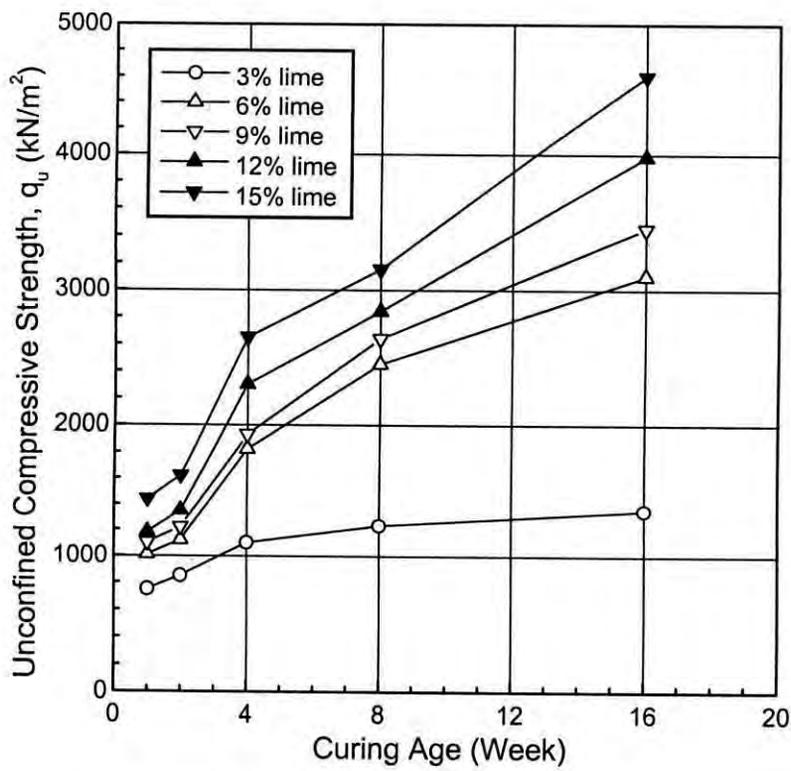


Fig 4.18 Effect of curing age on unconfined compressive strength

Table 4.8 Strength development index (SDI) of lime stabilised samples cured at different curing ages

Curing Age (week)	Values of SDI for Lime Stabilised Samples				
	3%	6%	9%	12%	15%
1	0.36	0.84	1.00	1.15	1.62
2	0.55	1.036	1.22	1.45	1.95
4	1.00	2.31	2.51	3.18	3.82
8	1.24	3.45	3.80	4.16	4.73
16	2.45	4.63	5.27	6.26	7.36

4.8 EFFECT OF MOULDING WATER CONTENT ON UNCONFINED COMPRESSIVE STRENGTH

In order to investigate the influence of moulding water content on unconfined compressive strength, unconfined compression strength tests were carried out on 2.8 inch diameter by 5.6 inch high samples, stabilised with 3%, 6%, 9%, 12% and 15% lime content and cured for 1 week, 2 weeks, 4 weeks, 8 weeks and 16 weeks. The samples were compacted according to the Standard Compaction test with two additional moulding water contents other than the optimum moisture content. The following additional water contents were used:

- (i) Water content corresponding to dry side of optimum moisture content at 95% of Standard Proctor compaction.
- (ii) Water content corresponding to wet side of optimum moisture content at 95% of Standard Proctor compaction.

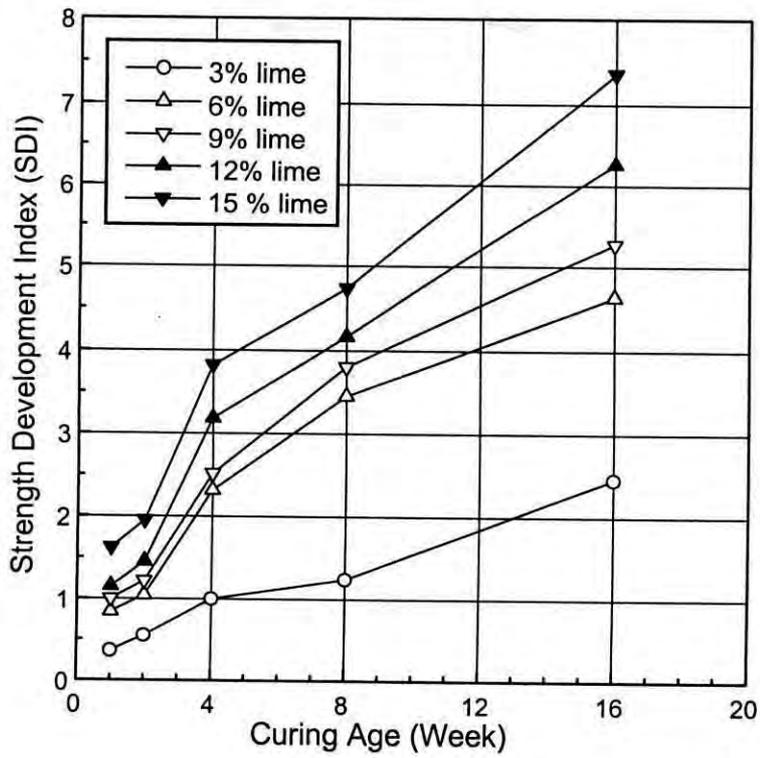


Fig. 4.19 SDI Versus curing age curves for lime treated expansive soil

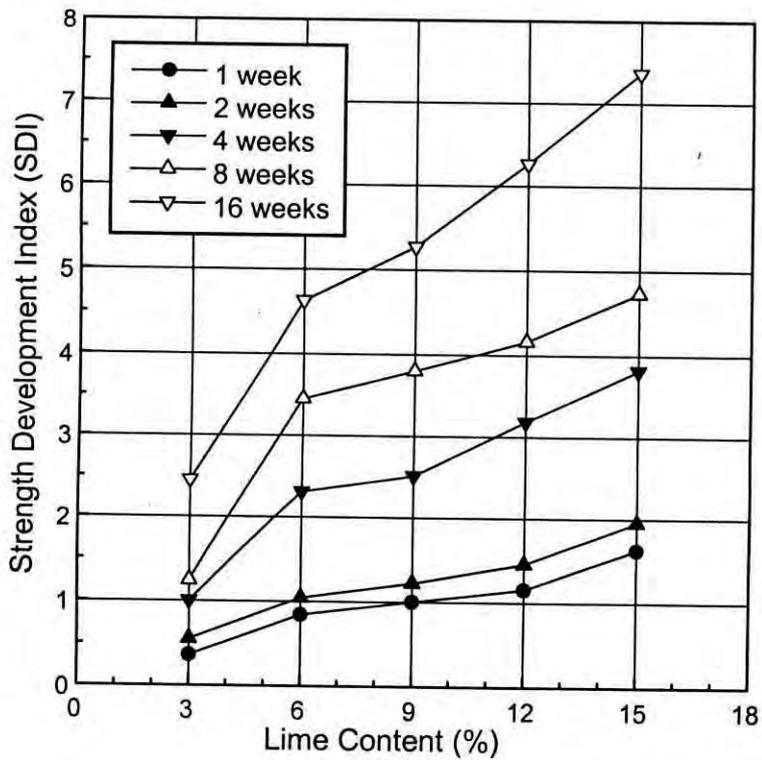


Fig. 4.20 SDI Versus lime content curves for lime treated expansive soil.

Comparisons of the values of q_u of samples stabilised with different lime contents and cured at different ages for the three moulding water contents are presented in Table 4.9. It can be seen from Table 4.9 that irrespective of curing ages and lime content, values of q_u is maximum and minimum at moulding moisture contents of wet side of optimum and dry side of optimum, respectively. Ahmed (1984) and Rajbongshi (1997) also found higher compressive strength for lime stabilised samples of sandy silt and silty clay samples of Bangladesh compacted at wet side of optimum moisture content than samples compacted at their optimum moisture contents.

The variation of q_u with curing age for samples treated with 3%, 6%, 9%, 12% and 15% lime and compacted with different moulding water contents are shown in Figs. 4.21, 4.22, 4.23, 4.24 and 4.25, respectively. It can be seen from Figs 4.21 to 4.25 that the values of q_u increase with the increase in curing age and that at any particular curing age, the values of q_u of samples compacted at wet side are higher than the values of q_u samples compacted at optimum or dry side of optimum water content. It therefore appears that in order to achieve adequate compressive strength, lime stabilised samples should be compacted at the wet side of their optimum moisture contents.

The relative increase in q_u -value i.e., $q_{u(\text{wet})} - q_{u(\text{dry})}$, when compacted using moulding water contents equal to wet side and dry side of optimum moisture content, has been found to depend on the lime content and curing age. This has been shown in Fig. 4.26. It can be seen from Fig. 4.26 that the increase in q_u -value increases with increasing lime content and curing age.

The variations of unconfined compressive strength with moulding water content have been presented in Figs. 4.27, 4.28, 4.29 and 4.30 for samples stabilised with 3%, 6%, 12% and 15%, respectively. Figs. 4.27 to 4.30 clearly show that irrespective of curing age, unconfined compressive strength increases with the increase in moulding water content. Molla (1997) also investigated the effect of moulding moisture content on unconfined compressive strength of lime treated samples of three regional soils of Bangladesh. Unconfined compressive strength of samples was found to increase with the increase in moulding moisture content.

Table 4.9 Unconfined compressive strength test results at different moulding water content for lime stabilised expansive soil

Moulding Water Content (w_{opt})	Curing Age (week)	q_u (kN/m ²)				
		3% Lime	6% Lime	9% Lime	12% Lime	15% Lime
Optimum moisture content (w_{opt})	1	750	1010	1100	1180	1440
	2	850	1120	1220	1350	1620
	4	1100	1820	1930	2300	2650
	8	1230	2450	2640	2840	3150
	16	1350	3100	3450	3980	4600
Dry side of w_{opt} at 95% compaction	1	465	610	660	700	865
	2	620	730	720	900	1000
	4	720	1200	1160	1560	1690
	8	800	1570	1560	1875	1970
	16	900	2050	2250	2510	3000
Wet side of w_{opt} at 95% compaction	1	880	1110	1210	1275	1570
	2	1110	1275	1330	1640	1780
	4	1260	1980	2130	2620	2934
	8	1355	2690	2905	3320	3545
	16	1480	3410	3795	4370	5000

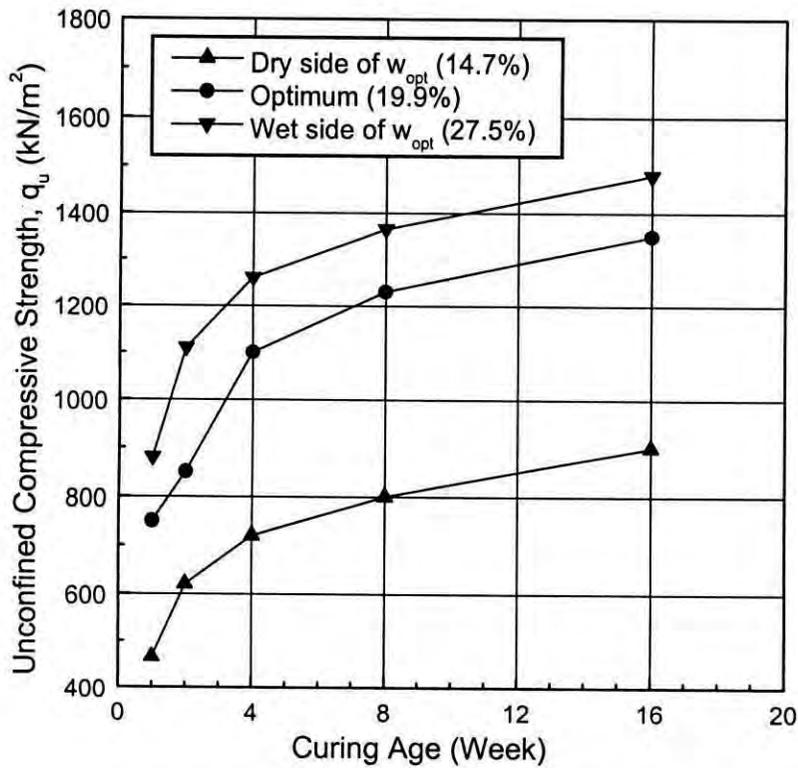


Fig 4.21 Effect of moulding water content on q_u for 3% lime treated expansive soil

