EFFECT OF LIME AND FLAYSH STABILIZATION ON GEOTECHNICAL PROPERTIES OF CHITTAGONG COASTAL SOILS

A Thesis

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MD. MONWARUL ISLAM

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By

Md. Monwarul Islam

Approved as to style and content by:

Mehedi Ahmed Ansary

Dr. Mehedi Ahmed Ansary : Chairman:
Associate Professor
Department of Civil Engineering
BUET

Dr. SK Sekender Ali : Member (Ex Officio)
Professor and Head
Department of Civil Engineering
BUET

Dr. Abu Siddique : Member
Professor
Department of Civil Engineering
BUET

Md. Mesbauddin Ahmed : Member (External)
Managing Director
M. Ahmed and Associate
Shymoly, Dhaka.

February 2004

CANDIDATE'S DECLARATION

It is hereby declared that this thesis or any part of it has not been submitted elsewhere for the award of any degree or diploma.

Signature of the candidate

Md. Monwarul Islam

Dedicated to my Parents

MD. MONWARUL ISLAM

Previous Education

(1) Master in Defence Studies from National University in 2003.

(2) Graduated in War Course from National Defence College in 2003.

(3) Graduated in Civil Engineering from BIT Rajshahi in 1991.

(4) Graduated in Science in 1981, Chittagong University.

(5) Graduated in Military Science & Staff Course from Defence Staff College, Mirpur Dhaka in 1993.

(6) Diploma in Operation and Maintenance of Diesel Engine from Amphibious Naval School, USA, in 1992.

(7) Diploma in combat engineering from Military Engineering College, Peoples Republic of China, in 1994.

(8) Diploma in Field Engineering from School of Military Engineering in 1983.

Experience:

(1) Survey of Gomoty River for dredging in 1982.

(2) Construction of flood protection dam on Gamoty River Comilla in 1982-83

(3) Worked as an Engineer Plan Officer, United Nation Operation in Mozambique and supervise all kinds of Civil and Military Engineering works in 1994.

(4) Worked as an Engineer Officer in Macedonia under United Nation in 1994-95.

(5) Serving in Bangladesh Army Engineering Corps since 1980.

(6) Worked as Deputy Project Director of Cox's Bazar Teknaf Marine Drive Road in 1999.

(6) Worked as Project Director of Chimbuk-Thanchi, Diginala-Marisha and Bagaihat-Sajek-Masalong Road Project.

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ABSTRACT

The use of mixture of flyash and lime has been studied to investigate the strength properties of stabilized soil. Compaction apparatus was employed to determine the strength of the stabilized soils. Strength tests were carried out on the specimens up to 28 days curing period The interested admixture was lime-flyash; the amount of lime was fixed at 3% with the amount of flyash 0,6%, 12% and 18%.

The results from the experimental investigation shows that by increasing the amount of flyash the strength properties of lime-flyash stabilized soils improve. However, a detailed investigation needs to be carried out on the use of flyash alone. A prolonged curing period increased the strength of stabilized soils. The use of lime-flyash gave better strength and it may be more economical.

In the present study, flyash (0, 6, 12 and 18%) stabilization with 3% lime of two selected soil (collected from Anwara and Banshkhali) of Chittagong coastal region were carried out in order to assess their suitability for use in road construction. The soils from Anwara and Banshkhali were respectively a clayey silt of low plasticity (LL=30, Pl=7) and a silty clay of medium plasticity (LL=44, Pl=19).

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

For samples of both the coastal soils, compared with the untreated samples, unconfined compressive strength (q_u) of flyash and lime treated samples increased significantly, depending on the additive content and curing age. It was found that compressive strength of samples treated with 6% to 18% Flyash and cured for 7 to 28 days satisfied the requirements of PCA (1956)

given at Annex A for the compressive strength of soil-cement mix and that for all flyash contents and all curing ages, Compressive strength of the stabilized samples fulfilled the requirements of soil-flyash mix for use in road sub-base and base subjected to light traffic. It was also found that the compressive strength of samples treated with 3% lime met the requirement for upgrading clays to sub-base material quality type, as proposed by Ingles and Metcalf (1972). Compared with the untreated samples, CBR of the flyash and lime stabilized samples increased considerably. It was found that CBR-values of flyash and limes stabilized samples increased up to about 11 times and 4.4 time respectively. CBR-values of samples of both the soils, treated with 6% to 18% flyash, fulfilled the requirements of soil-cement road sub-base and base for light traffic while CBR of sample stabilized with 3% 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 flyash and lime stabilized samples. Compared with the untreated samples, flexural strength and flexural modulus of the flyash and lime stabilized samples increased considerably, depending on the additive content. Compared with the untreated sample, the flexural strength and flexural modulus of flyash-treated samples increased up to about 4.6 and 4.7 times and 3 and 4.25 times respectively or both the soil while for 3% lime-treated samples the respective increases were about 1.2 times and 2.13 times.

It was found from comparisons that values of q_u, CBR, flexural strength and flexural modulus of the flyash-treated soil samples are significantly higher than those of the lime-treated samples. Moreover, it is expected that compared with soil-lime mix, soil-flyash mix would be much more durable in the weather conditions of tropical regions. It could be concluded that flyash stabilization of the coastal soils studied would be more suitable than lime stabilization for their use in road construction

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NOTATIONS

CBR	=	California bearing ratio
d	=	Average depth of sample
D	=	Average width of soil-cement and soil-lime beam sample
E	=	Flexural Modulus
E_h	=	Horizontal modulus
E_{mix}	=	Modlus (stiffness) of asphalt mix
E_v	=	Vertical modulus
E_y	=	Vertical modulus
I	=	Moment of inertia
K	=	Fatigue constant
L	=	Span length of sample
LL	=	Liquid limit
LS	=	Linear Shrinkage
SL	=	Shrinkage limit
N	=	Allowable number of load repetitions to fatigue
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 Anwra
Soil-B	=	Soil from Banshkhali
ω_{opt}	=	Optimum moisture content
Δ	=	Maximum deflection
ε_f	=	Axial strain at failure
γmax	=	Maximum dry density

Notations of Different Soil

Location of Soil	Code Used	Source
Bilamalia, Savar	SH-A	Shahjahan (2001)
Beraid, Badda	SH-B	Shahjahan (2001)
Patia, Rupganj	SH-C	Shahjahan (2001)
Rajendrapur, Gazipur	H-1	Hossain (2001)
Jamuna Bridge Site, Sirajganj	M-1	Molla (1997)
BUET Campas, Dhaka	M-2	Molla (1997)
Dhaleshari Bridge Site, Munshiganj	M-3	Molla (1997)
Katasur, Mohammadpur	H-A	Hossain (1991)
Chandina, Savar	H-B	Hossain (1991)
Anwara, Chittagong	R-A	Rajbingshi (1997)
Banshkhali, Chittagong	R-B	Rajbingshi (1997)
Nayarhat, Dhaka	M-A	Mustaque (1986)
Kaliakoir, Gazipur	M-B	Mustaque (1986)
Bank of Meghna	N-I	Uddin (1984)
Manikgang	N-II	Uddin (1984)
Sher-a-Bangla Nagar, Dhaka	N-III	Uddin (1984)
Aminbazar, Dhaka	AH-A	Abid Hassan (2002)
Bashundhara, Dhaka	AH-B	Abid Hassan (2002)
Anwara, Chittagong	MI-A	Present Study
Banshkhali, Chittagong	MI-B	Present Study

CHAPTER 1
INTRODUCTION

1.1 GENERAL

In early days, engineers could avoid unsuitable sites or unsuitable construction material sources whenever the required conditions for the construction were not fulfilled. There were plenty of sites and construction material sources available for any construction purpose. So it was ease of construction and ease in obtaining material, which governed the choice of site rather than economic factors.

It is evident that earth structures, such as embankments, highways, airport runways, dams, or reclamation appurtenance require soils with sufficiently good engineering properties: like low plasticity, high bearing capacity, low settlement, etc.

Although natural soil is a complex and variable material, which exhibits differing behavior under different conditions, because of its universal availability and the low cost for obtaining such material, it offers great opportunities for skillful use with engineering technology.

As time passed, due to the growth of population, people became more cautious about the economy or, for different reasons, it has been difficult to find suitable sites for construction or suitable material sites for earth structures, such as highways, dams, or runways, within an economic range. Since unsuitable materials, which have low bearing capacity, coupled with low stability and high settlement or excessive swelling or squeezing properties, are frequently encountered, it has been necessary to improve unsuitable materials to make them acceptable for construction.

Improvement of soil by altering its properties is known as soil stabilization. An increment in strength, a reduction in compressibility, improvement of the swelling or squeezing characteristics, and increasing the durability of soil are the main aims of soil stabilization.

1.2 SOIL STABILIZATION

Stabilization is one of the most economical and desirable methods for improving the strength, durability and resistance to deformation of in-situ soil. Soil stabilization always involves certain mechanical treatment of the natural soil or remixing the natural soil with admixtures followed by compaction of the mixture. Soil stabilization must not be confused with solidification aimed at increasing the strength of in-situ soil masses in natural design, generally without any interference with their structure. 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 different requirements of mechanical strength and of resistance to environmental forces.

In connection with road construction, stabilization is simply referred as means by which the engineering properties of sub grade and pavement materials can be improved in order to withstand traffic loading and weather effects. Stabilization of in situ soils by admixtures has become and increasingly accepted method for improving the bearing capacity of either the substructure or some other components of the pavement. Stabilization is contemplated when it would be more economical to overcome a deficiency in a readily available material rather than to bring in another which complies with the specified requirements, or when enhanced properties are required for pavement design purposes. Some examples of the use of stabilization are as follows (NAASRA, 1986):

(i) Improvement of the properties of an available material to permit an enhanced usage;
(ii) Improvement of the properties of a sub grade or sub-base to reduce the thickness requirement for overlying materials;
(iii) Reduction of the sensitivity to moisture of sub grade or lower courses when permeable bases are used;
(iv) Increase layer stiffness to reduce wearing surface strain.

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 optimize benefits. Hence the concept of stabilization should extend beyond the remedial type treatment to be a general tool applied to pavement design and construction, Stabilization also widens the range of materials that can be applied to pavement construction (NAASRA, 1986).

In connection with road works, Wintercorn (1975) reported the following major uses of soil stabilization:

(i) Providing bases and surfaces for secondary and farm-to-market roads, where good primary roads are already in existence;

(ii) Providing bases for high-type pavements where high–type rock and crushed gravel normally employed for such bases are not economically available;

(iii) For city and suburban streets where the noise absorbing and elastic properties of certain stabilized soil system possess definite advantages over other construction materials; and

(iv) For military and other emergency where an area must be trafficable within a short period of time.

There are a number of methods of soil stabilization for use in road works. The additives, which are commonly used, are granular materials, Portland cement, flyash, lime (quicklime and hydrated lime), lime-Flyash, bitumen and tar. There are other chemical used in soil stabilization named as chemical stabilization. The chemical being used is calcium chloride, sodium chloride and sodium silicate. Stabilization by heating and electricity to fire grained soil is other method of soil stabilization. Full use of the potential of stabilization requires an awareness of the various methods available, their preferred application 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 summarized in Table 1.1. 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 (19986) reported the usual range of suitability of the various types of stabilization, which have been presented in Figure 1.1.

Soil stabilization can be achieved by the conventional method of compaction, which is known as mechanical stabilization, and which is universally used. There is very soft clay or marshy land are encountered, coupled with increasing demands for economy and the need for better results, which may not be fulfilled solely by compaction. The addition of a chemical stabilizer and using the conventional compaction method may improve the properties of soil in an economical way.

Cement and lime as chemical stabilizers used for improving the properties of soil, such as plasticity, bearing strength or, settlement characteristics, are already well known in stabilization of granular soil and clayey soils.

Fly ash, an industrial waste material has been of great interest to engineers for use as stabilizing agent. It could be employed to stabilize granular soil and clayey soil as well.

Investigation of the use of industrial waste, such as Flyash, to counter soil stabilization problems has been underway since about 1950 at Iowa State university in the U.S.A. Even 2000 years ago, people in the western world used hydrated lime combined with volcanic cinders as a cementing material. Flyash is a waste product from modern industry, which is similar to volcanic ash.

It has been realized by many authors that with the addition of flyash the strength of lime treated soil markedly increases.

Environment protection has been of a great concern in the modern world. The use of flyash for stabilization may help to solve the problems of environmental concern. Most earth structures can be obtained in an economical way.

Economy and safety are often incompatible, and the main task of an engineer is to achieve a compromise with the optimum result.

Table-1.1 Mechanics and applications of stabilization (after NAASRA, 1986)

Type of Stabilization	Process	Effects	Applicable Soil Type*
Granular	Mixing of two or more materials to achieve planned particle size distribution.	Changes to soil strength, permeability volume stability.	Poorly graded soils, granular soils with a deficiency in some sizes.
Cement	Cementitious interparticle bonds are developed.	*Low additive contents: Decreases susceptibility to moisture changes improves strength. *High additive contents: increases modulus and tensile strength significantly. Possibility of reduced thickness requirements.	Not limited apart from deleterious components (organic, sulfates etc.) which retard cement reactions. Suitable for granular soils but inefficient in predominantly one sized materials. Expensive in cohesive soils.
Lime (including Hydrated Lime and Quicklime)	Cementitious interparticle bonds are developed but the rate of development is slow, relative to cement.	Improves handling properties of cohesive material. *Low additive contents: decreases susceptibility to moisture changes, improves strength. *High additive contents: increases modulus and tensile strength.	Suitable for cohesive soils. Requires clay components in soil that will react with lime. Organic material will retard reactions.
Lime plus Fly Ash, pulverized Blast furnace Slag	Lime and pozzolan modifies particle size distribution and develops cementitious bounds	Generally similar to cement but rate of gain of strength similar to lime. Also improves workability.	As for cement stabilization, 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.

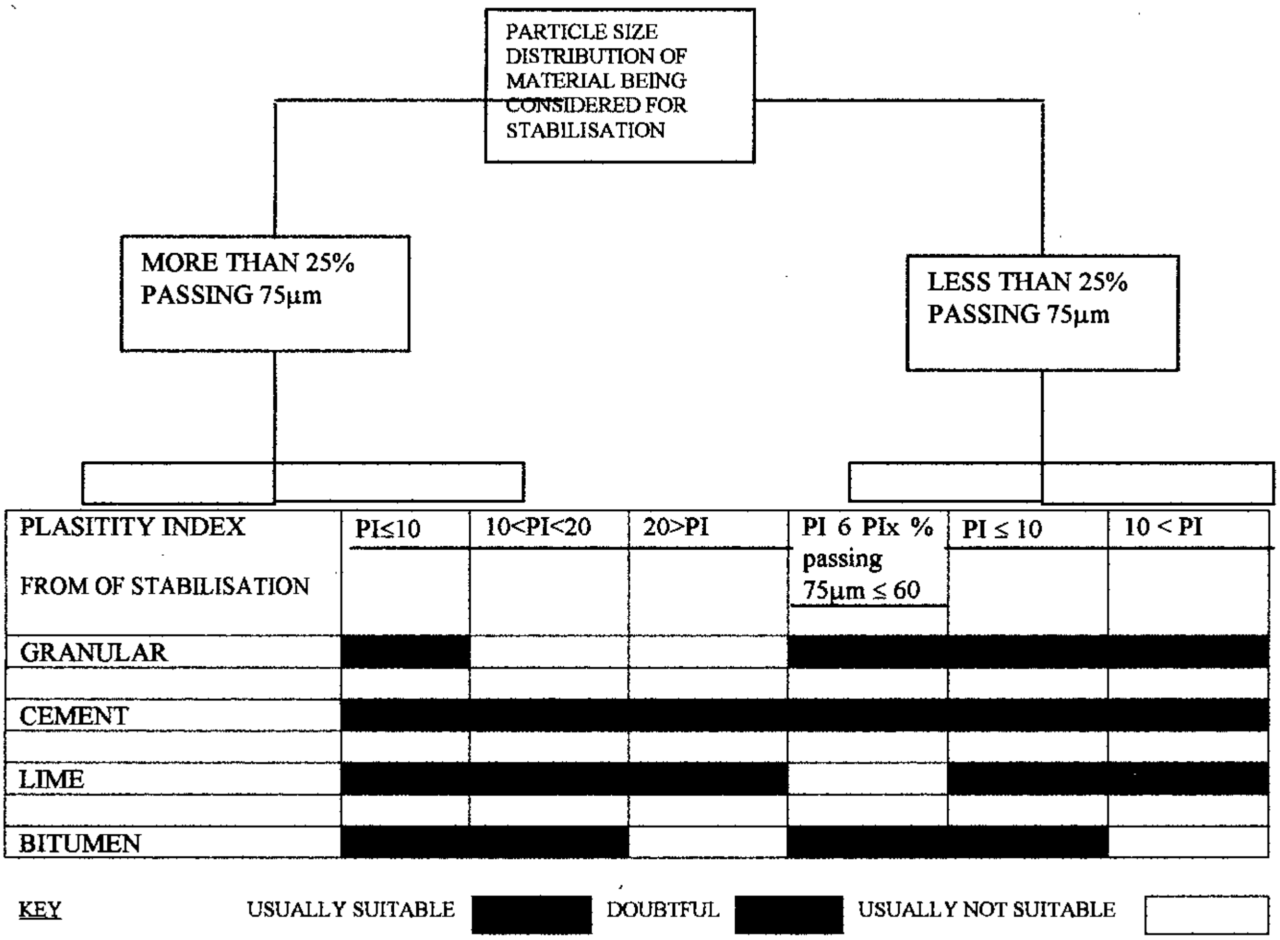

Fig. 1.1 Feasibility of stabilization techniques for different types of soils (after NAASRA, 1986).

1.3 GEOLOGICAL FORMATION OF SOILS OF CHITTAGONG COASTAL AREA

The Bengal basin has been filled in by sediment from the north, east and west. During this filling process, the basin has deepened and the sea level has varied considerably from its present position through a series of transgressions and regressions, which have occurred over a 60 million-year period. The alluvial plains and deltas formed in more recent periods by the Ganges, Barhmaputra and Meghna River cover the surface of the Bengal basin over a total area of 60,000 kilometer square. This huge delta is called the Bengal fan and consists of the world's largest-scale fan deposits.

Bangladesh is almost entirely and alluvial, deltaic plain with hills on the north-east, east and southeast margins. The alluvial plains extend about 400 km south eastward, falling gradually from an elevation of about 90 m in Tetulia in the far northeast to a coastal plain of less than 3 m in elevation south of a line joining Khulna-Narayanganj-Chandpur-Noakhali. These low-lying areas and the Chittagong coastal plain form the coastal areas of Bangladesh.

The coastal zone falls in a deepest part of Bengal Basin, Known as the Patuakhali through, which occupies the Hatiya, Barisal, and Faridpur areas and has sediments more than 18,000 m thick. The fan deposits from the Chittagong hills and deposits of coastal currents are mixed in complicated manner in the Chittagong coastal area. The geological formations and soil characteristics of this area are very complicated due to the multifold shallow bedrock of the above hills. The Chittagong Coastal plain comprises the generally narrow strip of land between the Chittagong hills and the sea, together with the Halda, the Karnafuly and the Sang floodplains and the offshore islands. This had known to be occupied by gently sloping piedmont alluvial fans with mainly loamy soils. Tidal clay plains occupy most of the offshore island. Most soil studies in the coastal area in the past have dealt with the soil types up to approximately 20-m below the ground surface. The surface layer mainly consists of silt and clay and has a thickness of some 50 m, except at the mouth of the Meghna river where the thickness is reduced to some 10 m. A more detailed examination reveals that the soil texture of the surface layer differs from one area to another in both the horizontal and vertical directions. The grain size, density and consistency also largely differ from one are to another. These differences reflect the sedimentation environment and are caused by frequent changes of the well-developed river and water channel courses. In general, the deposits of the major rivers are coarser than those of the sea currents.

1.4 SCOPE AND OBJECTIVE OF THE STUDY

The cardinal aim of this research work is to develop a general approach to soil improvement coastal area by using flyash for Road construction. The Study was limited to chemical stabilization by flyash and lime. A detailed study of the improvement of on the strength properties of coastal soil of Bangladesh have been made using flyash and lime as additives. Investigation was made using commercial lime and Flyash from brickfield, foundry shop and restaurant etc.

This research has been intended to evaluate the behavior and engineering properties (e.g. Moisture-density relations, California Bearing Ratio, Compressive strength, Flexural strength) of Flyash and lime stabilized soils from Chittagong coastal belt. The results of this investigation will enable to assess the suitability of using flyash and lime as stabilizer of these coastal soils particularly for their use in the construction of base/sub-base of roads in the coastal regions.

In this research work, flyash stabilization of two selected soils (collected from Anwara and Banshkhali) of Chittagong coastal region have been carried out. Flyash 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 behavior and engineering properties (e.g. Moisture-density relations, California Bearing Ratio, Compressive strength, Flexural strength of two selected soils stabilized with three different flyash contents (6%, 12% and 18%) having 3% lime as constant additives. The behavior and engineering properties of these two soils were also investigated without any treatment in order to examine the changes in behavior and engineering properties between the treated and untreated soils.

(ii) The behavior and engineering properties (e.g. Moisture-density relations, California Bearing Ratio, Compressive strength, Flexural strength and of a soil stabilized with 3% lime was investigated.

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

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

1.5 THE RESEARCH SCHEME

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

(i) Index property tests of the two coastal soils without any treatment were carried out to characterize the soils. Index tests include Atterberg limit tests, specific gravity and grain size analysis. Index property tests of the two sols stabilized with different flyash having 3% lime as constant and 3% lime contents were also performed.

(ii) The following tests were carried out on the two coastal soils without any treatment and stabilized with three different flyash contents (6%, 12% and 18%); having every time 3% lime and both the soil stabilized with 3% lime.

(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) tests.

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

Unconfined compressive strength tests and flexural strength test using simple beam with third point loading were carried out on flyash and lime stabilized samples cured at 7,14 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 stabilized soils. Laboratory CBR tests were carried out on the untreated samples and samples treated with flyash and lime using three levels of compaction energies.

1.6 THESIS ARRANGEMENT AND OUTLINE

The disseration is written in the following sequence:

A review on flyash, lime and other means of stabilization of soils is presented in Chapter 2. The review mainly includes the mechanisms of flyash and lime stabilization, factors governing the properties of flyash and lime-treated soils, the characteristics of flyash and lime stabilized soils and their applications.

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

Physical and engineering characteristics of the untreated soils and soils stabilized with fixed lime content (3%) and different flyash contents, as obtained from the laboratory investigation, are presented and discussed in Chapter 4. In addition, this chapter presents the comparison of properties of lime stabilization between different Regional soils of Bangladesh and the soil stabilized with fly ash in the present study.

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

LITERATURE REVIEW

2.1 GENERAL

The main reasons for increased study of soil improvement are to achieve more utilization of poor sites, support of existing structures, environmental protection, safety of high rise structures, an improved ability to handle problem soils, and engineering and construction in new and difficult environment. The objective of mixing additives with soil is to improve volume stability, strength and stress-strain properties, permeability, and durability. The development of high strength 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 factors affecting the quality of results. Most commonly used stabilizers for improving the physical and engineering properties of soil are Fly Ash, lime, cement and foundry sand, bitumen and chemicals like calcium chloride, sodium chloride and sodium silicate.

The use of admixtures for the stabilization of soils has been of great interest to highway engineers in recent years. Various organic and inorganic materials have been investigated for possible use as stabilizing agents. The aim has been to produce a material having better engineering properties then the original soil.The most extensively used stabilizing agents are cement and lime. Mixtures of lime and flyash are also among those that have shown promise. However, the latter have not been much used because their characteristics and behavior when added to soils are still to be investigated in detail. The flyash and other admixture and their uses have been discussed in subsequent paragraphs.

2.2 FLYASH

Hausmann (1990) stated that Fly ash is a solid waste product created by the combustion of coal and it is carried out of the boiler by flue gases and extracted by electrostatic precipitators or cyclone separators and filter bags. Its appearance is generally that of a light to dark gray powder of predominantly silt size.

Ash removed from the base of the furnace is termed bottom ash or boiler slag. It is coarser than fly ash, ranging in size from fine sand to gravel. As much as a quarter of the ash produced may be bottom ash.

Bottom ash serves well as structural fill and in road construction. Fly ash is regularly used as a partial replacement for cement in concrete because of its pozzolanic properties; it is also the form of ash, which has the greatest potential for use in ground modification.

In 1986 some 65 to 70 million metric tons of fly ash were produced in the United States alone, only 15 to 20% of this massive amount was used constructively: less than half of that was used in the manufacture of concrete. The rest is pumped in slurry form into lagoons or is conditioned by the addition of 10 to 15% water and disposed of as more or less engineered landfills.

Now a days coal is more and more frequently adopted as fuel for electric power plants. On the basis drawbacks of its use is the large quantity of produced ashes (up to 15% of the weight of coal). In the past, the coal ashes were disposed into abandoned open-pit mines or stream valleys; at present, it is becoming more and more necessary to use them for embankments and hydraulic or compacted fills.

Marking more productive use of fly ash would have considerable environmental benefits, reducing land, air, and water pollution: Increased use as a partial cement or lime replacement would also represent a savings in energy (fly ash has been called a high-energy waste material).

Besides using fly ash alone as a structural fill material scope exists for employing techniques of ground modification to find more medium-to high-volume applications in the following ways :

Add cement or lime to stabilize the fly ash.

Stabilize soil with cement-lime-fly-ash mixes.

Use fly ash in the containment of toxic wastes.

The Electric Power Research Institute has produced a comprehensive design manual for the use of fly ash in structural fills and highway embankments and for subgrade stabilization and land reclamation (EPRI, 1986). Another good source of information is the proceedings of conferences organized by the American Coal Ash Association, which provide a regular update in fly ash technology.

2.2.1 PROPERTIES OF FLY ASH

2.2.1.1 CHEMICAL COMPOSITION AND REACTIVITY

A microscopic view of fly ash reveals mainly glassy spheres with some crystalline and carbonaceous matter. The principal chemical constituents are silica (SiO_2), alumina (Al_2O_3), ferric oxide (Fe_2O_3), and calcium oxide (CaO). Other components are magnesium oxide (MgO), titanium oxide (TiO_2), alkalies (Na_2O and K_2O), sulphur trioxide (SO_3), phosphorous oxide (P_2O_5), and carbon (related to the "loss-on-ignition"). Water added to fly ash usually creates an alkaline solution, with a pH in the range from 6 to 11.

a. Fly ash is a heterogeneous material .The physical, chemical, and engineering properties of fly ash includes.

(1) Coal type and purity.
(2) Degree of pulverization.
(3) Boiler type and operation.
(4) Collection and stockpiling methods.

b. There is no single chemical or physical property which gives a reliable indication of the pozzolanic reactivity of fly ash. Cementitious calcium silicate and calcium aluminosilicate hydrates are formed when the glassy components of the fly ash ($(3A_21O_2)$ SiO_2 or "mullite") react with water and lime. Critical to the pozzolanity of fly ash are conditions such as

(1) Amount of silica and alumina in the fly ash.
(2) Presence of moisture and lime.
(3) Fineness of the fly ash (surface area).
(4) Low carbon content.

c. The degree of self-hardening of ash is also highly dependent on the ash's density, temperature, and age.

d. ASTM C618 distinguishes between class F and class C fly ash. Class F fly ash is normally produced from burning anthracite or bituminous coal; it has pozzolanic properties, which means that it will react with lime to form cementitious compounds. Class C fly ash is normally produced from burning subbituminous or lignite coal; in addition to being pozzolanic, it has cementitious properties of its own.

2.2.1.2 ENGINEERING PROPERTIES

The specific gravity of the ash particle ranges from 1.9 to 2.5, which is below that normally measured for soil solids. Some of the ash particles may actually float if they consist of hollow glass spheres (cenospheres); these have numerous industrial applications. The average grain size D_{50} of fly ash is likely to be in the range of 0.02 to 0.06mm. Fly ash is nonplastic and, in a dry state as collected, completely cohesionless. This lack of cohesion makes nonhardening fly ash highly erodible. In a moist, unsaturated state, surface tension of the pore water gives fly ash an apparent cohesion; if and when pozzolanic reaction occurs, considerable unconfined compressive strength is observed, increasing with age. The friction angle as measured in consolidated drained triaxial tests is typically on the order of 30^0, but values as low as 20^0 and as high as 40^0 have been reported. As a guide, compacted ash may have a dry density anywhere between 1.2 and 1.9t/m^3 and a corresponding optimum moisture content ranging from 30 down to 15%; however, more extreme values are also reported in the literature, such as γ_{dmax} =0.7 t/m^3 and w_{opt} = 60%. Low compacted density points to a potential advantage in the use of fly ash as backfill or embankment material: Low unit weight means low overburden pressures and, combined with a high friction angle, also low earth pressures.

EPRI (1986) reports that the compression index C_c of fly ash can range from0.05 to 0.37 for initial loading in recompression, these values are much lower: 0.006 to 0.04. The compressibility of compacted ash must rate as small when compared with clayey soils. Compacted dry fly ash may swell upon wetting if subjected to vertical pressures less than that equivalent to 0.5 to 1 m of fly ash fill. It was also reported that 11 to 14.5% free swell for a particular ash tested.

The permeability of a fly ash compacted to standard maximum dry density depends on the coal type it is derived from [EPRI (1986)].

Coal Type	Permeability of fly ash, cm/s
Bituminous	10^{-4} to 10^{-7}
Subbituminous	10^{-5} to 3×10^{-6}
Lignite	9×10^{-6} to 10^{-7}

Considerable capillary rise of water in fly ash fills can occur-on the order of 2 m and possibly more.

Fly ash is classed as a frost-susceptible material, which is a major drawback in and possibly more.

Negative environmental impacts from a fly ash fill are unlikely, but a study has to be made of the chemical composition of its leachate; its corrosivity on buried pipes, culverts, or other structural elements; and its radioactivity (Radium-226).

2.2.2. FLY ASH STABILIZED WITH LIME, CEMENT, AND/OR AGGREGATE

The use of mixtures of lime (L) or cement (C) and fly ash (F) with aggregate (A) giving LFA, CFA, or LCFA bases or sub bases for pavements is relatively well established in most countries. Guidelines for design and construction were given by Barenberg (1974) and other. Many local authorities have published criteria for the incorporation of pozzolanic materials with cement or lime in aggregate layers, either rated as bound or unbound layers, depending, e.g., on whether their indirect tensile strength is above or below 80 kPa (NAASRA, 1986).

To build a subbase or base course with lime-or cement-stabilized ash alone is not yet common, but this is one high-volume ash applications being promoted by ash producers.

Refering to British and American experience, EPRI (1986) quoted the following criteria as part of their design recommendation for a cement-stabilized fly ash base course.

Minimum Strength: The 7 day unconfined compressive strength of the mix, when cured under moist conditions at 21 + 22^{0}C, must exceed 2.8 to 3.1 MPa for cylindrical specimen having a length to diameter ratio 2:1.

Maximum Strength: An upper limit of strength 5.5 MPa is advised to avoid distinct cracking which may reflect through the asphalt surface.

Aging Criteria: The unconfined compressive strength of the mix is observed to increase with time.

Similar guidelines hold for lime-stabilized fly ash base courses, except that the design criteria refer to the 28-day, rather than the 7-day strength, because of the slower rate of cementation. The minimum strength required is also correspondingly higher (3.7 to 4.1 MPa). In some areas, standard strength tests must be complemented by the evaluation of durability, such as through freeze-thaw tests.

Figure 2.1 (a, b, c) shows the compaction and strength characteristics of compacted Australian fly ash with the addition of cement or lime. For this (class F) fly ash, lime was ineffective as a stabilizer: considerable amounts of cement were needed to achieve strengths as would be required in a base course. In these tests, the ash-cement combinations were compacted within 5 to 10 min after the addition of water. Ash-lime- water mixtures were allowed to cure overnight before compaction.

2.2.3 SOIL MODIFIED WITH FLY ASH AND CEMENT OR LIME

For cohesionless soils or soils with very low plasticity (plasticity index <10), cement will be more effective than lime, either alone or when combined with fly ash. For more plastic soils, either cement or lime may be added with fly ash. Only a soils testing program can indicate optimal mixes and relative economies. Fly ash could also serve as a filler in the bituminous stabilization of coarse-grained materials.

Fig 2.2 (a.b) demonstrates the effect of fly ash on the density and strength of a cement-stabilized sand. The sand in question is of medium gain size (D_{50} = 0.3 mm), is fairly uniform (USCS classification SP), and is from the Woy Woy area, New South Wales. The Miniature Harvard Compaction Test was used in these experiments; this procedure allows easy preparation of a large number of specimens, but the density results may not be equivalent to proctor compaction and the strength values may be affected by the small size of the specimen. The class F fly ash added acted primarily as a filler, enhancing the binding effect of the cement. All the materials were mixed in a dry state. Both, the density as well as the unconfined compressive strength showed maximum values when the mix was proportioned at around 20% fly ash to 80% sand.

Stabilization of a sandy road base with a fly-ash-cement mix, rather than cement alone, creates a less-permeable stiffer layer. This may result in reduced long-term maintenance. Initial financial benefits depend on local material and transport costs.

It has also been demonstrated that cement-fly-ash-sand or cement-fly-ash-gravel mixtures shrink less than soil-cement mixtures. Greater shrinkage is observed in these combinations if the cement is replaced by lime.

Fig 2.3 (a, b) presents some test results obtained with an inorganic clay of intermediate plasticity (LL = 45%, PI = 22%) from Gosford, New South Wales. The soil was air-dried and then broken down into small crumbs. The soil was premoistened for 24 hour before lime, fly ash, and additional water was added immediately before compaction.

As fig 2.3 (a, b) shows, lime and fly ash reduce the maximum dry density of clay. The corresponding optimum water content tends to increase, although results at low lime percentages (<2%) can be inconsistent. The unconfined compressive strength of this clay rose with the addition of fly ash to the lime (Fig 2.4) indications are the additional strength gains could have been achieved if the lime-fly-ash content would have been increased further. In these experiments the lime-fly-ash mix formed a coating around the soil crumbs, which was visible as a light-colored matrix in the compacted specimen.

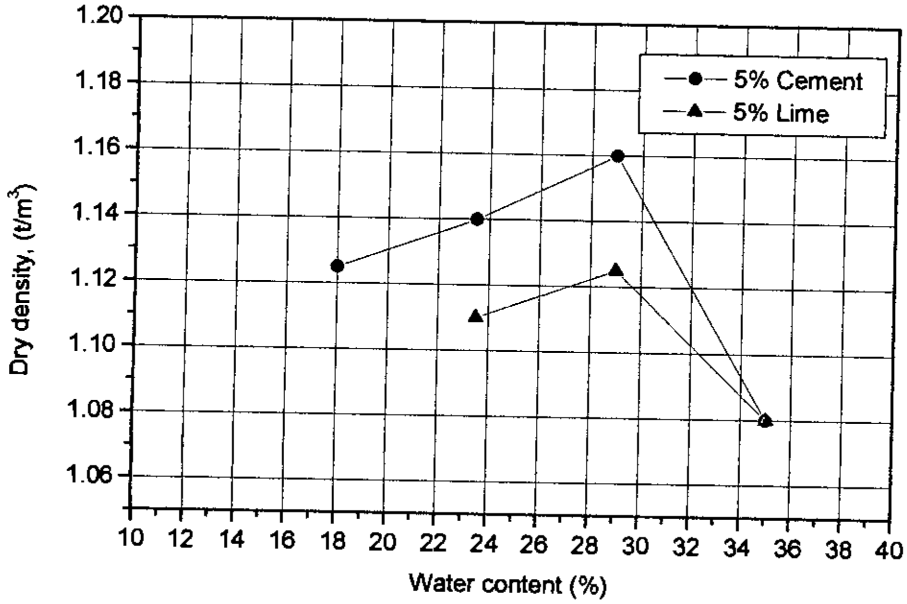

Fig 2.1 a. Compaction curves of fly ash with 5% additive (After Hausmann, 1990)

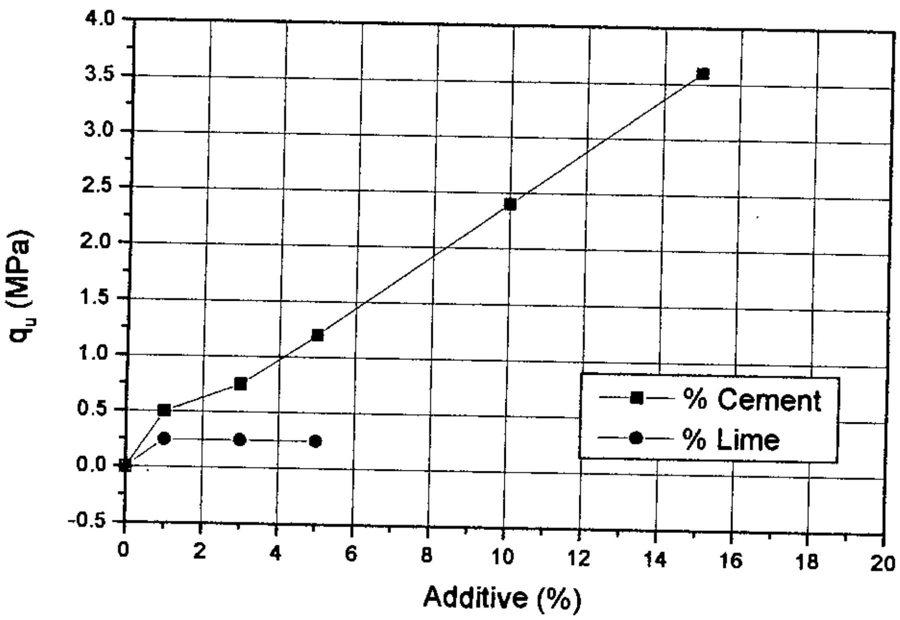

Fig 2.1 b. Unconfined compressive strength as a function of the additive content (specimen compacted near optimum water content with standard compactive effort) (After Hausmann, 1990)

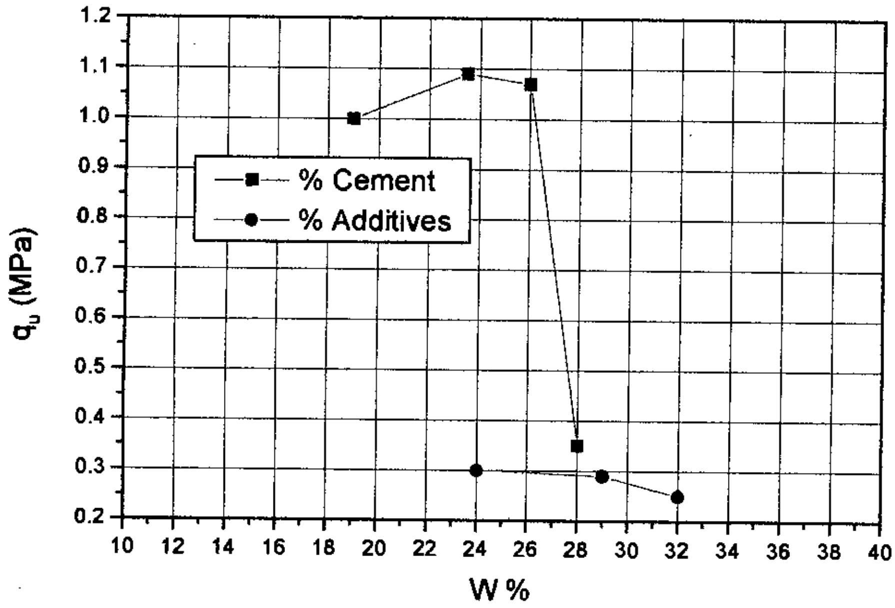

Fig 2.1.c Unconfined compressive strength as a function of the water content of compaction (After Hausmann, 1990).

