A STUDY ON CEMENT AND LIME STABILIZATION ON SOILS OF SELECTED RECLAIMED SITES OF DHAKA CITY

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Kazi Abid Hasan

Dedicated to my Parents

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NOTATIONS

CBR = California bearing ratio

D = Average width of soil-cement and soil-lime beam sample

E = Flexural modulus

E_h = Horizontal modulus

E_y = Vertical modulus

I = Moment of inertia

L = Span length of sample

LL = Liquid limit

P = Maximum applied load

PI = Plasticity index

PL = Plastic limit

q_u = Unconfined compressive strength

R = Modulus of rupture

SDI = Strength development index

Soil-A = Soil from Aminbazar

Soil-B = Soil from Bashundhara

w_{opt} = Optimum moisture content

Δ = maximum deflection

ε_f = axial strain at failure

γ_{max} = Maximum dry density

ABSTRACT

In the present study cement stabilization of reclaimed soils of two selected sites (Aminbazar and Bashundhara) and lime stabilization of one site (Bashundhara) in Dhaka City were carried out in order to assess their suitability for use in road construction. The soils from Aminbazar and Bashundhara were respectively a clayey silt of low plasticity (LL=41, PI=12) and a silty clay of high plasticity (LL=52, PI=29). As additives, ordinary Portland cement was used in percentage of 1, 3 and 5 for Aminbazar soil and 1, 3, 5 and 7 for Bashundhara soil while slacked lime was used in percentages of 1, 3, 5 and 7 for Bashundhara soil. Comparisons of different soil stabilization of regional soils of Bangladesh were also undertaken.

Index tests indicate that compared with the untreated samples, plasticity index and linear shrinkage of the cement and lime stabilized samples of the soils reduced. Shrinkage limit, however, reduced for cement-treated samples while it increased for lime-treated samples. For the cement and lime stabilized samples, maximum dry density increased and reduced respectively, while optimum moisture content reduced and increased for cement and lime stabilized samples respectively with the increase in additive content.

For samples of both the sites, unconfined compressive strength of cement and lime treated samples increased significantly than the untreated samples, depending on the additive content and curing age. It was found that compressive strength of samples treated with 3% cement and cured for 14 and 28 days satisfied the PCA (1956) for the compressive strength of soil-cement mix and that for all cement contents and all curing ages. Compressive strength of the stabilized samples fulfilled the requirements of soil-cement mix for use in road sub-base and base subject to light traffic, as proposed by Ingles and Metcalf (1972). It was also found that the compressive strength of samples treated with 5% and 7% lime met the requirements of upgrading heavy clays to sub-base materials quality type, proposed by Ingles and Metcalf (1972). In attempt to investigate the effect of molding water content on q_u, it appeared that in order to achieve maximum compressive strength, the cement and lime stabilized samples should be compacted at their optimum and wet side of optimum moisture content respectively. Compared with untreated samples, CBR of the cement and lime stabilized samples increased up to about 5.8 times and 4.1 times respectively. CBR values of samples of both the soils, treated with 3% and 5% cement, fulfilled the requirements of soil-cement road sub-base and base for light traffic while CBR values of samples stabilized with 7% lime did not satisfy the criteria of the minimum CBR for soil-lime mix for improvement of base material in road construction, as proposed by Ingles and Metcalf (1972).

The flexural stress versus deflection curves has been found to be approximately linear for both cement and lime stabilized samples. Compared with the untreated samples, flexural strength and modulus of the cement and lime stabilized samples increased considerably, depending on the additive content, compared with the untreated sample, the flexural strength and modulus of cement –treated samples increased up to 5.5 times and 5.3 times respectively, while for lime treated samples the respective increases were about 2.4 times and 2.6 times respectively. The loss in soil-cement of cement treated samples reduced with the increase in cement content. Although, the cement-treated samples did not meet the PCA (1956)

durability requirements, the samples treated with 3% and 5% cement however, fulfilled the requirements as suggested by Compendium 8 (1979).

It was found from comparison that the values of q_u, CBR, flexural strength and flexural modulus of the cement-treated samples of Bashundhara were significantly higher than those of the lime-treated samples. Moreover, it is executed that compared with soil-lime mix, soil-cement mix would be much more durable in the weather conditions of tropical regions. It could be concluded that the cement stabilization of the reclaimed soils studied would be more suitable than lime stabilization for their use in various purposes.

From previous researchers' findings (between 1984-2001) it has been found that eighteen regional soils of Bangladesh has so far been stabilized with different percent of cement and lime. Among them eleven soils were stabilized with cement and twelve soils were stabilized with lime. In general Unconfined Compressive Strength and CBR value increase with increase of cement (%). The range of Unconfined Compressive Strength is between 51.8 kN/m^2 to 4304 kN/m^2. Also flexural strength and modulus increase with increase of cement (%). The range of flexural strength is between 26.9 kN/m^2 to 286 kPa and flexural modulus varies between 17.3 MPa and 136 MPa. Durability of cement stabilization has been calculated by measuring soil-cement loss, which ranges from 10.6% to 42.7%. Soil-cement loss decreases with the increase of cement (%). In general Unconfined Compressive Strength and CBR values increases with the increases in lime content (%). The range of Unconfined Compressive Strength is between 39.3 kPa to 3452 kPa and CBR between 4 to 70. Three regional soils were investigated to find the flexural strength and modulus. For all the cases flexural strength and modulus increases with increase in lime content (%). The ranges of flexural strength are between 47.3 kPa to 243 kPa and flexural modulus varies from 23.3 MPa to 71.2 MPa.

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

Improvement of mechanical properties of soil may be essential to meet the required soil condition for construction in many areas. Stabilization is one of the most economical and desirable method for improving the strength, durability and resistance to deformation of both in situ and reclaimed soils. Soil stabilization 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 stabilization 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. Stabilization 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. Stabilization 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 stabilization 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 Stabilization for improving the physical and engineering properties. Undoubtedly, the most widely applied methods involve the use of inorganic cementation bonds between the particles in the soil system (Khan, 1989). Full use of the potential of Stabilization 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 Stabilization 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 stabilization, which have been presented in Fig. 1.1.

Table 1.1 Mechanics and applications of Stabilization (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	Cementations inter particle 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 inter particle 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.

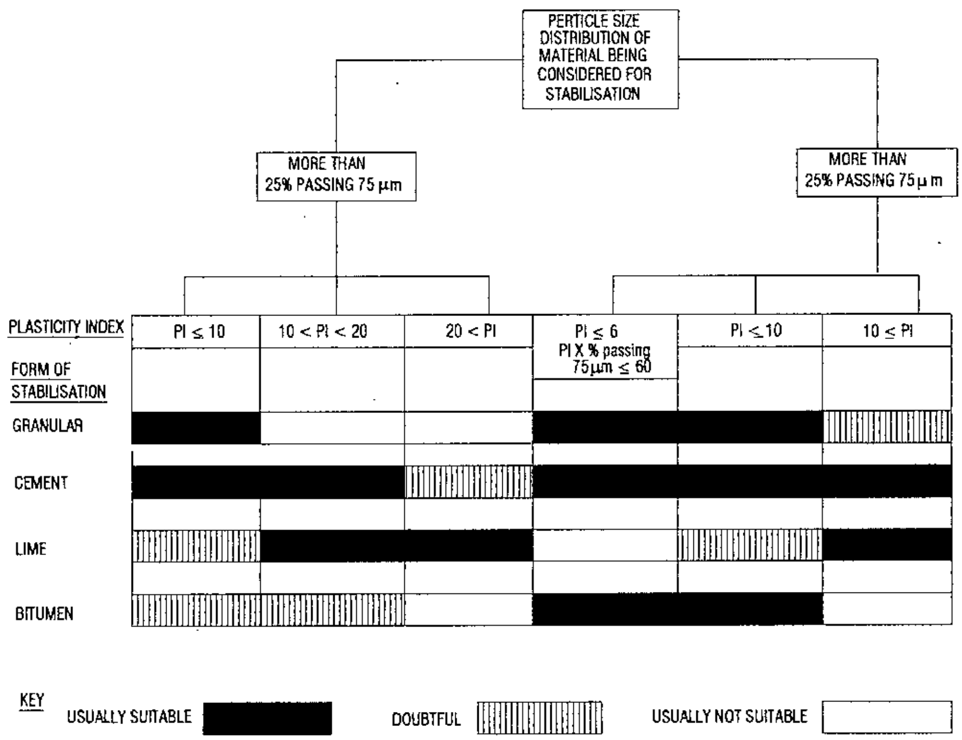

Fig. 1.1 Feasibility of Stabilization techniques for different types of soils (Reproduced after NAASRA, 1986)

Cement and lime stabilization are widely recommended for construction of roads (Ingles and Metcalf, 1972; Mitchell, 1981; IRC, 1976; Kezdi, 1979; Broms and Boman, 1979; NAASRA, 1986; TRB, 1987; Hausmann, 1990; TRL, 1993; Bell, 1993). The major engineering benefits of cement stabilization are increased strength and stiffness, better volume stability and increased durability. The main engineering benefits of lime Stabilization are improved workability, increased strength and volume stability. Cement Stabilization has been used successfully to stabilise granular soil, sands, silts and medium plastic clays. In the case of granular non-plastic soils, cement may bind the particles at points of contact. However, in the case of fine-grained soils, the soil particles on aggregating form cement coated matrix of soil and cement. Lime-stabilization has been successfully used to stabilise clayey soils. The clay minerals carry a negative charge on the surface, which adsorbed cautions of sodium, magnesium, potassium or hydrogen, and to a large extent responsible for imparting plasticity to the soil sample. Lime brings changes in the plasticity properties of a soil; it increases soil contact bonds and therefore the strength.

Stabilization of soil with cement and 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 Stabilization is commonly used for improving remoulded strength, loading capacity of foundations of (roads or embankments) and for erosion control (Macham, Diamond and Leo, 1977). Markus, Gibson, McKinlay and North (1979) recommended that stabilised cement and 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 Stabilization 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 in the Northern district of Bangladesh. A soil-cement trial pavement was also constructed at Dhalapara, Ghatail, Tangail under the Local Government Engineering Department (LGED) on the Ghatail-Sagardighi feeder road on either approach (east and west) of a bridge over River Bangshi. Bangladesh Transport Survey (1974) also recommended the possibility of cement Stabilization for non-plastic alluvial soils of flood plains of Bangladesh for sub-base and base construction of roads.

1.2 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, Compressive strength, Flexural strength and stiffness, Durability) of cement and lime stabilised reclaimed soils from two housing projects in Dhaka. The results of this investigation will enable to assess the suitability of using cement and lime as stabiliser of these reclaimed soils. Moreover, the engineering properties of sub grade and analyses of flexural stress, strain, deflection and stiffness properties using computer aided analytical technique will provide for the optimum design, i.e., thickness of stabilised base/sub-base of roads, improving remoulded strength, loading capacity of foundation, erosion controls etc.

In Bangladesh land development activities have been increased significantly in the recent time. Every year new residential, commercial and recreational areas are being developed by raising low lands. Houses, markets, roads etc. are frequently being constructed on newly reclaimed ground. In the reclamation process, filling soils are generally collected from readily available borrow pits. But the properties of the soils do not always comply with the specified requirements and thus creates problems in the construction phase. Longer monsoon heavy rainfall and flood are other problems in land development works in Bangladesh. To mitigate the problems it is necessary to improve the properties of in-situ soils and filling soils. Cement and lime stabilizations could play a vital role in this process.

In this research work, cement stabilization of two selected soils (collected from Aminbazar and Bashundhara) and lime stabilization of a soil (collected from Bashundhara) of Dhaka reclaimed region have been carried out. Ordinary Portland cement Type-I and hydrated lime (i.e., slaked lime) have been used as additives.

The major objectives of this research are as follows:

(i) To investigate the behaviour and engineering properties (e.g. Moisture-density relations, California Bearing Ratio, Compressive strength, Flexural strength and stiffness, and durability) of two selected soils stabilized with three different cement contents (1%, 3% and 5%) for Aminbazar soil and four different cement content (1%, 3%, 5% and 7%) for Bashundhara soil. The behaviour and engineering properties of these two soils without any treatment were also investigated in order to examine the changes in behaviour and engineering properties between the treated and untreated soils.

(ii) The behaviour and engineering properties (e.g. Moisture-density relations, California Bearing Ratio, Compressive strength, Flexural strength and stiffness) of Bashundhara-soil stabilised with four different lime contents (1%, 3%, 5% and 7%) was investigated. The behaviour and engineering properties of this soil without any additive was also studied.

(iii) To investigate the effect of the curing age on compressive strength and flexural properties of the cement and lime stabilised soils.

(iv) To examine the effect of compactive effort on California Bearing Ratio (CBR) of all the treated and untreated soils.

(v) The effect of moulding water contents on compressive strength for soils (collected from Aminbazar and Bashundhara) treated with particular cement and lime content (3 %) was investigated.

(vi) A study have made on different stabilized regional soils in Bangladesh and the present study soils.

1.3 THE RESEARCH SCHEME

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

(i) Index property tests of the two reclaimed soils without any treatment were carried out to characterise the soils. Index tests include Atterberg limit tests, specific gravity and grain size analysis. Index property tests of the two soils stabilised with different cement and lime contents were also performed.

(ii) The following tests were carried out on the two reclaimed soils without any treatment and stabilised with three different cement contents (1%, 3% and 5%); and one soil stabilised with three different lime contents (1%, 3%, 5% and 7%):

  (a) Modified compaction test

  (b) Unconfined compressive strength test (at curing age of 7, 14 and 28 days) on moulded cylindrical samples of 2.8 inch (71 mm) diameter by 5.6 inch (142 mm) high

  (c) California Bearing Ratio (4 days soaked CBR) test

  (d) Flexural strength test (at curing age of 7 and 28 days) using simple beam with third point loading system

  Unconfined compressive strength tests and flexural strength tests using simple beam with third point loading were carried out on cement and lime stabilised samples cured at 7 days and 28 days 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 stabilised soils, laboratory CBR tests were carried out on the untreated samples and samples treated with cement and lime using three levels of compaction energies. Absorption tests were also carried out on the portions of the cement-stabilized samples used in the flexural strength tests.

(iii) In order to investigate the durability of soil-cement mix, wetting and drying tests on hardened samples of the two reclaimed soils stabilized with 1%, 3% and 5% cement were carried out.

(iv) 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 soil samples of the soils from Aminbazar and Bashundhara treated with 3% cement and soil from Bashundhara treated with 3% lime contents which had been compacted according to the Modified 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 at dry side of the optimum moisture content.

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

(v) finally a study have made on different stabilized regional soil and the present study soils

1.4 THESIS LAYOUT

A review on cement and lime stabilization of fine-grained soils is presented in Chapter 2. The review mainly includes the mechanisms of cement and lime stabilisation, factors governing the properties of cement and lime-treated soils, the characteristics of cement and lime stabilised soils and their applications.

Chapter 3 presents the details of laboratory testing procedures and equipment used for investigating the effects of cement and lime stabilization on the physical and engineering characteristics of the soils studied.

Physical and engineering characteristics of the untreated soils and soils stabilised with different cement and lime contents, as obtained from the laboratory investigation, are presented and discussed in Chapter 4. A study have made on different stabilized regional soil and the present study soils

Chapter 5 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

CEMENT AND LIME STABILIZATION

2.1 GENERAL

For improving volume stability, strength and stress-strain properties, permeability, and durability it needs to stabilize soils. 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 stabilizers with soil is the most important factor affecting the quality of results. Two most commonly used stabilizers for improving the physical and engineering properties of soils are cement and lime.

The improvement of in-situ and reclaimed soils undergone by cement and lime treatment have been more widely employed in the past recent years, especially in stabilization of soils for various applications (Ingles and Metcalf, 1972; Mitchell, 1981; IRC, 1976; Macham et al., 1977; Kezdi, 1979; Broms and Boman, 1979; Markus et al., 1979; NAASRA, 1986; TRB, 1987; Hausmann, 1990; TRL, 1993; Bell, 1993). Stabilization by chemical admixtures such as cement and lime are intended to modify the interactions between cement/lime, water and soil by surface reactio2ns in such a manner as to make the behavior of the soil with respect to water and hardening agent effect most favorable for the given purpose.

This review looks into the fundamental concepts, mechanisms of cement and lime treatment, factors influencing the properties of cement and limes stabilized soils, characteristics of cement-treated and lime-treated soil, and the applications of cement and lime stabilization.

2.2 CEMENT STABILIZATION

The most commonly and successfully used stabilizer for soil stabilization is ordinary Portland cement due to its availability, easy quality control and easy handling characteristics. Soil stabilization with cement is also currently one of the most widely used methods. Portland cement and soil mix of the proper moisture content produce soil-cement, a structural material that is hard and durable. Soil-cement has been used mainly as bases under concrete pavement for highway and airfields. It is also used for construction of rural roads, earth dams and foundation of buildings. Any type of cement may be used for soil stabilization but ordinary Portland cement is most widely used. The two principal factors that determine the suitability of a soil for stabilization with ordinary Portland cement are, firstly, whether the soil and cement can be mixed satisfactorily and, secondly, whether, after mixing and compacting, the soil-cement will harden adequately.

Portland cement and blended cements are effective stabilizing agents applicable to a wide range of soils and situations. Cement has the following two important effects on soil behavior (NAASRA, 1986):

(i) It greatly reduces the moisture susceptibility of soils, giving to stabilized materials enhanced volume and strength stability under variable moisture conditions.

(ii) It can cause the development of inter particle bonds in granular materials, endowing the stabilized material with a useful tensile strength and high elastic modulus.

2.3 MATERIALS FOR CEMENT STABILIZATION

The materials to be considered in cement stabilization are the cement, soil and water. Water, both in quantity and quality, and a number of undesirable materials principally organic matter and sulphate salts are important.

2.3.1 CEMENT

Portland cement is the most commonly used and effective additive for soil stabilization. It has both adhesive and cohesive properties, enabling it to bind mineral fragments into a solid mass, i.e., those that can set and harden in the presence of water and so it is termed as "Hydraulic Cement". These consist primarily of silicates and aluminates of lime, made from limestones and clays (or shales), which are ground, blended and fused in a kiln and crushed to a powder. The usual hydraulic cement used is known as Portland cement. ASTM C150 defined Portland cement as a hydraulic cement produced by pulverizing clinker consisting essentially of hydraulic calcium silicates, usually containing one or more of the forms of calcium sulphate as an underground addition. Type I Portland cement is the most widely used in soil stabilization. Typical composition of ordinary Portland cement is presented in Table 2.1.

Table 2.1 Typical composition of ordinary Portland cement (after Mindess and Young, 1981)

Chemical Name	Chemical Formula	Weight (Per cent)
Tricalcium silicate	$3CaO.SiO_2$	50
Dicalcium silicate	$2CaO.SiO_2$	25
Tricalcium aluminate	$3CaO.Al_2O_3$	12
Tetracalcium-aluminoferrite	$4CaO.Al_2O_3.Fe_2O_3$	8
Calcium sulfate dihydrate (gypsum)	$CaSO_4.2H_2O$	3.5

2.3.2 SOIL

Any soil, with the exception of highly organic materials, may be treated with cement. Ingles and Metcalf (1972) reported that for cement stabilization, the upper limit of particle size is about 8 cm (3 in.) or one-third of the thickness of the compacted layer, but a maximum size of 2 cm (3/4 in.) is to be preferred to give a good surface finish. The lower limit is about 50 per cent passing the B.S. No. 200 sieve (0.08 mm), with a liquid limit not greater than 50 and a plasticity index not greater than 18.

NAASRA (1986) provides the following guide to property limits for effective cement stabilization :

Property Limit	Value
(a) Particle Size	
Maximum size*	75 mm
Passing 4.75 mm	> 50%
Passing 425 μm sieve	> 15%
Passing 75 μm sieve	< 50%
Finer than 2 μm+	< 30%
(b) Plasticity	
Liquid Limit	< 40
Plasticity Index	< 20

* Depends on mixing plant

+ At upper limit may need pretreatment with lime

2.3.3 WATER

There is no precise measure of the quality of water required, it being generally regarded that "potable" water is satisfactory. However, highly organic water or water containing high concentration of sulphates (e.g., above 0.05 per cent) may cause problems and should be avoided. Water with a high salt content (sulphates, or chloride in sea water) may be used, provided efflorescence is not likely to be a problem. Most importantly, the quantity of water added to cement-treated mix is determined by the requirements of the maximum dry density and not that needed for cement hydration.

2.4 MECHANISM OF SOIL CEMENT STABILIZATION

The reaction between cement and clay has been investigated by a number of investigators (Herzog, 1963; Moh, 1965; Saitoh, Suzuki and Shirai, 1985).

Major constituents of cement, which have a distinct effect on the strength aspect of soil-cement mix, are calcium disilicate, calcium trisilicate, and free lime. Calcium trisilicate sets fast and is responsible for immediate strength gain. Free lime may bring about base-exchange capacity and change the texture of the soil. Calcium disilicate is responsible for long term strength due to hydration reaction (Jah and Sinha, 1977).

Shetty (1982) explained that anhydrous cement compounds when mixed with water, react with each other to form hydrated compounds of very low solubility. The hydration of cement can be visualized in two ways. The first is "through solution" mechanism. In this the cement compounds dissolve to produce a supersaturated solution from which different hydrated products get precipitated. The second possibility is that water attacks cement compounds in the solid state converting the compounds into hydrated products starting from the surface and proceeding to the interior of the compounds with time. It is probable that both "through solution" and "solid state" types of mechanism may occur during the course of reactions between cement and water. The former mechanism may predominate in the early stage of hydration in view of large quantities of water being available, and the latter mechanism may operate during the later stages of hydration. Shetty (1982) estimated that on an average 23 per cent of water by weight of cement is required for chemical reaction. This 23 per cent of water chemically combine with cement and therefore it is called bound water. A certain quantity of water is imbibed within the gel-pores this water is known as gel-water. It can be said that bound water and gel-water are complementary to each other. It has been further estimated that about 15 per cent by weight of cement is required to fill the gel-pores. Therefore, a total 38 per cent of water by weight of cement is required for the complete chemical reactions and occupy the space within gel-pores.

In the fine-grained silty and clayey soils, the cement, on hydration develops strong linkage among and between the mineral aggregates and the soil aggregates to form a matrix which effectively encase the soil aggregates. The matrix forms a honeycomb type of structure on which the strength of the mixture depends, since the clay aggregations within the matrix have little strength and contribute little to the strength of the soil-cement. The matrix is effective in fixing the particles so they can no longer slide over each other. Thus the cement provides increased shear strength. The surface chemical effect of the cement reduces the water affinity and thus reduces the water holding capacity of clayey soils. The combination of reduced water affinity and water holding capacity, and a strong matrix provides an encasement of the larger raw soil aggregates. Because of its reduced water affinity and strength, this serves not only to protect them, but also to prevent them from swelling and softening from absorption of moisture. Moreover this helps to prevent them from suffering detrimental effects from wetting and drying. In the case of granular non-plastic soils, cement may bind the particles at point of contact. The more densely graded the soil, the smaller the voids, the more numerous and greater the contact areas and the stronger the cementing action (Jah and Sinha, 1977).

In addition, the hydration of cement leads to a rise in the pH value of the pore water, which is caused by the dissociation of the hydrated lime. The strong bases dissolved the soil silica and alumina (which are inherently acidic) from both the clay minerals and amorphous materials on the clay particle surfaces, in a manner similar to the reaction between a weak acid and a strong base. The hydrous silica and alumina will then gradually react with the calcium ions liberated from the hydrolysis of cement, to form insoluble compounds (secondary cementitious products) which harden when cured to stabilize the soil. This secondary reaction is known as the "pozzolanic reaction".

The reactions which take pace in cement stabilization can be represented in the following qualitative equations; the reactions given there are for the tricalcium silicates (C_3S) only, because they are the most important constituents of Portland cement:

$$C_3S + H_2O \rightarrow C_3S_2H_x(\text{hydrated gels}) + Ca(OH)_2 \quad (2.1)$$

$$Ca(OH)_2 \rightarrow Ca^{++} + 2(OH)^- \quad (2.2)$$

$$Ca^{++} + 2(OH)^- + SiO_2\,(\text{soil silica}) \rightarrow CSH \quad (2.3)$$

$$Ca^{++} + 2(OH)^- + Al_2O_3\,(\text{soil alumina}) \rightarrow CAH \quad (2.4)$$

The cementation strength of the primary cementitious products is much stronger than that of the secondary ones. At low pH values (pH < 12.6), the following reaction will occur:

$$C_3S_2H_x \rightarrow CSH + Ca(OH)_2 \quad (2.5)$$

In Equation (2.1), $C_3S_2H_x$ and $Ca(OH)_2$ are primary cementitious products while in Equations (2.3) and (2.4), CSH and CAH are secondary cementitious products. However, during the pozzolanic reaction, the pH drops, and a drop in the pH tends to promote the hydrolysis of $C_3S_2H_x$, to form CSH. The formation of CSH is beneficial only if it is formed by the (pozzolanic) reaction of lime and soil particles, but it is detrimental when it (CSH) is formed at the expense of the formation of the $C_3S_2H_x$, whose strength-generating characteristics are superior to those of CSH. The cement hydration and the pozzolanic reaction can last for months, or even years, after the mixing, and so, the strength of cement-treated clay is expected to increase with time.

2.5 FACTORS GOVERNING THE CHARACTERISTICS OF SOIL-CEMENT MIX

The hardening characteristics of cement treated soil mixtures are developed by a number of factors. A sound understanding of the behavior of the mixture is possible only by an extensive study of the nature and extent of these factors. Factors affecting the properties of soil-cement mix are broadly classified as soil factors and production factors. Soil factors deals with the composition of the untreated soil and its response to cement and the production factors include the quality of water and cement, the uniformity of mixing, compaction and curing conditions. A brief review of the important factors is presented in the following sections:

2.5.1 CHARACTERISTICS OF SOIL

Any type of soil, with the exception of highly organic soils or some highly plastic clays, may be stabilized with cement. Bell (1993) reported that although particles larger than 20 mm diameter have been incorporated in soil-cement, a maximum size of 20 mm is preferable since this allows a good surface finish. At the other extreme, not more than about 50% of the soil should be finer than 0.08 mm. Granular soils are preferred since they pulverise and mix more easily than fine-grained soils and so result in more economical soil-cement as they require less cement. Typically soils containing between 5 and 35% fines provide the most economical soil-cement stabilization. As the grain size of granular soils is larger than that of cement, the individual grains are coated with cement paste and bonded at their points of contact. Indian Road Congress (1973a) does not recommend cement stabilization for clay soils having plasticity index greater than 22.

Soils with large clay content are difficult to mix and high additive contents are required for an appreciable change in properties. Under laboratory conditions, with elaborate attention to mixing, such heavy clays may be successfully stabilized but, in practice, it is not usual to attempt directly to stabilize with cement a clay soil with a liquid limit exceeding 45 and plasticity indices above 18% (Croft, 1968)

It is often possible, however, to cement stabilize such heavy clays after pre-treatment (modification), with either cement or, more commonly, hydrated lime. The purpose of the pre-treatment with 2-3 per cent of cement or lime content to be used is to reduce the plasticity and render the soil more workable. After curing (compacted or loose) for one to three days, the modified soil is then stabilized with cement in the usual manner.

IRC (1973a) does not recommend cement stabilization for road construction for soils having organic matter content greater than 2 per cent. Ahmed (1984) showed for a silty soil of ASSHO group A-5 having organic matter content of about 4% by weight that strength increase of the soil beyond 8% cement content is insignificant.

A comprehensive description of physical and chemical properties of soil affecting the characteristics of soil-cement mix has been presented by Ahmed (1984) and Hossain (1986).

2.5.2 CEMENT CONTENT

For a given soil that reacts normally with cement, the cement content determines the nature of the cement-treated soil mixture. The proportion of cement alters the plasticity, the volume change, the elastic properties, the resistance to wet-dry alternations and other properties in different degrees for different soils. Catton (1940) and Portland Cement Association, PCA (1956) recommended average cement requirement for moisture-density and wetting-drying tests of various fine-grained soils, which have been reported by Hossain (1986). Cement content significantly influences the physical and engineering properties of fine-grained soils (Mitchell, 1976; Ahmed, 1984; Hossain, 1986; Serajuddin and Azmal, 1991; Serajuddin, 1992; Bell, 1993). The influence of cement content on various properties of soil-cement mix has been presented in section 2.6. In general, increasing cement content has got the following effects on soil-cement mix:

(i) Increase in the values of liquid and plastic limits

(ii) Reduction in the values of plasticity index, shrinkage limit, swell, volume change and linear shrinkage

(iii) Increase or reduction in maximum dry density

(iv) Increase in unconfined compressive strength, flexural strength and stiffness, and CBR.

It is to be noted that quantity of cement required for stabilization increases as soil-plasticity increases. For highly plastic soil as much as 15 to 20% cement by weight is required to bring about the hardening of the soil (Yoder and Witczak, 1975).

2.5.3 TYPES OF CEMENT

Felt (1955) made experiments on three different types of soils to find out the effect of cement type on cement-treated soil mixtures. Felt (1955) compared the results of compaction test, compressive strength tests and the wet-dry tests made on soils treated by normal Portland cement (Type-I) and air-entraining Portland cement (Type-IA). It was found that moisture-density relationships, compressive strengths and the soil-cement losses in the wet-dry tests were almost the same. This indicates that these two types of cement can be used interchangeably in soil-cement construction.

It was further observed on experimentation with Type-III cement that the optimum moisture contents and maximum densities obtained are approximately the same for Type-I and Type-III cements. Felt (1955) also found that influence of Type-III cement on strength of different soils varies. For loamy sand, the 7 and 28-day strength for Type-III cement were about 2 and 1.4 times those for Type-I cement respectively. For a silty-clay loam, the strength for Type-III was only slightly higher than that for Type-I cement.

2.5.4 MIXING AND COMPACTION

To achieve better results by cement stabilization, efficient mixing and compaction are essential pre-requisites. Equipment used and the time lag between mixing and compaction also influence both the strength and durability characteristics of soil-cement mixtures. The degree of mixing using particular equipment and following a specific procedure depends on the soil type as well as on its degree of pulverization and its moisture content. The efficiency of mixing also depends on the mixing time. An increased wet mixing time usually increases the optimum moisture content, reduces the compressive strength and increases the weight losses during the wet-dry tests.

Studies on cement hardening and certain in-situ experiences gave rise to the idea that waiting between wet mixing and compaction could increase the compression strength of the soil-cement mix. In such cases, consolidation can even start during this rest period, while in the course of compaction cement cover under development would be torn off and prepared for further hydration. This results in an increased strength. Hungarian experience supported this assumption. But Marshall (1954) claimed that this waiting period would lead to strength reduction in case of several soils. Felt (1955) also showed that the compressive strength of cement treated soil mixtures is reduced with the increasing period of mixing. In Britain, the current specifications require that compaction be completed within 2 hours of mixing being initiated (Maclean and Lewis, 1963). Ingles and Metcalf (1972) also reported that prolonged delays between mixing and compaction reduce the magnitude of unconfined compressive strength significantly.

2.5.5 CURING TIME AND CURING TEMPERATURE

The environmental conditions under which curing takes place have significant influences on the extent to which a soil may be stabilized with cement. The unconfined compressive strength, flexural strength and stiffness of soil-cement mix increases with the increase in the curing age. Soil-cement must be moist cured during the initial stages of its life so that moisture sufficient to meet the hydration needs of the cement can be maintained in the mixtures. Curing in the laboratory moist room meets the requirements of humidity and temperature. But in field a loose material such as straw, foliage, reed, earth etc must cover the fresh surface. Another way is to cover the surface with a waterproof protective coating, usually bituminous, which then keeps the water in the pavement.

Curing temperature also markedly influences the strength of cement-treated soil mixtures. Clare and Pollard (1953) showed that when the test-temperature is around 25°C (77°F), the 7-day compressive strength increases with the increase in temperature by 2 to 2.5 percent per degree. They also found that taking the compressive strength as the sole-criterion of quality of cement-treated soil mixture, less cement is needed in warm weather than in cold weather.

2.6 PROPERTIES OF CEMENT STABILIZED SOIL

The properties of soil-cement mixtures vary with several factors as mentioned in the previous sections. The major benefits of cement stabilized soils are increased strength and stiffness, better volume stability and increased durability. The properties of soil-cement mix have been summarized by a number of investigators (Ingles and Metcalf, 1972, Kezdi, 1979; Mitchell, 1981; NAASRA, 1986; Bell, 1993). In the following sections the various physical and engineering properties of cement-treated soil have been reviewed.

2.6.1 PLASTICITY

In general, liquid limit and plastic limit of the soil generally increases with increasing cement, while the plasticity index reduces with the increase in cement content. Felt (1955) showed that the plasticity index for a plastic granular soil reduced considerably when treated with cement.

Willis (1947), however, showed that the cement admixture reduces slightly the liquid limit of mixtures made from soils having liquid limit greater than 40. Willis (1947) also showed that liquid limit increases for soils having liquid limits less than 40 when treated with cement.

Ahmed (1984) showed that for sandy silt (LL = 40, PI = 10) and silty clay (LL = 43, PI = 21), plastic limits increased while plasticity indices reduced as cement content increased. Rajbongshi (1997) found that with the increase in cement content, for coastal soil (Type: A-4, LL=41, PI=7) liquid limit and plastic limit increased while plasticity index reduced. For coastal soil (Type: A-7-6, LL = 44, PI = 19) liquid limit reduced which is presented in Figs. 2.1 and 2.2 respectively.

2.6.2 VOLUME AND MOISTURE STABILITY

Small additions of cement have profound effects on the volume stability of expansive materials without necessarily endowing the material with significant strength gains (NAASRA, 1986). Cement, by binding the particles, greatly reduces moisture induced shrinkage and swell.

The volume and moisture change of soil-cement mixtures are of particular importance with respect to pavement cracking. Crack formation is a natural characteristic of soil--cement mixes whose tendency to crack is related to strength. Apart from fractures due to loading, cracks are caused by volume changes. If a cohesive soil is treated with cement, then the shrinkage due to water-content variation of the soil-cement thus obtained will certainly be less than that of the original soil. Shrinkage decreases with increased cement content, owing to the development of a soil-cement matrix (Willis, 1947; Mehra and Uppal, 1950; Jones, 1958). With the increase in cement content, the soil-cement matrix assumes more stable configuration resulting in decreased shrinkage.

The volume change of soil cement is determined by the usual wetting and drying test method through direct volume measurement or by linear measurement of height. Cement addition has been seen to reduce the specific volume variation up to 33 or even 50 percent. Fig. 2.3 illustrates the reduction of linear shrinkage in three different cohesive soil while Fig. 2.4 shows the reduction in swell with the increase in cement content for an expansive clay. Temperature variation may also cause volume change of soil-cement mix. According to measurements performed in India, the thermal expansion coefficient depends on the cement content and density (Kezdi, 1979).

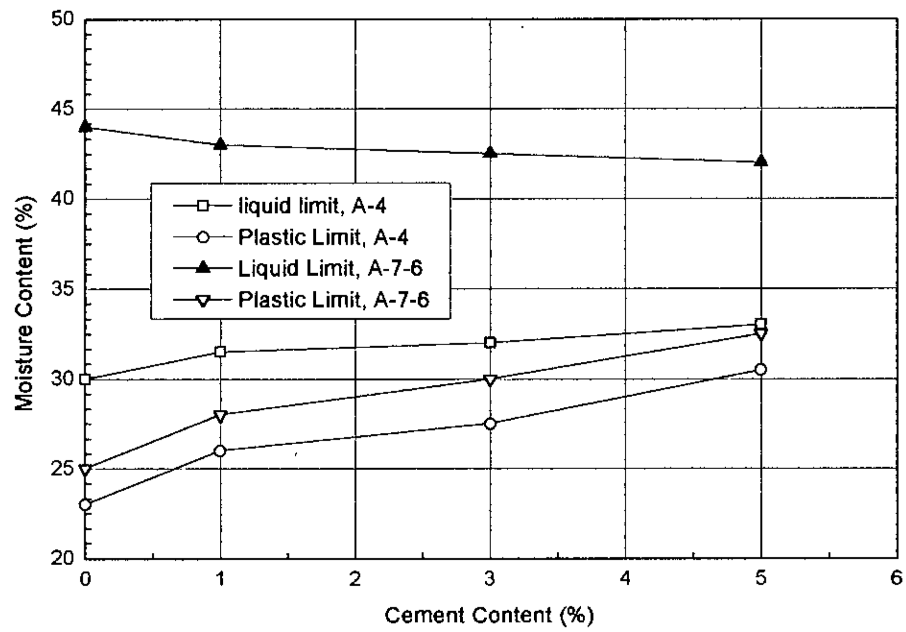

Fig. 2.1 Effect of cement content on Atterberg limits for a coastal soil.
(reproduced after Rajbongshi, 1997).

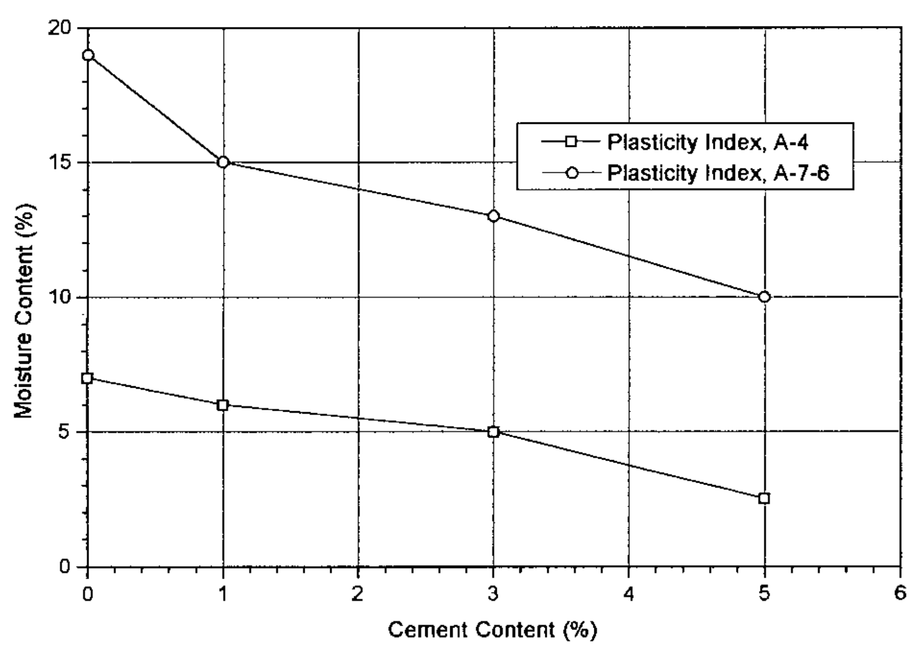

Fig. 2.2 Effect of cement content on Plasticity Index for a coastal soil.
(reproduced after Rajbongshi, 1997).

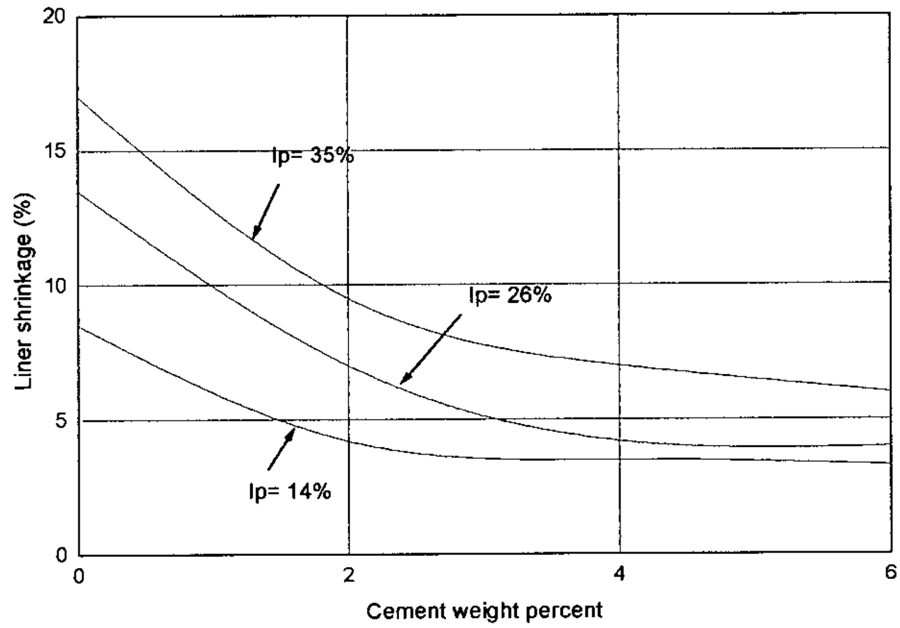

Fig. 2.3 Effect of cement content on linear shrinkage of three soils of different plasticity. (reproduced after Kezdi, 1979)

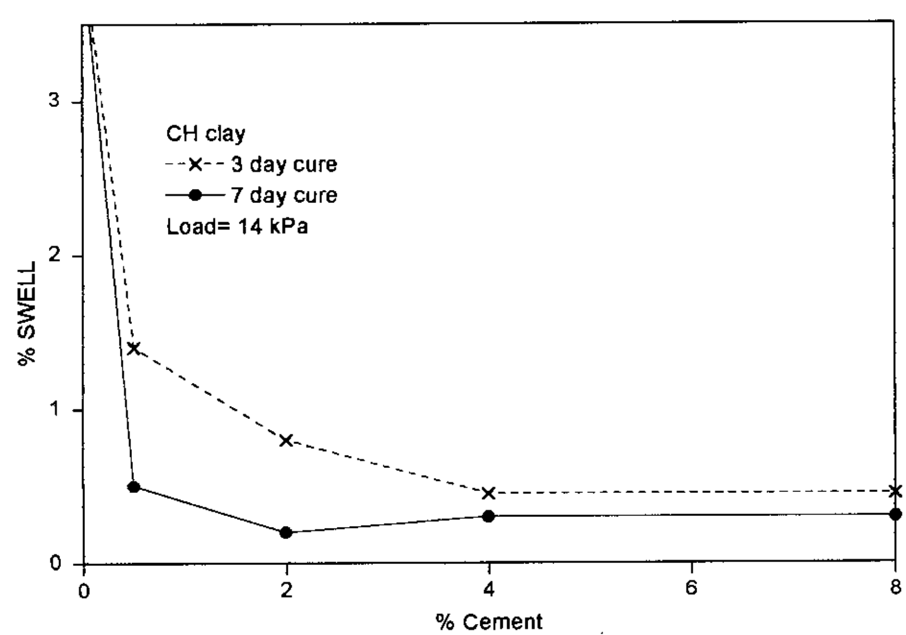

Fig. 2.4 Volumetric stability of an expansive soil stabilized with cement at optimum moisture content and standard maximum dry density. (reproduced after NAASRA, 1986)

2.6.3 MOISTURE-DENSITY RELATIONS

The density achieved in largely a function of compaction effort, soil texture and, in the case of clay soils, the type of clay minerals present, which determine the soil moisture response. Adequate compaction is essential for successful stabilization but prolonged delays between mixing and compaction reduce the maximum density attainable (Ingles and Metcalf, 1972). The addition of cement produces small increases in the compacted densities of both kaolinitic and illitic clay soils, but not those containing montmorillonite; in fact the latter gives rise to small reductions in compaction densities (Bell, 1993)

With the addition of cement, maximum dry density of sand increases; no change is observed for light to medium clays whereas it increases slightly for fat clays and for silts, density decreases on treatment with cement (Kezdi, 1979). Small changes can also be observed in the optimum moisture content. This has been illustrated in Fig. 2.5. Felt (1955) also reported that for sand and sandy soils the density increases with the increasing cement content.

Ahmed (1984) found that for sandy silt and silty clay soils, the maximum dry density reduced for increase in cement content up to 3 to 5% and then it increased with further increase in cement content. Hossain (1986), however, found reduction in maximum dry density with increasing cement content for a clayey silt. Serajuddin and Azmal (1991) reported that the maximum dry density increased while the optimum moisture content reduced with the increase in cement content for two fine-grained regional soils of Bangladesh. Results of moisture-density relations of filling sands treated with 3, 5 and 7 cement contents have been reported by BRTC (1995b). It has been found that, compared with the untreated sand, the maximum dry densities increased with the increase in cement content while the values of optimum moisture contents reduced with increasing cement contents.

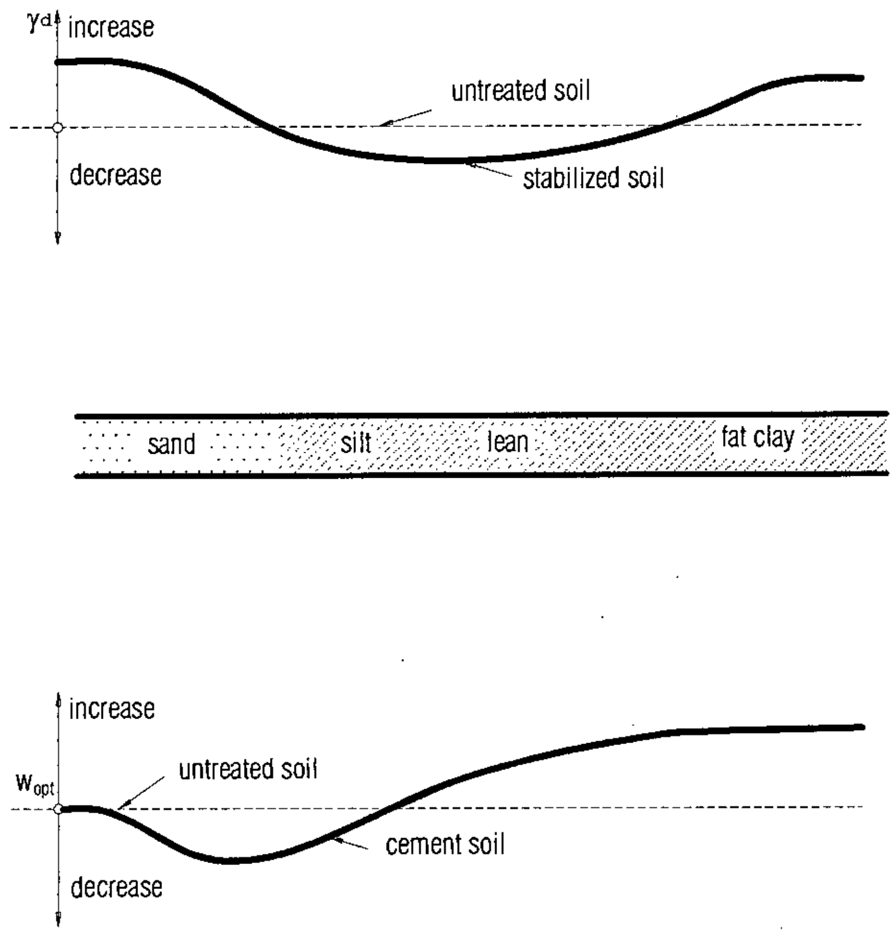

Fig. 2.5 Effect of cement content on maximum dry density and optimum moisture content of different soils (reproduced after Kezdi, 1979)

2.6.4 UNCONFINED COMPRESSIVE STRENGTH

The compression strength value can characterize the degree of soil-cement-water reaction and the progress of hardening. It is usually the compression strength value, which serves as a criterion for determining cement requirements for the construction of soil-cement. Evaluation of stabilized soil with admixture like cement is widely made with the help of compressive strength of stabilized mix. In Britain, usual practice is to specify the desired stabilities of most soil-cement mix in terms of minimum unconfined compressive strengths. The most recent specification for soil-cement requires a minimum 7-day value of 400 psi for moist-cured cylindrical specimens having a height/diameter ratio of 2:1 and 500 psi for cubical specimens (Ministry of Transport, UK, 1969).

Portland Cement Association, PCA (1956) established the range of compressive strength of cement treated soils under three broad textural soils groups, namely, sandy and gravelly soils, silty soils and clayey soils. The range of 7-day and 28-day unconfined compressive strength for soil-cement is shown in Table 2.2. The cement contents of the soil-cement mixtures for which strength values are given are those, which will satisfy the accepted stability criteria for soil-cement.

Table 2.2 Range of compressive strength of soil-cement (after PCA, 1956)

Soil Type	**Compressive Strength (psi)**	
	7 days	**28 days**
Sandy and Gravelly soils: AASHO group A-1, A-2, A-3 Unified group GW, GC, GP, GM, SW, SC, SP, SM.	300-600	400-1000
Silty soils: AASHO group A-4, A-5 Unified group ML and CL	250-500	300-900
Clayey soils: AASHO group A-6, A-7 Unified groups MH, CH	200-400	250-600

In general, unconfined compressive strength increases linearly with cement content, but at different rates for fine-grained and coarse-grained soils as shown in Fig. 2.6. Curing time is also important because strength increases gradually with age of curing. The effect of curing age on unconfined compressive strength for fine-grained and coarse-grained soils stabilized with 10% cement is shown in Fig. 2.7.

A most comprehensive review of the strength properties of cement stabilization was reported by Mitchell (1981). The unconfined compressive strength, q_u is generally described as increasing linearly with the cement content percentage, C. This increase is more pronounced for coarse-grained soil than for silt and clays.

Mitchell (1981) reported the following relationships between curing time and q_u

$$q_u(d) = q_u(d_0) + K \log \frac{d}{d_0} \qquad (2.6)$$

where, $q_u(d)$ = Unconfined compressive strength at d days, kPa

$q_u(d_0)$ = Unconfined compressive strength at d_0 days, kPa

K = 480C for granular soils and 70C for fine-grained soil

C = Cement content, % by weight

Ramaswamy, Aziz, Kheok and Lee (1984) reported that the values of q_u of cement-treated silty clay subgrade soil samples for road construction continued to increase with the increase in cement content and curing age.

Ahmed (1984), Hossain (1986) and Rajbongshi (1997) investigated the effect of cement stabilization on unconfined compressive strength (1.4 in. diameter by 2.8 in. high samples) of a number of regional soils of Bangladesh. Ahmed (1984) and Hossain (1986) found that compared with the untreated soil, unconfined compressive strength of the cement-treated samples increased markedly, depending on the cement content and curing age. The effect of cement content and age on compressive strength, and the rate of gain in strength with cement content for coastal soils reported from Rajbongshi (1997) are shown in Figs. 2.8 and 2.9 respectively.