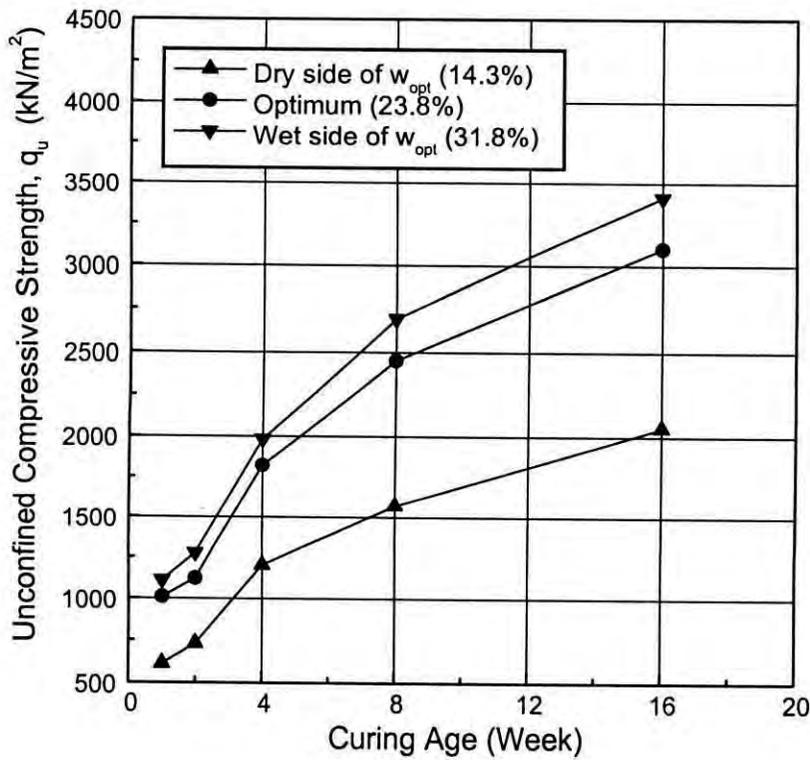


Fig 4.22 Effect of moulding water content on q_u for 6% lime treated expansive soil

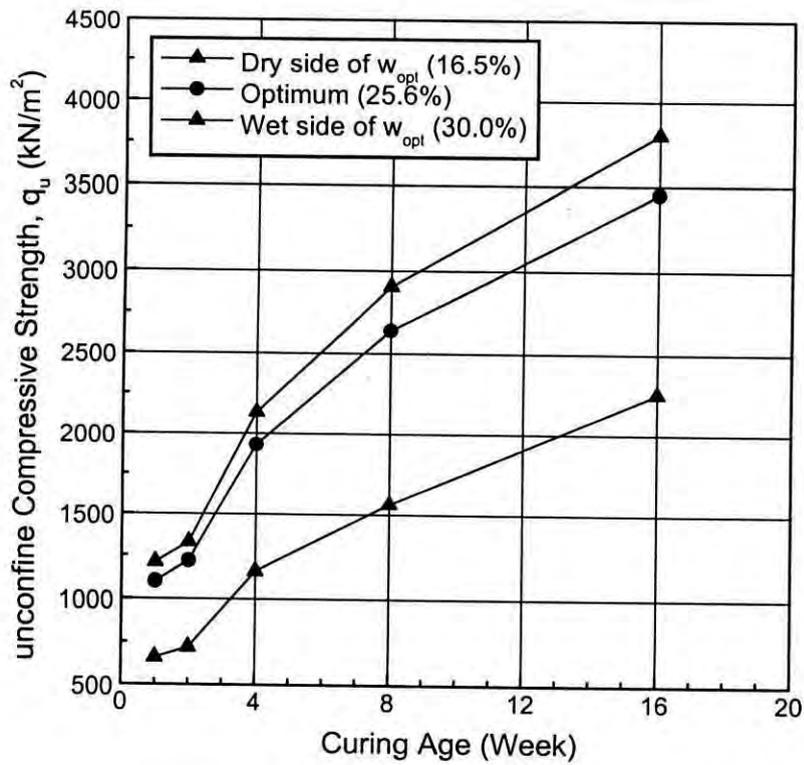


Fig 4.23 Effect of moulding water content on q_u for 9% lime treated expansive soil

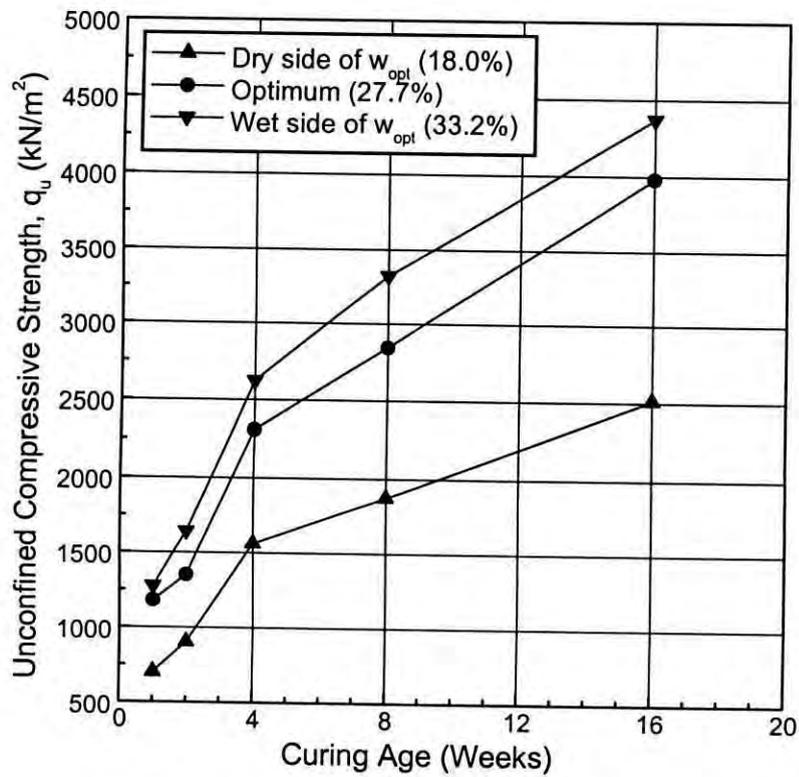


Fig 4.24 Effect of moulding water content on q_u for 12% lime treated expansive soil

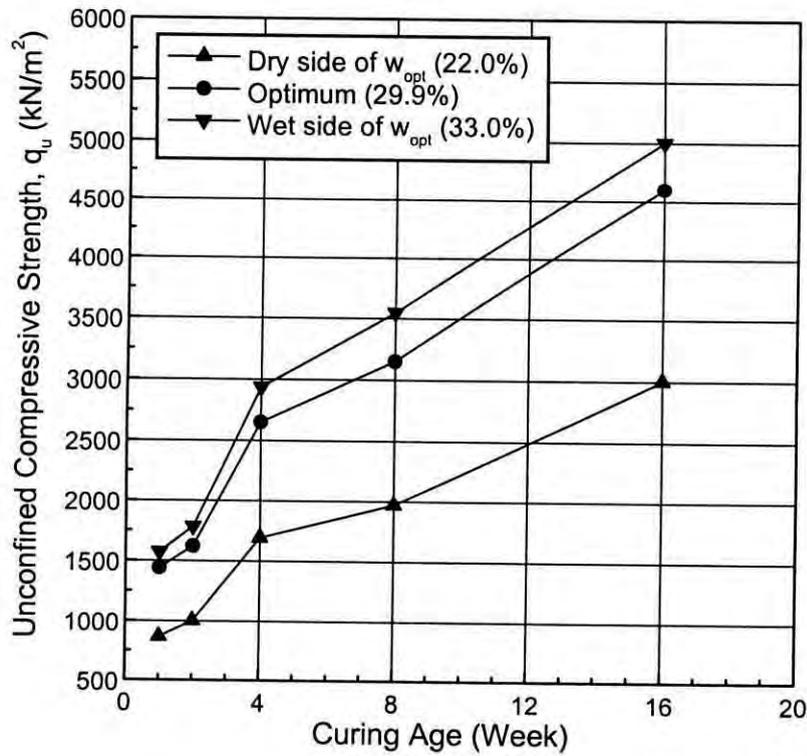


Fig 4.25 Effect of moulding water content on q_u for 15% lime treated expansive soil

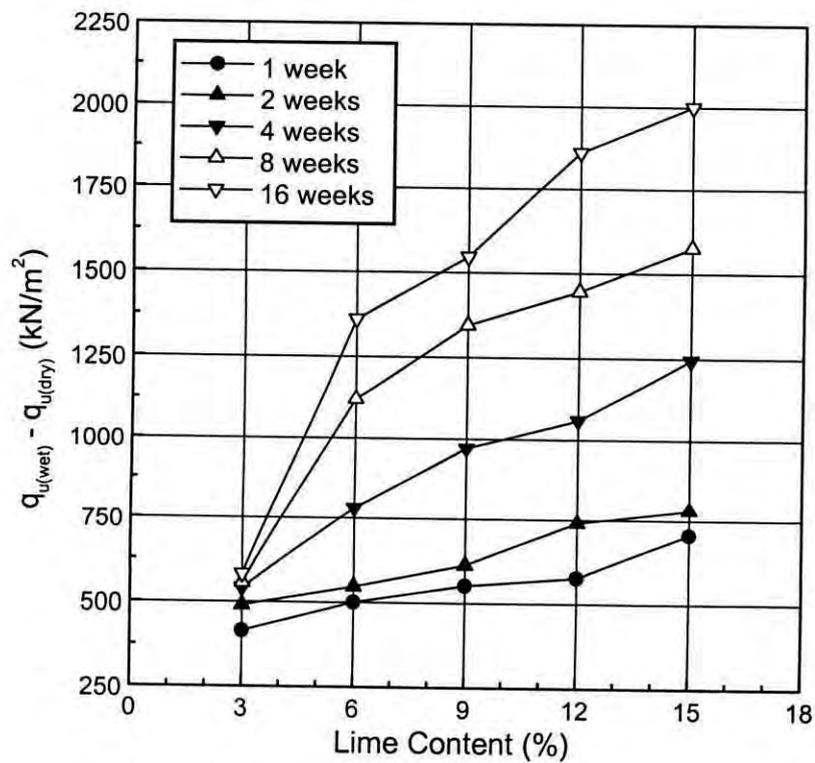


Fig. 4.26 Effect of lime content on change in q_u between wet side and dry side moulding water content