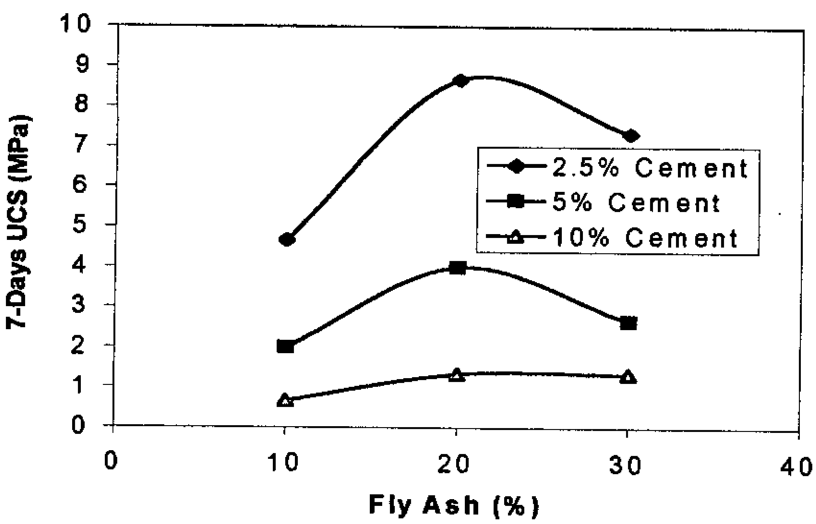

Fig 2.2 a. Maximum dry density of sand-fly-ash-cement mixes fly-ash corresponds to 90% sand, while % cement is expressed terms of the fly-ash-soil mix) (After Hausmann, 1990).

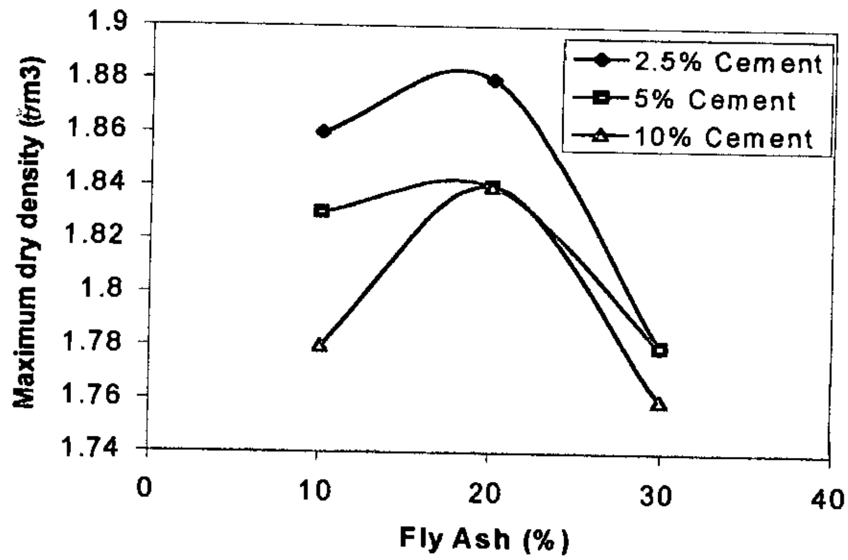

Fig 2.2 b. Unconfined compressive strength (After Hausmann, 1990).

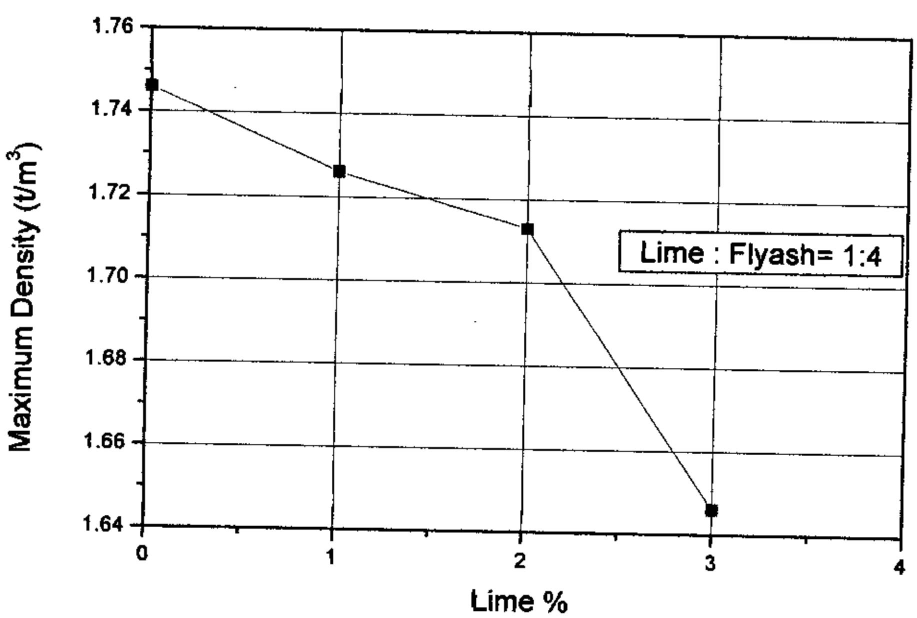

Fig 2.3 a. Maximum dry density (Proctor compaction) (After Hausmann, 1990)

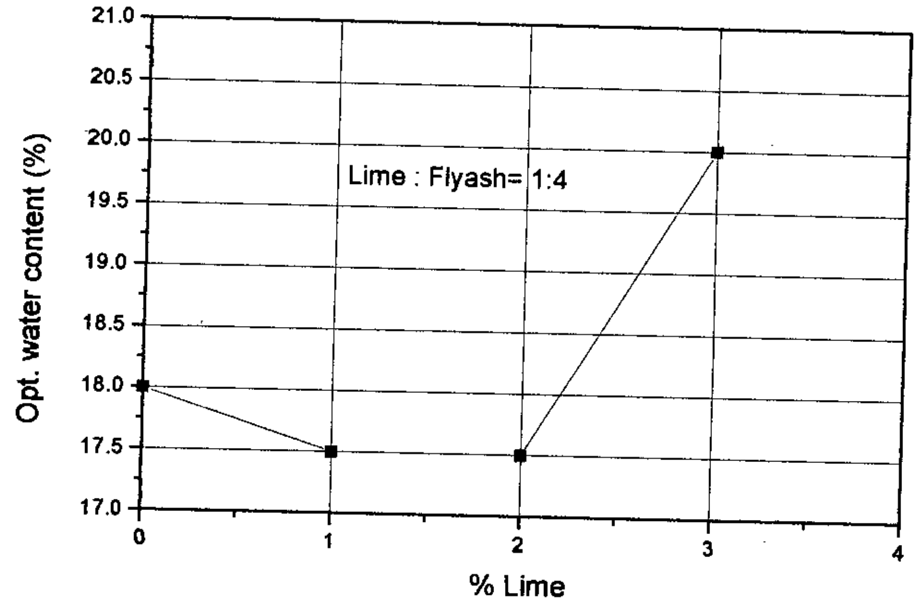

Fig 2.3 b. Optimum water content (Proctor compaction) (After Hausmann, 1990)

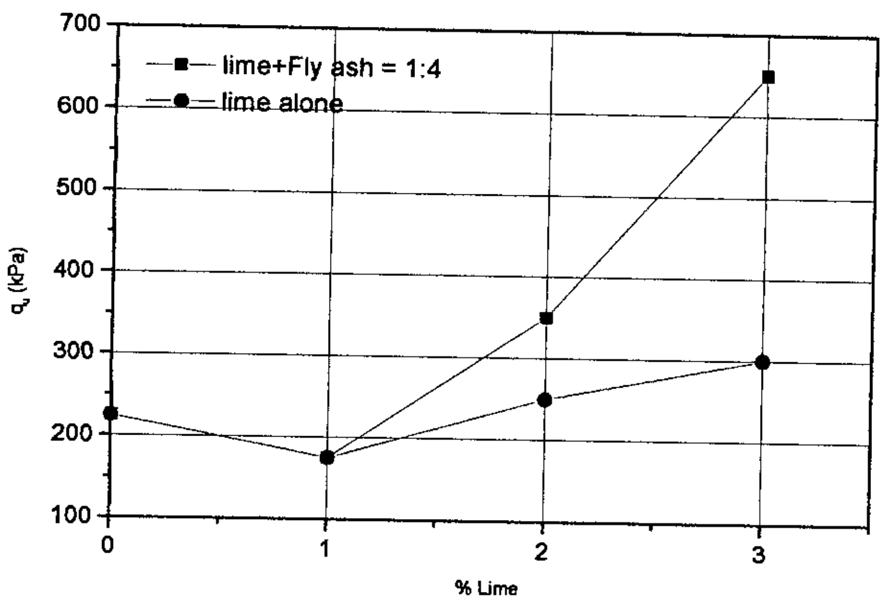

Fig 2.4 Unconfined compressive strength of a medium plastic clay with lime and lime-fly-ash additive (After Hausmann, 1990)

2.2.4 MECHANICAL BEHAVIOUR OF COAL FLY ASHES

2.2.4.1 GENERAL

Gatti and Tripiciano (1981) found out the mechanical behavior of coal fly ashes and the results of a series of laboratory tests of coal ashes are presented on the basis of which the shear strength and deformability characteristics of the waste material can be defined. Different conditions of placing and ageing are considered. A stress strain relationship, suitable for practical applications, is derived which allows for relating the material behavior to the ageing time.

2.2.4.2. DESCRIPTION OF THE MATERIAL AND SAMPLE PREPARATION

The chemical composition of the ashes, deriving from coal burnt with 5% of gas oil, is was follows:

:

a.	Unburned materials	5.0%.
b.	Silica (SiO_2)	46.0%
c.	Iron (Fe_2O_3)	3.5%
d.	Alluminium (Al_2O_2)	34.5%
e.	Calcium (CaO)	7.0%
f.	Sodium (NaO)	0.2%.
g.	Magnesium (MgO)	1.8%.
h.	Potassium (K_2O)	0.5%.
j.	Sulphat (SO_3)	1.5%

The results of the mineralogical analysis, by means of x-ray diffractometer are reported in fig 2.5. If they concern both virgin ashes and residual products after heating up to 1000 C. The identified crystalline phases are: Mullite ($Al_6\ Si_2O_{13}$), about 25%, and quartz (SiO_2), about 3%. Traces of Cristobalite are present in the residual material after heating. The presence of Mullite is due to Alluminous clay minerals contained in the coal. Since the amount of crystalling phases is about

25% to 30% of the ash total weight, the nature of amorphous components was determined by thermal analysis (fig 2.6) showing a behavior typical of carbonous substances.

The results of grain size analyses are reported in fig 2.7. The virgin ashes are very uniform and have small particle size; the specific weight of the grains is between 22.2 and 22.6 kN/m^3 and atterbergs limits are PL= 36%, LL= 39%. Since the material presents a pozzolan like behavior with time, two series of tests were performed on the virgin ashes (V) and on the recovered ashes (R), obtained by pulverizing ashes the sitting and hardening of which were allowed for 40 days. The R-ashes, whose grain size distribution is reported in fig 2.7, are characterized by the following index properties: Density = 22.5 – 22.6 kN/m^3; PL = 41%, LL = 44%. In order to simulate the field conditions of hydraulic and compacted fills two different procedures were used for sample preparation adopting:

a. Ashes mixed with water at a percentage much higher than the liquid limit (68% - 75% for V-ashes; 75% - 95% for R-ashes), without compaction, but allowing hardening with time (V-A and R-A samples).

b. Ashes compacted (Modified proctor Test-ASTM D-1557-58T) at the optimum water content and at a water content smaller than the optimum one (V-B and R-B samples).

In order to simulate various environmental situations the samples were aged under the following conditions :

a. Room temperature and humidity (not controlled).
b. Room temperature, 100% relative humidity.
c. Room temperature, under water.
d. 20 to 22 ^{0}C temperature, 65% - 75% relative humidity.
e. 20 to 22 ^{0}C temperature, 100% relative humidity.
f. 20 to 22 ^{0}C temperature, under water.

The tests were carried out after 7 to 180 days (ageing time) from the sample preparation. This in order to investigate the variation of the geotechnical properties with time.

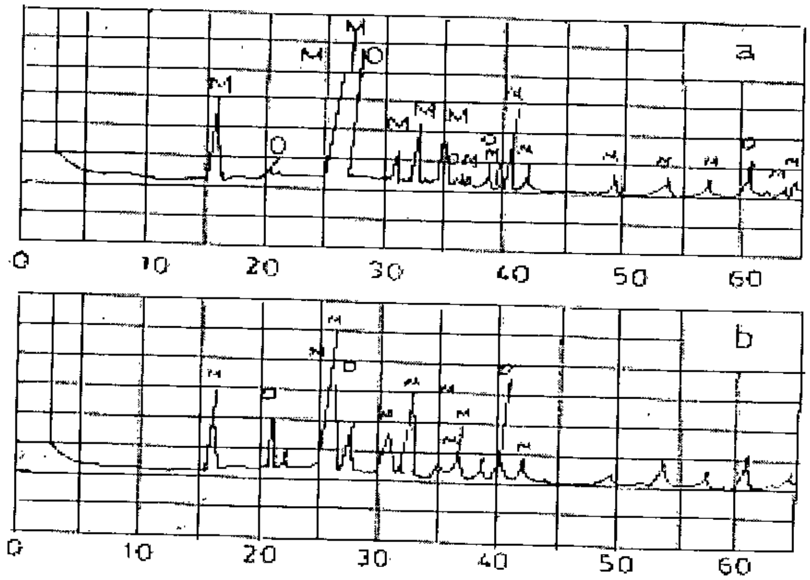

Fig 2.5. X-ray diffractometer traces: a-before, b-after thermal analysis (After Gatti and Tripiciano, 1981)

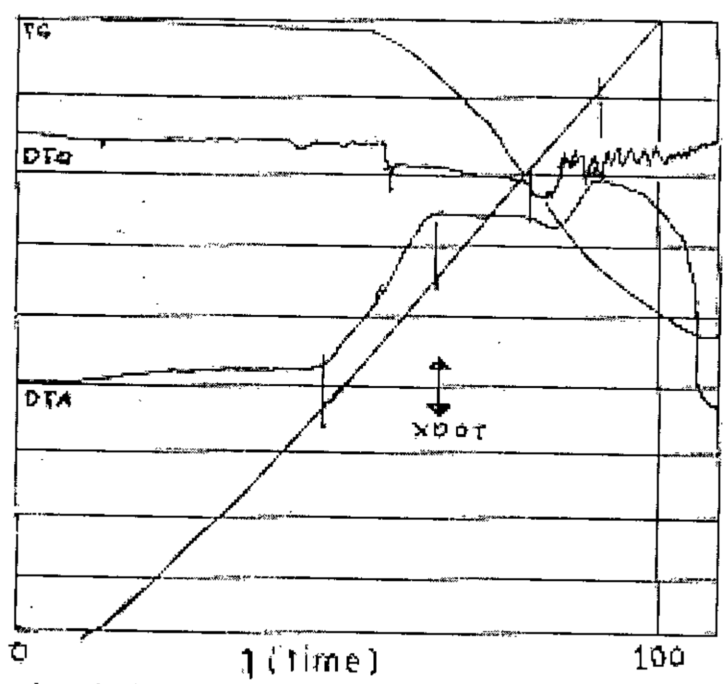

Fig 2.6. Qualitative thermal analysis : TG weight loss : weight loss vel : DTA esot. eff : T °C. (After Gatti and Tripiciano, 1981)

2.2.4.3 LABORATORY TESTS

A. SETTING TIME

The setting time was determined on samples prepared with V-ashes, mixed with various percentage of water, which were kept at content temperature (20^0C) and humidity (75%). The results of tests are reported in Table 2.1.

Table 2.1 Setting Time Tests

Initial water content %	Beginning of setting (hour)	Time of setting (hour)
53	12	7
64	36	8
75	53	19

B. COMPACTION AND CBR TESTS

The results of tests are reported in fig 2.8. For V-and R-ashes, respectively, the optimum water content was 28% and 36% and the maximum dry density was 12, 24 and 11.40 kN/m^3. The CBR tests were performed on samples of V-B—ashes at optimum water content. After immersion in water for 3 and 28 days the values of CBR were, respectively, 25% and 250%.

C. PERMEABILITY TESTS

The tests were carried out on V-B samples compacted at the water content of 11.5% and 28%. In fig 2.9, it is shown that the permeability decreases with time; as expected, the samples with water content close to the optimum have coefficient of permeability smaller than that of the other samples.

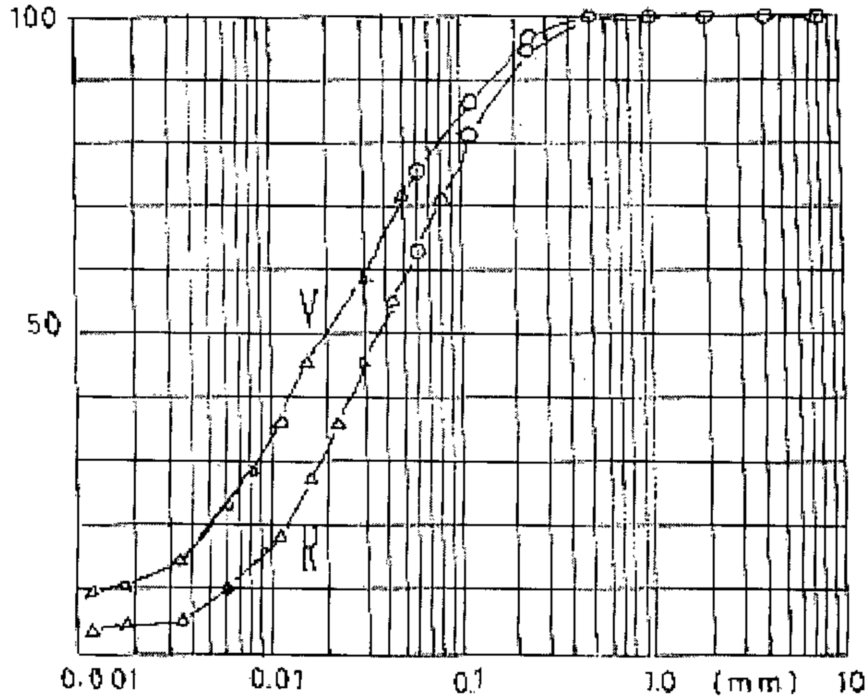

Fig 2.7. Grain size composition (After Gatti and Tripiciano, 1981)

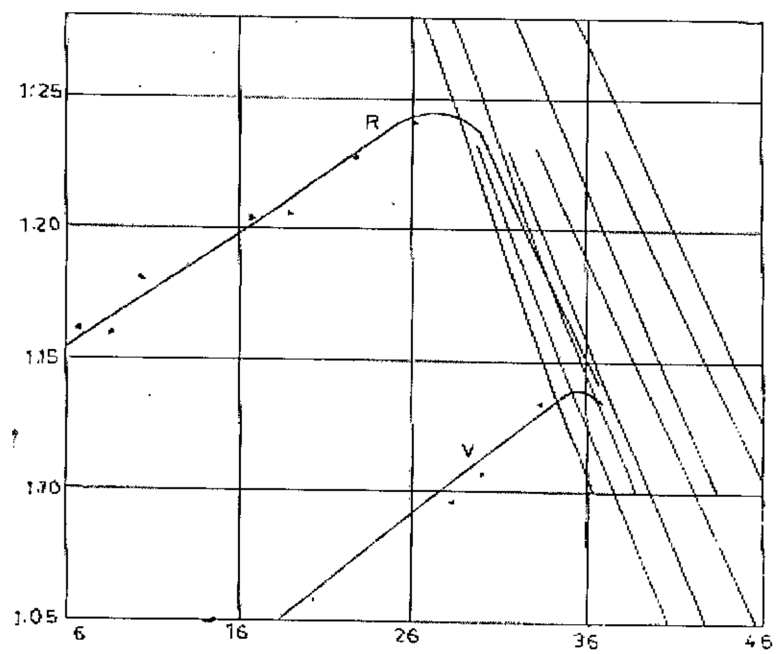

Fig 2.8. Compaction test (After Gatti and Tripiciano, 1981)

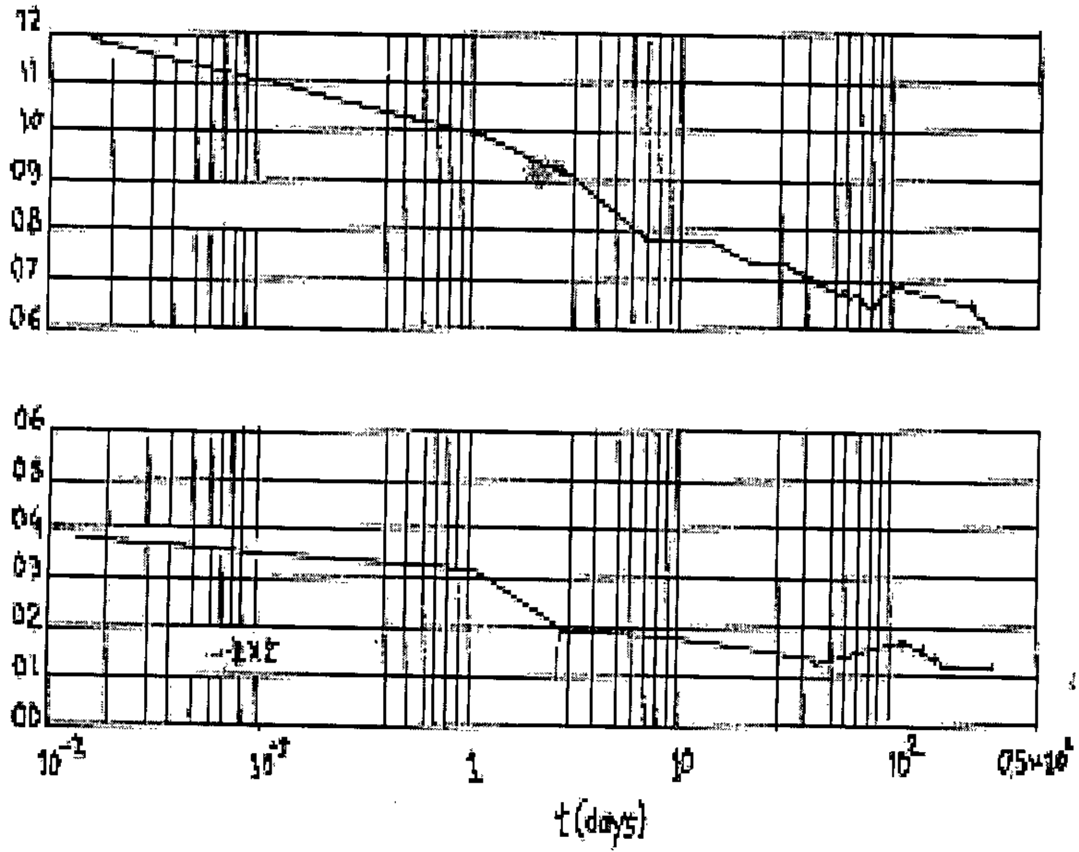

Fig 2.9 coefficient of permeability (k) of V-ashes vs. time (days) (V-B samples) (After Gatti and Tripiciano, 1981)

D. UNCONFINED COMPRESSION TESTS

The tests were performed on V-A samples stored under conditions 1 of 6, and on R-A-5 and R-A-6 samples, with T ranging from 2 to 180 days. The samples were prepared during February, August and November. For all tests the unconfined compression strength appears to decrease with increasing moisture. The high resistance of V-A-1 and V-A-4 samples is probably due to noticeable reduction of the water content. The vertical strains at failure decreases with increasing T; the reduction is more appreciable for samples stored under conditions 1 and 4 than for the other samples. The R-A samples have very low compression strength, about 10kpa, practically constant with T. The different behavior of V-A and R-A samples is clearly due to the pozzolan type properties of the V-ashes.

E. TRIAXIAL TESTS

Triaxial CIU tests were performed on V-A samples aged. Other tests were performed on V-B samples compacted at W=12% and 28%, and on R-B samples compacted at w=30% and 36%, without ageing. In 1980 CID tests have been carried out only on 2.5% and 4% while that at 50% of the failure load is between 0.7% and 1%.

The limited experimental information so far obtained from CID tests does not allow for assessing the influence of T on the stress-strain relation ship. However, it appears that the normalization of the experimental data is possible by means of equation of fig 2.10 V-A-5 samples at T=7 days) on by the equation proposed by Richard and Abbott (1975). The value of E^1 are between 0.7 and 1.8 10^4 kPa and those of E^1 are between 0.7 and 1.3 10^4 kPa and those of E* are between 0.4 and 10.10^4 kPa. Knowing the vertical ε_1 and volumetric ε_V strains it is easy to obtain the shear strain γ_{13}. In fig 2.11 the $\tau_{13}=(\sigma_1 - \sigma_3)/2$vs. τ_{13} is reported for V-A-5 samples with T=7 days.

F. OEDOMETER TESTS.

Both V-A and V-R samples were tested. No ageing was allowed for V-A samples thus the setting process of the virgin ashes developed during the tests. In order to investigate the effect of this phenomenon non standard odometer tests were performed in addition to the standard ones, with load increments applied either every 12 hours or every hour. The results of such tests are reported in fig 2.12 together with those of the tests on R-A samples.

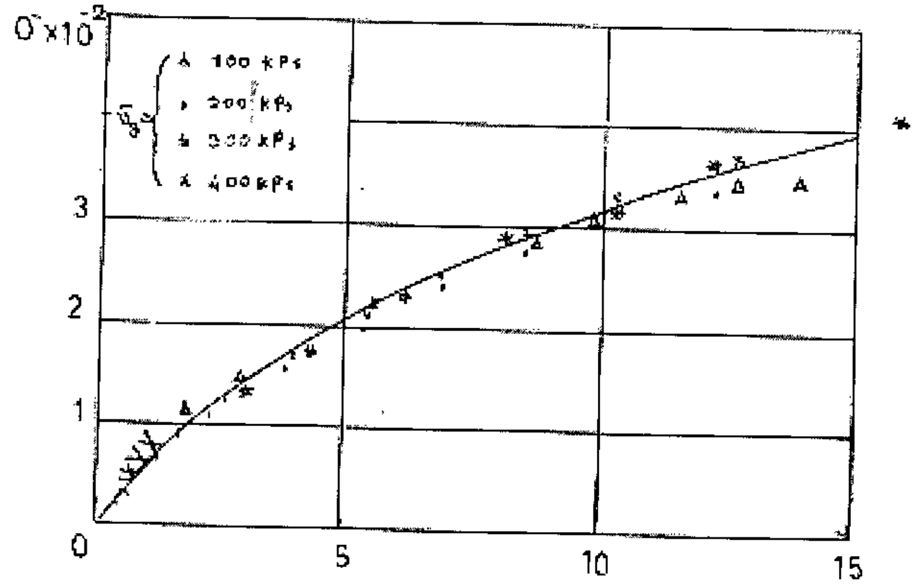

Fig.2.10 CID triaxial test experimental values (v-A-5 sample at 7=days) (After Gatti and Tripiciano, 1981)

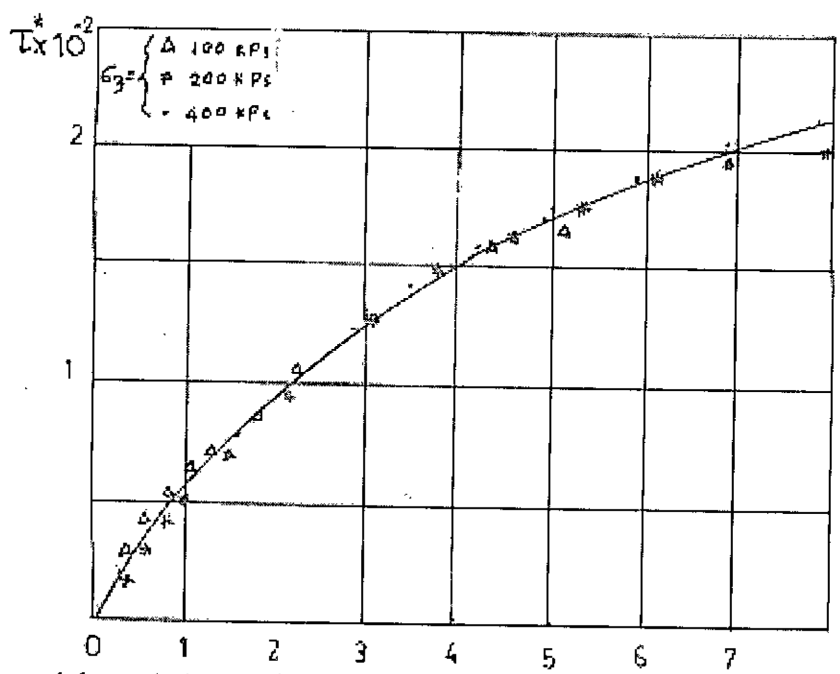

Fig. 2.11 CID in axial test (τγ) experimental values (v-A-5 samples at T=7 days) (After Gatti and Tripiciano, 1981)

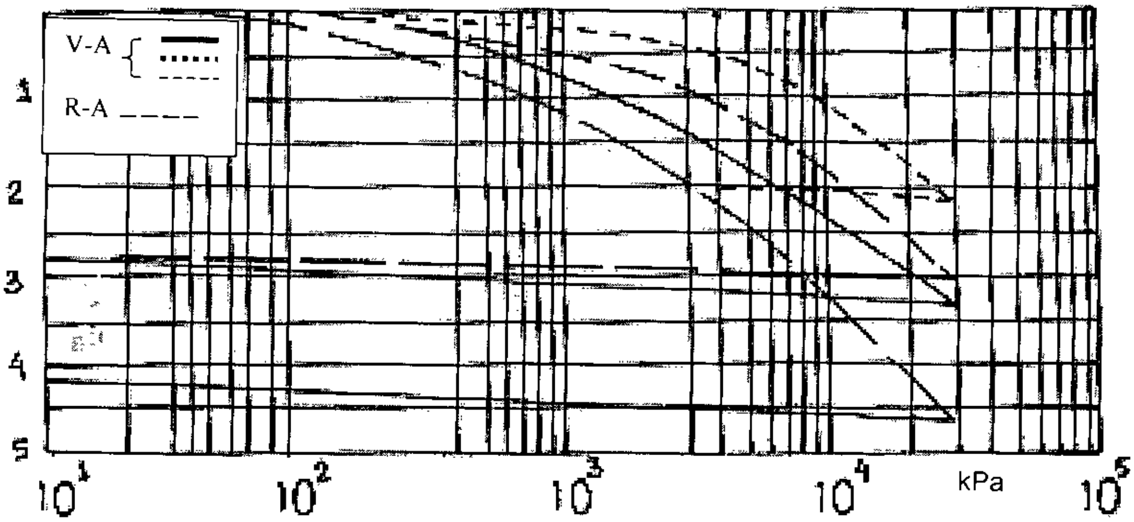

Fig 2.12. Typical odometer test (After Gatti and Tripiciano, 1981)

2.2.4.4 Finally the main findings of the experimental study so far carried out can be summarized as follows:

a. The virgin coal ashes type A (i.e. mixed with water and not compacted) snow a remarkable variation of strength and deformability characteristics with time due to the presence of alluminous clay minerals in the coal.
b. On the contrary, for the recovered ashes type A such a variation is quite limited.
c. A stress-strain relationship applicable to practical problems was determined for ashes type A on the basis of triaxial CIU tests.
d. The experimental results show that the use of virgin ashes is preferable for fields under water, while for compacted embankments both virgin and recovered ashes can be adopted.

2.2.5 PERMEABILITY OF FLOWABLE SLURRY MATERIALS CONTAINING FOUNDRY SAND AND FLY ASH

2.2.5.1. General

Naik and Singh (1997) made a study to evaluate the effect of foundry sand and fly ash on permeability of flowable slurry mixtures. In this work, two reference flowable fly ash slurry mixtures were proportioned for strength levels in the range of 0.34-0.69 MPs (50-100 psi) at 28 day using two different sources of ASTM Class F fly ash. Other mixtures contained clean and used foundry sands as a replacement for fly ash in the range of 30-85%. The permeability of the flowable mixtures was affected by an increase in either the water to cementitious materials ratio or the foundry sand content. The permeability values were either comparable to or lower titious materials ratio or the foundry sand content. The permeability values were either comparable to or lower than those reported for granular compacted fills up to 85% fly ash replacement with foundry sand. The type of foundry sand (clean or used) did not materially affect permeability of the mixtures tested. The permeability values for the mixtures tested varied from 3×10^{-6} to 74×10^{-6} cm/s.

Previously Naik and his associates have conducted a great deal of research. A large number of investigations have been carried out to develop excavatable CLSM mixtures having the 28d compressive strength in the range of 0.34-0.69 MPa. However, no information is available on properties of CLSM containing foundry sand. Very few measured the permeability of CLSM mixtures. Generally, the pereability of excavatable slurry mixtures is identical to that of compacted granular fills, ranging between 10^{-5} and 10^{-5} cm/s. To reduce permeability, the cementitious content of flowable slurry material needs to be increased. The permeability reported by the Electric Power Research Institute for Class F fly ash slurry material ranged from 19.3×10^{-6} cm/s for 5% cement slurry to an average of 3.3 x 10-4 cm/s for 20% cement slurry. This study also found that Class F fly ash slurry mixtures had higher permeability than mixtures containing Class C fly ash.

Latter on he carried out a research to evaluate the strength and permeability of excavatable flowable slurry materials incorporating clean as well as used foundry sands with fly ash. The result of this work would be useful in establishing mixture proportion and production technology for foundry sand containing flowable slurry materials for field applications.

2.2.5.2 MATERIALS

A. SAND

Both clean (unused) and used foundry sands were obtained for this investigation. The clean sand was obtained from a sand mining company is Wisconsin and the used foundry sand was obtained from Maynard Steel Casting C. Milwaukee. For the purpose of comparison. some properties of regular concrete sand were also measured. Physical properties of foundry sands were determined using appropriate ASTM standards. However, a modified ASTM C 88 was used to measure soundness of the foundry sands. In accordance with ASTM C 88 test standards, the test sample was such that it contained 100 g of all materials retained on each of the no. 4 (4.75mm). no. 8 (2.36mm), no. 16 (1.18mm), no. 30 (600um), and no. 50 (300 um) sieves, and respectively passed through the following sieves : 9.5 mm (3/8 in.), no 4 (4.75mm), no 8 (2.36mm), no. 16 (1.18 mm), and no. 30 (600um). Since foundry sand was finer than the no. 30 sieve, only about 0.2-2.1% of the sands was retained on the no.4 (4.75mm)-no. 30 (600um) sieves. Therefore, the ASTM sample requirement was modified to evaluate the soundness of the foundry sands for this investigation. Only one sample was used (100 g passing through the no. 30 (600 um) sieve and retained on hte no. 50 (300um) sieve.

The physical properties data for the two foundry sands and regular concrete sand are shown in Table 2.2. The properties of used foundry sand vary greatly due to the type of metal cast and foundry processing equipment used, the type of additive for mold marking, the number of times the sand is recycled in the process, and the type and amount of binder used. The unit weight of the used sand was greater than that of the clean, sand which may be attributed to particles of such materials as steel pellets bonded to the sand during the foundry process. Both the clean and the used foundry sands exhibited high absorption values compared to the regular concrete sand. However, the difference between the values for clean and used foundry sands was insignificant. The materials finer than the no. 200 (75um) sieve were slightly higher for the used foundry sand, relative to the clean foundry sand. The difference it the results probably was due to the presence of binders in the used foundry sand. The ASTM limit for deleterious substance in fine aggregate is 5% for all concretes (for concrete subjected to abrasion, it is 3%). The results for the sand used for the project showed low values of clay lumps and friable particle for all sands tested: all the values were less than the allowable ASTM limit. However, the used foundry sand has the highest value of all the sands tested. This is primarily due to the presence of binders in the used foundry sand, which were probably dissolved during the soaking in water for 24 h and then were washed away when sieved in accordance with ASTM C 142. The mass losses suffered were 10% for the regular concrete and 10.5% for the clean foundry sand when subjected to the soundness test in accordance with ASTM C 88. Thus, both the sands showed values below the ASTM limit of 12%. However, the loss for the used foundry sand was very high (54.9%). This occurred because the used sand particles were weakness due to temperature shock that occurs during molding operations. This led to cracking and quicker deterioration of the used sand particles in the chemicals used for the soundness test pet ASTM.Since CLSM is a low-strength material, the used of weaker sand produced during molding will not cause any problems in obtaining the strength levels varying from 0.34 to 0.69 MPa (50 to 100 psi). Therefore, higher values of mass loss during the soundness testing will not have any appreciable effect on the performance of used foundry sand containing CLSM. These foundry sands have also shown acceptable performance in concrete. The sieve analysis data are presented in Table-2.3.

Table 2.2. Physical properties of sand (After Naik, T.R. And Singh, 1997)

	As Received Moisture Content (%)	Unit Weight (kg/m^3)	Bulk Soecific Gravity	Bulk Specific Gravity (SSD)	Apparent Specific Gravity	SSD Absorption (%)	Void (%)	Fineness Modulus	Clay Lumps and Friable Particles (%)	Soundness of Aggregates (%)	Material Finer Than no 200 (75 cm) Sieve
Sand type (1)	ASTM C566 (2)	ASTM C29 (3)	ASTM C128 (4)	ASTM C128 (5)	ASTM C128 (6)	ASTM C128 (7)	ASTM C29 (8)	ASTM C136 (9)	ASTM C136 (10)	ASTM C88 (11)	ASTM C117 (12)
Sand 1	0.39	1.840	2.43	2.47	2.52	1.0	25.0	3.57	0.2	10.0	1.40
Sand 2	0.19	1.730	2.38	2.50	2.70	4.9	33.8	2.33	0.1	10.5	0.17
Sand 3	0.25	1.784	2.44	2.57	2.79	5.0	34.8	2.32	0.4	54.9	10.8

Note Sand 1 – regular concrete sand; Sand 2 – clean foundry sand (FS1) : and sand 3 – used foundry sand (FS2)

Table 2.3 Sieve Analysis Results for Sand (ASTM C 136)

	Percent Retained on Each Sieve			Cumulative Percent Retained			Cumulative Percent Passing			Required ASTM c 33
Sieva size (1)	Sand 1 (2)	Sand 2 (3)	Sand 3 (4)	Sand 1 (5)	Sand 2 (6)	Sand 3 (7)	Sand 1 (8)	Sand (9)	Sand 3 (10)	Cumulative passing (%) (11)
No 4	0.1	0.0	0.0	0.1	0.0	0.0	99.9	100	100	95-100
No 8	12.8	0.0	0.0	13.0	0.0	0.0	87.0	100	100	80-100
No 16	13.6	0.0	0.0	26.6	0.0	0.0	73.4	100	100	50-45
No 30	18.9	0.1	0.5	45.5	0.1	0.5	54.5	99.9	95.5	25-60
No 50	32.2	41.4	46.1	77.7	41.5	46.6	22.3	58.5	53.4	10-30
No 100	16.6	54.6	47.1	94.2	96.1	93.7	5.8	3.9	6.3	2-10

Note : Sand 1 – regular concrete sand : Sand 2 – Clean foundry sand (FS1) : and Sand 3 – Used foundry sand (FS2)

The sieve analysis grading curves were plotted along with the ASTM standard grading requirements for regular sand used in concrete mixtures. From the plot it is found that both the clean foundry sand and the used foundry sand are finer and they are outside the ASTM limits. It was found that the foundry sands contain predominantly finer particles compared of those of regular sand. Approximately 50-60% of the clean and used foundry sands passed through the no. 50-60% of the clean and used foundry sands passed through the no. 50 sieve (95-100% passed through the no. 30 sieve). The chemical properties of the foundry sands were also determined and are reported elsewhere (Naik and Singh 1994).