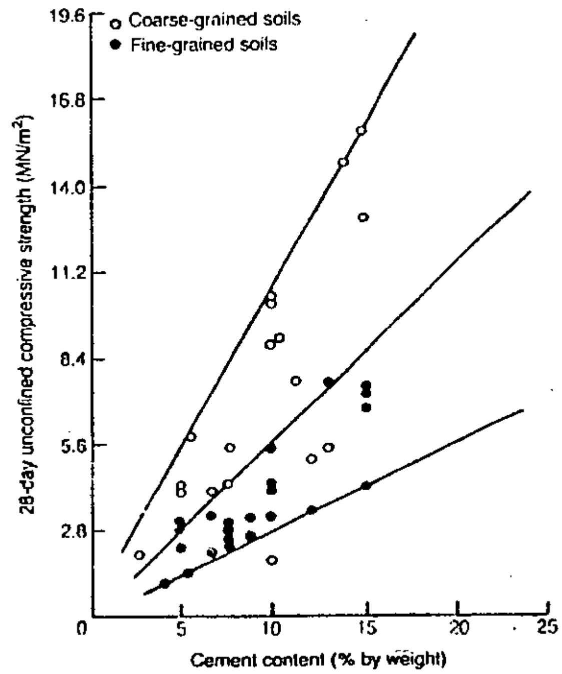

Fig. 2.6 Effect of cement content on unconfined compressive strength of cement stabilised coarse-grained and fine-grained soils (after Anon, 1990)

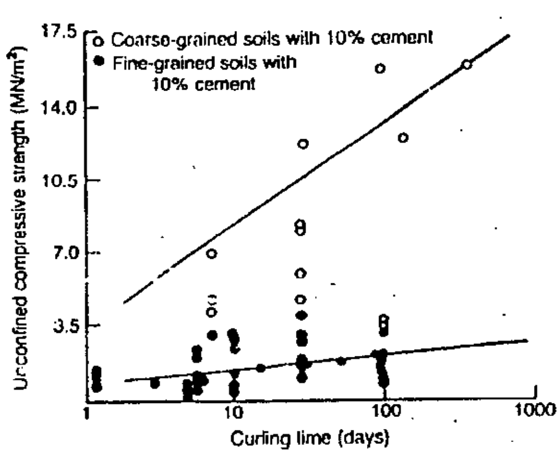

Fig. 2.7 Effect of curing time on unconfined compressive strength of cement stabilised coarse-grained and fine-grained soils (after Anon, 1990)

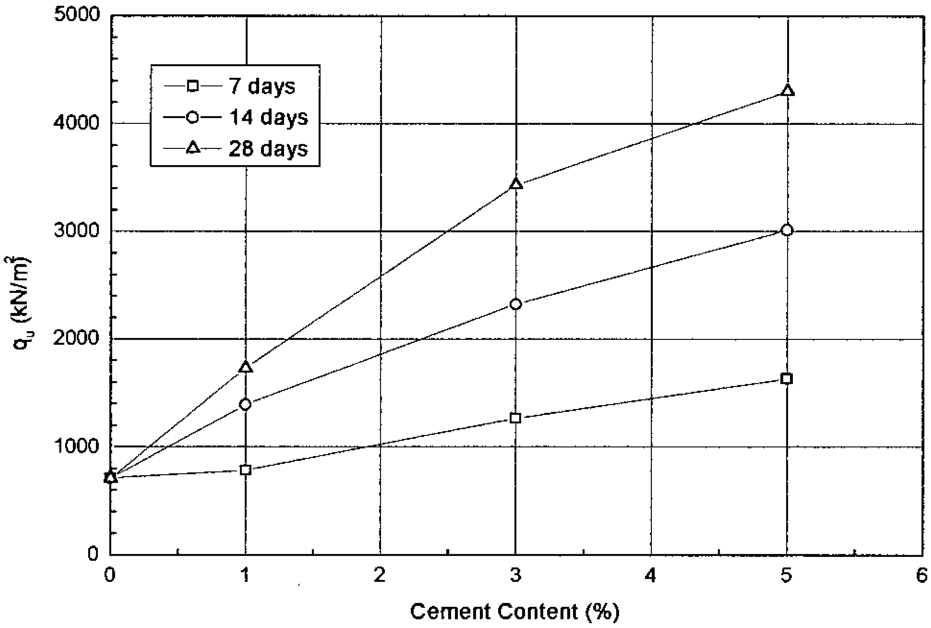

Fig. 2.8 Effect of cement content on unconfined compressive strength of a sandy silt (reproduced after Rajbongshi, 1997)

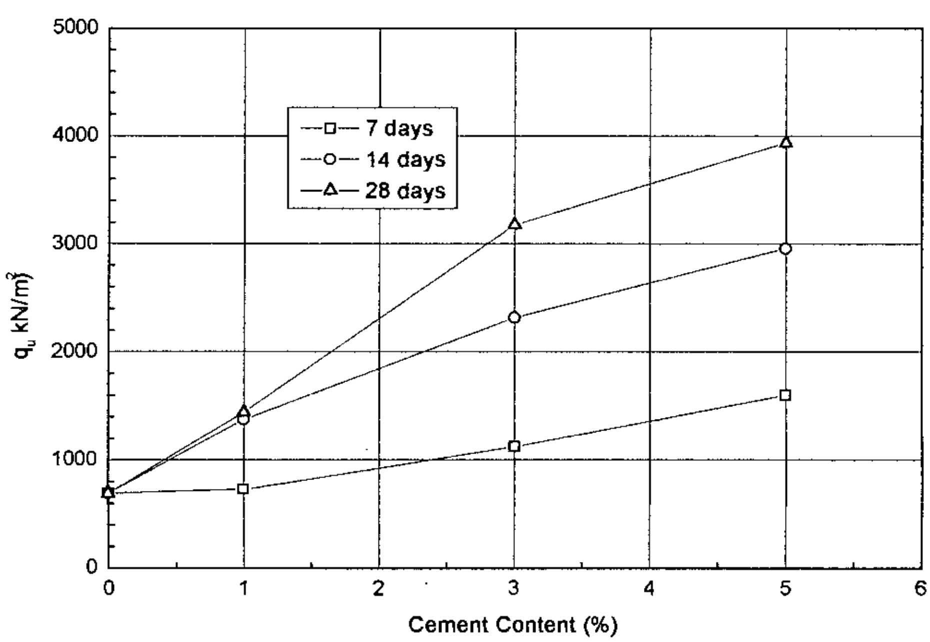

Fig. 2.9 Gain in unconfined compressive strength for a cement-treated silty clay over untreated soil (reproduced after Rajbongshi, 1997)

Serajuddin and Azmal (1991) and Serajuddin (1992) reported the effect of cement content and curing age on unconfined compressive strength (50 mm diameter and 100 mm high samples) of regional alluvial soils of Bangladesh. Typical results are presented in Figs. 2.10 and 2.11. Both Figs. 2.10 and 2.11 show that compressive strength of samples stabilized with cement increases with the increase in cement content and curing age.

Hong (1989) and Uddin (1995) reported the effect of cement content and curing age on unconfined compressive strength of soft Rangsit clay of Bangkok. Hong (1989) reported results of samples (LL = 104, PI = 63) stabilized with 5% to 15% cement and cured for 7 days to 56 days while Uddin (1995) reported results of samples (LL = 70 to 117, PI = 50 to 78) treated with 5% to 40% cement and cured for 1 week to 40 weeks. Hong (1989) and Uddin (1995) found considerable increase in unconfined compressive strength, depending on the cement content and curing age.

2.6.5 CALIFORNIA BEARING RATIO (CBR)

Small additions of cement to well-graded granular materials commonly lead to large increases in measured CBR. Mitchell (1976) reported the following empirical relationship for the CBR of compacted cement stabilized soil:

$$CBR = 0.0038\, q_u^{1.45} \tag{2.7}$$

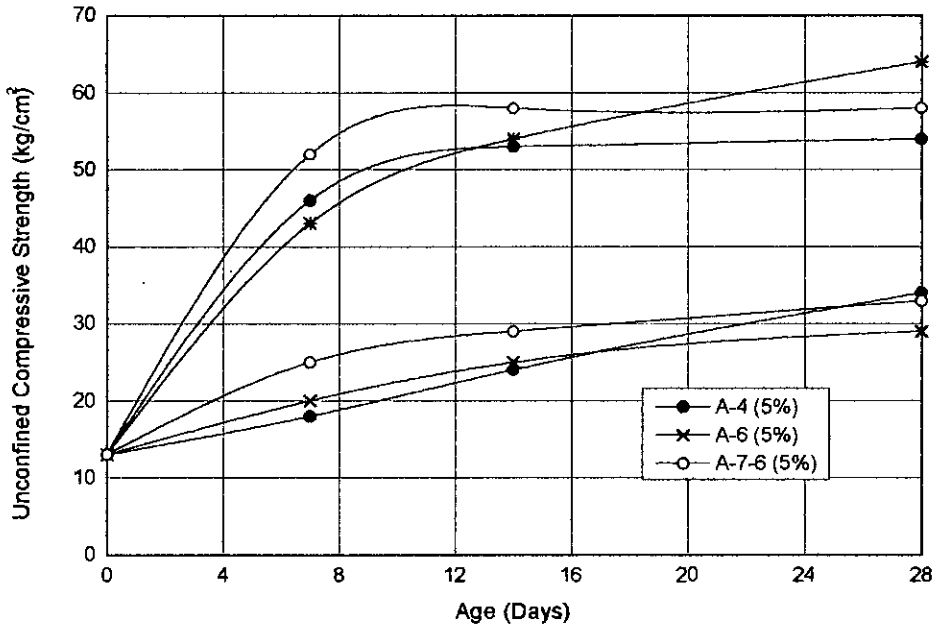

Fig. 2.10 Effect of curing age and cement content on unconfined compressive strength for soil-cement mix specimens of three typical silty soils (reproduced after Serajuddin and Azmal, 1991)

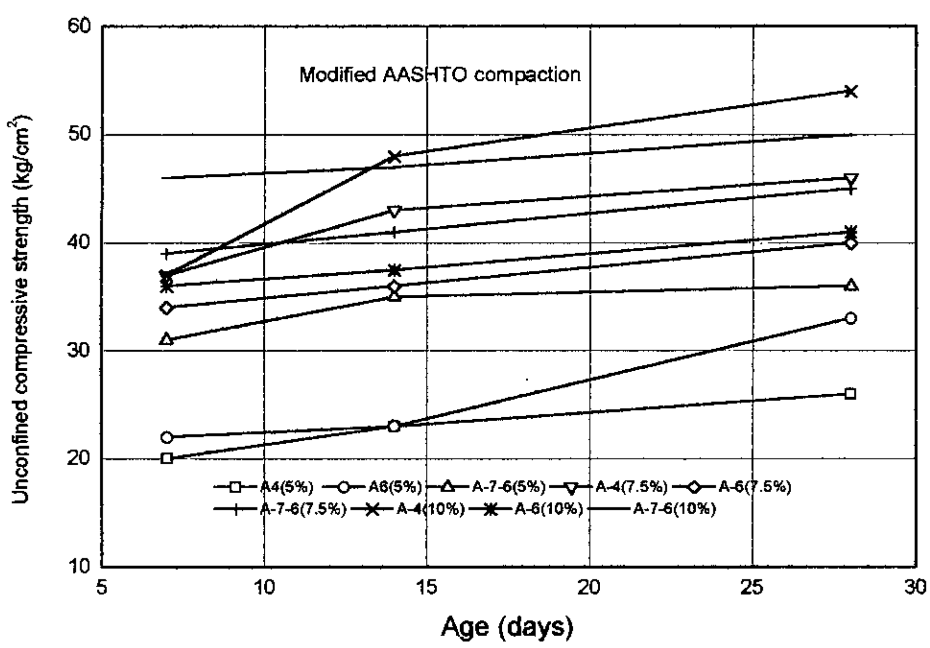

Fig. 2.11 Effect of curing age and cement content on unconfined compressive strength for cement stabilized three typical regional soils (reproduced after Serajuddin, 1992)

In Equation (2.7), q_u is the unconfined compressive strength in kPa. Ingles and Metcalf (1972) reported the following CBR-values for different types of cement treated soils:

Soil type (Unified Soil Classification)	CBR
GW, GP, GM, GC, SW	600
SM, SC	600
SP, ML, CL	200
ML, CL, MH, VH	< 100
CH, OL, OH, Pt	< 50

According to NAASRA (1986) there is, however, limitation to the use of CBR in the pavement design with stabilized materials. Use of CBR in pavement design has been reported to be inapplicable to cement-treated layers. In well graded granular materials, small additions of cement may give rise to significant increase in CBR, which is inappropriate for design. NAASRA (1987) also suggested that, as far as sub grades are concerned, stabilized soils should not be assigned a CBR value greater than 15.

Results of CBR tests of filling sands treated with 1%, 3%, 5% and 7% cement contents have been reported by BRTC (1995b) and Rajbongshi (1997). Effect of cement content on CBR for three levels of compaction efforts was investigated. It has been found that CBR increased with the increase in cement content and dry density of the soil-cement mix as shown in Figs. 2.12 and 2.13 respectively.

2.6.6 TENSION AND FLEXURAL PROPERTIES

When a soil cement pavement fails under a wheel load, the failure may be caused by tensile stresses on the underside of the slab under the load or by surface stresses some distance from the load. The tensile strength of soil-cement is, therefore, important in connection with road pavement. Indirect tensile test has been used for evaluating the tensile strength of soil-cement mixtures. The indirect tensile test is essentially a diametric compression test in which the material fails in tension along the loaded diameter of the cylindrical test specimen. Tensile strength is influenced by particle size

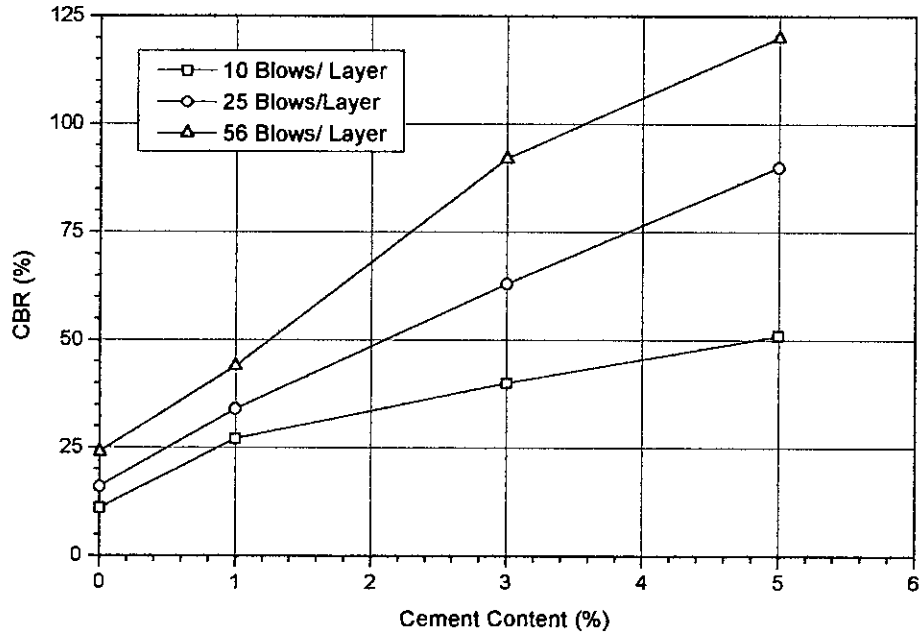

Fig. 2.12 Effect of cement content on CBR values for cement treated coastal soils (reproduced after Rajbongshi, 1997)

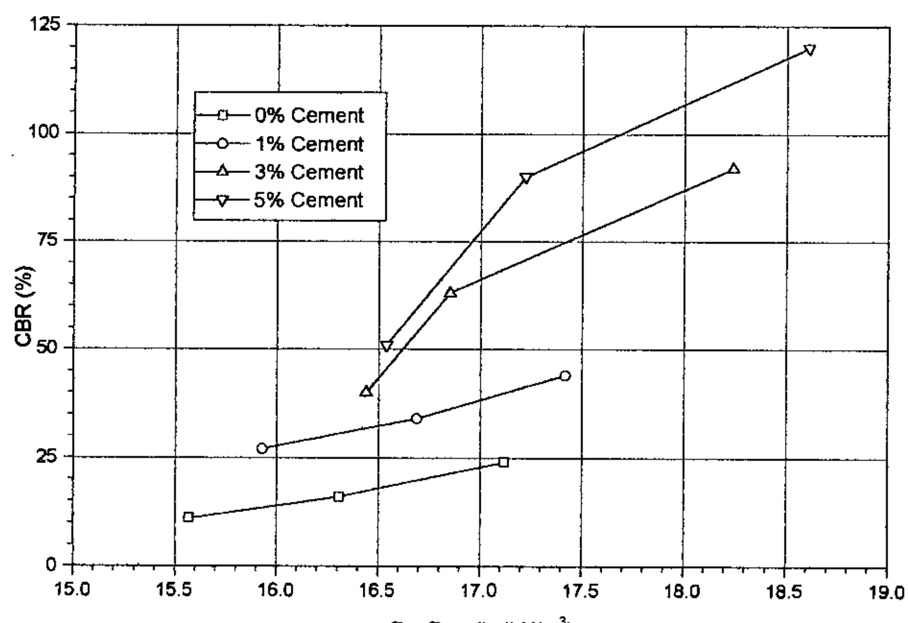

Fig. 2.13 Relationship between dry density and CBR values for cement treated coastal soils (reproduced after Rajbongshi,1997)

distribution, moisture content, cement content, and density. Maximum tensile strength would usually occur for materials compacted close to optimum moisture content. Ingles and Metcalf (1972) reported the relationship between unconfined compressive strength and indirect tensile strength at the optimum moisture content and maximum density which is shown in Fig. 2.14. It can be seen from Fig. 2.14 that the indirect tensile strength at the optimum moisture content and maximum density is approximately equal to 10 per cent of the compressive strength at this condition.

Cement bound materials fail in tension under relatively low strain. The critical strain usually decreases with increasing modulus. As with other fatigue relationships any increase in-place strain reduces the critical number of repetitions to failure. For equivalent moduli the critical tensile strain for cement bound materials is likely to be much lower than the critical strain for asphalt.

From tests of small soil-cement beams in flexure, Ingles and Metcalf (1972) reported the following features:

(i) Non-linear stress-strain behavior, although the relation is usually linear up to 60-70 per cent of the failure load

(ii) non-recoverable strain on repeated loading, and

(iii) Gradually increasing strain under constant loading (i.e., creep).

Mitchell (1981) reported that the flexural strength is in the order of one-fifth to the one-third of the unconfined compressive strength.

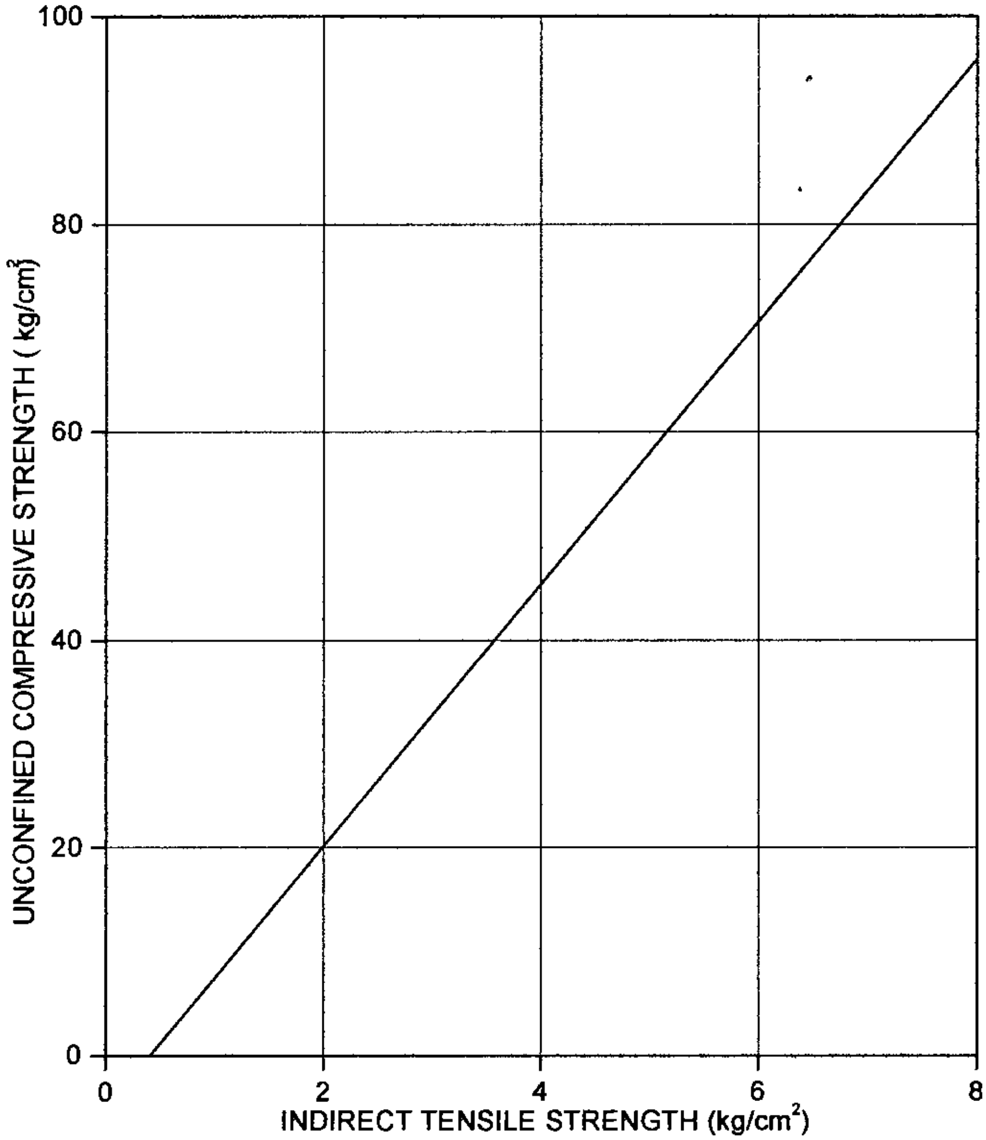

Fig. 2.14 Relationship between unconfined compressive strength and indirect tensile strength for cement stabilized soils (reproduced after Ingles and Metcalf, 1972)

2.6.7 DURABILITY

Durability of soil-cement mixture is its resistance to repeated drying and wetting or freezing and thawing. Cement provides stability against freeze-thaw and cyclic wetting and drying. Criteria based on the freeze-thaw and cyclic wetting and drying are widely used in Northern America. Materials satisfying these criteria would normally fall into the bound class with unconfined compressive strengths of about 3 MPa or higher (NAASRA, 1986). In the United States, the desired cement content is normally selected to meet durability. Portland Cement Association (1956) reported the values of maximum soil-cement loss in the wet-dry test which is as follows:

AASHO Soil Groups	Freeze-Thaw and Wet-Dry Losses (%)
A-1-a, A-1-b, A-3, A-2-4 and A-2-5	14
A-2-6, A-2-7, A-4 and A-5	10
A-6, A-7-5 and A-7-6	7

Compendium 8 (1979), suggested that in tropical and sub-tropical conditions, where freeze and thaw tests are not essentials, a q_u-value of 150 psi (1034 kN/m^2) at 7 days curing age is adequate to stand 12 cycles of wetting and drying which satisfies the weathering conditions in the tropics.

Rajbongshi (1997) investigated the effect of durability of soil-cement mixes of coastal soils by performing wetting and drying tests. He found that soil-cement loss sharply reduced as cement content increased which is shown in Fig. 2.15. It can be seen from Fig. 2.15 that addition of about 5% cement in the soil studied by Rajbongshi (1997) would result a durable soil-cement mix, which does not satisfy the PCA (1956) criteria.

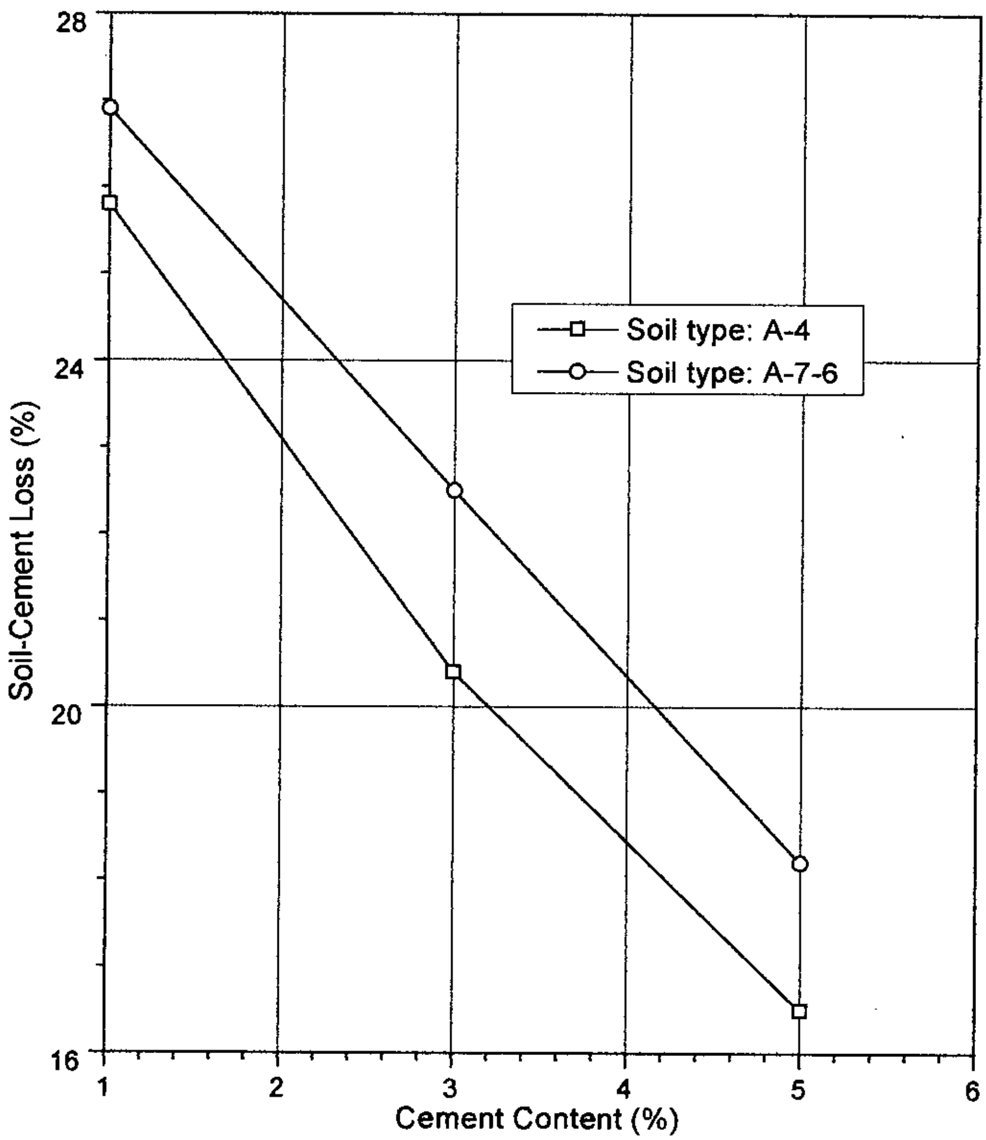

Fig. 2.15 Effect of cement content on soil-cement loss of two regional soils in wetting and drying test (reproduced after Rajbongshi, 1997)

2.7 APPLICATIONS OF SOIL-CEMENT

The principal use of soil-cement is as a base material underlying pavements. One of the reasons soil-cement is used as a base is to prevent pumping of fine-grained sub grade soils into the pavement above. The thickness of the soil-cement base depends upon sub grade strength, pavement design period, traffic and loading conditions and thickness of the wearing surface. Frequently, however, soil-cement bases are around 150-200 mm in thickness.

Ingles and Metcalf (1972) recommenced the values of various engineering properties of cement-treated materials for various purposes that are shown in Table 2.3. In Table 2.3, the recommended values of unconfined compression strength, CBR, swell and loss in wetting and drying tests are presented. Ingles and Metcalf (1972) also suggested the cement contents for various soil types for pavement construction that are sown in Table 2.4.

Bell (1993) reported various other uses of soil-cement. Soil-cement has been used to afford slope protection to embankment dams, soil-cement made from sandy soils giving a durable erosion-resistant facing. Soil-cement has also provided slope protection for canals, river banks, spillways, highway and railway embankments and coastal cliffs. Where slopes are exposed to moderate to severe wave action or rapidly flowing water, the soil-cement generally is placed in horizontal layers 150 mm to 225 mm thick and 2-3 m wide adjacent to the slope, that is, as "stair step slope protection". In situations where conditions are less severe, a layer of soil-cement 150 mm to 225 mm thick may be placed parallel to the slope of the face. In addition to water storage reservoirs, soil-cement has been used to line waste-water treatment lagoons, sludge-drying beds, ash-settling ponds and sanitary landfills. The soil-cement linings are commonly 100 mm to 150 mm thick.

Table 2.3 Values of engineering properties of soil-cement (after Ingles and Metcalf, 1972)

Purpose	q_u (7- Day Cured)		Four-Day Soaked CBR	Swell	Loss in Wet-Dry Test
	kg/cm^2	psi		%	%
Road sub-base, formation backfill for trenches etc	3.5-10.5	50-150	20-80	2	7
Road sub-base, base for light traffic*	7-14	100-200	50-150	2	10
Base for heavy traffic* Building blocks	14-56	200-800	200-600	2	14
Embankment protection Floodways (too strong for general use under thin surfacing)	>56	800	600	2	14

* Lower strengths may be adequate for well-drained areas in the tropics

Table 2.4 Cement contents for various soil types for pavement construction (after Ingles and Metcalf, 1972)

Soil Type	Cement Requirement (per cent)
Fine crushed rock	½ - 2(1)
Well graded sandy clay gravels	2 - 4
Well graded sand	2 - 4
Poorly graded sand	4 - 6(2)
Sandy clay	4 - 6
Silty clay	6 - 8
Heavy clay	8 - 12
Very heavy clay	12 - 15(3)
Organic soils	10 - 15(4)

(1) Used as a construction expedient to aid "set up" on compaction, to reduce sensitivity to compaction moisture content and prevent reveling under construction traffic.

(2) Compaction may be very difficult, and segregation of the cement may occur.

(3) Mixing may be very difficult - pretreatment with lime may help.

(4) Pretreatment with lime or addition of 2 per cent calcium chloride may help.

2.8 LIME STABILIZATION

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 stabilizer with soil is the most important factor affecting the quality of results. Feasibility of stabilization techniques for different types of soils the most commonly used stabilizers for improving the physical and engineering properties of soils is lime. The use of lime to stabilize subgrade soil has been known to engineers all over the world for a long time. Lime stabilization in widely recommended for construction of roads (Ingles and Met calf, 1972; NASARA, 1986, Hausmann, 1990). The pressure injection method of lime stabilization 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 stabilization 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 stabilized 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 stabilized soils, characteristics of lime treated soil, and applications of lime stabilization.

2.9 MATERIALS FOR LIME STABILIZATION

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

2.9.1 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 stabilization. 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 stabilization 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.

The efficiency of lime stabilization 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 favorable to strength gain (Broms, 1986).

Table 2.5 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/m3)	450 to 560	1050	1250

2.9.2 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 stabilize 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 pulverizing 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 stabilization as a construction expedient.

2.9.3 WATER

Potable water is preferred for lime stabilization. Acidic (organic) water should be avoided. Seawater can be used but should be avoided where a bituminous seal is to be placed, as crystallization of salts may lift the seal. The amount of water used in lime stabilization 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 pulverization and mixing stage is less important than in the case of cement stabilization.

2.10 MECHANISMS OF LIME STABILIZATION

It is recognized 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.

2.10.1 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 montmorllinite, kaolinite, and illite begin by the replacement of existing cations between the silicate sheets with Ca++. Following the saturation of inter layer positions with Ca++, the whole clay minerals deteriorate without the formation of substantial new crystalline phases.

2.10.2 CEMENTATION

Cementation is the main contributor to the strength of the stabilized 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:

$$Ca^{++} + 2(OH)^{-} + SiO_2\,(soil\ silica) \Rightarrow CSH_2 \qquad (2.14)$$

$$Ca^{++} + 2(OH)^{-} + Al_2O_3\,(soil\ alumina) \Rightarrow CAH_3 \qquad (2.15)$$

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 stabilization. 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 would 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 stabilization, 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 cementetious value, in finely divided form and in the presence of moisture; chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementetious properties. Asserson et al. (1974) worked with red tropical soils suggested that after the initial 7 days of curing, strength increases as a 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.

2.10.3 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.

2.11 FACTORS AFFECTING LIME STABILIZATION

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.

2.11.1 SOIL CHARACTERISTICS

2.11.1.1 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 stabilization.

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 stabilization 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 stabilization.

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 stabilization. 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 stabilized using lime. It was fond that the strength of lime stabilized 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 stabilized 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 stabilization on these types of soil is such that the stabilized 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 stabilized with lime. He 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-stabilization. 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 sulphate 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.

2.11.1 ORGANIC MATTER PRESENT IN THE SOIL

One of the important factors that inhibit lime-soil reaction is the organic content. One of the possible reasons 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 stabilization. However, IRC (1973a) recommended a maximum limit of 2% organic content for lime stabilization.

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

Arman and Muhfakh (1972) studied the effect of the percent of organic matter on the unconfined compressive strength of the lime stabilized soil. It has been found that the presence of organic matter in the soil reduce the strength of the stabilized 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 stabilized 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.

2.11.2 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 stabilization 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.

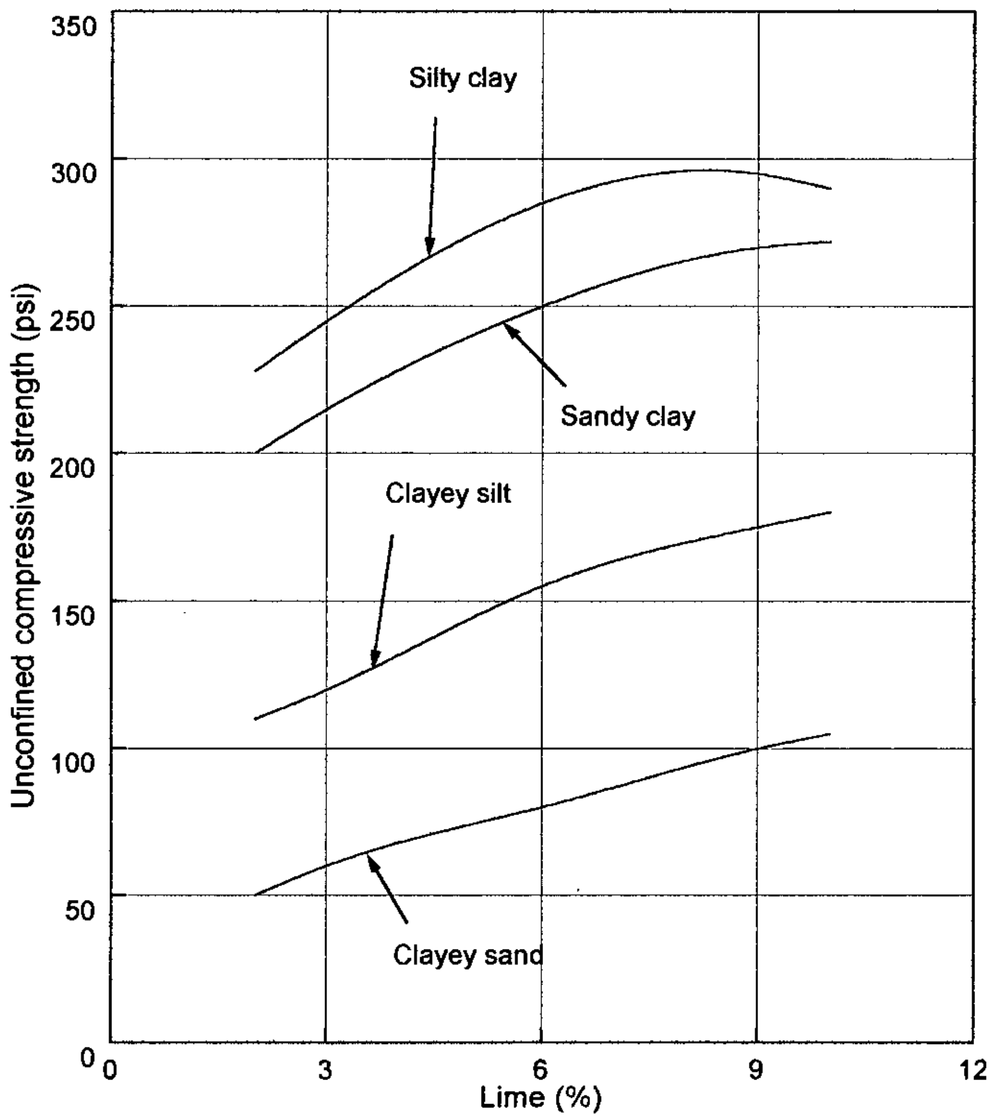

Fig. 2.16 Variation of unconfined compressive strength (q_u) with lime content for various types of soil (reproduced after Locat et al., 1990)

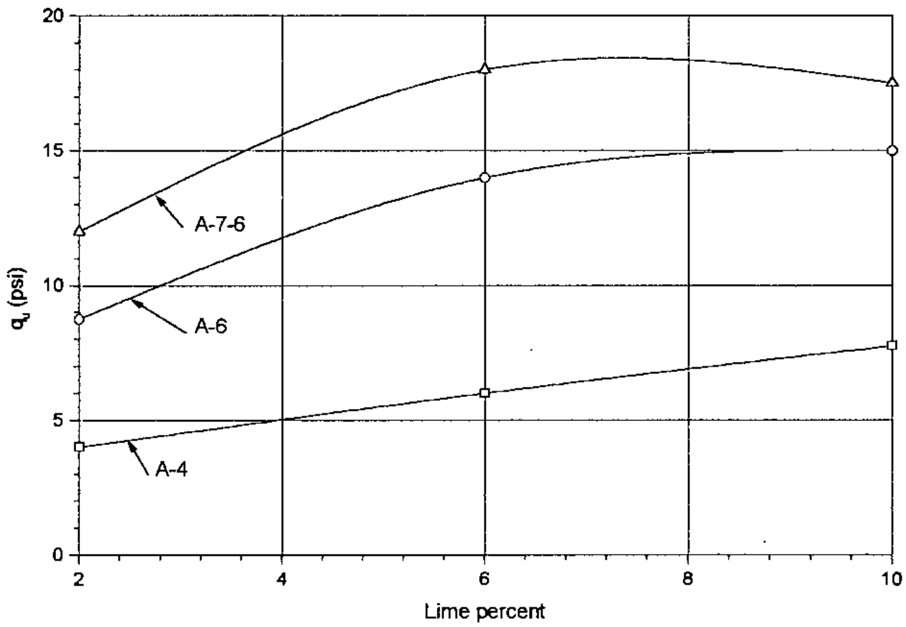

Fig. 2.17 Variation of unconfined compressive strength with lime content for different types of soil (reproduced after Serajuddin, 1992)

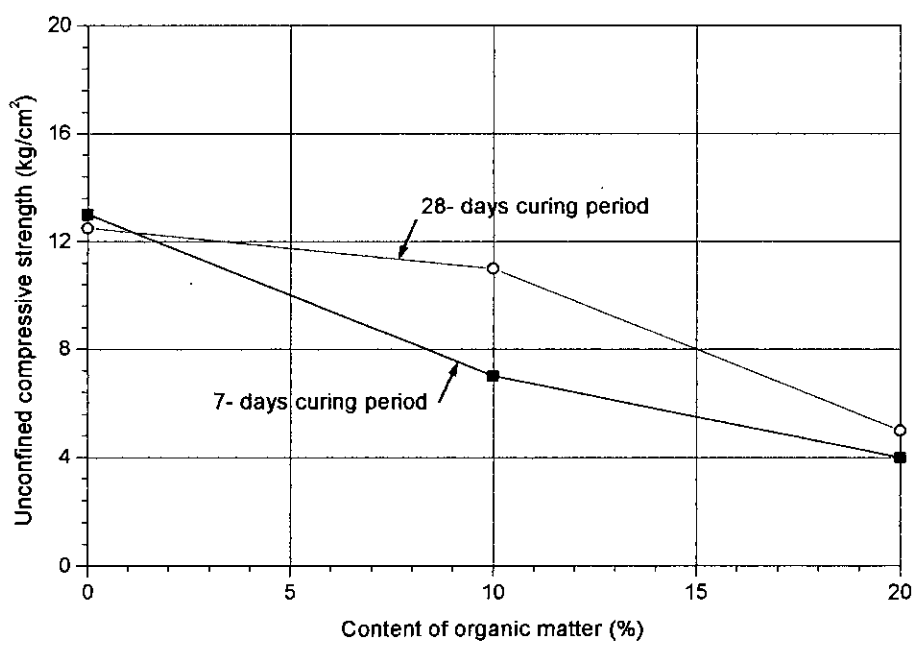

Fig. 2.18 Effect of organic matter on unconfined compressive strength of lime treated soil (reproduced after Arman and Muhfakh, 1972)

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 stabilization of silty clay, and 3% to 8% was proposed for stabilization 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 stabilization varies from 2% to 8%. Variation of the unconfined compressive strength of the lime stabilized 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 stabilized 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 stabilized 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 stabilized 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.

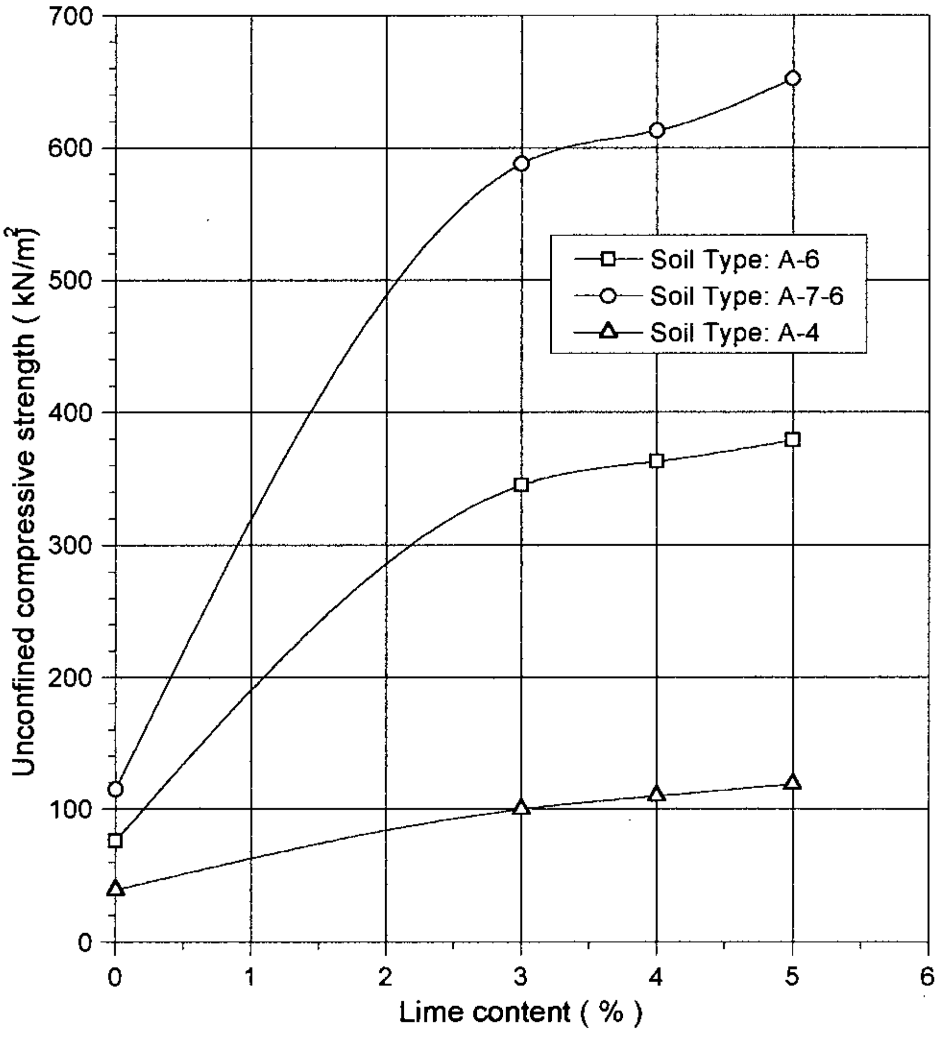

Fig. 2.19 Variation of the unconfined compressive strength of lime stabilized soil due to variation of lime content (reproduced after Molla, 1997)

2.11.3 MIXING AND COMPACTION PROCEDURE

2.11.3.1 COMPACTIVE EFFORT

The success of lime-soil stabilization 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 AASHO compaction. For the same conditions, but with modified AASHO 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 stabilized 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 stabilized 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 stabilized 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 stabilized soil. Serajuddin (1992) found that the CBR value of the stabilized 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 stabilized 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.

2.11.3.2 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 tbe strength of the specimen up to 30% as compared to the specimen prepared by compacting immediately after mixing.

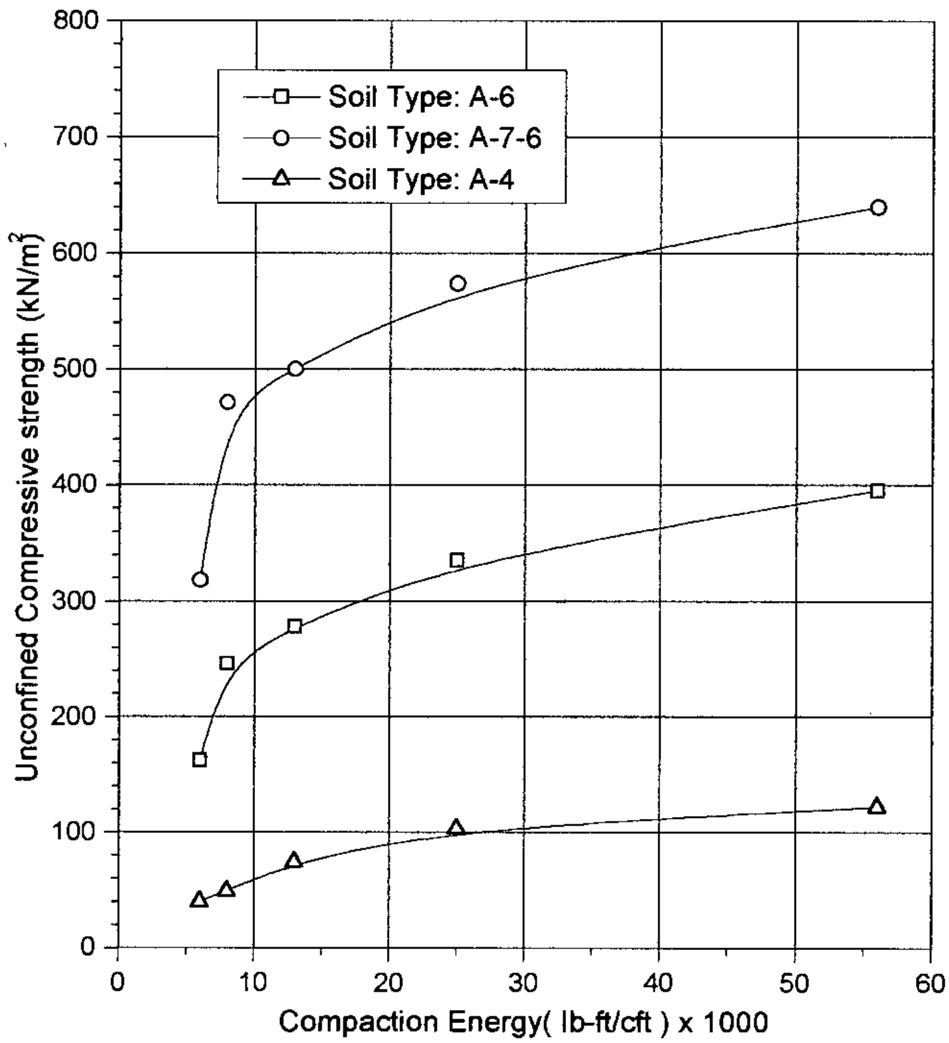

Fig. 2.20 Variation of unconfined compressive strength (qu) at different compactive effort for stabilized soils using 3% lime (reproduced after Molla, 1997)

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 continuous 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 lose soil particles by week cementation, as the soil mellows.

Shahjahan (2001) investigated the effect of compaction delay time on unconfined compressive strength of three regional soils of Bangladesh. He reported that unconfined compressive strength decreases with the increase in compaction delay time. This trend is presented in Fig. 2.21 for two soils.

2.11.4 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 stabilized clays will normally be higher than that of untreated clay after mixing.