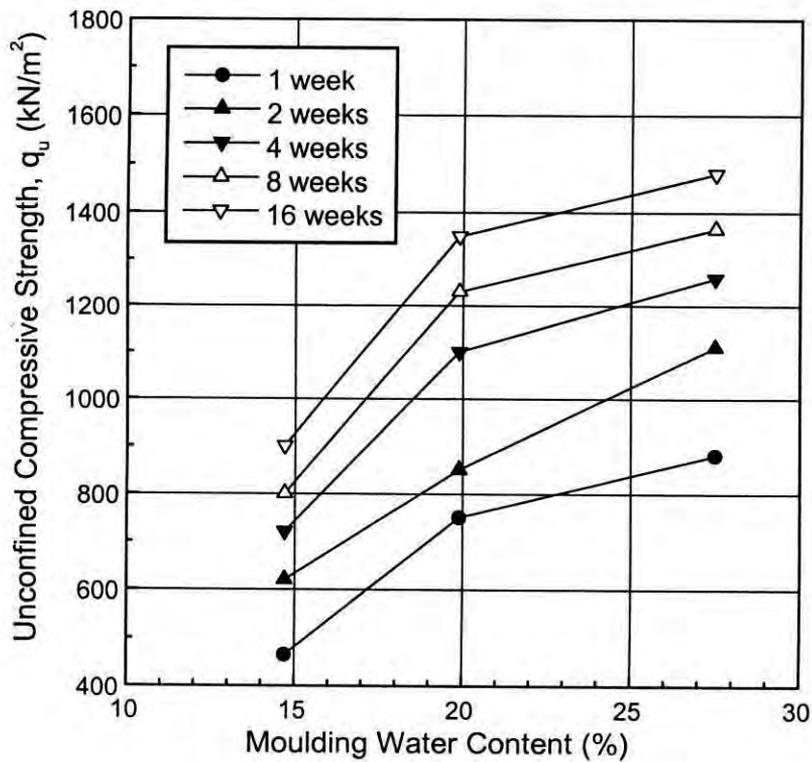


Fig. 4.27 q_u Versus moulding water content curves for 3% lime treated soil

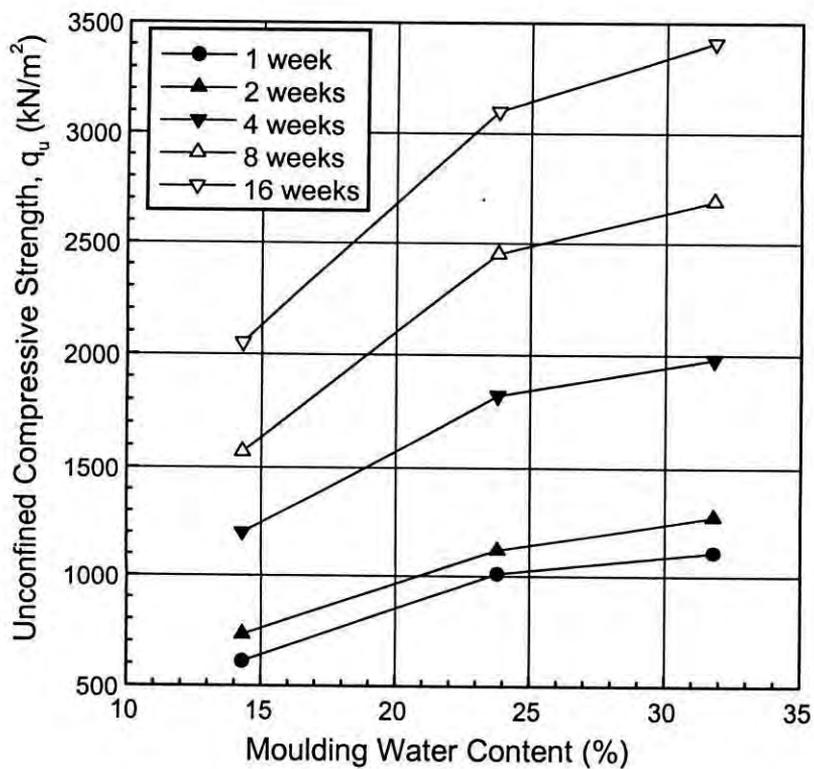


Fig. 4.28 q_u Versus moulding water content curves for 6% lime treated soil

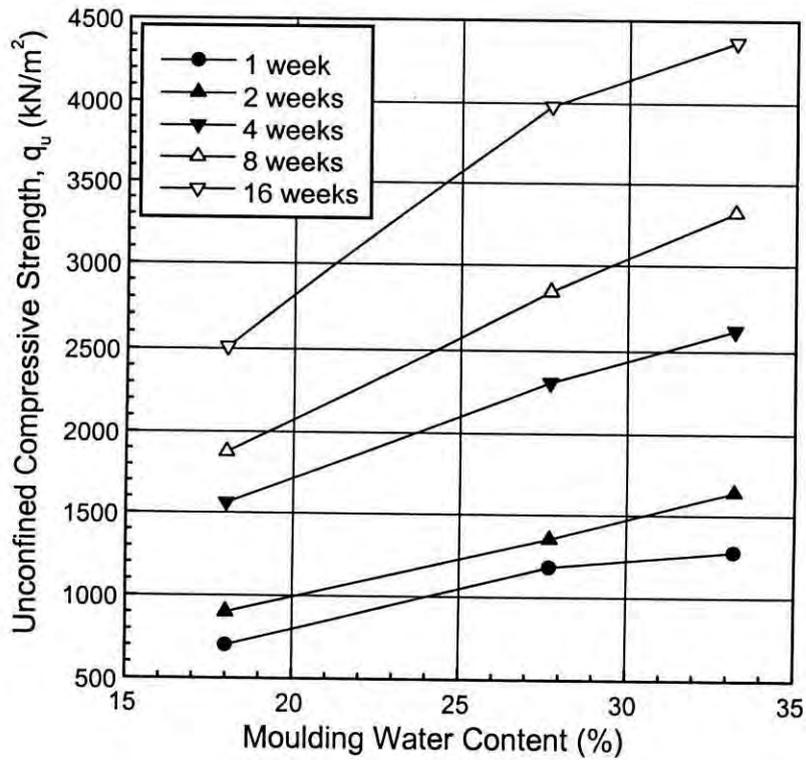


Fig. 4.29 q_u Versus moulding water content curves for 12% lime treated soil

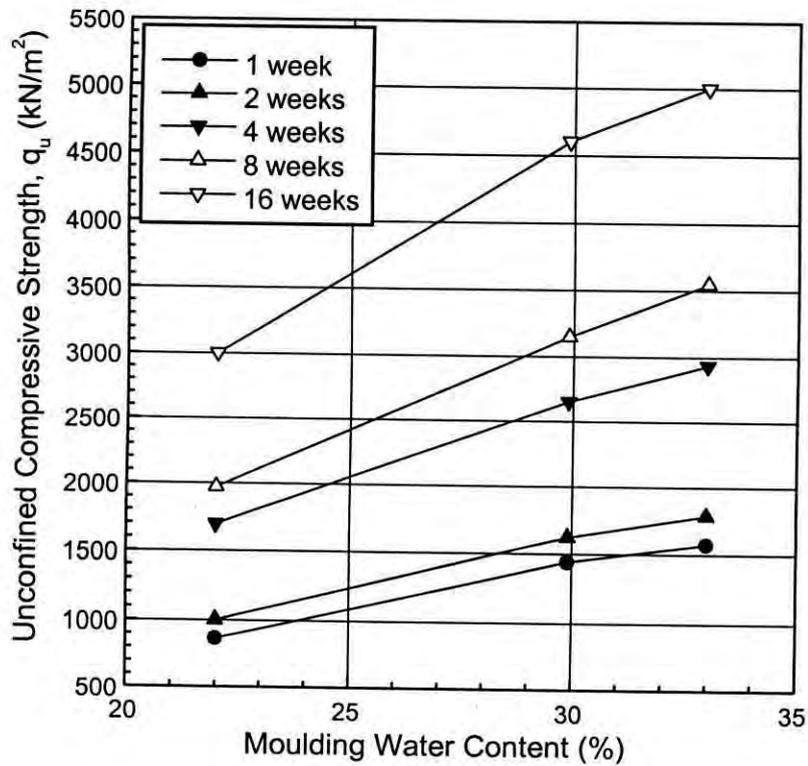


Fig. 4.30 q_u Versus moulding water content curves for 15% lime treated soil

4.9 EFFECT OF LIME STABILISATION ON CALIFORNIA BEARING RATIO (CBR)

A summary of the CBR test result for samples of the expansive soil is presented in Table 4.10. In order to investigate CBR-dry density relationship for untreated and stabilised samples, CBR tests were performed on samples compacted using three levels of compaction energies, e.g., low compaction energy (10,000ft-1b/ft³), medium compaction energy (25,000 ft-1b/ft³) and high compaction energy (56,000ft -1b/ft³). It can be seen from Table 4.10 that compared with the untreated sample, CBR-values of the treated samples at all levels of compaction increased considerably. The variation of CBR with lime content is shown in Fig 4.31, while Fig 4.32 presents the CBR versus compaction energy plots for the same samples. It can be seen from Fig 4.31 that at all levels of compaction, CBR increases markedly with increasing lime content while Fig 4.32 shows that at any particular lime content, CBR increases significantly with the increase in compaction energy. It has been found that compared with the untreated sample, CBR-values of treated samples increased by 4.75 to 8.75 times due to increase in lime content from 3% to 15%. Molla (1997) and Rajbongshi (1997) also investigated the effect of lime on CBR-values of three regional soils and a coastal soil of Bangladesh, respectively. It was found that CBR value of stabilised samples increased with increasing lime content. Rajbongshi (1997) also reported that at any particular lime content, CBR increased significantly with the increase in compaction energy. TRB (1987) reported the effect of lime treatment on CBR-values for three plastic clays (LL=35 to 59, P₁=15 to 30) and showed that for all the soils CBR increases markedly with increasing lime content. Table 4.10 also shows that compared with the untreated sample, the amount of swell of the stabilised samples reduced. The percentage swell of the treated samples are insignificant.

Ingles and Metcalf (1972) recommended that for improvement of base material in road construction the minimum CBR-values of soil lime mix should be 80, It can be seen from Table 4.10 that CBR values of samples of the expansive soil treated with maximum 15% lime and compacted at high energy is 70 which does not fulfill the criteria proposed by Ingles and Metcalf (1972). Therefore, a slightly higher lime content may be required to achieve higher CBR values in order to fulfill the above criteria suggested by Ingles and Metcalf (1972).

Table 4.10 Summary of CBR test results of untreated and lime treated samples of expansive soil

Lime Content	Compaction Energy	Dry Density (kN/m ³)	CBR	Swell (%)
0	Low	15.70	4	2.0
	Medium	16.05	5	1.70
	High	17.08	8	1.40
3	Low	13.72	26	0.7
	Medium	14.95	29	0.5
	High	15.50	38	0.3
6	Low	13.50	30	0.3
	Medium	14.60	32	0.2
	High	15.30	41	0.1
9	Low	13.0	35	0.05
	Medium	14.15	37	0.02
	High	14.65	46	0.01
12	Low	12.60	41	0.04
	Medium	13.95	50	0.01
	High	14.55	60	0.01
15	Low	12.30	52	0.03
	Medium	13.50	58	0.02
	High	14.0	70	0.01