B. FLY ASH

Two ASTM Class fly ashes (F1 and F2), obtained from two different sources in Wisconsin, were used in this work. Their physical properties were determined in accordance with ASTM C 311. The test data on these fly ashes are shown in Table 2.4. All the physical properties of the fly ashes, except the strength activity index at 7 d for fly ash F1 and loss on ignition for fly ash F2, satisfied the requirements of the ASTM C 618 for Class F fly ash. The chemical properties data on these fly ashes are reported in Table 2.5.

375 CEMENT

A type 1 cement secured from one source was used throughout this investigation. Its physical and chemical properties methods. The result of the physical and chemical properties of the port land cement used in this work is shown in Tables 2.6 and 2.7, respectively. The cement met the ASTM C 150 specification for Type 1 cement.

Table 2.4. Physical properties of fly ashes. (After Naik, T.R. And Singh, 1997)

Physical Properties (1)	Fly ash F1 (2)	Fly ash F2 (3)	ASTM C 618 Class F fly ash (4)
Fineness retained on no. 325 sieve (%) strength activity index with cement, 7	20.3	13.7	34 maximum
d (% of control)	62.5	83.7	75 minimum
Strength activity index with cement, 28			
d (% of control)	75.0	86.9	75 minimum
Water requirement (% of control)	100	10.33	105 maximum
Autoclave expansion (%)	0.00	-0.03	=0.8
Specific gravity	2.22	2.19	-
Required for use in concrete			

Table 2.5 Chemical Composition of Portland Cement and Fly Ashes (After Naik, T.R. And Singh, 1997)

Analyte (1)	Lafarge, Type 1 cement (%) (2)	ASTM C 150 Type i (%) (3)	F1 (%) (4)	F2 (%) (5)	ASTM C 618 Class F (%)
S_iO_2	20.3	-	48.4	46.1	-
Al_iO_3	4.3	-	27.0	24.4	-
Fe_2O_3	2.6	-	6.6	21.6	-
Total $S_iO_2 + Al_iO_3 \div Fe_2O_3$	27.2	-	82.2	92.1	70.0 minimum
SO_3	-	-	0.6	1.5	5.0 maximum
MgO	2.2	6.0 maximum	2.0	1.0	5.0 maximum
CaO	63.6	-	8.5	3.2	-
T_iO_2	0.3	-	1.3	0.9	-
K_2O	0.80	-	1.0	1.4	-
Moisture content	-	-	0.2	0.4	3.0 maximum
Loss on ignition	0.6	3.0 maximum	2.8	10.7	6.0 maximum
Required for use in concrete					

Table 2.6. Physical Properties of Portland Cement. (After Naik, T.R. And Singh, 1997)

Physical test (1)	Lafarge, Type 1 cement (2)	ASTM C 150 Type 1 cement (3)
Air content of cement mortar (%)	9.1	12 maximum
Fineness: specific surface (air permeability test) (m^3/kg)	367	280 minimum
Autoclave expansion (%)	-0.01	0.8 maximum
Compressive strength (MPa)	3.12	-
1day	14.4	-
3day	26.6	12.3
28day	38.1	-
Vicat time of initial setting (min)	207	45 minimum
Vicat time initial setting (mim)	-	375 maximum

2.2.5.3. MIXTURE PROPORTIONS FOR FLOWABLE SLURRY MATERIALS

In this work, two reference flowable fly ash slurry mixtures were used. The first one was proportioned with fly ash F1. The second mixture was proportioned with fly ash F2. Both mixtures were proportioned to obtain flowable slurry. For each reference mixture, additional mixtures were proportioned with foundry sand as a partial replacement of fly ash. All mixtures were proportioned to have the 28 day compressive strength in the range of 0.34-0.69 MPa (50-100 psi). A total of 18 different fly ash slurry mixtures were proportioned and produced at the CBU Concrete Research Laboratory. Of these, two were the control mixtures without foundry sand, and the remaining 16 had four different replacement of fly ash by the foundry sand was on a weight basis. The mixture proportions are presented in Tables 2.7 and 2.8. The flow/spread wad determined in accordance with the ACI 229, method, as reported earlier by Naik et, al (1990).

Table 2.7. Mixture Proportions and Fresh Slurry Properties for FlyAsh F1 Mixture (After Naik, T.R. And Singh, 1997)

Item	Mixture Number								
(1)	S1 (2)	S2 (3)	S3 (4)	S4 (5)	S5 (6)	S6 (7)	S7 (8)	S8 (9)	S9 (10)
Foundry Sand (%)	0	30 (FS1)	50 (FS)	70 (FS1)	85 (FS1)	30 (FS2)	50 (FS2)	70 (FS2)	85 (FS2)
Cement (Kg/m^3)	36	44	37	35	46	44	37	36	46
Fly ash (kg/m^3)	1.044	899	737	482	244	899	737	490	248
Foundry sand (kg/m^3)	0	398.	756	1.149	1.274	405	757	1.104	1.434
Water (Kg/m^3)	540	450	406	363	363	450	405	368	369
[W/(C+FA)]	0.50	0.48	0.52	0.70	1.25	0.48	0 52	0 70	1.25
Flow/spread (mm)	413	406	400	406	406	406	406	400	413
Air content (%)	1.2	1.2	0.6	0.6	0.7	1.2	0.8	0 7	0.7
Air temperature (^{0}C)	13.9	11.1	14.4	16 7	14.4	13.9	14.4	14.4	14.4
Slurry temperature (^{0}C)	16.1	16.1	15.6	16.7	16.1	17.2	19.4	17.4	17.2
Slurry density (kg/m^3)	1.621	1.791	1.948	2.027	2.065	1.797	1.932	2 054	2.108
*FS1 = clean foundry sand : and FS2 = used foundry sand									

Table 2.8. Mixture Proportions and Fresh Slurry Properties. (After Naik, T.R. And Singh, 1997)

Item	Mixture Number								
(1)	P1 (2)	P2 (3)	P3 (4)	P4 (5)	P5 (6)	P6 (7)	P7 (8)	P8 (9)	P9 (10)
Foundry Sand (%)	0	30 (FS1)	50 (FS)	70 (FS1)	85 (FS1)	30 (FS2)	50 (FS2)	70 (FS2)	85 (FS2)
Cement (Kg/m^3)	47	46	44	47	44	47	46	47	45
Fly ash (kg/m^3)	834	795	634	451	242	812	666	478	249
Foundry sand (kg/m^3)	0	356	633	1.105	1.461	549	710	1.166	1.503
Water (Kg/m^3)	685	561	507	297	322	361	467	351	311
[W/(C+FA)]	0.78	0.67	0.75	0 60	1.12	0 42	0.66	0.67	1.05
Flow/spread (mm)	298	292	305	305	330	305	311	337	318
Air content (%)	0.8	1.2	0.4	0.5	0.4	1.3	0.5	0 3	0.3
Air temperature (^{0}C)	14.4	34.4	32.8	14.4	16.1	15.5	14.4	16.1	16.1
Slurry temperature (^{0}C)	17.2	18 9	18.3	18.9	19.6	17.2	17.8	19.4	20.6
Slurry density (kg/m^3)	1.567	1.756	1.847	1.900	2.067	1.769	1.906	2.038	2 108
*FS1 = clean foundry sand : and FS2 = used foundry sand									

2.2.5.4 MANUFACTURING TECHNIQUE

All the constituent materials for the fly ash slurry mixtures were mixed at the CBU Concrete Research Laboratory using a 0.25 m^3 capacity power-driven revolving drum mixer. At the present time, a standard mixing procedure for slurry is not available. As a result, the mixing procedure, as described next, was developed by the writers at CBU (Naik et al. 1990). For the control mixtures without foundry sand, the inside of the mixer was initially sprayed with water, and then the mixer drum was drained of any excess water. All the cement and half of the mixing water were added in the mixer and mixed for three minutes. Then, half of the fly ash wad added and mixed for three more minutes. The remaining water and fly ash were alternately added in smaller quantities to obtain the required consistency. Finally, the entire batch was mixed for five minutes. However, for all other mixtures, after spraying the mixer with water and draining the excess water, all the foundry sand and cement were mixed together for three minutes, then half of the water required was added and mixed for another three minutes. Thereafter, half of the fly ash was added and mixed for three minutes and the remaining fly ash and water were added alternately in small quantities. Finally, the entire batch was mixed for five minutes. The flow/spread, air content, temperature, density, etc. were determined for each test mixture before casting test specimens.

2.2.5.5. PREPARATION AND TESTING OF SPECIMENS

Cylinders (150 x 300mm) were made for the measurement of plastic properties as well as the compressive strength of the flowable slurry materials. To measure permeability of mixtures, 100 x 125 mm cylinders were cast. All specie preparations were done in accordance with ASTM C 192. Cylinders up to 14 d were also evaluated for bleed water. Tlement 50mm (2 in.) 16 penny nail penetration, and shrinkage cracks. These parameters were determined at ages of 1, 3, 5, 7, 10 and 14 d. The settlement was determined measuring the height of the cylindrical specimen (150 x mm) placed in a plastic mold at each test age. A decrease height of the cylindrical specimen was used as a measure settlement of the mixture. The nail penetration test was formed by applying moderate pressure (22-44N) on the mm long nail. At each age, the compressive strength was sured in accordance with ASTM D 4832. The permeability the mixtures was evaluated in accordance with ASTM d 5.

2.2.5.6. TEST RESULTS AND ANALYSIS

A. PLASTIC PROPERTIES

The plastic properties of the flowable slurry mixture mined were flow/spread, temperature, unit weight, settlen bleed water, shrinkage cracks, and conditions of set. A discussion of these plastic properties is included in this p. A detailed description of the results on plastic properties these mixtures has been reported elsewhere (Nail and S 1994). The unit weight of slurry material was found to in the range of 1.570-2.115 kg/m^3.

The mixtures made with fly ash F1 showed some bleed at the 1 hage, and the bleed water decreased generally time up to 1 day. In the case of the fly ash F2 mixtures, mixtures except the 85% foundry sand mixtures exhibited sence of bleed water even at the 1h hage. This may be attri to the greater fineness of fly ash F2 and the lesser amo water used in these mixtures compared to the fly ash mixtures. All the fly ash F2 mixtures became hard at the age of 5 days.

Mostly due to setting and hardening of the mixtures, the depth of nail penetration decreased with age. Test data showed a slight increase in settlement up to 3d. Thereafter, the settlement became approximately constant. In general, total settlement was found to be less than 18 mm for the F1 mixtures and 3.2 mm of the F2 mixtures with or without foundry sand up to 14 days. The settlement substantially decreased with decreasing water content of the mixtures evaluated in this work. For maintaining settlement less than or equal to 3 mm (1/8 in), the water content of the mixtures should be maintained so as to have a flow of 279 mm of less. All of the test specimens showed absence of shrinkage cracks up to the 14 days age.

B. COMPRESSIVE STRENGTH

The compressive strength data for the slurry mixtures tested are plotted in Figs 2.13-2.15. The compressive strength for all the slurry mixtures with and without foundry sand varied from 0.17 to 0.41 MPa (25 to 60 psi) at the 7 days age (Fig 2.13). The compressive strength values ranged from 0.27 to 0.55 MPa (40 to 80 psi) for the fly ash F1 mixtures and 0.31 to 0.62 MPa (45 to 90 psi) for the fly ash F2 mixtures at 28 days (Fig. 2.14). These values are in the range specified at 28 d of 0.3400.69 MPa (50-100 psi).

As expected, the compressive strength increased with age for all the mixtures tested. In general, the compressive strength increased with increasing amount of foundry sand up to a certain limit and then decreased (Fig 2.15). The level of foundry sand corresponding to the maximum compressive strength depended greatly upon mixture proportions, type of foundry sand (clean or used), and age. Based on the compressive strength results, it was concluded that flowable slurry with up to 85% fly ash replacement with clean or used foundry can be manufactured without significantly affecting the strength of the reference mixtures. However, for obtaining relatively high strength for ages at 28 days and beyond for the mixtures tested, fly ash replacement with the foundry sands should vary between 30 to 50%.

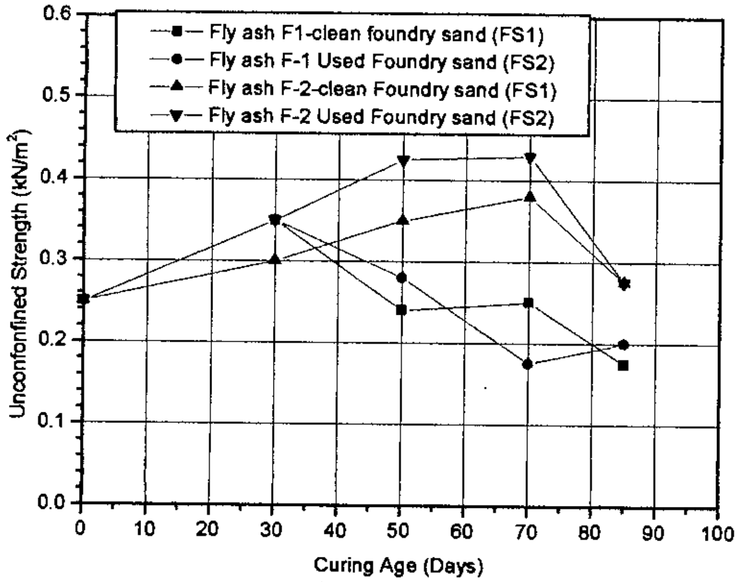

Fig 2.13. Compressive Strength versus Percentage of Foundry Sand at 7days Age. (After Naik, T.R. And Singh, 1997)

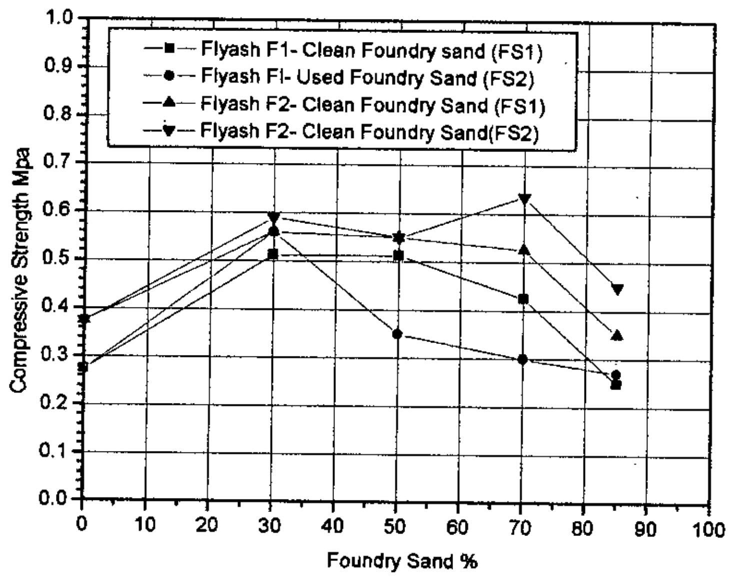

Fig 2.14. Compressive Strength versus Percentage of Foundry Sand at 28 days Age. (After Naik, T.R. And Singh, 1997)

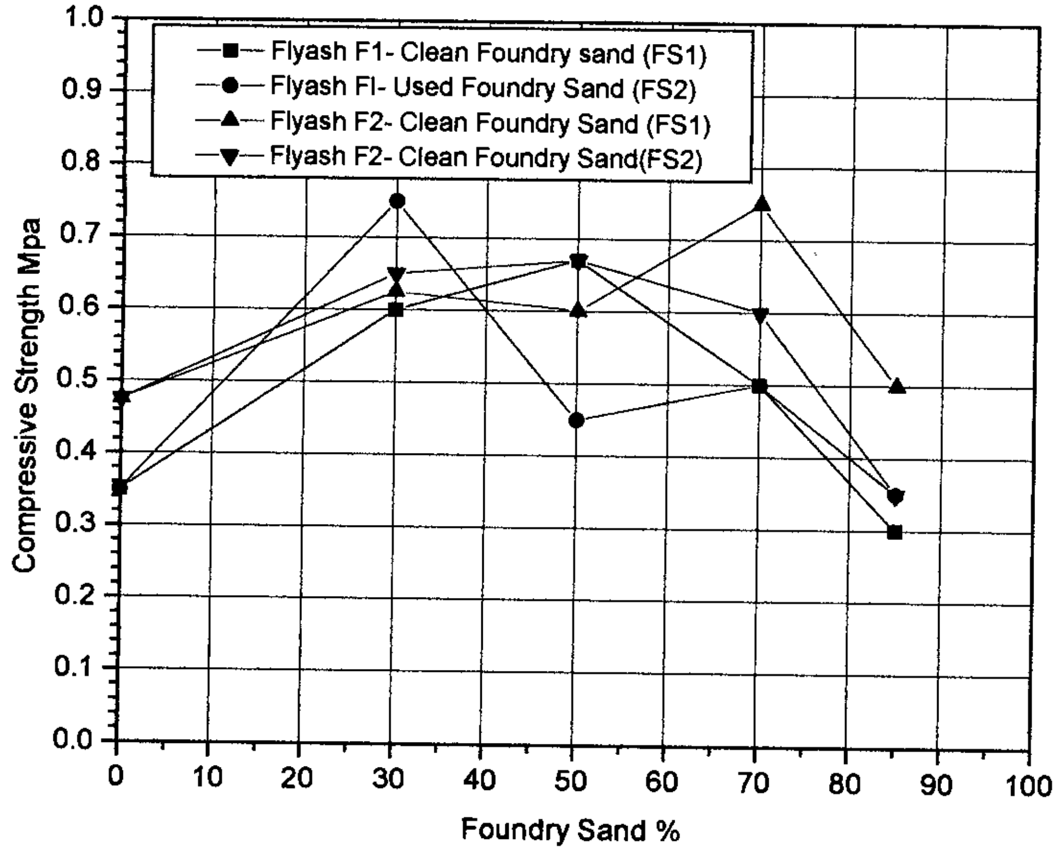

Fig 2.15. Compressive Strength versus Percentage of Foundry Sand at 91days Age. (After Naik, T.R. And Singh, 1997)

C. PERMEABILITY

Test results are presented in Table 2.9 for the fly ash F1 mixtures and in Table 2.10 for the fly ash F2 mixtures. The results are also presented in Figs 2.16 - 2.18. The permeability of the fly ash F1 slurry mixtures varied from 4×10^{-6} cm/s to 72×10^{-6} cm/s, and for fly ash F2 the slurry mixtures varied from both the fly ash mixtures were only slightly affected by the increasing foundry sand content for up to 70% fly ash replacement at the age of 30 days (Figs 16 and 17). The minimum permeability value was observed age 30% fly ash replacement with foundry sand. However, it increased abruptly when the replacement levels for the fly ashes with foundry sand were increased to 85% from 70%. The increase may be attributed to the increase in voids produced by the increase in the amount of foundry sand, and to the decrease in the amount of find particles of the fly ash in the mixture. Additionally, they may be a decrease in grain and pore refinement of the materials due to decreases in the pozzolonic reaction of the fly ash.

There was no significant effect of types of foundry sand (clean or used) on the permeability values of the mixture tested (Table 2.9 and 2.10 and Figs 2.16 and 2.17). The effect of the source of the fly ash on the permeability was also insignificant for the mixtures tested. The effect of the water to cementitious materials ratio [W1(C + FA)] on the permeability of it mixtures is shown in Figs. 2.17 - 2.19. The permeability increase with an increase in the water to cementitious materials ratio. The results show low permeability values up to 70% fly ash replacement with foundry sand in the S1(C + FA) range 0.4-0.8. The permeability values for the mixtures with a without foundry sands were found to vary between 4 x 10 and 72×10^{-6} cm/s. These values are comparable to the observed for granular fill materials, ranging between $10^{-6} \times 10$ and 100×10^{-6} cm/s.

Table 2.9. Permeability of Fly Ash F1 Mixtures with and without Foundry Sand

(After Naik, T.R. And Singh, 1997)

Mixture no (1)	Foundry sand (%) (2)	Fly ash F1 (%) (3)	W1 (C + FA) (4)	C/W ratio (5)	Cement content (%) (6)	Specimen no (7)	Permeability ($x 10^{-6}$ cm/s)	
							Actual (8)	Average (9)
S1	0	100	0.50	0.067	2.2	1	10.6	109
S1	0	100	0.50	0.067	2.2	2	10.7	
S1	0	100	0.50	0.067	2.2	3	11.4	
S2	30 (FSI)	70	0.48	0.098	2.5	1	3.9	4.3
S2	30 (FSI)	70	0.48	0.098	2.5	2	-	
S2	30 (FSI)	70	0.48	0.098	2.5	3	4.6	
S3	50 (FSI)	50	0.52	0.091	1.9	1	8.0	7.5
S3	50 (FSI)	50	0.52	0.091	1.9	2	7.0	
S3	50 (FSI)	50	0.52	0.091	1.9	3	-	
S4	70 (FSI)	30	0.70	0.097	1.7	1	15.6	13.8
S4	70 (FSI)	30	0.70	0.097	1.7	2	12.1	
S4	70 (FSI)	30	0.70	0.097	1.7	3	13.6	
S5	85 (FSI)	15	1.25	0.126	2.4	1	-	74.2
S5	85 (FSI)	15	1.25	0.126	2.4	2	75.1	
S5	85 (FSI)	15	1.25	0.126	2.4	3	73.3	
S6	30 (FS2)	70	0.48	0.098	2.5	1	4.8	5.6
S6	30 (FS2)	70	0.48	0.098	2.5	2	5.8	
S6	30 (FS2)	70	0.48	0.098	2.5	3	6.3	
S7	50 (FS2)	50	0.52	0.091	1.9	1	7.8	7.9
S7	50 (FS2)	50	0.52	0.091	1.9	2	8.0	
S7	50 (FS2)	50	0.52	0.091	1.9	3	-	
S8	70 (FS2)	30	0.70	0.097	1.8	1	11.7	11.1
S8	70 (FS2)	30	0.70	0.097	1.8	2	11.0	
S8	70 (FS2)	30	0.70	0.097	1.8	3	10.5	
S9	85 (FS2)	15	1.25	0.125	2.2	1	73.6	71.9
S9	85 (FS2)	15	1.25	0.125	2.2	2	62.5	
S9	85 (FS2)	15	1.25	0.125	2.2	3	79.5	

Table 2.10. Permeability of Fly Ash F2 Mixtures with and without Foundry Sand

(After Naik, T.R. And Singh, 1997)

Mixture no (1)	Foundry sand (%) (2)	Fly ash F1 (%) (3)	W1 (C + FA) (4)	C/W ratio (5)	Cement content (%) (6)	Specimen no (7)	Permeability ($x 10^{-6}$ cm/s)	
							Actual (8)	Average (9)
P1	0	100	0.78	0.0683	3.0	1	9.2	9.8
P1	0	100	0.78	0.0683	3.0	2	9.9	
P1	0	100	0.78	0.0683	3.0	3	10.3	
P2	30 (FSI)	70	0.67	0.0816	2.6	1	9.1	8.3
P2	30 (FSI)	70	0.67	0.0816	2.6	2	7.2	
P2	30 (FSI)	70	0.67	0.0816	2.6	3	8.6	
P3	50 (FSI)	50	0.75	0.0855	2.4	1	10.1	11.6
P3	50 (FSI)	50	0.75	0.0855	2.4	2	12.8	
P3	50 (FSI)	50	0.75	0.0855	2.4	3	11.9	
P4	70 (FSI)	30	0.60	0156	2.5	1	15.6	13.4
P4	70 (FSI)	30	0.60	0156	2.5	2	11.2	
P4	70 (FSI)	30	0.60	0156	2.5	3	-	
P5	85 (FSI)	15	1.12	0.135	2.1	1	65.9	69.3
P5	85 (FSI)	15	1.12	0.135	2.1	2	72.7	
P5	85 (FSI)	15	1.12	0.135	2.1	3	-	
P6	30 (FS2)	70	0.42	0.130	2.7	1	4.4	4.8
P6	30 (FS2)	70	0.42	0.130	2.7	2	4.9	
P6	30 (FS2)	70	0.42	0.130	2.7	3	5.1	
P7	50 (FS2)	50	0.64	0.100	2.5	1	7.0	6.5
P7	50 (FS2)	50	0.64	0.100	2.5	2	6.0	
P7	50 (FS2)	50	0.64	0.100	2.5	3	-	
P8	70 (FS2)	30	0.67	0.130	2.2	1	13.1	12.6
P8	70 (FS2)	30	0.67	0.130	2.2	2	12.2	
P8	70 (FS2)	30	0.67	0.130	2.2	3	12.5	
P9	85 (FS2)	15	1.05	0.145	2.1	1	66.3	64.9
P9	85 (FS2)	15	1.05	0.145	2.1	2	60.9	
P9	85 (FS2)	15	1.05	0.145	2.1	3	67.5	

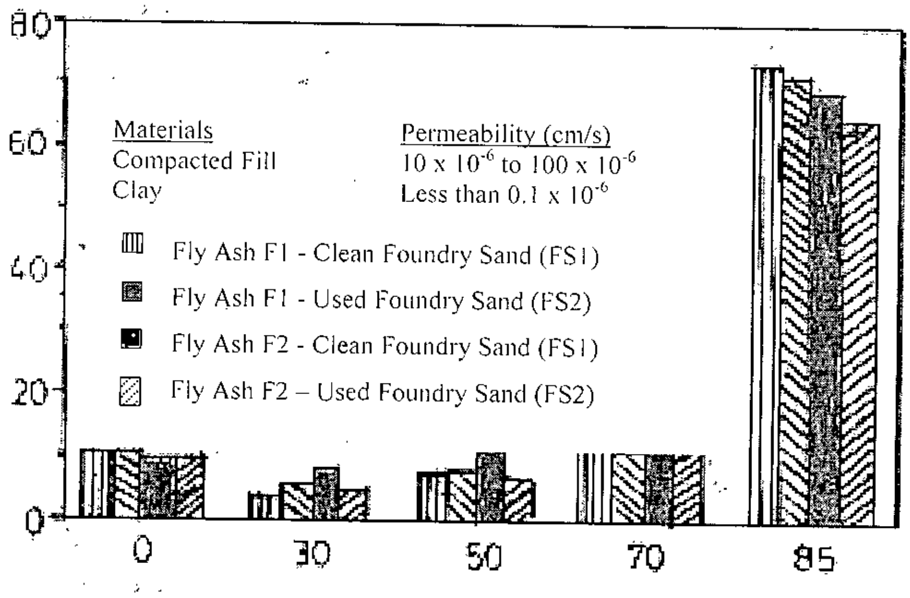

Fig 2.16. Permeability of Slurry Mixtures as Function of Four dry Sand Content. (After Naik, T.R. And Singh, 1997)

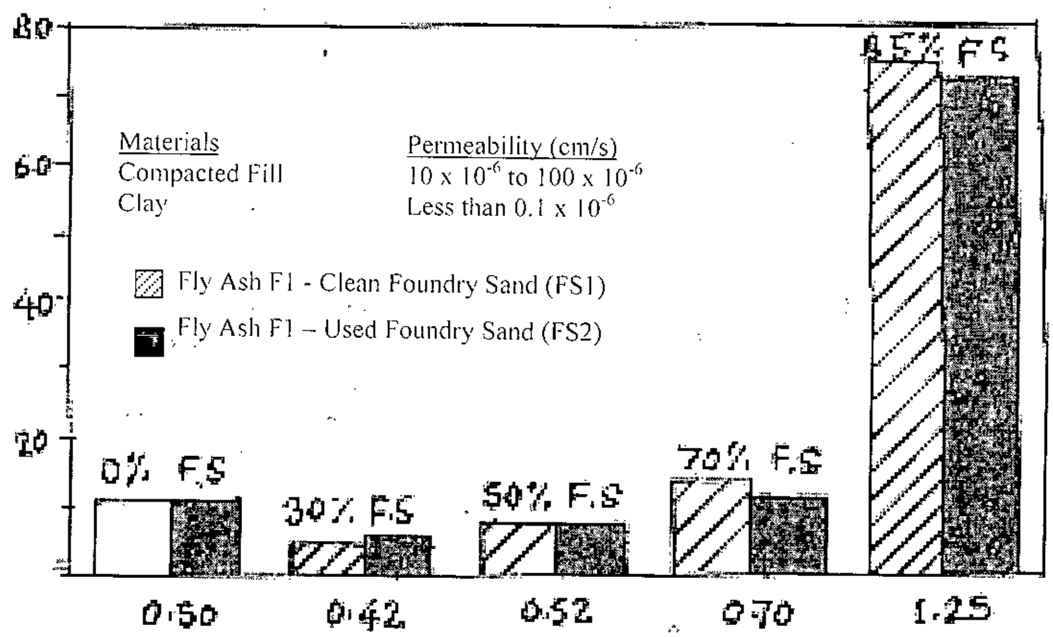

Fig 2.17 Permeability versus Water to Cementitious Materials Ratio for Fly Ash F1 Slurry Mixtures with and without Foundry. (After Naik, T.R. And Singh, 1997)

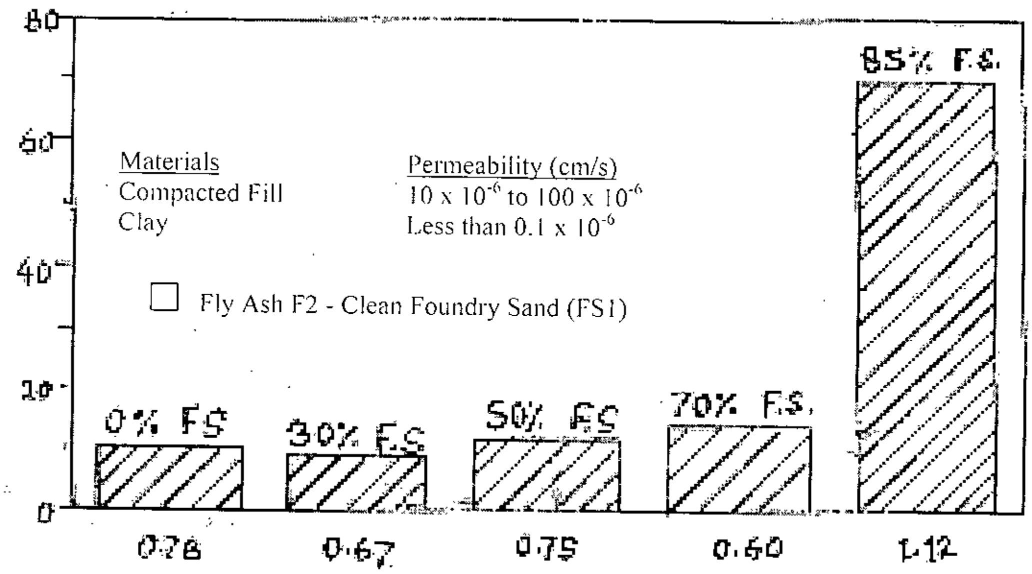

Fig 2.18. Permeability versus Water to Cementitious Materials Ratio for Fly Ash F2 Slurry Mixtures Containing Clean Foundry. (After Naik, T.R. And Singh, 1997)

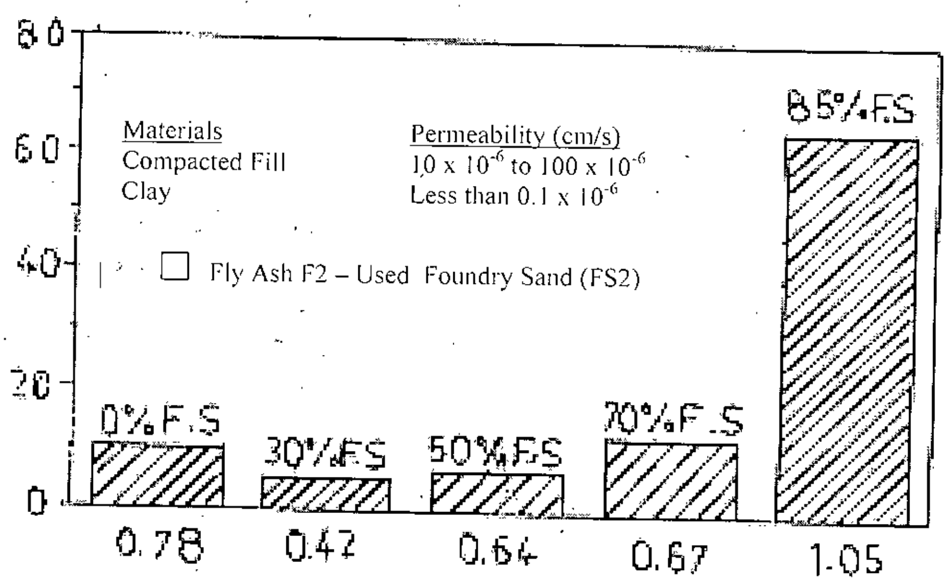

Fig 2.19. Permeability versus Water to Cementitious Materials Ratio for Fly Ash F2 Slurry Mixtures Containing Used Foundry Sand (FS2). (After Naik, T.R. And Singh, 1997)

2.2.5.7. Finally the following major conclusions were drawn from the basis of the work completed in his investigation.

a. Flowable slurry materials can be manufactured using foundry sand as a replacement of fly ash up to 85% for strength levels in the range of 0.27 - 0.61 MPa (40-90 psi) at 28d.

b. The two sources of fly ash used in this study produced the same effect on the permeability. The same was also true for the two types of foundry sands used.

c. The minimum permeability was observed at 30% fly ash replacement with foundry sand. However, the permeabilities of the test mixtures were not significantly influenced by the inclusion of foundry sand up to 70% fly ash replacement.

d. At 85% replacement of foundry sand, a sharp increase in permeability was observed.

e. The permeability values of the mixtures were low for the water to cementitious materials ratio values ranging between 0.4 and 0.8. Above the water to cementitious materials ratio of 0.8, the permeability values increased rapidly for mixtures tested in the investigation.

2.3 LIME FLYASH-SOIL MIXTURE

Nettleton (1962) showed improvement of the strength properties of residual clay using flyash as sole additive. However, he further observed that the additional of a small amount of lime in flyash-soil mixtures further improved the strength of the stabilized soil. Subsequently. many authors have indicated the use of flyash as an additive in lime-soil mixtures to achieve better strength of the mixture. However, there exist about little literature describing the use of flyash a lesson soil stabilization.

In mechanism of lime, flyash, and soil in stabilization as describe by Chu et al (1955) as follows: Flyash is a gray, dust like ash which results from burning powered coal. The coal is burned while in suspension in air, and the resulting ash consists largely of tiny spheres of silica and alumina glass. The ash is similar to volcanic ash used in early Roma construction. It is a pozzolanic material; at is, it is not itself a cement, but it reacts with lime and water to form cementitious material. However, it is the reaction of lime and flyash which is utilized to stabilize soils. After mixing the proper proportion the mixtures in a moist, non plastic state, but it can be readily compacted to form a dense mass.

Leonard and Davidson,(1959) reported that because of the slow reaction of lime absorption, the development of the slow reaction of lime absorption, the development of compressive strength of a soil, lime and flyash mixture is slow. Therefore, the rate of development of compressive strength of lime-layash reaction is directly related to the rate of lime absorption by the flyash. The rate of lime absorption is limited by the rate of diffusion of the calcium through the reaction product.

Gray and Lin (1972) stated that flyash exhibits age hardening, or pozzolanic, properties. This pozzolanic behavior tends to limit the extent of actual field settlement in the long run. Partial saturation also accounts for a considerable difference of behaviour in compression.

Pozzolanic materials are defined as siliceous, or siliceous and aluminous materialy which themselves exhibit little or no cementitious value, but will, in the presence of moisture, chemically react with calcium hydrulic ordinary temperatures to form compounds possessing cementitious properties.

One of the main questions in soil, lime, and flyash stabilization is how much lime and flyash are needed. The amount and proportions of the lime and flyash admixtures are governed by the desired strength in the stabilized soil and by economy.

Mateos and Davidson (1962) stated that there is no optimum amount, nor optimum ratio, of lime and flyash for stabilizing all soils. The amounts of lime and flyash to be used depend greatly on the kinds of flyash and soil, and some what on the kind of lime. The authors found that the amount of hydrated lime for granular soils should be from 3 to 6 percent, and the amount of flyash between 10 and 25 percent. For clay soils, the amount of lime should be between 5 and 9 percent, and the amount of flyash between 10 and 25 percent. At low lime contents (about 3 percent) calcitic hydrated lime was found to be more efficient than dolomitic monohydrated lime for stabilizing clayey soils, with or without flyash. At higher lime contents, dolomitic monohydrated lime gave better strength than calcitic hydrated lime.

Minnick and Miller (1950) found that the coarser the material to the stabilized with lime and flyash, the higher the volume of flysh that is required.

Mainfort (1955) stated that several inorganic cementing materials are capable of hardening or otherwise modifying the physical characteristics of soils. However, inorganic cementing agents are particularly susceptible to moisture attack during conditions of freezing and thawing.

VISCOCHIL et al (1958) have shown that the density of soil, lime, and flyash mixtures is dependent on the compactive effort applied, but the density also depends on the lime/flyash ratio. The density is decreased by higher contents of lime because of two factors : Lime itself is less dense than soil or flyash, and lime cause aggregation of clay. The authors further stated that the unconfined compressive strength is primarily influenced by cementation and does not give a true measure of the frictional strength developed in a confined state. Therefore, a stabilized granular material with relatively low unconfined compressive strength may show satisfactory stability.