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

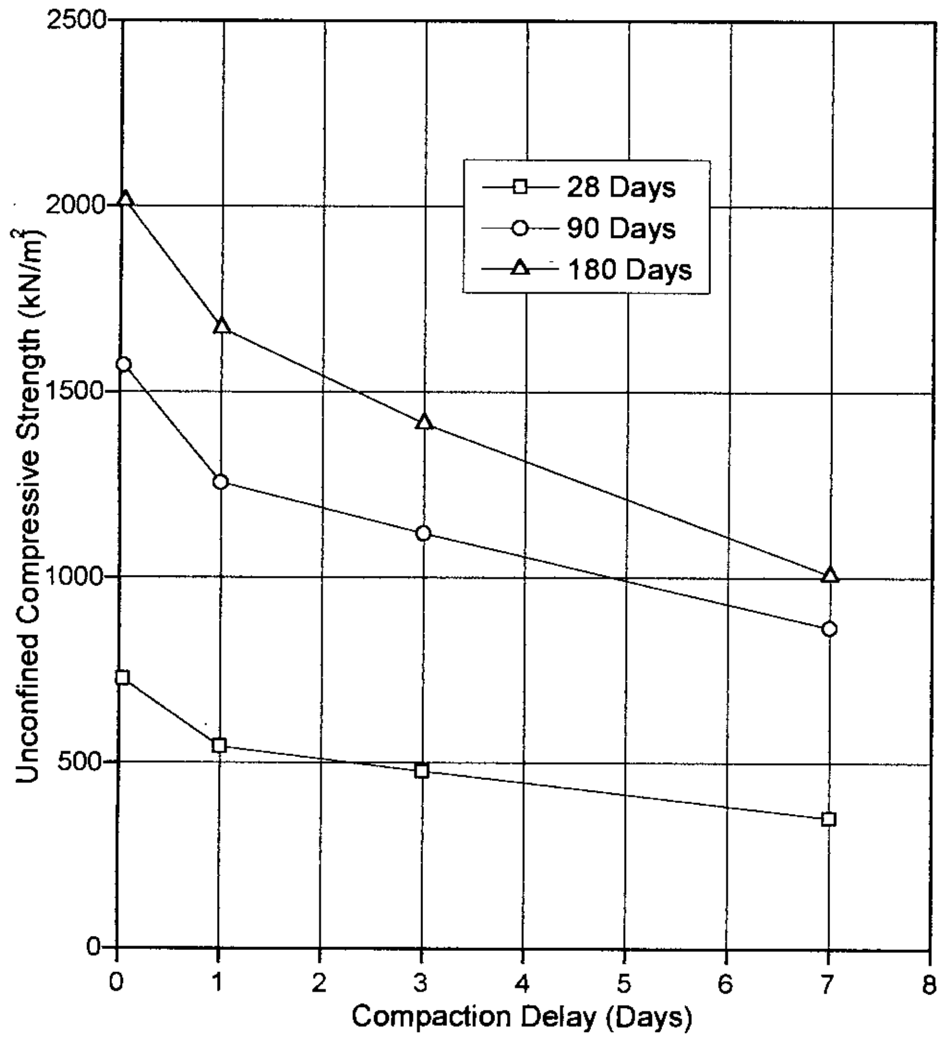

Fig. 2.21 Variation of unconfined compressive strength (q_u) with compaction delay time Soil type-ML/CL (reproduced after Shahjahan, 2001)

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 stabilized 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 favored by a high temperature. In fact, one of the limitations of soil-lime stabilization is the climatic factor. It is found that reactions are not effective at temperatures below 50°F and therefore under such circumstances, soil-lime stabilization 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 stabilized 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.

2.12 PROPERTIES OF LIME STABILIZED SOIL

The main benefits of lime stabilization of clays are imported workability, increased strength, and volume stability. The properties of soil-lime mix have been summarized 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 stabilized soils are reviewed.

2.12.1 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).

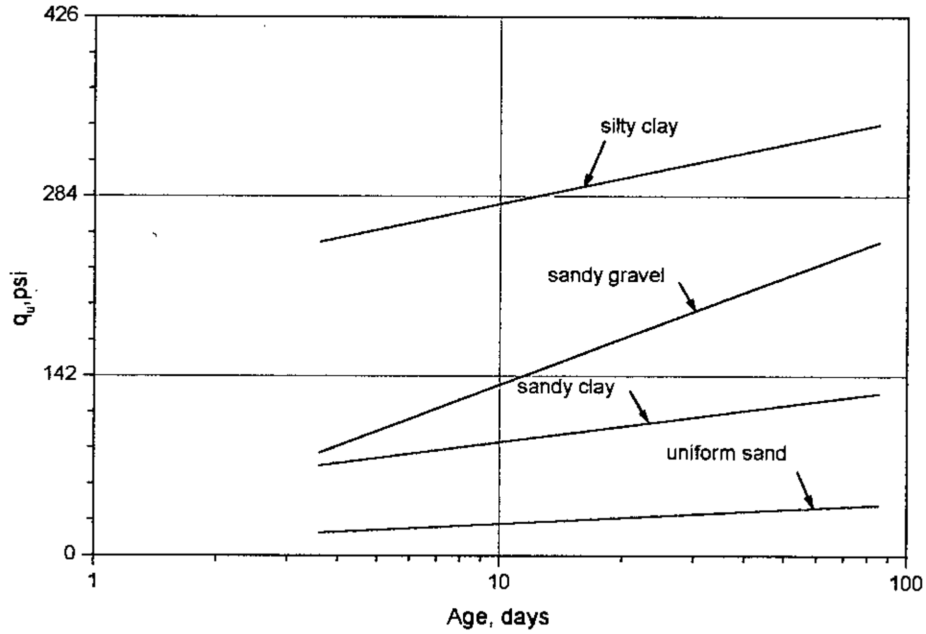

Fig. 2.22 Effect of curing age on unconfined compressive strength (q_u) for various types of soils stabilized with 5% lime (reproduced after Ingles and Metcalf, 1972)

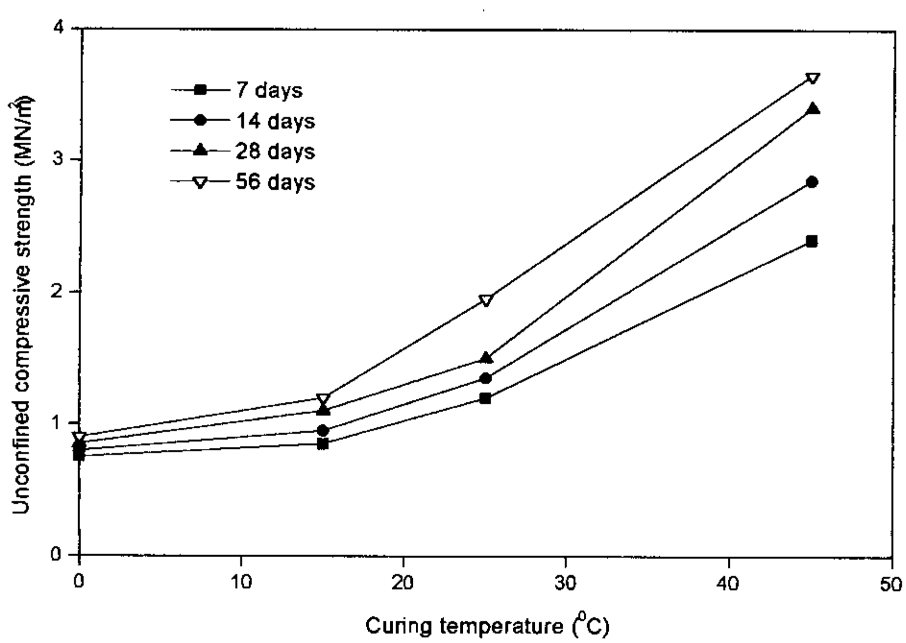

Fig. 2.23 Effect of curing temperature and curing age on unconfined compressive strength of a clay of high plasticity stabilized with 5% lime (reproduced after Bell, 1988)

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; Rodroguez 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.

2.12.2 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 stabilization 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 stabilization.

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 the 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.

2.12.3 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 stabilized materials. Lime stabilized 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.

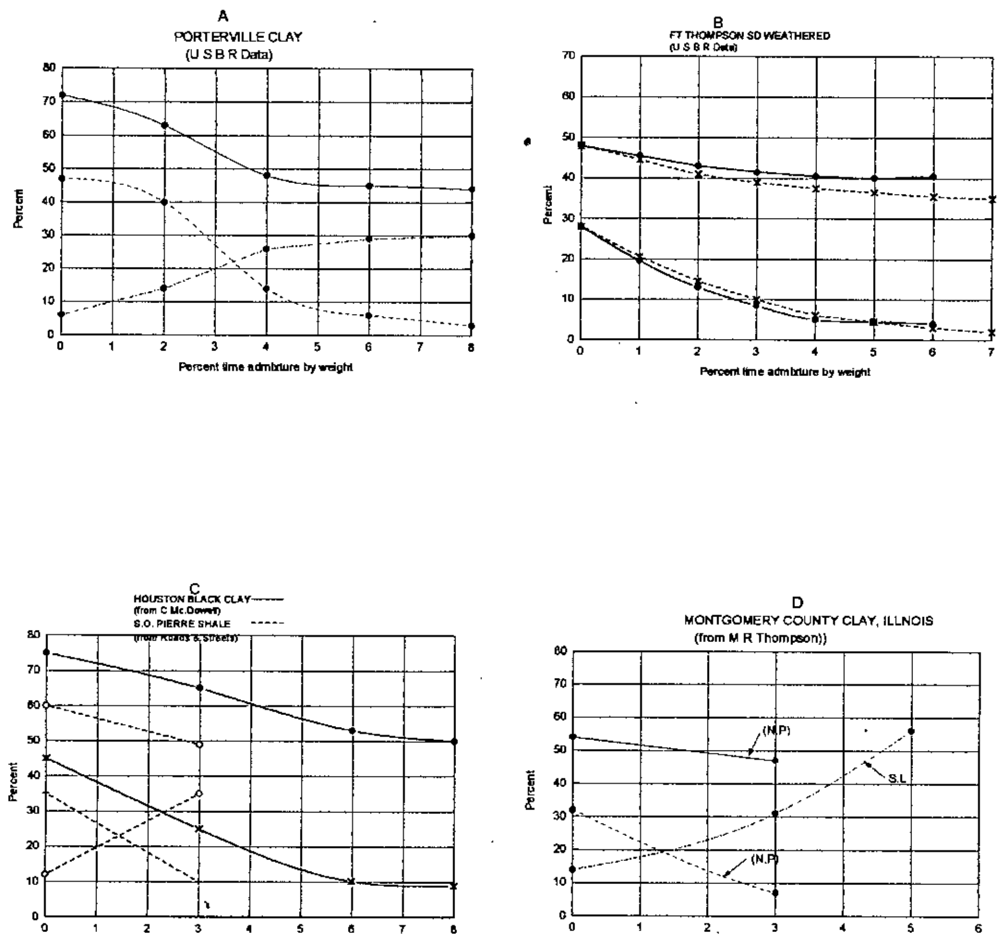

Fig. 2.24 Effect of lime on plastic characteristics of expansive montmorillonite clays (reproduced after Holtz, 1969)

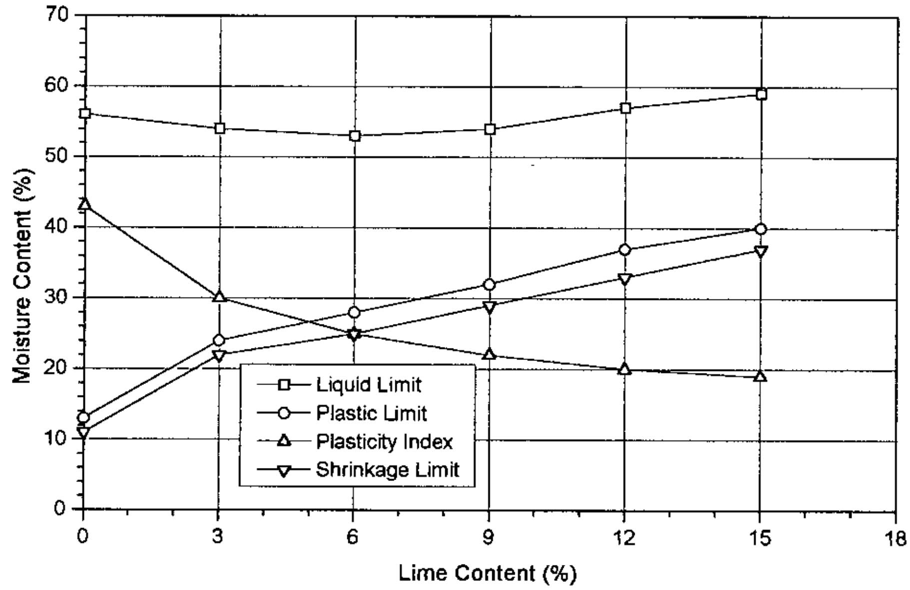

Fig. 2.25 Effect of lime content on Atterberg limits and shrinkage limit of a expansive soil (reproduced after Hossain, 2001)

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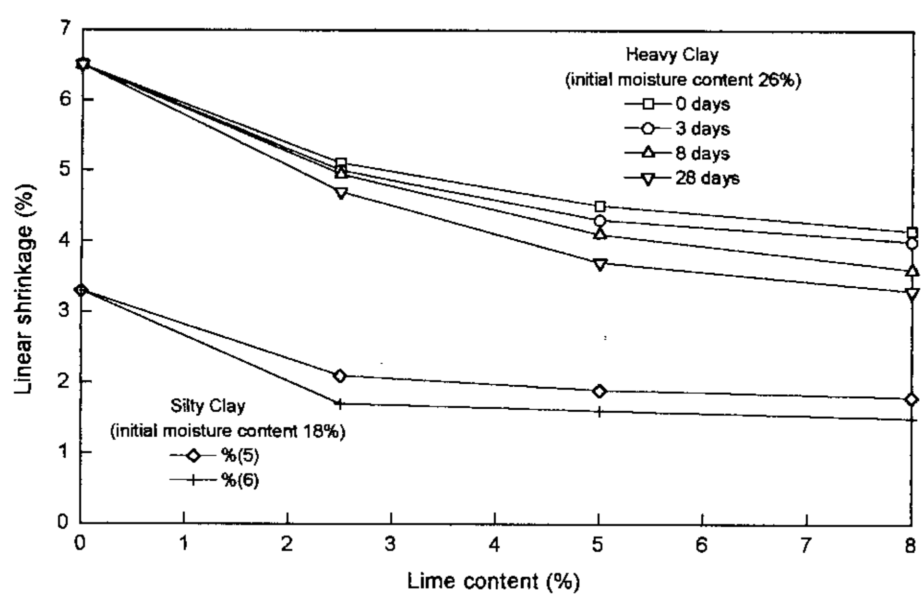

Fig. 2.26 Effect of lime content on linear shrinkage of clays (reproduced after Bell, 1988)

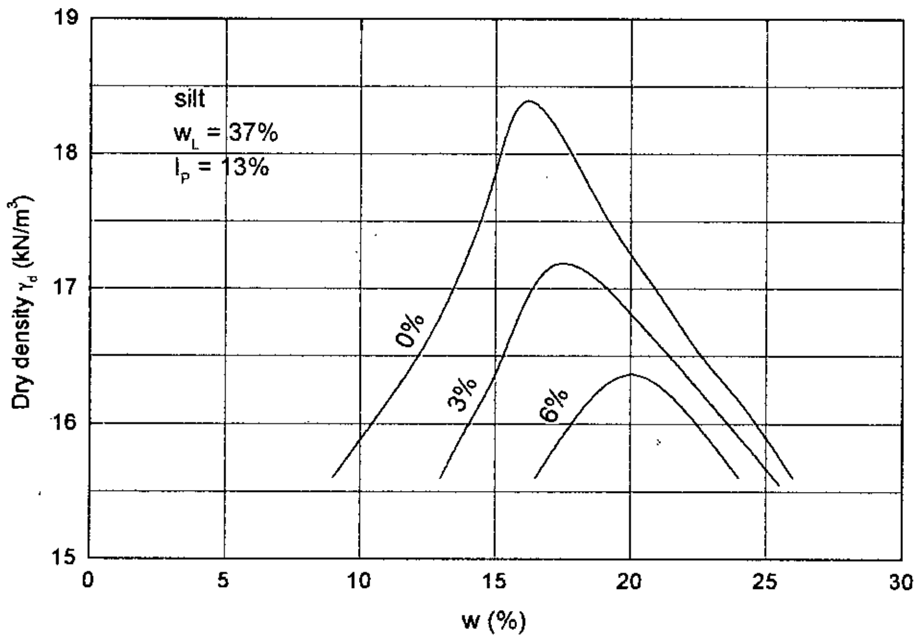

Fig. 2.27 Effect of lime content on maximum dry density and optimum moisture content of a lime-treated silt (reproduced after Kezdi, 1979)

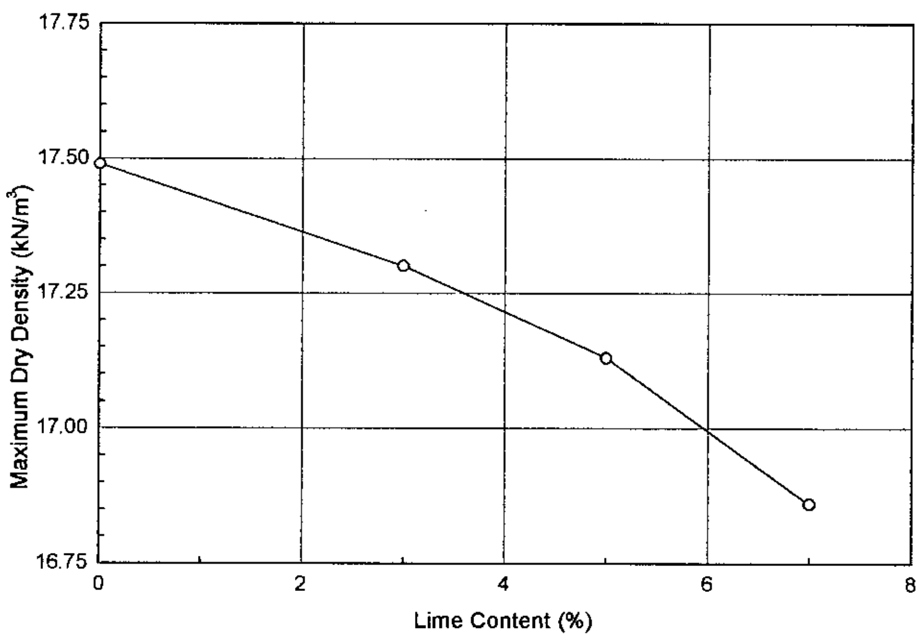

Fig. 2.28 Effect of lime content on maximum dry density of a lime-treated coastal soil (reproduced after Rajbongshi, 1997)

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. Shahjahan (2001) 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):

$$SDI = \frac{\textit{Strength of stabilised sample - Strength of untreated sample}}{\textit{Strength of untreated sample}} \tag{2.16}$$

Plotting 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 molding moisture content on unconfined compressive strength of lime-treated samples. Unconfined compressive strength of samples was found to increase with increasing molding 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.

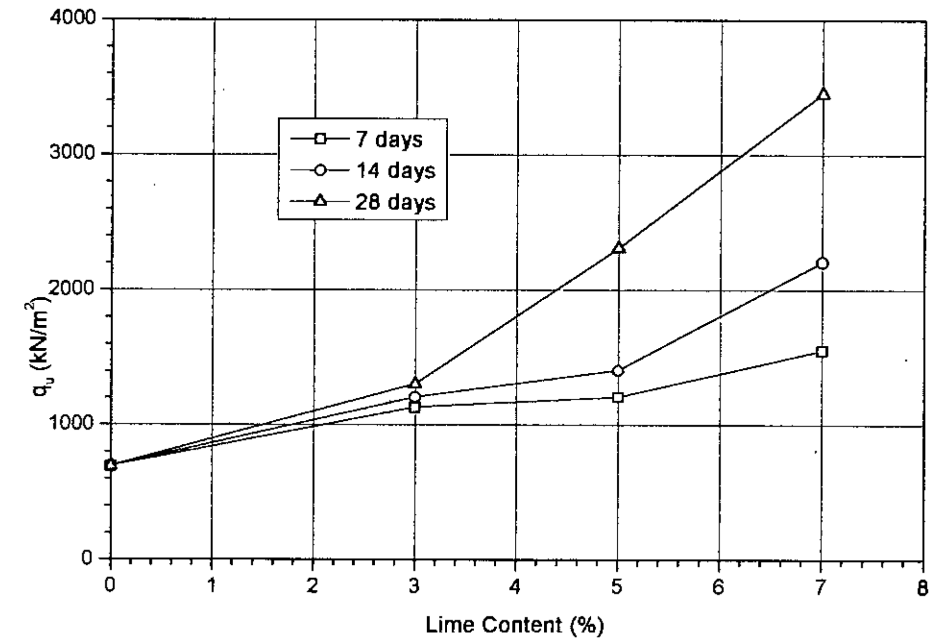

Fig. 2.29 Effect of lime contents on unconfined compressive strength (qu) of a coastal soil at different curing age (reproduced after Rajbongshi, 1997)

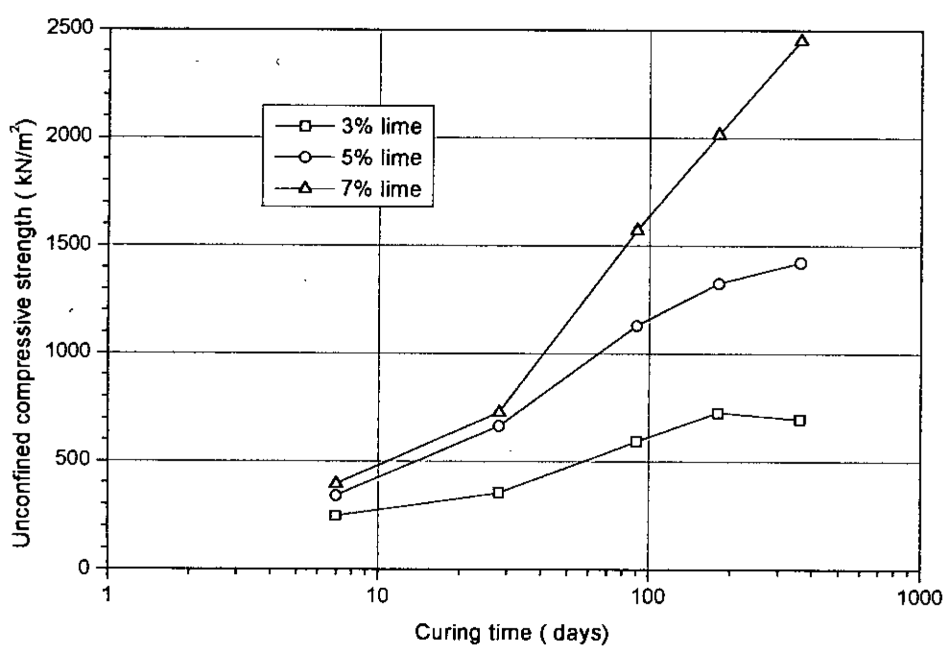

Fig. 2.30 Effect of curing age on unconfined compressive strength (q_u) of a soil (Type-ML/CL) at different lime content (reproduced after Shahjahan, 2001)

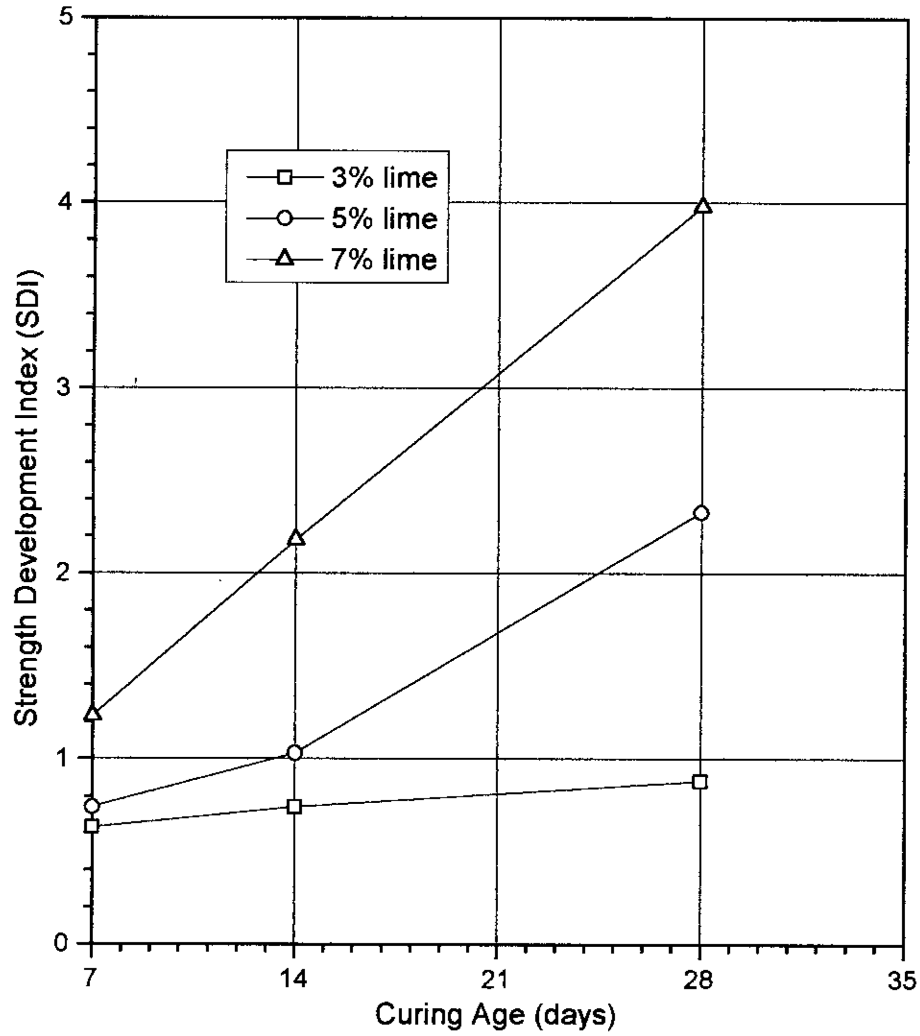

Fig. 2.31 SDI versus curing age curves for samples of a lime-treated coastal soil
(reproduced after Rajbongshi, 1997)

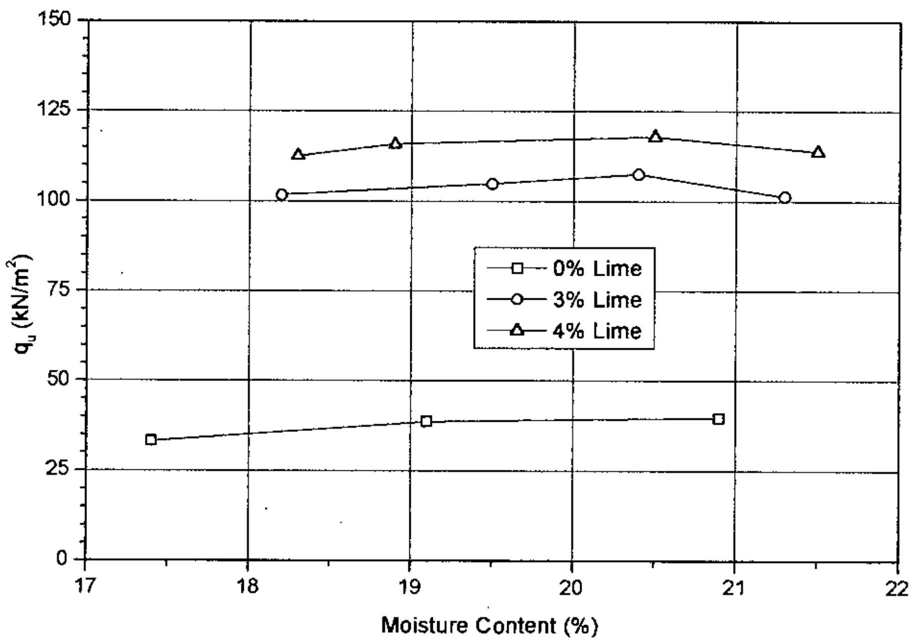

Fig. 2.32 Variation of unconfined compressive strength (qu) with moulding moisture content for a lime-treated silty clay soil (reproduced after Molla, 1997)

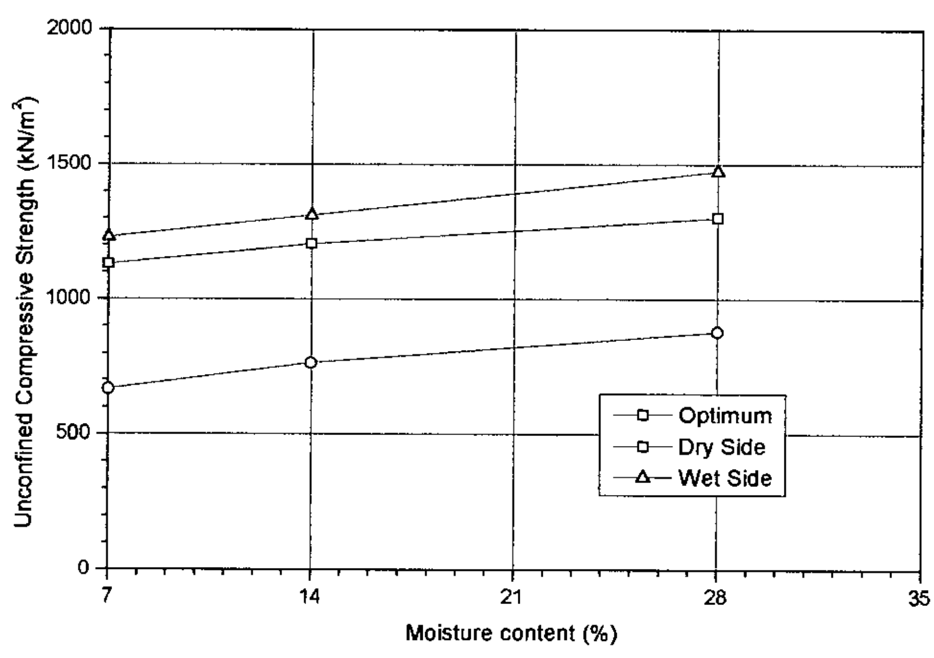

Fig. 2.33 Variation of q_u with curing age for a coastal soil treated with 3% lime and compacted at different moulding water contents (reproduced after Rajbongshi, 1997)

2.12.4 CALIFORNIA BEARING RATIO (CBR)

The CBR test has been extensively used to evaluate the strength of lime stabilized 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 stabilized 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 stabilized 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 (471 kN-m/m^3), medium compaction (1178 kN-m/m^3) and high compaction (2638 kN-m/m^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.

2.12.5 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:

$$ST = 6.89 + 50.6\, q_u \qquad (2.17)$$

Where, ST is the tensile strength in pounds per square inch and qu is the unconfined compressive strength in kips per square inch.

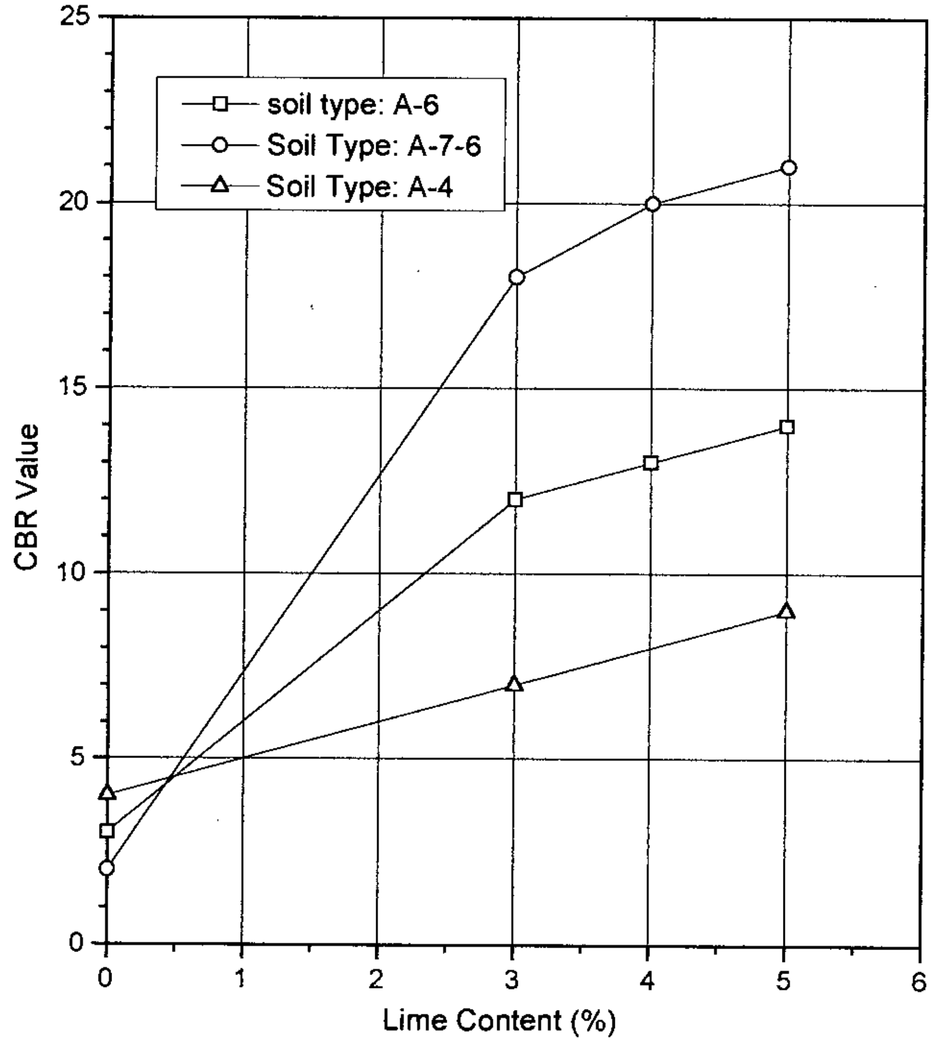

Fig. 2.34 Variation of CBR value with lime content for three regional soils
(reproduced after Molla, 1997)

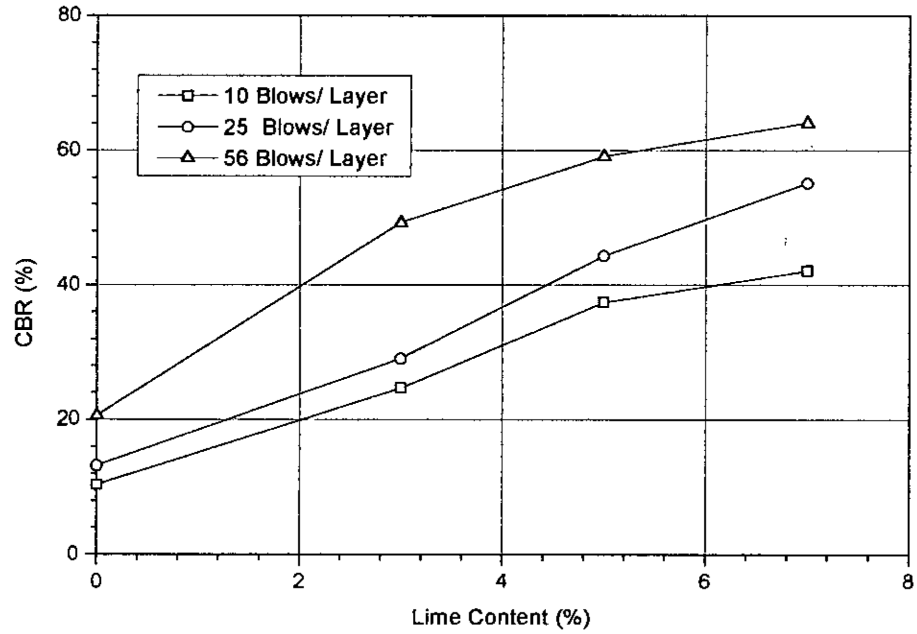

Fig. 2.35 Effect of lime content on CBR values of a coastal soil (reproduced after Rajbongshi, 1997)

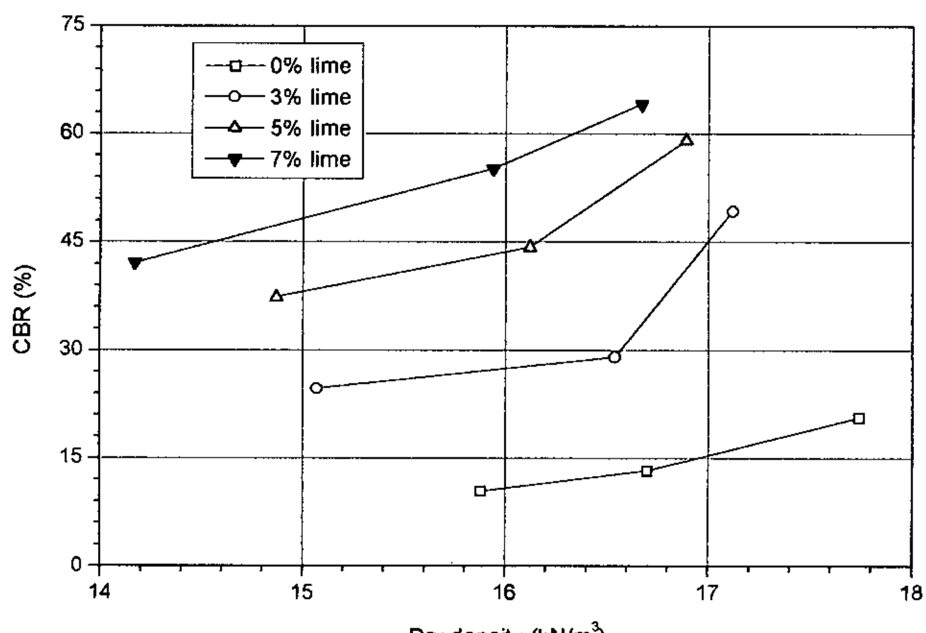

Fig. 2.36 CBR versus dry density curves of a lime-treated coastal soil (reproduced after Rajbongshi, 1997)

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 percent of ultimate strength, will decrease and the fatigue life of the mixture will increase.

The flexural properties of untreated and stabilized samples of a coastal soil has 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.

2.12.6 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.

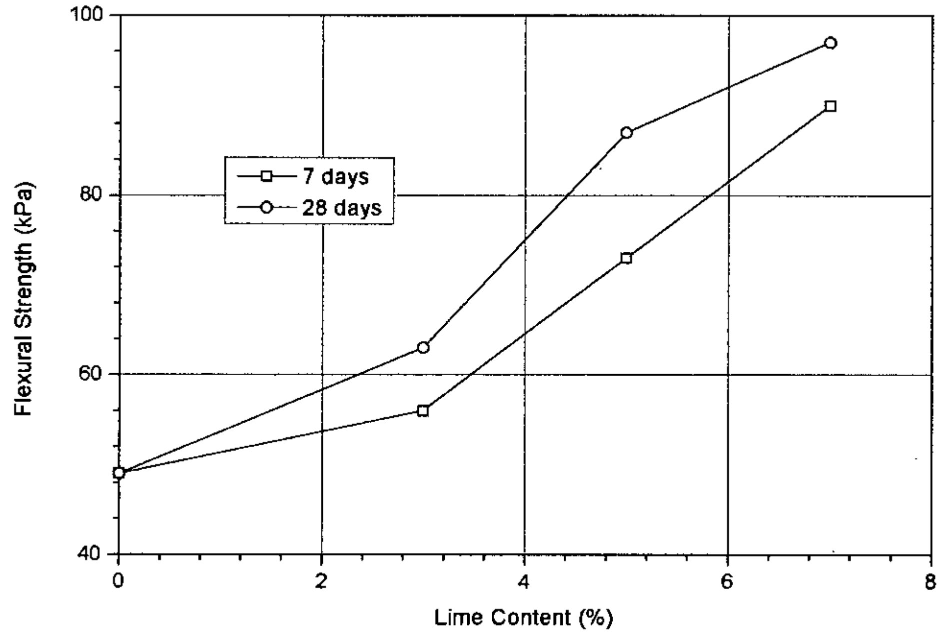

Fig. 2.37 Effect of lime content on flexural strength of a coastal soil
(reproduced after Rajbongshi, 1997)

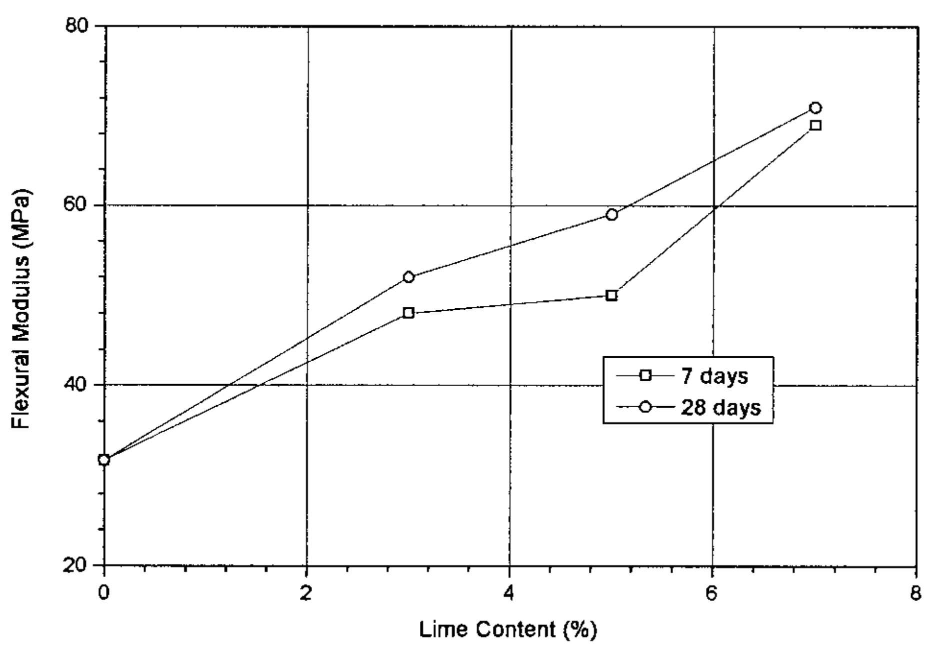

Fig. 2.38 Effect of lime content on flexural modulus of a coastal soil
(reproduced after Rajbongshi, 1997)

2.13 APPLICATIONS OF LIME STABILIZATION

The principal use of the addition of lime to soil is for subgrade and sub-base stabilization and as a construction expedient on wet sites where lime is used to dry out the soil. As far as lime stabilization for roadways is concerned, stabilization is brought about by the addition of between 3 and 6% lime (by dry weight of soil). When lime stabilization 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 necessarily 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 cohesion less 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 stabilized with lime. The range of materials for sub grade, sub-base and base course that can be treated with lime or cement are fairly similar. Lime stabilization 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 stabilize small earth dams constructed of dispersive soil and so avoid piping failure. Lime has also been used to stabilize low-angled slopes, a surface layer of soil about 150 mm thick being mixed in place.

CHAPTER 3

EXPERIMENTAL INVESTIGATIONS

3.1 GENERAL

The investigations in the laboratory were conducted on the untreated and stabilised samples of the two reclaimed soil samples collected from two housing projects (Aminbazar and Bashundhara) of Dhaka are discussed in details in this chapter.

3.2 SAMPLING AND COLLECTION OF SOIL SAMPLES

The present investigations are carried out on two disturbed reclaimed soils collected from Aminbazar and Bashundhara of Dhaka City. These sites are shown in Fig. 3.1. Soil sampling was carried out according to the procedure outlined in ASTM D420-87. For each location, approximately 2 m by 2 m area was excavated to a depth of 2 m to 3 m using hand shovels. Water table was below excavated pit. 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 samples were packed in large polythene bags covered by gunny bags and were eventually transported to the Geotechnical Engineering Laboratory of Bangladesh University of Engineering and Technology, Dhaka. The natural moisture content of samples were 17% to 21%. The soil samples were designated as follows:

Soil-A : collected from Aminbazar

Soil-B : collected from Bashundhara

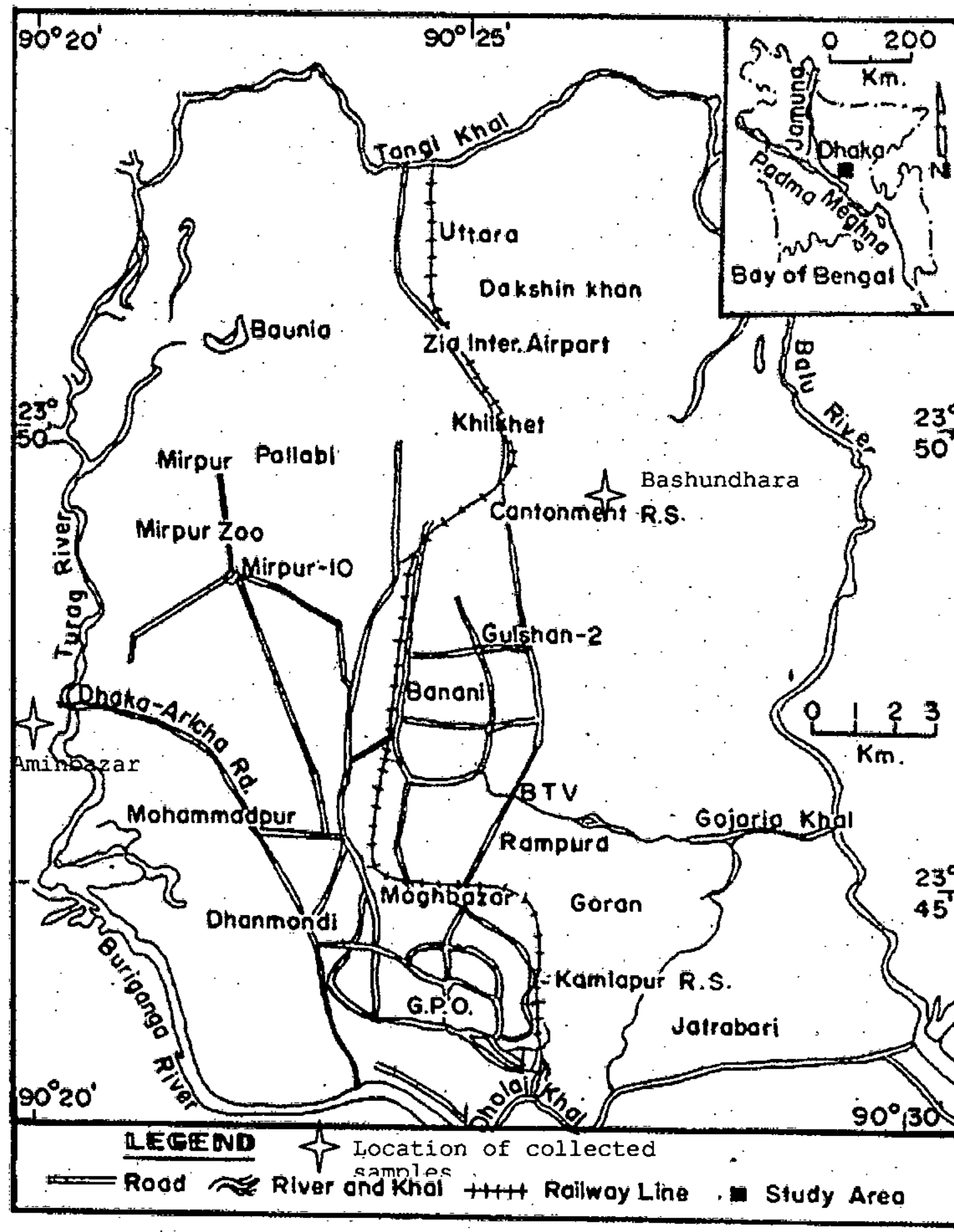

Fig 3.1 Location map of collected samples

3.3 LABORATORY TESTING PROGRAMME

In order to examine the physical, index and engineering characteristics of the untreated soils and soils stabilised with cement and lime a comprehensive laboratory investigation programme was undertaken. Portland cement type-I and air-slaked lime were used as additives for stabilisation. Soil-A were stabilised with Portland cement in percentages of 1, 3 and 5 and Soil-B were stabilised with Portland cement in percentages of 1, 3, 5 and 7 while Soil-B was treated with lime only in percentages of 1, 3, 5 and 7. The whole laboratory-testing programme consisted of carrying out the following tests on samples of the two reclaimed soils:

(i) Index property tests on samples of the two reclaimed soils without any treatment. Index tests included specific gravity test, Atterberg limit tests, linear shrinkage test and grain size analysis. Atterberg limit tests and linear shrinkage tests on samples of the two soils stabilised with different cement and lime contents were also performed.

(ii) The following tests on Soil-A and Soil-B without any treatment, Soil-A stabilised with three different cement contents (1%, 3% and 5%) and Soil-B stabilised with four different cement contents (1%, 3%, 5% and 7%) and also stabilised with four different lime contents (1%, 3%, 5% and 7%):

(a) Modified compaction test

(b) Unconfined compressive strength test on moulded cylindrical samples of 2.8 inch (71 mm) diameter by 5.6 inch (142 mm) high

(c) California Bearing Ratio (CBR) test

(d) Flexural strength test using simple beam with third point loading system

(iii) Wetting and drying test on hardened samples of Soil-A stabilised with three different cement contents (1%, 3% and 5%) and Soil-B stabilised with four different cement contents (1%, 3%, 5% and 7%)

(iv) Absorption test on hardened samples of Soil-A stabilised with three different cement contents (1%, 3% and 5%) and Soil-B stabilised with four different cement contents (1%, 3%, 5% and 7%)

Unconfined compressive strength tests and flexural strength tests using simple beam with third point loading were carried out on cement and lime stabilized samples cured at these different ages (7 days, 14 days and 28 days) while the flexural strength tests using simple beam with third point loading were carried out cement and lime stabilized samples cured at 7 days and 28 days. CBR tests were carried out on the untreated samples and samples treated with different cement and lime contents using three levels of compaction. Absorption tests were carried out on the portion of the samples used in the flexural strength tests. In order to investigate the effect of moulding water content on the compressive strength, unconfined compression strength tests were also carried out on Soil-A and Soil-B treated with 3% cement and 3% lime content on soil-B and compacted according to the Modified Compaction test with two moulding water contents corresponding to 95% of maximum dry density at dry side of optimum moisture content and corresponding to 95% of maximum dry density at wet side of optimum moisture content. Details of laboratory testing programme showing the tests carried out, type of samples tested and number of tests performed are presented in Table 3.1

Table 3.1 Details of laboratory tests performed on samples of the two reclaimed soils

Type of Test	Sample	No. of Tests	
		Soil - A	Soil - B
Specific Gravity of Solids	Untreated soil	1	1
Liquid Limit and Plastic Limit	Untreated soil	1	1
	Soil-cement mixture	3	4
	Soil-lime mixture	-	4
Shrinkage Limit and Linear Shrinkage	Untreated soil	1	1
	Soil-cement mixture	3	4
	Soil-lime mixture	_	4
Particle Size Distribution	Untreated soil	1	1
Modified Compaction Test	Untreated soil	1	1
	Soil-cement mixture	3	4
	Soil-lime mixture	-	4
Unconfined Compression Test (Curing age of 7, 14 and 28 days)	Untreated soil	1	1
	Soil-cement mixture	9	12
	Soil-lime mixture	-	12
CBR Test (4 days soaked) at Three Levels of Compaction	Untreated soil	3	3
	Soil-cement mixture	9	12
	Soil-lime mixture	-	12
Flexural Strength Test using Simple Beam with Third Point Loading System (Curing age of 7 and 28 days)	Untreated soil	1	1
	Soil-cement mixture	6	8
	Soil-lime mixture	-	8
Wetting and Drying Test	Soil-cement mixture	3	4
Absorption Test	Soil-cement mixture	6	8
Moulding water content adding 3% cement and lime at 95% compaction at wet and dry side	Soil-cement mixture	6	6
	Soil-lime mixture	-	6

3.4 PHYSICAL AND INDEX PROPERTIES OF UNTREATED SOILS

The samples collected from the field were disturbed samples. These samples were then air-dried for about three months and the soil lumps were broken carefully with a wooden hammer so as to avoid breakage of soil particle. The required quantities of soil were then sieved through sieve No. 40 (0.425 mm). The following Standard test procedure were followed in determining the physical and index properties of the untreated soils:

Specific gravity	ASTM D854
Liquid limit (Cone Penetrometer Method)	BS 1377
Plastic limit and plasticity index	BS 1377
Shrinkage limit	ASTM D427
Linear shrinkage	BS 1377
% of material in soils finer than No. 200 sieve	ASTM D1140
Grain size distribution	ASTM D422

The grain size distribution curves of the samples of the two reclaimed soils are presented in Fig. 3.2. The different fractions of sand, silt and clay of samples of Soil-A and Soil-B 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). The soils were also classified according to AASHTO Soil Classification System (AASHTO M145-49). Table 3.2 presents the values of index and shrinkage properties, grain size distribution and classifications of Soil-A and Soil-B.