Note: Low compaction energy = 10,000 ft - lb/ft³

Medium compaction energy = 25,000 ft - lb/ft³

High compaction energy = 56,000ft - lb/ft³

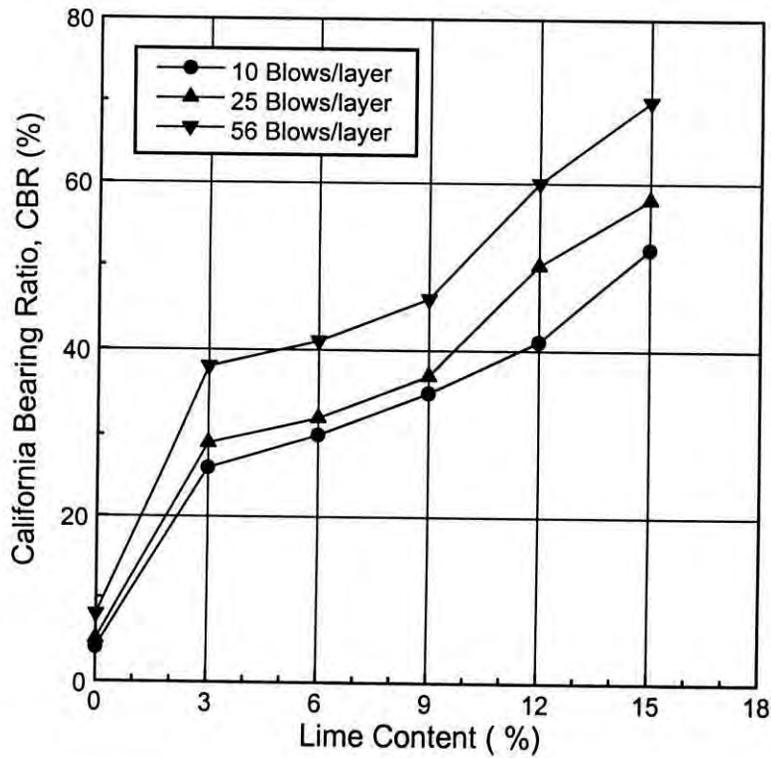


Fig 4.31 Effect of lime content on CBR values of lime treated expansive soil

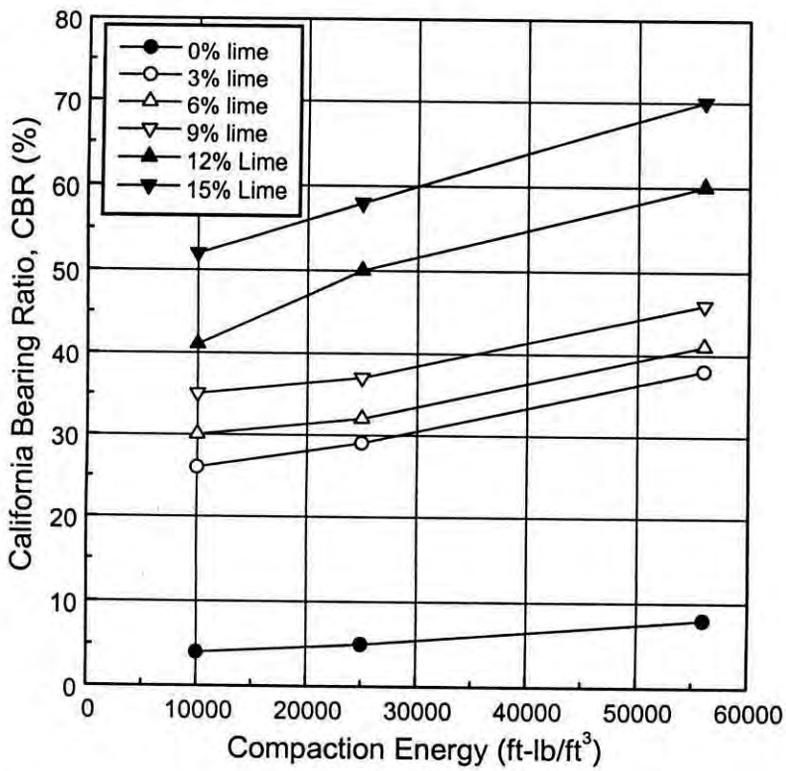


Fig 4.32 CBR versus compaction energy curves of lime treated expansive soil

4.10 EFFECT OF LIME STABILISATION ON FLEXURAL STRENGTH AND FLEXURAL MODULUS

The flexural properties of untreated and stabilised samples of the expansive soil have been investigated by carrying out flexural strength test using simple beam test with third point loading. Typical flexural stress versus deflection curves for different lime stabilised samples are presented in Appendix-C. It can be seen from Figs. C-1 to C-5 that flexural stress versus deflection curves are approximately linear. Similar trend between flexural stress and deflection has also been found by Rajbongshi (1997) for lime stabilised samples of a coastal soil. From the flexural stress and deflection data flexural strength and flexural modulus were determined. The flexural properties of lime stabilised samples are presented in Table 4.11. It can be seen from Table 4.13 that compared with the untreated sample, the flexural strength and flexural modulus of the treated samples cured for 1 week, 2 weeks, 4 weeks and 8 weeks increased significantly. It has been found that compared with the untreated sample, the flexural strength and flexural modulus of samples of the expansive soil cured at 8 weeks increased by about 66% to 209% and 37% to 76%, respectively, due to stabilisation with 3% to 15% lime content. The maximum deflection and failure strain of untreated and stabilised soil-lime beams were in the range of 0.288 mm to 0.550 mm and 0.405% to 0.711%, respectively.

The effect of lime content on flexural strength are shown in Fig 4.33 while Fig 4.34 presents the effect of curing age on flexural strength for same samples. Fig 4.33 shows that at any particular curing age, flexural strength increases with the increase in lime content while Fig 4.34 shows that at any lime content, that flexural strength increases with increasing curing age.

The effect of lime content on flexural modulus is shown in Fig. 4.35 while Fig. 4.36 presents the effect of curing age on flexural modulus for same samples. It can be seen from Fig. 4.35 that at any particular curing age, flexural modulus increases with the increase in lime content while Fig 4.36 shows that at any lime content, that flexural modulus increases with increasing curing age.

Table 4.11 Summary of flexural properties of untreated and lime treated samples of expansive soil

Lime Content (%)	Curing Age (week)	Flexural Strength (kN/m²)	Maximum Deflection (mm)	Flexural Modulus (MPa)	Failure Strain (%)
0	-	97.19	0.304	46.0	0.405
3	1	113.40	0.287	54.32	0.405
	2	129.60	0.288	57.30	0.440
	4	145.00	0.290	61.0	0.465
	8	161.0	0.30	63.0	0.499
6	1	162.0	0.390	58.0	0.534
	2	178.0	0.41	60.0	0.575
	4	211.0	0.42	63.0	0.645
	8	227.0	0.430	66.8	0.660
9	1	194.0	0.421	65.7	0.576
	2	210.0	0.431	67.0	0.610
	4	243.0	0.441	69.0	0.677
	8	259.0	0.452	72.6	0.694
12	1	226.7	0.430	71.0	0.576
	2	243.0	0.46	73.5	0.640
	4	275.0	0.511	75.5	0.711
	8	285.0	0.523	77.0	0.711
15	1	243.0	0.480	73.50	0.63
	2	259.0	0.495	76.0	0.66
	4	285.0	0.535	78.0	0.703
	8	300.0	0.550	81.0	0.694

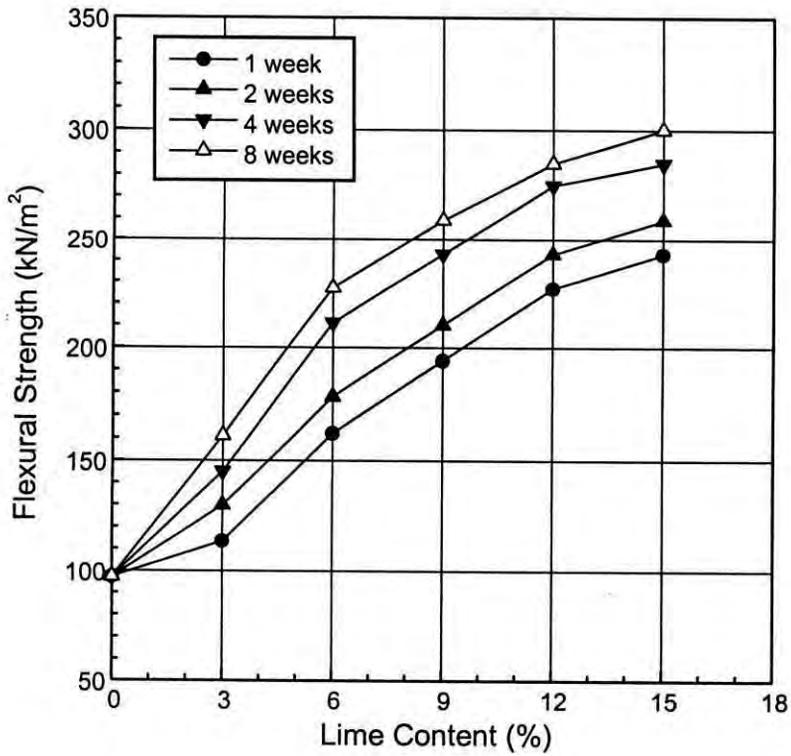


Fig. 4.33 Effect of lime content on flexural strength of lime treated expansive soil

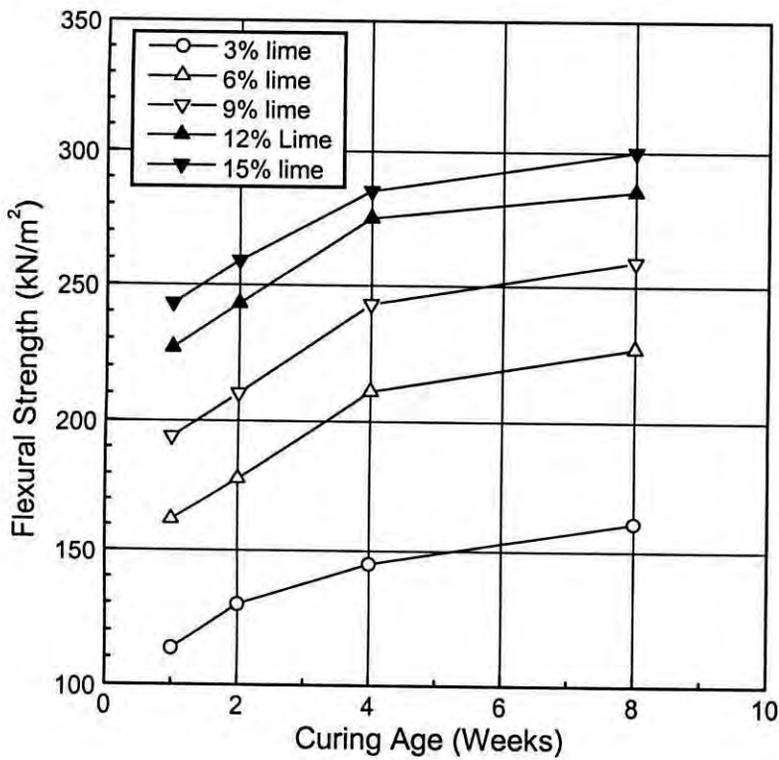


Fig. 4.34 Effect of curing age on flexural Strength of lime treated expansive soil

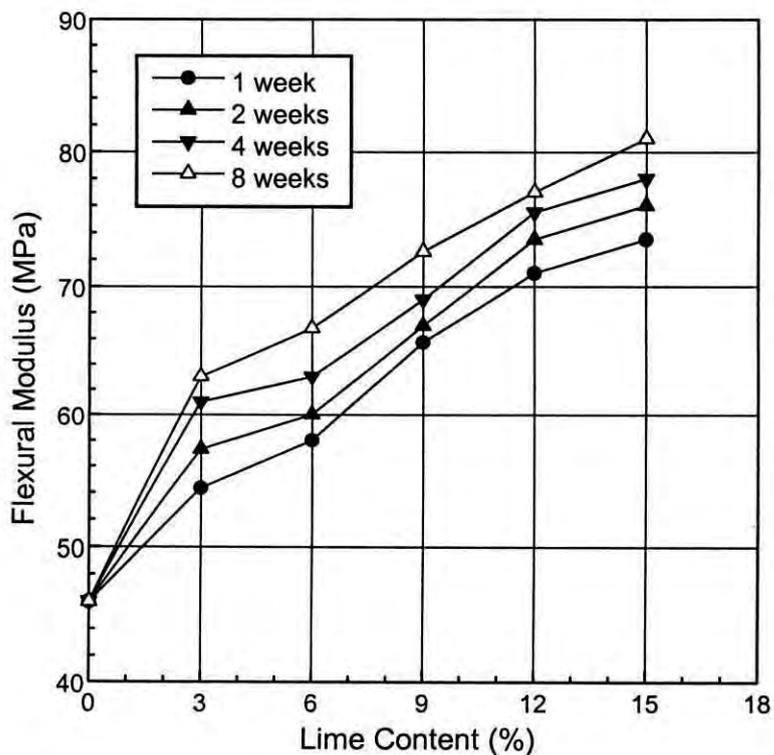


Fig. 4.35 Effect of lime content on flexural modulus of lime treated expansive soil