2.3.1 FACTOR INFLUENCING LIME- FLYASH STABILIZATION

The stability of lime-flyash-soil mixtures is affected by many variables. According to CHU et al (1955), the following are main factors which affect the stability of a stabilized soil.

a. **Properties of Soil**: Cohesive soil possesses properties which are different to those of cohesionless soil. The stability a lime-flyash-soil mixture is thus greatly influenced the kind of soil. Montmorillonitic soil is more affected by reaction to lime and flyash than illitac soil.

b. Amount and ratio of lime and flyash in the mixture the amount of additional or variations in the ratio of lime to flyash considerably affect the strength of a stabilized soil. For a given ratio of lime to flyash, increasing the amount of the additives increases the compressive strength of the stabilized soil. The effect may not be great for some soils when the variation is within a certain range.

c. **Properties of Lime and Flyash** Depending on the source of raw material contents vary considerably from sources to sources therefore the compressive strength is affected by the source of a flyash.

d. Aging of lime greatly influences the compressive strength of lime, flyash, and soil mixtures. Increased amounts of carbonates contained in lime, due to aging to aging, reduce the reactivity of lime and hence deduce the unconfined compressive strength of the mixture of lime, flyash, and soil.

e. Moisture contents of mixture : The unconfined compressive strength of lime and flyash stabilized soil is greatly affected by the moisture content of the mixture at the time of mixing. The optimum moisture content of a lime and flyash stabilized soil increases slightly during the curing period. Thus, it is better to compact the mixture on the wet side to optimum moisture content as found from the proctor compaction curve.

f. Method and degree of compaction : The unconfined compressive strength of lime-flyash-soil mixture is remarkably affected by the method and degree of compaction.

g. Length of curing period : The compressive strength of mixtures of lime, flyash, and soil increases with an increase in the duration of curing. Experiments have shown that a fairly long curing period, perhaps 28 days, is desirable before evaluating the stability of mixtures of lime, flyash, and soil.

h. Condition during curing : Elevated temperatures result in much higher compressive strengths than those obtained by curing at about $22 \pm 3C$ (Chu et al, 1955). However, higher temperatures may not be attainable in the field under conventional methods of curing. The effect of the relative humidity during curing on compressive strength is not so distinct. Maximum compressive strength is achieved with relative humidity lower than 100 percent. However, lower than 90 percent of relative humidity is not recommended (CHU et al, 1955).

2.4. LIME-SOIL MIXTURE

2.4.1. LIME STABILISATION

Stabilization of soils with lime is similar to cement stabilization in that similar criteria and testing and construction techniques are employed. There are however, significant differences in the nature and rate of the cementitious reactions and these often permit a clear basis of choice between cement and lime.

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.

2.4.2 PREVIOUS WORK AND CONCLUSIVE RESULTS

Considerable research has been conducted by numerous investigators to determine the suitability of the various forms of lime as a soil stabilizing agent. The physical properties of plastic soils are considerably modified by the addition of small quantities of lime. The plasticity is reduced, and the soil become more friable, hence easier to mix and mold to uniform density.

Glin and Handy (1963) stressed that hydrated lime added to clayey soils caused to beneficial modifications: (a) Rapid depression of the plasticity index, and (b) Long-term cementation attributable to chemical reactions between the lime and siliceous minerals, or glasses.

Mateos (1964) stated that quick lime lowers the plasticity of a soil more than an equivalent amount of hydrated lime. Quick lime was also more effective in improving the shrinkage properties of a soil than hydrated lime.

Diamond and Kinter (1965) described the response of soil to treatment with lime as complex and often dramatic. They further stated that various explanations have been proposed to account for these responses, including:

a. Cation exchange i.e, replacement by calcium cations (derived from lime) of the exchangeable sodium, magnesium, or other cations previously held by soil:

b. Flocculation of clay, and a consequent increase in effective grain size:

c. Carbonation, i.e., reaction of lime with carbon dioxide from the atmosphere to form calcium carbonate, which can exert cementing action; and

d. Pozzolanic reaction with soil constituents to generate new minerals of a cememtitious natures.

Herzog and Mitoeil (1963) described the responses of lime differently in that the reactions taking place between clay minerals and lime may be divided into two distinct types those which are completed rapidly (ion exchange and flocculation) and reactions which proceed slowly (carbonation, pozzolanic reactions, and the formation of new materials).

The authors further explained that the addition of lime to clays causes floculation because of the increased electrolyte content of therefore water and also a result of ion exchange the clay to the calcium form.

Although floculation and ion exchange may be completed in a new days, the slower reaction producting cementitious material in Lime-clay mixtures may be formed by carbonation and by chemical reactions between clay contents and lime. Carbonation is normally confined to surfaced exposed to air and involves the conversion of lime to calcium carbonate by carbon dioxide absorbed from the air. Calcium carbonate cements soil particles together and enhances their stability.

Clay minerals and some other soil components possess pozzolanic properties. The addition of lime to soil causes an instantaneous rise in the pH of the molding water due to solution and dissociation of the Ca (OH)2. The high pH increases the reactivity of surface silica and alumina. The calcium ion combine with the reactive hydrous silica and alumina and from gradually hardening cementitious material.

The cation exchange capacity depends very much on the pH of the soil water and on the type of clay mineral in the soil. Among the types of clay minerals, montmorillonite has the highest, and the kaolinite has the lowest, cation exchange capacity.

Eades and Grin (1960) stated that the quantity of lime needed to treat effectively a clay mineral is dependent on the type of mineral present.

Grim (1953) stated that the general order of replaceability of the common cations associated with soils is given by the lyotropic series, $Na^+ < K^+ < Ca^{++} < Mg^{++}$ cations tend to replace cations to the left in the series and monovalent cations are usually replaceable by multivalent cations.

The addition of lime and soil supplies an excess of Ca and cation exchange will ouuur, with Ca^{++} replacing dissimilar cations from the exchange complex of the soil. In some cases, the exchange complex is practically Ca saturated before the lime addition and cation exchange does not occur. or is minimized.

Herrin and Mitchell (1961) showed that there is a rapid increase in strength of soil-lime mixtures at the beginning increase becomes less and less. After considerable curing time, the strength appears to be still increasing, but very slightly.

Ladd et al (1960) reported that lime reduces the compacted density of soils and increases the optimum water content. Lime increases the soaked strength of soils after humid curing periods, but the effectiveness, of lime treatment varies considerably with soil type.

The less plastic soils, such as silts and organic soils, are often less responsive to lime than soils of increased plasticity, such as clays.

The maximum soaked strength of lime-stabilized soils usually occurs at optimum water content for compaction, except for plastic or organic clays where the maximum strength may occur on the wet side of the optimum moisture content:

2.4.3 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.

2.4.3.1 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. Table 2.11 summarizes the properties of hydrated, quick and slurry lime.

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 2.11 Properties of lime (after NAASRA, 1986)

Parameters	Hydrated Lime	Quick Lime	Slurry Lime
Composition	$Ca(OH)_2$	CaO	$Ca(OH)_2$
Form	Fine Powder	Granular	Slurry
Equivalent $Ca(OH)_2$/Unit Mass	1.00	1.32	0.56 to 0.33
Bulk Density (kg/m^3)	450 to 560	1050	1250

2.4.3.2 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.

2.4.3.3 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.

2.4.4 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.

2.4.4.1 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.

2.4.4.2 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:

$$Ca^{++} + 2(OH)^- + SiO_2\,(Soil\ Silica) \Rightarrow CSH_I \qquad (2.1)$$

$$Ca^{++} + 2(OH)^- + Al_2O_3\,(Soil\ alumina) \Rightarrow CAH_I \qquad (2.2)$$

In equations 2.1 and 2.2, CSH and CAH are cementitious products. The above reactions represented by Equations 2.1 and 2.2 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 will increase strength only up to the point where all the silica of the clay is used up; adding too much lime can actually be counterproductive. This contrasts with cement 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.

2.4.4.3 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.

2.4.5 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.

2.4.5.1 SOIL CHARACTERISTICS

2.4.5.1.1 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. 2.20 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. 2.21. It has been found that silty soil has much lower unconfined compressive strength than the clay types of soil.

The P^H 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 P^H 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.

2.4.5.1.2 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. 2.22.

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.

2.4.5.2 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.

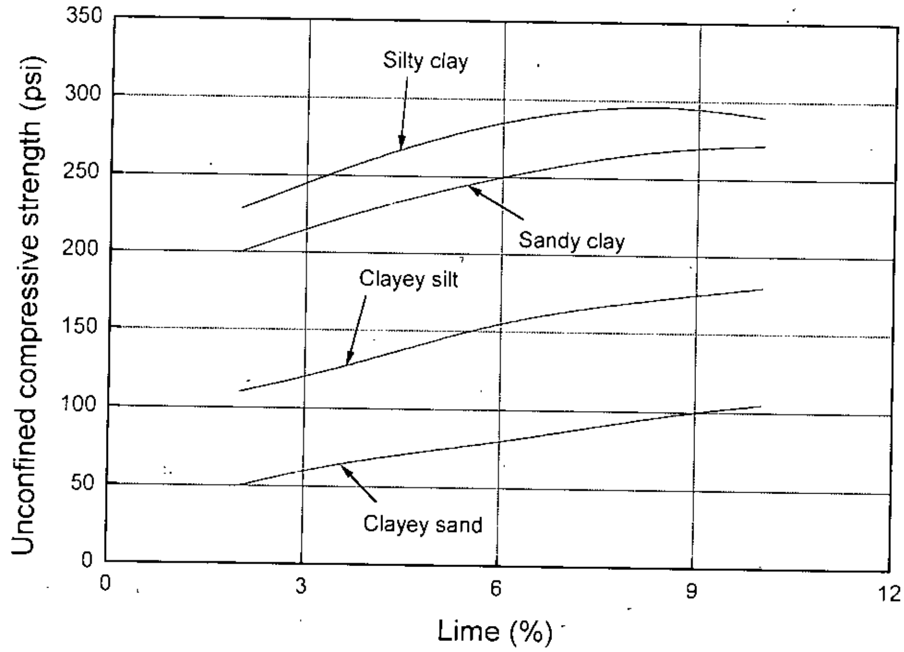

Fig. 2.20 Variation of unconfined compressive strength (qu) with lime content for various types of soil (after Locat et al., 1990)

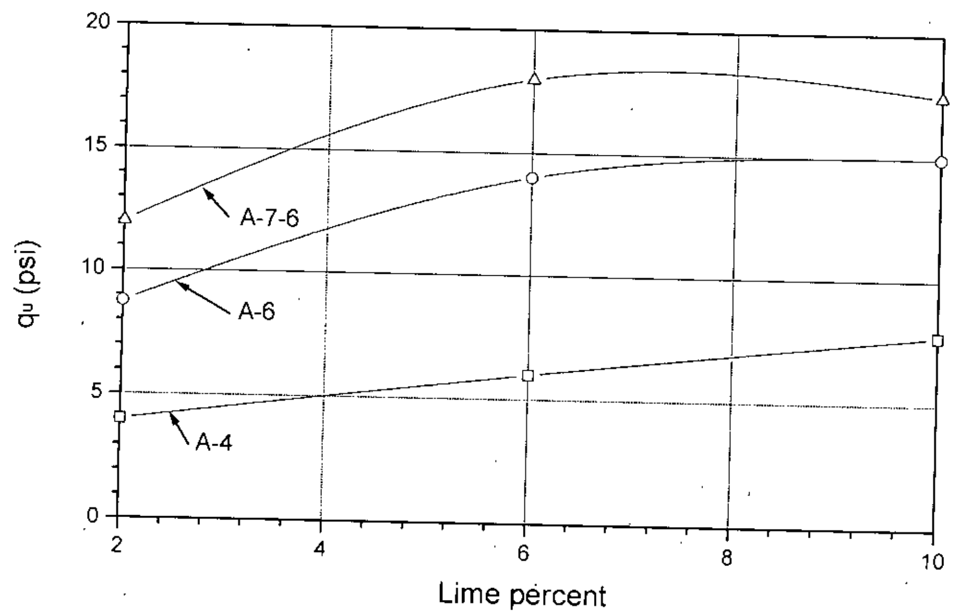

Fig. 2.21 Variation of unconfined compressive strength with lime content for different types of soil (after Serajuddin, 1992)

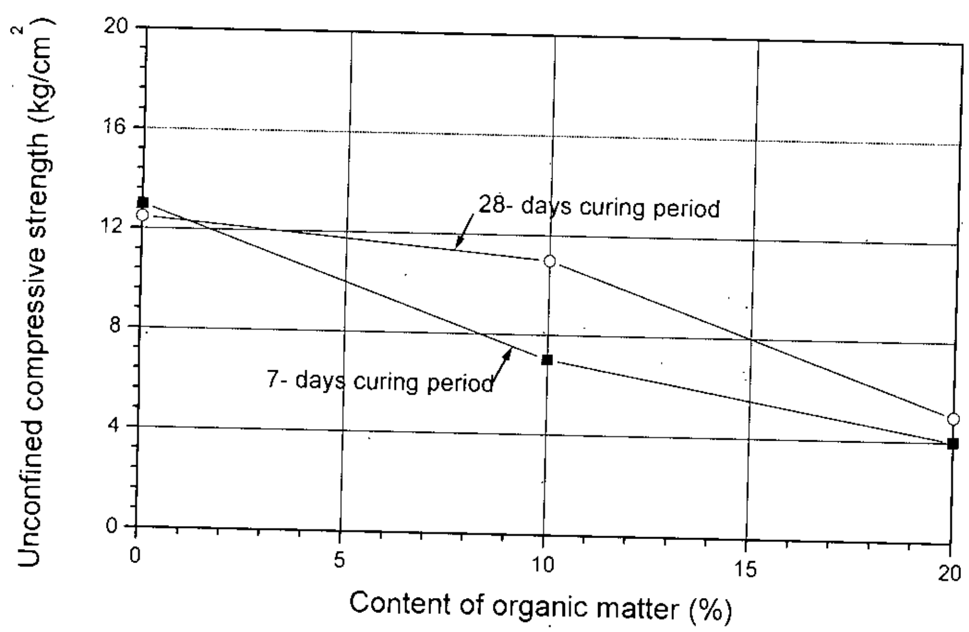

Fig. 2.22 Effect of organic matter on unconfined compressive strength of lime treated soil (after Arman and Muhfakh, 1972)

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. 2.23 for three regional soils of Bangladesh. It can be seen from Fig. 2.23 that the unconfined compressive strength of the lime stabilized soil increase with the increase.of lime content for all the three soil types. In another investigation of soil of Dhaka, Abid (2002) also found that unconfined compressive strength increases with increase of lime content (Fig-2.24).

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.

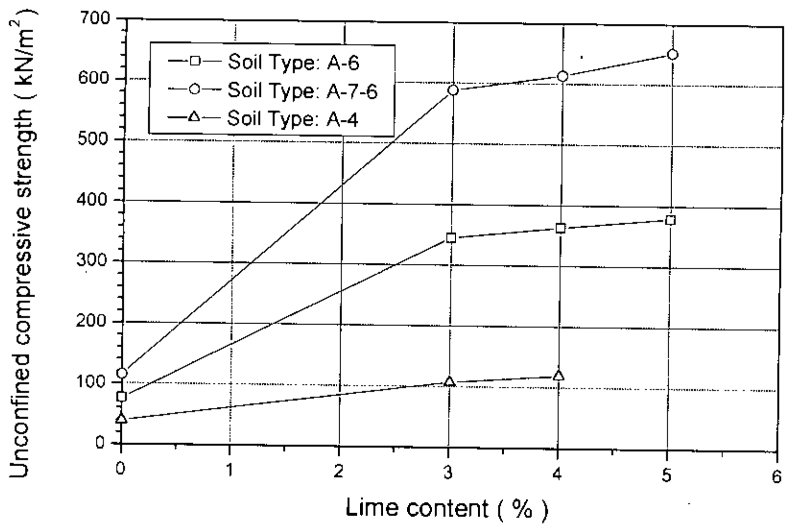

Fig. 2.23 Variation of the unconfined compressive strength of lime stabilized soil due to variation of lime content (after Molla, 1997)

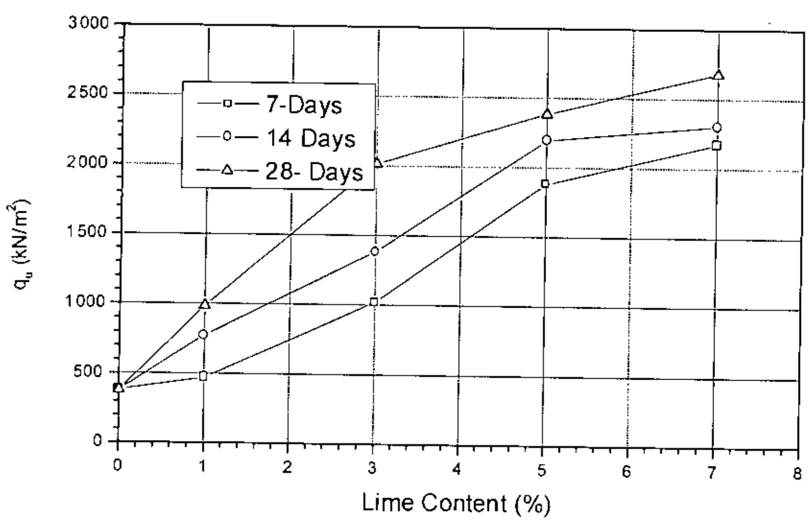

Fig. 2.24 Effect of lime content on unconfined compressive strength of lime treated soil of Dhaka (After Hassan 2002).

2.4.5.3 MIXING AND COMPACTION PROCEDURE

2.4.5.3.1 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, Iw = 9 - 26) of Bangladesh. Molla (1997) reported that unconfined compressive strength increases with the increase in compaction energy as shown in Fig. 2.25.

2.4.5.3.2 COMPACTION DELAY TIME

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

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

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

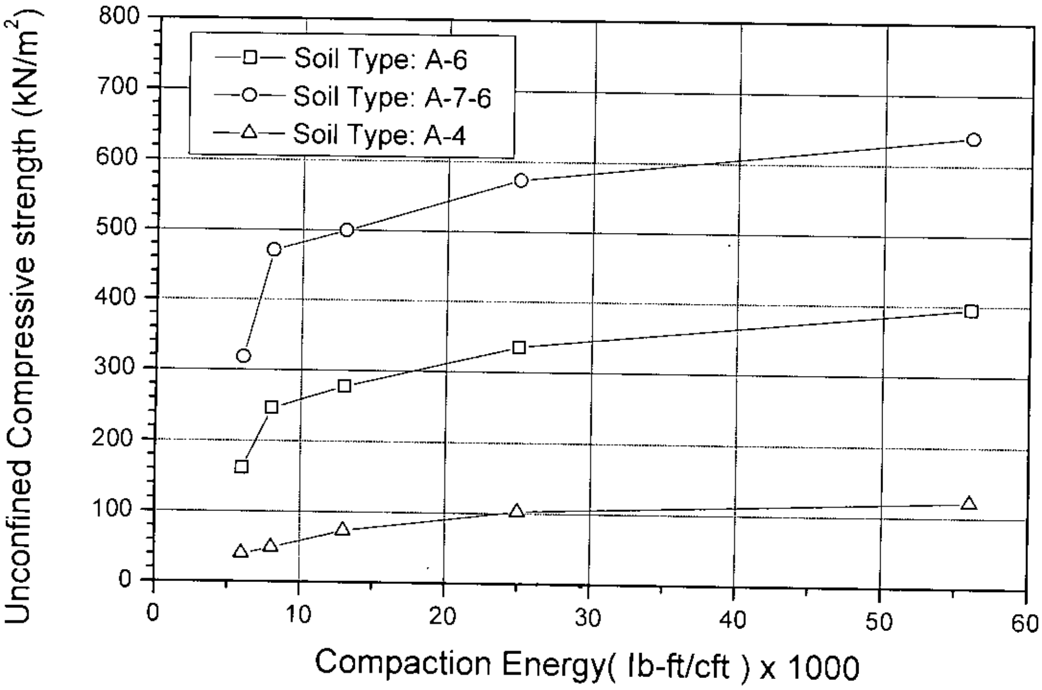

Fig. 2.25 Variation of unconfined compressive strength (qu) at different compactive effort for stabilized soils using 3% lime (after Molla, 1997)

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

Molla (1997) investigated the effect of compaction delay time on unconfined compressive strength of three regional soils of Bangladesh. Molla (1997) reported that unconfined compressive strength decreases with the increase in compaction delay time. This trend is presented in Fig. 2.26 for two soils which found by Shahjahan 2001.

2.4.5.4 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.

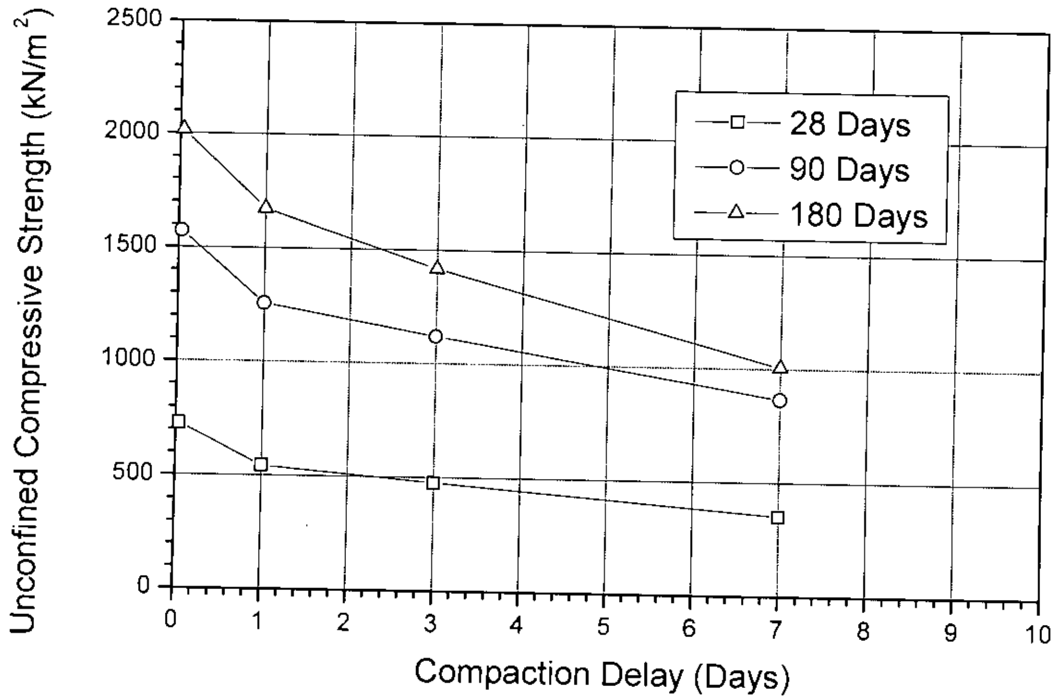

Fig. 2.26 Variation of unconfined compressive strength (q_u) with compaction delay time Soil type-ML/CL (after Shahjahan, 2001)

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. 2.27. From Fig. 2.27, 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. 2.28. It can be seen from Fig. 2.28 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.

Hassan (2002) recommanded that degree of strength gain resulted due to increasing lime content and curing age of AminBazar and Basundara, Dhaka soil. Fig 2.29 and 2.30 shows relationship between qu and curing period and strength development index increase with increasing curing age and lime content.

2.4.6 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.

2.4.6.1 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).

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. 2.31. 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. 2.31.

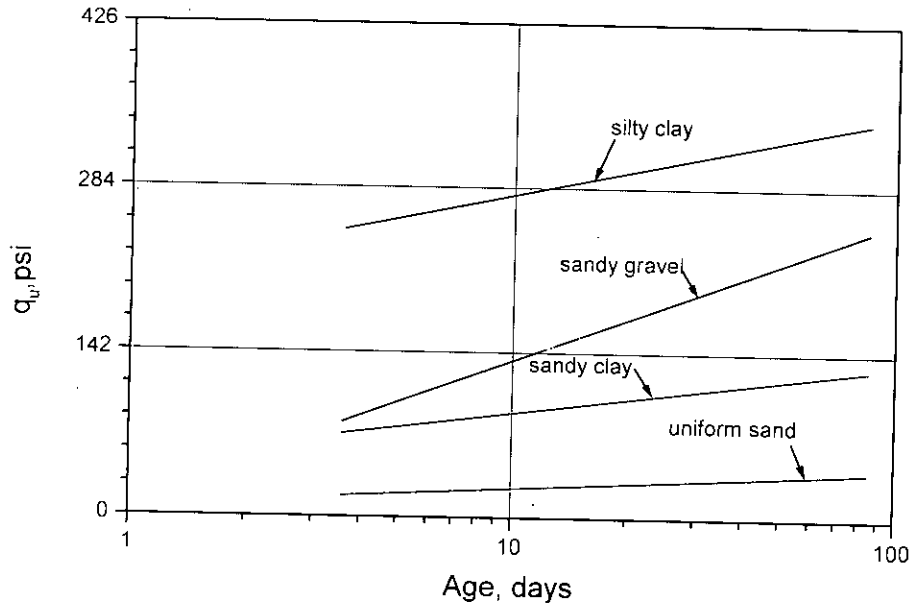

Fig. 2.27 Effect of curing age on unconfined compressive strength (q_u) for various types of soils stabilized with 5% lime (after Ingles and Metcalf, 1972)

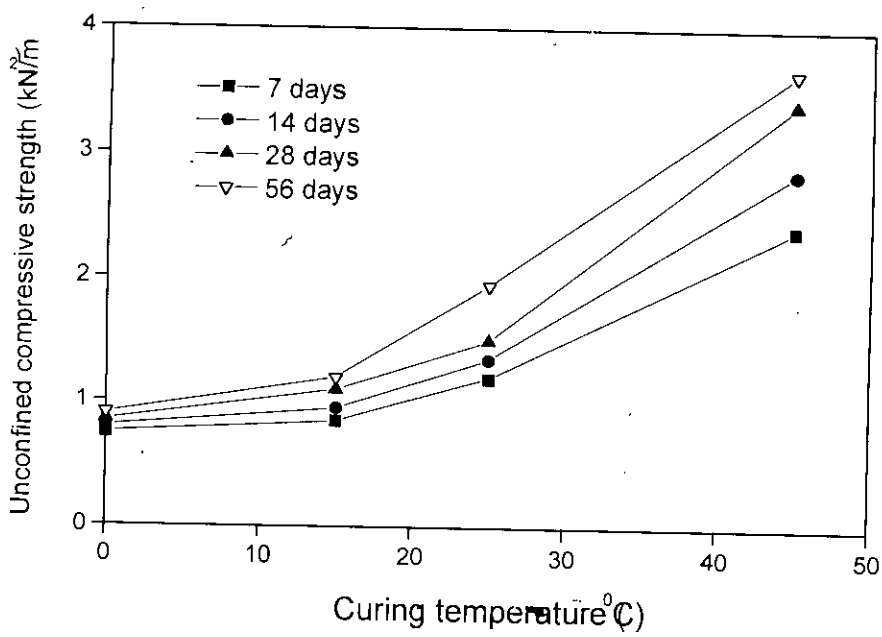

Fig. 2.28 Effect of curing temperature and curing age on unconfined compressive strength of a clay of high plasticity stabilized with 5% lime (after Bell, 1988)

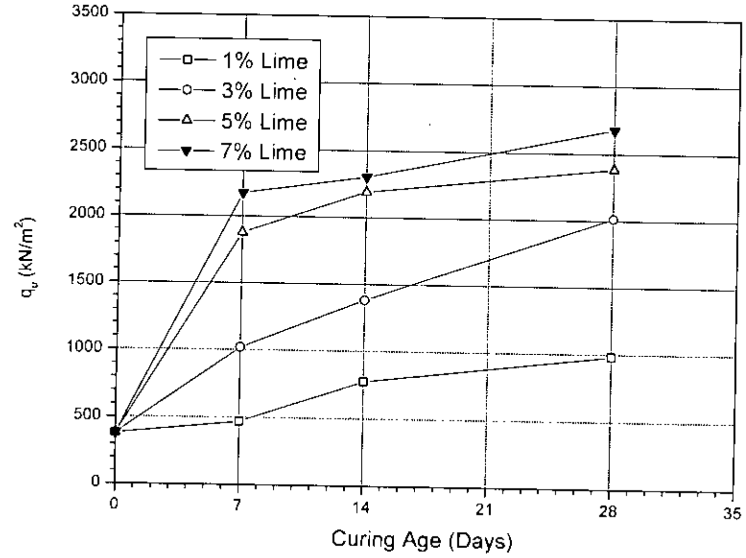

Fig. 2.29 Effect of curing age on unconfined compression strength of lime-treated soil (After Hassan, 2002)

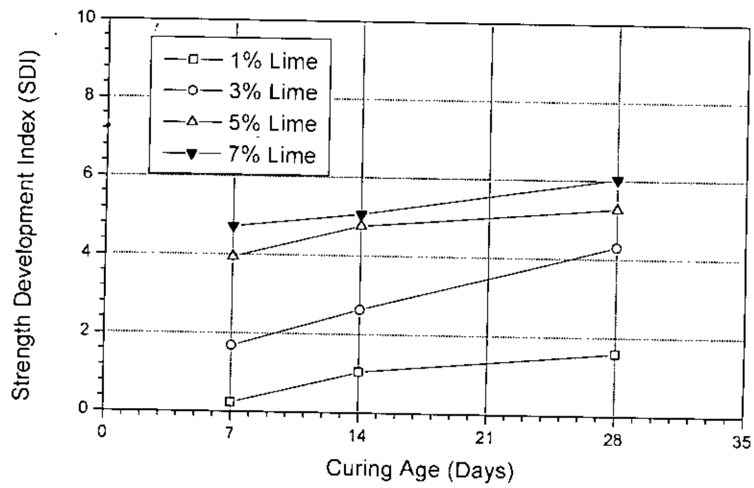

Fig. 2.30 SDI versus curing age curves of samples of lime-treated soil (After Hassan, 2002)

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 (1986), 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, PL = 12 and 20) of Bangladesh. Hossain (2001) 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, PL = 19) of Bangladesh. Hossain (2001) 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. 2.32. 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. 2.33. It can be seen from Fig. 2.33 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. Hasan (2002) also found plastic limit and shrikage limit increases with increasing lime content while liqurd limit and plasticity index reduced with increase in lime content. Fig 2.34 shows the results of Hasan (2002) analysis.

2.4.6.2 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. 2.35. 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. 2.36. 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.

2.4.6.3 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. 2.37, 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 (1986) also found an increase in unconfined compressive strength with the increase in lime content and curing age lime for two regional soils of Bangladesh.

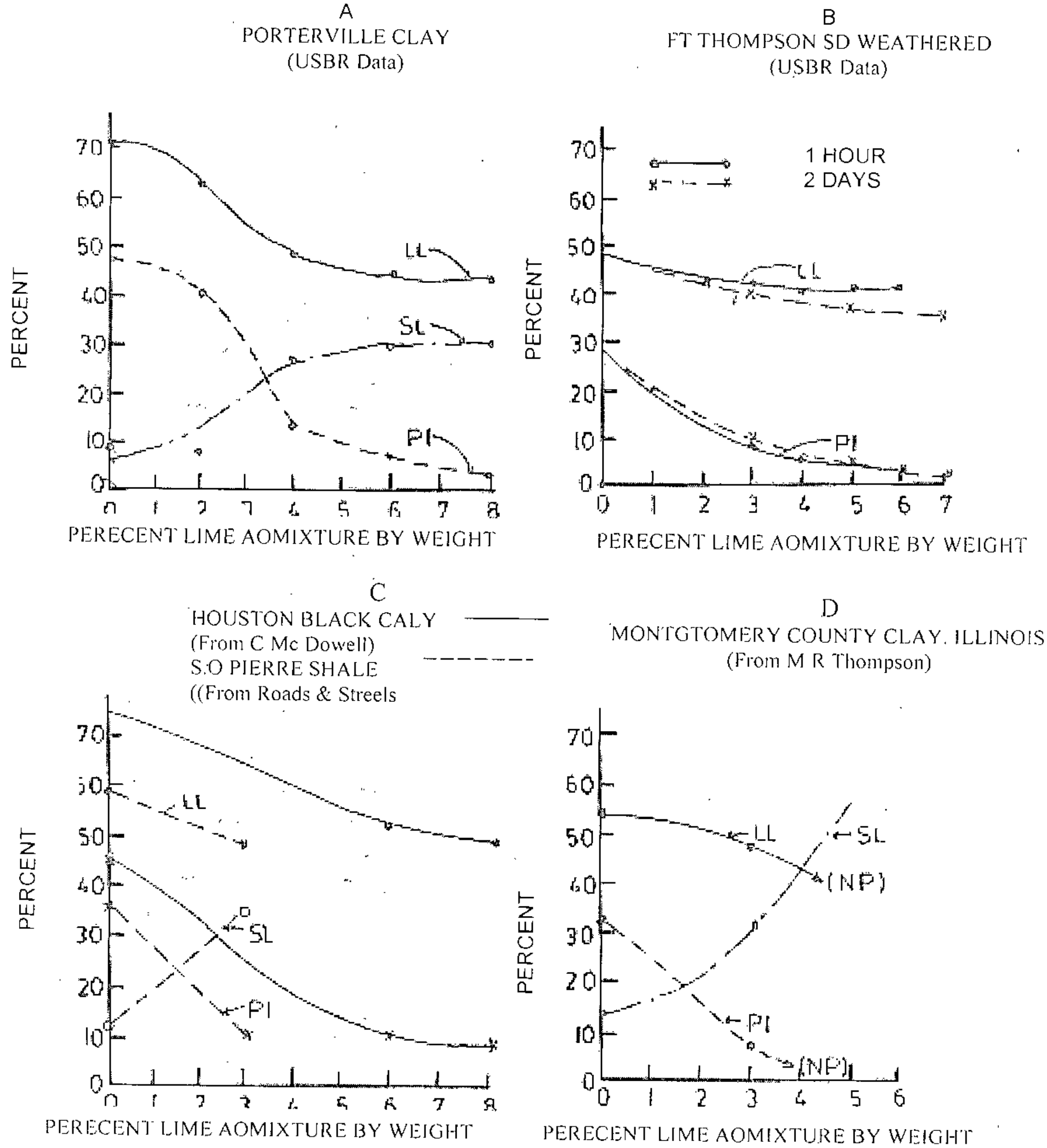

Fig. 2.31 Effect of lime on plastic characteristics of expansive montmorillonite clays (after Holtz, 1969)

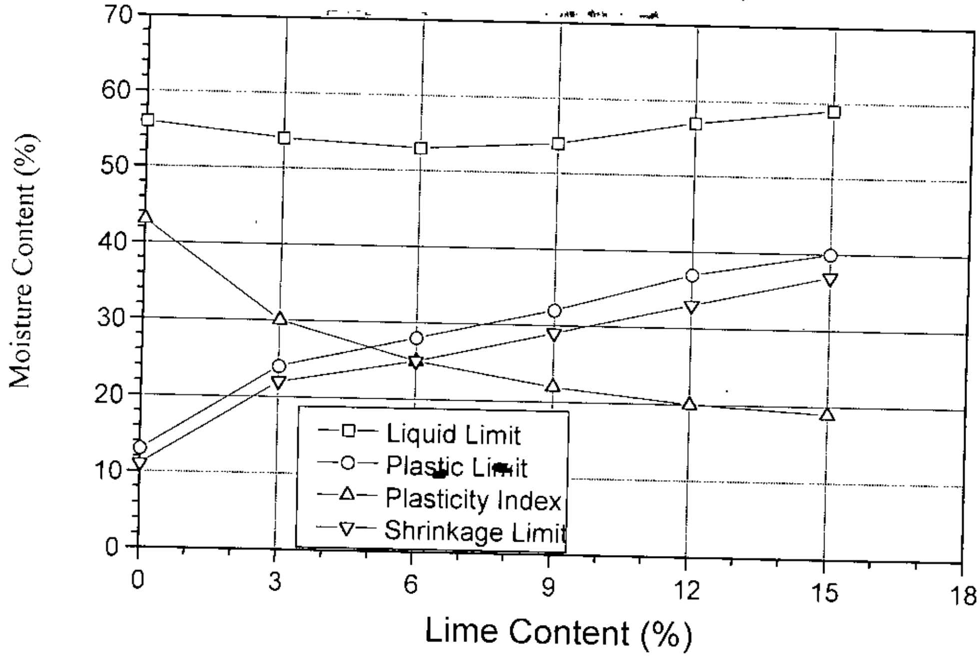

Fig. 2.32 Effect of lime content on Atterberg limits and shrinkage limit of a expansive soil (after Hossain, 2001)

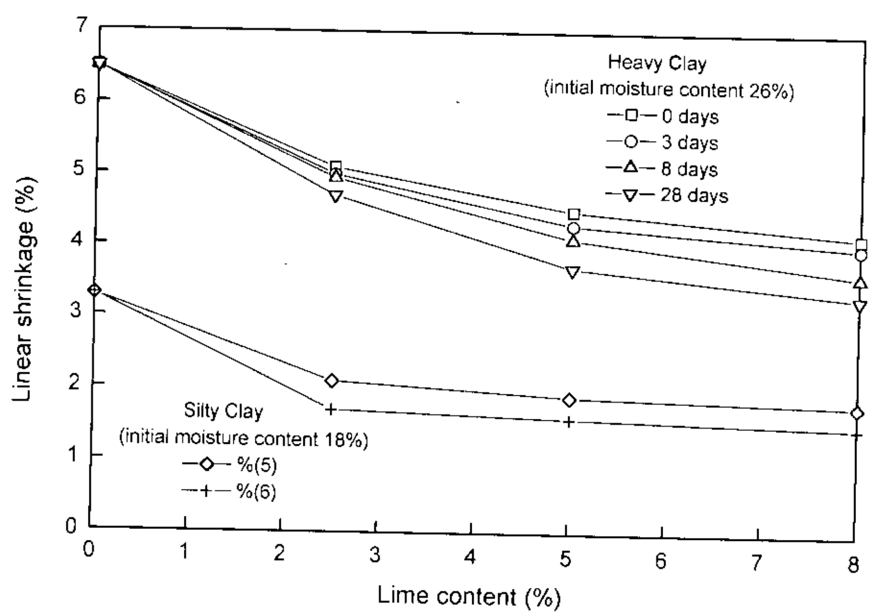

Fig. 2.33 Effect of lime content on linear shrinkage of clays (after Bell, 1988)

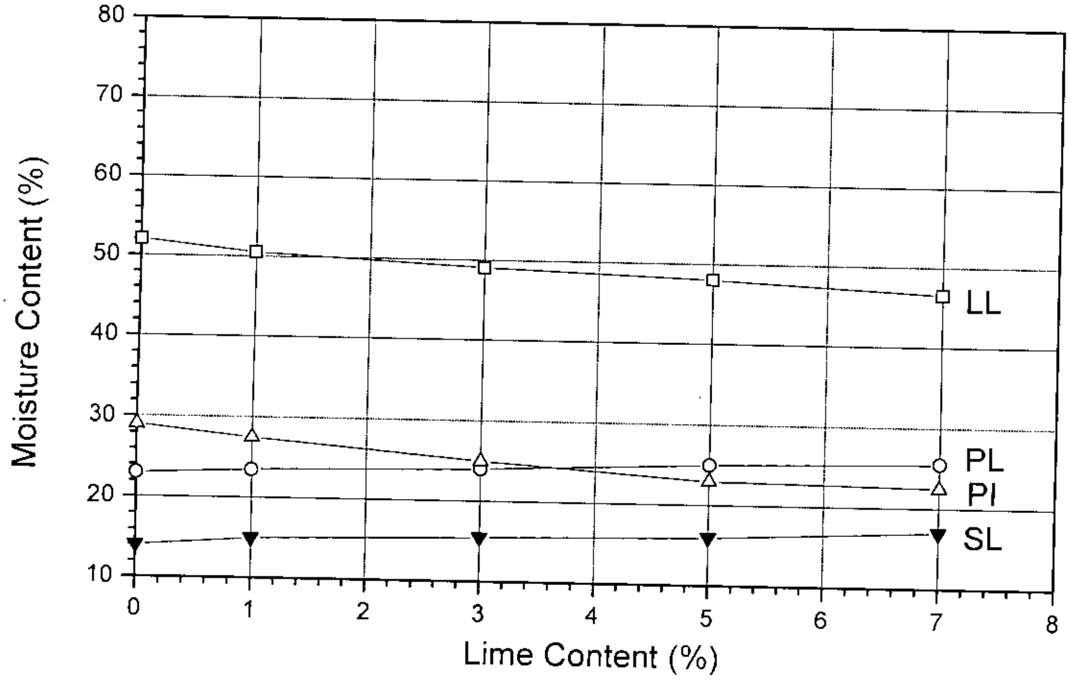

Fig: 2.34 Effect of lime content on Atterberg limits and shrinkage limit of soil (After Hasan, 2002).