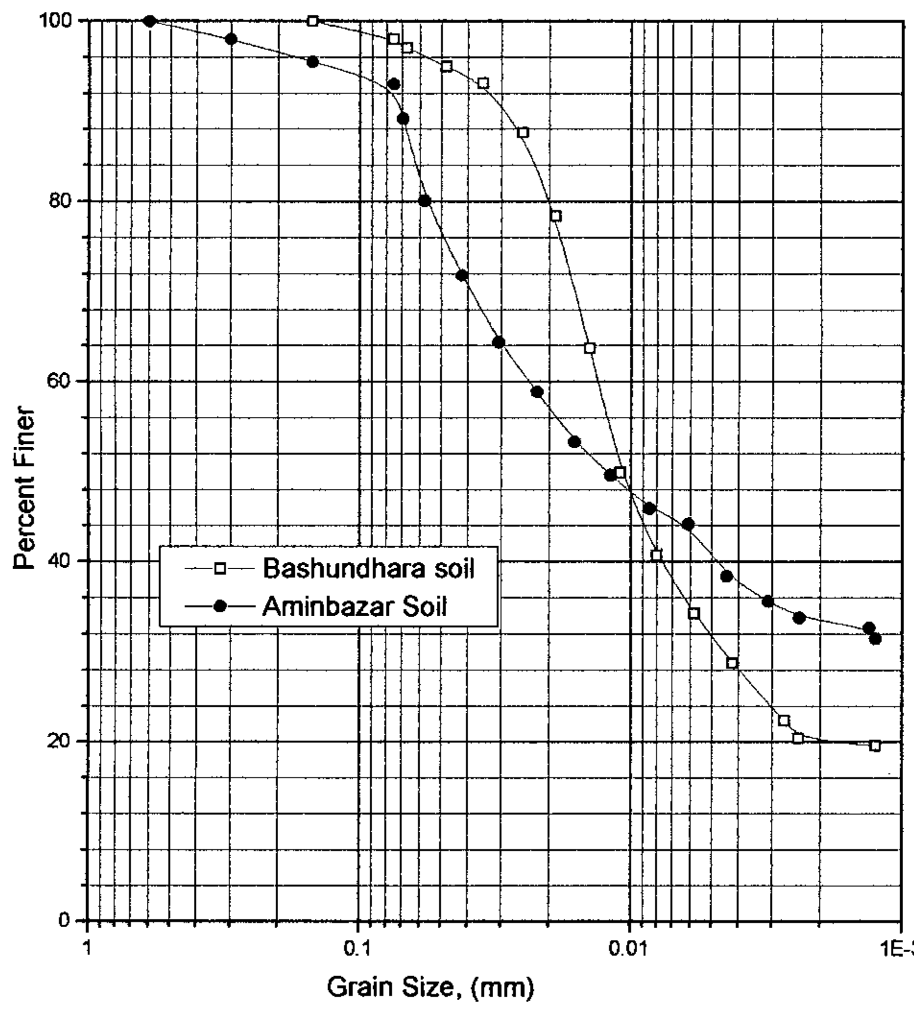

Fig: 3.2 Grain size distribution curves for soil-A and soil-B.

Table 3.2 Index properties and classification of the reclaimed soils used

Index Properties and Classification	Soil-A	Soil-B
Specific Gravity	2.74	2.67
Liquid Limit	41%	52%
Plastic Limit	29%	23%
Plasticity Index	12%	29%
Shrinkage Limit	25.5%	22%
Linear Shrinkage	10%	13%
% Sand (2 mm to 0.075 mm)	8%	2%
% Silt (0.075 mm to 0.002 mm)	58.5%	78%
% Clay (<0.002 mm)	33.5%	20%
% of Material Finer than No. 200 Sieve	92%	98%
Unified Soil Classification	ML	CH
AASHTO Soil Classification	A-5	A-7-5

3.5 PROPERTIES OF CEMENT USED FOR SOIL STABILISATION

For this research, ordinary Portland cement Type-I has been used for the stabilisation of Soil-A and Soil-B. For the determination of normal consistency of cement paste, setting times (initial and final setting times) of cement pastes and compressive strength of 50 mm (2 inch) cube specimens, the standard test procedures outlined in ASTM C187, C191 and C109 respectively were followed. The results of these tests are shown in Table 3.3. In this research, hydrated lime (i.e., slaked lime), which is commercially available in the market, has been used for the stabilisation of Soil-B.

Table 3.3 Test results of ordinary Portland cement (Type-I)

Properties		Results
Water for Normal Consistency		21.5 per cent
Setting Time	Initial setting time	135 minutes
	Final setting time	255 minutes
Compressive Strength	3 days	1625 psi
	7 days	2575 psi

3.6 INDEX PROPERTY TESTS ON STABILISED SOIL SAMPLES

Liquid limit, plastic limit, plasticity index and shrinkage characteristics including shrinkage limit and linear shrinkage of samples of the two reclaimed soils (from Aminbazar and Bashundhara) stabilised with cement and lime were determined. Portland cement Type-I and hydrated lime (i.e., slaked lime) were used as additives. Portland cement was used in percentages of 1, 3 and 5 while the lime contents were used in percentage of 1, 3, 5 and 7. Liquid limit, plastic limit and plasticity index of the stabilised samples were carried out on air-dried pulverised samples. The required quantities of pulverised soil were sieved through sieve No. 40 (0.425 mm). The cement and 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 and after mixing of water the samples were kept 24 hours in polythene bags to bring uniform moisture content in soils. Liquid limit, plastic limit and plasticity index of the stabilised samples were determined following the standard procedure outlined in BS 1377 and ASTM D424 respectively. The shrinkage factor comprising the shrinkage limit was determined in accordance with the procedure specified in ASTM D427. Linear shrinkage of the cement and lime treated samples were determined following the procedure outlined in BS 1377. Fig 3.3 shows the photograph of hydrometer tests.

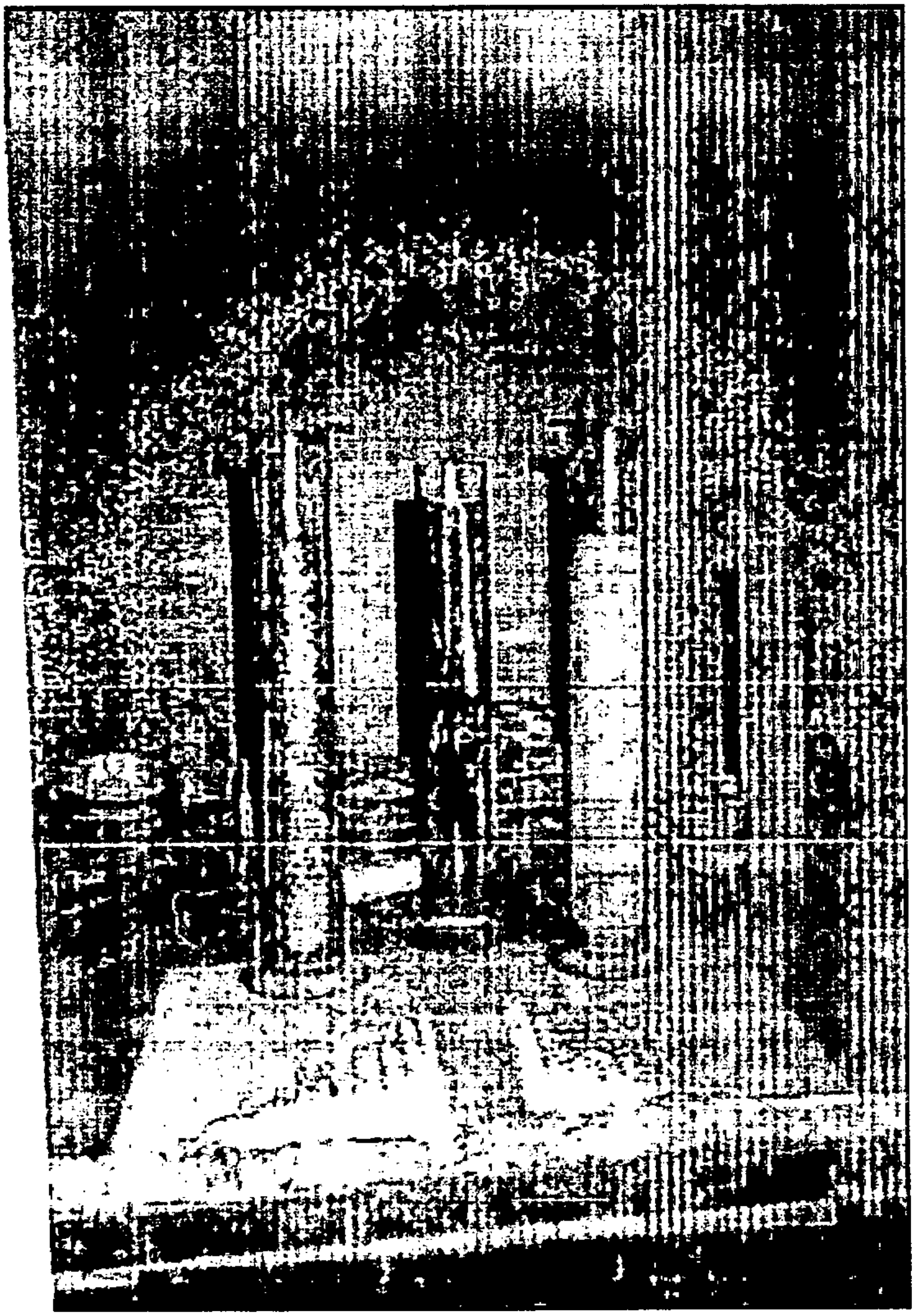

Fig.3.3 Photograph showing the Hydrometer Test of two samples.

3.7 COMPACTION TEST

The moisture content versus dry density relationships of the untreated samples of the two reclaimed soils were investigated by carrying out Modified Compaction test. These tests were performed according to the standard procedure outlined by ASTM D1557. Air-dried samples passing through No. 4 sieve was used for compaction. For compaction of the moist samples, a cylindrical mould of 6 inch (152.4 mm) inside diameter and of volume 0.075 ft^3 was used. A series of moist samples of varying moisture contents were 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 falling from a free height of 18 inch (457 mm). The amount of material used was such that the fifth compacted layer was slightly above the top of the mould but not exceeding 6 mm. During compaction the mould was placed on an uniform rigid foundation. Finally, moisture content and dry density determinations were made on each compacted sample of Soil-A and Soil-B. Fig 3.4 shows the photograph of compaction test.

For the cement and lime treated samples of the two reclaimed soils, samples for moulding specimens were prepared according to the procedure outlined in ASTM D558. A series of soil-cement and soil-lime samples of varying moisture contents were prepared. These samples were subsequently compacted in a cylindrical mould of 6 inch (152.4 mm) inside diameter and of volume 0.075 ft^3 in accordance with the above procedure as outlined in ASTM D1557. The different cement contents used for preparing samples were 1%, 3% and 5% for soil-A and 1%, 3%, 5% and 7% for soil-B while lime contents of 1%, 3%, 5% and 7% were used for soil-B. Finally, moisture content and dry density determinations were made on each of the compacted stabilized sample of Soil-A and Soil-B.

3.8 UNCONFINED COMPRESSIVE STRENGTH TEST

3.8.1 PREPARATION AND MIXING OF SOILS

Untreated Soil-A and Soil-B were 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 quantities of pulverized soil were then sieved through sieve No. 4 (4.76 mm). All the soil retained on this sieve was discarded. Representative soil

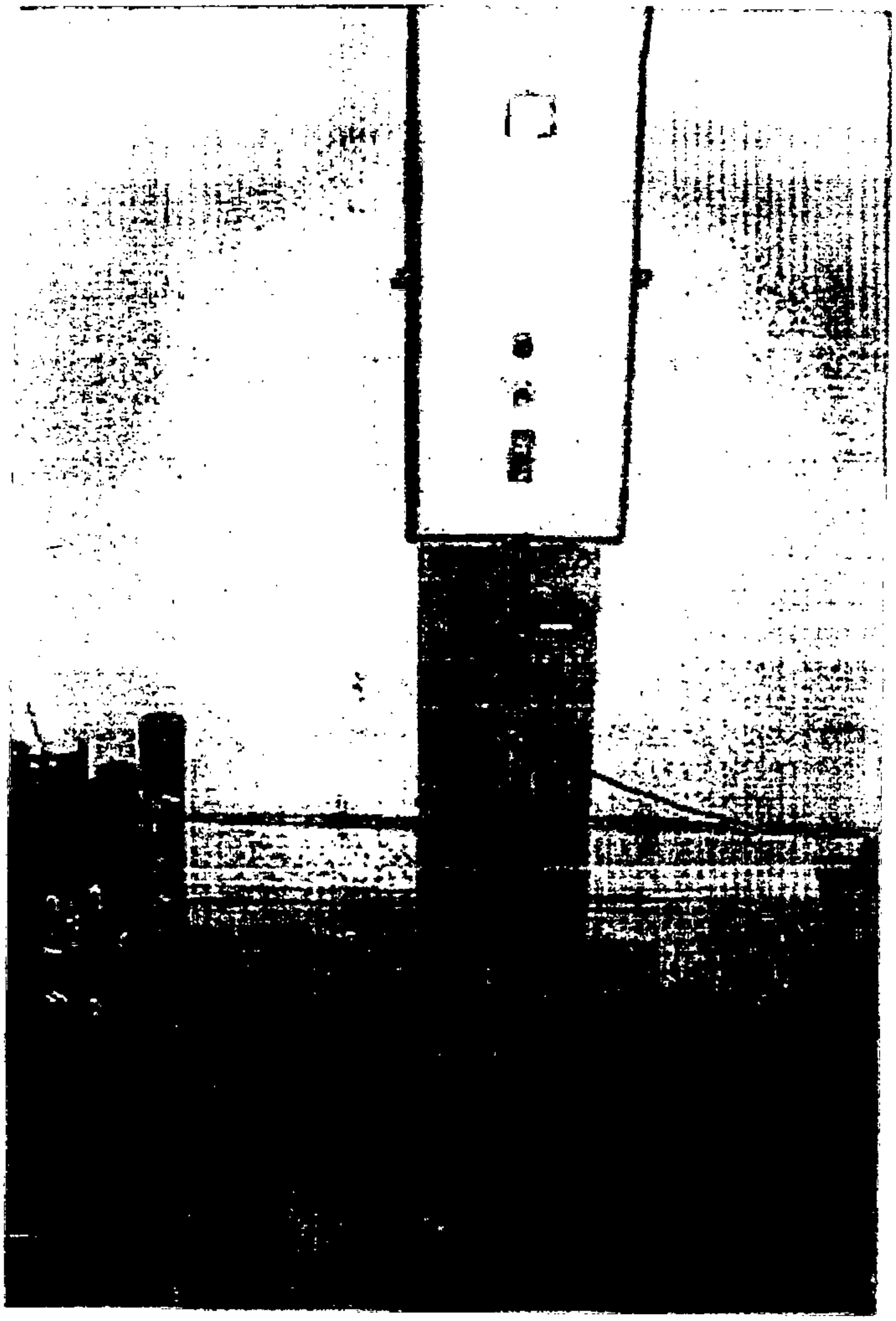

Fig. 3.4 Photograph showing the compaction test apparatus.

sample of required quantity was taken to prepare test sample of desired density, i.e., the maximum dry density obtained in the Modified Compaction test. Moisture content of air-dry soil sample was determined. Portland cement was used in percentage of 1, 3 and 5 for Soil-A and 1, 3 5 and 7 for Soil-B and the lime content in percentages of 1, 3, 5 and 7 were used for Soil-B. The percentages of the additives were calculated on the basis of air-dry (three months) weight of the soil samples. Soils were mixed by hand with cement and lime in a laboratory mixer in batches. This mixing was carried out in a steel pan. Water was added on the basis of its respective optimum moisture content, getting from compaction test of various cement and lime content. Water added into the soil mass until it was thoroughly blended. In order to attain the required design moisture content for compaction, the water required in addition to air-dry state was calculated and with this additional water required for hydration was added to the soil and additives. For the hydration of cement extra water required was 38 per cent by weight of the cement (Shetty, 1982) and for lime it 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 as obtained from the Modified Compaction tests for the untreated soils and soils stabilized with different cement and lime contents.

3.8.2 MOULD FOR COMPRESSION TEST

The mould used for compacting untreated soil, soil-cement and soil-lime mix were fabricated using locally available mild steel seamless pipe. The mould complies 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 samples of soil-cement and soil-lime in the laboratory under accurate control of quantities of materials and test conditions. The design and dimensions of the mould are shown in Fig. 3.5. Mould having an inside diameter of 2.8 ± 0.01 in. (71 ± 0.25 mm) and a 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. All together six mould with necessary accessories were fabricated for this research work.

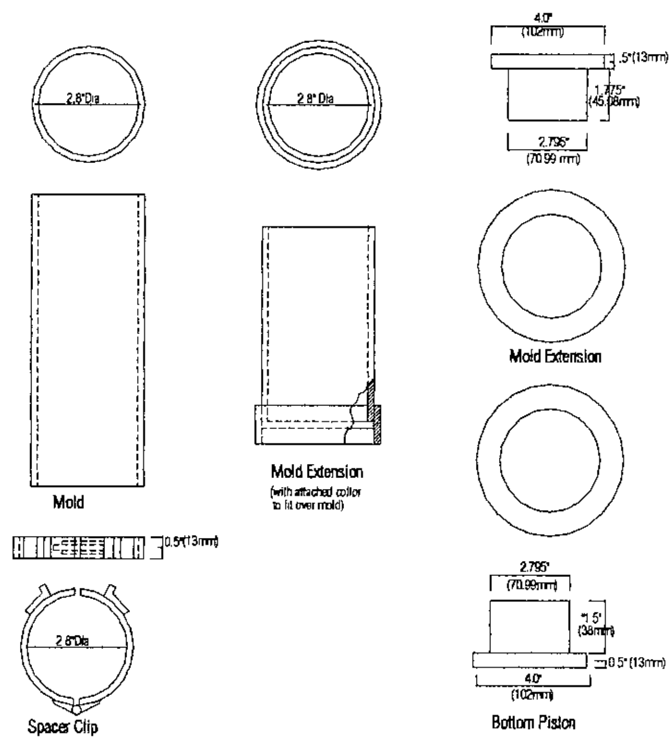

Fig. 3.5 The design and dimensions of the compression test mould.
(reproduced after ASTM, 1989)

3.8.3 COMPACTION OF SAMPLES

Compression test samples of untreated and treated soils were prepared with the cylinder of size with 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 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 ½-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).

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 Modified Compaction test performed according to the standard procedures outlined in ASTM D1557.

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 at normal room temperature (19 C-25 C) on a level table covered with wetted jute Hessian cloth to maintain moist condition. Every day the wetted cloth checked and when it became dry, it again wetted and covered the samples. 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 7, 14, and 28 days. It may be mentioned that the soil samples which were prepared without adding cement or lime, i.e., the untreated samples were not cured.

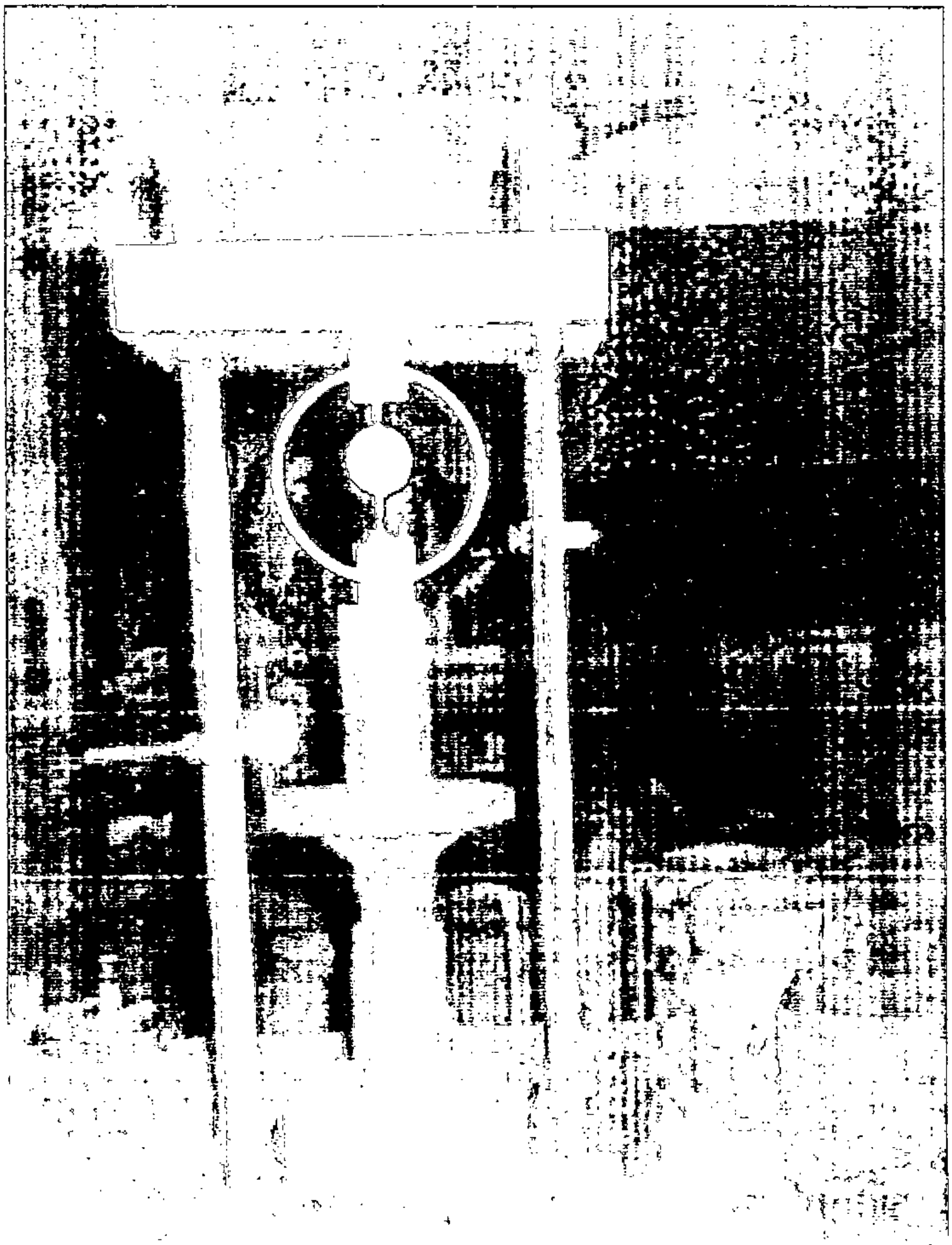

Fig. 3.6 Photograph showing the compression test of a sample.

3.8.5 COMPRESSION TEST

The stabilized samples were placed on the compression-testing machine directly 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. 3.6 presents photograph of the compression test apparatus showing a sample being tested.

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

3.9.1 PREPARATION AND MIXING OF SOILS

The untreated and soils treated with various cement and lime contents were prepared and mixed in accordance with the procedure outlined in section 3.9.1 For the stabilised samples, Portland cement was used in percentage of 1, 3 and 5 for Soil-A and 1, 3, 5 and 7 for Soil-B and the lime content in percentages of 1, 3, 5 and 7 were used for Soil-B. The design moisture content of the untreated samples and samples stabilised with cement and lime were equivalent to the respective values of optimum moisture contents as obtained from the Modified Compaction tests (ASTM D1557) for the untreated soils and soils stabilised with different cement and 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

Fig. 3.7 Photograph showing CBR test of a sample.

samples and samples treated with cement and lime using another two levels of compaction energies equivalent to 10 and 25 blows in five approximately equal layers with a rammer of weight 10 lbs and 18 inches free fall and compacted in a mould of volume 0.075 cft. After the 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.

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, which was 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. Fig 3.7 swows the photograph showing soaking of CBR test samples. 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.

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. Fig. 3.7 presents a photograph of the CBR test apparatus.

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 cement and lime contents were prepared and mixed in accordance with the procedure outlined in section 3.9.1 For the stabilised samples, Portland cement was used in percentages of 1, 3 and 5 for Soil-A and 1, 3, 5 and 7 for Soil-B and the lime content in percentages of 1, 3, 5 and 7 were used for Soil-B. The design moisture content of the untreated samples and samples stabilised with cement and lime were equivalent to the respective values of optimum moisture contents as obtained from the Modified Compaction tests (ASTM D1557) for the untreated soils and soils stabilised with different cement and lime contents.

3.10.2 MOULD FOR FLEXURE TEST

The mould used for compacting untreated soil, soil-cement and soil-lime mixtures were fabricated using locally available mild steel plates, which comply with the requirements of ASTM D1632. 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 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. 3.7. 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 0.375 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. Fig 3.9 and Fig. 3.10 shows the schematic diagrams of flexure test apparatus.

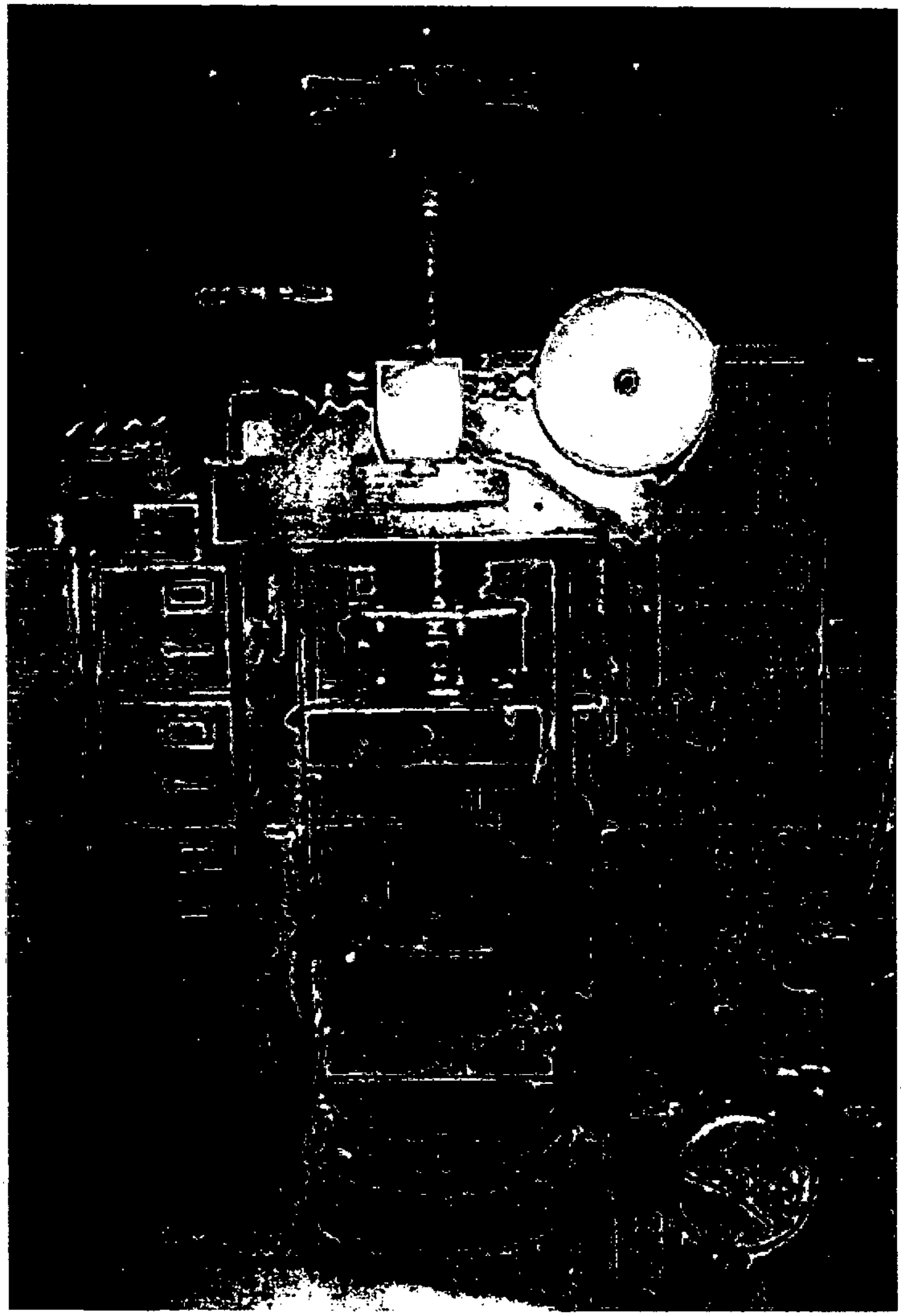

Fig. 3.8 Photograph showing the mould for flexure test.

3.10.3 MOULDING AND CURING OF SAMPLES

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. (10 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 Modified 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-in. (13 mm) diameter smooth steel rod repeatedly through the mixture from the top down to the point of refusal. Approximately 90 rodding were distributed uniformly over the cross section of the mould. These layers of compacted sample were levelled by hand and layer two and three were compacted in the similar way. The sample at this time was made approximately 3¾ in. 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-cement and soil-lime samples were removed from the mould they were kept in a ice box 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 7 and 28 days. The soil samples, which were prepared without adding cement or lime were not cured. The treated samples were carried for testing purpose directly from the moist curing environment.

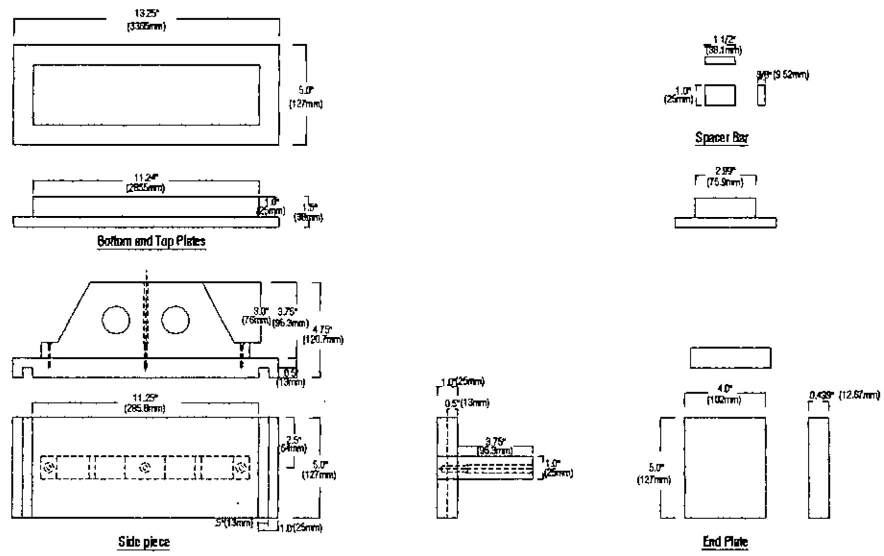

Fig. 3.9 A schematic diagram of mould for preparation of specimen for flexural strength test of soil, soil-cement and soil-lime samples by third point loading. (reproduced after ASTM, 1989)

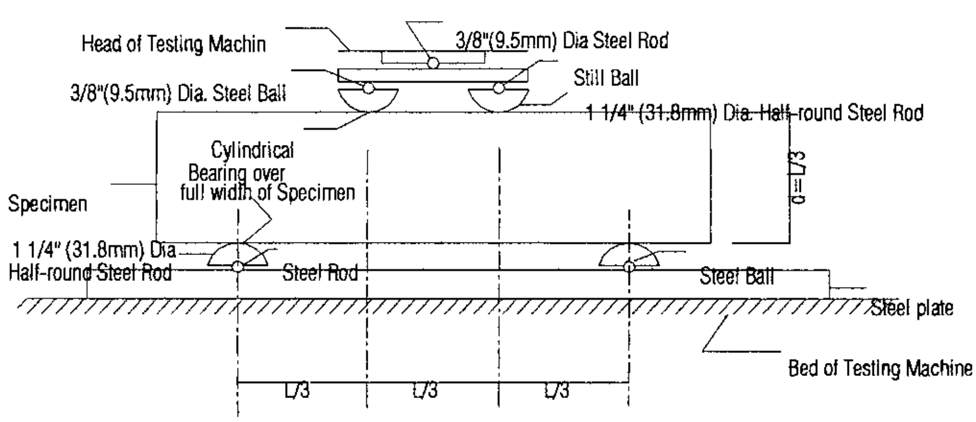

Fig. 3.10 A schematic diagram of the apparatus for flexural strength test of soil, soil-cement samples by third point loading. (reproduced after ASTM, 1989)

3.10.4 FLEXURAL STRENGTH TEST

The flexure tests of untreated soil, soil-cement 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 in. by 3 in. by 11¼ 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 in.). 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 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. Fig. 3.8 shows the mould for preparation of specimen for flexural strength test and a schematic diagram of the apparatus for flexure test of soil, soil-cement and soil-lime samples by third point loading is shown in Fig. 3.9 and Fig 3.10.

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:

$$R=\frac{PL}{bd^2} \tag{3.1}$$

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:

$$R = \frac{3Pa}{bd^2} \qquad (3.2)$$

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, soil-cement and soil-lime beam samples, as found from flexural strength tests were calculated using the following expression of simple beam theory:

$$E = \frac{23PL^3}{1296I\Delta} \qquad (3.3)$$

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

3.11 ABSORPTION TESTS ON SOIL-CEMENT

Absorption tests were conducted on the portion of the soil-cement beams, which were used in flexural strength tests. As soon as the flexure tests were completed, the broken parts of the beam were taken to determine the absorption capacity of the soil-cement. First, making the straight edge of the broken side with the help of a saw made the broken parts to a regular shape and then the sample was marked and weighed. After weighing the sample was kept in the oven to at 100°C for 24 hours. The sample after taking out from the oven were allowed to cool for 15 minutes and then again weighed. Then the sample was kept immersed in water for 24 hours. The sample was taken out from water and the free water from the surface of the sample was removed and the sample was weighed. Water absorption was calculated on the basis of oven-dry weight of the sample.

3.12 WETTING AND DRYING TEST

This test is carried out in order to assess the reaction of the stabilized soil to the effect of repeated drying and wetting. Wetting and drying tests of soil-cement mix were performed in accordance with the procedure outlined in ASTM D559.

The samples were prepared by compaction following ASTM Method D698. Dimensions of the samples tested were identical to those of the Standard Proctor mould i.e. 4.0 inches in diameter and 4.6 inches height. The air-dried soils were passed through No. 4 sieve. Air-dry moisture content was calculated. For cement-stabilized samples, cement contents of percentages of 1, 3 and 5 by weight of air-dried soil were used. The moisture content taken was that corresponds to optimum moisture content.

In order to attain the required moisture content, the water required in addition to air-dried state was calculated and for cement stabilized soil, an additional amount of water with previous amount for hydration were added to the soil and the admixture. For hydration of cement water required was 38 per cent by weight of cement (Shetty, 1982).

Approximately 8 lbs of soil sample was taken and the required amount of water and admixture were added. The mixture was compacted according to ASTM Standard D698 except that the surface of each compacted layer was roughened prior to the application of the next by scratching a square grid lines 1/8 inch wide and 1/8 inch deep having approximately ¼ inch spacing. During compaction the water content of a representative sample was determined. After compaction, the mould was weighed for determination of density. The compacted sample was then extracted from the mould by an extruder.

Each test required two samples: one for testing the volume and moisture changes while the second one was used for soil-cement loss determination. The ready-made samples were weighed and stored for 24 hours in humid surrounding. Then the samples were cured for 7 days in desiccators, keeping the samples over a filter paper just touching the water below. Weight and dimensions are checked in curing period. Following the 7-day treatment, the samples were submerged in tap water for 5 hours at room temperature, leaving a water layer of 1 inch above them. After removal the weight and dimensions of specimen No. 1 were checked, then both samples were placed into an oven at 100 °C for 42 hours. This was followed by another weight check, then specimen No. 2 was brushed by standard ASTM brush by eighteen to twenty strokes on sides and four on each end. The force applied was 3 lbs and it was done on a consolidation-test-machine platform. Finally, a third weighing was performed to determine the loss in weight.

The operations described above represent a single durability or wetting-drying test cycle. For a sample, a maximum of 12 cycles of wetting and drying were carried out.

Thereafter, volume and moisture change were calculated as a percentage of original volume and moisture content. The soil-cement loss was expressed as a percentage of the original oven dry weight. Fig. 3.11 shows the photograph of compaction of soil-cement loss samples.

Fig. 3.11 Photograph showing mould for wetting and drying test of a sample

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 GENERAL

The findings of the laboratory investigations on the characteristics of untreated and stabilized samples of the two reclaimed soils are presented and discussed in the following sections of this chapter. These results demonstrate the effect of additives, e.g., cement and lime on the physical and engineering properties of the samples investigated. Results of analytical investigations are also presented.

4.2 PHYSICAL AND ENGINEERING PROPERTIES OF CEMENT-TREATED SOILS

In the following sections, the physical and engineering characteristics comprising plasticity and shrinkage properties, moisture-density relations, unconfined compressive strength, California Bearing Ratio (CBR), flexural properties, absorption and durability (e.g., volume change and soil-cement loss) of untreated and cement-treated samples of the two reclaimed soils are presented and discussed. Comparisons of changes in the properties between the untreated and stabilized samples have been made.

4.2.1 PLASTICITY AND SHRINKAGE CHARACTERISTICS

The values of plasticity and shrinkage properties of the untreated and cement-treated soil samples are shown in Tables 4.1 and 4.2 for Soil-A (Aminbazar) and Soil-B (Bashundhara) respectively. It can be seen from Tables 4.1 and 4.2 that compared with the untreated samples of Soil-A and Soil-B, plastic limit of the stabilized samples increased while plasticity index, shrinkage limit and linear shrinkage reduced. Compared with the untreated sample, the value of liquid limit of the treated sample increased in Soil-A while it is reduced in case of Soil-B. Fig. 4.1 shows the variation of liquid limit and plastic limit while Fig. 4.2 shows the variation of plasticity index with the increment of cement addition. It can be seen from Fig. 4.1 that for Soil-A (LL = 41, PI = 12), both liquid limit and plastic limit increased while for Soil-B (LL =52, PI = 25) liquid limit reduced and plastic limit

Table 4.1 Index and shrinkage properties of cement-treated Soil-A (Aminbazar)

Index and Shrinkage Properties	Cement Content (%)			
	0	1	3	5
Liquid Limit	41.0	42.0	44.5	46.5
Plastic Limit	29.0	31.0	34.5	37.0
Plasticity Index	12.0	11.0	10.0	9.0
Shrinkage Limit	25.5	25.0	24.0	22.0
Linear Shrinkage	10.0	7.5	6.5	5.5

Table 4.2 Index and shrinkage properties of cement-treated Soil-B (Bashundhara)

Index and Shrinkage Properties	Cement Content (%)				
	0	1	3	5	7
Liquid Limit	52.0	51.0	48.0	46.5	45.5
Plastic Limit	23.0	25.0	30.0	31.5	32.0
Plasticity Index	29.0	26.0	18.0	15.0	13.5
Shrinkage Limit	22.0	21.5	20.0	19.0	18.0

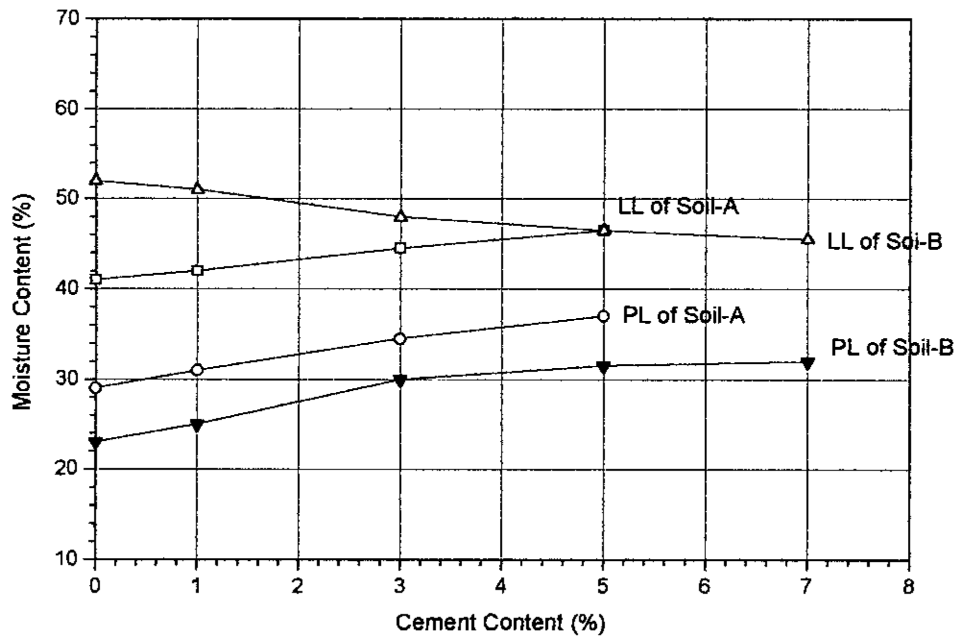

Fig: 4.1 Effect of cement content on liquid limit and plastic limit of soil-A and soil-B.

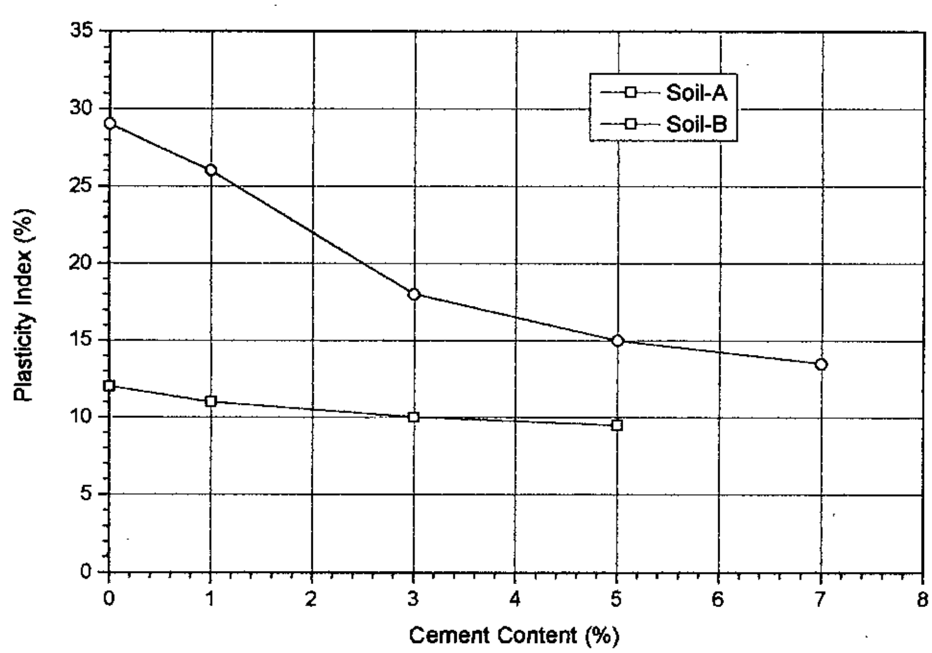

Fig: 4.2 Effect of cement content on plasticity index of soil-A and soil-B.

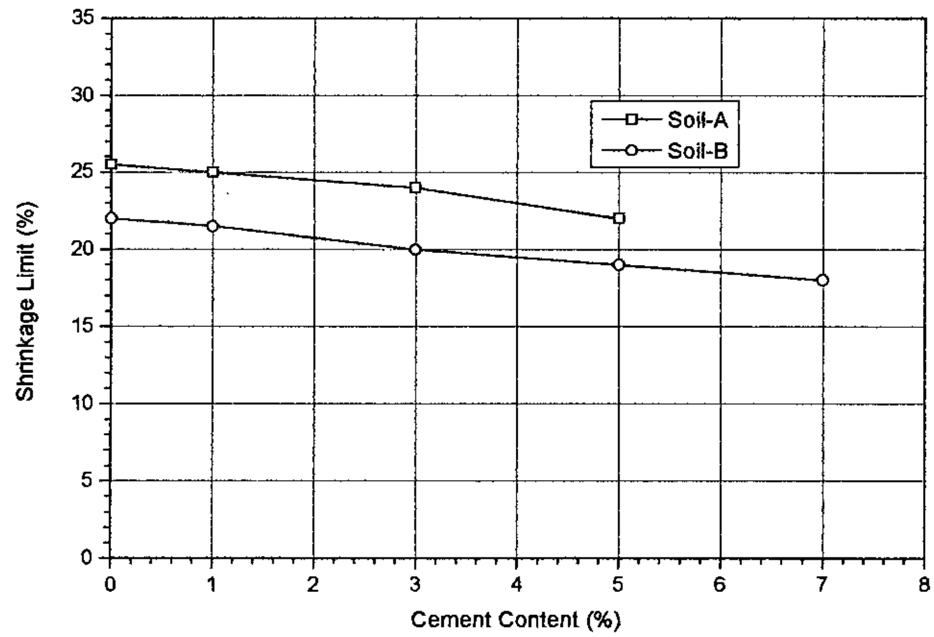

Fig: 4.3 Effect of cement content on shrinkage limit of soil-A and soil-B.

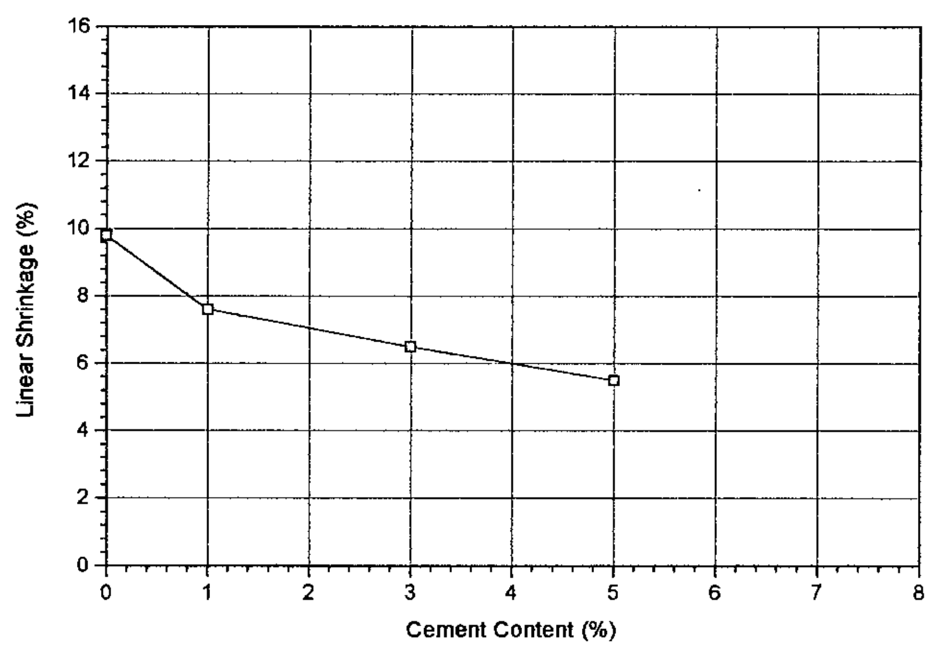

Fig: 4.4 Effect of cement content on linear shrinkage of soil-A.

increased with increasing cement content. These results are in agreement with those reported by Willis (1947), Felt (1955), Ahmed (1984) Rajbongshi (1997) and Hossain (2001). Ahmed (1984) and Hossain (1986) found that with the increase in cement content, both liquid and plastic limit increased while plasticity index reduced for a sandy silt (LL = 40, PI = 10) and a clayey silt (LL = 33, PI = 6) respectively. However, for a silty clay (LL = 44, PI = 19), Rajbongshi (1997) found a reduction in liquid limit and plasticity index, and an increase in plastic limit with increasing cement content.

The changes in shrinkage limit due to increase in cement content are shown in Fig. 4.3 while Fig. 4.4 presents the variation of linear shrinkage with the increase in cement content. It can be seen from Figs. 4.3 and 4.4 that for both the soils shrinkage limit and linear shrinkage reduced slightly with the increase in cement content. Reduction of shrinkage limit with the increase in cement content was reported by Willis (1947), Mara and Up pal (1950) and Jones (1958). Kezdi (1979) reported reduction in linear shrinkage due to increase in cement content in three clayey soils of different plasticity.

4.2.2 MOISTURE-DENSITY RELATIONS

The moisture-density relations of untreated and cement-treated samples of Soil-A and Soil-B are shown in Figs. 4.5 and 4.6 respectively. From the relations presented in Figs. 4.5 and 4.6, the maximum dry density (γ_{max}) and optimum moisture content (w_{opt}) of Soil-A and Soil-B have been determined which are presented in Table 4.3. It can be seen from Table 4.3 that for both the soils, with the increase in cement content, values of γ_{max} increased while the values of w_{opt} reduced. The increase in γ_{max} with the increase in cement content for the two soils is shown in Fig. 4.7. Compared with the untreated sample, the values of γ_{max} increased up to 7.8% for 5% cement of soil-A and 5.9% for 7% cement of Soil-B. The decrease in w_{opt} with the increase in cement content for the two soils is shown in Fig. 4.8. The values of w_{opt} reduced up to 11.1% for 5% cement of soil-A and 12.2% for 7% cement of Soil-B. Kezdi (1979) reported that with the addition of cement, maximum dry density of sand, fat clays and silts increase while optimum moisture content reduces for sands and silts. Felt (1955) also reported that for sand and sandy soils the density increases with the increasing cement content. Rajbongshi (1997) found that for coastal soils of Bangladesh, the maximum dry density increased with increase in cement content. Hossain (1986), however, found reduction in maximum dry density with increasing cement content for regional clayey silt. For filling sands treated with 3%, 5% and 7% cement contents, it has been found that, compared with the untreated sand, the maximum dry densities increased with the increase in cement content while the values of optimum moisture contents reduced with increasing cement contents (BRTC, 1995). Rajbongshi (1997) reported that the maximum dry density increased while the optimum moisture content reduced with the increase in cement content for two coastal soils of Bangladesh.