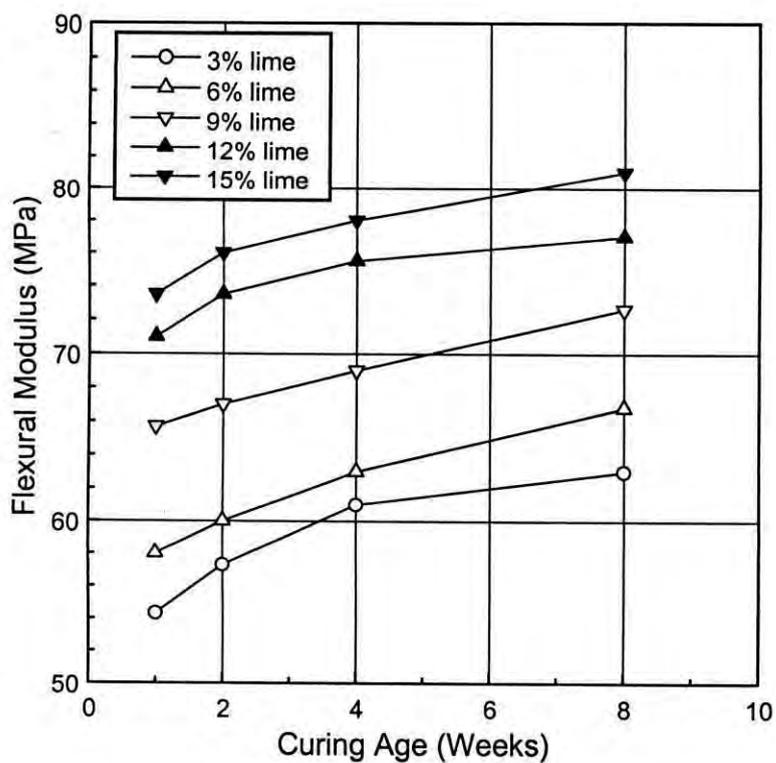


Fig. 4.36 Effect of curing age on flexural modulus of lime treated expansive soil

The flexural properties of untreated and lime stabilised samples of a coastal soil have been investigated by Rajbongshi (1997). It has been found that compared with the untreated sample, flexural strength and modulus of the treated samples cured at 7 and 28 days increased significantly.

It is also evident from Figs. 4.34 and 4.36 that at any particular lime content, the rate of increase in flexural strength and flexural modulus with curing time is prominent for curing ages up to 4 weeks. Curing age has got insignificant effect on increase in flexural strength and modulus when samples have been cured beyond 4 weeks. Rajbongshi (1997) also found insignificant effect of curing age on the increase in flexural strength and flexural modulus of lime treated samples of a coastal soil.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDY

5.1 CONCLUSIONS

This study has been aimed to investigate the geotechnical properties of a lime stabilised expansive soil. Initially, index and swelling properties of untreated samples collected from six different locations of Rajendrapur Cantonment, Gazipur were determined. By comparing the laboratory measured values of index and swelling properties of the six samples with the various recommended criteria outlined in Article 2.6 of Chapter 2, the relative degree of expansion of the six soil samples has been evaluated in order to select the most expansive soil used for subsequent lime stabilisation. On the basis of the recommended criteria, it has been found that the overall degree of expansion of Soil-A, collected from POL store area of Rajendrapur Cantonment, Gazipur is high while the overall degree of expansion of Soil-D, collected from Main & RD office area of Rajendrapur Cantonment, Gazipur is medium. The overall degree of expansion of the rest four soils, i.e., Soil-B (collected from Air condition store office area of Rajendrapur Cantonment, Gazipur), Soil-C (collected from AMO Store area of Rajendrapur Cantonment, Gazipur), Soil-E (collected from COD store area of Rajendrapur Cantonment, Gazipur) and Soil-F (collected from married ORS quarter area of Rajendrapur Cantonment, Gazipur) has been found to be low to medium. Therefore, Soil-A was selected for lime stabilisation. Samples of this soil were stabilised with five different lime contents of 3%, 6%, 9%, 12% and 15%. Finally, the physical and engineering properties of the lime stabilised expansive soil have been evaluated in order to study the influence of lime stabilisation on geotechnical properties of this expansive soil.

The major findings and conclusions drawn from the experimental investigations of lime stabilisation on the selected expansive soil may be summarized as follows:

- Compared with the untreated sample, liquid limit of the stabilised samples initially decreased with the increment of lime content up to 6% and then increased with increasing lime content.

- Compared with the untreated sample, plastic limit of the treated samples increased drastically with increasing lime content. Plastic limits of samples have been found to increase by 1.8 to 3 times due to increase in lime content from 3% to 15%.
- Compared with the untreated sample, plasticity index of samples of lime treated expansive soil decreased markedly due to increase in lime content from 3% to 15%. Plasticity indices of the lime treated samples were found to reduce by 30% to 56%.
- Linear shrinkage of the stabilised samples decreased significantly with the increase in lime content while shrinkage limit increased drastically with increasing lime content. Compared with the untreated sample, linear shrinkage was found to reduce by 35% to 75% while shrinkage limit has been found to increase by 2 to 3.36 times.
- Volumetric shrinkage and shrinkage ratio of samples of the treated soil decreased considerably with increasing lime content. Compared with the untreated sample, volumetric shrinkage and shrinkage ratio were found to reduce by 48% to 82% and 31% to 46%, respectively.
- Free swell and free swell index of the stabilised samples decreased significantly with increasing lime content. Free swell and free swell index were found to reduce by 17% to 87% and 38% to 95%, respectively, due to stabilisation with 3% to 15% lime.
- Swelling pressure and swelling potential of the treated samples reduced markedly. Swelling pressure and swelling potential become zero when the soil has been stabilised with 9% and 12% lime.
- Compared with the untreated sample, volume change of the stabilised samples from air-dry to saturated condition decreased considerably. Volume change of the stabilised samples from air-dry to saturated condition were found to decrease by 55% to 83% due to increase in lime content from 3% to 15%.
- The experimental results on the influence of lime stabilisation on swelling properties of the expansive soil clearly demonstrate that lime could be considered a very effective additive in reducing the various swelling properties of an expansive soil.
- It has been found that with the increase in lime content, values of maximum dry density (γ_{dmax}) reduced while the values of optimum moisture content (w_{opt}) increased. Compared with the untreated sample, the values of γ_{dmax} reduced by 10%

to 17% while the values of w_{opt} have been found to increase by 10% to 65%, due to increase in lime content from 3% to 15%.

- Compared with the untreated sample, the values of unconfined compressive strength (q_u) of the treated samples increased significantly, depending on the lime content and curing age. It was found that the values of q_u of treated samples cured at any particular age increased with increasing lime content while the values of q_u of samples treated with a particular lime content increased with the increase in curing age. It has been found that unconfined compressive strength of samples of this expansive soil treated with 6%, 9%, 12% and 15% lime fulfilled the requirements for its use for upgrading heavy clays to sub-base material quality type, as proposed by Ingles and Metcalf (1972).
- The rate of strength gain with curing time has been evaluated in terms of the parameter termed strength development index (SDI) as proposed by Uddin (1990). It was found that the values of SDI increases with increasing curing age and increasing lime content. It has been observed from the present investigation that long-term curing (up to 16 weeks) has marked influence on the gain in strength. It has been found that at any particular lime content, unconfined compressive strength continued to increase with the increase in curing age. The effect of long-term curing on the increase in unconfined compressive strength has been found to be more pronounced when samples were stabilised with higher lime contents (more than 3%). It was found that the value of q_u of sample treated with 15% lime and cured for 16 weeks was about 8.4 times higher than the strength of the untreated sample.
- Comparisons of the values of q_u of samples stabilised with different lime contents and cured at different ages for the three moulding water contents showed that irrespective of curing age and lime content, values of q_u is maximum and minimum at moulding moisture contents of wet side of optimum and dry side of optimum, respectively. It therefore appears that in order to achieve adequate compressive strength, lime stabilised samples should be compacted at the wet side of their optimum moisture contents.
- The relative increase in q_u -value [i.e., $q_{u(wet)} - q_{u(dry)}$], when compacted using moulding water contents equal to wet side and dry side of optimum moisture content, has been found to depend on the lime content and curing age. It was found that the increase in q_u -value increases with increasing lime content and curing age.

- Compared with the untreated sample, CBR-values of the treated samples at all levels of compaction increased considerably with increasing lime content. It was also found that at any particular lime content, CBR increased significantly with the increase in compaction energy. It has been found that compared with the untreated sample, CBR-values of treated samples increased by 4.75 to 8.75 times due to increase in lime content from 3% to 15%. A CBR value of 70 obtained for the sample of the soil stabilised with 15% lime, however, did not fulfil the criteria of the minimum CBR-value of 80 for soil-lime mix for improvement of base material in road construction as proposed by Ingles and Metcalf (1972). Therefore, a slightly higher lime content may be required to achieve higher CBR values for use as base material for roads.
- The flexural stress versus deflection curves has been found to be approximately linear. The maximum deflection and failure strain of untreated and stabilised soil-lime beams were in the range of 0.288 mm to 0.550 mm and 0.405% to 0.711%, respectively.
- Compared with the untreated sample, flexural strength and flexural modulus of the stabilised samples increased significantly, depending on the lime content and curing age. It has been found that the flexural strength and flexural modulus of samples of the expansive soil cured for 8 weeks increased by about 66% to 209% and 37% to 76%, respectively, due to stabilisation with 3% to 15% lime content. It was observed that although at any particular lime content, unconfined compressive strength continued to increase with curing age up to 16 weeks, the rate of increase in flexural strength and flexural modulus with curing time, however, is prominent for curing ages up to 4 weeks only.

5.2 RECOMMENDATIONS FOR FUTURE STUDY

Several aspects of the work presented in this thesis require further study. Some of the important areas of future research could be as follows:

- (i) The present study was carried out on a expansive sample collected from Rajendrapur Cantonment, Gazipur. Similar investigations may be carried out with expansive soils collected from other regions of Bangladesh, e.g., Barind Tract of Rajshahi, Lalmai Hill areas of Comilla, Sreepur, Gazipur. The results may be

compared with those obtained in the present investigations. Such a study would also be helpful in preparing a database of the generalised behaviour of lime stabilised expansive soils of Bangladesh.

- (ii) In this investigation, lime has been used as additive for stabilisation. Investigations on the physical and engineering properties could be carried out by stabilising this expansive soil with other additives, e.g., fly-ash or lime plus fly-ash to assess their suitability.
- (iii) Further research could be carried out to study the effect of other state variables, e.g., compactive effort, compaction delay time, mixing method on the strength and flexural properties of lime stabilised expansive soil.
- (iv) Using the CBR and flexural properties of the lime stabilised expansive soil, numerical investigations could be carried out for paved soil-lime roads (continuous soil-lime sub-base overlying untreated natural subgrade and underlying unbound granular base with a asphalt wearing surface) subjected to light to heavy traffic. Such a study would be helpful for optimum design of lime stabilised paved roads in the areas where expansive soil occurs.