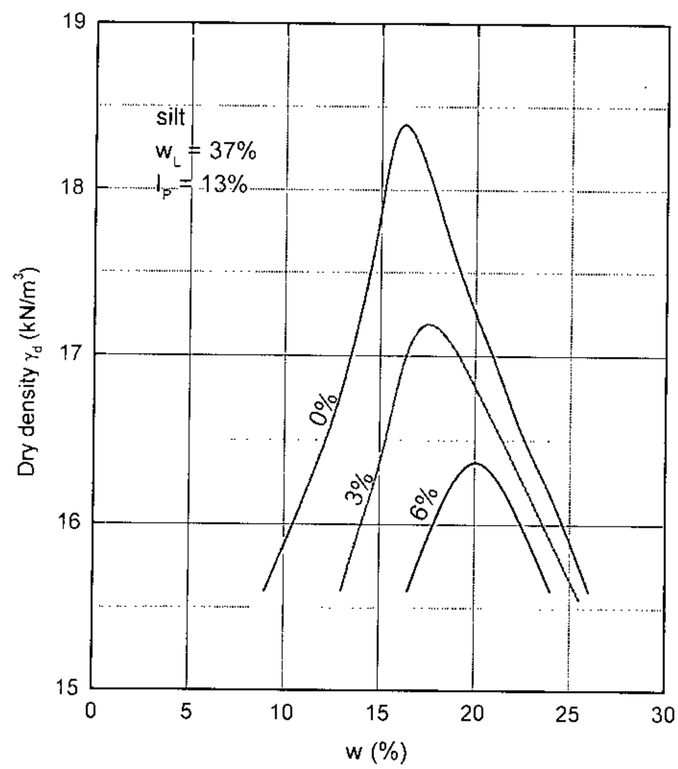

Fig. 2.35 Effect of lime content on maximum dry density and optimum moisture content of a lime-treated silt (after Kezdi, 1979)

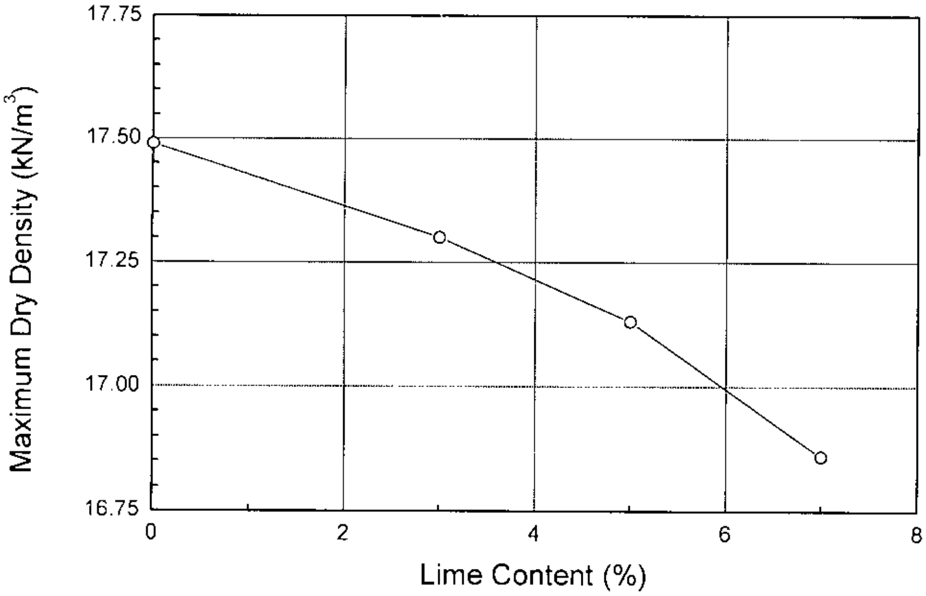

Fig. 2.36 Effect of lime content on maximum dry density of a lime-treated coastal soil (after Rajbongshi, 1997)

Rajbongshi (2001) 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. 2.38. Molla (1997) also found that unconfined compressive strength of lime-treated samples increased with the increase in lime content and curing age for three regional soils of Bangladesh.

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

$$SDI = \frac{\textit{Strength of stabilised sample - Strength of untreated sample}}{\textit{Strength of untreated sample}} \qquad (2.3)$$

Plotting of SDI with curing age of samples of a lime treated coastal soil is shown in Fig. 2.39. Fig. 2.39 shows that the values of SDI increases with increasing curing time and lime content as well. Fig. 2.39 clearly shows the relative degree of strength gain resulted due to increasing lime content and curing age. As can be seen from Fig. 2.39 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. 2.40. 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. 2.41. The values of unconfined compressive strength of samples compacted at dry side of optimum moisture content has been found to the least.

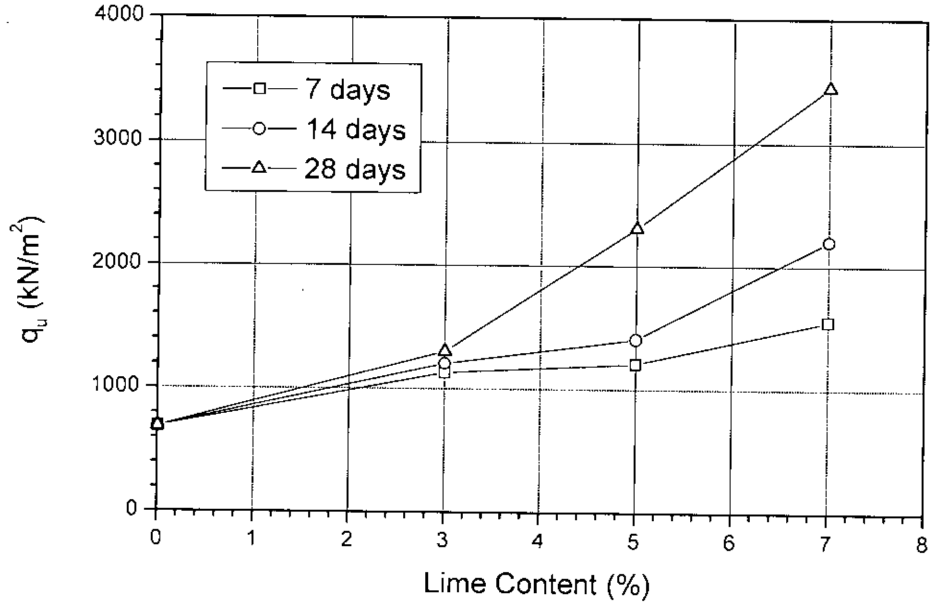

Fig. 2.37 Effect of lime contents on unconfined compressive strength (qu) of a coastal soil at different curing age (reproduced after Rajbongshi, 1997)

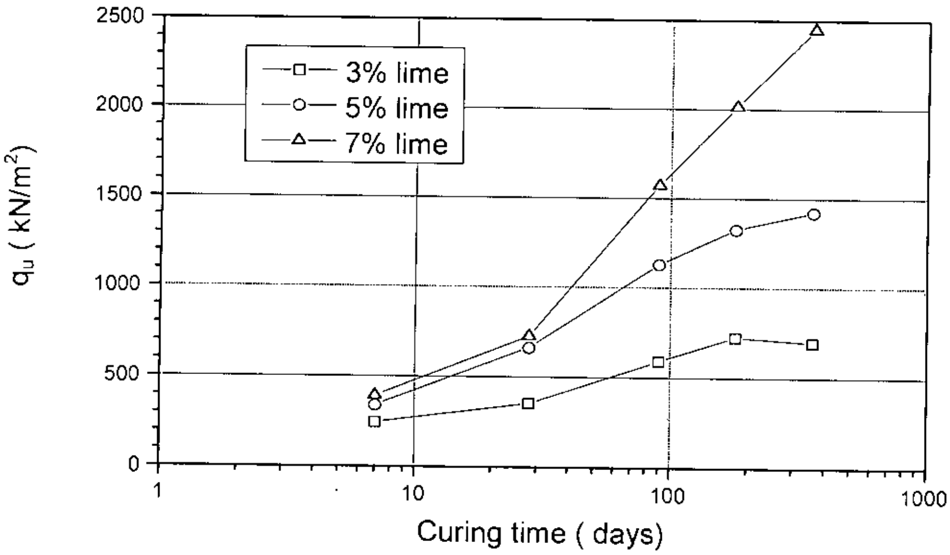

Fig. 2.38 Effect of curing age on unconfined compressive strength (q_u) of a soil (Type- ML/CL) at different lime content (after Shahjahan, 2001)

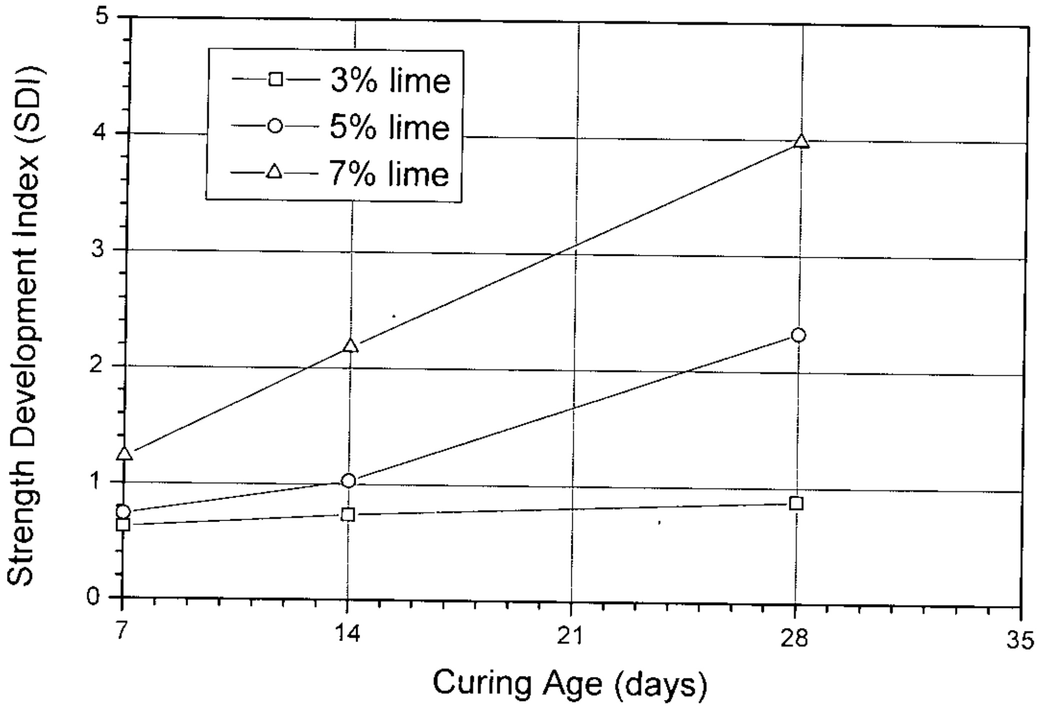

Fig. 2.39 SDI versus curing age curves for samples of a lime-treated coastal soil (after Rajbongshi, 1997)

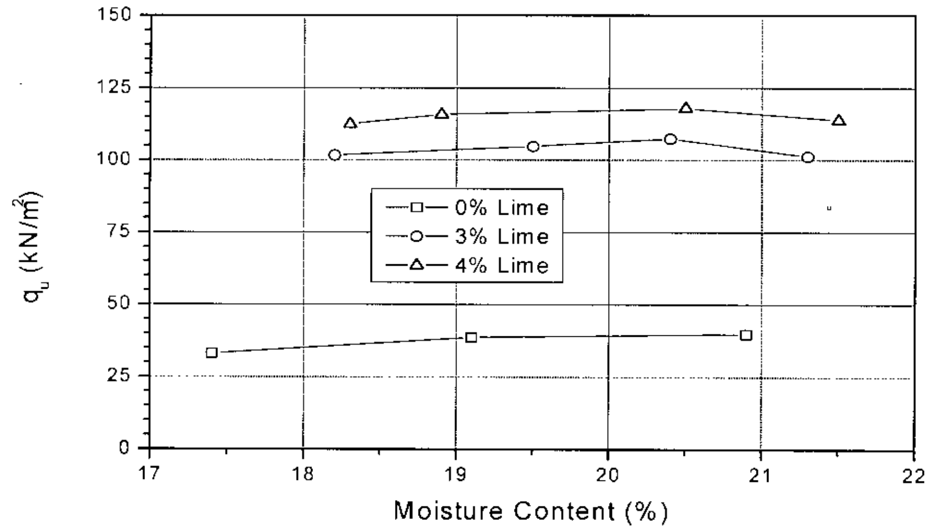

Fig. 2.40 Variation of unconfined compressive strength (qu) with moulding moisture content for a lime-treated silty clay soil (after Molla, 1997)

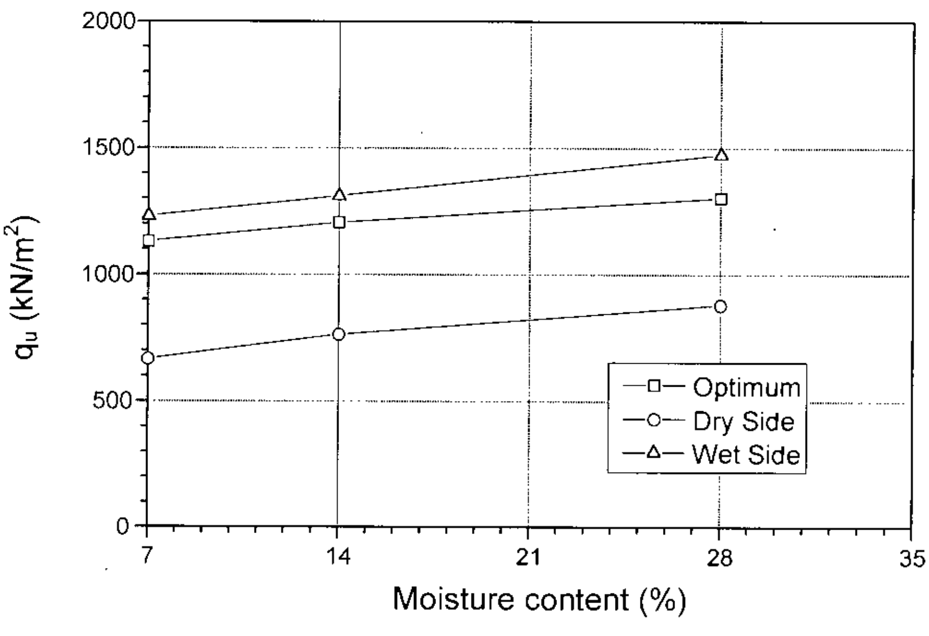

Fig. 2.41 Variation of q_u with curing age for a coastal soil treated with 3% lime and compacted at different moulding water contents (after Rajbongshi, 1997)

2.4.6.4 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 (1986) investigated the effect of lime on CBR-values of two subgrade soils of Bangladesh stabilized with 2%, 4%, 6%, 8% and 10% lime. Hossain (1986) 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. 2.42 for three soils of different plasticity. From Fig. 2.42, 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. 2.43 while Fig. 2.44 presents the CBR-dry density relationships for the same samples. It can be seen from Fig. 2.43 that at all levels of compaction, CBR increases markedly with increasing lime content while Fig. 2.44 shows that at any particular lime content, CBR increases significantly with the increase in dry density. Hasan (2002) also performed CBR test on Dhaka soil found same kind of result. Fig 4.45 and 4.46 shows the relationship of CBR and lime content and CBR versus drydensity, respectively.

2.4.6.5 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:

$$ST = 6.89 + 50.6\,qu \qquad (2.4)$$

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

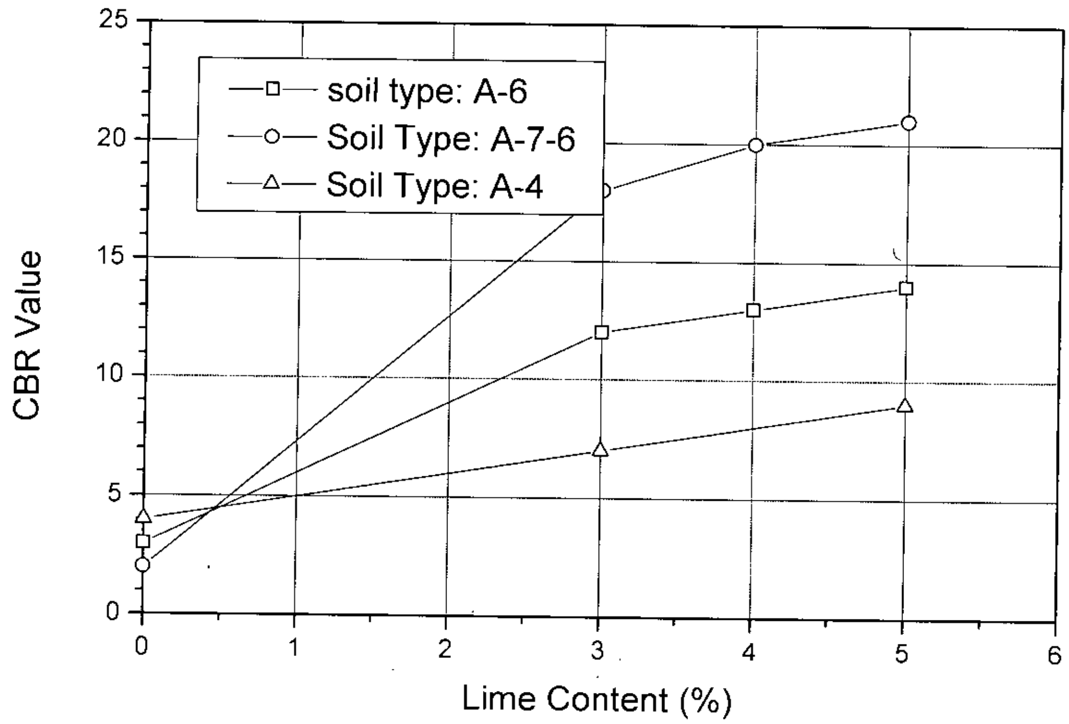

Fig. 2.42 Variation of CBR value with lime content for three regional soils (after Molla, 1997)

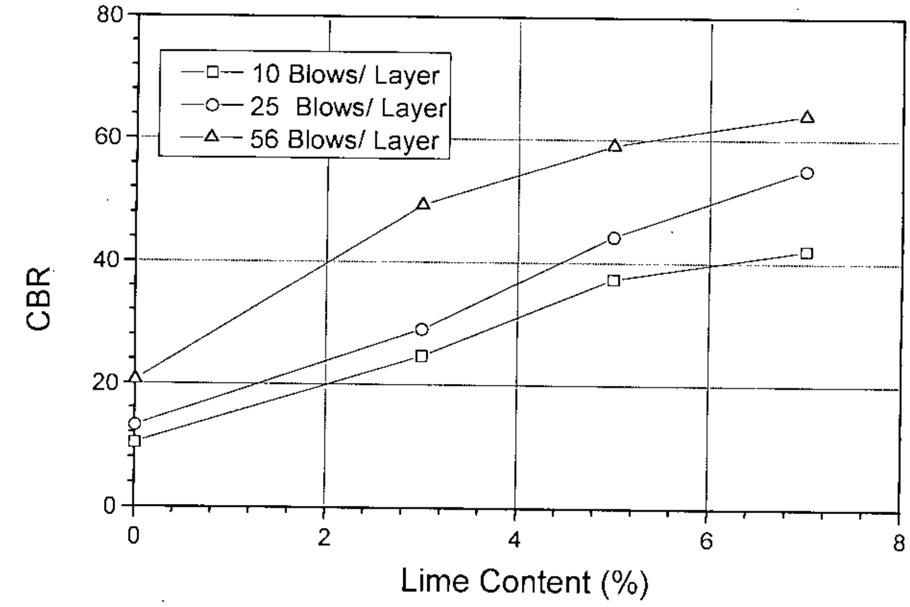

Fig. 2.43 Effect of lime content on CBR values of a coastal soil (after Rajbongshi, 1997)

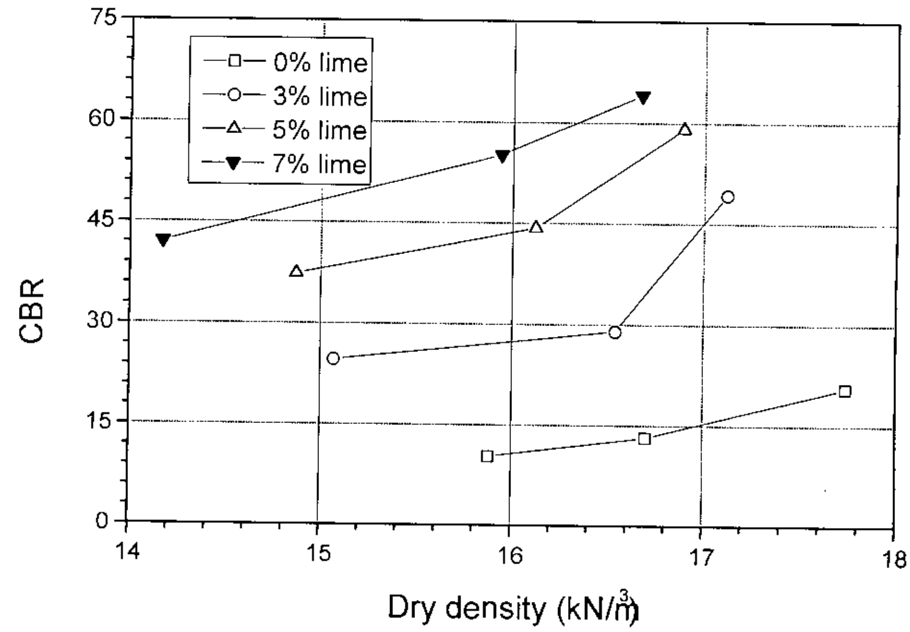

Fig. 2.44 CBR versus dry density curves of a lime-treated coastal soil (after Rajbongshi, 1997)

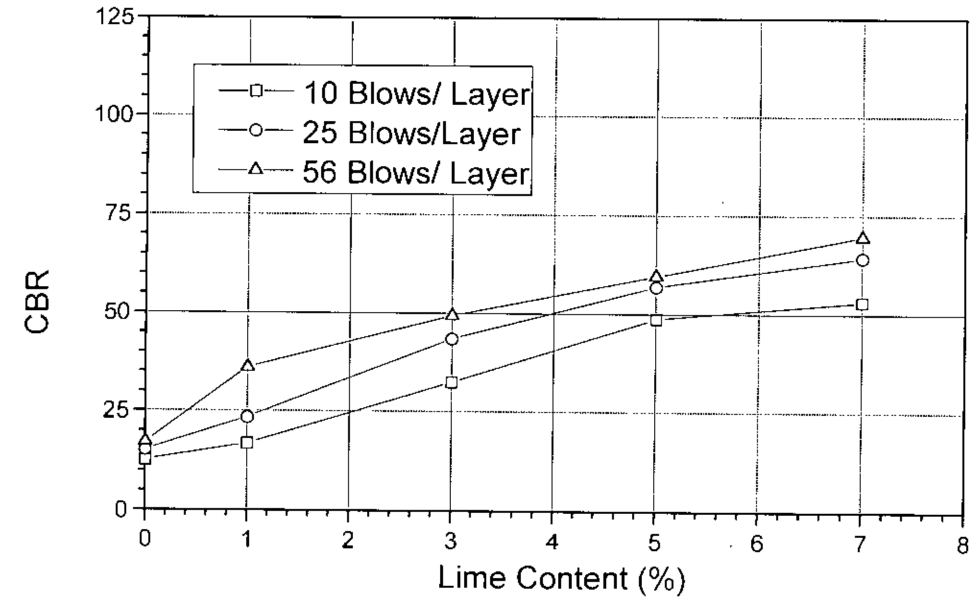

Fig: 2.45 Effect of lime content on CBR values of soil (After Hasan, 2002).

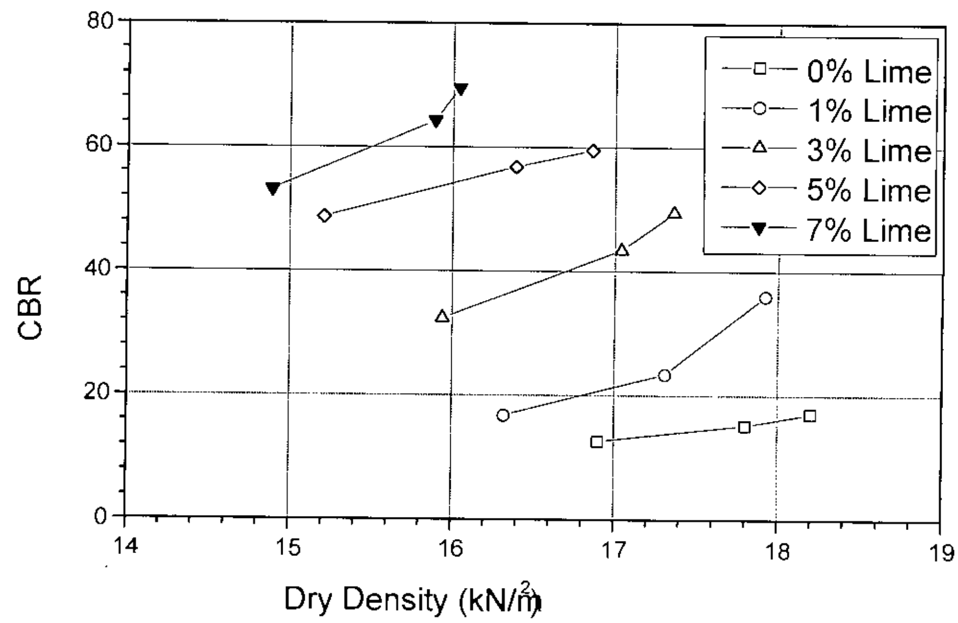

Fig: 2.46 CBR versus dry-density curves of lime-treated soil (After Hasan, 2002).

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. 2.47 while Fig. 2.48 presents the effect of lime content on flexural modulus. Figs. 2.47 and 2.48 show that flexural strength and modulus increases with increasing lime content. It is evident from Figs. 2.47 and 2.48 that curing age has got insignificant effect on increase in flexural strength and modulus. The flexural properties of untreated and stabilized (with lime) sample of Dhaka soil has been also investigated by Hasan, 2002). Fig 2.49 and 2.50 shows the flexúrál stress versus stress versus lime content.

2.4.6.6 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.

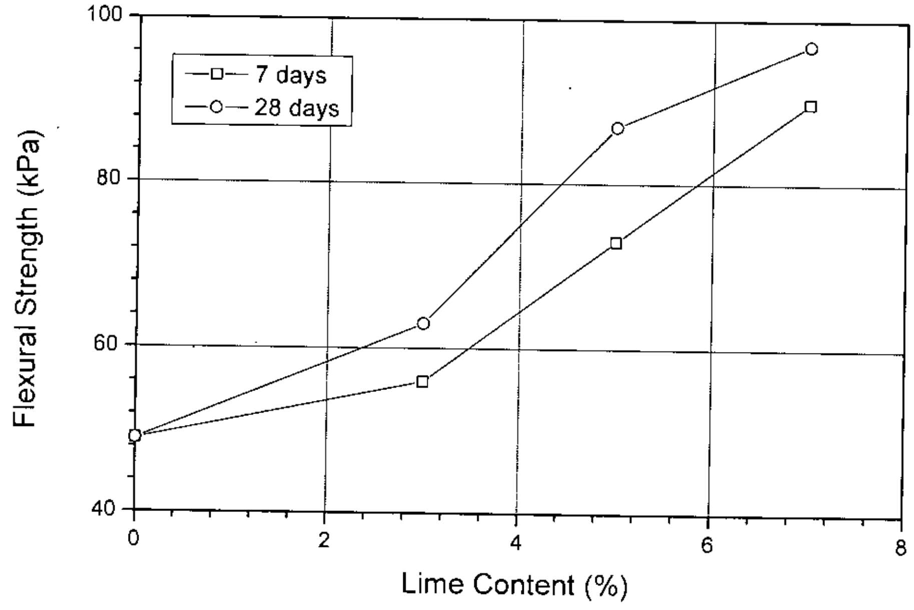

Fig. 2.47 Effect of lime content on flexural strength of a coastal soil (after Rajbongshi, 1997)

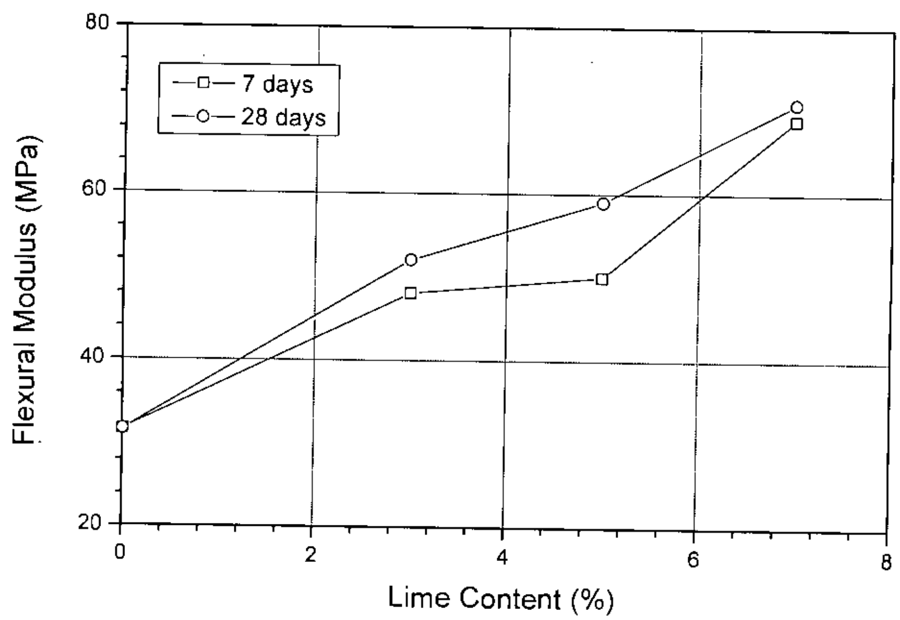

Fig. 2.48 Effect of lime content on flexural modulus of a coastal soil (after Rajbongshi, 1997)

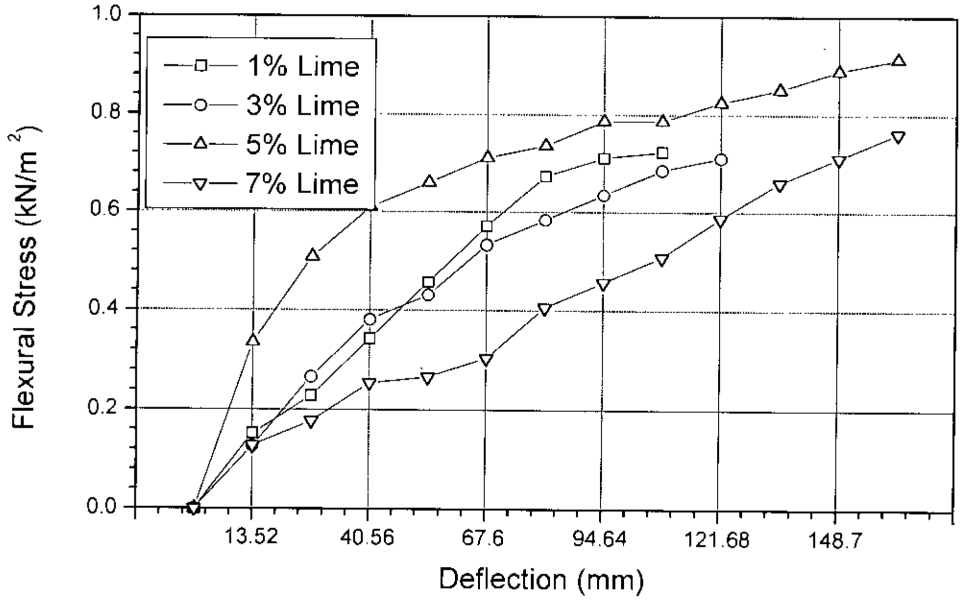

Fig: 2. 49 Flexural stresses versus deflection curve of lime-treated soil (After Hasan 2002).

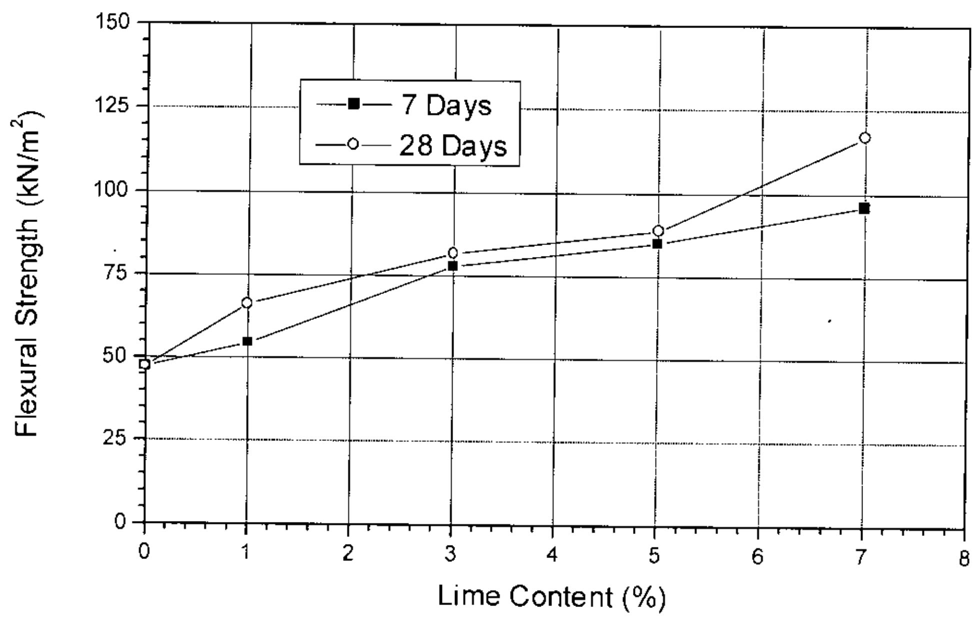

Fig: 2.50 Effect of lime content on flexural strength of soil (After Hasan, 2002)

2.4.7 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 subgrade, 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.

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

CHAPTER 3

LABORATORY INVESTIGATION

3.1 INTRODUCTION

The laboratory investigations carried out on the untreated and stabilized samples of the two soil samples collected from coastal region of Chittagong have been described in details in this chapter.

3.2 SAMPLING AND COLLECTION OF SOIL SAMPLES

Disturbed soils from two selected sites, namely Anwara and Banshkhali of Chittagong coastal region were collected for the present investigation. 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. 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 polyphone bags covered by gunny bags and were eventually transported to the Geotechnical Engineering Laboratory of Bangladesh University of Engineering and Technology, Dhaka. The soil samples were designated as follows:

Soil-A: collected from Anwara.
Soil-B: collected from Banshkhali.

3.3 GEOLOGICAL CONDITION

Bangladesh can be divided into three major physiographic units, namely, the Tertiary Hills, The Palestine Terrace and the Recent Plain land. Chittagong lies within the Tertiary hill unit, which is characterized by hills, valleys, cliffs, plain and beaches. These features developed as result of varying degrees of faulting, folding, upliftment and subsidence due to tectonic activities which have been modified by erosion and depositional activities. The rocks of the Tertiary Hills have been disseted to form long and sub-parallel Nort-Shoth trending ranges following the trend of fold axes.

Between these linear hill ranges extensive low-lying areas of strongly dissected relief occur as plains.

The eastern coastal areas of administrative units of Anwara and Banshkhali are tidal plans. These units are situated as a narrow strip between the Chittagong hilly uplands and the Bay of Bengal. The surface environment of these areas are mainly controlled by shallow seawater and the flood plain activities of the rivers, Carnally, Halda and Shinju. The subsoil's are mainly composed of very soft to medium stiff clay silts and fine grained silty sands with some decomposed organic material near the surface. Tectonically these sites are part of the folded flank of the Bengal Basin.