Table 4.3 Values of maximum dry density and optimum moisture content of untreated and cement-treated Soil-A and Soil-B

Cement Content (%)	Soil-A		Soil-B	
	γ_d (kN/m^3)	w_{opt} (%)	γ_d (kN/m^3)	w_{opt} (%)
0	16.5	18.0	18.5	11.5
1	16.8	17.7	18.8	11.3
3	17.2	17.1	19.1	10.8
5	17.8	16.0	19.5	10.3
7	-	-	19.6	10.1

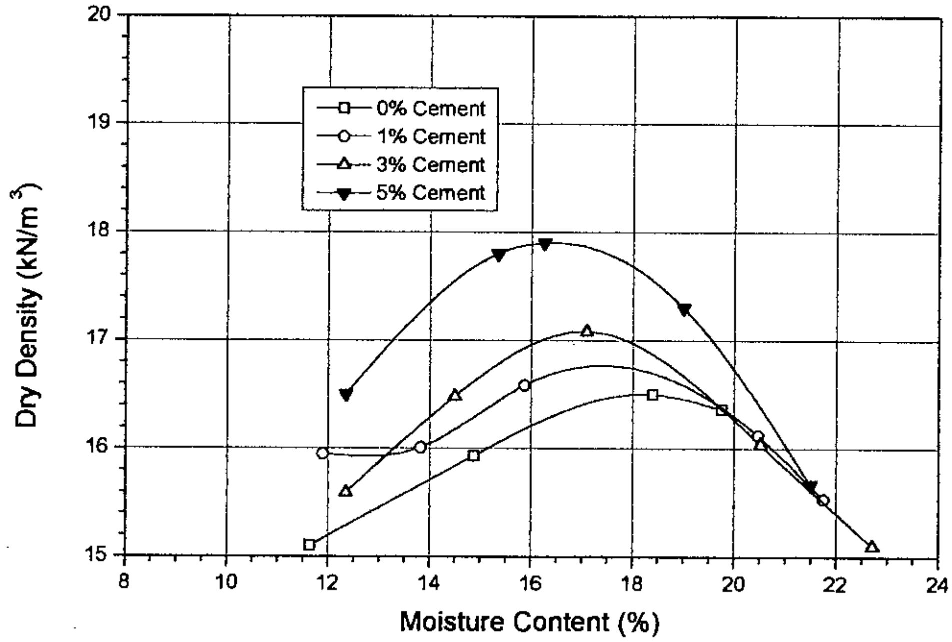

Fig: 4.5 Moisture-density relations of cement treated soil-A.

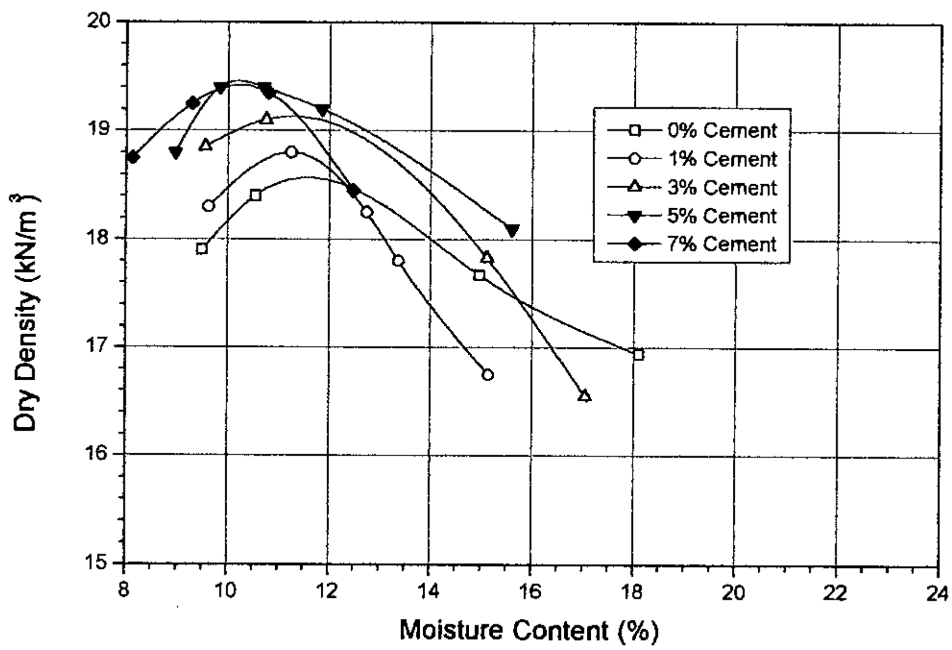

Fig: 4.6 Moisture-density relations of cement treated soil-B.

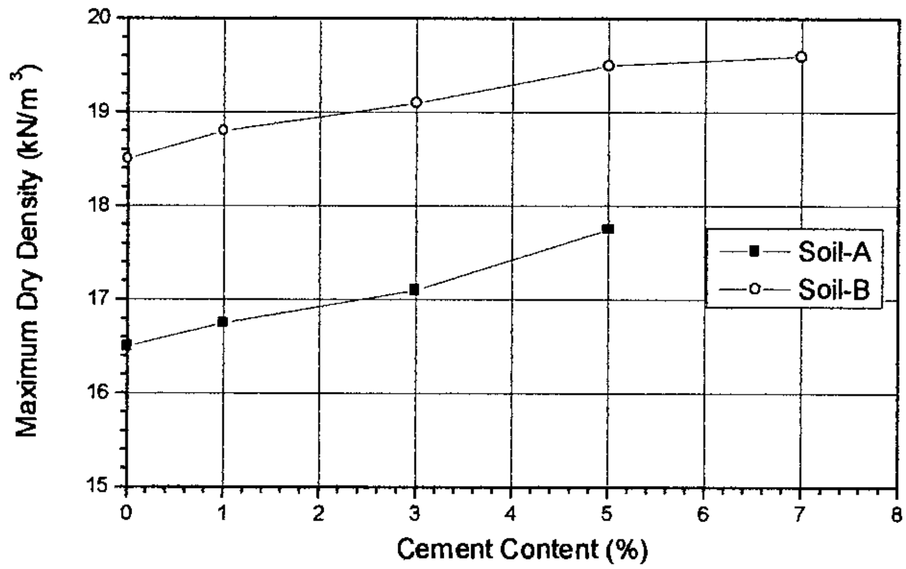

Fig: 4.7 Effect of cement content on maximum dry density of cement-treated soil-A and soil-B.

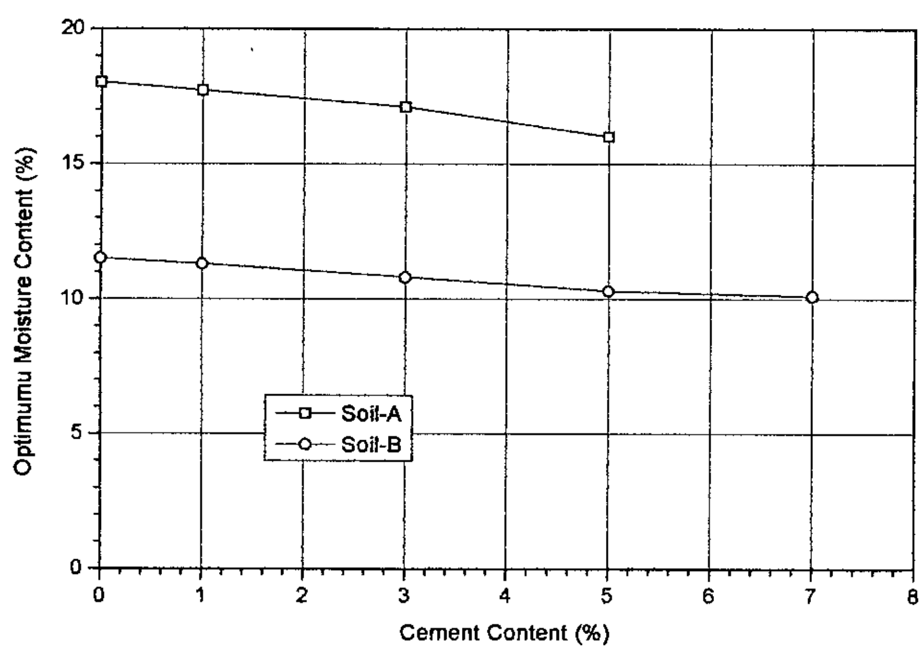

Fig: 4.8 Effect of cement content on optimum moisture content of cement-treated soil-A and soil-B.

4.2.3 UNCONFINED COMPRESSIVE STRENGTH

Table 4.4 shows a summary of the unconfined compression test results for Soil-A and Soil-B. In Table 4.4, the values of unconfined compressive strength (q_u) for the untreated samples and samples treated with different cement contents (1% and 3% for soil-A and 1%, 3%, 5% and 7% for soil-B) and cured for 7, 14 and 28 days are presented. It can be seen from Table 4.4 that for both the soils, compared with the untreated samples, the values of q_u of the treated samples increased significantly, depending on the cement content and curing age. Similar results have also been reported by Hossain (1986), Serajuddin and Azmal (1991) and Rajbongshi (1997) for fine-grained soils of Bangladesh for use in road construction. It can be seen from Table 4.4 that the values of q_u of samples of Soil-A treated with 3% cement and Soil-B treated with 7% cement and cured at 28 days were found to be about 6.3 times and 9.4 times higher than the strength of the untreated samples. It is also evident from Table 4.4 that the gain in strength with increasing cement content and curing age is higher in less plastic Soil-A (PI = 29) than in more plastic Soil-B (PI = 12). PCA (1956) recommended that values of q_u of soil-cement cured at 7 days and 28 days for soils belonging to ML and A-5 groups should be in the range of 250 to 500 psi (1723 to 3445 kPa) and 300 to 900 psi (2067 to 6201 kPa) respectively. PCA (1956) also recommended that values of q_u of soil-cement cured at 7 days and 28 days for soils belonging to A-7 group should be in the range of 200 to 400 psi (1378 to 2756 kPa) and 250 to 600 psi (1723 to 4134 kPa) respectively. It can be seen from Table 4.4 that values of q_u of samples of Soil-A (belongs to A-5 group) stabilized with 3% cement and Soil-B (belongs to A-7 group) stabilized with 3%, 5% and 7% cement and cured for 28 days satisfied the requirements of PCA (1956). Ingles and Metcalf (1972), however, recommended that the values of q_u of soil-cement road sub-base and base for light traffic should be in the range of 100 to 200 psi (689 to 1378 kPa). Table 4.4 also shows that for all cement contents and all curing ages, the values of q_u of treated samples fulfilled the requirements of soil-cement road sub-base and base for light traffic as proposed by Ingles and Metcalf (1972).

The relation between q_u for samples cured at different ages and cement contents are presented in Figs 4.8 and 4.9 for Soil-A and Soil-B respectively. Figs. 4.10 and 4.11 show the relations between q_u and curing age for Soil-A and Soil-B respectively. It can be seen from Figs. 4.8 to 4.11 that the values of qu of treated samples increased with increasing cement content and curing age. These results are in agreement with those reported by a number of researchers (Ramaswamy et al., 1984; Ahmed, 1984; Hossain, 1986; Hong, 1989; Anon, 1990; Serajuddin and Azmal, 1991; Serajuddin, 1992; Uddin, 1995; Rajbongshi, 1997).

Table 4.4 Unconfined compressive strength test results of untreated and cement treated Soil-A and Soil-B

Cement Content (%)	Curing Age (Days)	q_u (kPa)	
		Soil-A	Soil-B
0	-	460	380
1	7	576	482
	14	1040	653
	28	1380	1020
3	7	2320	946
	14	2832	1636
	28	2933	2464
5	7	-	2188
	14	-	2551
	28	-	3075
7	7	-	2671
	14	-	2892
	28	-	3588

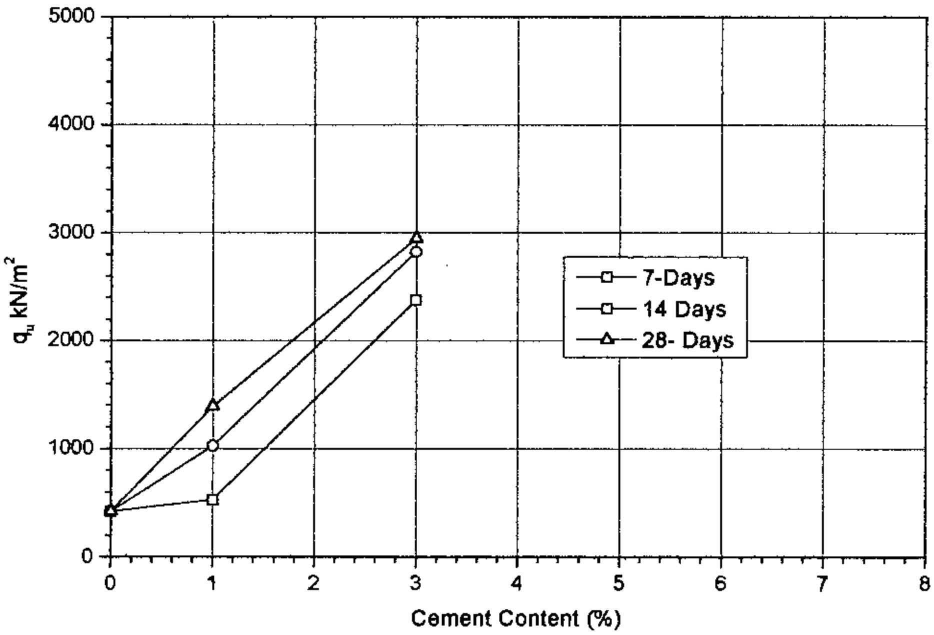

Fig: 4.9 Effect of cement content on compressive strength of cement-treated soil-A.

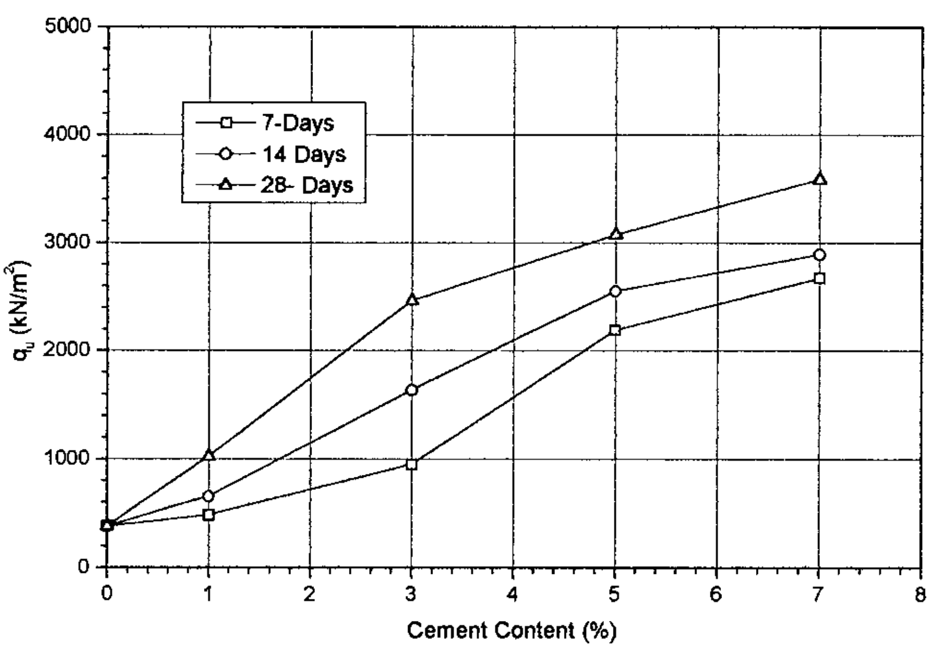

Fig: 4.10 Effect of cement content on compressive strength of cement-treated soil-B.

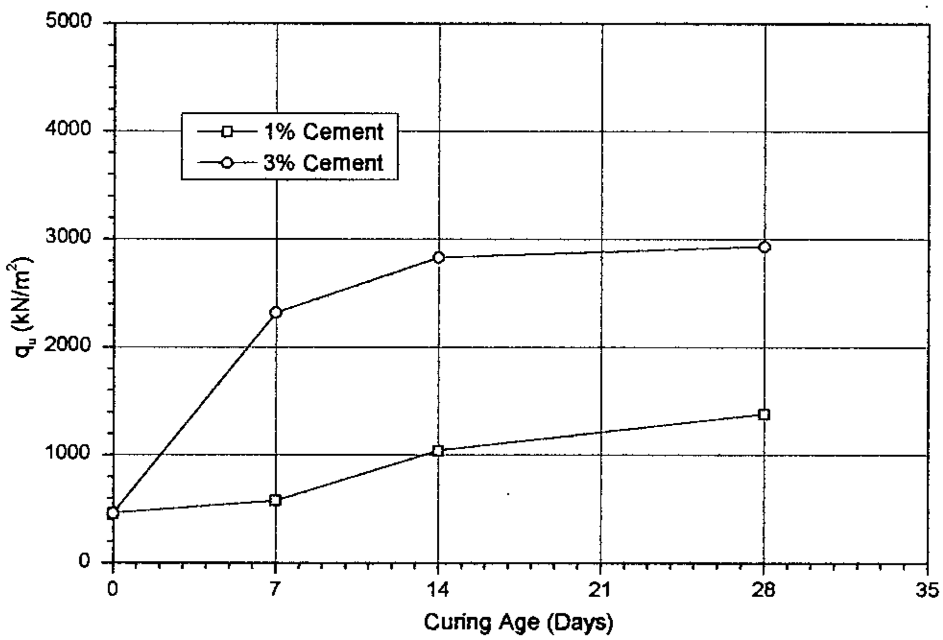

Fig: 4.11 Effect of curing age on unconfined compressive strength of cement-treated soil-A.

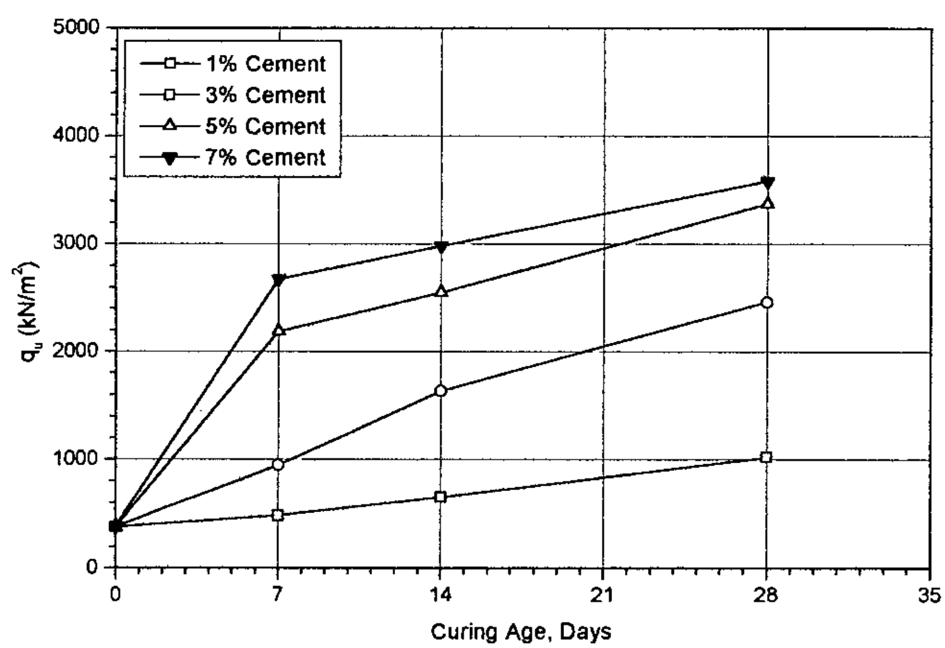

Fig: 4.12 Effect of curing age on unconfined compressive strength of cement-treated soil-B.

Plot of SDI with curing age of treated samples of soil-A and Soil-B are shown in Figs. 4.12 and 4.13 respectively. It can be seen from Figs 4.12 and 4.13 that the values of SDI increases with increasing curing age and cement content as well. These figures clearly demonstrate the relative degree of strength gain resulted due to increasing cement content and curing age. Uddin (1995) also reported an increase in SDI with increasing curing time and cement content for samples of Rangsit clay of Bangkok (LL = 70 to 117, PI = 50 to 78) treated with 5% to 40% cement and cured for 1 week to 40 weeks. It can be seen from Figs. 4.12 and 4.13 that the strength gain for samples of Soil-A and Soil-B treated with 1% cement are relatively much slower than those of samples treated with 3%, 5% and 7% cement.

In order to investigate the effect of molding water content on the compressive strength, unconfined compressive strength tests were also carried out on 2.8 inch diameter by 5.6 inch high untreated soil samples of Soil-B treated with 3% cement content and cured for 7, 14 and 28 days. The samples were compacted according to the Modified Compaction test with two additional molding water contents other than the optimum moisture content. The following water contents were used for compaction:

(a) water content corresponding to 95% of maximum dry density at dry side of the optimum moisture content.

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

Comparisons of the values of q_u at different curing ages for three molding moisture contents of Soil-B are presented Table 4.5. Table 4.5 shows that irrespective of curing ages values of q_u are maximum and minimum respectively at molding moisture contents of optimum and wet side of optimum. Rajbonshi (1997) also found higher compressive strength for cement-stabilized samples of coastal soils compacted at their optimum moisture contents than samples compacted at wet and dry side of optimum moisture content. It therefore appears that in order to achieve adequate compressive strength, samples should be compacted at their optimum moisture contents.

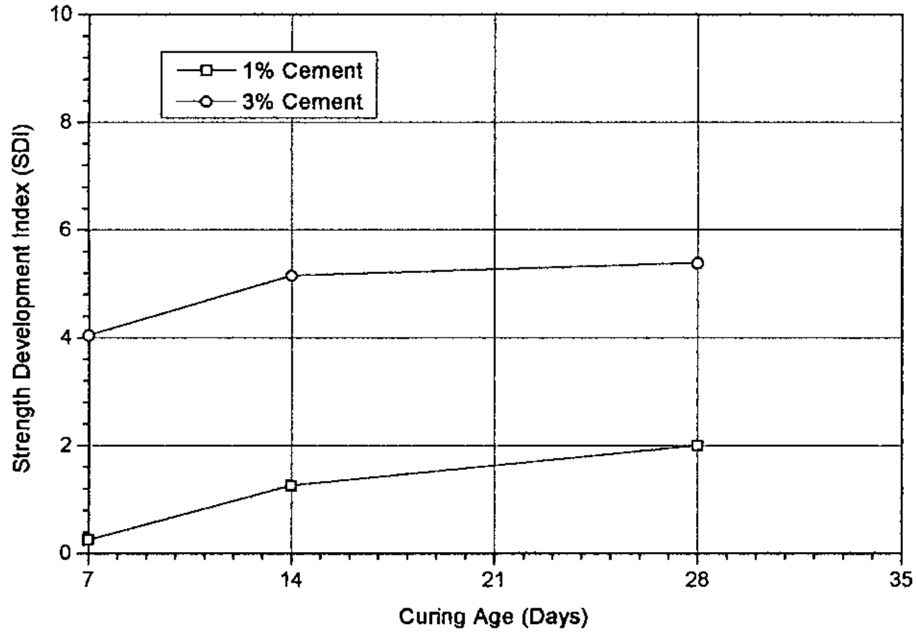

Fig: 4.13 SDI versus curing age curves for cement-treated soil-A.

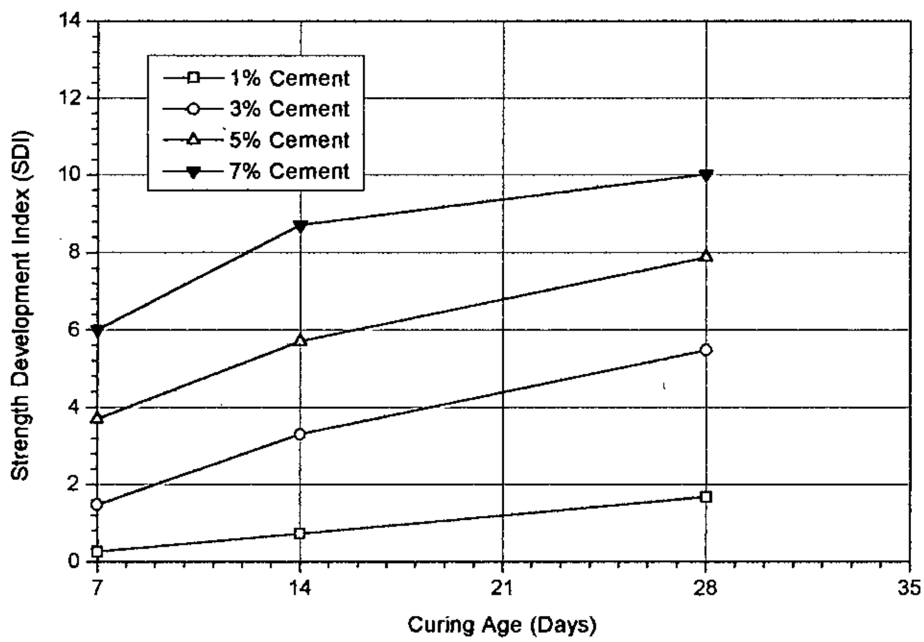

Fig: 4.14 SDI versus curing age curves for cement-treated soil-B.

Table 4.5 Unconfined compressive strength test results for samples of Soil-B treated with 3% cement and compacted with different molding water contents

Molding Water Content (%)	Curing Age (Days)	q_u (kPa)
Optimum Moisture Content (w_{opt})	7	946
	14	1636
	28	2464
Dry side of w_{opt} at 95% compaction	7	850
	14	1478
	28	2091
Wet side of w_{opt} at 95% compaction	7	754
	14	1293
	28	1946

The variation of q_u with curing age for samples of Soil-B treated with 3% cement and compacted with different molding water contents are shown in 4.14 respectively. It can be seen from Figs 4.14 that for both Soil-B, the values of q_u increases with the increase in curing age and that at any particular curing age the values of q_u of samples compacted at optimum water content are higher than the values of q_u of samples compacted at dry side or wet side of optimum water content.

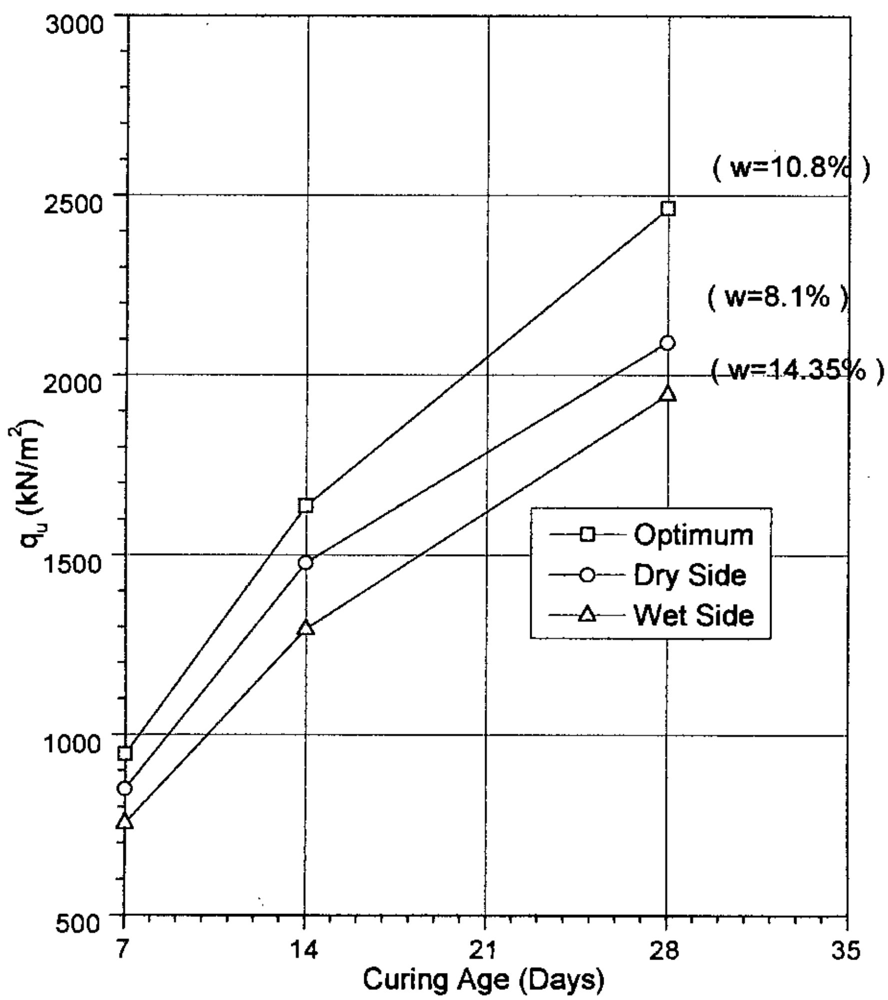

Fig: 4.15 Unconfined compressive strength versus curing age curves for samples of soil-B treated with 3% cement and compacted with different molding water contents.

4.2.4 CALIFORNIA BEARING RATIO (CBR)

A summary of the CBR test results for Soil-A and Soil-B is presented in Table 4.6. In order to investigate CBR-dry density relationship for untreated and stabilized samples, CBR tests were performed on samples compacted according to modified compaction test using three levels of compaction energies, e.g., low compaction (471 kN-m/m^3), medium compaction (1178 kN-m/m^3) and high compaction (2638 kN-m/m^3). It can be seen from Table 4.6 that for both Soil-A and Soil-B, compared with the untreated sample, CBR-values of the treated samples at all levels of compaction increased considerably. The variation of CBR with different cement content for Soil-A and Soil-B are shown in Figs. 4.16 and 4.17 respectively while Figs. 4.18 and 4.19 present the CBR-dry density relationships for Soil-A and Soil-B respectively. It can be seen from Figs. 16 and 17 that at all levels of compaction, CBR increases with increasing cement content while Figs. 4.18 and 4.19 show that at any particular cement content, CBR increases significantly with the increase in dry density. Similar trend of increasing CBR with the increase in cement content and dry density have been found for regional soils stabilized with different cement content (BRTC, 1995; Rajbonshi, 1997).

It can be seen from Table 4.6 that CBR-values of Soil-A and Soil-B stabilized with 5% cement increased up to 5 times 5.3 times than those of the respective untreated samples. It is also evident from the CBR data presented in Table 4.6 that the CBR-values of samples of the less plastic Soil-A (PI = 12) is lower than those for the samples of more plastic Soil-B (P = 29).

Ingles and Metcalf (1972) recommended that four-day soaked CBR-values of soil-cement road sub-base and base for light traffic should be in the range of 50 to 150. It can be seen from Table 4.6 that CBR values of Soil-A treated with 3% to 5% cement and compacted with low to high energy and Soil-B treated with 3% cement, compacted with high energy and that CBR values of Soil-B treated with 5% cement and compacted with low to high energy met the requirements of soil-cement road sub-base and base for light traffic as proposed by Ingles and Metcalf (1972).

Table 4.6 Summary of CBR test results of untreated and cement-treated Soil-A and Soil-B.

Cement Content (%)	Compaction Energy	Soil-A		Soil-B	
		Dry Density (kN/m^3)	4-Day Soaked CBR	Dry Density (kN/m^3)	4-Day Soaked CBR
0	Low	15.4	15	16.9	13
	Medium	16.5	18	17.8	15
	High	16.8	21	18.2	17
1	Low	15.5	39	17.2	23
	Medium	16.3	44	18.1	39
	High	17.0	52	18.8	56
3	Low	15.5	55	17.7	30
	Medium	16.6	60	18.4	41
	High	17.39	78	19.3	70
5	Low	15.96	71	17.9	50
	Medium	17.38	89	18.9	65
	High	18.32	102	19.9	90
7	Low	-	-	18.0	63
	Medium	-	-	19.2	79
	High	-	-	20.0	100

Note: Low compaction energy = 471 kN-m/m^3 (10000 lb-ft/ft^3)
Medium compaction energy = 1178 kN-m/m^3 (25000 lb-ft/ft^3)
High compaction energy = 2638 kN-m/m^3 (56000 lb-ft/ft^3)

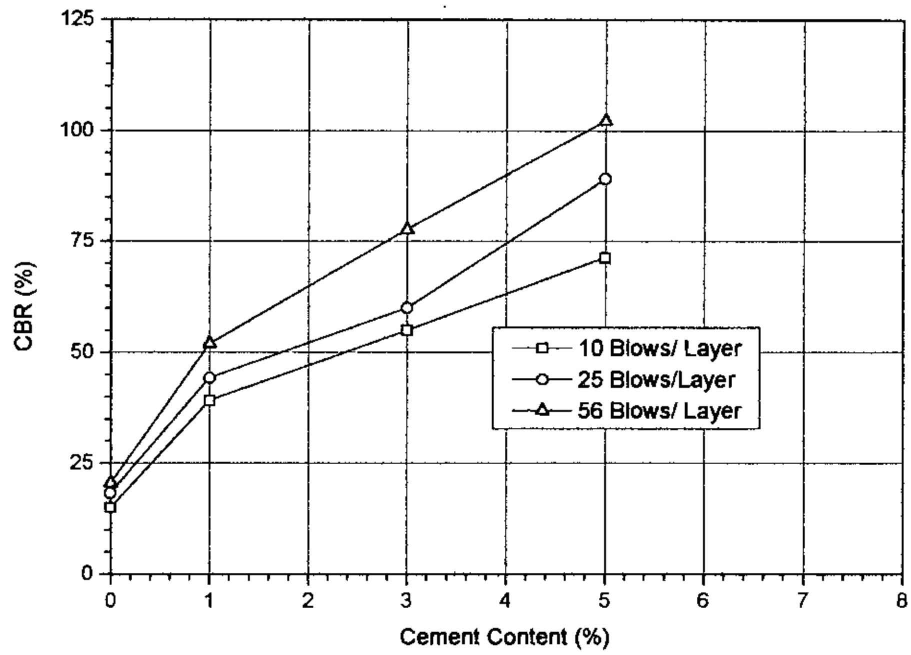

Fig: 4.16 Effect of cement content on CBR values of soil-A.

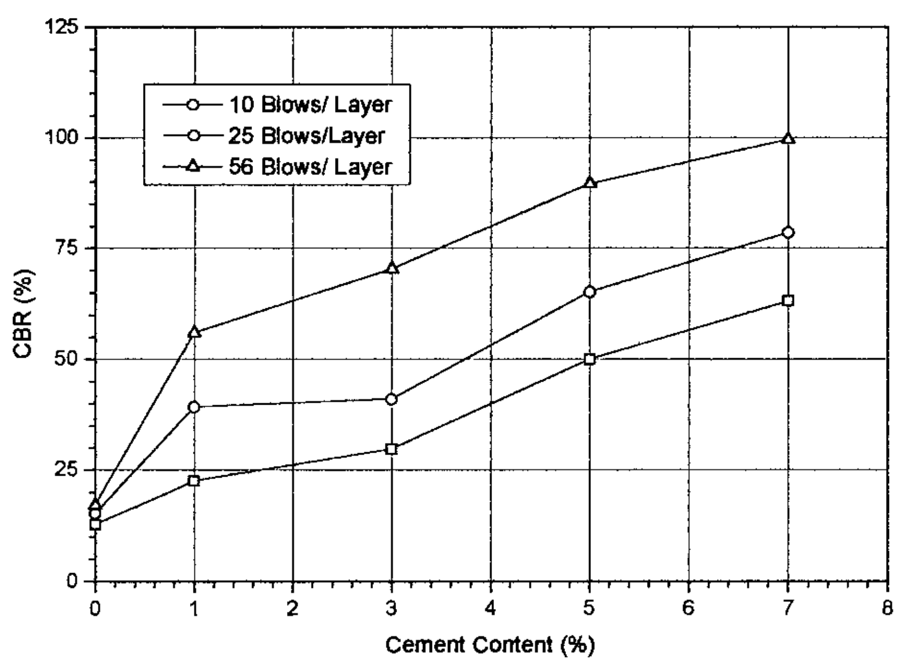

Fig: 4.17 Effect of cement content on CBR values of soil-B.

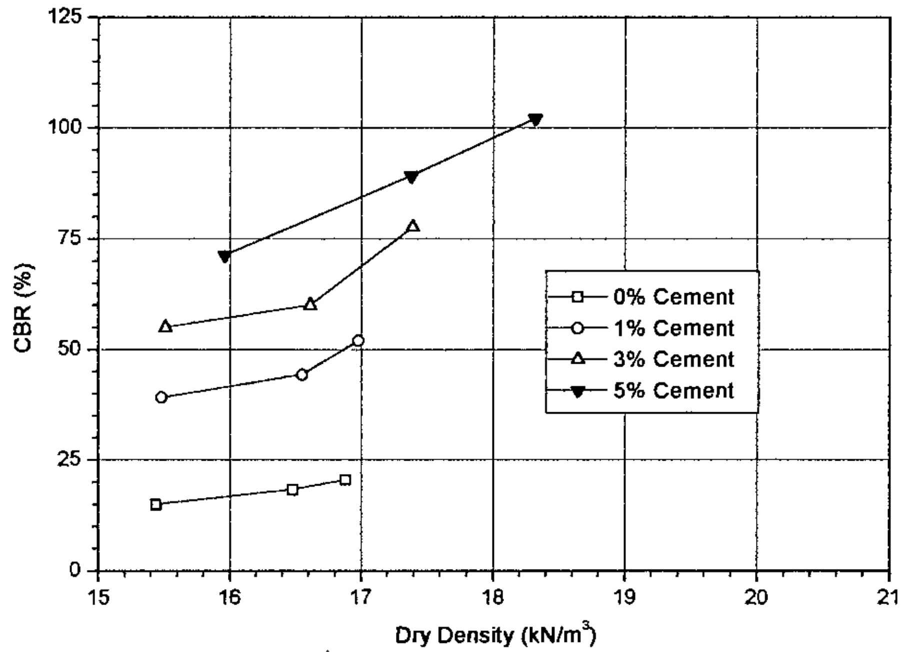

Fig: 4-18 CBR versus dry-density curves of cement treated soil-A.

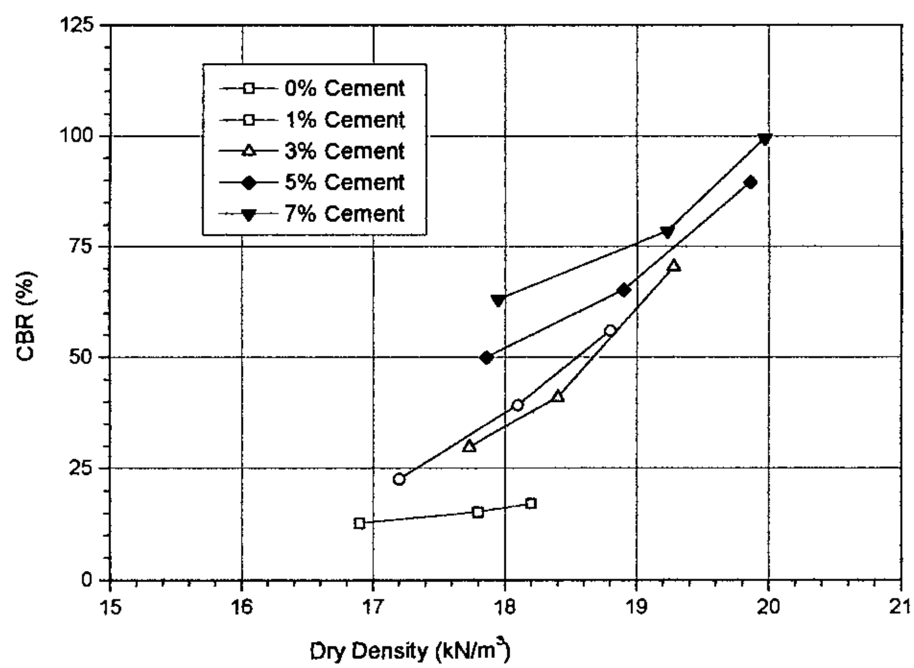

Fig: 4.19 CBR versus dry-density curves of cement treated soil-B.

4.2.5 FLEXURAL STRENGTH AND MODULUS

The flexural properties of untreated and stabilized samples of the two soils have been investigated by carrying out flexural strength test using simple beam test with third point loading. Typical flexural stress versus defection curves for two stabilized samples of Soil-A and Soil-B are presented in Figs. 4.20 and 4.21 respectively. From the flexural stress and deflection data flexural strength and modulus were determined. The flexural properties of Soil-A and Soil-B are presented in Tables 4.7 and 4.8 respectively. It can be seen from Tables 4.7 and 4.8 that for both Soil-A and Soil-B, compared with the untreated sample, flexural strength and modulus of the treated samples cured at 7 and 28 days increased significantly. It can be seen from Table 4.7 that compared with the untreated sample, the flexural strength and modulus of Soil-A treated with 1, 3 and 5 percent cement and cured at 28 days are respectively about 4.5 times and 3.4 times higher while Table 4.8 shows that the flexural strength and modulus of Soil-B treated with 1, 3, 5 and 7% cement and cured at 28 days are respectively about 5.5 times and 5.4 times higher than those of the untreated samples. The maximum deflections of untreated and stabilized soil-cement beams of soil-A are 0.2 to 0.76 mm and of soil-B are 0.33 to 0.86 mm and failure strain are 0.21 to 0.35% for soil-A and 0.14 to 0.31% for soil-B. Comparing the flexural strength and modulus of Soil-A with those of Soil-B, it is evident that the values of flexural strength and modulus of samples of more plastic Soil-B (P = 29) is higher than the less plastic Soil-A (PI =12).

The effect of cement content on flexural strength for Soil-A and Soil-B are shown in Figs. 4.22 and 4.23 respectively while Figs. 4.24 and 4.25 present the effect of cement content on flexural modulus of Soil-A and Soil-B respectively. Figs. 4.22 to 4.25 shows that flexural strength and modulus increases with increasing cement content. It is evident from Figs. 4.22 to 4.25 that curing age has got insignificant effect on increase in flexural strength and modulus. Rajbonshi (1997) shows that for coastal soils flexural strength and modulus increases with increase in cement content and curing time.

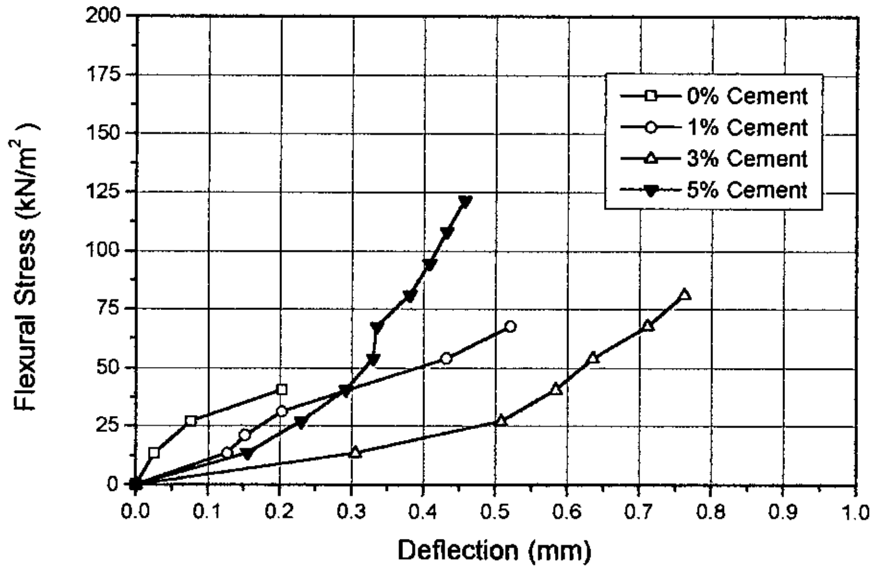

Fig: 4.20 Flexural stress versus deflection curve of cement treated soil-A for 28 days curing.

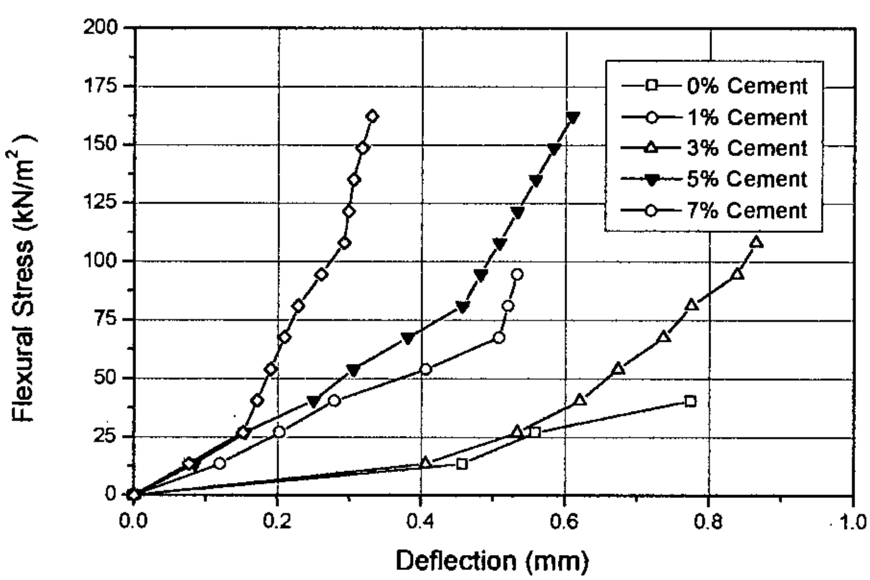

Fig: 4.21 Flexural stress versus deflection curve of cement treated soil-B at 28 days curing.

Table 4.7 Flexural properties of untreated and cement-treated Soil-A

Cement Content (%)	Curing Age (Days)	Flexural Strength (kPa)	Maximum Deflection (mm)	Flexural Modulus (MPa)	Failure Strain (%)
0	-	26.9	0.20	17.3	0.16
1	7	54.1	0.64	18.6	0.29
	28	60.8	0.51	25.5	0.24
3	7	71.0	0.64	24.5	0.29
	28	81.1	0.76	28.5	0.35
5	7	104.6	0.38	48.5	0.22
	28	121.7	0.46	58.3	0.21

Table 4.8 Flexural properties of untreated and cement-treated Soil-B

Cement Content (%)	Curing Age (Days)	Flexural Strength (kPa)	Maximum Deflection (mm)	Flexural Modulus (MPa)	Failure Strain (%)
0	-	47.3	0.76	23.4	0.20
1	7	81.1	0.68	26.0	0.31
	28	94.6	0.51	40.8	0.23
3	7	108.2	0.36	66.6	0.16
	28	128.4	0.86	82.6	0.16
5	7	135.2	0.33	97.1	0.14
	28	162.2	0.61	116.2	0.14
7	7	196.0	0.43	99.4	0.20
	28	263.6	0.32	125.5	0.21

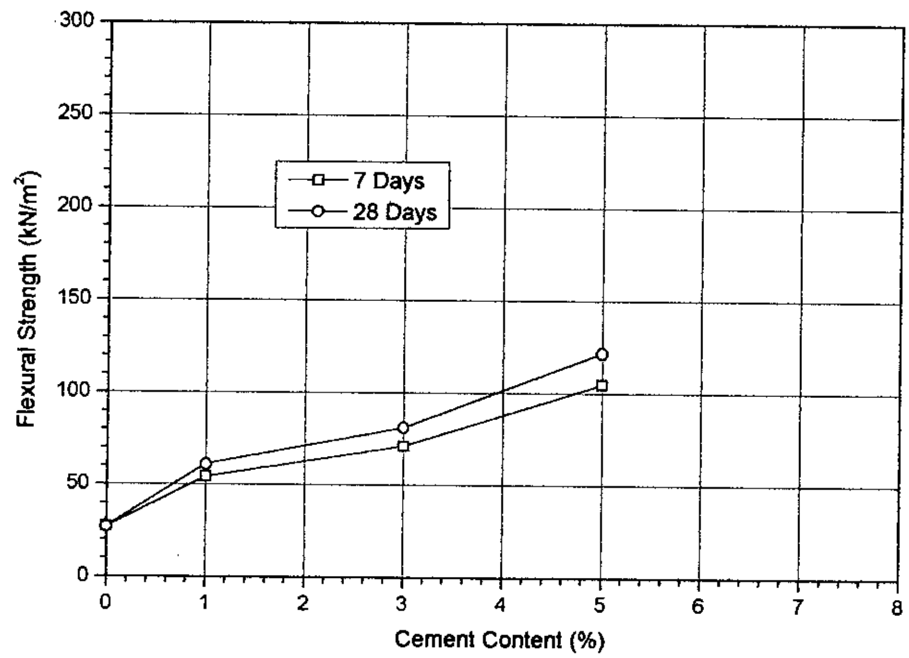

Fig: 4.22 Effect of cement content on flexural strength of soil-A.