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APPENDIX-A

**GRAIN SIZE DISTRIBUTION CURVES FOR
SAMPLES OF UNTREATED SOILS**

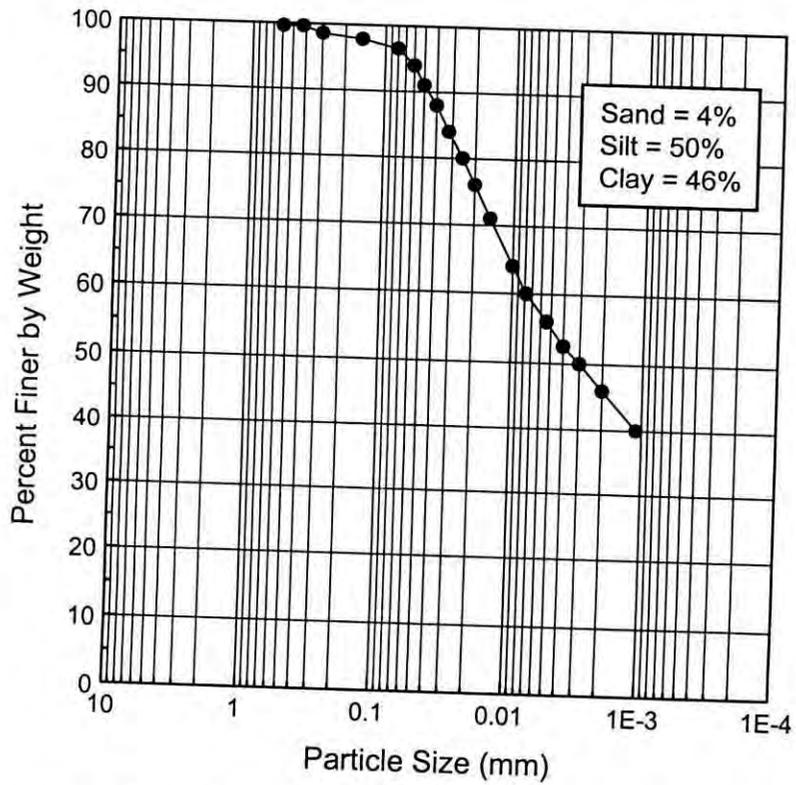


Fig. A-1 Particle size distribution curve of soil-A collected from POL store at Razendrapur cantonment, Gazipur

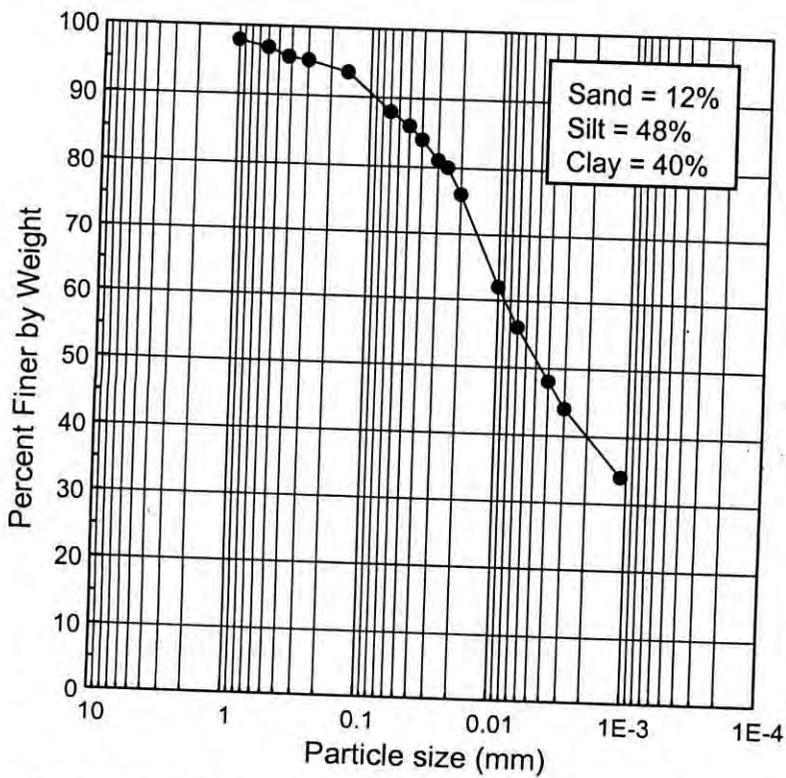


Fig. A-2 Particle size distribution curve of Soil-B collected from Air condition store office at Rajendrapur cantonment, Gazipur

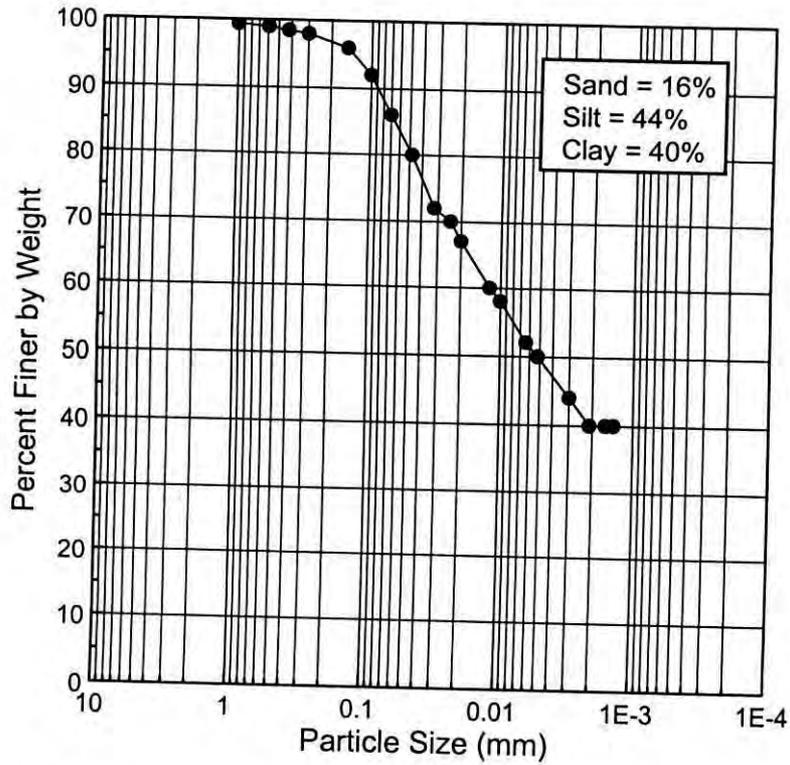


Fig. A-3 Particle size distribution curve of Soil-C collected from AMO store at Razendrapur cantonment, Gazipur

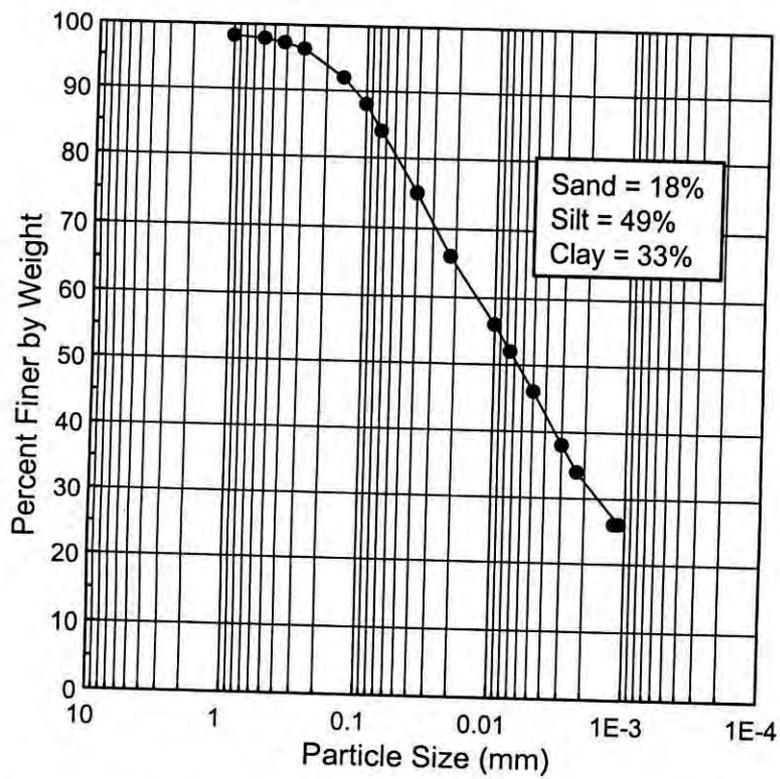


Fig. A-4 Particle size distribution curve of Soil-D collected from Main & RD office at Razendrapur cantonment, Gazipur

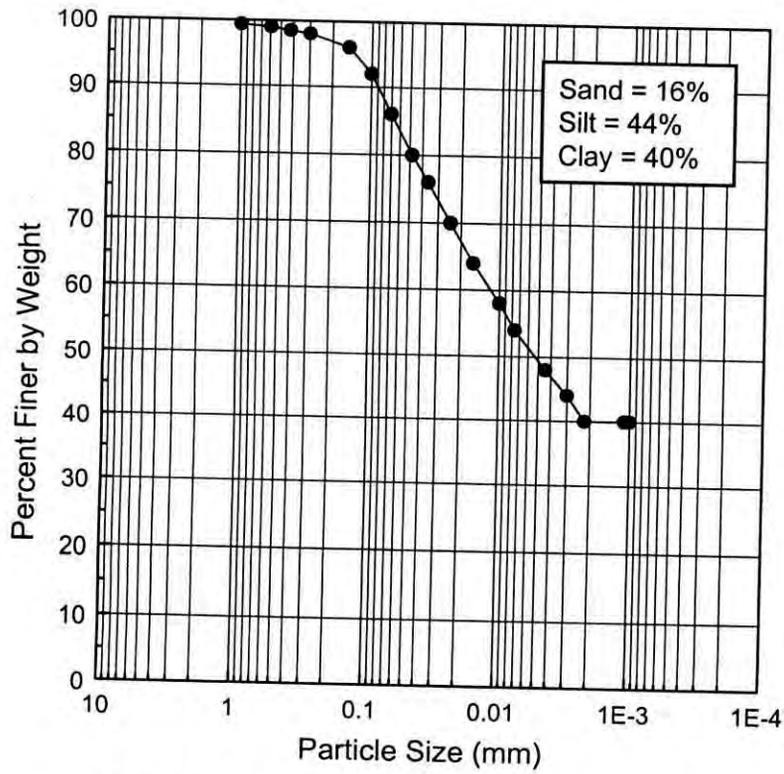


Fig A-5 Particle size distribution curve of Soil-E collected from COD store at Rajendrapur cantonment, Gazipur

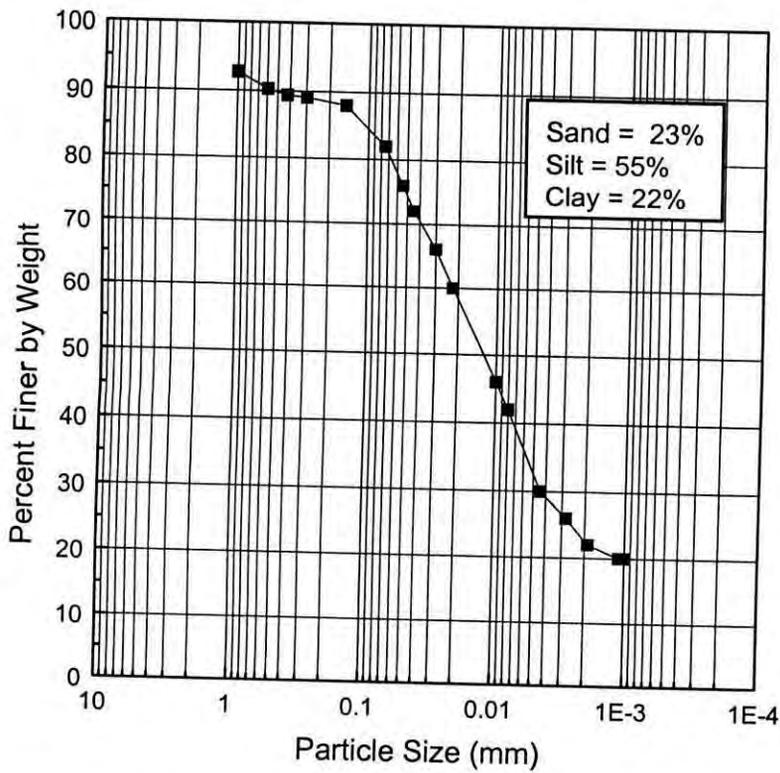


Fig. A-6 Particle size distribution curve of Soil-F collected from ORS QTR area at Rajendrapur cantonment, Gazipur