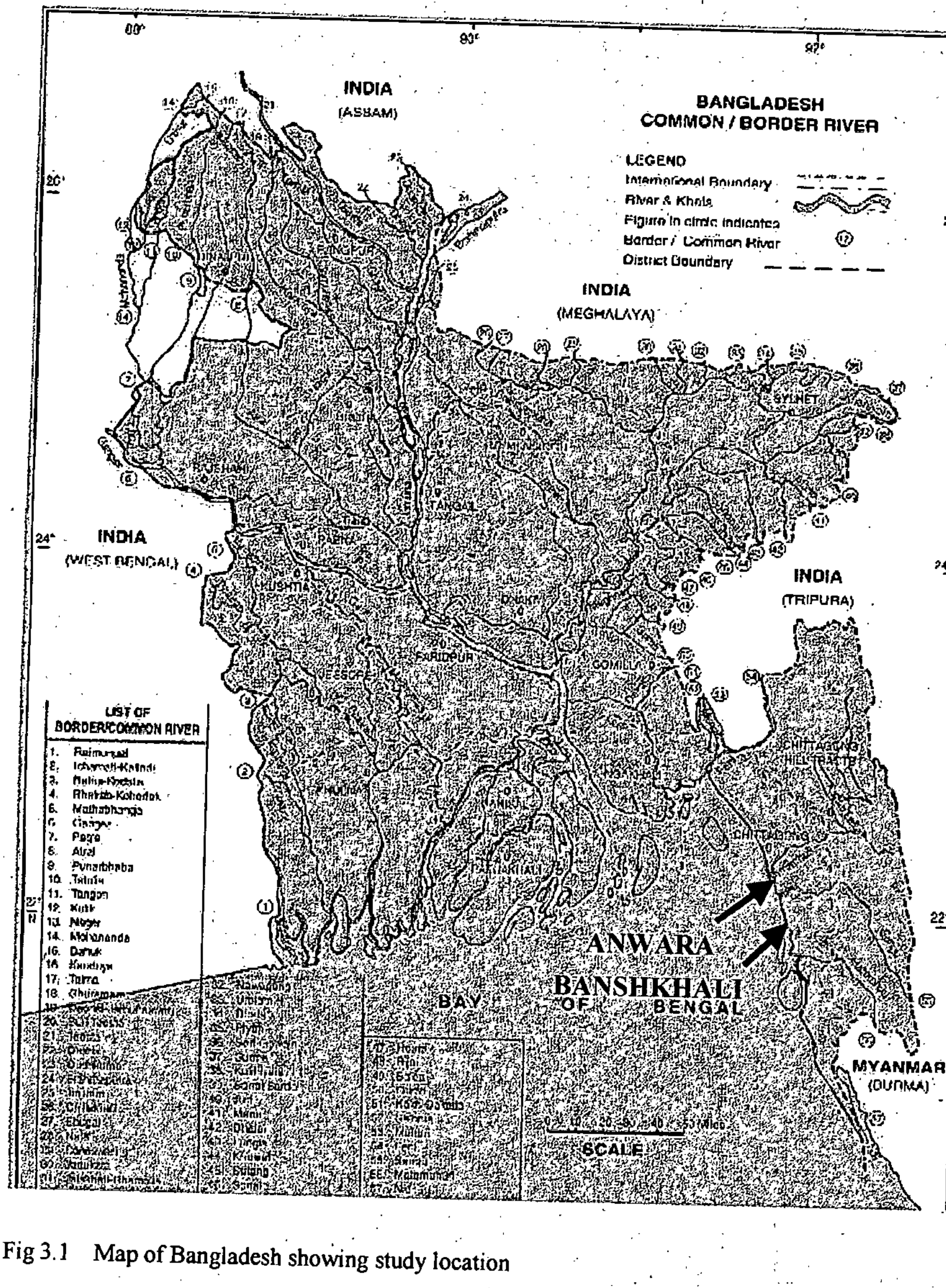

Fig 3.1 Map of Bangladesh showing study location

3.4 LABORATORY TESTING PROGRAMME

A comprehensive laboratory investigation program was undertaken in order to examine the physical, index and engineering characteristics of base soils (i.e., untreated soils) and soils stabilized with flyash and lime. Flyash and air-slaked lime were used as additives for stabilization. Both Soil-A and Soil-B were stabilized with Portland flyash in percentages of 6. 12 and 18 keeping 3% lime constant. Besides both the soil was treated with 3% lime independently. The whole laboratory-testing program consisted of carrying out the following tests on samples of the two coastal soils:

(i) Index property tests on samples of the two coastal soils without any treatment with 3% lime and with different flyash (6, 12, 18). Index tests included specific gravity test, Atterberg limit tests, linear shrinkage test and grain size analysis.

(ii) Chemical analysis of fly ash was done to find out properties.

(iii) The following tests on Soil-A and Soil-B without any treatment with 3% lime. Soil-A and with three different flyash contents (6%, 12% and 18%) were carried out.

(a) Modified compaction test.

(b) Unconfined compressive strength test on molded cylindrical samples of 2.8-inch diameter by 5.6 inch (142 mm) high.

(c) California Bearing Ratio (CBR) tests.

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

Unconfined compressive strength test and flexural strength tests using simple beam with third point loading were carried out on flyash and lime stabilized samples cured at three different ages (7 days. 14 days and 28 days). CBR tests were carried out on the untreated samples and samples treated with different flyash with 3% lime contents using three levels of compaction. Details of laboratory testing programmed 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 coastal soils

Type of Test	Sample	No of Tests	
		Soil-A	Soil-B
Specific Gravity of Soils	Untreated soil/treated	1	1
Liquid Limit and Plastic Limit	Untreated soil	1	1
	Soil-Flyash mixture	3	3
	Soil-Lime mixture	1	1
Shrinkage Limit and Linear Shrinkage	Untreated soil	1	1
	Soil- Flyash mixture	3	3
	Soil-Lime mixture	1	1
Pertiete size Distribution	Untreated soil	1	1
Modified Compaction Test	Untreated soil	1	1
	Soil- Flyash mixture	3	3
	Soil-Lime mixture	1	1
Unconfined Compaction Test	Untreated soil	1	1
	Soil- Flyash mixture	9	9
	Soil-Lime mixture	4	4
CBR Test at Three Levels of Compaction	Untreated soil	1	1
	Soil- Flyash mixture	3	3
	Soil-Lime mixture	1	1
Flexural Strength Test using Simple Beam with Third Point Loading System	Untreated soil	1	1
	Soil- Flyash mixture	9	9
	Soil-Lime mixture	3	3
Chemical Analysis of fly ash		1	

3.5 PHYSICAL AND INDEX PROPERTIES OF UNTREATED SOILS

3.5.1 PREPARATION OF SOIL SAMPLE

The samples collected from the field were disturbed samples. These samples were then air-dried and the soil lumps were broken carefully with a wooden hammer so as to avoid breakage of soil particle. The required quantities of soil were then sieved through sieve No.40. (0.435 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 samples of the two coastal 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) and the positions of the two soils (Soil-A and Soil-B) on the plasticity Chart is shown in Fig.3.3. The soils were also classified according to ASSET Soil Classification System (AASHTO M145-49). Table 3.2 presents are values of index and shrinkage properties, grain size distribution and classifications of Soil-A and Soil-B.

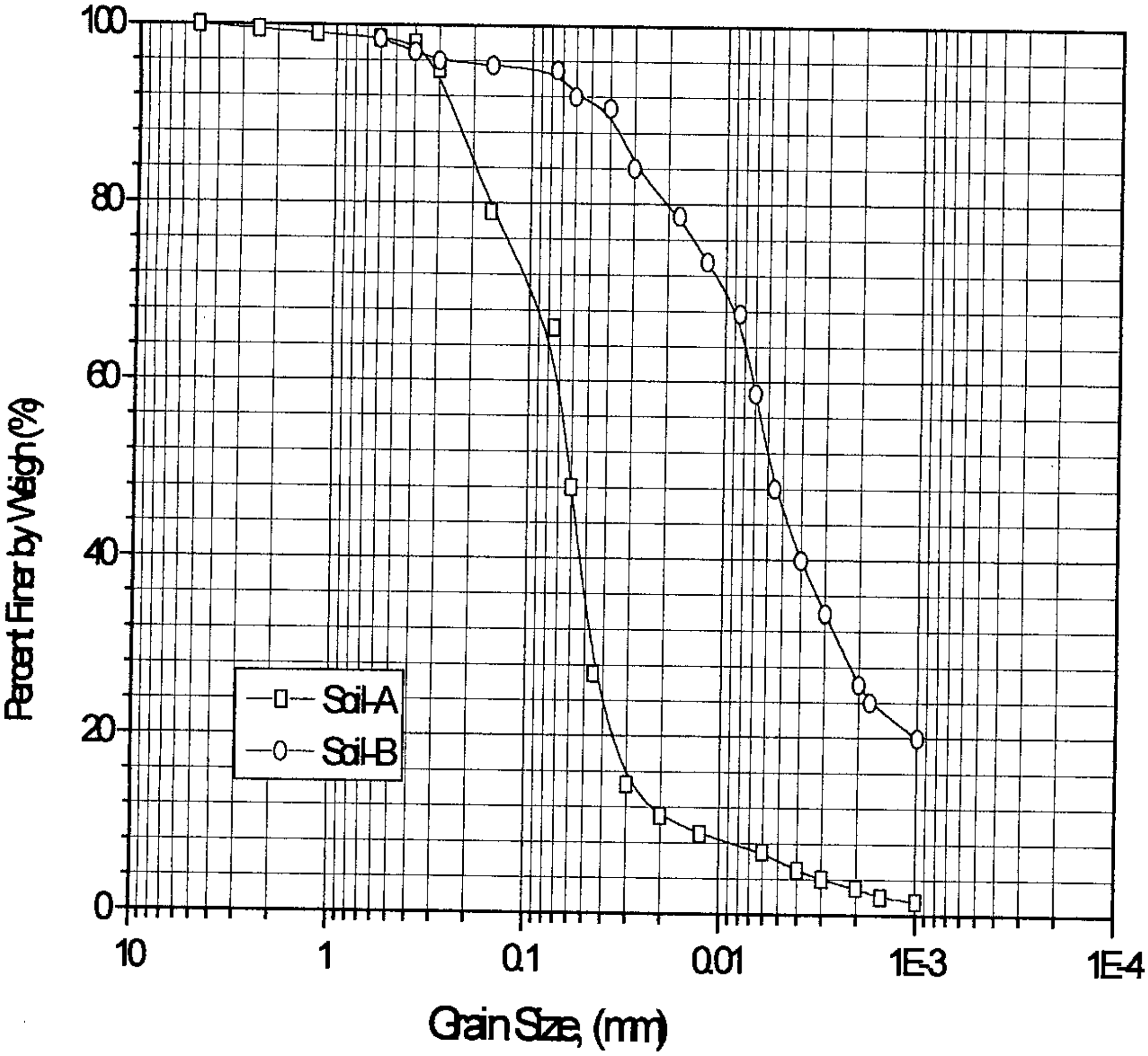

Fig: 3.2 Grain size distribution curves of Soil- A and Soil-B

Fig: 3.3 Plasticity Chart showing the positions of Soil-A and Soil-B

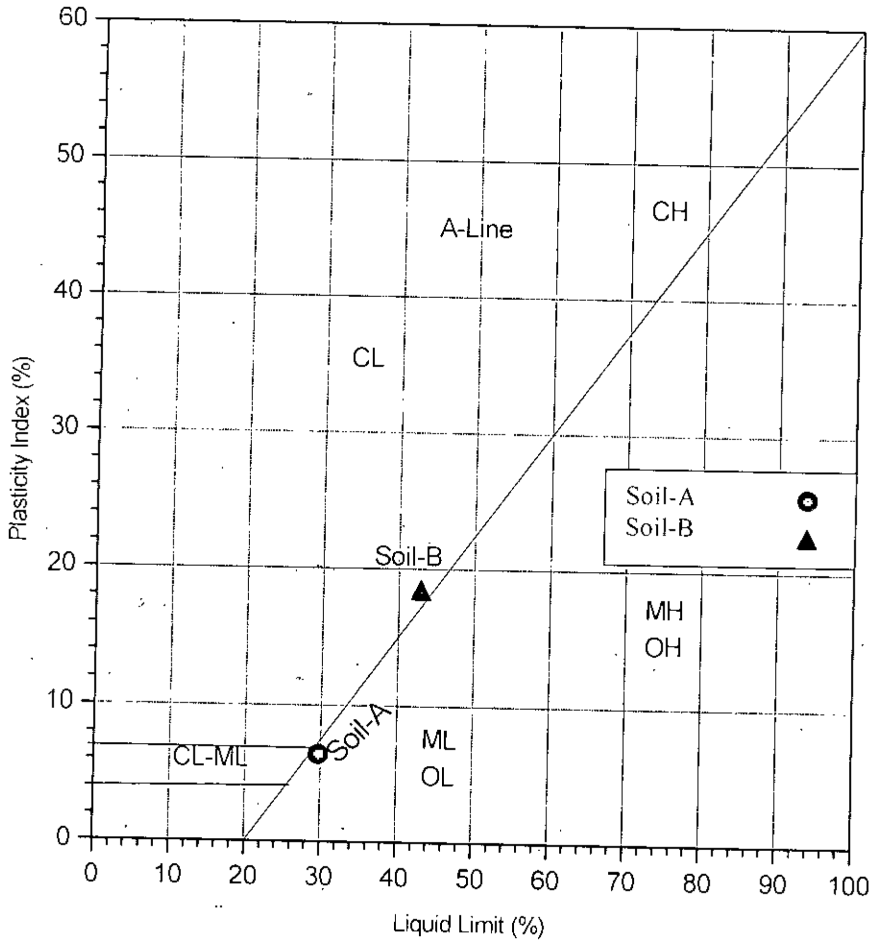

Table 3.2 Index properties and Classification of the coastal soils used

Index Properties and Classification	Soil-A	Soil-B
Specific Gravity	2.70	2.80
Liquid Limit	30	44
Plastic Limit	23	25
Plasticity Index	7	19
Shrinkage Limit	20	23
Linear Shrinkage	7	8
% Sand (0.60 mm to 2 mm)	34	6
% Silt (0.002 mm to 0.06 mm)	62	68
% Clay (< 0.002 mm)	4	26
% of Material Finer than No. 200 Sieve (0.074mm)	68	94
Unified Soil Classification	ML	CL
AASHTO Soil Classification	A-4	A-7-6

3.6 PROPERTIES OF FLYASH USED FOR SOIL STABILISATION

The flyash was obtained from different source. The chemical analysis was made by the Department of Chemistry, DU, Bangladesh. Presented in bellow Table 3.3 the chemical composition of fly ash :

Table. 3.3 Chemical Analysis of Flyash

Sample	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SiO_3	Total
Flyash	10.695	5.00	16.0329	40.5348	24.6545	3.4281	100.3453

Data based on chemical analysis by department of chemistry, Dhaka University

3.7 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 soils (from Anwara and Banshkhali) stabilized with flyash and lime were determined. Flyash and hydrated lime (i.e., slaked lime) were used as additives. Flyash was used in percentages of 6, 12 and 18 while the lime contents were used in percentage of 3 only, Liquid limit, plastic limit and plasticity index of the stabilized samples were carried out on air-dried pulverized samples. The required quantities of pulverized soil were sieved through sieve. No. 40 (0.425 mm). The flyash 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 pulverized to pass through no. 40 sieve. Liquid limit, plastic limit and plasticity indexes of the stabilized samples were determined following the standard procedure outline 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 fly ash and lime treated samples were determined following the procedure outlined in BS 1377.

3.8 COMPACTION TEST

The moisture content versus dry density relationship of the untreated samples of the two coastal soils was investigated by carrying out Modified Compaction test. These tests were performed according to 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 most 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 (457mm). The amount of material used was such that the fifth compacted layer was slightly above the top to the mold but not exceeding 6 mm. During compaction the mould was placed on a uniform rigid foundation. Finally, moisture content and dry density determination were made on each compacted sample of Soil-A and Soil-B.

For the lime and flyash with lime treated samples of the two coastal soils, samples for moulding specimens were prepared according to the procedure outlined ASTM D558. A series of soil-flyash with lime 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 in accordance with the above procedure as outlined in ASTM D1557. The different flyash contents used for preparing samples were 6%, 12% and 18% keepin 3% lime constent while for lime treated samples; lime contents of 3% were use. Finally, moisture content and dry density determination were made on each of the compacted stabilized sample of Soil-A and Soil-B.

3.9 UNCONFINED COMPRESSIVE STRENGTH TEST

3.9.1 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 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. Flyash was used in percentage of 6, 12 and 18 for Soil-A and Soil-B with 3% Lime and the percentages of the additives were calculated on the basis of air-dry weight of the soil samples. Soils were mixed with flyash and lime in a laboratory mixer in batch. This mixing was carried out in a steel pan. Required quantity of water was added into the soil mass until it was thoroughly blended. In order to attain the required 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. 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 flyash and lime contents.

3.9.2 MOULD FOR COMPRESSION TEST

The moulds used for compaction untreated soil, Soil-flyash and soil-lime mix were fabricated using locally available mild steel seamless pipe. The mould complies with the requirement 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-flyash 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 3.4. 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. (142mm) 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 moulds with necessary accessories were fabricated for this research work.

3.9.3 COMPACTION OF SAMPLES

Compaction 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 moulds were held in place with the spacer clip over the bottom piston so that the spacer clip extended about 25 mm into the cylinder. A separation 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).

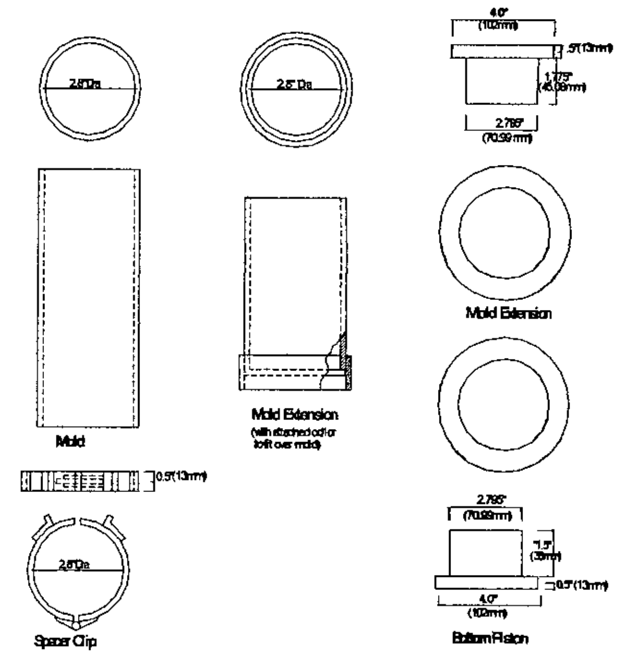

Fig : 3.4 Soil-Flyash mould for compressive strength test (after ASTM, 1989)

A separating disk was placed on the surface of the sample after removal of the extension sleeve. Spacer clip was then removed from the bottom of the piston. The top piston was placed in contact with the top surface of the sample and a static load was applied by a hydraulic compression machine until the sample became 5.6 inch (142 mm) high. The sample was then ejected from the mould using a hydraulic ejector. The compacted dry density of the samples were approximately equal to their respective maximum dry density achieved in the Modified compaction test performed according to the standard procedures outlined in ASTM D1557.

3.9.4 CURING OF SAMPLES

As soon as the samples were ejected from the mould, the samples prepared for unconfined compressive strength were then kept on a level table covered with wetted jute hessian cloth to maintain most condition. The samples were never cured with direct water spray or under submerged condition. The samples were always protected from free water for the specified most curing periods of 7, 14 and 28 days. It may be mentioned that the soil samples that were prepared without adding flyash or lime, i.e., the untreated samples were not cured.

3.9.5 COMPRESSION TEST

The stabilized samples were placed on the compression-testing machine directly after removal from the moist curing condition at different ashes. 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. 91 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. 3.5 presents photograph of the compression test apparatus showing a sample being tested.

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

3.10.1 PREPARATION AND MIXING OF SOILS

The untreated and soils treated with various flyash and lime contents were prepared and mixed in accordance with procedure outlined in section 3.9.1 for the stabilized samples, flyash was used in percentage of 1,3 and 5 for Soil-A and Soil-B. The design moisture flyash of the untreated samples and samples stabilized with flyash 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 stabilized with different flyash and lime contents.

3.10.2 COMPACTION 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 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 lbs (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 stabilized soils, laboratory CBR tests were carried out on the untreated samples and samples treated with flyash and lime using another two levels of compaction energies equivalent to 10, 25 and 56 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 was 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.

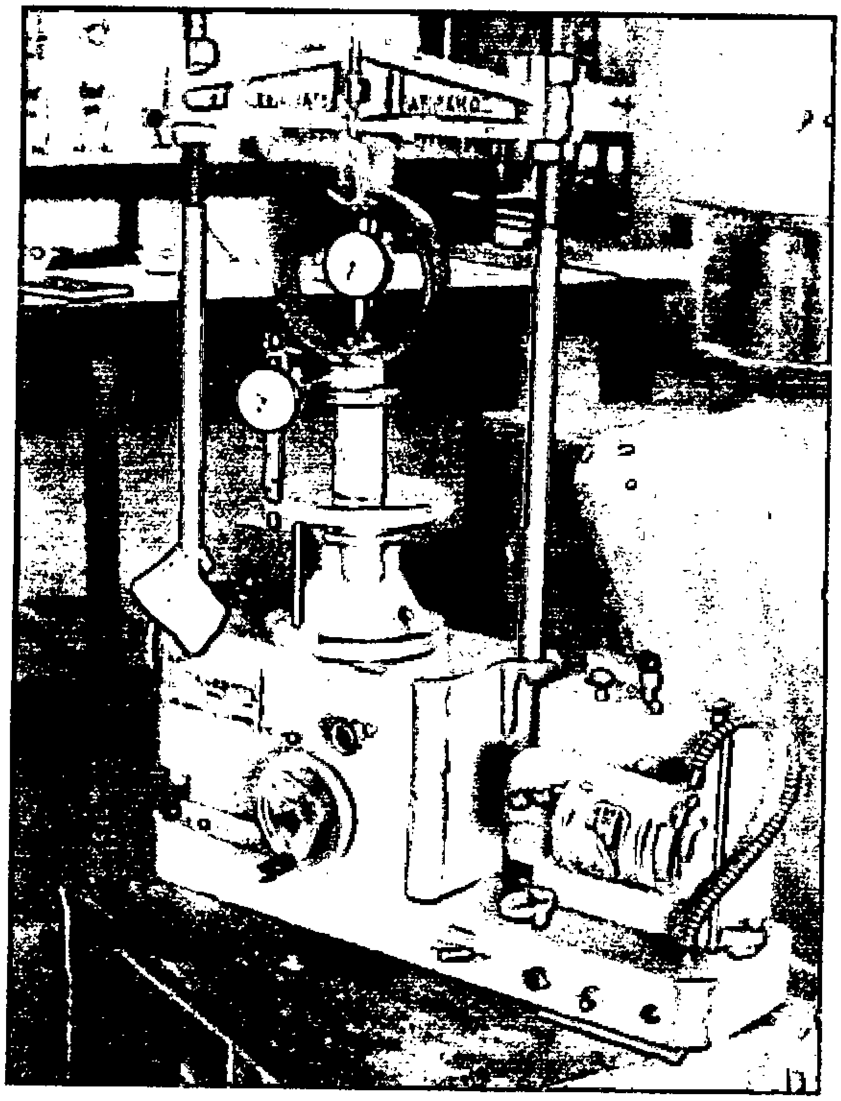

Fig : 3.5 Photograph showing the set-up for unconfined compression test.

3.10.3 SOAKING OF SAMPLE

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

3.10.4 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 he 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 penetration. All these tests were performed following the standard procedure outlined in ASTM D1883. Fig 3.6 presents a photograph of the bearing test apparatus.

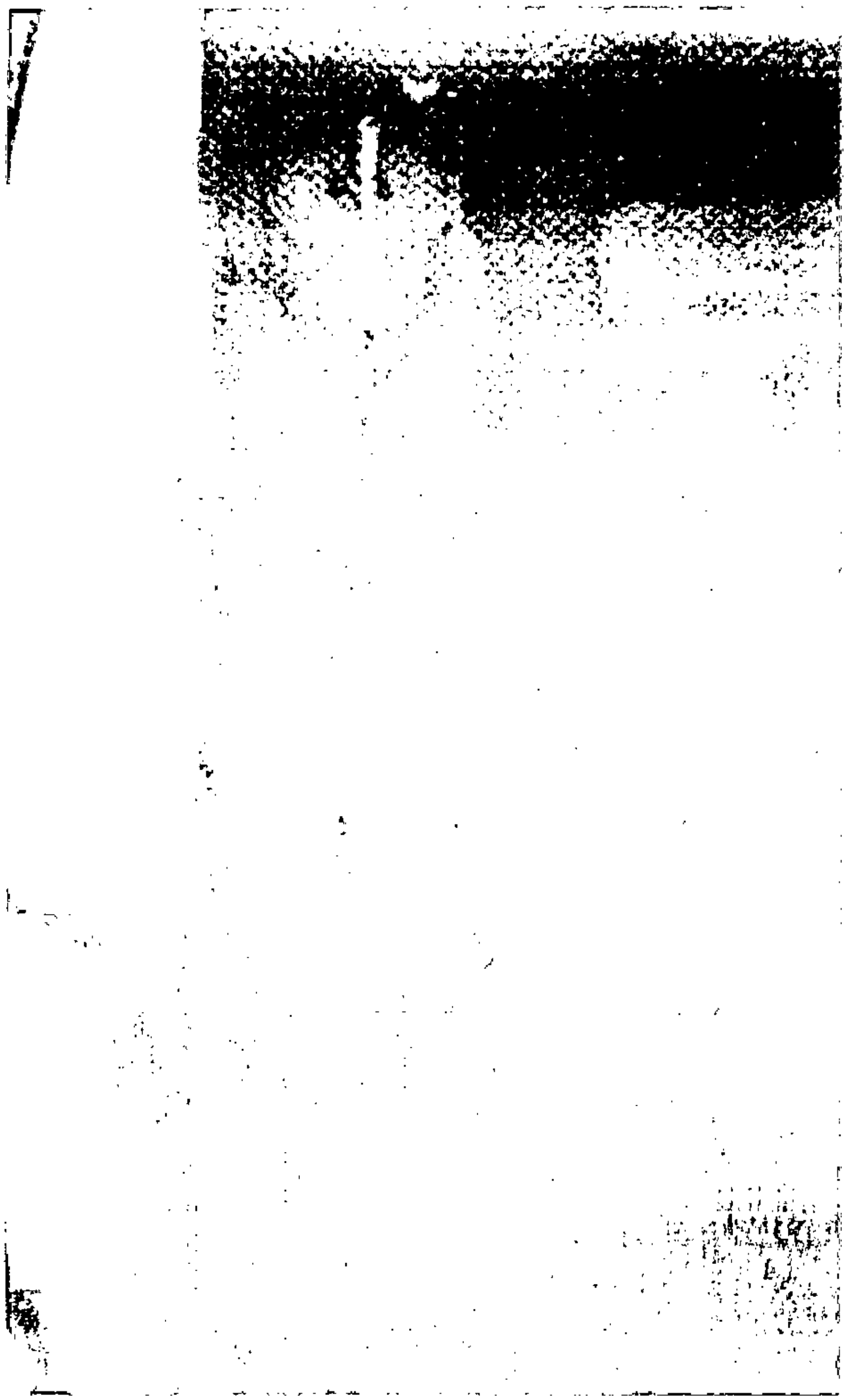

Fig. 3.6 Photograph showing the arrangements of CBR test.

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

3.11.1 PREPARATION AND MIXING OF SOILS

The untreated and soils treated with various flyash and lime contents were prepared and mixed in accordance with the procedure outlined in section 3.9.1 For the stabilized samples. Flyash was used in percentage of 6, 12 and 18 for Soil-A and Soil-B with the lime content in percentage of 3. The design moisture content of the untreated samples and samples stabilized with flyash and lime were equivalent to the respective values of optimum moisture contents of the untreated samples and samples stabilized with flyash 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 stabilize different flyash and lime contents.

3.11.2 MOULD FOR FLEXURE TEST

The moulds used for compacting untreated soil, soil-flyash and soil-lime mixture were fabricated using locally available mild steel plates, which comply with the requirements of ASTM D1632.

The fabrication procedure of these moulds was difficult as compared with that for 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 dimension of in 3 in. by 11 ¼ in (76.2 mm by 76.2 mm by 285.8 mm) for moulding specimens of the same size. The mould was manufactured in such a way the sample could be moulded with its longitudinal axis in a horizontal position. The parts of the mould were made to be tight fitting and held together. The sides of the mould were sufficiently rigid to prevent spreading or warping. The interior faces of the mould were machined to plane surfaces within a variation, in any 3 in (76.2-mm) line on a surface, of 0.002 in (0.051 mm). The distance between opposite sides was within 3 0.01 in (76.20 =0.25 mm). The height of the mould was made 3 in, (76.20 mm) within the variation of –0.01 in (-0.25 mm). Four 3/8 in. (9.25 mm) spacer bars and top and bottom machined steel plates were provided. The plates fit the mould with a 0.005-in. (0.13-mm) clearance on all sides.

3.11.3 MOULDING AND CURING OF SAMPLE

The test samples were prepared with the longitudinal axis horizontal. The inside parts of the mould were first lightly oiled. Then the mould was assembled with the sides and ends separated from the base plate by the 3/8 in. (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 leveled 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 roddings were distributed uniformly over the cross section of the mould. This layer of compacted sample was leveled by hand and layers two and three were compacted in the similar way. The sample at this time was made approximately 3 ¾ 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.

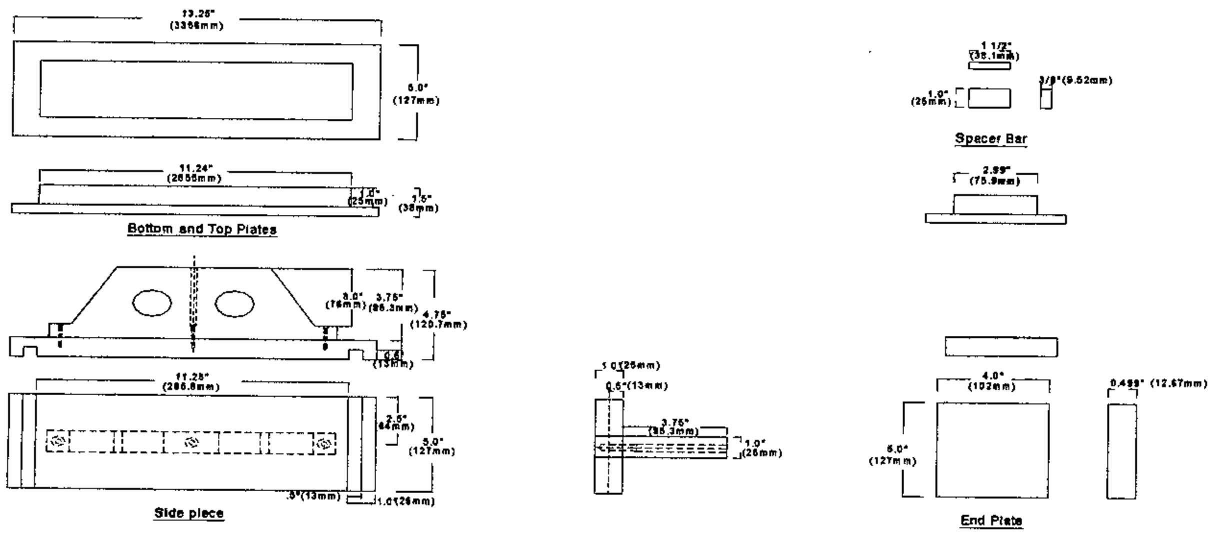

Fig. 3.7 Schematic diagram of soil-flyash beam mould for flexural test with third point loading system (after ASTM, 1989)

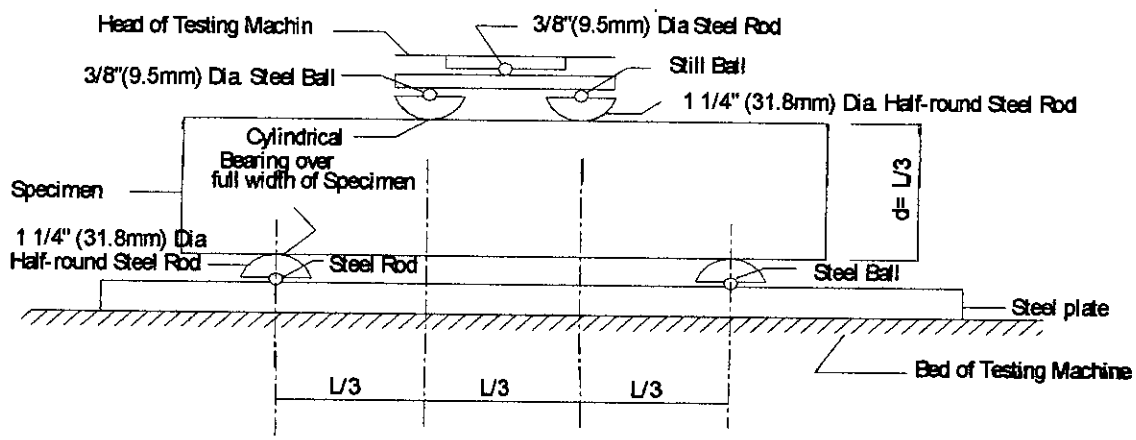

Fig. 3.8 Schematic diagram of the set-up for flexural test with third point loading system (after ASTM, 1989)

As soon as the soil-flyash and soil-lime samples were removed from the mould they were kept on a table covered with wetted jute hessian cloth. The samples were never cured with direct water spray or under submerged condition. The samples were always protected from free water for the specified moist curing periods of 7, 14 and 28 days. The soil samples that were prepared without adding flyash or lime were not cured. The treated samples were carried for testing purpose directly from the moist curing environment.

3.11.4 FLEXURAL STRENGTH TEST

The flexure tests of untreated soil, soil-flyash 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., 9in.). The load applying assembly block was placed in contact with the upper surface of the beam at the third points between the supports. The center of the beam was aligned with the center of the thrust of the spherically seated head block of the machine. The movable part of this head block was 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 load proving ring of capacity 10 KN. Load was applied at a deformation rate of approximately 0.05in/min. (0.02 mm/s) Two dial gauges were fitted under the beam specimen to record the deflection of the beam. The total loads until failure of the specimen was recorded. A schematic diagram of the apparatus for flexure test of soil, soil-flyash and soil-lime samples by third point loading is shown in Fig. 3.8 while Fig 3.9 shows the a photograph of a test set-up.

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:

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

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:

$$R = \frac{3Pa}{bd^2}$$

Where:

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

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

$$E = \frac{23PL3}{1296I\,\Delta}$$

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

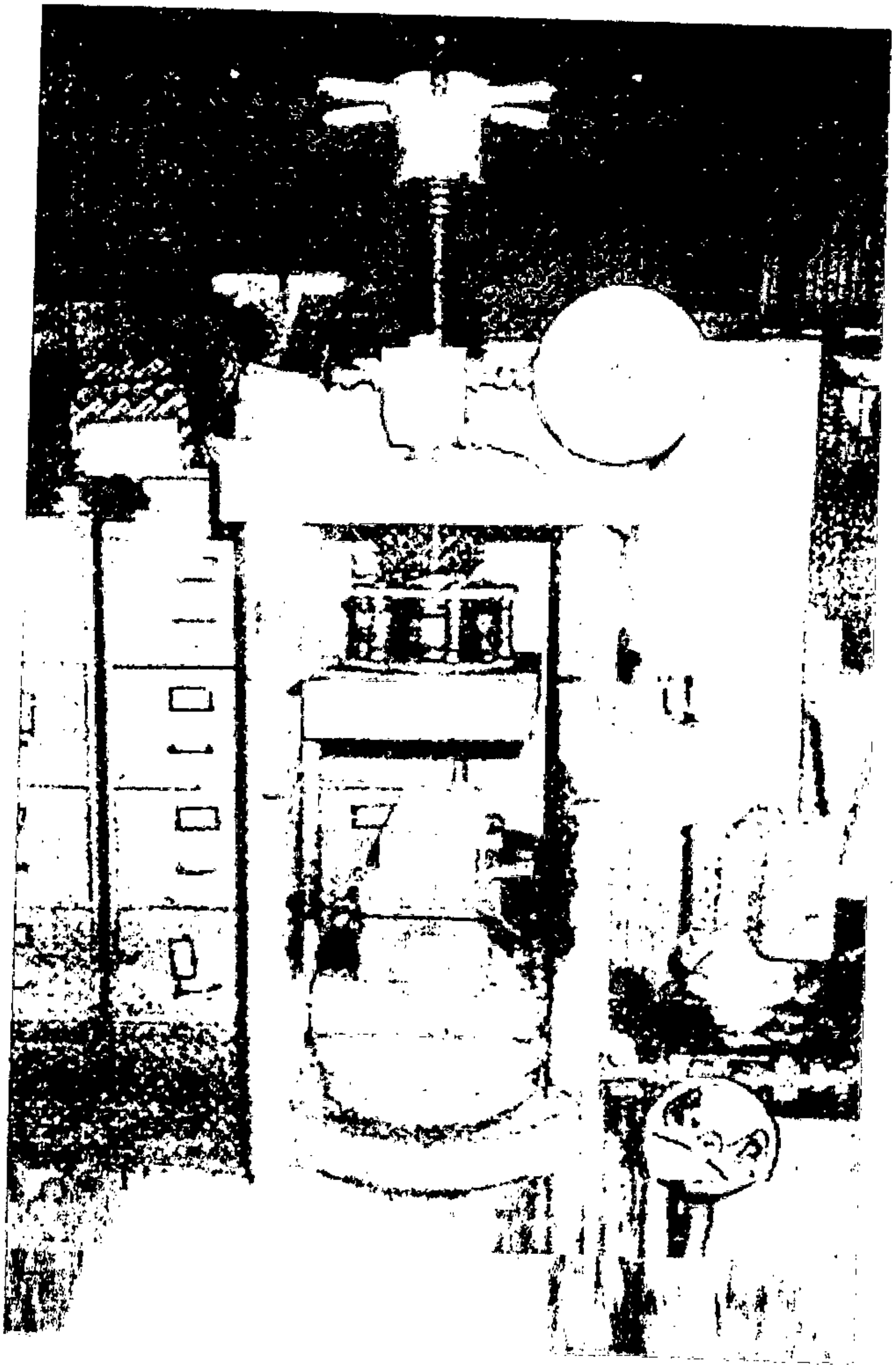

Fig. 3.9 Photograph showing the set-up for flexural test with third point loading system

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 INTRUDUCTION

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

4.2 PHYSICAL AND ENGINEERING PROPERTIES OF FLYASH-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, of untreated and flyash-treated samples of the two coastal soils are presented and discussed.