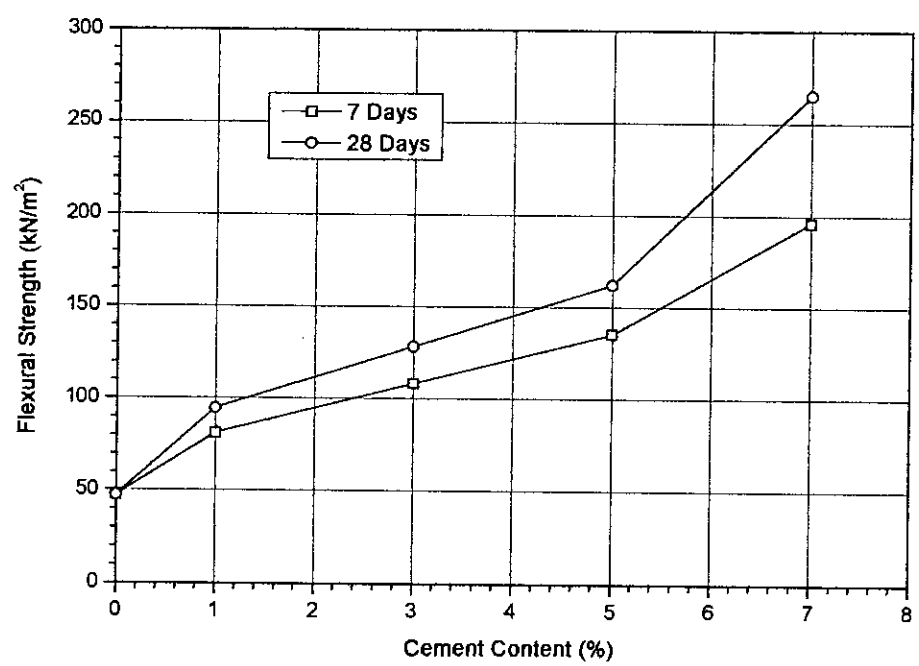

Fig: 4.23 Effect of cement content on flexural strength of soil-B.

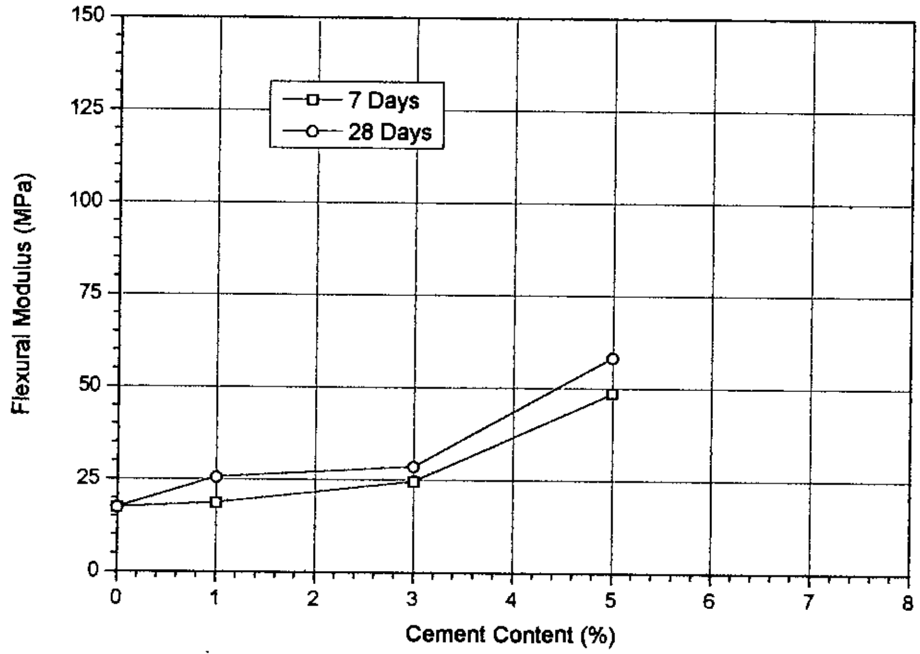

Fig: 4.24 Effect of cement content on flexural modulus of soil-A.

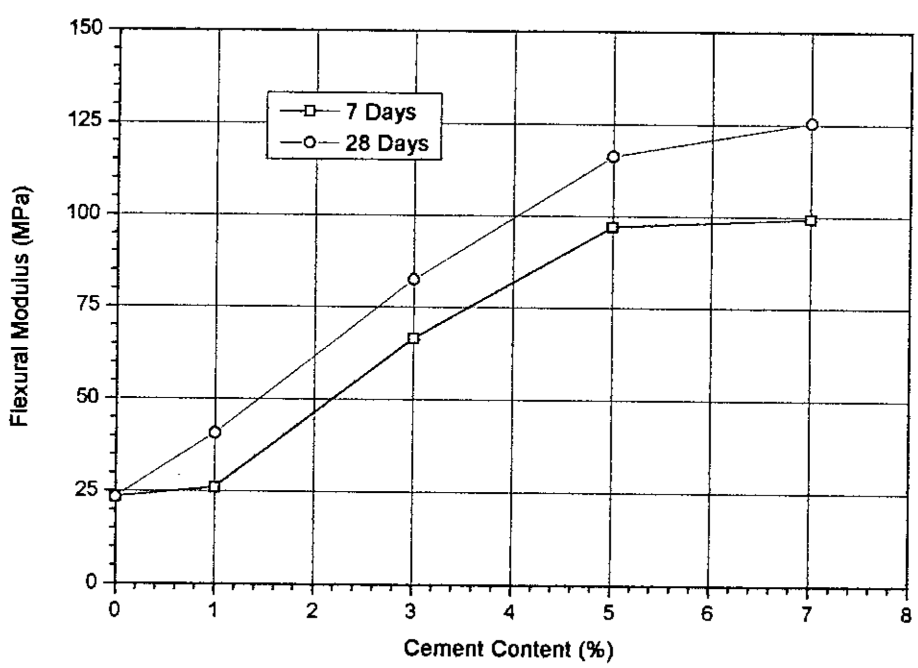

Fig: 4.25 Effect of cement content on flexural modulus of soil-B.

4.2.6 ABSORPTION CAPACITY OF SOIL-CEMENT MIX

Absorption tests were conducted on the portion of the soil-cement beams, which were used in flexural strength tests. The values of absorption capacity of samples treated with 1%, 3% and 5% cement for Soil-A and 1, 3, 5 and 7% cement for Soil-B and cured at 7 days and 28 days are presented in Table 4.9. Table 4.9 shows that the absorption capacity of stabilized samples of Soil-A and Soil-B are in the range of 15.40% to 26.40%. Rajbonshi (1997) investigated the water absorption capacity of clayey silt stabilized with 5% cement and found that absorption capacity varied between 20.4% and 28.5%.

Table 4.9 Water absorption capacities of soil-cement beams of Soil-A and Soil-B

Cement Content (%)	Curing Age (Days)	Absorption Capacity (%)	
		Soil-A	Soil-B
1	7	25.0	19.1
	28	26.4	19.6
3	7	25.1	18.0
	28	25.9	18.6
5	7	24.3	17.2
	28	25.3	17.6
7	7	-	15.4
	28	-	16.0

4.2.7 DURABILITY

Durability of hardened soil-cement mix samples have been assessed by performing repeated wetting and drying tests. Results of wetting and drying tests showing moisture and volume changes for different cycles performed on samples of soil stabilized with 1%, 3% and 5% cement of Soil-A and 1%, 3%, 5% and 7% cement of Soil-B are presented in Tables A.1 to A.6 of Appendix-A. Soil-cement losses in wetting and drying tests for hardened soil-cement mix samples of Soil-A and Soil-B are shown in Table 4.10. The relationship between soil-cement loss and cement content for Soil-A and Soil-B are presented in Fig. 4.26. It can be seen from Table 4.10 and Fig. 4.26 that the loss in soil-cement reduced with the increase in cement content. Similar results have also been reported by Hossain (1986) for two regional soils of Bangladesh. It can be seen from Table 4.10 that the soil-cement loss for 5% cement content are 18.6% and 16.8% respectively for Soil-A and Soil-B. The Portland Cement Association, PCA (1956) suggested that a maximum of 10% loss of soil-cement in the wet and dry test is allowable for this type of soil. Therefore, the soils studied should be stabilized with higher percentages of cement content in order to fulfill the PCA (1956) durability criteria. However, Compendium 8 (1979), mentioned that in tropical and sub-tropical conditions, where freeze and thaw tests are not essentials, a q_u-value of 150 psi (1034 kPa) at 7 days curing is adequate to stand 12 cycles of wetting and drying which satisfies the weathering conditions in the tropics. It can be seen from Table 4.10 that the values of q_u of samples of Soil-A and Soil-B treated with 3% and 5% cement and cured at 7 days fulfill the requirement as proposed by Compendium 8 (1979). Hossain (1986) found that addition of about 8% cement in a clayey soil (LL = 33, PI = 6) and non-plastic silt produced durable soil-cement mix, which satisfied the PCA (1956) criteria. Rajbonshi (1997) shows that the values of qu of coastal soils treated with 3% and 5% cement and cured at 7 days fulfill the conditions of Compendium 8 (1979).

Table 4.10 Soil-cement loss in wetting and drying tests for cement-treated samples of Soil-A and Soil-B (curing age: 28 days)

Cement Content (%)	Soil-A		Soil-B	
	q_u (kPa)	Soil-Cement Loss (%)	q_u (kPa)	Soil-Cement Loss (%)
1	1380	28.6	1020	25.5
3	2933	24.3	2464	22.1
5	3050	18.6	3375	16.8
7	-	-	3588	11.7

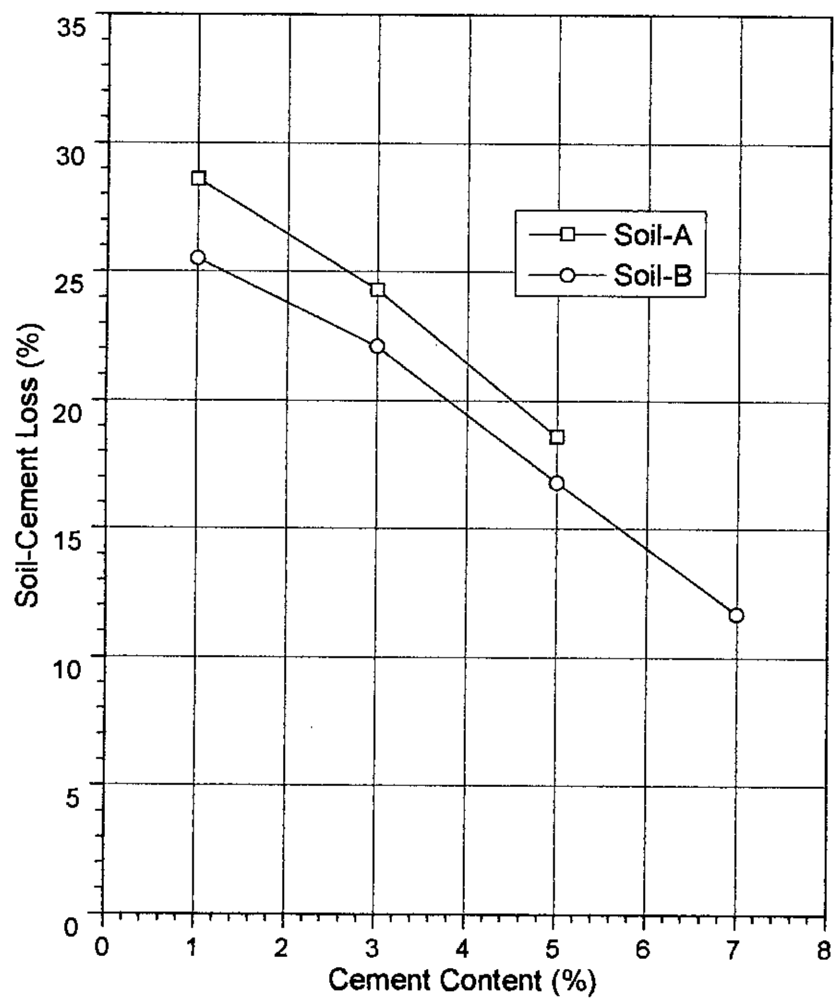

Fig: 4.26 Comparison of soil-cement loss of cement-treated soil-A and soil-B from wetting and drying test.

4.3 PHYSICAL AND ENGINEERING PROPERTIES OF LIME-TREATED SOIL

In the following sections, the physical and engineering characteristics comprising plasticity and shrinkage properties, moisture-density relations, unconfined compressive strength, California Bearing Ratio (CBR) and flexural properties of untreated and lime-treated samples of the Soil-B (Bashundhara) are presented and discussed. Comparisons on changes in the properties between the untreated and stabilized samples have been made.

4.3.1 PLASTICITY AND SHRINKAGE CHARACTERISTICS

The values of plasticity and shrinkage properties of the untreated and lime-treated soil samples are shown in Table 4.11. Fig. 4.27 shows the variation of liquid limit, plastic limit plasticity index and shrinkage limit with the increment of lime addition. It can be seen from Fig. 4.27 that plastic limit and shrinkage limit increased with increasing lime content while liquid limit and plasticity index reduced with the increase in lime content. These results are in agreement with those reported by IRC (1976), Ahmed (1984) ,Rajbongshi (1997), Hossain (2001) and Shahjahan (2001). Table 4.11 shows the linear shrinkage of the stabilized samples reduced with increasing lime content. Similar results were also reported by Bell (1993).

Table 4.11 Index and shrinkage properties of lime-treated Soil-B

Index Shrinkage Properties	Lime Content (%)				
	0	1	3	5	7
Liquid Limit	52.0	50.5	49.0	48.0	46.5
Plastic Limit	23.0	23.5	24.0	25.0	25.5
Plasticity Index	29.0	27.5	25.0	23.0	22.5
Shrinkage Limit	14.0	15.0	15.5	16.0	17.0

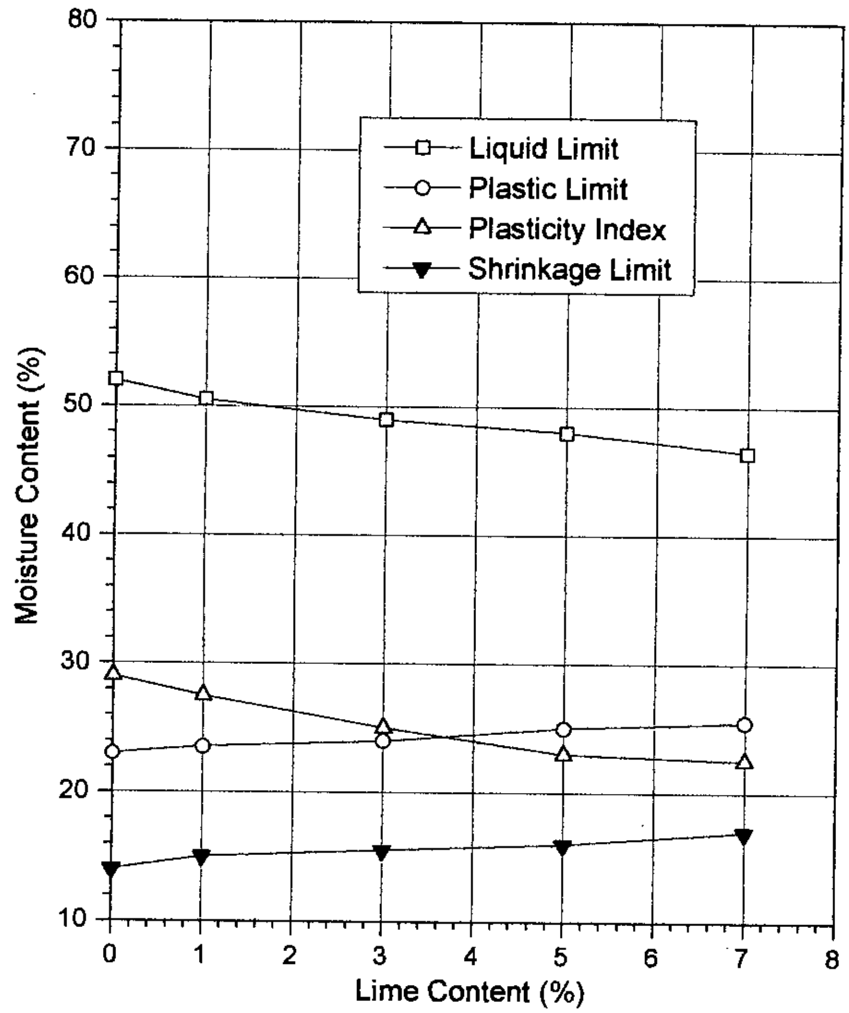

Fig: 4.27 Effect of lime content on Atterberg limits and shrinkage limit of soil-B.

4.3.2 MOISTURE-DENSITY RELATIONS

The moisture-density relationship of untreated and lime-treated samples is shown in Fig. 4.28. From the relations presented in Fig. 4.28, the maximum dry density (γ_{max}) and optimum moisture content (w_{opt}) of Soil-B have been determined which are presented in Table 4.28. It can be seen from Table 4.28 that with the increase in lime content values of γ_{max} reduced while the values of w_{opt} increased. The reduction in γ_{max} with the increase in lime content for the soil is shown in Fig. 4.29. Compared with the untreated sample, the values of γ_{max} reduced up to 4.2% for an increase in lime content up to 7%. The values of w_{opt} increased up to 20.8% for an increase in lime content up to 7%. Ahmed (1984) found that for sandy silt and silty clay soils of Bangladesh, the maximum dry density reduced with the increase in lime content. Rajbongshi (1997) also found that compared with untreated sample the maximum dry density of lime-treated samples of two fine-grained coastal soils reduced while optimum moisture content slightly increased. Reduction in the values of γ_{max} with increasing lime content has also been reported by a number of researchers (Kezdi, 1979; TRB, 1987; Hausmann, 1990; Bell, 1993).

4.3.3 UNCONFINED COMPRESSIVE STRENGTH

Table 4.13 shows a summary of the unconfined compression test results of the untreated and treated samples of Soil-B. The values of unconfined compressive strength (q_u) for the untreated samples and samples treated with different lime contents (1%, 3%, 5% and 7%) and cured for 7, 14 and 28 days are presented in Table 4.13. Table 4.13 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. 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), Serajuddin (1992) and Rajbongshi (1997) also reported that the unconfined compressive strength of regional alluvial soils and coastal soils of Bangladesh treated with 3% to 10% hydrated lime increased with the increase in lime content and curing age. Table 4.13 shows that the value of q_u of sample treated with 7% lime and cured at 28 days was found to be about 7 times higher than the strength of the untreated sample.

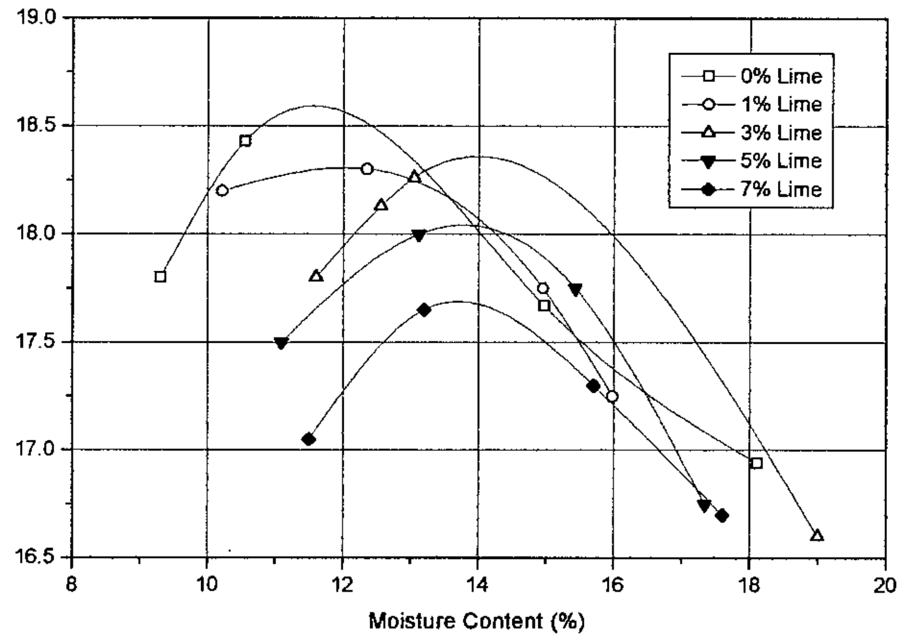

Fig: 4.28 Moisture-density relations of lime-treated soil-B.

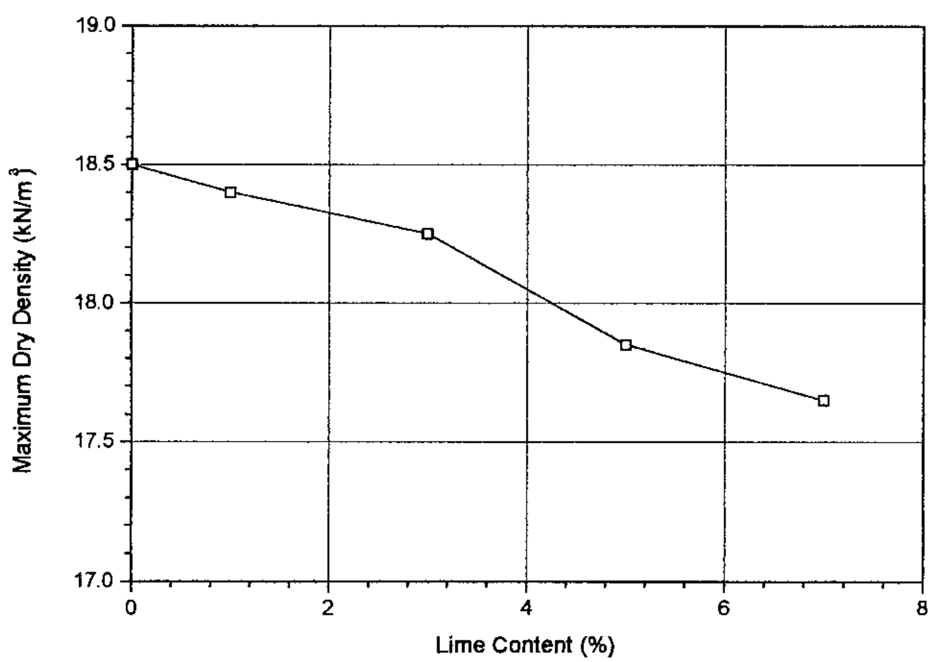

Fig: 4.29 Effect of lime content on maximum dry density of lime-treated soil-B.

Table 4.12 Values of maximum dry density and optimum moisture content of untreated and lime-treated Soil-B

Lime Content (%)	Maximum Dry Density, γ_d (kN/m^3)	Optimum Moisture Content, w_{opt} (%)
0	18.5	11.5
1	18.4	11.9
3	18.3	13.0
5	17.9	13.6
7	17.7	13.9

Table 4.13 Unconfined compressive strength test results of untreated and lime-treated Soil-B

Lime Content (%)	Curing Age (Days)	q_u (kPa)
0	-	380
1	7	470
	14	775
	28	984
3	7	1020
	14	1381
	28	2015
5	7	1877
	14	2192
	28	2385
7	7	2173
	14	2304
	28	2678

Ingles and Metcalf (1972) recommended that when lime stabilization has to be used in order to upgrade heavy clays to sub-base material quality type, a q_u-value of 250 psi (1722 kPa) or 150 psi (1033 kPa) to 450 psi (3100 kPa) at seven days is required. Table 4.13 shows that the unconfined compressive strength of samples treated with 5% and 7% lime contents fulfilled the requirements as proposed by Ingles and Metcalf (1972). The relationship between q_u for samples cured at different ages and lime contents are shown in Figs. 5.30 and 5.31 for Soil-B. Figs. 5.30 and 5.31 show the relations between q_u and curing period for Soil-B. These results are in agreement with those reported by a number researchers (Ingles and Metcalf, 1972; Ahmed, 1984; Serajuddin and Azmal, 1991; Serajuddin, 1992; Bell, 1993; Rajbongshi 1997; Mollah 1997 and Hossain 2001). Shahjahan(2001) shows that at 7% lime content and 360 days of curing age q_u increase 5 times than that of 7 days curing.

The rate of strength gain with curing time has been evaluated in terms of the parameter strength development index (SDI). A Plot of SDI with curing age of treated samples is shown in Fig.4.32. Fig.4.32 shows that the value of SDI increases with increasing curing time and lime content as well. This figure clearly shows the relative degree of strength gain resulted due to increasing lime content and curing age.

In order to investigate the effect of molding water content on the compressive strength, unconfined compression tests were also carried out on 2.8 inch diameter by 5.6 inch high soil samples stabilized with 3% lime content and cured for 7, 14 and 28 days. The samples were compacted according to the modified Compaction test with two additional molding water contents other than the optimum moisture content. The following water contents similar to that used in case of cement treated samples were used for compaction:

(a) water content corresponding to 95% of maximum dry density at dry side of the optimum moisture content.

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

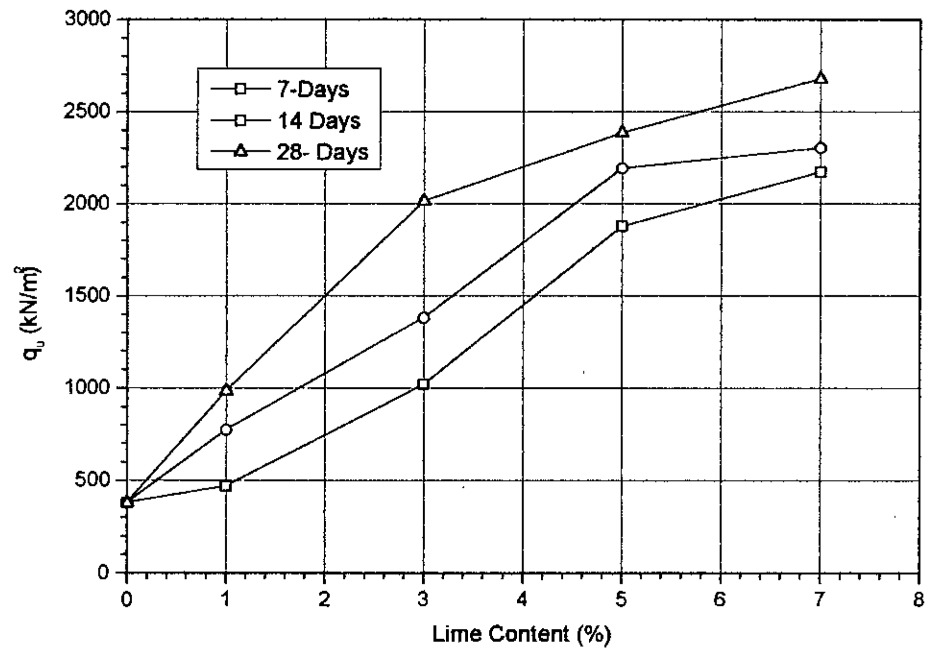

Fig: 4.30 Effect of lime content on unconfined compression strength of lime-treated soil-B.

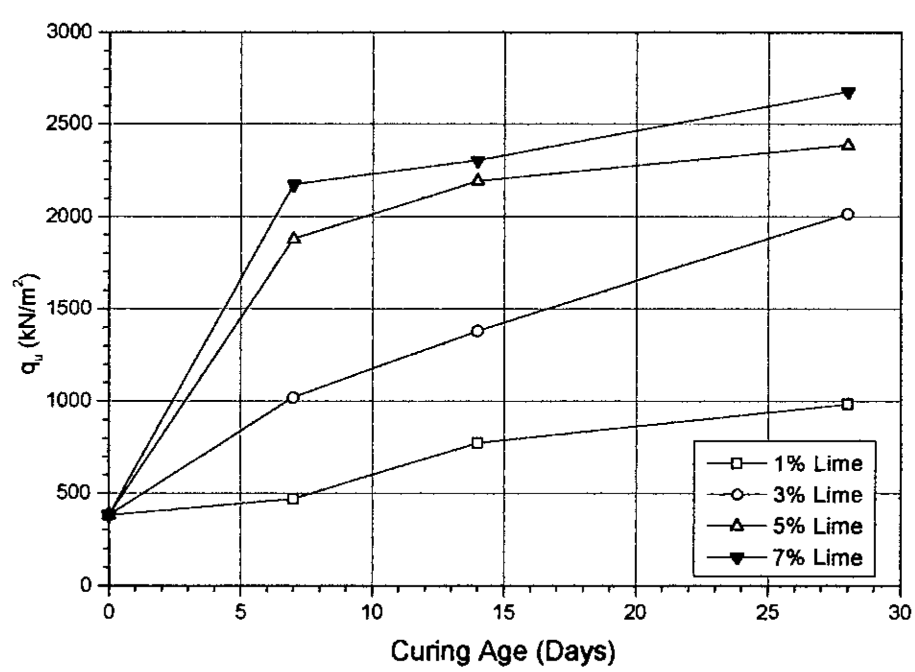

Fig: 4.31 Effect of curing age on unconfined compression strength of lime-treated soil-B.

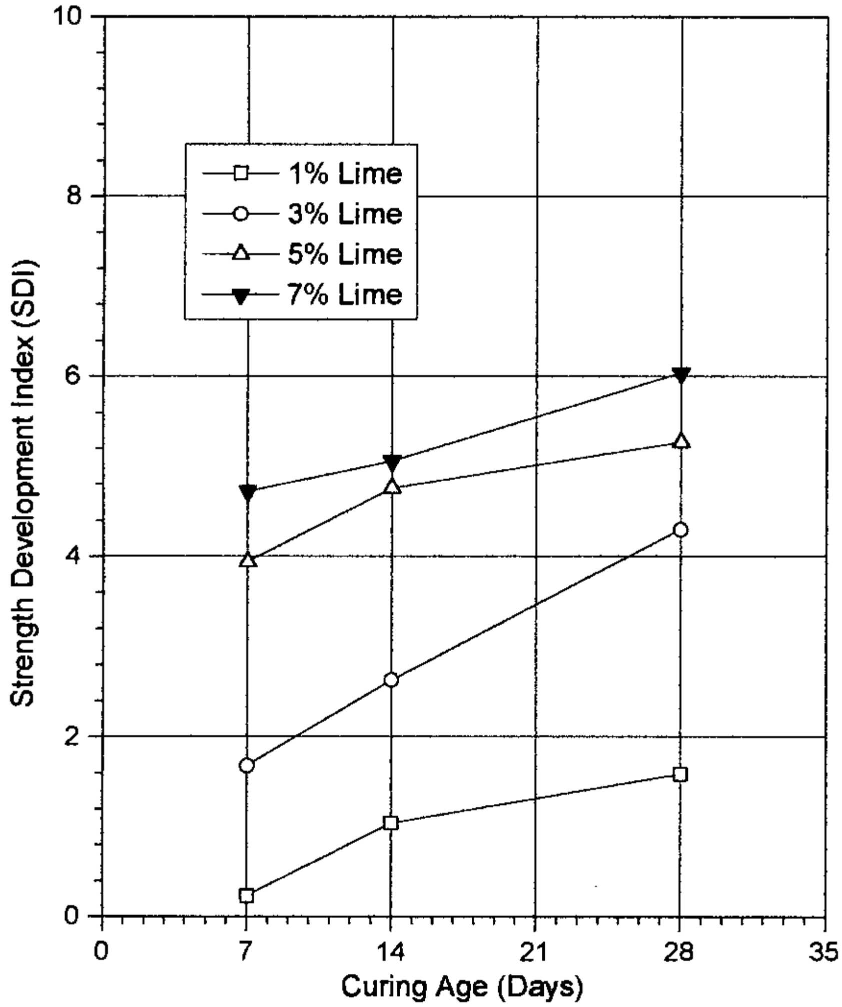

Fig: 4.32 SDI versus curing age curves of samples of lime-treated soil-B.

Comparisons of q_u at different curing ages for three molding moisture contents of Soil-B are presented in Table 4.14. It can be seen from Table 4.14 that irrespective of curing ages, values of q_u is maximum and minimum at molding moisture contents of wet side of optimum and dry side of optimum respectively. Ahmed (1984) also found higher compressive strength for lime stabilized samples of sandy silt and silty clay compacted at wet side of optimum moisture content than samples compacted at their optimum moisture contents. Rajbongshi (1997) also found the same results for coastal soils. It therefore appears that in order to achieve adequate compressive strength, lime stabilized samples should be compacted at their wet side of their optimum moisture contents.

The variation of q_u with curing age for samples of Soil-B treated with 3% lime and compacted with different Molding water contents are shown in Fig.4.33. It can be seen from Figs. 5.33 that the values of q_u increases 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 of samples compacted at optimum or dry side of optimum water content.

Table 4.14 Unconfined compressive strength test results at different molding water content for 3% lime content of Soil-B

Molding Water Content	Curing Age (Days)	q_u (kPa)
Optimum moisture content (w_{opt})	7	1020
	14	1381
	28	2015
Dry side of w_{opt} at 95% compaction	7	883
	14	1348
	28	1745
Wet side of w_{opt} at 95% compaction	7	1160
	14	1590
	28	2441

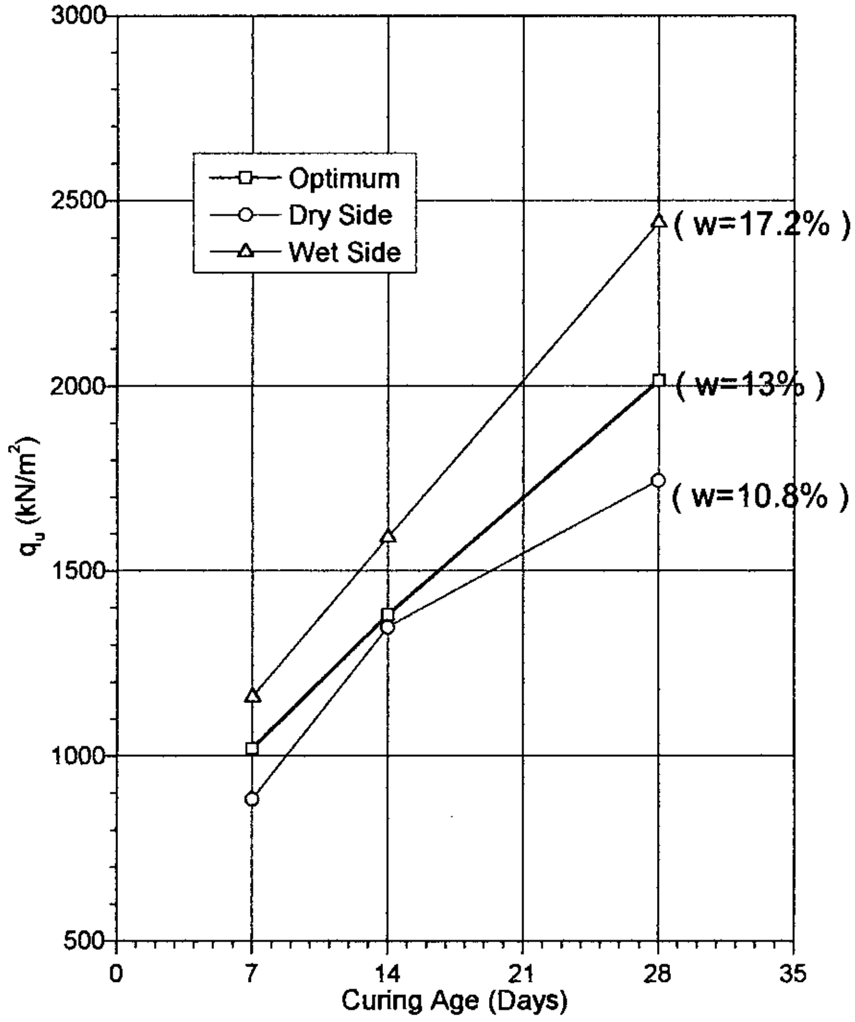

Fig: 4.33 q_u versus curing age curves for samples of soil-B treated with 3% lime and compacted with different molding water content.

4.3.4 CALIFORNIA BEARING RATIO (CBR)

A summary of the CBR test results for Soil-B is presented in Table 4.15. In order to investigate CBR-dry density relationship for untreated and stabilized samples, CBR tests were performed on samples compacted according to Modified Compaction test using three levels of compaction energies, e.g., low compaction (471 kN-m/m^3), medium compaction (1178 kN-m/m^3) and high compaction (2639 kN-m/m^3). It can be seen from Table 4.15 that for Soil-B, 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 for Soil-B is shown in Fig.4.34 while Fig.4.35 presents the CBR-dry density relationships for the same soil. It can be seen from Fig.4.34 that at all levels of compaction, CBR increases markedly with increasing lime content while Fig.4.35 shows that at any particular lime content, CBR increases significantly with the increase in dry density. TRB (1987) reported the effect of lime treatment on CBR-values for three plastic clays (LL = 35 to 59, PI = 15 to 30) and showed that for all the soils CBR increase markedly with increasing lime content. It can be seen from Table 4.15 that CBR-values of Soil-B stabilized with 7% lime increased up to about 4.1 times that of the respective untreated samples.

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 4.15 that CBR values of Soil-B treated with maximum 7% lime and compacted at high energy is 70, which does not fulfill the criteria proposed by Ingles and Metcalf (1972). Therefore, 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). Mollah (1997) and Rajbongshi (1997) show that at all levels of compaction, CBR increases markedly with the increase in lime content also with the increase in dry density.

Table 4.15 CBR test results of untreated and lime-treated Soil-B

Lime Content	Compaction Energy	Dry Density (kN/m^3)	4 days soaked CBR
0	Low	16.90	13
	Medium	17.80	15
	High	18.20	17
1	Low	16.32	17
	Medium	17.31	23
	High	17.92	36
3	Low	15.94	32
	Medium	17.04	43
	High	17.36	49
5	Low	15.21	49
	Medium	16.39	57
	High	16.86	60
7	Low	14.89	53
	Medium	15.89	64
	High	16.04	70

Note: Low compaction energy = 471 kN-m/m^3 (10000 lb-ft/ft^3)
Medium compaction energy = 1178 kN-m/m^3 (25000 lb-ft/ft^3)
High compaction energy = 2639 kN-m/m^3 (56000 lb-ft/ft^3)

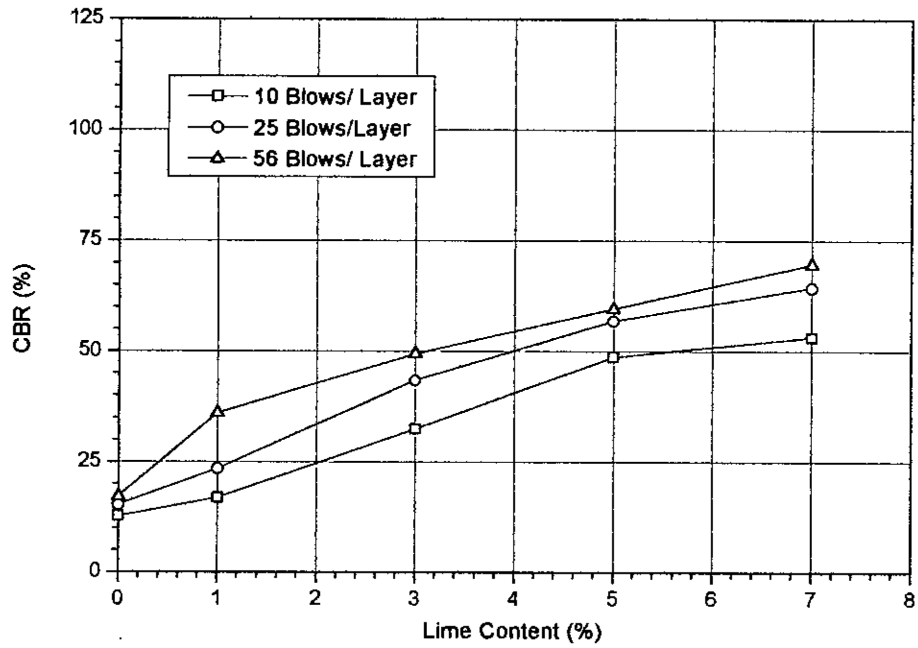

Fig: 4.34 Effect of lime content on CBR values of soil-B.

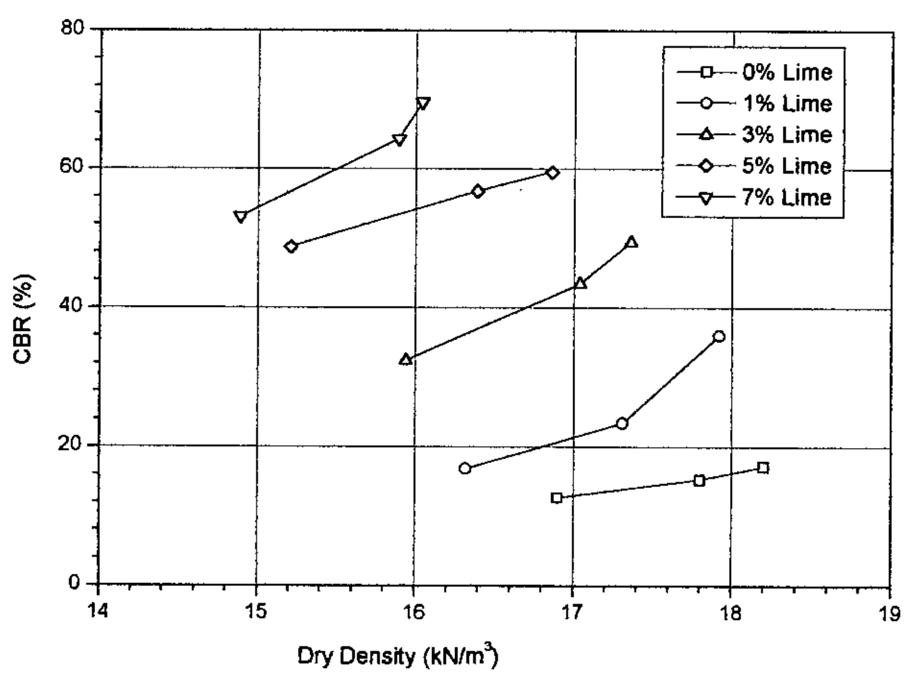

Fig: 4.35 CBR versus dry-density curves of lime-treated soil-B.

4.3.5 FLEXURAL STRENGTH AND MODULUS

The flexural properties of untreated and stabilized samples of Soil-B have been investigated by carrying out flexural strength test using simple beam test with third point loading. Typical flexural stress versus defection curves for stabilized sample of Soil-B is presented in Fig.4.36. From the flexural stress and deflection data, flexural strength and modulus were determined. The flexural properties of Soil-B are presented in Table 4.16. It can be seen from Table 4.16 that for Soil-B, compared with the untreated sample, flexural strength and modulus of the treated samples cured at 7 and 28 days increased significantly. It can be seen from Table 4.16 that compared with the untreated sample, the flexural strength and modulus of Soil-B treated with 7% lime and cured at 28 days are respectively about 2.4 times and 2.6 times higher than those of the untreated samples. The maximum deflection and failure strain of untreated and stabilized soil-lime beams were in the range of 0.49 mm to 1.06 mm and 0.22% to 0.48% respectively. Rajbongshi (1997) shows that compared with the untreated soils flexural strength and modulus of treated samples cured at 28 days increase significantly.

The effect of lime content on flexural strength for Soil-B is shown in Fig.4.37 while Fig.4.38 presents the effect of lime content on flexural modulus of Soil-B respectively. Figs.4.37 and 4.38 show that flexural strength and modulus increases with increasing lime content. It is evident from Figs.4.37 and 4.38 that curing age has got insignificant effect on increase in flexural strength and modulus.

Table 4.16 Flexural properties of lime-treated Soil-B

Lime Content (%)	Curing Age (Days)	Flexural Strength (kPa)	Maximum Deflection (mm)	Flexural Modulus (MPa)	Failure Strain (%)
0	-	47.3	0.76	23.4	0.35
1	7	54.4	0.86	24.0	0.37
	28	66.3	0.72	29.4	0.33
3	7	77.9	0.50	38.8	0.23
	28	81.7	0.71	52.7	0.32
5	7	85.0	0.58	43.1	0.27
	28	88.7	0.86	57.7	0.39
7	7	96.2	0.84	53.9	0.38
	28	116.8	1.06	62.7	0.49

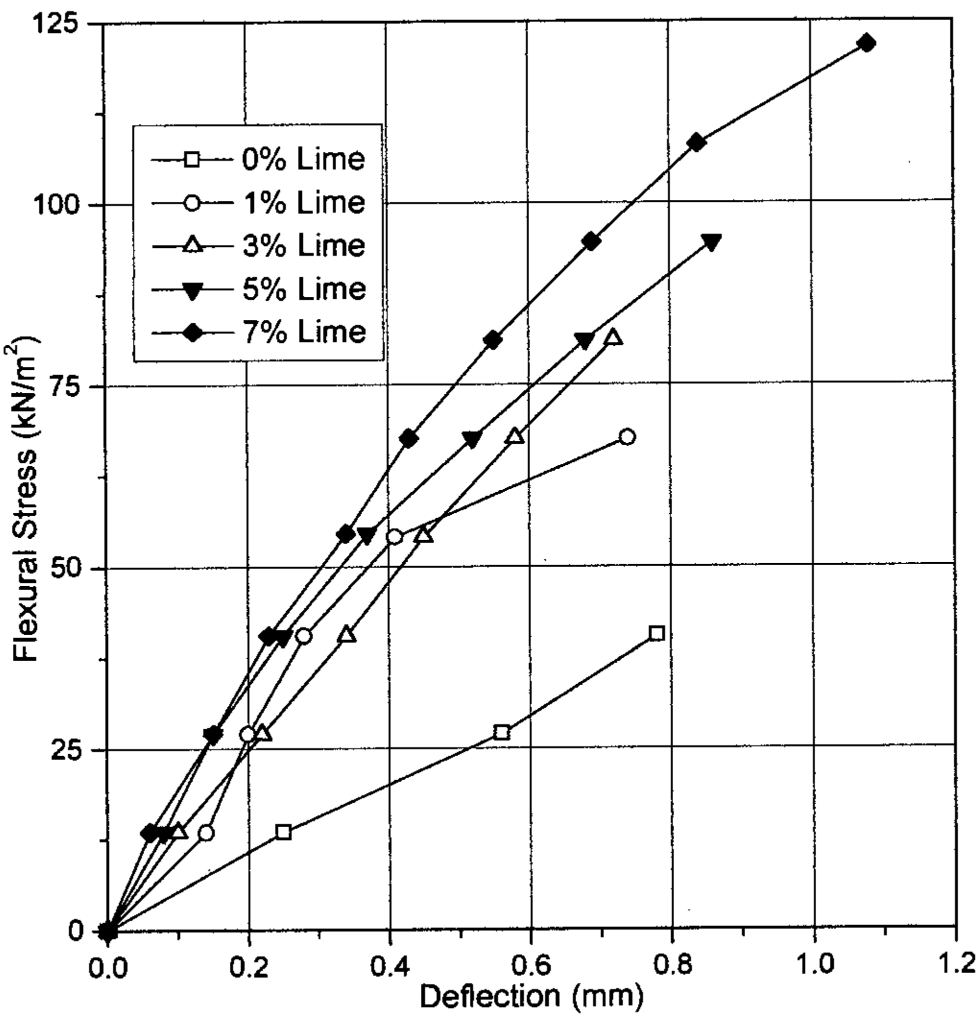

Fig: 4.36 Flexural stresses versus deflection curve of lime-treated soil-B.

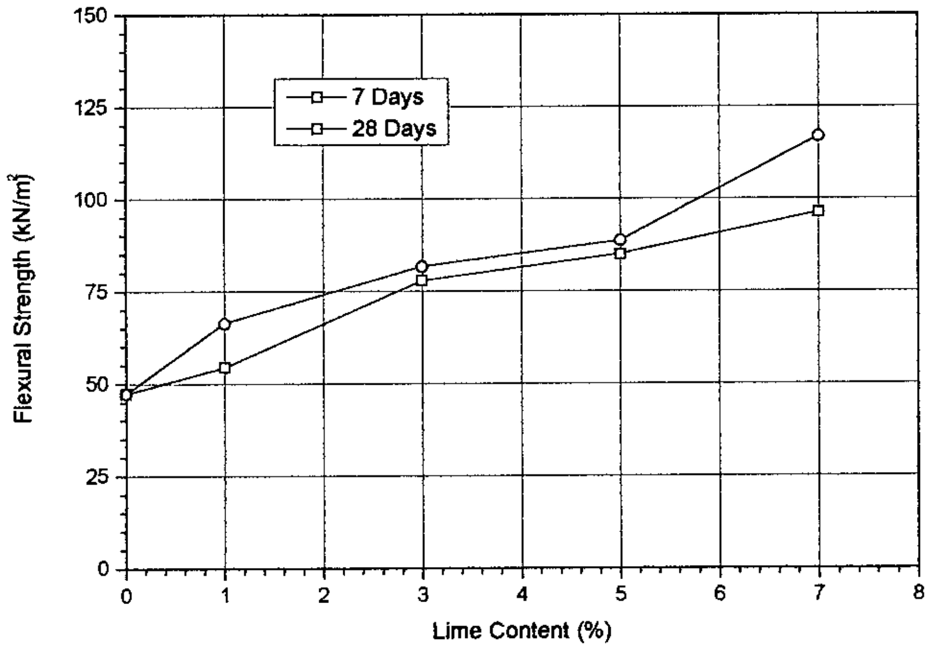

Fig: 4.37 Effect of lime content on flexural strength of soil-B

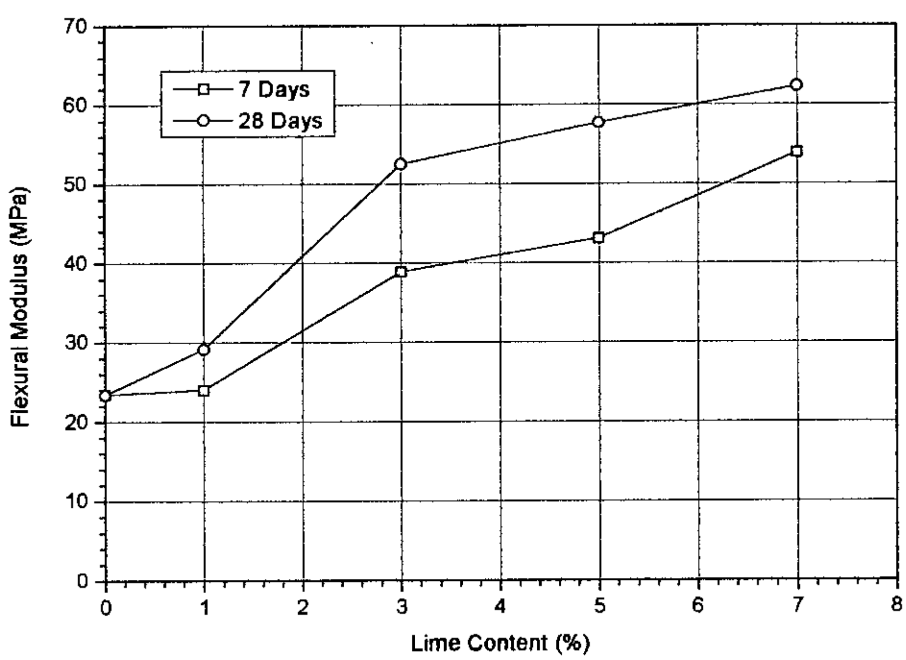

Fig: 4.38 Effect of different molding water content on the unconfined compressive strength for 3% cement and lime content of soil-B.