APPEXDIX-B

**VOLUME CHANGE VERSUS SURCHARGE PRESSURE
CURVES FOR SAMPLES OF UNTREATED SOILS**

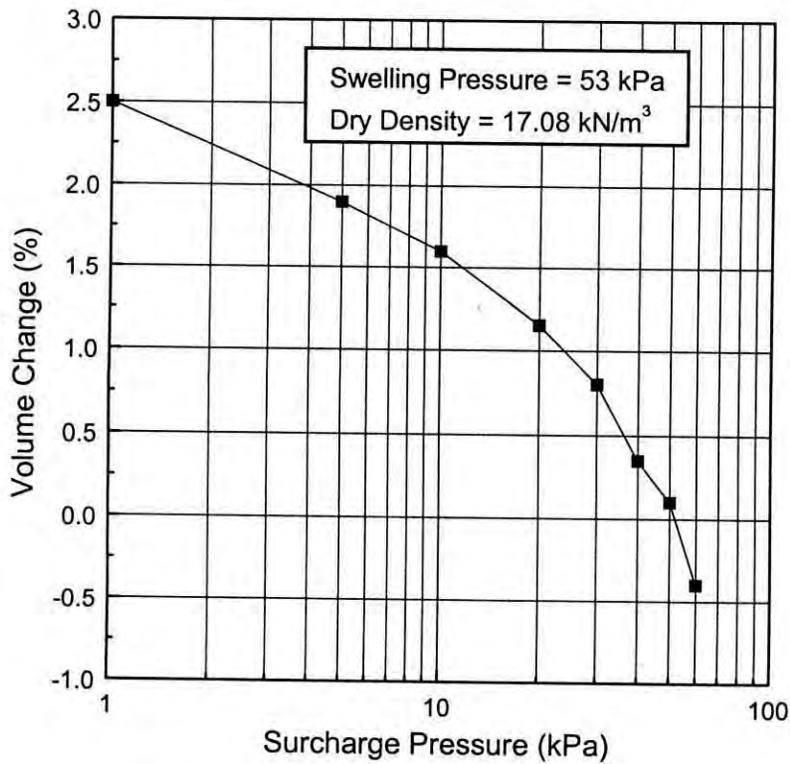


Fig. B-1 Volume change versus surcharge pressure curve of laboratory compacted Soil-A

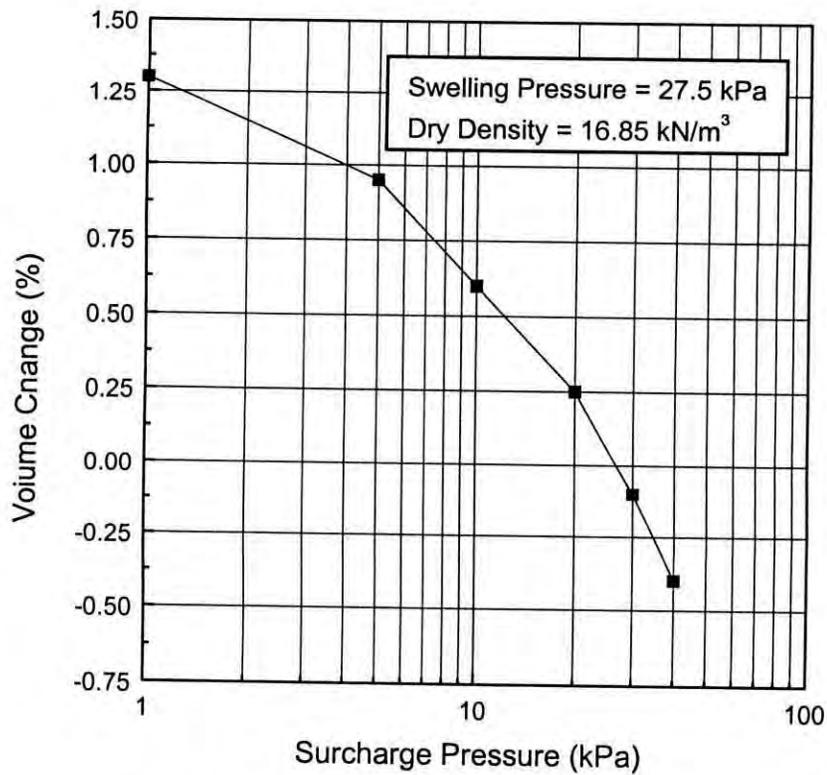


Fig. B-2 Volume change versus surcharge pressure curve of laboratory compacted Soil-B

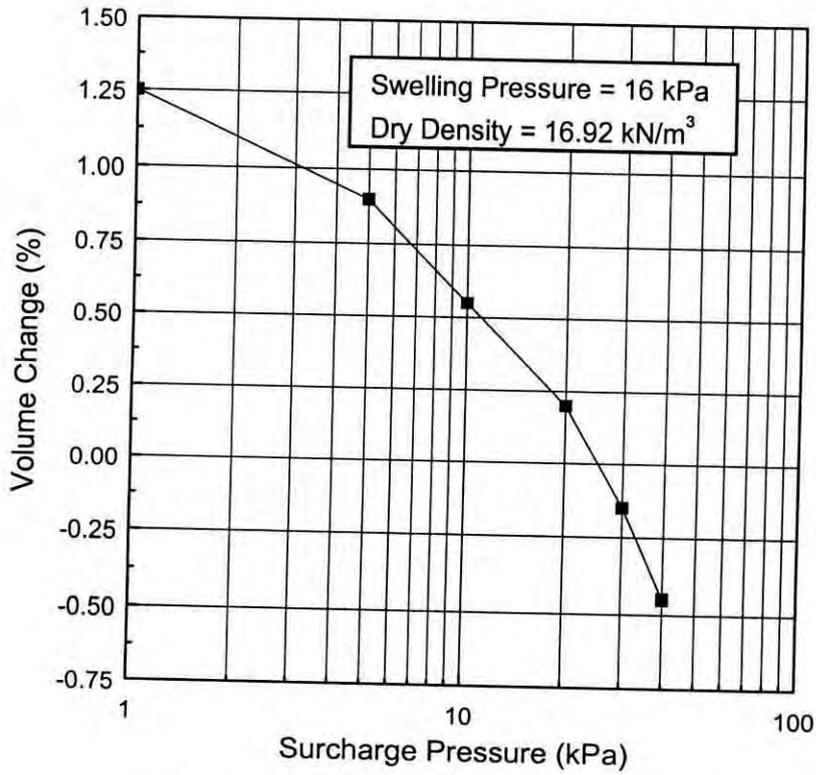


Fig. B-3 Volume change versus surcharge pressure curve of laboratory compacted Soil-C

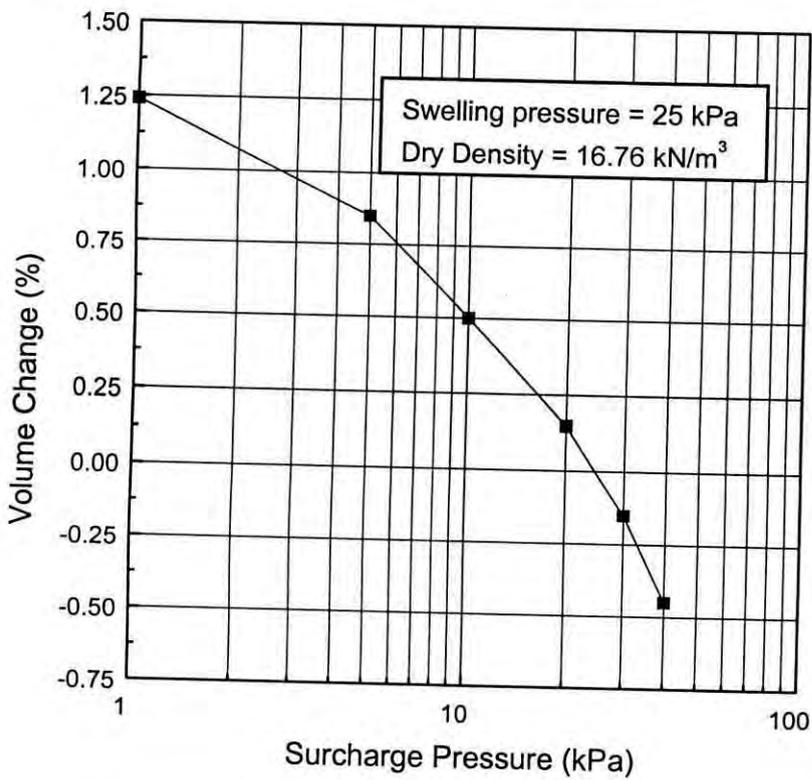


Fig. B-4 Volume change versus surcharge pressure curve of laboratory compacted Soil-D

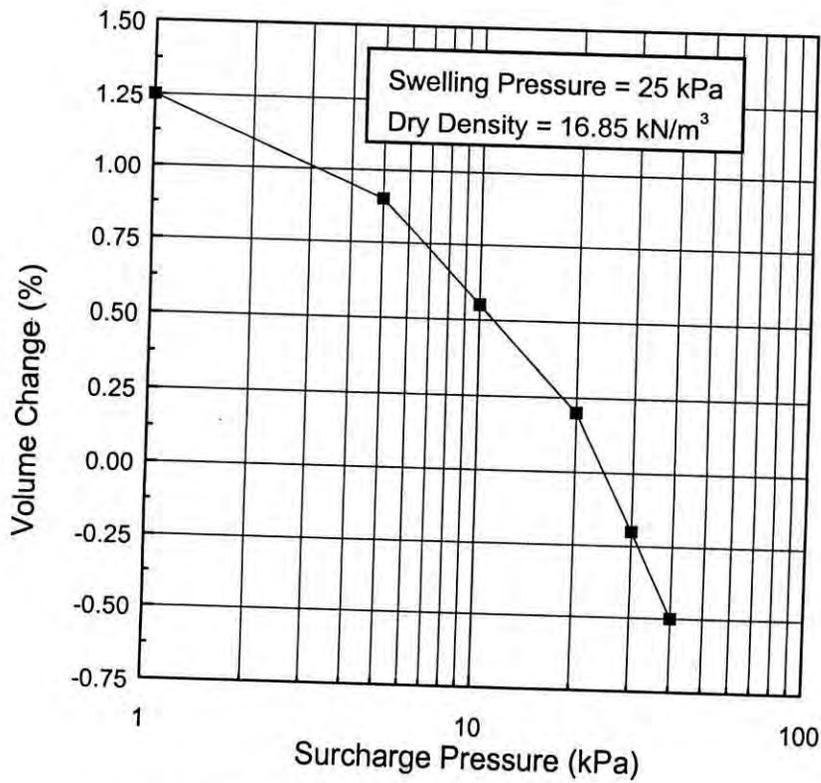


Fig. B-5 Volume change versus surcharge pressure curve of laboratory compacted Soil-E