4.2.1 PLASTICITY AND SHRINGKAGE CHARACTERSTICS

The values of plasticity and shrinkage properties of the untreated and flyash-treated soil samples are shown in Tables 4.1 and 4.2 for Soil-A and Soil-B 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 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 flyash additions. It can be seen from Fig.4.1 that for Soil-A (LL=30, PI=7), both liquid limit and plastic limit increased while for Soil B- (LL=44, PI=19) liquid limit reduced and plastic limit

Table 4.1 Index and shrinkage properties of flyash-treated Soil-A

Index and Shrinkage properties		Lime/flyash content			
		Lime	Flyash content with 3% lime		
	0	3% lime	6	12	18
Liquid Limit	30	30	33	37	39
Plastic Limit	24	25	29	34	36
Plasticity Index	6	5	4	3	3
Shrinkage Limit	20	20	19	19	18
Linear Shrinkage (%)	7	6	6	5	5

Table 4.2 Index and shrinkage properties of flyash-treated Soil-B

Index and Shrinkage properties		Lime/flyash content			
		Lime	Flyash content with 3% lime		
	0		6	12	18
Liquid Limit	44	43	43	42	42
Plastic Limit	26	30	31	32	34
Plasticity Index	18	13	12	10	8
Shrinkage Limit	23	26	22	20	20
Linear Shrinkage (%)	8	7	7	6	6

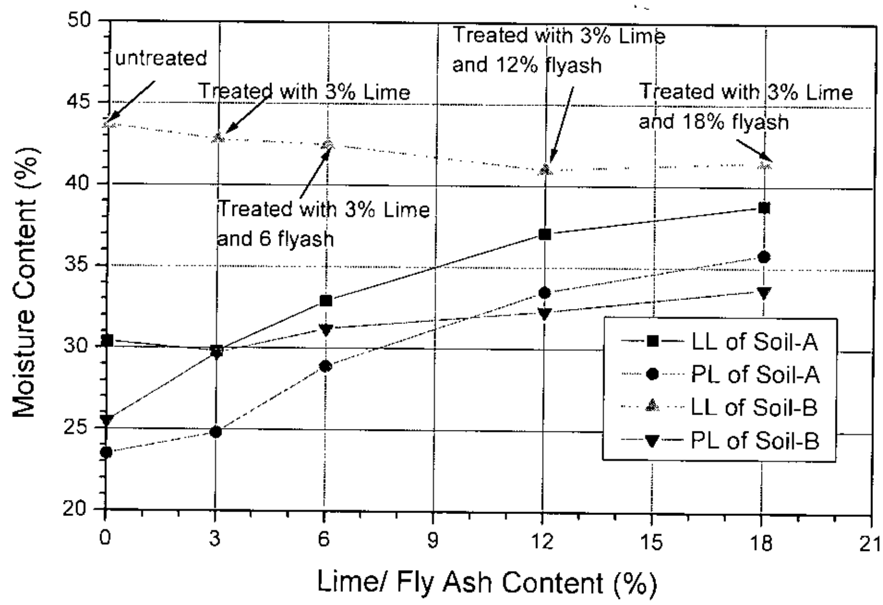

Fig. 4.1 Effect of lime and fly ash on liquid limit and plastic limit of Soil-A and Soil-B

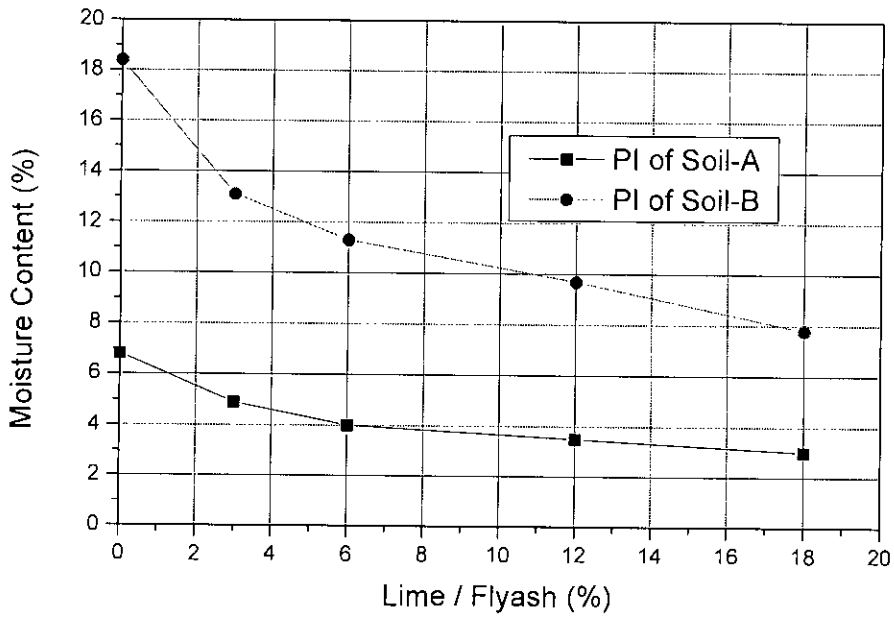

Fig. 4.2 Effect of lime and fly ash content on plasticity indices of Soil-A and Soil-B

Increased with increasing flyash content except LL decreases with 3% lime for both soil. These results are in agreement with those reported by Willis (1974, Felt (1955) and Ahmed (1984). 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 clayey silt (LL=33, PI=6) respectively. However, for a silty clay (LL=43, PI=21), Ahmed (1984) found a reduction in liquid limit and plasticity index, and an increase in plastic limit with increasing cement content. Glen and Handy also stressed that lime added to clay soil cause rapid depression of PI.

The changes in shrinkage limit due to increase in flyash content are shown in Fig 4.3 while Fig 4.4 presents the variation of linear shrinkage with the increase in flyash 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 flyash content. Where as shrinkage limit slightly increase with 3% lime content for both the soil. Reduction in shrinkage limit with increased cement content has been reported by Willis (1947), Mehra and Uppal (1950) and Jones (1958). Kezdi (1979) reported reduction in linear shrinkage due to increase in cement content in three clayey soils of different plasticity.

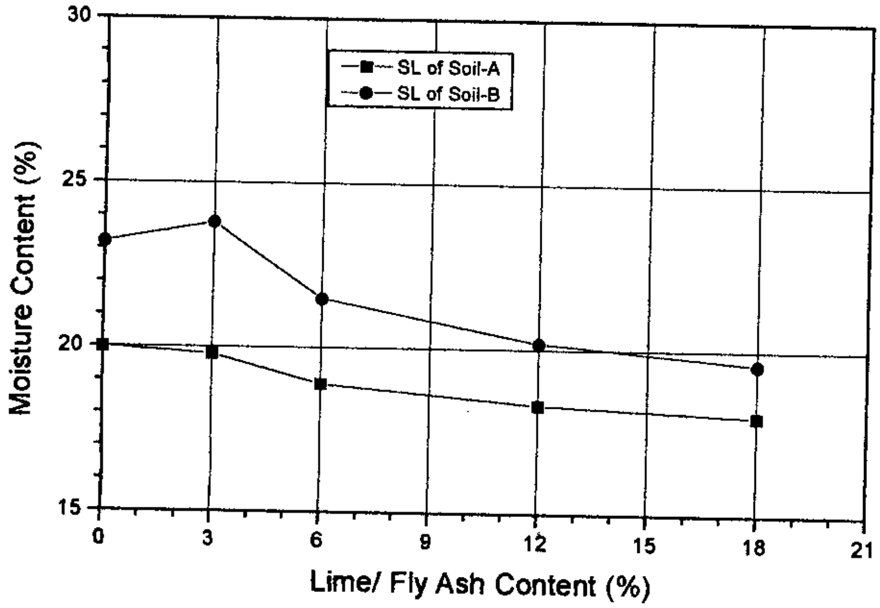

Fig. 4.3 Effect of lime and fly ash content on shrinkage limit of Soil-A and Soil-B

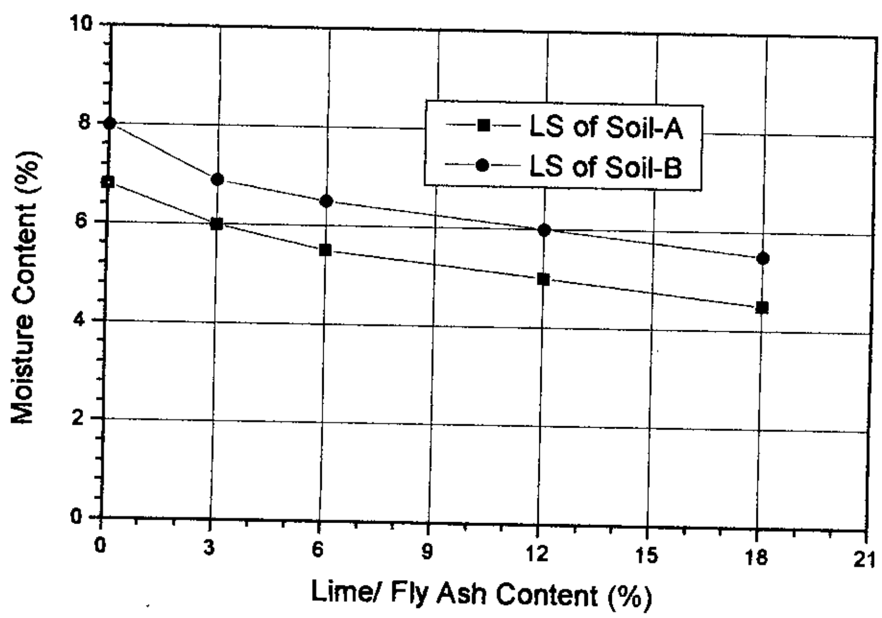

Fig. 4.4 Effect of lime and fly ash content on linear shrinkage of Soil-A and Soil-B

4.2.2 COMPARISON OF INDEX PROPERTIES

Comparison of index properties of lime treated regional soils and soils investigated in the present study with flyash are described by the table 4.3. For all the cases the shrinkage limit increase with the increases in lime content When Shrinkage limit decreases with increase of flyash 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.

Table 4. 3 Comparison of Index properties of lime treated regional soils

Soil Code	Lime/ Flyash 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
MI-A	0	30	24	20
	3% Lime + 6% Ash	33	29	19
	3% Lime + 12% Ash	37	34	18
	3% Lime + 18% Ash	39	36	18
MI-B	0	44	26	23
	3% Lime 6% Ash	43	30	22
	3% Lime + 12% Ash	42	31	20
	3% Lime + 18% Ash	41	34	20

4.2.3 MOISTURE DENSITY RELATIONS

The moisture-density relations of untreated and flyash-treated samples of Soil-A and Soil-B are shown in Figs 4.5 and 4.6, the maximum dry density (Υmax) and optimum moisture contents (Wopt) of Soil-A and Soil-B have been determined which are presented in Table 4.4 that for both the soils, with the increase in flyash content with 3% lime, values of Υmax increased while the values of Wopt reduced. While only with 3% lime both and ωopt reduced for both the soil, which satisfied Leonord & Daidson experiment. The increase in Υmax with the increase in flyash content for the two soils is shown in Fig. 4.7. Compared with the untreated sample, the values of Υmax increased up to 8% and 7% for Soil-A and Soil-B respectively. The values of ωopt reduced up to 9% and 10% respectively for Soil-A and 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. Ahmed (1984) found that for sandy silt and silty clay soils of Bangladesh, 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 regional 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 (a clayey silt of low plasticity and a silty clay of medium plasticity) of Bangladesh. 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).

Table 4.4 Values of maximum dry density and optimum moisture Content of untreated and flyash-treated Soil-A and Soil-B

Lime/Flyash Content %	Soil-A		Soil-B	
	γ_d (kN/m^3)	ω_{opt} %	γ_d (kN/m^3)	ω_{opt} %
0	17.30	16.50	17.50	15.3
3% Lime	17.10	15.50	17.30	15.7
3% Lime + 6% flyash	17.40	15.0	17.90	14.80
3% Lime + 12% flyash	18.20	14.50	18.40	14.30
3% Lime + 18% flyash	18.60	14.0	18.80	13.80

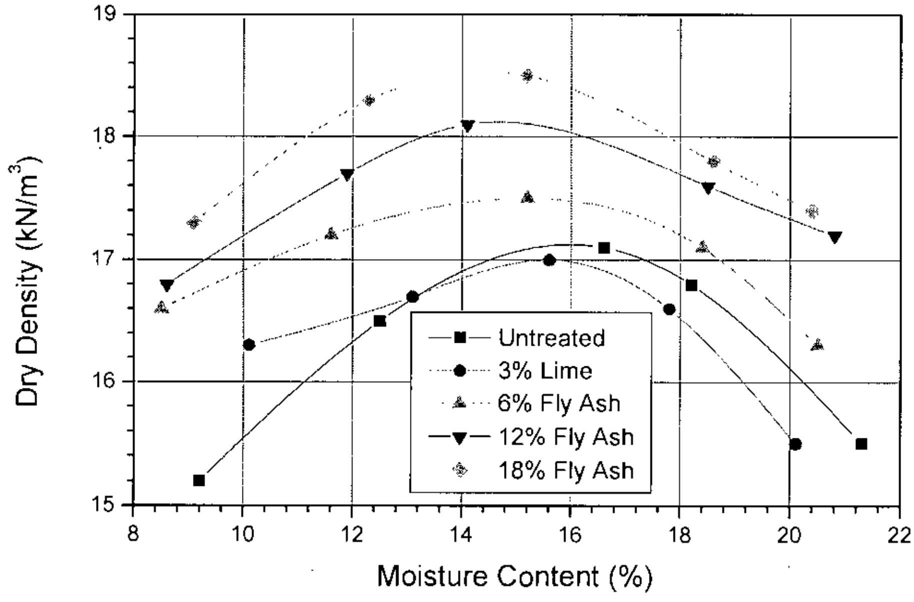

Fig. 4.5 Moisture-density relations of untreated, lime and fly ash –treated Soil-A

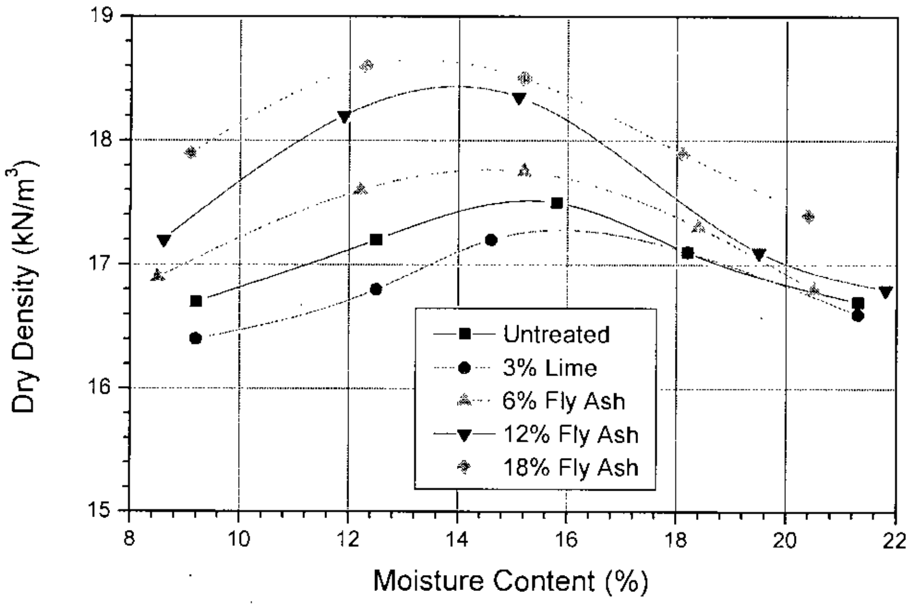

Fig. 4.6 Moisture-density relations of untreated, lime and fly ash –treated Soil-B

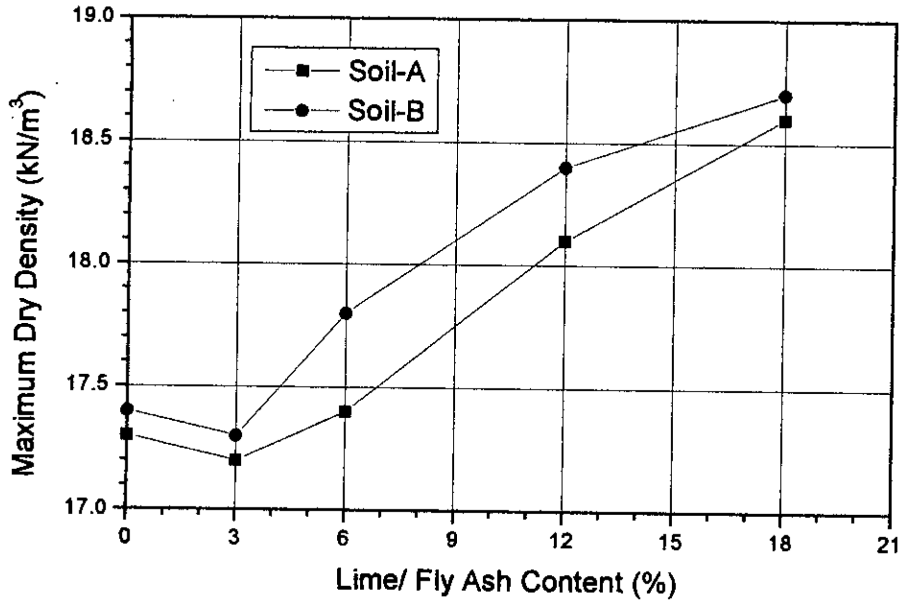

Fig. 4.7 Effect of fly ash content on maximum dry density of lime and fly ash-treated Soil-A and Soil-B

4.2.4 COMPARISON OF MOISTURE DENSITY RELATION

The moisture-density relations of untreated and lime treated regional soils are shown in Fig 4.8 and Fig 4.9 From the relations shown in fig 4.8 and fig 4.9 the maximum dry density and optimum moisture content have been determined which are presented in Table 4.5 From all the regional soils data shown in the table 4.5 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 Kezi (1979), TRB,(1987), Hausmann (1990), Bell (1993). While optimum water content decreases with increase of flyash content and dry density increases with increase of flyash content. Similar result observed by Govinda Ram 1983 and Rasbangshi work on cement stabilization. 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 18.82 kN/m^3 to 13.8 kN/m^3- while vis versa in the case of flyash as additives.

Table 4.5 Comparison of moisture – density relations for lime stabilized regional soils of Bangladesh

Soil Code	Content (%)	Optimum Moisture Content (%)	Maximum Dry Density (kN/m³)	Soil Code	Content (%)	Optimum Moisture Content (%)	Maximum Dry Density (kN/m³)
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-I	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-I	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	MI-A	0	16.5	17.3
M-3	0	18.8	15.8		3% Lime 6% Ash	15	17.43
	3	19.4	15.5		3% Lime 12% Ash	14.5	18.22
	5	19.8	15.4		3% Lime 18% Ash	14	18.6
				MI-B	0	15.3	17.5
					3% Lime 6% Ash	14.81	17.9
					3% Lime 12% Ash	14.3	18.4
					3% Lime 18% Ash	13.8	18.82

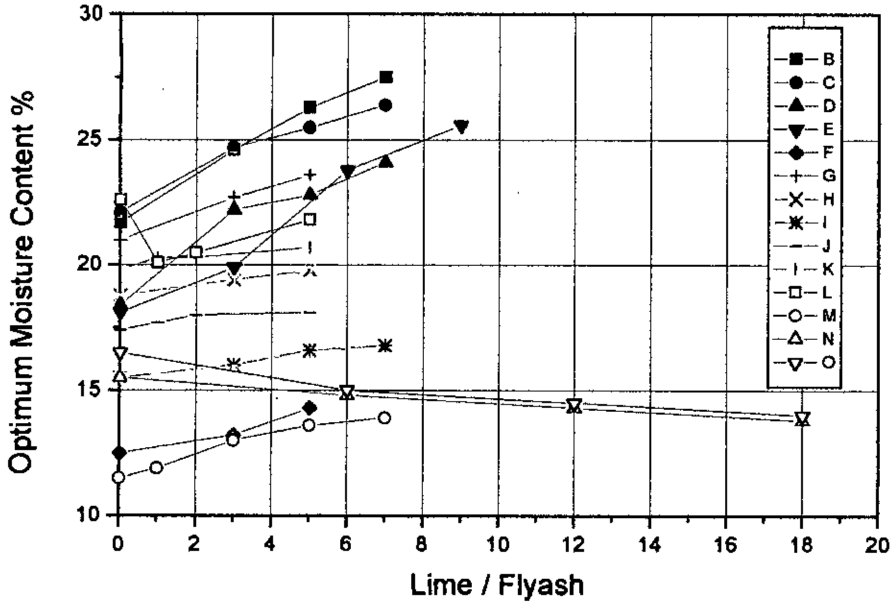

Fig. 4.8 Comparison of the effect of lime stabilization on optimum moisture content between different regional soils of Bangladesh and soils used in the present study.

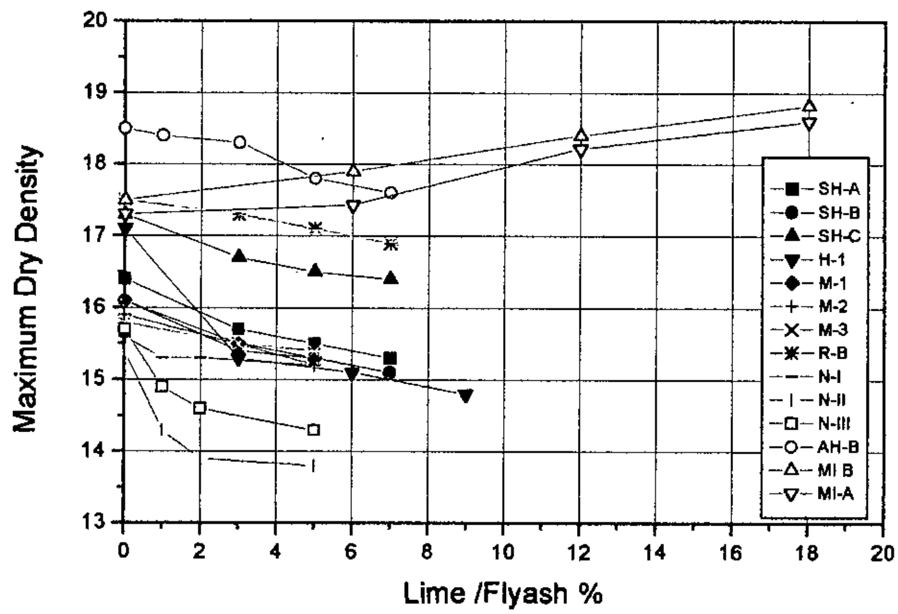

Fig. 4.9 Comparison of the effect of lime stabilization on dry density between different regional soils of Bangladesh and soils used in the present study

4.2.5 UNCONFINED COMPRESSIVE STRENGTH

Table 4.6 shows a summary of the unconfined compression test results for Soil-A and Soil-B. In Table 4.6. the values of unconfined compressive strength (qu) and axial strain at failure ε_f for the untreated samples and samples treated with 3% lime and different flyash contents (6%, 12% and 18%) and cured for 7, 14 and 28 days are presented. It can be seen from Table 4.6 that for both the soils, compared with the untreated samples, the values of qu of the treated samples increased significantly, depending on the flyash content and curing age. Leonard and Davidson (1959) reported that because of the slow reaction of lime absorption, the development of compressive strength of soil directly related with lime absorption by flyash. Methods shows that for clayey soils. the amount of lime should be 3 – 9 percent and the amount of flyash between 10 to 25 percent for soil stabilization. Similar results have also been reported by Hossain (1986), Serajuddin and Azmal (1991) and Serajuddin (1992) for fine-grained soils of Bangladesh for use in road construction. It can be seen from Table 4.6 that the values of qu of samples of Soil-A and Soil-B treated with 6% flyash and cured at 28 days were found to be about 4 times higher than the strength of the untreated samples and with 18% fly ash it about 5 times higher than untreated sample. While only with 3% lime it increases only 2 times. It is also evident from Table 4.6 that the gain in strength with increasing flyash content and curing age is higher in less plastic Soil-A (PI=7) than in more plastic Soil-B (PI=19). PCA given in Annex. A (1956) recommended that values of q_u of soil-cement cured at 7 days and 28 days for soils belonging to ML and A-4 groups should be in the range of 250 psi (1723 kN/ m^2) to 500 psi (3445 kN/m^2) and 300 psi (2067kN/ m^2) to 900 psi (6201 kN/ m^2) respectively. PCA (1956) also recommended that values of qu of soil-Cement cured at 7 days and 28 days for soils belonging to A-7 group should be in the range of 200 psi (1378 kN/ m^2) to 400 psi(2756 kN/ m^2) and 250 psi (1723 kN/ m^2) to 600 psi (4134 kN/ m^2) respectively. It can be seen from Table 4.4 that values of qu of samples of Soil-A (belonging to A-4 group) and Soil-B (belonging to A-7 group) stabilized with 6% to 18% flyash and cured for 14 and 28 days satisfied the requirements of PCA (1956). Ingles and Metcalf (1972), however, recommended that the values of qu of soil-cement road sub-base and base for light traffic should be in the range of 100 psi (689 kN/m^2) to 200 psi (1378 kN/m^2). Table 4.6 also shows that for all flyash contents and all curing ages. the values of qu of treated samples fulfilled the requirements of soil-flyash road sub-base for light traffic as proposed by Ingles and metcalf (1972) with cement. It can also be seen from Table 4.6 that compared with the untreated samples, the values of ε_f of the stabilized samples reduced and that values of ε_f of the treated samples reduced with the increase in flyash content. The relation between qu for samples cured at different ages and flyash contents are presented in Figs 4.10 and 4.11 for Soil-A and Soil-B respectively. Figs 4.12 and 4.13 show the relations between qu and curing period for Soil-A and Soil-B respectively. It can't be seen from Figs 4.10 to 4.13 that the values of qu of treated samples increased with increasing flyash content and curing age. These results are in agreement with those reported by a number of researchers like Mateos and Davidson (1962), Vischosil (1958). It also satisfied with the experiment done by (Ramaswamy et al, 1984; Ahmed, 1984; Hossain, 1986, Hong, 1989; Anon, 1990; Serajuddin and Azmal 1991; Serajuddin 1992; Uddin, 1995)-with cement.

Table 4.6 Unconfined compressive strength test results of Untreated and flyash-treated Soil-A and Soil-B

Additives	Curing Age (Days)	Soil-A	Soil-B
		q_u (kN/m^2)	q_u (kN/m^2)
0	-	715	690
3% Lime	7	1193	1125
	14	1279	1201
	28	1393	1306
3% Lime + 6% Flyash	7	1496	1282
	14	2233	1740
	28	2834	2656
3% Lime + 12% Flyash	7	1872	1496
	14	2728	2341
	28	3406	2943
3% Lime + 18% Flyash	7	2214	1849
	14	3123	2632
	28	3642	3290

The rate of strength gain with curing time has been evaluated in terms of the parameter strength development index (SDI) as defined by the following expression (Uddin, 1995):

$$SDI = \frac{\text{Strength of stabilized sample-Strength of untreated sample}}{\text{Strength of untreated sample}}$$

Posting of SDI with curing age of treated samples of soil-A and Soil-B are shown in Figs. 4.14 and 4.15 respectively. It can be seen from Figs. 4.14 and 4.15 that the values of SDI increase with increasing curing time and cement content as well. These figures clearly content and curing age. Uddin (1995) also reported an increase in SDI with ;increasing curing time and cement content for samples of Rangsit dlay of Bangkok (LL=70 to 117, PI=50 to 78) treated with 5% to 40% cement and cured for 1 week to 40 weeks. As can be seen from Figs 4.14 and 4.15 that the strength gain for samples of Soil-A and Soil-B treated with 6% flyash are relatively much slower than those of samples treated with 12% and 18% flyash that aggress with the Leonerd & Davidson.

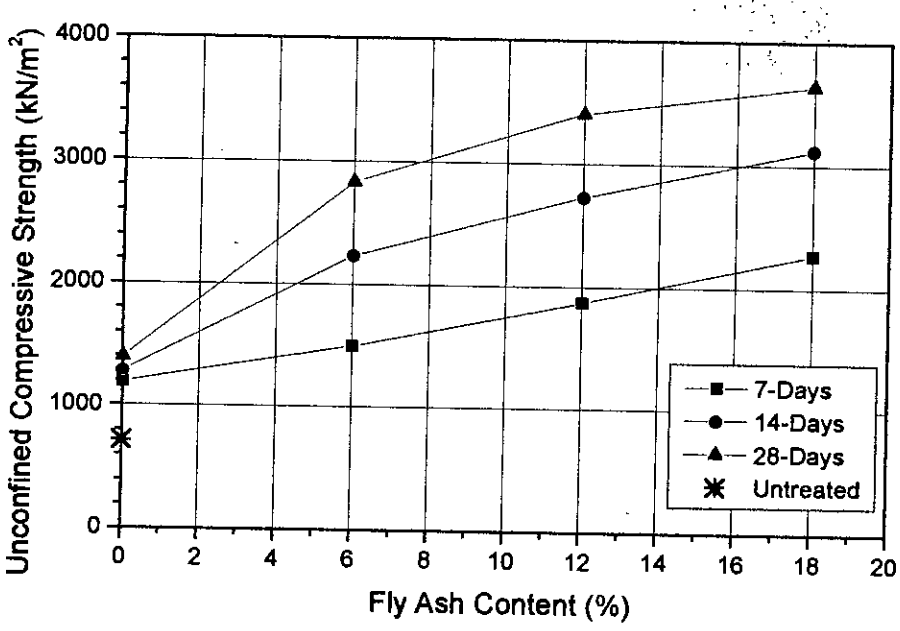

Fig. 4.10 Effect of fly ash on compressive strength of lime and fly ash-treated Soil-A

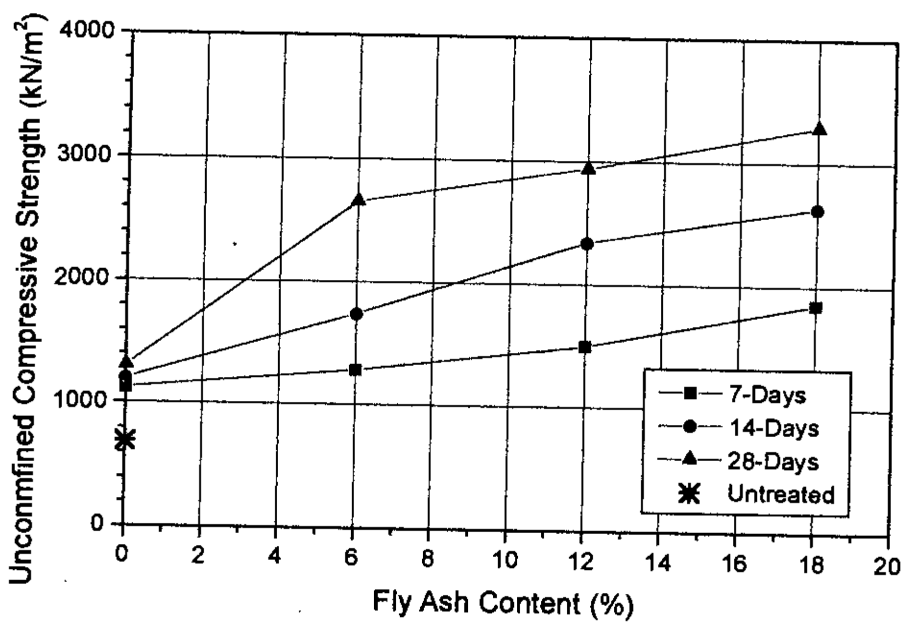

Fig. 4.11 Effect of fly ash content on compressive strength of lime and fly ash –treated Soil-B

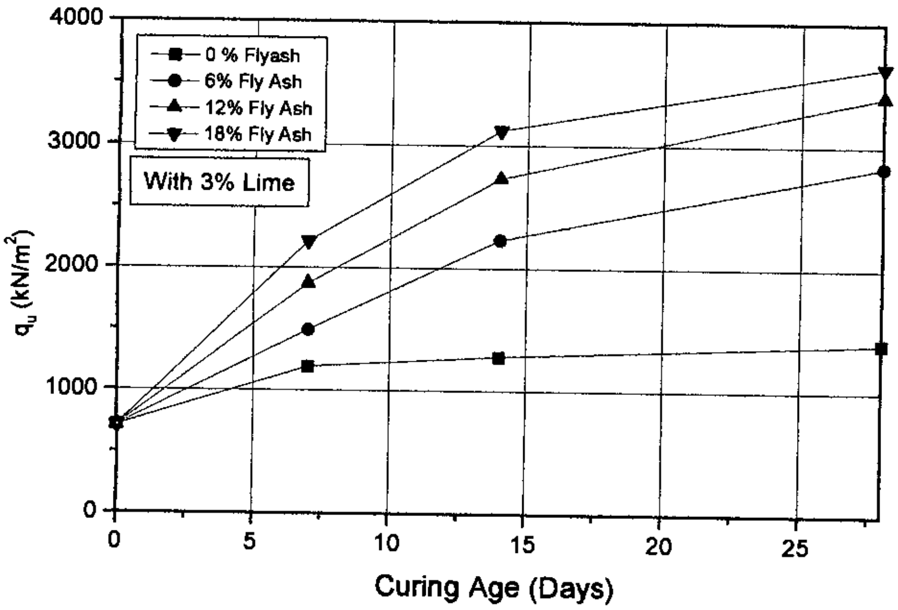

Fig. 4.12 Effect of curing age on unconfined compressive strength of lime and fly ash treated Soil-A

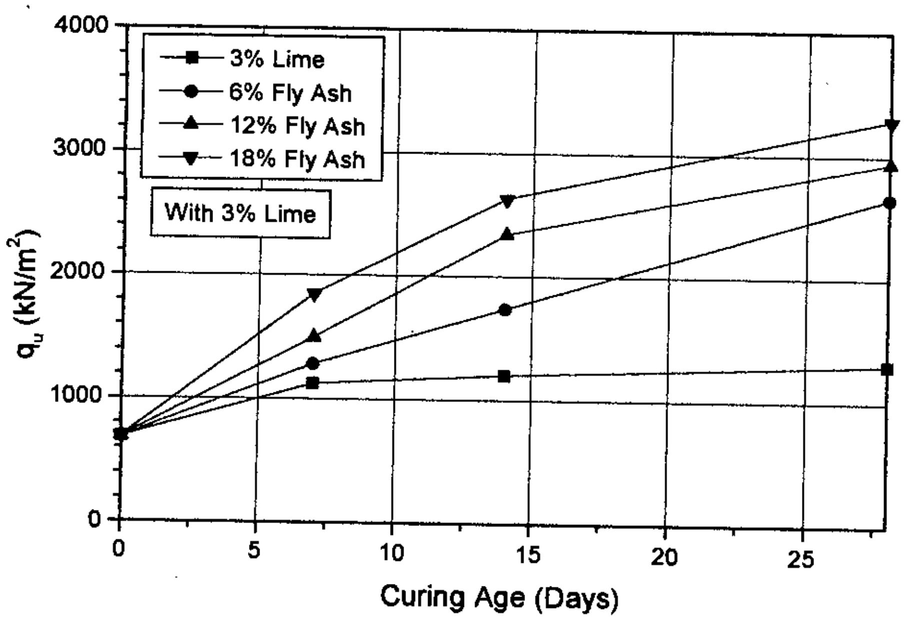

Fig. 4.13 Effect of curing age on unconfined compressive strength of lime and fly ash-treated Soil-B

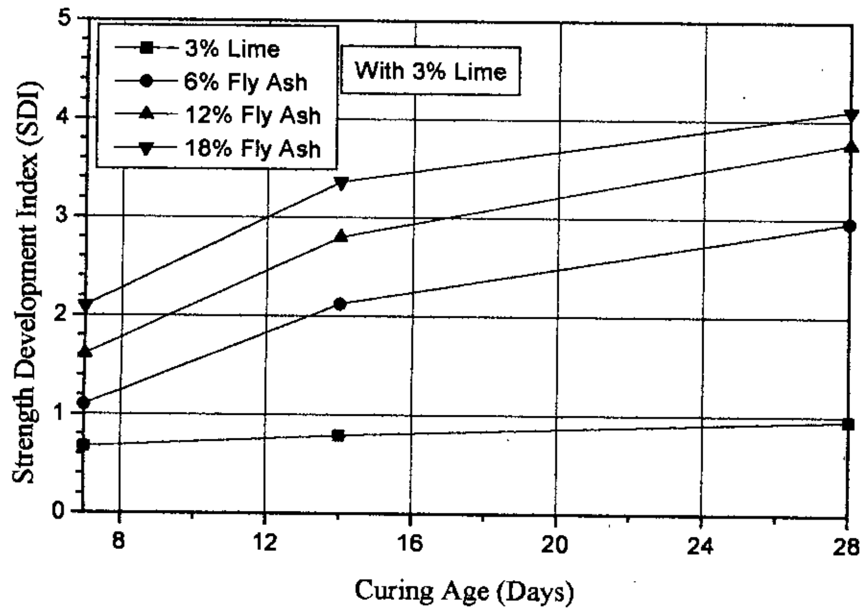

Fig. 4.14 SDI versus curing age curves for fly ash-treated Soil-A

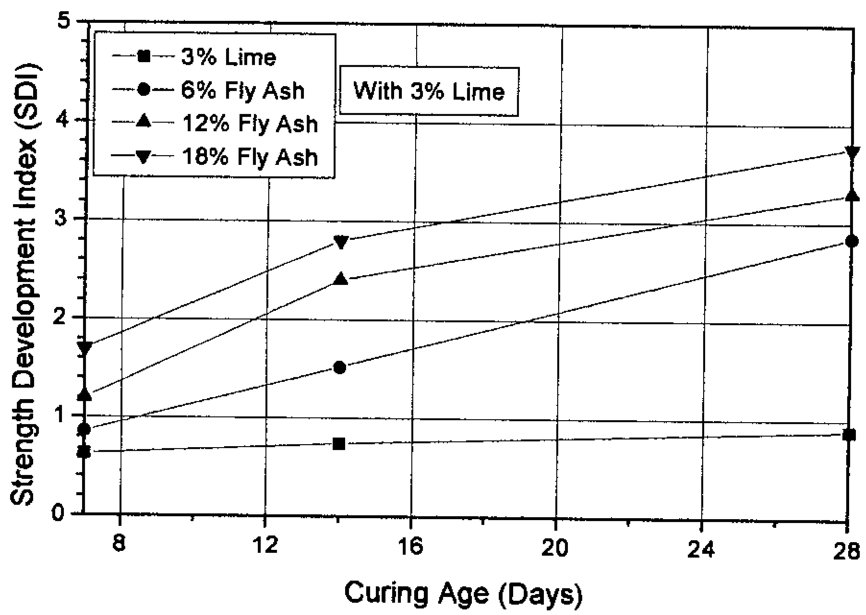

Fig. 4.15 SDI versus curign age curves for fly ash-treated Soil-B

4.2.6 COMPARISON OF UNCONFINED COMPRESSIVE STRENGTH

Table 4.7 shows summary of unconfined compressive strength of regional soils of Bangladesh. The unconfined compressive strength of untreated samples and samples treated different lime and flyash 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 and flyash 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. 4.16 shows the relation of unconfined compressive strength and lime and flyash 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 and flyash content of eleven regional soils are shown in table 4.7 From this table it is shown that for 28 days curing age the unconfined compressive strength increases with the increase in lime and flyash content and ranges from 39.3 kN/m^2 to 345 kN/m^2. The compressive strength of lime and flyash 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 the same. The compressive strength found to be almost same as 7% lime and 18% flyash as additives.