4.4 COMPARISON OF PROPERTIES OF CEMENT AND LIME STABILIZED SAMPLES

In the previous sections, the properties of cement-stabilized samples (1%, 3% and 5% cement contents) of Soil-A and (1%, 3%, 5% and 7% cement contents) of Soil-B, and the properties of lime-stabilized samples (1%, 3%, 5% and 7% lime contents) of Soil-B have been presented. Attempt has been made to compare the physical and engineering characteristics of cement and lime stabilized samples of Soil-B treated with only 1%, 3% 5% and 7% additives.

4.4.1 INDEX AND SHRINKAGE PROPERTIES

A comparison of index and shrinkage properties of cement and lime treated samples of Soil-B is presented in Table 4.17.

Table 4.17 Comparison of index properties and shrinkage properties of cement and lime stabilized samples of Soil-B

Percent of Additive	Type of Additive	Properties				
		Liquid Limit	Plastic Limit	Shrinkage Limit	Linear Shrinkage	Plasticity Index
1	Cement	51	25.0	21.5	13	26
	Lime	50.5	23.5	15.0	13	27.0
3	Cement	48.0	30.0	20.0	12.0	18.0
	Lime	49.0	24.0	15.5	11.0	25.0
5	Cement	46.5	31.5	19.0	10.0	15.0
	Lime	48.0	25.0	16.0	8.0	23.0
7	Cement	45.5	32.0	18.0	9.0	13.5
	Lime	46.5	25.5	17.0	6.0	22.5

It can be seen from Table 4.17 that insignificant changes in the values of liquid limit, plastic limit and plasticity index and linear shrinkage of the cement and lime stabilized samples have occurred. However, Table 4.17 shows that the values of shrinkage limit of the cement treated soil reduced with the increase in cement content and increases with the increase in lime content.

4.4.2 MOISTURE DENSITY RELATIONS

A comparison of optimum moisture content and dry density of cement and lime treated soils at different percent of additive are shown in table 4.18. Fig 4.39 shows that for cement treated soil maximum dry density increases with the increase in cement content while the optimum moisture content decreases. Fig 4.39 also shows that for lime treated soil maximum dry density decreases with the increase in lime content while the optimum moisture content increases. For cement treated soil with 7% cement, maximum dry density increases by 6 percent while for lime treated soil with 7% lime, maximum dry density decreases by 4.3 percent. For cement treated soil with 7% cement, optimum moisture content decreases by 1.4 percent while for lime treated soil with 7% lime, optimum moisture content increases by 2.4 percent. Fig 4.40 shows the effect of cement and lime content on optimum moisture content of cement and lime stabilized soils.

Table 4.18 Values of maximum dry density and optimum moisture content of untreated and cement/ lime treated Soil-B

Cement /Lime Content (%)	Cement Treated Soil-B		Lime Treated Soil-B	
	γ_d (kN/m^3)	w_{opt} (%)	γ_d (kN/m^3)	w_{opt} (%)
0	18.5	11.5	18.5	11.5
1	18.8	11.3	18.4	11.9
3	19.1	10.8	18.3	13.0
5	19.5	10.3	17.9	13.6
7	19.6	10.1	17.7	13.9

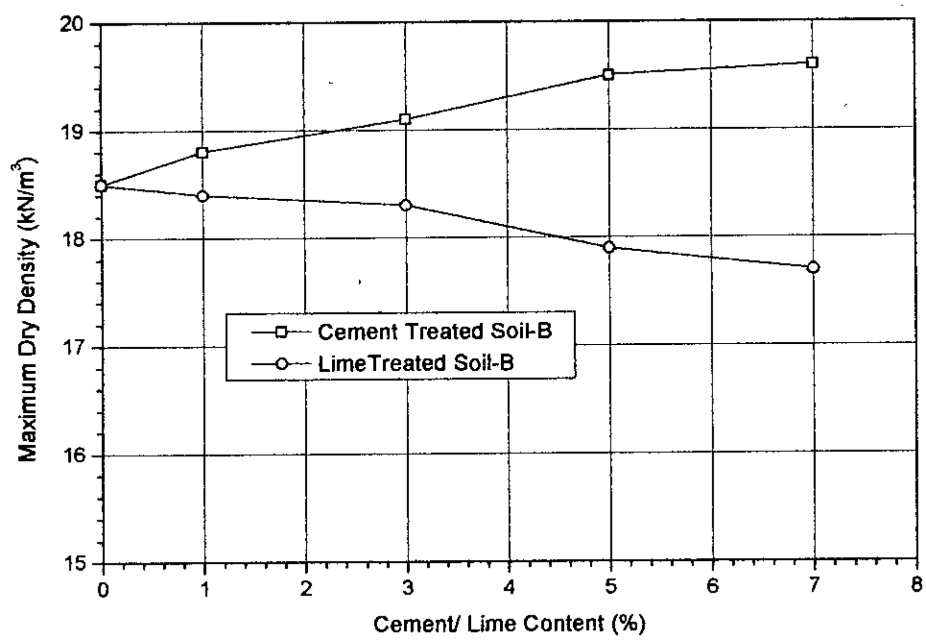

Fig 4.39 Effect of cement and lime content on maximum dry density of cement and lime stabilized soil-B.

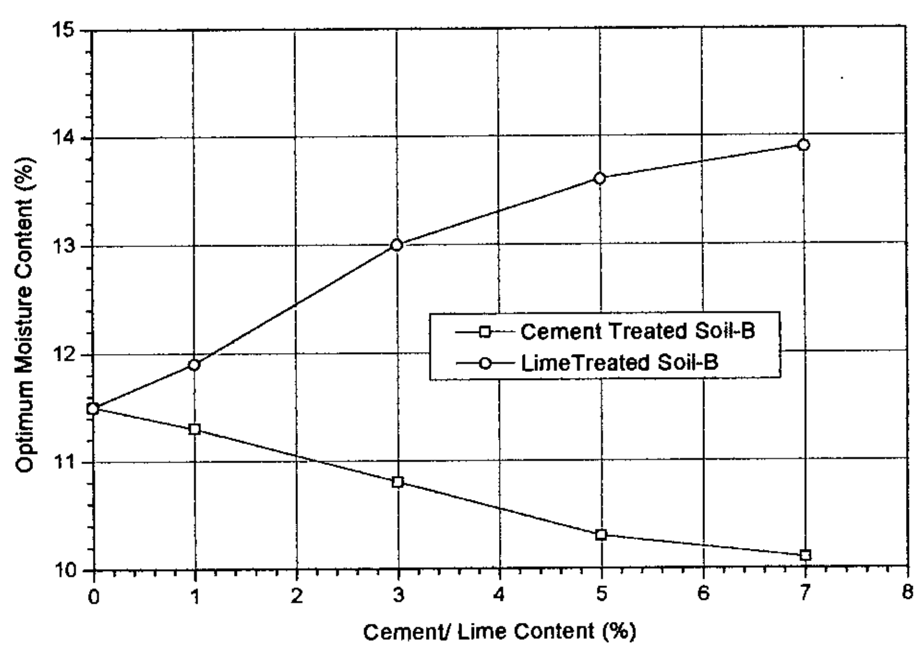

Fig 4.40 Effect of cement and lime content on optimum moisture content of cement and lime stabilized soil-B.

4.4.3 UNCONFINED COMPRESSIVE STRENGTH

A comparison of unconfined compressive strength (q_u) of the cement and lime stabilized samples cured at different ages is shown in Fig.4.41. It can be seen from Fig.4.41 that at a particular percentage of additive, the values of q_u are significantly higher for the cement treated samples than for the lime stabilized samples. Similar results have also been reported by Ahmed (1984), Serajuddin and Azmal (1991), Serajuddin (1992), Rajbongshi (1997), Mollah (1997) and Shahjahan (2001) for a number of regional soils of Bangladesh stabilized with various cement and lime contents. It has been found that the values of q_u of samples treated with 3%, 5% and 7% cement and cured at 28 days are respectively 22%, 28% and 34% higher than the respective lime stabilized samples. A comparison of unconfined compression strength test results of cement and lime treated soils are shown in table 4.19. A comparison of the effect of Molding water content on unconfined compressive strength of the cement and lime stabilized samples is presented in Fig.4.42. Fig.4.42 shows that at all molding water contents, the values of q_u of the cement treated samples are considerably higher than those of the lime treated samples.

Table 4.19 Unconfined compressive strength test results of untreated and cement /lime treated Soil-B

Cement/ Lime Content (%)	Curing Age (Days)	Cement Treated Soil-B	Lime Treated Soil-B
		q_u (kPa)	q_u (kPa)
0	-	380	380
1	7	482	470
	14	653	775
	28	1020	984
3	7	946	1020
	14	1636	1381
	28	2464	2015
5	7	2188	1877
	14	2551	2192
	28	3075	2385
7	7	2671	2173
	14	2892	2304
	28	3588	2678

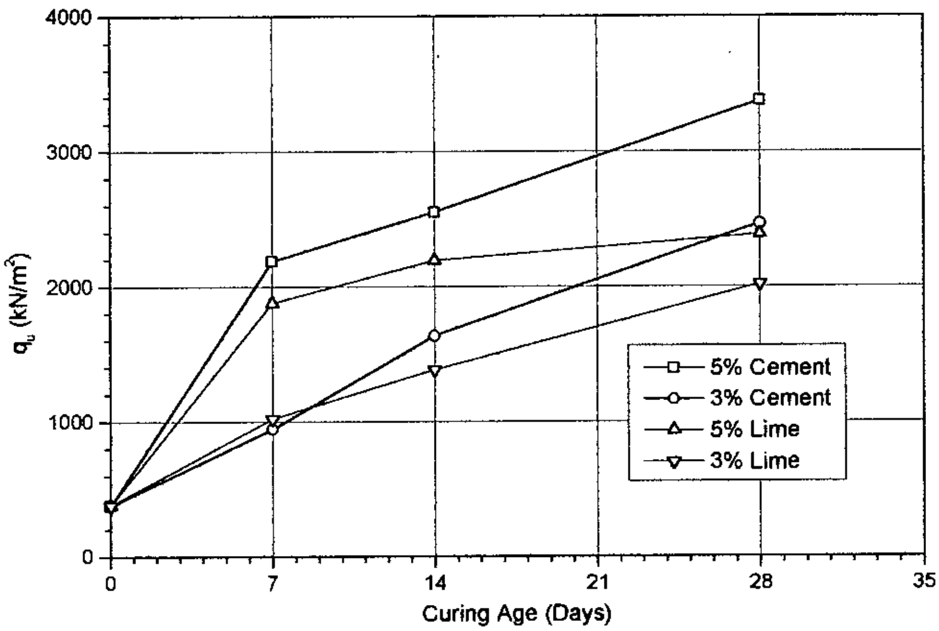

Fig: 4.41 Effect cement and lime content on unconfined compressive strength of soil-B for different curing ages.

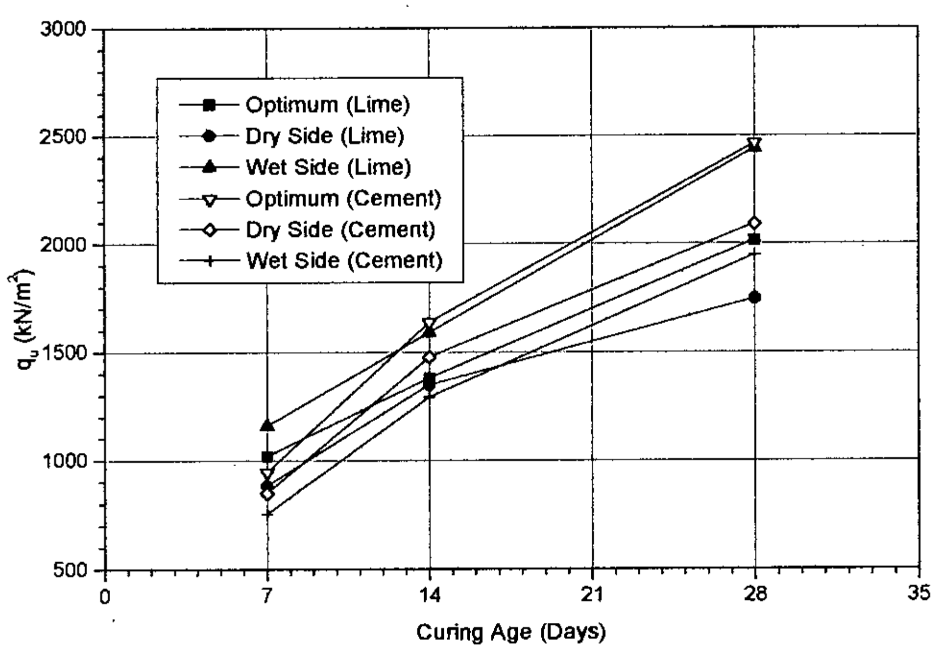

Fig: 4.42 Effect of curing age on q_u for soil B treated with 3% cement and lime and compacted with different moulding water contents.

4.4.4 CALIFORNIA BEARING RATIO (CBR)

Table 4.20 shows a comparison of CBR-values at different levels of compaction for the cement and lime stabilized samples. It can be seen from Table 4.20 that at all levels of compaction efforts; the values of CBR of the cement treated samples are considerably higher than those of the lime treated samples. It has been found that the values of CBR of samples treated with 1%, 3%, 5% and 7% cement are respectively 55%, 42%, 50% and 43% higher than the respective lime stabilized samples. Fig: 4.43 shows the comparison of CBR values of cement and lime-treated soil-B at different level of compaction.

Table 4.20 Comparison of CBR values of samples of cement and lime-treated Soil-B at different levels of compaction

Percent of Additive	Type of Additive	4 days soaked CBR		
		Low Compaction	Medium Compaction	High Compaction
1	Cement	23	39	56
	Lime	17	23	36
3	Cement	30	41	70
	Lime	32	43	49
5	Cement	50	65	90
	Lime	49	57	60
7	Cement	63	79	100
	Lime	53	64	70

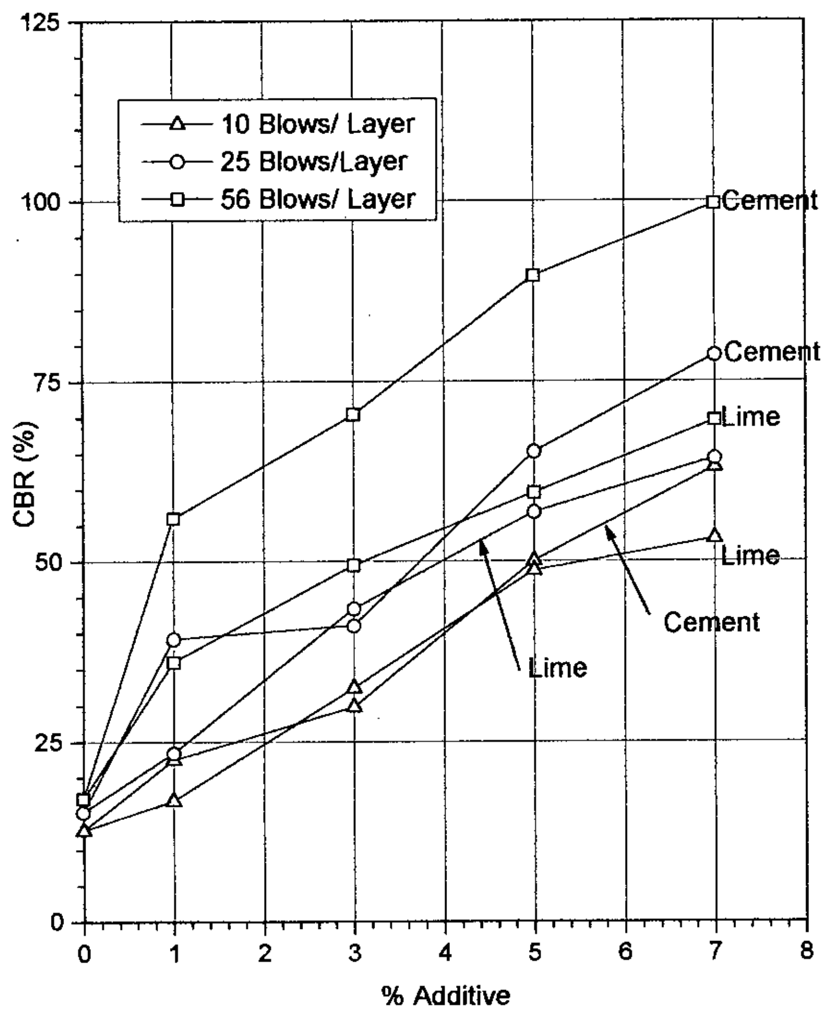

Fig: 4.43 Comparison of CBR values of cement and lime-treated soil-B at different level of compaction.

4.4.5 FLEXURAL PROPERTIES

A comparison of flexural strength and flexural stiffness of cement and lime stabilized samples cured at 7 and 28 days is presented in Table 4.21. Fig 4.44 and 4.45 shows the comparison of flexural strength and flexural modulus of cement and lime stabilized samples cured at 7 and 28 days. Table 4.21 shows that at both the percentages of additive and at all curing ages, the flexural strength and flexural modulus of cement stabilized samples are significantly higher than those of the respective lime stabilized samples. From Table 4.21, it has been found that the flexural strength values of samples treated with 3%, 5% and 7% cement and cured at 28 days are respectively 57%, 83% and 125% higher than the respective lime stabilized samples. It has also been found that the values flexural modulus of samples treated with 3%, 5% and 7% cement and cured at 28 days are respectively 91%, 101% and 101% higher than the respective lime stabilized samples.

It is evident from the comparisons of the engineering properties between cement-treated and lime-treated samples of Soil-B, as presented in the previous sections, that unconfined compressive strength, CBR, flexural strength and flexural modulus of the cement-treated samples are significantly higher than those of the lime-treated samples. Besides, compared with soil-lime mix, soil-cement mix is usually much more durable in the prevailing weather conditions of tropical regions. Serajuddin (1993) concluded that lime stabilization is not suitable for samples of fine-grained soils from Chittagong due to very small increase in unconfined compressive strength in stabilized samples. Rajbongshi (1997) concluded that for road construction cement stabilized coastal soils is more effective than lime stabilized coastal soils.

Table 4.21 Comparison of flexural properties of samples of cement and lime-treated Soil-B

Percent of Additive	Type ofAdditive	Curing Age (Days)	Flexural Strength (kPa)	Flexural Modulus (MPa)
1	Cement	7	81.1	26.0
		28	94.6	40.8
	Lime	7	54.4	24.0
		28	66.3	29.1
3	Cement	7	108.2	66.6
		28	128.4	82.3
	Lime	7	77.9	38.8
		28	81.7	52.5
5	Cement	7	135.2	97.1
		28	162.2	116.2
	Lime	7	85.0	43.1
		28	88.7	57.7
7	Cement	7	196.0	99.4
		28	203.6	125.5
	Lime	7	96.2	53.9
		28	116.8	62.3

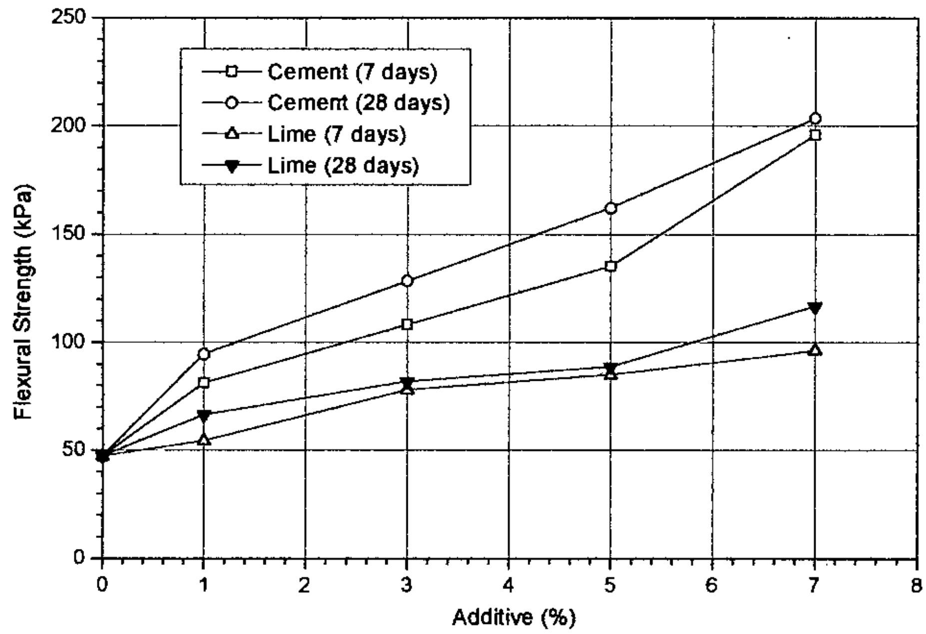

Fig: 4.44 Comparison of flexural strength of cement and lime-treated soil-B at different level of compaction.

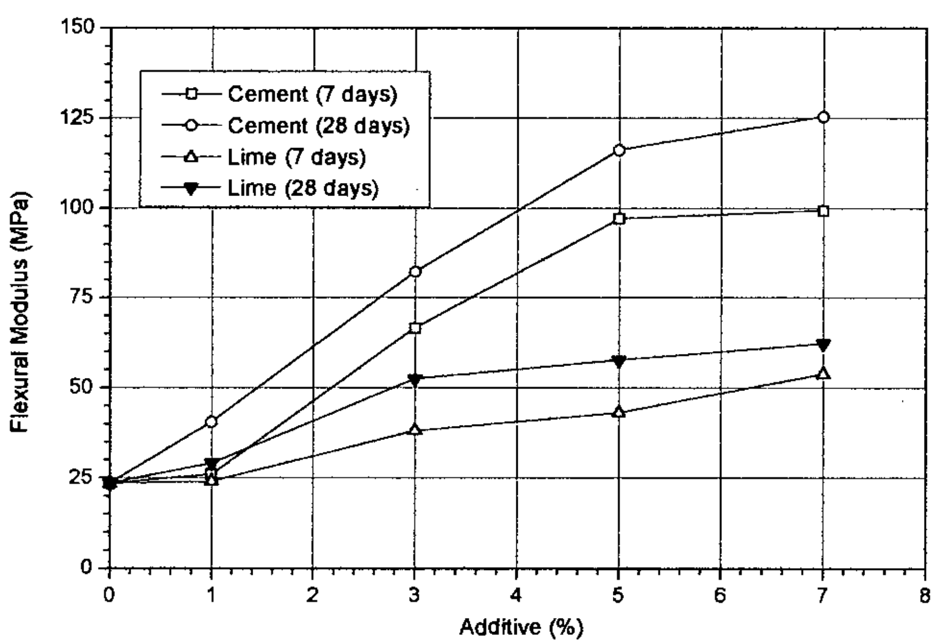

Fig: 4.45 Comparison of flexural modulus of cement and lime-treated soil-B at different level of compaction.

4.5 A STUDY OF PROPERTIES OF CEMENT AND LIME STABILIZATION BETWEEN DIFFERENT REGIONAL SOILS OF BANGLADESH AND THE SOIL USED IN THE PRESENT STUDY

The present investigations were carried out on two disturbed reclaimed soils collected from Aminbazar and Bashundhara. Aminbazar soil were stabilized with cement in percentages of 1, 3 and 5, and Bashundhara soil were stabilized with cement in percentages of 1, 3, 5 and 7, while Bashundhara soil were stabilized with lime in percentages of 1, 3, 5 and 7. Presently a study has been made in the following two articles between different regional soils of Bangladesh and the soil used in the present study.

Uddin (1984) worked on Meghna, Manikganj and Dhaka soils, Ahmed (1986) worked on Dhaka and Gazipur soils, Hossain (1991) worked on Dhaka soils, Mollah (1997) worked on Dhaka, Munshiganj and Shirajganj soils, Rajbongshi (1997) worked on coastal soils of Chittagong coastal area, Hossain (2001) worked on expansive soils of Rajendrapur and Shahjahan (2001) worked on long time curing effect on stabilized Dhaka soils. Among them Mollah (1997), Hossain (2001) and Shahjahan (2001) worked on lime stabilization and other researchers' worked on both lime and cement stabilization.

In the following sections a study has been made among the cement stabilized regional soils on the basis of physical and engineering properties. The physical and engineering characteristics comprising plasticity and shrinkage properties, moisture-density relations, unconfined compressive strength, CBR, flexural properties and durability of untreated and cement treated regional soils of Bangladesh are presented below.

4.5.1 PLASTICITY AND SHRINKAGE CHARACTERISTICS

Atterberg Limits of eighteen regional soils of Bangladesh is presented in table 4.22. The liquid limit of the investigated regional soils varies between 25 to 56 and plastic limit varies between 13 to 30. Shrinkage limit from the given data varies from 11 to 23 and plasticity index ranges from 5.5 to 43.

Table 4.21 presents specific gravity, percent clay, silt and some other parameters. AASHTO and Unified Classification also presents in the table 4.23. The percent clay varies from 0 to 46, percent silt varies from 38 to 89.5 and the percent sand varies from 1 to 62. The specific gravity ranges from 2.57 to 2.76. AASHTO classification numbers A-4, A-5, A-6, A-7-5 and A-7-6 and unified classification symbols are CL, CH, ML.

Study of index properties of cement treated regional soils have presented by table 4.24. Out of six treated samples, it is found that liquid limit varies from 25 to 52, plastic limit varies from 13 to 41.5 and shrinkage limit ranges from 18 to 25.5.

Table 4.22 Atterberg Limits of different regional soils of Bangladesh

Location of Soil	Code Used	Source	Liquid Limit	Plastic Limit	Plasticity Index	Shrinkage Limit
Bilamalia, Savar	SH-A	Shahjahan (2001)	46	27	19	-
Beraid, Badda	SH-B	Shahjahan (2001)	48	28	20	-
Patia, Rupganj	SH-C	Shahjahan (2001)	40	23	17	-
Rajendrapur, Gazipur	H-1	Hossain (2001)	56	13	43	11
Jamuna Bridge Site, Sirajganj	M-1	Molla (1997)	34	21	13	-
BUET Campas, Dhaka	M-2	Molla (1997)	47	21	26	-
Dhaleshari Bridge Site, Munshiganj	M-3	Molla (1997)	37	28	9	-
Katasur, Mohammadpur	H-A	Hossain (1991)	25	13	12	-
Chandina, Savar	H-B	Hossain (1991)	42	22	20	-
Anwara, Chittagong	R-A	Rajbongshi (1997)	30	23	7	19.8
Banshkhali, Chittagong	R-B	Rajbongshi (1997)	44	25	19	23.0
Nayarhat, Dhaka	M-A	Hossain (1986)	-	-	-	-
Kaliakoir, Gazipur	M-B	Hossain (1986)	33	27.5	5.5	-
Bank of Meghna	N-I	Uddin (1984)	-	-	-	-
Manikgang	N-II	Uddin (1984)	40	30	10	-
Sher-a-Bangla Nagar, Dhaka	N-III	Uddin (1984)	43	22	21	-
Aminbazar, Dhaka	AH-A	Present Study	41	29	12	19
Bashundhara, Dhaka	AH-B	Present Study	52	23	29	14

Table 4. 23 Classification of different regional soils of Bangladesh

Soil Code	(%) Clay	(%) Silt	(%) Sand	Specific Gravity	Maximum Dry Density (kN/m³)	Optimum Moisture Content (%)	Classification	
							AASHTO	Unified
SH-A	24	72	4		16.4	21.7	A-7-6	CL
SH-B	26	71	3		16.1	22.1	A-7-6	CL
SH-C	20	64	16		17.3	18.4	A-6	CL
H-1	46	50	1	2.57	17.1	18.1	-	CH
M-1	7	81	12		16.1	12.5	A-6 (11)	CL
M-2	21	60	19		15.9	21.0	A-7-6 (20)	CL
M-3	4	86	10		15.8	18.8	A-4 (9)	ML
H-A	22.8	69.2	8	2.67	15.7	-	A-6 (8)	-
H-B	20	67	13	2.70	14.7	-	A-7-6 (8)	-
R-A	4	62	34	2.68	17.1	16.4	A-4	ML
R-B	26	68	6	2.70	17.5	15.5	A-7-6	CL
M-A	0	86	14	2.63	15.3	15.0	A-4	CL
M-B	4	89.5	6.5	2.68	16.2	18.0	A-4 (0)	CL
N-I	0	38	62	2.61	16.0	17.5	A-4	ML
N-II	5	87	8	2.73	15.7	20	A-4 (12)	ML
N-III	10	63	27	2.76	15.9	21	A-7-6 (17)	CL
AH-A	20	76	2	2.74	16.5	18	A-5	ML
AH-B	34	50	16	2.67	18.5	11.5	A7-5	CH

4.5.2 MOISTURE-DENSITY RELATION OF CEMENT STABILIZED SOILS

The moisture-density relations of untreated and cement treated regional soils are shown in Fig 4.46 and Fig 4.47 From the relations shown in fig 4.46 and fig 4.47 the maximum dry density and optimum moisture content have been determined which are presented in Table 4.25 From all the regional soils data shown in the table 4.25 it is observed that the optimum moisture content decreases while the maximum dry density increase with the increase in cement content. Similar results have reported by Kezdi (1979), Felt (1955), BRTC (1995) and Rajbonshi (1997). Cement stabilization of six regional soils is compared. The optimum moisture content is found to be decrease with the increase in cement content and ranges from 22.6% to 10.1%. The maximum dry density increases with the increase in cement (%) and ranges from 14.5 kN/m^3 to 19.6 kN/m^3.

Table 4. 24 Index properties of cement treated regional soils

Soil Code	Cement Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)
AH-A	0	41.0	29.0	25.5
	1	42.0	31.0	25.0
	3	44.5	34.5	24.0
	5	46.5	37.0	22.0
AH-B	0	52.0	32.0	22.0
	1	51.0	31.5	21.5
	3	48.0	30.0	20.0
	5	46.5	25.0	19.0
	7	45.5	23.0	18.0
H-A	0	25.0	13.0	-
	2	33.0	24.5	-
	4	34.0	29.0	-
	6	35.5	32.0	-
	8	36.0	33.5	-
	10	38.0	37.0	-
H-B	0	42.0	22.0	-
	2	36.5	22.0	-
	4	38.0	25.5	-
	6	39.5	29.0	-
	8	43.0	35.0	-
	10	44.5	41.5	-
R-A	0	30.0	23.0	19.8
	1	31.5	26.0	19.5
	3	32.0	27.5	19.0
	5	33.0	30.5	18.8
R-B	0	44.0	25.0	23.0
	1	43.0	28.0	22.3
	3	42.5	30.0	21.2
	5	42.0	32.5	20.0

Table 4. 25 Moisture-density relations of different cement treated regional soils of Bangladesh

Soil Code	Content (%)	Optimum Moisture Content (%)	Maximum Dry Density (kN/m^3)
R-A	0	16.4	17.1
	1	15.5	17.4
	3	15.2	18.2
	5	14.9	18.5
R-B	0	15.5	17.5
	1	15.0	17.7
	3	14.0	18.3
	5	13.9	18.7
N-I	0	17.4	15.6
	2	17.0	15.1
	5	17.8	17.1
	10	17.4	15.9
N-II	0	19.8	15.4
	2	20.4	14.5
	5	20.3	14.5
	10	20.1	14.6
N-III	0	22.6	15.7
	2	21.4	15.3
	5	20.1	14.8
	10	21.3	15.0
AH-A	0	18.0	16.5
	1	17.8	16.8
	3	17.1	17.1
	5	16.0	17.8
AH-B	0	11.5	18.5
	1	11.3	18.8
	3	10.8	19.1
	5	10.3	19.5
	7	10.1	19.6

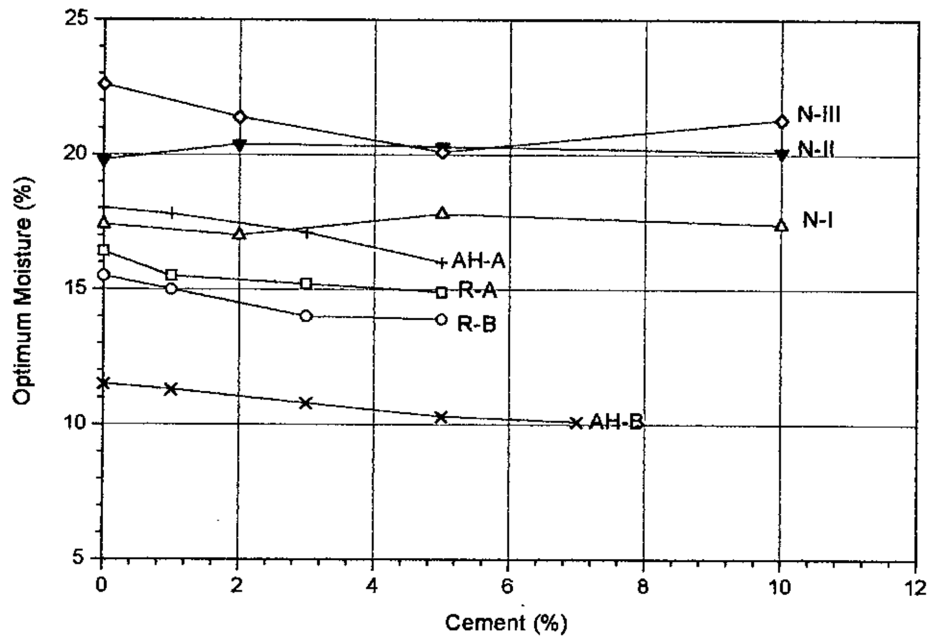

Fig: 4.46 Effect of cement stabilization on optimum moisture content between different regional soils of Bangladesh and soils used in the present study.

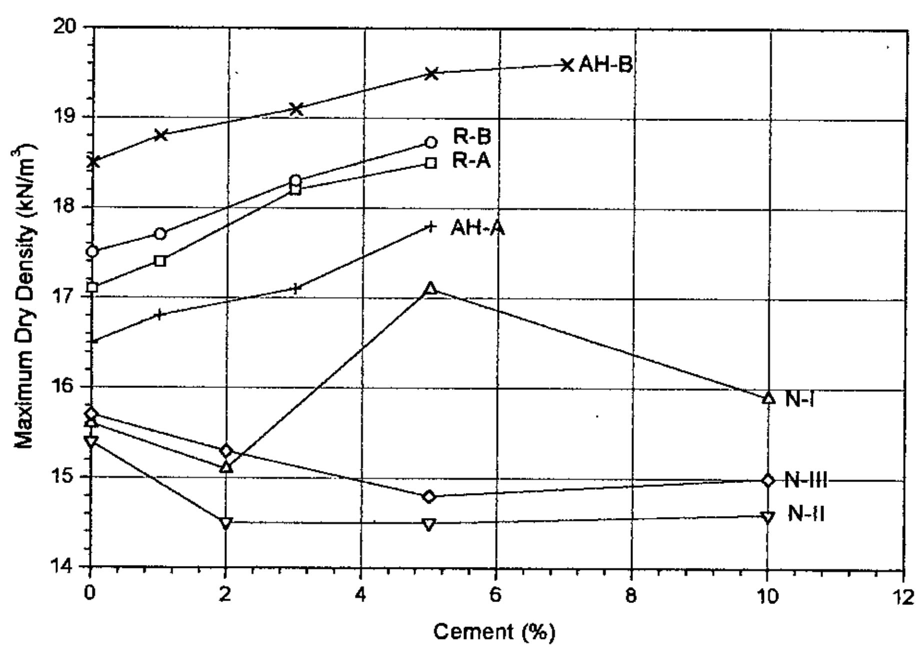

Fig: 4. 47 Effect of cement stabilization on dry density between different regional soils of Bangladesh and soils used in the present study.

4.5.3 UNCONFINED COMRESSIVE STRENGTH OF CEMENT STABILIZED SOILS

Table 4.26 shows summary of unconfined compressive strength of regional soils of Bangladesh. The unconfined compressive strength of untreated samples and samples treated different cement contents and cured for 28 days are presented. It can be seen that for all the regional soils, compared with the untreated soils, the values of unconfined compressive strength increases significantly with the increase in cement. The unconfined compressive strength of the cement stabilized soils increases 2 to 6 times compared with the untreated soils. Fig. 4.48 shows the relation of unconfined compressive strength and cement content. These results are in agreement with those reported by number of researcher (PCA, 1956; Igles and Metcalf, 1972; Ramaswamy et. al., 1984; Hong, 1989; Anon, 1990). In general unconfined compressive strength increases with the increase in cement content. From the eleven investigated samples it is found that the ranges of unconfined compressive strength is between 51.8 kN/m^2 to 4304 kN/m^2. The compressive strength of coastal soils was found to be higher than other regional soils. The trend of increase in compressive strength of coastal and reclaimed soils was found same.

Table 4. 26 Unconfined compressive strength (28 days) for cement stabilized regional soils of Bangladesh.

Soil Code	Cement Content (%)	Unconfined Compressive Strength (kN/m^2)	Soil Code	Cement Content (%)	Unconfined Compressive Strength (kN/m^2)
H-A	0	-	N-I	0	51.8
	2	894.5		2	273.6
	4	2539.6		5	839.4
	6	2835.0		10	1969.1
H-B	0	-	N-II	0	348.4
	2	515.4		2	304.5
	4	838.42		5	1404.2
	6	1293.5		10	1628.2
R-A	0	710	N-III	0	424.3
	1	1729		2	213.3
	3	3431		5	368.3
	5	4304		10	2447.1
R-B	0	692	AH-A	0	460
	1	1442		1	1380
	3	3177		3	2933
	5	3935		5	3050
M-A	0	-	AH-B	0	380
	2	731.1		1	1020
	8	1042.9		3	2464
	10	1224.3		5	3375
M-B	0	-		7	3588
	2	872.0			
	8	1088.5			
	10	1306.5			

4.5.4 CALIFORNIA BEARING RATIO (CBR) OF CEMENT STABILIZED SOILS

A study of CBR test for different levels of compaction energy and 4 days soaking time of regional soils are represented in table 4.27. In these tests upto 8% cement is used. It shows that, compared with the untreated soils, CBR values of the treated soils at all levels increase considerably with increase in cement content. The variation of CBR with cement content of regional soils are shown in Fig. 4.49.

It can be seen from Table 4.27 that the CBR values of the regional treated soils increase upto about 6 times than those of the respective untreated samples. It is also evident from the CBR data present in table 4.27 that the CBR values of samples of less plastic soils are moderately higher than those for the samples of more plastic soils. Similar trends of increasing CBR with the increasing cement content have been found for regional stabilized soils. (BRTC, 1995). From six tested results at high compaction energy and 4 days curing period, shown in table 4.27, CBR values ranges from 17 kPa to 120 kPa.

Table 4. 27 CBR values for cement stabilized regional soils of Bangladesh

Soil code	Cement Content (%)	CBR Value		
		Low Compaction	Medium Compaction	High Compaction
H-A	0	-	-	-
	2	4	23	31
	4	21	35	44
	6	29	46	61
H-B	0	-	-	-
	2	3	7	19
	4	14	25	28
	6	18	32	39
R-A	0	11	16	24
	1	27	34	44
	3	40	63	92
	5	51	90	120
R-B	0	10	13	21
	1	25	33	43
	3	42	60	87
	5	49	79	91
AH-A	0	15	18	20
	1	39	44	52
	3	55	60	77
	5	71	89	102
AH-B	0	12	15	17
	1	22	39	56
	3	29	41	70
	5	50	65	90
	7	63	78	99

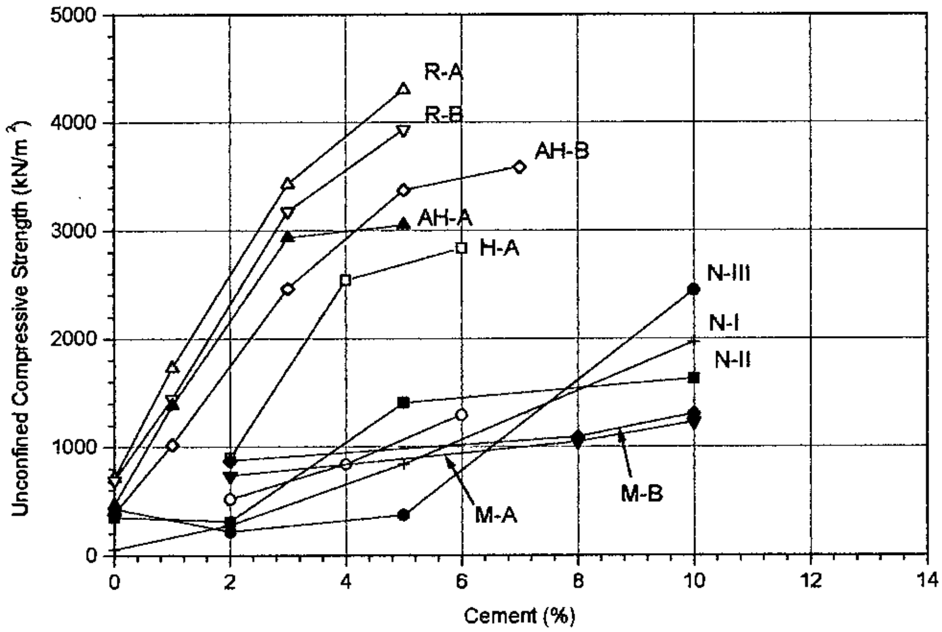

Fig: 4. 48 Effect of cement stabilization on unconfined compressive strength between different regional soils of Bangladesh and soils used in the present study.

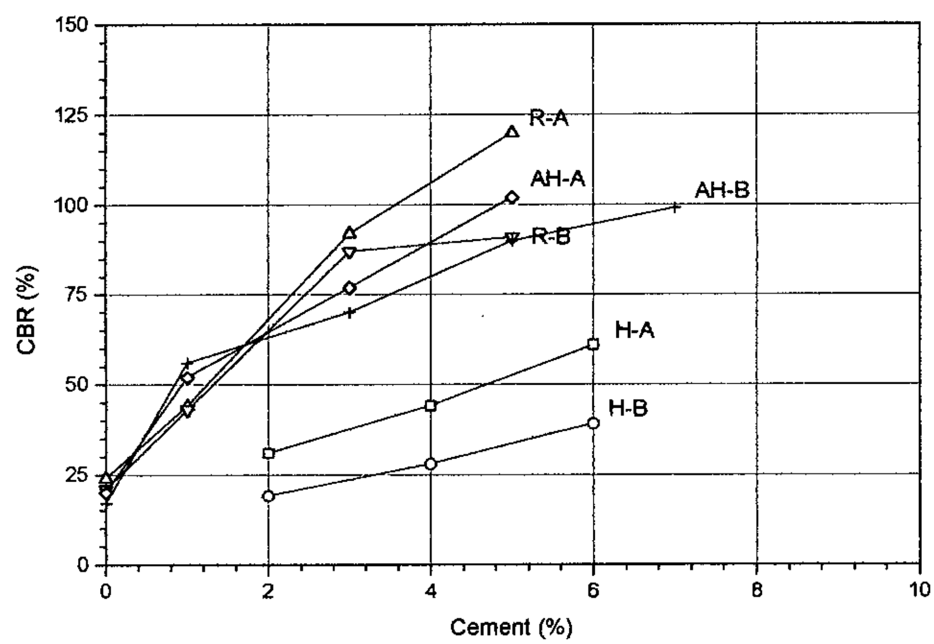

Fig: 4. 49 Effect of cement stabilization on CBR between different regional soils of Bangladesh and soils used in the present study.

4.5.5 FLEXURAL STRENGTH AND MODULUS OF CEMENT STABILIZED SOILS

The flexural strength and modulus of treated and untreated samples of regional soils for 28 days curing and treated with various cement content are shown in table 4.28. It is found that with the increase in cement content both the flexural strength and modulus increase significantly. The flexural strength cement treated soils increases upto 6 times and the flexural modulus of cement treated soil increases upto 5 times higher than those of the untreated soils. It is also evident from the data present in table 4.28 that the flexural strength and flexural modulus values of samples of less plastic soils are moderately higher than those for the samples of more plastic soils. The effect of cement content on flexural strength for regional soils is shown in Fig. 4.50 while Fig. 4.51 shows the effect of cement content on flexural modulus for regional soils. The range of flexural strength is between 26.9 kPa to 286 kPa and flexural modulus varies between 17.3 kPa to 136 kPa. The flexural strength of coastal soils was found to be higher than other reclaimed soils. The trend of increase in flexural strength of coastal and reclaimed soils was found same.

Table 4. 28 Flexural properties (28 days) for cement stabilized regional soils of Bangladesh

Soil code	Cement Content %	Flexural Strength (kPa)	Flexural Modulus (MPa)
R-A	0	67.5	49.8
	1	84.4	80.9
	3	128.3	86.8
	5	266.7	133.3
R-B	0	49.1	32.7
	1	101.2	89.7
	3	134.9	98.7
	5	286.8	136.0
AH-A	0	26.9	17.3
	1	60.8	25.5
	3	81.1	28.5
	5	121.6	58.2
AH-B	0	47.3	23.3
	1	94.6	40.7
	3	128.4	82.3
	5	162.2	116.2
	7	203.6	125.5

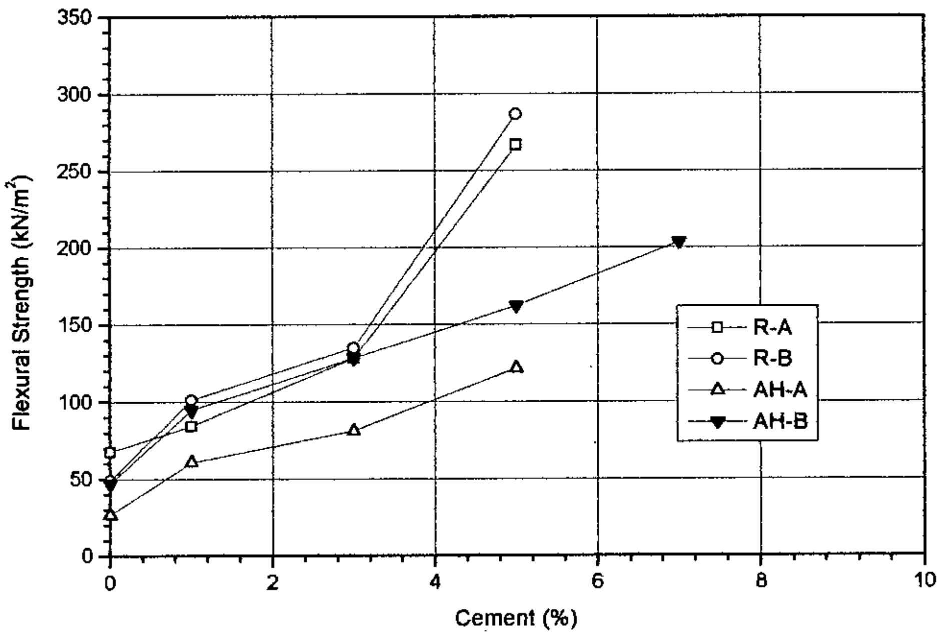

Fig: 4.50 Effect of cement stabilization on flexural strength between different regional soils of Bangladesh and soils used in the present study.

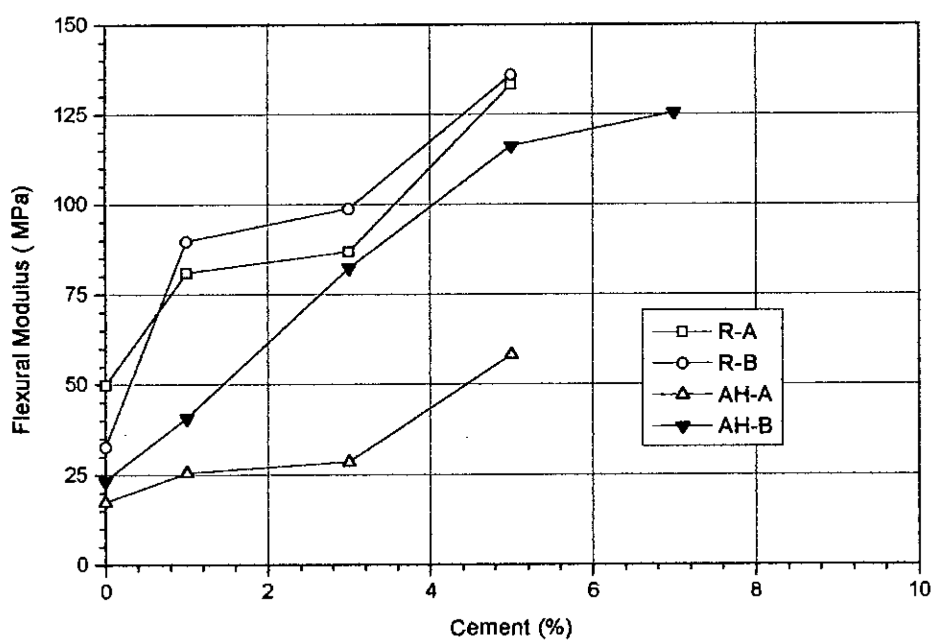

Fig: 4.51 Effect of cement on flexural modulus for the regional soils of Bangladesh.

4.5.6 DURABILITY OF CEMENT STABILIZED SOILS

Durability of hardened soil-cement mixed soil have been assessed by performing repeating wetting and drying tests. Soil-cement loss in wetting and drying tests for hardened soil-cement mixed samples for different regional soils stabilized upto 8% cement is shown in the table 4.29. From all the data it was found that the loss in soil-cement reduced with the increase in cement content. PCA (1956) suggested that a maximum of 10% loss of soil-cement in the wetting and drying test is allowable for all type of soils. It needs addition of higher percentages of cement to satisfy the PCA criteria. Hossain and Molla found that addition of about 8% cement satisfied the PCA (9156) criteria. The relationship between soil cement loss and cement content for regional soils are presented in Fig. 4.52. Durability of cement stabilized soil has been checked by measuring soil cement loss, which were ranges from 10.6 to 42.7%.