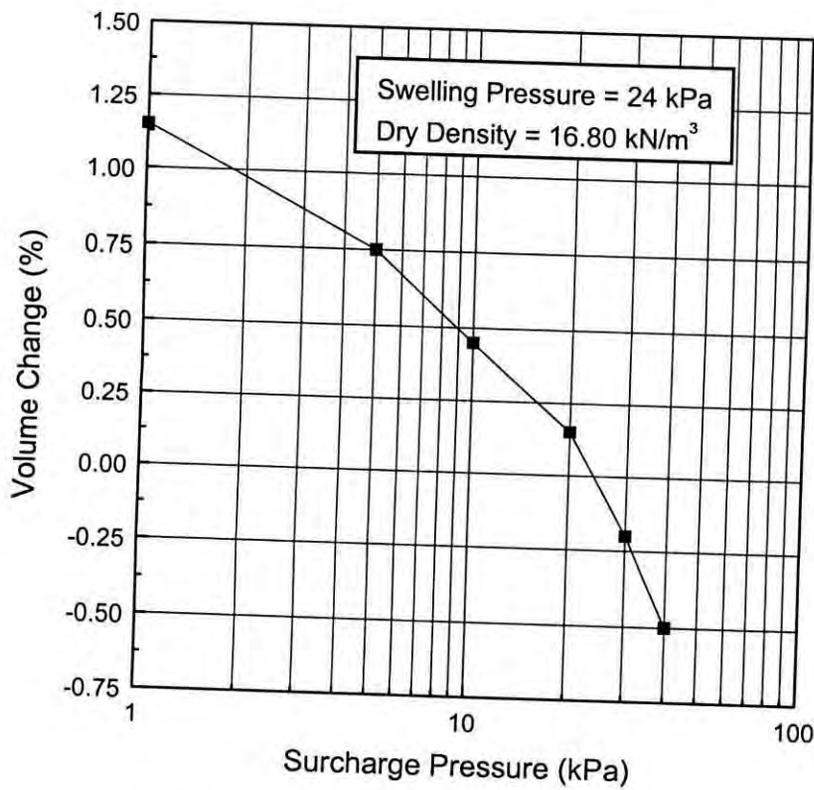


Fig. B-6 Volume change versus surcharge pressure curve of laboratory compacted Soil-F

APPENDIX-C

**FLEXURAL STRESS VERSUS DEFLECTION CURVES
FOR SAMPLES OF LIME TREATED EXPANSIVE SOIL**

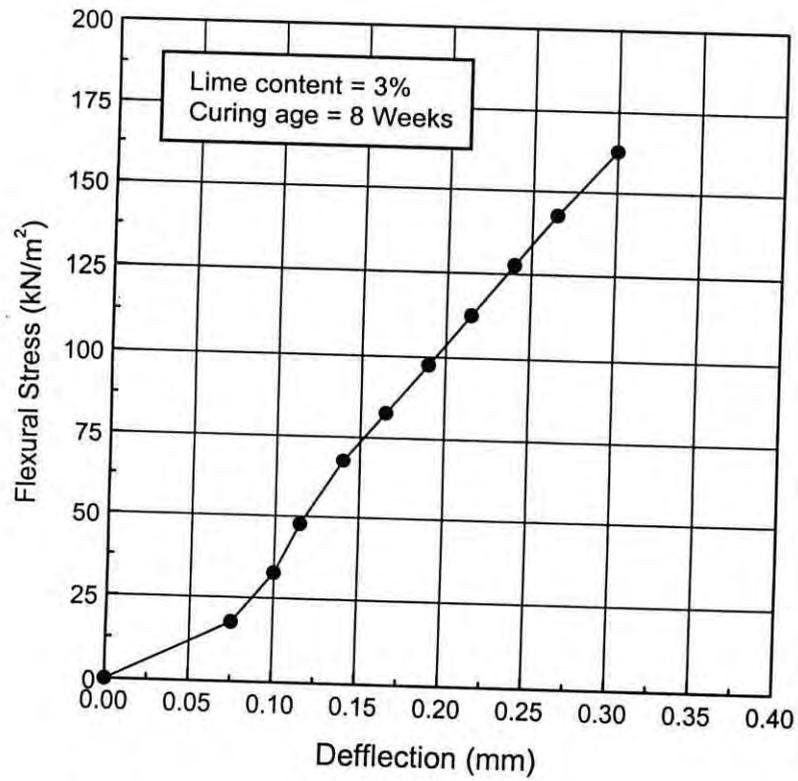


Fig. C-1 Flexural stress versus deflection curve for 3% lime treated expansive soil

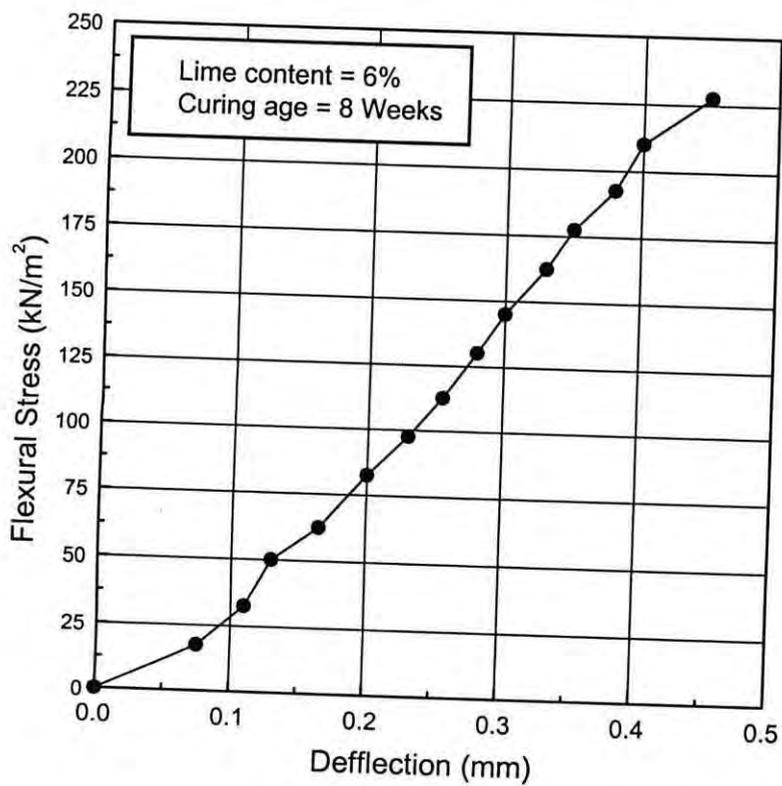


Fig. C-2 Flexural stress versus deflection curve for 6% lime treated expansive soil

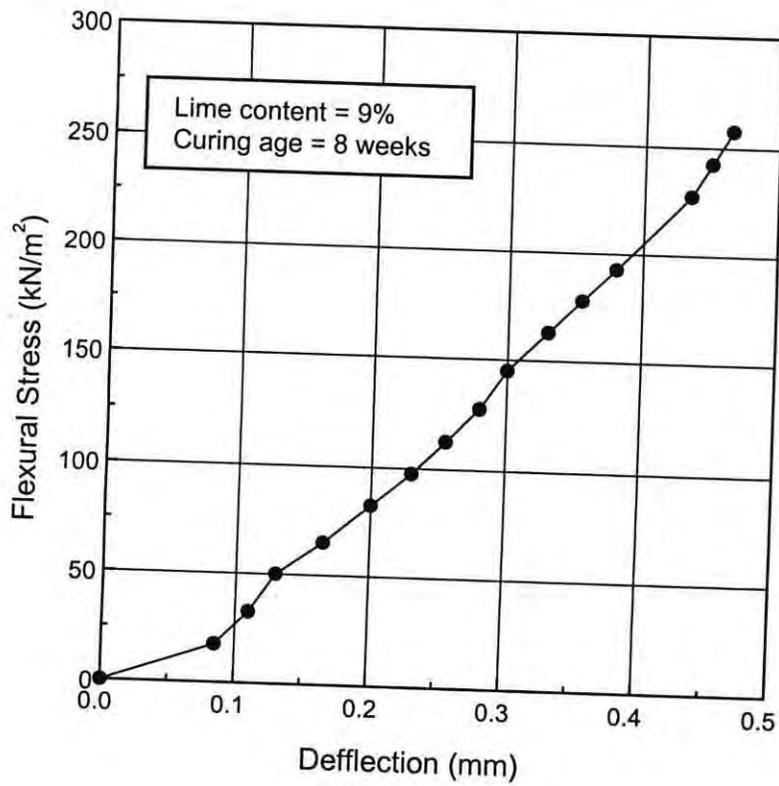


Fig. C-3 Flexural stress versus deflection curve for 9% Lime treated expansive soil

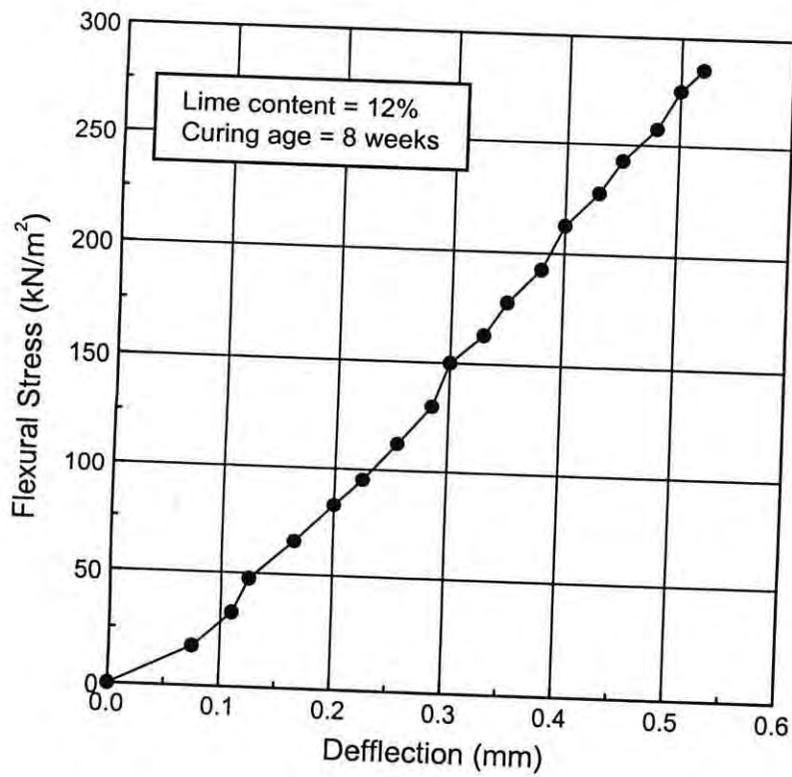


Fig C-4 Flexural stress versus deflection curve for 12% lime treated expansive soil

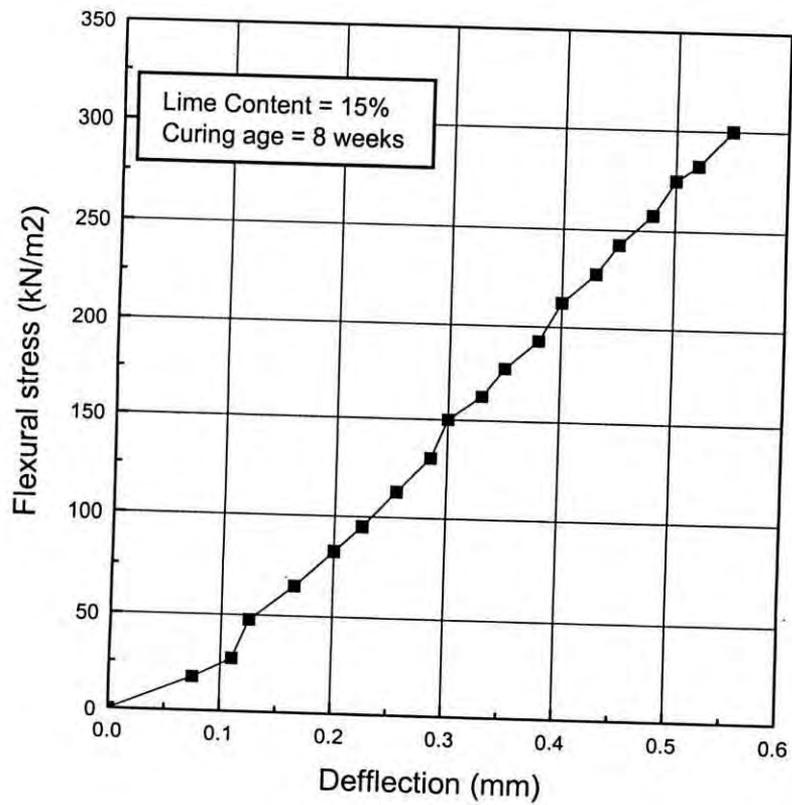


Fig. C-5 Flexural stress versus deflection curve for 15% lime treated expansive soil