Table 4. 7 Comparison of unconfined compressive strength (28 days) of lime stabilized regional soils of Bangladesh

Soil Code	(%) Lime	Unconfined Compressive Strength (kN/m^2)
SH-A	0	243
	3	353
	5	663
	7	726
SH-B	0	229
	3	233
	5	278
	7	369
SH-C	0	279
	3	1244
	5	1384
	7	1499
H-1	0	550
	3	1100
	6	1820
	9	1930
M-1	0	75.9
	3	346
	5	3710
M-2	0	115
	3	388
	5	652
MI-A	0	715
	3% Lime 6% Ash	2834
	3% Lime 12% Ash	3406
	3% Lime 18% Ash	3642

Soil Code	(%) Lime/flyash	Unconfined Compressive Strength (kN/m^2)
M-3	0	39
	3	107
	5	119
H-A	0	-
	2	171
	4	220
	6	235
H-B	0	-
	2	211
	4	340
	6	408
R-B	0	692
	3	1302
	5	2308
	7	3452
AH-B	0	380
	1	984
	3	2015
	5	2385
	7	2678
MI-B	0	690
	3% Lime 6% flyash	2656
	3% Lime 12% Flyash	2943
	3% Lime 18% Flyash	3290

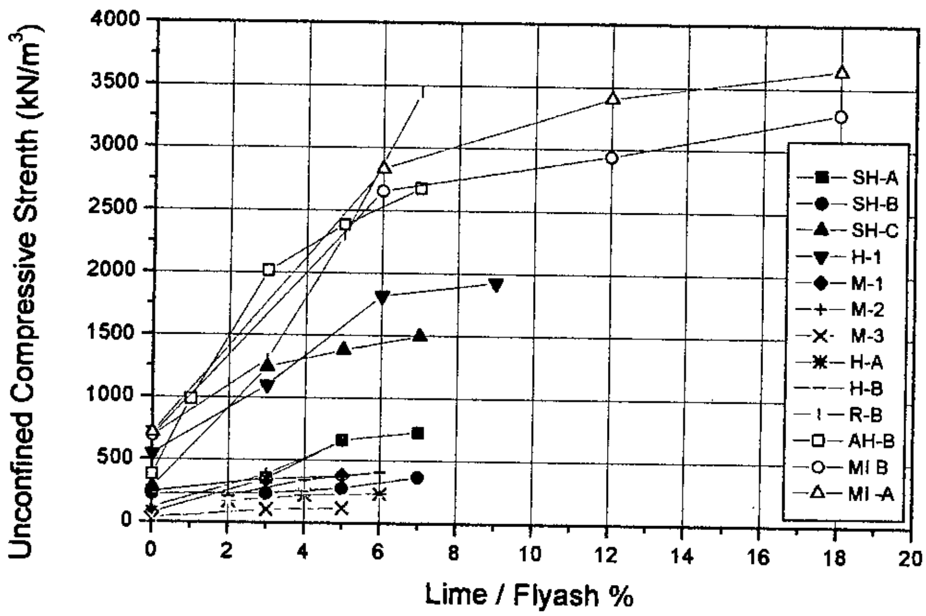

Fig. 4.16 Comparison of the effect of lime stabilization on unconfined compressive strength between different regional soils of Bangladesh and soils used in the present study.

4.2.7 CALIFORNIA BEARING RATIO (CBR)

A summary of the CBR test results for Soil-A and Soil-B is presented in Table 4.8. 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 (10,000 ft-lb/ft3),It can be seen from Table 4.8 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 flyash content for Soil-A and Soil-B are shown in Figs. 4.17 and 4.18 respectively while Figs. 4.19 and 4.20 present the CBR dry density relationships for Soil-A and Soil-B respectively. It can be seen from Figs.4.17 and 4.18 that at all levels of compaction, CBR increases markedly with increasing flyash content while Figs 4.19 and 4.20 show that at any particular flyash content, CBR increase 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 filling sands stabilized with 3%, 5% and 7% cement (BRTC,1995).

It can be seen from Table 4.8 that CBR-values of Soil-A and Soil-B stabilized with 6% flyash increased up to about 4.5 times and 3.7 times those of the respective untreated samples. It is also evident from the CBR data presented in Table 4.8 that the CBR-values of samples of the less plastic Soil-A (PI=7) are moderately higher than those from the samples of more plastic Soil-B (PI=19).

Ingles and metcalf (1972) recommended that four-day soaked CBR-values of soil-cement road sub-base for light traffic should be in the range of 50 to 150. It can be seen from Table 4.8 that CBR of samples of Soil-A and Soil-B treated with 6% flyash and compacted with medium and high energy and that CBR samples of Soil-A and Soil-B treated with 12% flyash and compacted with low to high energy met the requirements of soil-flyash road sub-base and base for light traffic as proposed by Ingles and Metcalf (1972) with cement.

Table 4.8 **Summary of CBR test results of untreated, lime and flyash-treated Soil-A and Soil-B**

Additives (%)	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	11	16	10
	Medium	16	16	17	13
	High	17	24	18	21
3% lime	Low	15	26	15	18
	Medium	16	31	16	23
	High	17	44	17	35
3% lime + 6% fly ash	Low	16	38	16	27
	Medium	17	49	17	37
	High	17	58	18	46
3% lime + 12% fly ash	Low	16	56	17	38
	Medium	17	68	18	54
	High	18	88	19	86
3% lime + 18% fly ash	Low	17	68	16	46
	Medium	18	94	18	77
	High	19	117	19	92

Note: Low compaction energy = 10,000 ft-lb/ft^3

Medium compaction energy = 25,000 ft-lb/ft^3

High compaction energy = 56,000 ft-lb/ft^3

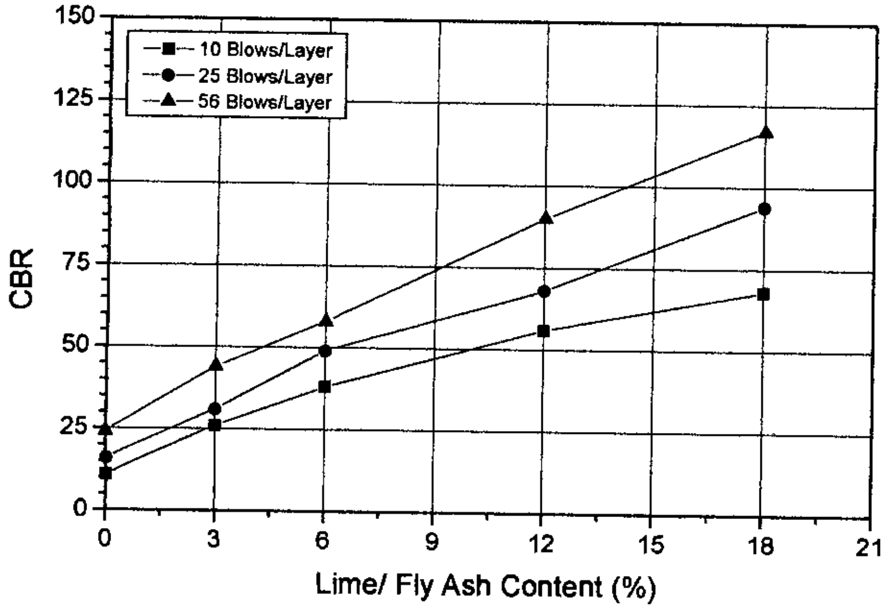

Fig. 4.17 Effect of fly ash content on CBR values of Soil-A

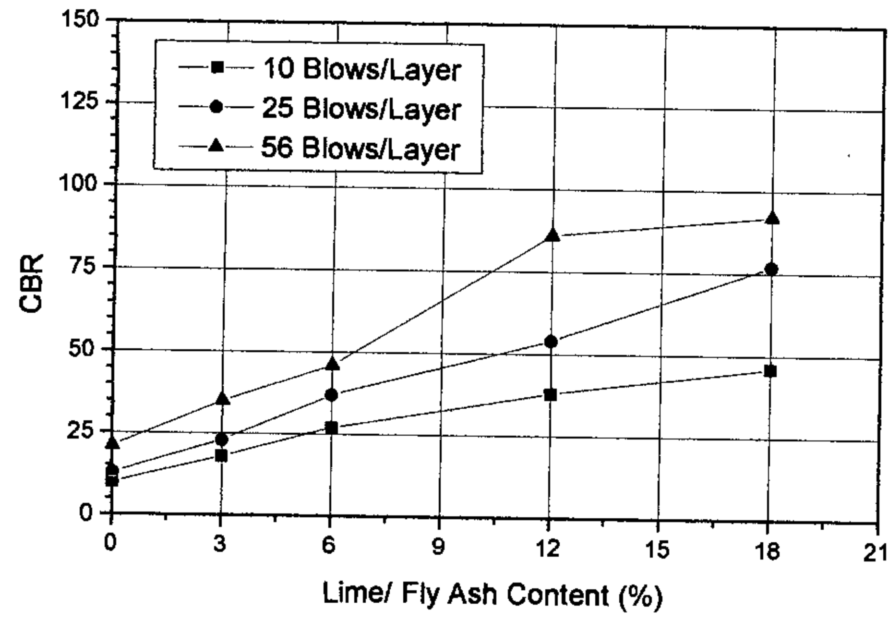

Fig. 4.18 Effect of fly ash content on CBR values of Soil-B

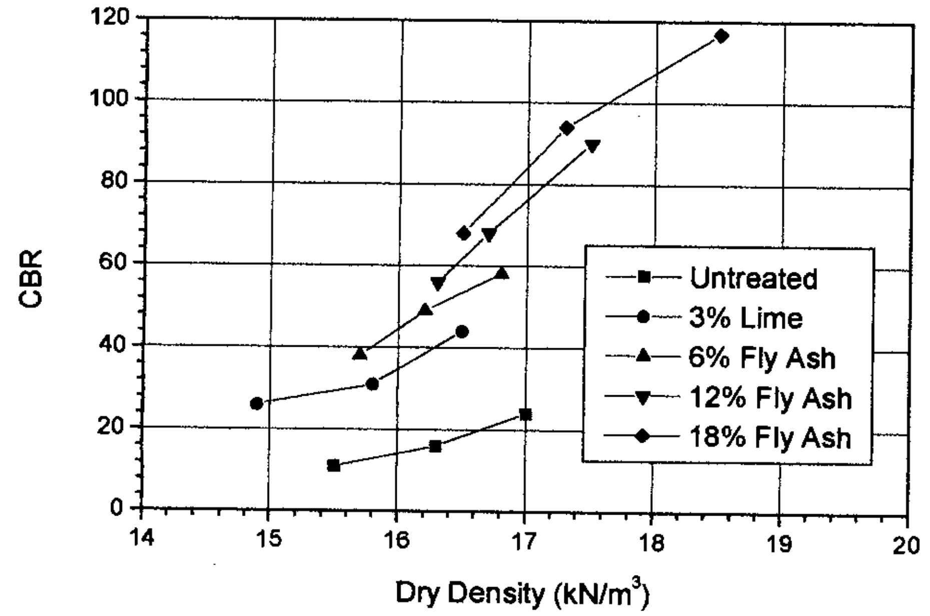

Fig. 4.19 CBR vs dry-density curves of fly ash treated Soil-A

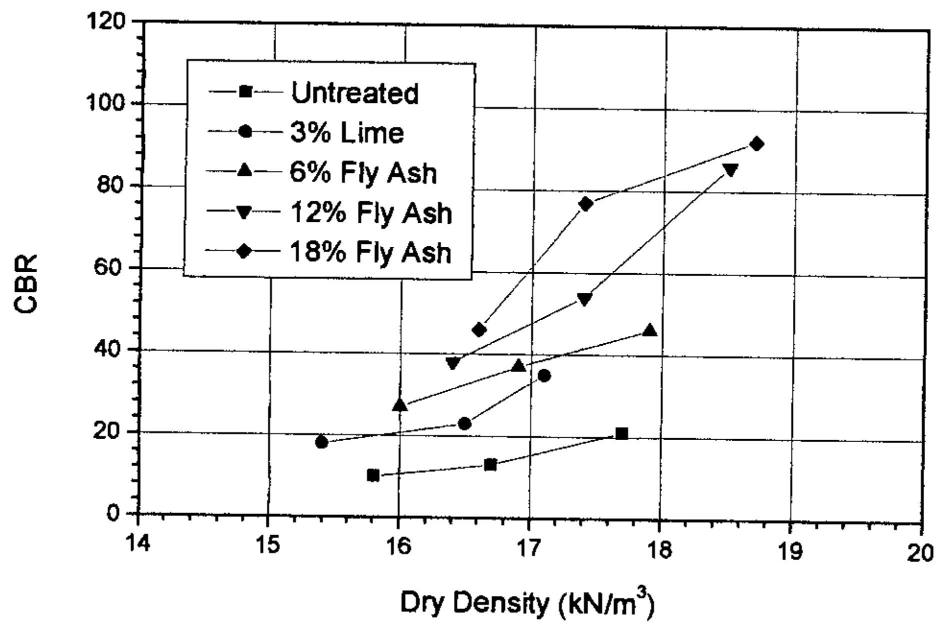

Fig. 4.20 CBR vs dry-density curves of fly ash treated Soil-B

4.2.8 COMPARISON OF CALIFORNIA BEARING RATIO (CBR)

A comparative study of CBR test for three levels of compaction energy and 4 days soaking time of regional soils are represented in table 4.9 In these tests up to 9% lime and 18% flyash are used. It shows that, compared with the untreated soils, CBR values of the treated soils at all levels increase considerably with increase in lime and flyash content. The variations of CBR with lime and flyash content for regional soils are shown in Fig. 4.21

It can be seen from Table 4.9 that the CBR values of the regional treated soils with lime 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. Nine samples are taken for comparison of CBR values for different percent of lime content ad different level of compaction energy. From the table 4.9 it can be shown that the CBR values increases with the increase in lime and flyash content and ranges from 4 to 69 and 92 respectively. 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. Where as it increases almost 10 times when treated with 18% flyash.

Table 4.9 Comparison of CBR values of lime stabilized regional soils of Bangladesh

Soil code	Lime Content (%)	CBR Value		
		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
MI-A	0	11	16	24
	3% Lime/ 6% flyash	38	49	58
	3% Lime 12% flyash	56	68	88
	3% Lime 18% flyash	68	94	117
MI-B	0	10	13	21
	3% Lime/ 6% flyash	27	37	46
	3% Lime 12% flyash	38	54	86
	3% Lime 18% flyash	46	77	92

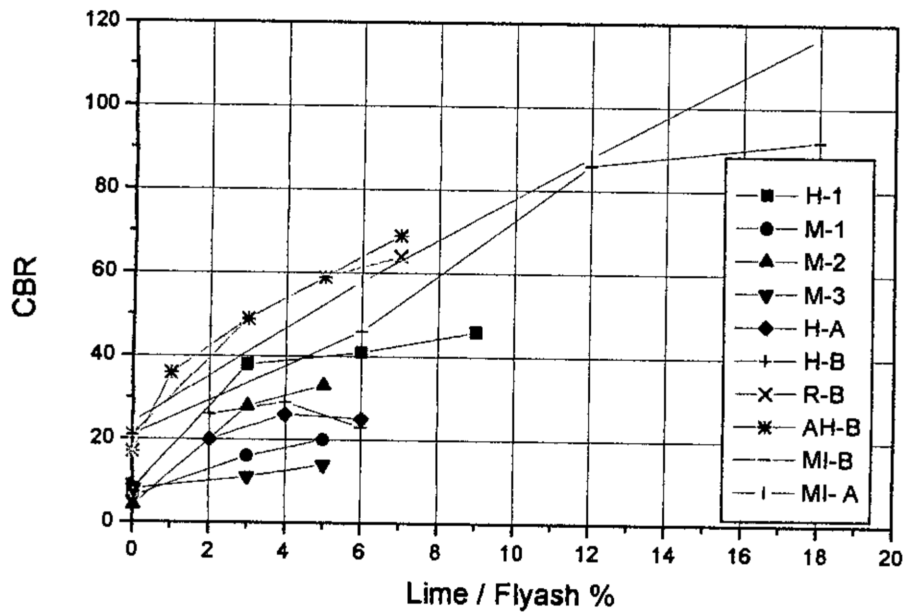

Fig: 4.21 Comparison of the effect of lime stabilization on CBR between different regional soils of Bangladesh and soils used in the present study.

4.2.9 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 is presented in Figs.4.22 and 4.23 respectively. It can be seen from Figs. 4.22 and 4.23 that flexural stress-deflection curves are approximately linear. 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 4.10 and 4.11 respectively. It can be seen from Tables 4.10 and 4.11 that for both Soil-A and Soil-B, compared with the untreated sample, flexural strength and modulus of the treated samples cured at 7, 14 and 28 days increased significantly. It can be seen from Table 4.10 that compared with the untreated sample, the flexural strength and modulus of Soil-A treated with 6%, 12% and 18% flyash and cured at 28 days are respectively about 1.5, 3, 4.6 times and 1.8, 2 and 3 times higher respectively. Table 4.11 shows that the flexural strength and modulus of Soil-B treated with 6%, 12% and 18% flyash and cured at 28 days are respectively about 2, 1.5, 6.7 times and 2.6, 3, 4.4 times higher respectively than those of the untreated samples. The maximum deflection and of untreated and stabilized soil-flyash beams were in the range of 0.15 mm to 0.35 mm respectively. 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(PI=19) is higher than the less plastic Soil-A (PI=7).

The effect of flyash content on flexural strength for Soil-A and Soil-B are shown in Figs.4.24 and 4.25 respectively while Figs.4.26 and 4.27 present the effect of flyash content on flexural modulus of Soil-A and Soil-B respectively. Figs. 4.24 to 4.27 show that flexural strength and modulus increases with increasing flyash content. It is evident from Figs.4.24 to 4.27 that curing age has got insignificant effect on increase in flexural strength and modulus.

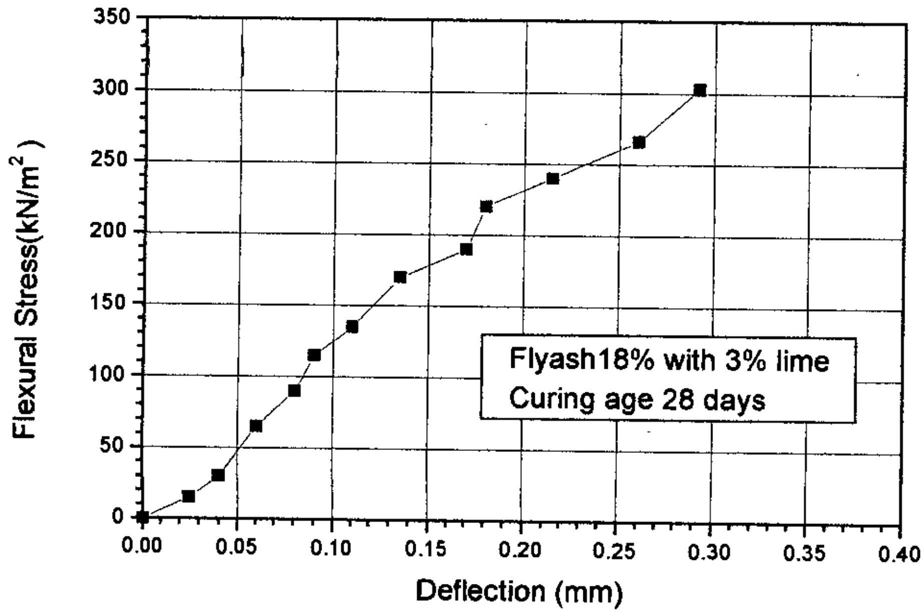

Fig. 4.22 Flexural stress versus deflection curve of fly ash-treated Soil-A

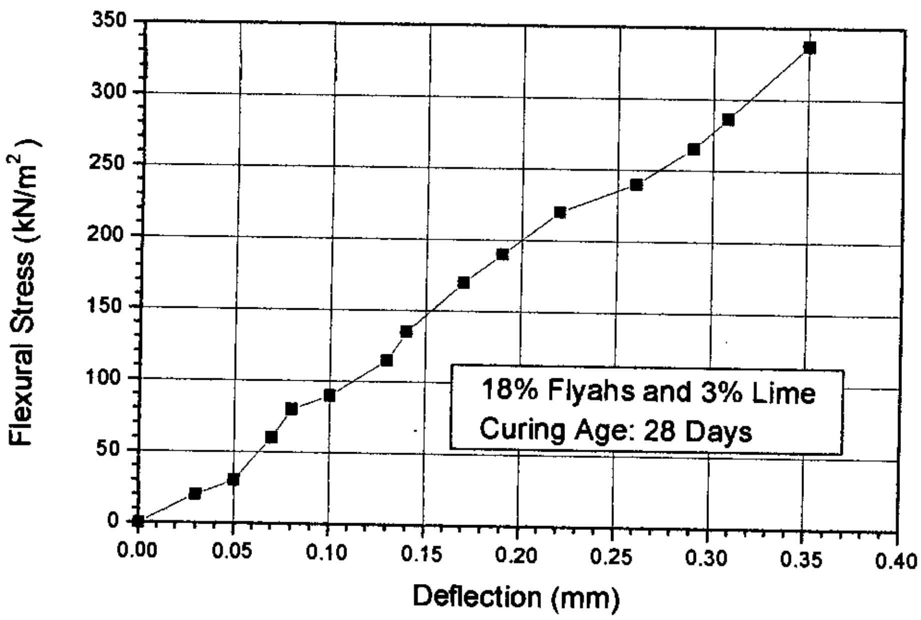

Fig. 4.23 Flexural stress versus deflection curve of fly ash-treated Soil-B

Table 4.10 Flexural properties of untreated and flyash-treated Soil-A

Flyash/Lime Content (%)	Curing Age (Days)	Flexural Strength kN/m^2	Maximum deflection (mm)	Flexural Modulus (MPa)
0	-	65	.198	47
3% lime	7	71	.1778	59
	14	75	.1651	61
	28	82	.152	64
6% flyash + 3% lime	7	83	.144	81
	14	88	.152	84
	28	101	.1651	85
12% flyash + 3% lime	7	120	.1905	86
	14	145	.21	92
	28	202	.254	98
18% flyash + 3% lime	7	243	.2667	122
	14	257	.279	131
	28	303	.304	143

Table 4.11 Flexural properties of untreated and flyash-treated Soil-B

Flyash Content (%)	Curing Age (Days)	Flexural Strength kN/m^2	Maximum deflection (mm)	Flexural Modulus (MPa)
0	-	50	.226	33
3% lime	7	57	.1728	49
	14	61	.176	51
	28	64	.179	54
6% flyash + 3% lime	7	88	.152	71
	14	94	.152	83
	28	100	.152	88
12% flyash + 3% lime	7	121	.20	83
	14	137	.20	100
	28	162	.2286	105
18% flyash + 3% lime	7	275	.30	119
	14	293	.33	128
	28	337	.35	140

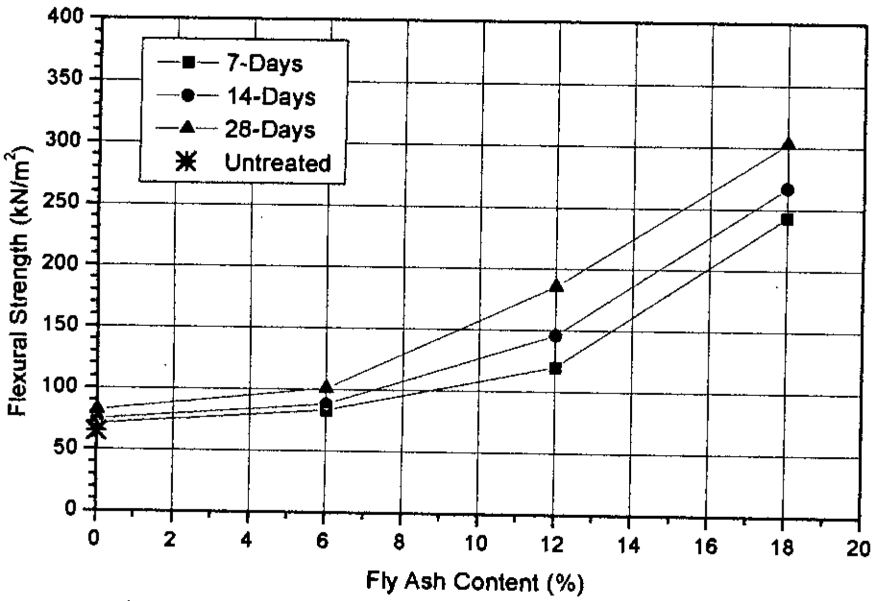

Fig. 4.24 Effect of fly ash content ob flexural strength of Soil-A

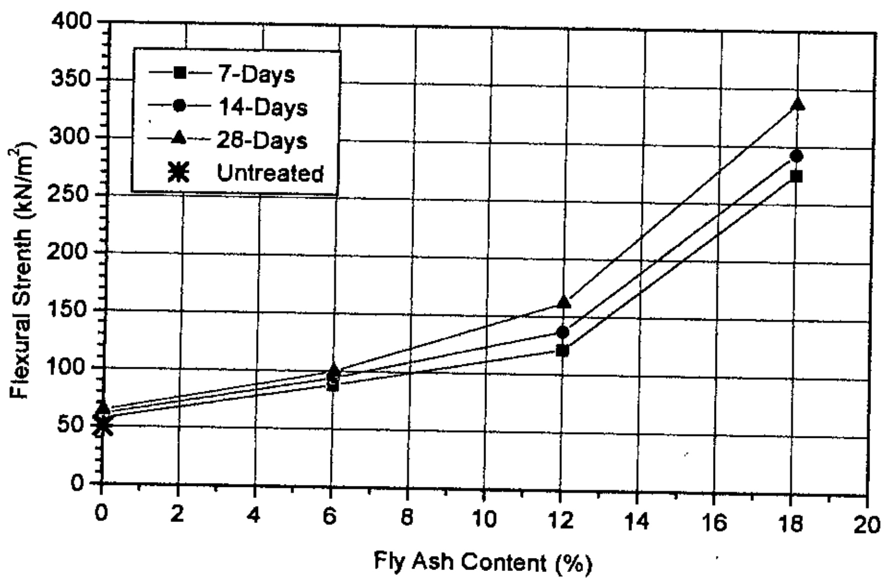

Fig. 4.25 Effect of fly ash content of flexural strength of Soil-B

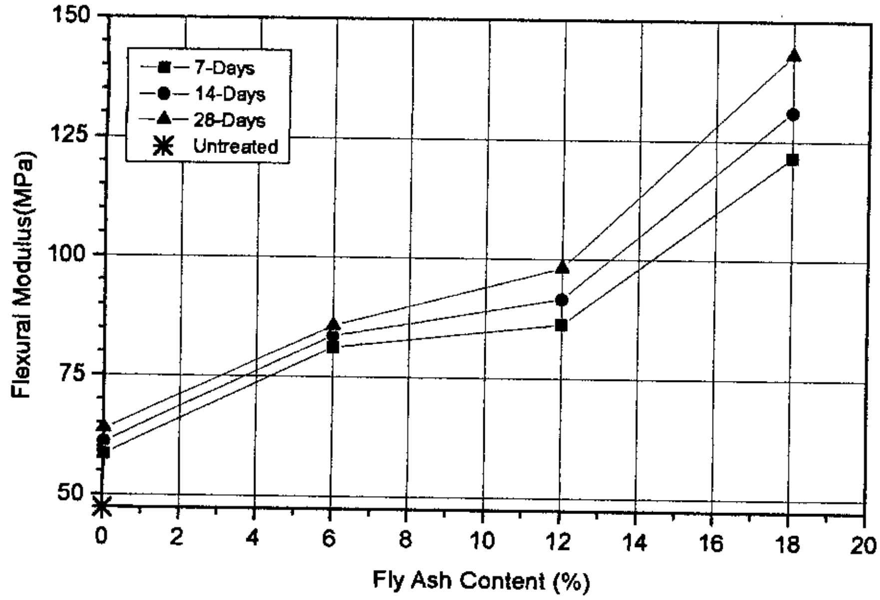

Fig. 4.26 Effect of fly ash content on flexural modulus of Soil-A

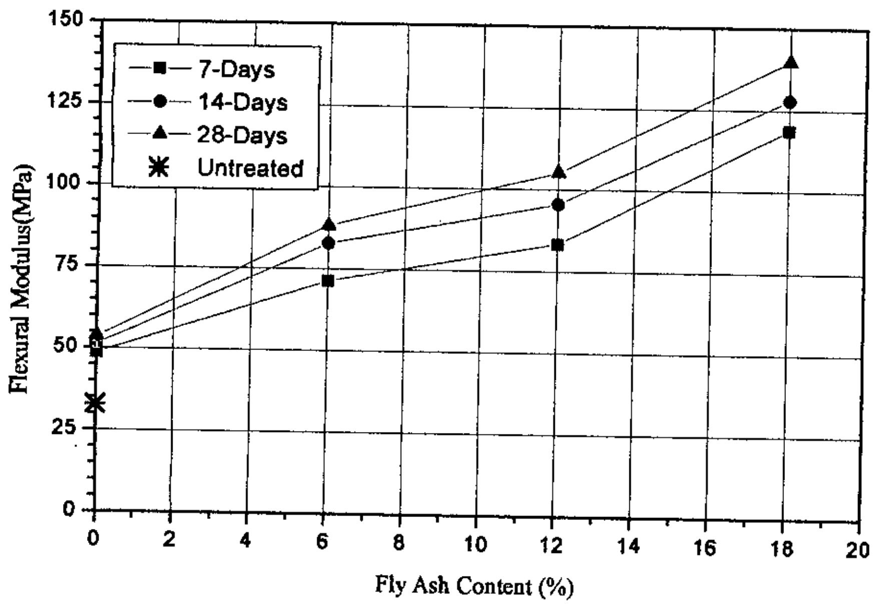

Fig. 4.27 Effect of fly ash content on flexural modulus of Soil-B

4.2.10 COMPARISON OF FLEXURAL STRENGTH AND MODULUS

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 4.12 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. 4.28 while Fig. 4.29 shows the effect of lime content on flexural modulus for regional soils. The ranges of flexural strength are between 47 kN/m^2 to 243 kN/m^2 and flexural modulus varies between 23 MPa to 71 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 4.12 Comparison of flexural properties (28 days) of lime stabilized regional soils of Bangladesh

Soil code	Lime/Flyash (%)	Flexural Strength (kN/m^2)	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
MI-A	0	65	47.2
	3	82	63.9
	6	101.23	85.3
	12	202	98.2
	18	303	143
MI-B	0	50.0	32.8
	3.0	64.0	53.7
	6	100.0	88.2
	12	162.0	105.03
	18	337	139.6

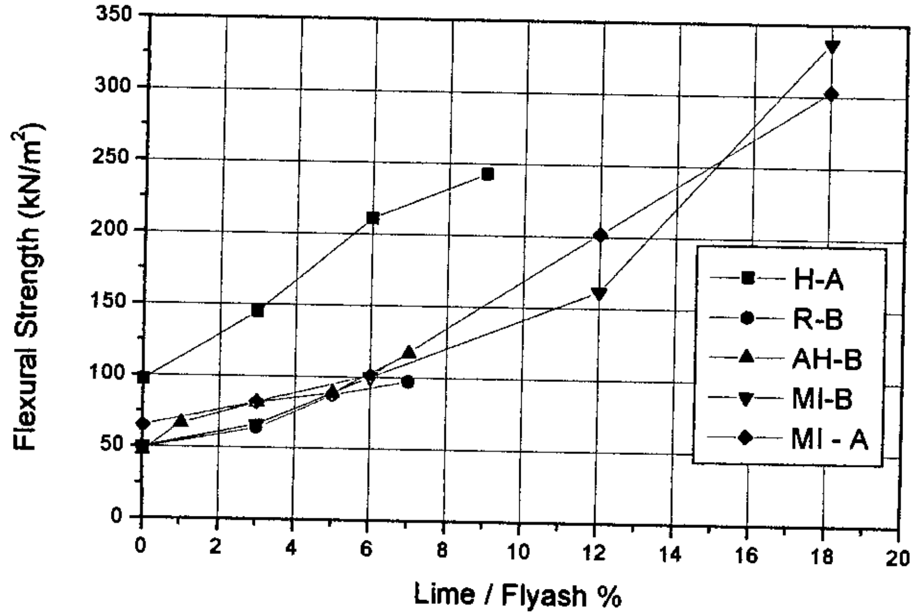

Fig: 4.28 Comparison of the effect of lime stabilization on flexural strength between different regional soils of Bangladesh and soils used in the present study.

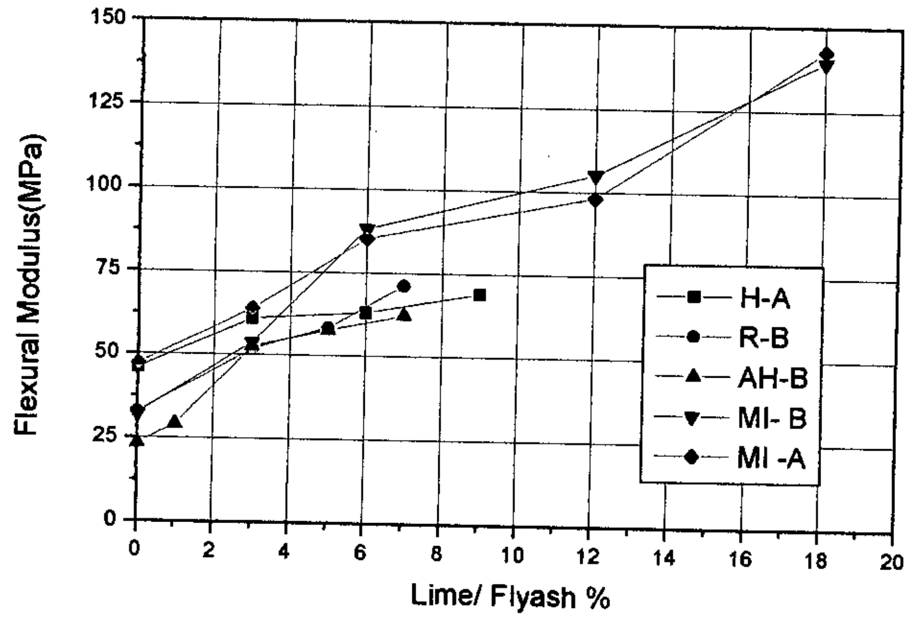

Fig: 4.29 Comparison of the effect of lime stabilization on flexural modulus between different regional soils of Bangladesh and soils used in the present study.

CHAPTER-5

CONCLUSION AND RECOMMENDATIONS FOR FUTURE STUDY

5.1 CONCLUSIONS

In this research work, flyash stabilization of two selected soil (collected from Anwara and Banshkhali) of Chittagong coastal region have been carried out . Flyash has been used in percentage of 6.12 and 18 while lime has been added in percentages of 3 as additives with and without flyash. The physical and engineering properties of flyash and lime stabilised soil have been determined in order to asses the suitability of flyash and lime stabilization further use in road construction. The major findings and conclusions have been separated into three sections relating to the following areas:

(1) The influence of flyash with lime stabilization on the physical and engineering properties on samples of the two coastal soil from Anwara (i.e, Soil-A) and Banshkhali (i.e.,Soil-B) from two selected location of Chittagong coastal region.

(2) The effect of lime stabilistion on the physical and engineering properties on samples.

5.1.1 INVESTIGATIONS ON THE EFFECT OF FLYASH STABILISATION

This section presents the findings and conclusions obtained from the experimental investigation of the influence of flyash stabilization on smaples of the two coastal soils studied. The major findings and conclusions may be summarized as follows :

(i) Compared with the untreated samples of Soil-A and Soil-B, plstic limit of the stabilised 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 reducted in case of Soil-B. For Soil-A (LL= 30 PI =7) both liquid and plastic limit increased while for Soil-B (LL=44, PI=19) liquid limit reduced and plastic limit increased with increased flyash content.

(ii) For samples of both the soils, compared with the untreated samples, the values of maximum dry density (ϒmax) increased with flyash content and decreased with 3% lime while the values of optimum moisture content (opt) reduced. Compared with the untreated sample, the values of ϒmax increased up to 7.5% for both the soil with 18% flyash. The values of Wept reduced upto 15% and 10% respectively for samples of Soil-A and Soil-B.

(iii) For samples of both the coastal soils, compared with the untreated samples, the values unconfined compressive strength (qu) of the treated samples increases significantly, depending on the flyash content and curing age. The values of qu of samples of Soil-A and Soil-B treated with 6% and 18% flyash and cured at 28 days were found to be about 4 and 5 times higher than the strength of he untreated samples respectively. It has also been fount that the gain in strength with increasing flyash content and curing age is higher in the less plastic Soil-A(PI= 7) than in the more plastic Soil-B (PI=19). Compared with the untreated samples, the values of axial strain at failure (ε_f) of the stabilised samples reduced with the increase in flyash content which evidently indicated that the treated samples became more brittle as flyash content increased. The rate of strength gain with curing time (determined in terms of strength Development Index, SDI) for samples of Soil-A and Soil-B treated with 6% flyash are relatively much slower than those of samples treated with 12% and 18% flyash.

It was found that values of qu of samples of Soil-A (belonging to A-4 group) and Soil-B (belonging to A-7 group) treated with 6%, 12% and 18% flyash with 3% lime and cured for 7,14 and 28 days satisfied the requirements of PCA (19956) for the unconfined compressive strength of soil-flyash mix.

(iv) Compared with the untreated sample, CBR-values of the treated samples for both Soil-A and Soil-B increased considerably irrespective of the level of compaction effort. It was found that the CBR-values of Soil-A and Soil-B stabilized with 6% flyash increased up to about 4.5 times and 3.7 times than those of the respective untreated samples. It has also been observed that the CBR-values of samples of the less plastic Soil-A (PI=7) are moderately higher than those for the samples of more plastic Soil-B PI=19. Comparing with the criteria for four-day soaked CBR-values of soil-flyash mix for use in road sub-base and base subjected to light traffic as proposed by Ingles and Metcalf (1972).

(v) The flexural stress versus deflection curves have been found to be approximately linear 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 flyash content. For comparison, the flexural strength and flexural modulus of Soil-A treated with 18% flyash and cured at 28 days are respectively about 4 times and 2.7 times higher than those for the untreated sample. The flexural strength and modulus of Soil-B treated with 18% flyash and cured at 28 days are respectively about 6 times and 4.3 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 (PI=19) is higher than the less plastic Soil-A (PI=7). The maximum deflection and failure strain of untreated and stabilised soil-flyash beams were very small and have been found in the range of 0.15 mm to 0.35 mm and 0.101% to 0.240% respectively.

From the aforementioned findings, it is evident that for both samples of the two coastal soils studied, flyash stabilization provided a substantial improvement in the engineering properties as compared with the samples of the untreated soils. It has also been found that, in general, samples of both the soils stabilized with 12% and 18% flyash with 3% Limes satisfied the requirements of compressive strength, CBR and durability for their use as base or sub-base materials in roads subjected to light traffic.

5.2 RECOMMENDATION FOR FUTURE STUDY

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

(1) The present study was carried out on samples collected from the two selected location of Chittagong coastal region. Similar investigations may be carried out with soils collected from other coastal regions of Bangladesh Like Cox's Bazar, Bhola, Barisal, Mongla and the results may be compared with those obtained in the present investigations.

(2) In this research work, samples of the two coastal soils were stabilized with a maximum of 18% flyash and 3 % 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 percentage of flyash and lime in order to evaluate the optimum additive content which fulfills all the necessary requirements for their use in road construction in rural areas.

(3) In this research work, the stabilized samples were cured for a maximum ages of 28 days while investigation their engineering properties. The influence of long term curing age (at least up to 90 days) on engineering properties of the stabilized samples, particularly flyash treated samples of the two soils studied could be investigated

(4) In this investigation, flyash and Lime have been used as additives for stabilization. Investigations of the physical and engineering properties could be carried out by stabilizing the soils studied with other additives (flyash+cement and only flyash) in order to assess the most suitable type of additive for stabilizing these coastal soils for use in road construction.

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ANNEX A

Soil Type	Compressive Strength kN/ m^2	
	7 days	28 days
Sandy and Gravelly soils: AASHO group A-1, A-2, A-3 Unified group GW, GC, FP, GM, SW, SC. SP, SM	2067-4134	2756-6890
Silty soils: AASHO group A-4, A-5 Unified group ML and CL	1723-3445	2067-6201
Clayey soils: AASHO group A-6, A-7 Unified group MH, CH	1378-2752	1723-4134