Table 4. 29 Soil-cement loss for cement stabilized regional soils of Bangladesh

Soil Code	Cement Content (%)	Soil-cement loss (%)
R-A	1	25.8
	3	20.4
	5	16.5
R-B	1	26.9
	3	22.5
	5	18.2
M-A	2	31.4
	4	22.2
	6	15.4
	8	10.6
M-B	2	42.7
	4	18.3
	6	16.8
	8	12.4
AH-A	1	28.6
	3	24.3
	5	18.6
AH-B	1	25.5
	3	22.1
	5	16.8
	7	11.7

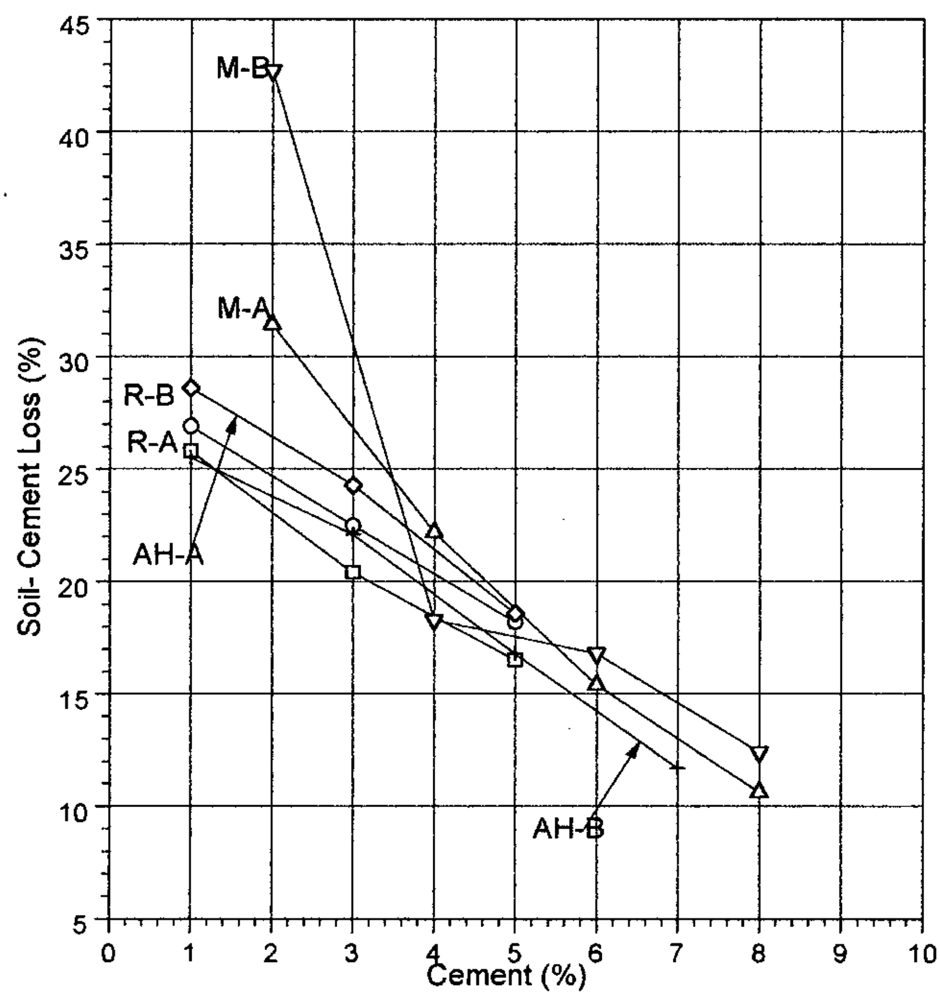

Fig: 4.52 Effect of cement stabilization on soil-cement loss between different regional soils of Bangladesh and soils used in the present study.

4.5.7 INDEX PROPERTIES AND MOISTURE DENSITY RELATIONS OF LIME STABILIZED SOILS

Index properties of lime treated regional soils and soils investigated in the present study are described by the table 4.30. For all the cases the shrinkage limit increase with the increases in lime content. The shrinkage limit varies from 11 to 37. The liquid limit varies from 25 to 59 and plastic limit varies from 13 to 45.

The moisture-density relations of untreated and lime treated regional soils are shown in Fig 4.53 and Fig 4.54. From the relations shown in fig 4.53 and fig 4.54 the maximum dry density and optimum moisture content have been determined which are presented in Table 4.31. From all the regional soils data shown in the table 4.31 it is observed that the optimum moisture content increases while the maximum dry density decreases with the increase in lime content. Similar results are reported by Kezdi (1979), TRB (1987), Hausmann (1990), Bell (1993). The optimum moisture content and maximum dry density of lime stabilized twelve regional soils are compared. In all investigation it can be summarized that the optimum moisture content increases with the increase in the lime content and ranges from 11.5% to 25.6% and maximum dry density decreases with increase in lime content and ranges from 18.5 kN/m^3 to 13.8 kN/m^3.

Table 4. 30 Index properties of lime treated regional soils

Soil Code	Lime Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)
AH-B	0	52.0	23.0	14.0
	1	50.5	23.5	15.0
	3	49.0	24.0	15.5
	5	48.0	25.0	16.0
	7	46.5	25.5	17.0
H-A	0	25.0	13.0	-
	2	31.0	30.5	-
	4	33.0	33.0	-
H-B	0	42.0	22.0	-
	2	39.5	37.0	-
	4	41.5	40.0	-
	6	45.0	45.0	-
H-1	0	56.0	13.0	11.0
	3	54.0	24.0	22.0
	6	53.0	28.0	25.0
	9	54.0	32.0	29.0
	12	57.0	37.0	33.0
	15	59.0	40.0	37.0
R-B	0	44.0	25.0	23.0
	3	43.0	30.0	26.5
	5	42.5	32.0	27.0
	7	41.0	36.0	32.5

Table 4. 31 Moisture – density relations for lime stabilized regional soils of Bangladesh

Soil Code	Content (%)	Optimum Moisture Content (%)	Maximum Dry Density (kN/m^3)	Soil Code	Content (%)	Optimum Moisture Content (%)	Maximum Dry Density (kN/m^3)
SH-A	0	21.7	16.4	R-B	0	15.5	17.5
	3	24.6	15.7		3	16.0	17.3
	5	26.3	15.5		5	16.6	17.1
	7	27.5	15.3		7	16.8	16.9
SH-B	0	22.1	16.1	N-I	0	17.4	15.6
	3	24.7	15.4		1	17.7	15.3
	5	25.5	15.3		2	18.0	15.3
	7	26.4	15.1		5	18.1	15.2
SH-C	0	18.4	17.3	N-II	0	19.8	15.4
	3	22.2	16.7		1	20.3	14.3
	5	22.8	16.5		2	20.3	13.9
	7	24.1	16.4		5	20.7	13.8
H-1	0	18.1	17.08	N-III	0	22.6	15.7
	3	19.9	15.3		1	20.1	14.9
	6	23.8	15.1		2	20.5	14.6
	9	25.6	14.8		5	21.8	14.3
M-1	0	12.5	16.1	AH-B	0	11.5	18.5
	3	13.2	15.5		1	11.9	18.4
	5	14.3	15.3		3	13.0	18.3
M-2	0	21.0	15.9		5	13.6	17.8
	3	22.7	15.5		7	13.9	17.6
	5	23.6	15.2				
M-3	0	18.8	15.8				
	3	19.4	15.5				
	5	19.8	15.4				

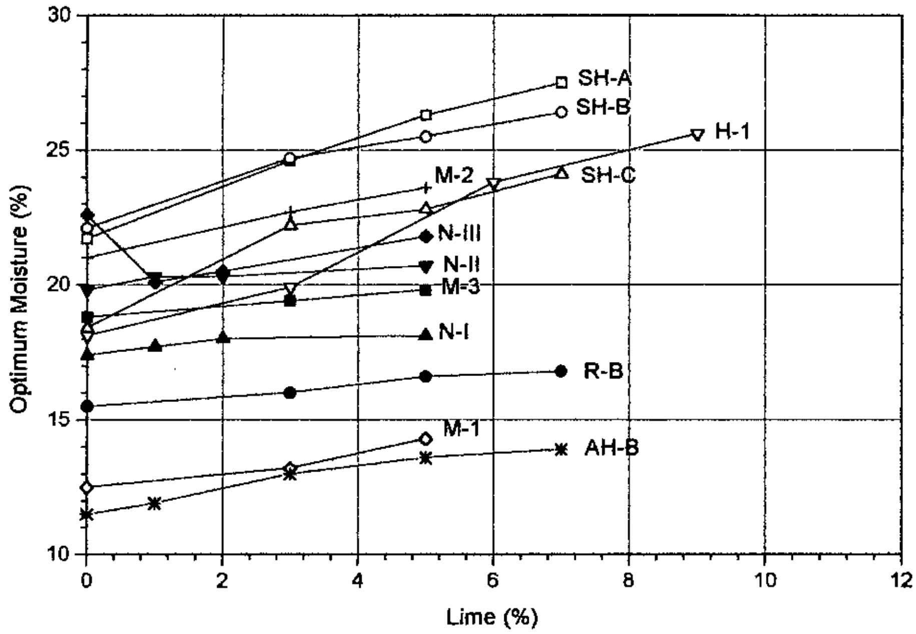

Fig: 4.53 Effect of lime stabilization on optimum moisture content between different regional soils of Bangladesh and soils used in the present study.

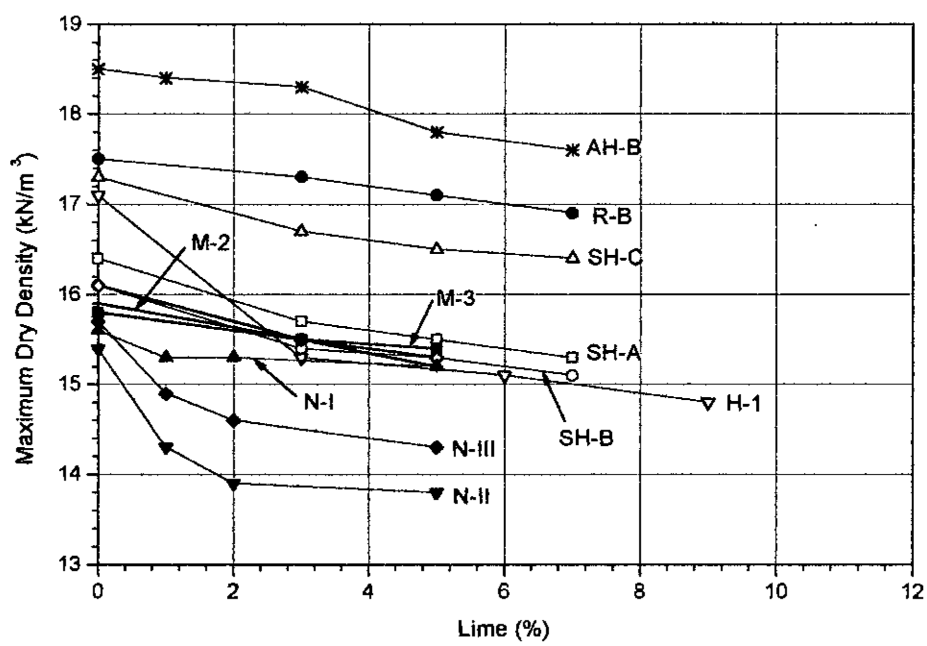

Fig: 4.54 Effect of lime stabilization on dry density between different regional soils of Bangladesh and soils used in the present study.

4.5.8 UNCONFINED COMPRESSIVE STRENGTH OF LIME STABILIZED SOILS

Table 4.32 shows summary of unconfined compressive strength of regional soils of Bangladesh. The unconfined compressive strength of untreated samples and samples treated different lime contents and cured for 28 days are presented. It can be seen that for all the regional soils, compared with the untreated soils, the values of unconfined compressive strength increases significantly with the increase in lime content and curing age. The unconfined compressive strength of the lime stabilized soils increases 2 to 5 times higher compared with those of the untreated soils. Fig. 4.55 shows the relation of unconfined compressive strength and lime content. These results are in agreement with those reported by number of researcher (Igles and Metcalf, 1972; Bell, 1993). The unconfined compressive strength of untreated samples and samples treated with different lime content of eleven regional soils are shown in table 4.32. From this table it is shown that for 28 days curing age the unconfined compressive strength increases with the increase in lime content and ranges from 39.3 kN/m^2 to 3452 kN/m^2. The compressive strength of lime stabilized coastal soils was found to be higher than other regional soils. The trend of increase in compressive strength of coastal and reclaimed soils was found to be similar.

Table 4. 32 Unconfined compressive strength (28 days) of lime stabilized regional soils of Bangladesh

Soil Code	(%) Lime	Unconfined Compressive Strength (kN/m^2)	Soil Code	(%) Lime	Unconfined Compressive Strength (kN/m^2)
SH-A	0	243.2	M-3	0	39.3
	3	352.5		3	106.8
	5	662.7		5	118.6
	7	725.8	H-A	0	-
SH-B	0	228.6		2	171.4
	3	232.7		4	220.0
	5	276.9		6	234.9
	7	368.2	H-B	0	-
SH-C	0	278.1		2	210.9
	3	1243.9		4	339.5
	5	1383.5		6	408.4
	7	1498.7	R-B	0	692
H-1	0	550		3	1302
	3	1100		5	2308
	6	1820		7	3452
	9	1930	AH-B	0	380
M-1	0	75.9		1	984
	3	345.5		3	2015
	5	379.3		5	2385
M-2	0	115.2		7	2678
	3	387.5		-	-
	5	652.3		-	-

4.5.9 CALIFORNIA BEARING RATIO (CBR) OF LIME STABILIZED SOILS

A study of CBR test for three levels of compaction energy and 4 days soaking time of regional soils are represented in table 4.33. In these tests up to 9% lime is used. It shows that, compared with the untreated soils, CBR values of the treated soils at all levels increase considerably with increase in lime content. The variations of CBR with lime content for regional soils are shown in Fig. 4.56.

It can be seen from Table 4.33 that the CBR values of the regional treated soils increase up to about 8 times than those of the respective untreated samples. TRB (1987) reported the effect of lime treatment on CBR values for three plastic clays (LL = 35 to 59, PI= 15 to 30) and showed that for all the soils CBR increase markedly with the increase in lime content. Eight samples are taken for comparison of CBR values for different percent of lime content and different level of compaction energy. From the table 4.31 it can be shown that the CBR values increases with the increase in lime content and ranges from 4 to 69. The CBR values of coastal soils were found to be higher than other regional soils. The trend of increase in CBR values of coastal and reclaimed soils was found same.

Table 4. 33 CBR values of lime stabilized regional soils of Bangladesh

Soil code	Lime	CBR Value		
	Lime Content (%)	Low	Medium	High
H-A	0	4	5	8
	3	26	29	38
	6	30	32	41
	9	35	37	46
M-1	0	2	3	6
	3	3	12	16
	5	4	14	20
M-2	0	1	2	4
	3	4	18	28
	5	5	21	33
M-3	0	3	4	8
	3	4	7	11
	5	4	9	14
H-A	0	-	-	-
	2	1.5	8	20
	4	3	10	26
	6	3	10	25
H-B	0			
	2	2	13	26
	4	2	14	29
	6	3	8	23
R-B	0	10	13	21
	3	25	29	49
	5	37	44	59
	7	42	55	64
AH-B	0	12	15	17
	1	16	23	36
	3	32	43	49
	5	48	56	59
	7	53	64	69

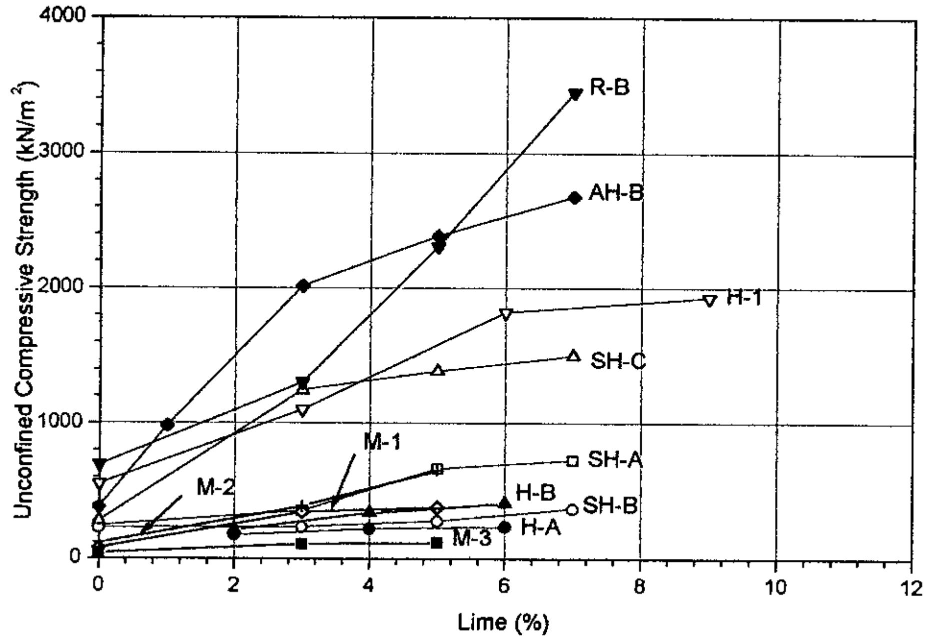

Fig: 4.55 Effect of lime stabilization on unconfined compressive strength between different regional soils of Bangladesh and soils used in the present study.

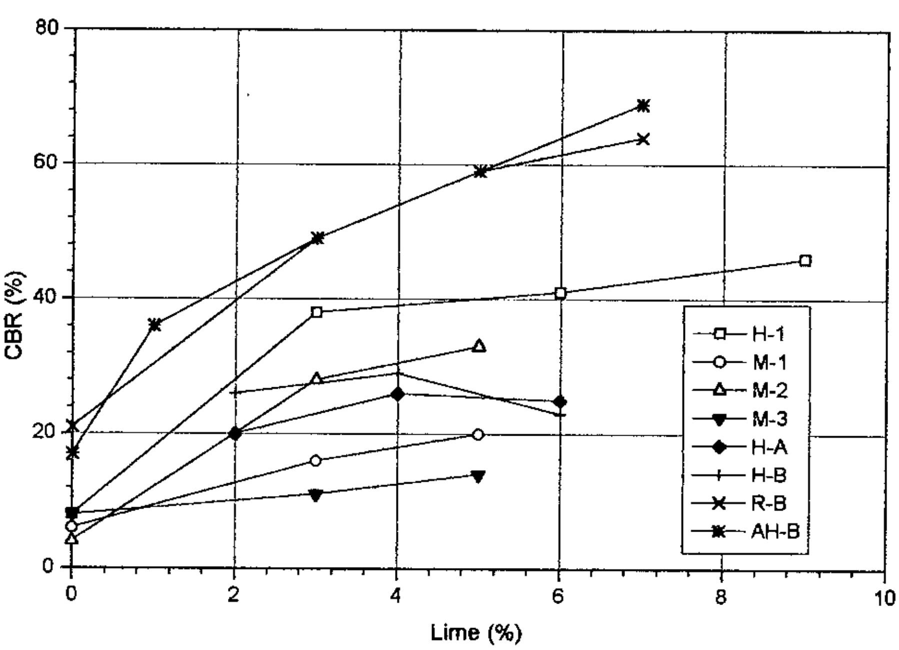

Fig: 4.56 Effect of lime stabilization on CBR between different regional soils of Bangladesh and soils used in the present study.

4.5.10 FLEXURAL STRENGTH AND MODULUS OF LIME STABILIZED SOILS

The flexural strength and modulus of untreated and treated samples of regional soils for 28 days curing and treated with various lime content are shown in table 4.34. It is found that with the increase in lime content both the flexural strength and modulus increase significantly. The flexural strength of lime treated soils increases up to 3 times and the flexural modulus of lime treated soil increases up to 2.5 times higher than those of the untreated soils. The effects of lime content on flexural strength for regional soils are shown in Fig. 4.57 while Fig. 5.58 shows the effect of lime content on flexural modulus for regional soils. The ranges of flexural strength vary between 47.3 kPa to 2430 kPa and flexural modulus varies between 23.3 MPa to 71.2 MPa. The flexural strength of expansive soils was found to be higher than other coastal and reclaimed soils. The trend of increase in flexural strength of coastal, expansive and reclaimed soils was found to be the same.

Table 4. 34 Flexural properties (28 days) of lime stabilized regional soils of Bangladesh

Soil code	Lime Content (%)	Flexural Strength (kPa)	Flexural Modulus (MPa)
H-A	0	97.2	46.0
	3	145.0	61.0
	6	211.0	63.0
	9	243.0	69.0
R-B	0	49.1	32.7
	3	63.5	51.8
	5	87.2	58.4
	7	97.1	71.2
AH-B	0	47.3	23.3
	1	66.3	29.1
	3	81.6	52.5
	5	88.7	57.7
	7	116.8	62.3

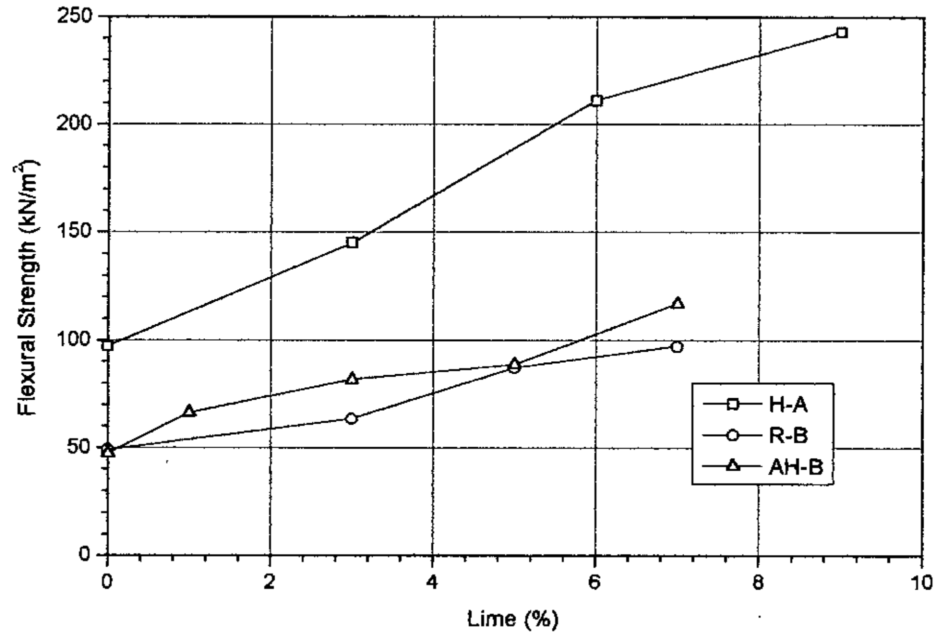

Fig: 4.57 Effect of lime stabilization on flexural strength between different regional soils of Bangladesh and soils used in the present study.

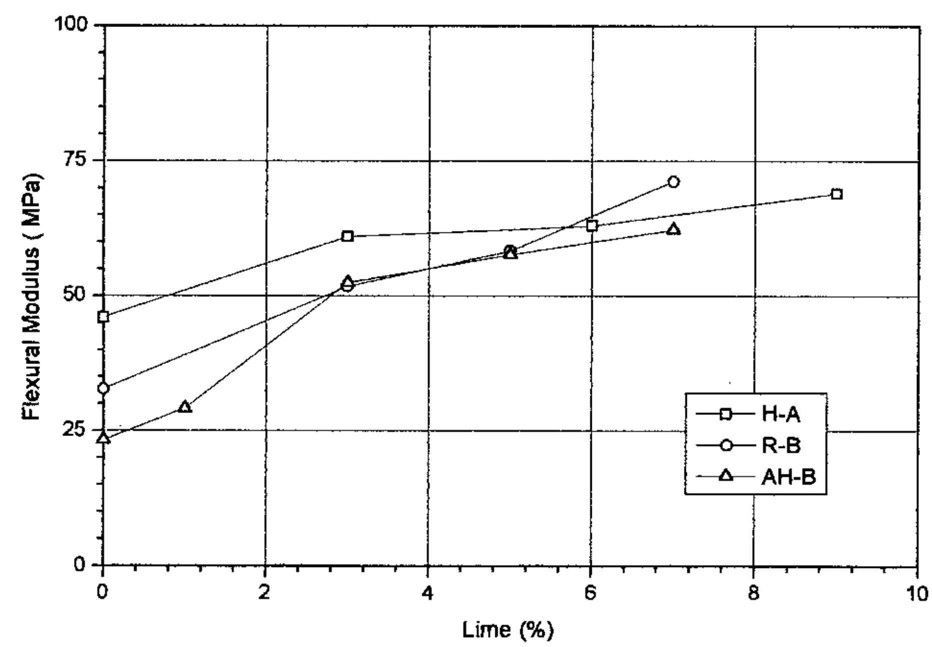

Fig: 4.58 Effect of lime stabilization on flexural modulus between different regional soils of Bangladesh and soils used in the present study.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDY

5.1 CONCLUSIONS

Strength development of reclaimed stabilized soil was investigated in this research work. Soils from two reclaimed selected sites namely Bashundhara and Aminbazar were collected. The different cement contents used for preparing sample were in percentages of 1,3 and 5 with Aminbazar soil and in percentages of 1, 3, 5 and 7 with Bashundhara soil, while for lime treated sample, lime contents were in percentages of 1,3,5 and 7 with Bashundhara soil. The major findings and conclusions have been described in three sections relating to the following areas:

(1) The influence of cement stabilization on the physical and engineering properties on samples of the two reclaimed soils from Aminbazar (i.e., Soil-A) and Bashundhara (i.e., Soil-B) from two selected locations of Dhaka city.

(2) The effect of lime stabilization on the physical and engineering properties on samples of the Soil-B.

(3) A study of properties of cement and lime stabilization between different regional soils in Bangladesh and the soils used in the present study.

5.1.1 INVESTIGATIONS ON THE EFFECT OF CEMENT STABILISATION

The major findings and conclusions drawn from the present research work of cement stabilized reclaimed soil are as follows:

(i) Compared with the untreated sample, the value of liquid limit of the treated sample increased in Soil-A while it is reduced in case of Soil-B. On the other hand for Soil-A and Soil-B, plastic limit of the stabilized samples increased while plasticity index, shrinkage limit and linear shrinkage reduced.

(ii) With the increase in cement content, the values of maximum dry density increased up to 7.8% and 5.4% for samples of Soil-A and Soil-B and the values of optimum moisture content reduced up to 11.1% and 10.1% respectively for samples of Soil-A and Soil-B.

(iii) The values of qu of samples of Soil-A and Soil-B treated with 5% cement and cured at 28 days were found to be about 6 times and 8 times higher than the strength of the untreated samples. The rate of strength gain with curing time for samples of Soil-A and Soil-B treated with 1% cement are relatively much slower than those of samples treated with 3%, 5% and 7% cement. In an attempt to investigate the effect of molding water content on qu, it has been found that irrespective curing ages, the values of qu are maximum and minimum respectively at molding moisture contents of optimum and wet side of optimum. It therefore appears that in order to achieve adequate compressive strength with cement stabilization, samples should be compacted at their optimum moisture contents.

(iv) It was found that the CBR-values of Soil-A and Soil-B stabilized with 5% cement increased up to about 5 times and 5.3 times than those of the respective untreated samples. It was found that the CBR of samples of Soil-A and Soil-B treated with 3% cement and compacted with medium and high energy and that CBR samples of Soil-A and Soil-B treated with 5% cement and compacted with low to high energy satisfied the requirements of soil-cement road sub-base and base for light traffic as proposed by Ingles and Metcalf (1972).

(v) For both Soil-A and Soil-B, compared with the untreated sample, flexural strength and flexural modulus of the treated samples increased significantly, depending on the cement content. For comparison, the flexural strength and flexural modulus of Soil-A treated with 5% cement and cured at 28 days are respectively about 4.5 times and 3.3 times higher than those for the untreated sample. The flexural strength and modulus of Soil-B treated 7% cement and cured at 28 days are respectively about 5.5 times and 5.3 times higher than those of the untreated samples. The curing age, however, has got insignificant effect on increase in flexural strength and modulus. It was also found that the values of flexural strength and modulus of samples of more plastic Soil-B (P=29) is higher than the less plastic Soil-A (PI=12).

(vi) The loss in soil-cement for samples of Soil-A and Soil-B treated with 5% cement are 18.6% and 16.8% respectively which did not meet the PCA values of 10% loss and have to need more cement contents.

From the aforementioned findings it is evident that for both samples of the two reclaimed soils studied, cement stabilization provided a substantial improvement in the engineering properties as compared with the samples of the untreated soils.

5.1.2 INVESTIGATIONS ON THE EFFECT OF LIME STABILISATION

This section presents the findings and conclusions from the experimental investigation of lime stabilization on samples of Soil-B collected from Bashundhara. The main findings and conclusions are as follows:

(i) The plastic limit and shrinkage limit increased with increasing lime content while liquid limit and plasticity index reduced with the increase in lime content. Compared with the untreated sample, linear shrinkage of the stabilized samples reduced with increasing lime content.

(ii) Compared with the untreated sample, the values of maximum dry density reduced up to 4.2% for an increase in lime content up to 7% while the values of optimum moisture content increased up to 20.8 % for an increase in lime content up to 7%.

(iii) The value of q u of sample treated with 7% lime and cured for 28 days was found to be about 7 times higher than the strength of the untreated sample. It has been found that the values of q u of samples treated with 5% and 7% lime contents fulfilled the requirements for upgrading heavy clays to sub-base material quality type, as proposed by Ingles and Metcalf (1972). Unlike cement stabilization, it was found that irrespective of curing ages, values of q_u is maximum and minimum at molding 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 stabilized samples should be compacted at the wet side of their optimum moisture contents.

(iv) Compared with the untreated sample, CBR values of the treated samples increased considerably at all levels of compaction effort, depending on the lime content. The values of CBR of Soil-B stabilized with 7% lime increased up to about 4.1 times that of the respective untreated samples. A CBR value of 70 obtained for the sample of the soil stabilized with 7% lime, however, did not fulfill 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).

(v) A slightly higher lime content it was found that curing age has got significant effect on the increase in flexural strength and modulus of sample treated with 7% lime and cured at 28 days are respectively about 2.4 times and 2.6 times higher than those of the untreated samples. The maximum deflection and failure strain of untreated and stabilized soil-lime beams were found to be small and were the maximum of 1.06 mm and 0.48% respectively.

5.1.3 A STUDY OF RESULTS OF DIFFERENT STABILIZED SOILS

Different researchers have so far carried out soil stabilization of eighteen samples of different region of Bangladesh. The main findings are:

(i) The liquid limit of the investigated soils varying between 25% to 56% and plastic limit varying from 13% to 30%.

(ii) The percent clay varying from 0 to 46; percent silt varying from 38 to 89.5; the percent sand varying from 1 to 62.

(iii) The maximum dry density varying between 14.7 kN/m^3 to 18.5 kN/m^3 and optimum moisture content varying from 11.5% to 22.1%.

(iv) Cement stabilization of six regional soils is compared. The optimum moisture contents are found to increase with the increase in cement content and ranges from 10% to 22.6%. The maximum dry density increases slightly with increase in cement (%) ranges between 14.5 kN/m^3 and 19.6 kN/m^3.

(i) In general Unconfined Compressive Strength and CBR value increase with increase of cement (%). The Unconfined Compressive Strength ranges from 51.8 kN/m^3 to 4304 kN/m^3.

(ii) Also flexural strength and modulus increase with increase of cement (%). The range of flexural strength is between 26.9 kPa to 286 kPa and flexural modulus varies between 17.3 Mpa and 136 Mpa.

(iii) Durability of cement stabilization has been calculated by measuring soil-cement loss, which ranges from 10.6% to 42.7%. Soil-cement loss decreases with the increase of cement (%).

(iv) Lime stabilization of twelve regional soils was compared. Five regional soils were investigated for finding index properties. From these results, it is found that shrinkage limit increases with the increases in lime content (%). The optimum moisture content increases with the increases in lime content and varies from 11.5% to 25.6% and maximum dry density decreases with increases in lime content and ranges from 18.5 kN/m^3 to 13.8 kN/m^3.

(v) In general Unconfined Compressive Strength and CBR values increases with the increases in lime content (%). The Unconfined Compressive Strength ranges between 39.3 kPa to 3452 kPa and CBR from 4 to 70.

(vi) Three regional soils were investigated to find the flexural strength and modulus. For all the cases flexural strength and modulus increases with increase in lime content (%). The flexural strength ranges between 47.3 kPa to 243 kPa and flexural modulus varies from 23.3 MPa to 71.2 MPa.

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 samples collected from the two selected reclaimed sites of Dhaka city. Similar investigations may be carried out with soils collected from other reclaimed sites of any region of Bangladesh and the results may be compared with those obtained in the present investigation.

(ii) In this research work, samples of the two reclaimed soils were stabilized with a maximum of 7% cement and 7% lime contents. The scope of the present work could be extended by determining the physical and engineering properties of these soil samples stabilized with higher percentages of cement and lime in order to evaluate the optimum additive content.

(iii) In this research work, the reclaimed stabilized samples were cured for a maximum age of 28 days while investigating their engineering properties. The influence of long term curing age on engineering properties of the stabilized samples of the two soils studied could be investigated.

(iv) In this investigation, cement and lime have been used as additives for stabilization. Investigations on the physical and engineering properties could be carried out by stabilizing the soils studied with other additives, e.g., fly ash, lime plus fly ash and cement plus fly ash in order to assess the most suitable type of additive for stabilizing these reclaimed soils.

(v) In this research work, analysis was carried out for investigation of physical and engineering properties of reclaimed soils. Further analysis can be carried out for stabilization of rural roads which are subjected to light traffic and highway roads subjected to relatively heavy traffic loadings, e.g., mini bus and trucks, runway etc.

(vi) The behavior and engineering properties of two reclaimed soils were carried out by moisture density relations, California Bearing Ratio, unconfined compressive strength, flexural strength and modulus in this research work. Further study can be carried out considering the behavior and engineering properties of the reclaimed soils by pore pressure development and consolidation characteristics.

REFERENCES

Ahmed, N.U. (1984), "Geotechnical Properties of Selected Local Soils Stabilized With Lime and Cement", M.Sc. Engineering Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka.

Arman, A.H. and Muhfakh, G.A. (1972) "Lime Stabilization Organic Soils", HRB Bul. no 381, Highway Research Board, Washington, D.C.

ASTM (1989), "Annual Book of ASTM Standards", Volume 04.08, Soil and Rock, Building Stones, Geotextiles.

Bell, F.G. (1988), "Lime Stabilisation of Clay Soils: Part-1, Basic Properties", Ground Engineering, Vol. 21 No.1 pp. 10-15

Bell, F.G. (1993), "Engineering Treatment of Soils", E & FN Spon, An Imprint of Chapman & Hall, First Edition, London, U.K.

Boominathan, A. and Prasad, S. R., (1992), "Laboratory Investigation of Effectiveness of Lime Treatment in Polluted Soils", Proceedings of the Indian Geotechnical Conference, pp. 497-500.

Brandal, H. (1981), "Alteration of Soil Parameters by Stabilization with Lime", Proceedings of the Tenth ICSMFE vol.3 PP. 587- 594.

Broms, B. and Boman, P. (1977), "Stabilization of Soil with Lime Columns", Design Hand Book, Royal Institute of Technology.

Broms, B.B. and Boman, P. (1979), "Lime Columns. A New Foundation Method", Proc., American Society Civil Engineers, Journal Geotechnical Engineering Division, Vol.105, No. GT4, pp. 539-56.

Broms, B. B. (1986), "Stabilization of Soft Clay with Lime and Cement Columns in Southeast Asia", Nanyang Technical Institute, Applied Research Project RP 10/83.

BRTC (1997), "A Report on the Investigation of the Distress in Small Structures at Sreepur, Gazipur and Katur Road, Rajshahi Food Godown Sites Caused by Expansive Soils", Department of Civil Engineering, Bureau of Research Testing and Consultation, Bangladesh University of Engineering and Technology, Dhaka.

BS 1377 (1975), "Method of Tests For Soils Civil Engineering Purposes", British Standards Institution, Gaylard and Son Ltd., London.

Chen, F.H. (1975), "Foundation on Expansive Soils", Developments in Geotechnical Engineering Vol. 12, First Edition, Elseveir Science Publishers B.V., The Netherlands.

Compendium-8 (1987), "Lime Stabilization, Reactions, Properties Design and Construction", State of Art Report-5, U.S. Transportation Research Board, Washington D.C.

Croft, J.B. (1964), "The Processes Involved in the Lime Stabilisation of Soils", Proc., Australian Road Research Board, Vol. 2, Part 2, pp. 1169-1204.

Diamond, S. and Kinter, E. F. (1965), "Mechanism of Soil Lime Stabilization, an Interpretive Review", Highway Research Board, Bulletin No. 92, pp. 83-96.

Dunlop, R. J. (1977), Lime Stabilization for New Zealand Roads, RRU technical recommendation no.2 National Roads Board Wellington, New Zealand.

Eades, J. L. and Grim, R. H. (1960), "Reaction of Hydrated Lime with Pure Clay Minerals in Soil Stabilization", Highway Research Board, Washington D.C.buU.no.262, pp51-63.

Eades, J. 1., Nichols, F. P. and Grim, R.E. (1962), "Foundation of New Materials with Lime Stabilization as Proven by Field Experiments in Virginia", Bulletin No. 335, Highway Research Board, Washington D.C. pp. 31-39.

Faisal, H. A., Aminuddin, A. and Choy. K. C. (1992), "Use of Rice Husk to Enhance Lime Treatment of Soil", Canadian Geotechnical Journal, Vol. 29, pp. 843-852

Grim, R.E. (1968), "Clay Mineralogy", McGraw-Hill Book Co.

Heidema (1957), "Bar-Linear Shrinkage Test and Its Importance", Proc., Fourth International Conference on Soil Mechanics and Foundation Engineering, London, Vol. 1, pp. 14.

Hammer, M.J. and Thompson, O.B.(1966), "Foundation Clay Shrinkage Caused by Large Trees", Journal ASCE, Soil Mechanics and Foundation Division, Vol. 92, No. SM 6.

Hausmann, M.R. (1990), "Engineering Principles of Ground Modification", McGraw-Hill Publishing Company, Singapore.

Herzog, A. and Mitchell, J. K. (1963), "Reactions Accompanying the Stabilization of Clay with Cement", Highway Research Record, No. 36, Washington D.C. pp. 146-171.

Herrin, M. and Mitchell, H. (1961), "Soil Lime Mixture", Highway Research Board, Washington D.C, Bulletin No. 304, pp 99-138.

Hilt, G. H. and Davidson, D. T. (1960), "Lime Fixation in Clayey Soils", Highway Research Board, Washington D.C, Bulletin No. 262.

Ho, C. and Handy, R.L. (1963), "Characteristics of Lime Reaction by Montmorillonite Clays, "Highway Research Record No. 29, Highy Research Board, Washington.

Holm, G., Trank, R., Ekstrom, A. and Toratensson, M. (1983), "Lime Column Under Embankment- a Full Scale Test", Proc., 8th ICSMFE, Helsinki, pp. 909-912.

Hossain, M. M. (1983), "Swelling Properties of Selected Local Soils" M. Sc. Engineering thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.

Hossain, MD. A. (2001), "Geotechnical Behaviour of a Lime Treated Expansive Soil." M. Sc. Engineering thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.

Hossain, K.M.A. (1991), "Stabilization of Subgrade Soil" M. Engineering thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.

Holtz, W.G. (1959), "Expansive Clay – Properties and Problems", Colorado School of Mines Quaterly, Vol. 54, No.4.

Holtz, W.G. (1969), "Volume Change in Expansive Clay Soils and Control by Lime Treatment", Proceedings of the Second International Research and Engineering Conference on Expansive Soils, Texax A& M Press.

Holtz, W.G. and Gibbs, H.J. (1956), "Engineering Properties of Expansive Clays", ASCE Transactions, Vol. 121, Paper No. 2814, pp. 641-663, Discussion, pp. 664-677.

Holtz, W.G. and Gibbs, H.J. (1954), "Engineering Properties of Expansive Clays", Proceedings, ASCE, Vol. 80.

Ingles, O.G. and Metcalf, J.B. (1972), "Soil Stabilization-Principles and Practice", Butterworths, Pty. Limited, Australia.

IRC (Indian Road Congress) (1973a), "Recommended Design Criteria for the Use of Soil-Lime Mixes in Road Construction", IRC Highway Research Board:51-1973, New Delhi.

IRC (Indian Road Congress) (1973b), "Handbook of Quality Control for Construction of Roads and Runways", The IRC Special Publication: 11-1973, New Delhi.

IRC, (Indian Road Congress)(1976), "State-of-the-Art: Lime-Soil Stabilization", Special Report, IRC Highway Research Board, New Delhi.

IS: 2720 (1972) `Methods of Test for Soils', Part VI, Indian Standards Institution, New Delhi.

IS: 1948 (1972), "Classification and Identification of Soils for General Engineering Purposes", Indian Standards Institution, New Delhi.

IS: 2720, Part XLI, (1977), "Measurement of Swelling Pressure of Soils", Indian Standards Institution, New Delhi.

IS: 2911, Part III (1980), "Code of Practice for Design and Construction of Pile Foundations", Part III, Under reamed Piles (First Revision), Indian Standards Institution, New Delhi.

Jones, C. W. (1958), "Stabilization of Expansive Clay with Hydrated Lime and with Portland Cement," Bulletin, Highway research Board, No. 193.

Kassiff, G. and Baker, R. (1971), "Aging Effects on Swell Potential of Compacted Clay", Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 97, SM 3. Proc. March.

Kezdi, A. (1979), "Stabilized Earth Roads", Elsvier Scientific Publishing Company,New York.

Kulkarni, G.J. (1977), "Engineering Materials", Start Book Service, Ponna, India.

Locat, J., Berube, M. A. and Marc, C. (1990), "Laboratory Investigations on the Lime Stabilized of Sensitive Clay : Shear Strength Development", Canadian Geotechnical Journal, Vol. No. 3, pp. 294-304.

Macham, L.R., Diamond, S., and Leo, E. (1977), "Laboratory Study of the Effectiveness of Cement and Lime Stabilization for Erosion Control", Transprotation Research Board, No. 641, pp. 24-28.

Markus, T.A., Gibson, H., Mckinlay, D.G. and North, E.R. (1979), "Stabilised Soil: An Under-Utilised Resource for Low-Cost Building in Developing Countries", University of Strathclyde, Glasgow.

Mateos, M. (1964), "Soil-Lime Research at Lowa State University", Journal of the Soil Mechanics and Foundation Engineering Division, ASCE vol. 90, pp 127-153.

Mc Dowell, C., Ghazzali D. 1. and Michael 0. Nill (1977), "Swell Potential Related to Building Performance", Soil Mechanics and Foundation Division, ASCE Vol. 103, GT12, December.

Mielenz, R. C. and King, M. E. and King, M. E. (1955), "Physical-Chemical Properties and Engineering Performance of Clays", Clay and Clay Technolgy, Bulletin 169, State of California, USA.

Murthy, V.N.S (1992), "Soil Mechanics and Foundation Engineering", Third Edition, Sai Kripa Technical Consultants.

Mitchell, J. K. and Hooper D. R. (1961), "Influence of Time between Mixing and Compaction on Properties of Lime Stabilized Expensive Clay", Highway Research Board, Washington D.C., Bulletin No. 304, pp. 14-31.

Mitchell, J.K. (1981), "Soil Improvement-State-of-the-Art Report", Proceedings of the Tenth International Conference on Soil Mechanics and Foundation Engineering, Vol 4, Stockholm.

Molla, M.A.A. (1997), "Effect of Compaction Conditions and State Variables on Engineering Properties of the Lime Stabilized Soil", M.Sc. Engineering Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka.

Morgan, J.P. and McIntire, W.G. (1959), "Quartenary Geology of the Bengal Basin, East Pakistan and India", Bulletin of Geotechnique Society, America, Vol. 70. pp. 319-336.

NAASRA (1986), "Guide to Stabilization in Roadworks", Sydney, Australia.

NASSRA (1970), Guide to Stabilization to in Roadwork, National Association of Australia State Road Authorities

Rajbongshi, B. (1997), "A Study on Cement and Lime Stabilized Chittagong Coastal Soils for Use in Road Construction", M.Sc. Engineering Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka.

Ramie, A. B. (1987), "Strength and Compressibility Characteristics of Soft Muar Clay Stabilized with Lime and Lime Rice Husk Ash", M.Sc. Engineering Thesis no. GT. 87-16, Asian Institute of Technology, Bangkok.

Rodriguez, A. R., Caastillo, H. D. and Sowers, G. F.(1988), "Soil Mechanics in Highway Engineering", Part-2, pp. 751-755.

Sastry, M. V. B. and Rao, A. S. (1987), "Cinder Ash as Soil Stabilization", Journal of the Institution of Engineers (India) Vol. 67, Part CI6, pp 316-321.

Seed, H.B., Woodward, R.J. and Lundgren, R. (1962), "Prediction of Swelling Pressure for Compacted Clays", Journal of Soil Mechanics and foundations Division, ASCE, Vol. 88.

Serajuddin, M. and Azmal, M. (1991), "Fine-Grained Soils of Bangladesh for Road Construction", Proc., Ninth Asian Regional Conferece on Soil Mechanics and Foundation Engineering, Vol. 1, Bangkok, Thailand, pp. 175-178.

Serajuddin, M. (1992), "Chemical Stabilization of Bangladesh Fine-Grained Soils for Improvement of Road Sub-Grade", Proceedings of the Indian Geotechnical Conference, Vol. 1, pp. 223-226.

Shahjahan, A.B.M., (2001), "Long Term Strength Development of Lime Stabilized Soils" M.Sc. Engineering Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka.

Singh, A. (1975), "Soil Engineering in Theory and Practice – Fundamentals and General Pinciples", Asia Publishing House, India..

Singh, A. (1992), "Modem Geotechnical Engineering ", Third Edition, CBS Publishers and Distributors Pvt. Ltd.

Skempton, A.W. (1964), "Long Term Stability of Clay Slopes", Geotechnique, Vol. 14, p. 77.

Spangler, M. G. and Patel, 0. H. (1949), " Modification of a Gumbotil Soil by Lime and Portland Cement Admixtures," Proceedings Highway Research Board, Vol. 29.

Thompson, M. R. (1966a), "Shear Strength and Elastic Properties of Lime Soil Mixtures", Highway Research Board, Washington D.C. vol. no. 139, pp 61-72.

Thompson, M.R. (1966b), "The Split-Tensile Strength of Lime-Stabilised Soils", Highway Reserch Recorder, National Research Council, Washington, D.C. pp. 69-82.

Thompson, M.R. (1969), "Engineering Proeprties of Lime-Soil Mixtures, Journal of Materials, ASTM, Vol. 4, No. 4.

Townsend, D. L. and Klyn, T. W. (1970), "Durability of Lime Stabilized Soil, Highway Research Board, Washington D.C., Vol. 315, pp 81-89.

TRB (Transportation Research Board) (1987), "State-of-the-Art Reprot: Lime Stabilization Research, Proeprties, Design, and Construction", National Research Council, Washington, D.C.

TRL (Transport Research Laboratory) (1993), "A Guide to in Structural Design of Bitumen Surfaced Road in Tropical and Sub-Tropical Countries", Development Administration (ODA), London, U.K.

Uddin, M.K. (1995), "Strength and Deformation Characteristics of Cement-Treated Bangkok Clay", Ph.D. Thesis, Asian Institute of Technology, Bangkok, Thailand.

Van London, W. J. (1953), "Waterproofing Value of Asphalt Membranes in Earth Fills for Gulf Freeway," Proceedings, Association Asphalt Paving Technologists, Vol. 22.

Winterkorn, H. F. (1975), "Soil Stabilization", Foundation Engineering Handbook, Van Nastrand Reinhold Company, New York, pp. 312-336.

Yu Kuen (1975), "A Feasibility Study of the Use of Lime Piles for Soil Stabilization", M. Sc. Engineering Thesis no. 779, Asian Institute of Technology, Bangkok.

APPENDIX-A

WETTING AND DRYING TEST RESULTS OF CEMENT TREATED SAMPLES OF SOIL-A AND SOIL-B

Table A.1 Wetting and drying test of cement treated soil-A

Cement Content	Cycle No.	Moisture Content (%)	Volume Change (%)
1%	1	22.4	2.58
	2	21.8	
	3	21.00	
	4	Discontinued	
	5	-	
	6	-	
	7	-	
	8	-	
	9	-	
	10	-	
	11	-	
	12	-	

Table A.2 Wetting and drying test of cement treated soil-A

Cement Content	Cycle No.	Moisture Content (%)	Volume Change (%)
3%	1	20.3	2.43
	2	19.9	
	3	19.2	
	4	18.7	
	5	18.2	
	6	18.1	
	7	18.1	
	8	Discontinued	
	9	-	
	10	-	
	11	-	
	12	-	

Table A.3 Wetting and drying test of cement treated soil-A

Cement Content	Cycle No.	Moisture Content (%)	Volume Change (%)
5%	1	22.4	2.14
	2	22.1	
	3	21.8	
	4	21.8	
	5	21.5	
	6	20.3	
	7	20.0	
	8	20.8	
	9	20.4	
	10	19.9	
	11	19.6	
	12	19.4	

Table A.4 Wetting and drying test of cement treated soil-B

Cement Content	Cycle No.	Moisture Content (%)	Volume Change (%)
1%	1	25.8	1.76
	2	25.3	
	3	25.0	
	4	Discontinued	
	5	-	
	6	-	
	7	-	
	8	-	
	9	-	
	10	-	
	11	-	
	12	-	

Table A.5 Wetting and drying test of cement treated soil-B

Cement Content	Cycle No.	Moisture Content (%)	Volume Change (%)
3%	1	25.6	1.42
	2	25.2	
	3	25.0	
	4	25.0	
	5	24.7	
	6	24.8	
	7	25.6	
	8	23.8	
	9	23.2	
	10	Discontinued	
	11	-	
	12	-	

Table A.6 Wetting and drying test of cement treated soil-A

Cement Content	Cycle No.	Moisture Content (%)	Volume Change (%)
5%	1	26.4	1.06
	2	26.1	
	3	26.0	
	4	25.9	
	5	25.4	
	6	25.2	
	7	24.8	
	8	24.8	
	9	24.5	
	10	24.2	
	11	24.1	
	12	24.5	