STABILIZATION OF SUBGRADE SOILS

By

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A Project Report
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ABSTRACT

Two subgrade soils were stabilized using Portland Cement and lime admixtures. The whole research work was conducted for the purpose of evaluating the subgrade strength and the cost effectiveness of stabilizer usage.

With the variation of cement and lime, from 2 percent to 10 percent, stabilized samples were prepared at their maximum dry densities and optimum moisture contents. They were cured and tested for evaluating durability, volume and moisture change characteristics, unconfined compressive strength and California Bearing Ratio (CBR).

The results obtained show that cement treated soils satisfy the durability criteria recommended by the Portland Cement Association (PCA) at about 2 percent and 7 percent cement content respectively. Both the soils fail to satisfy the unconfined compressive strength criteria of PCA for cement content at which the durability criteria is satisfied. Unconfined compressive strengths of lime treated soils increase due to addition of lime.

The CBR value increases at an increasing rate for higher cement contents. This value increases slightly after addition of lime.

A correlation between Dynamic Cone Penetrometer (DCP) and CBR was developed in order to provide a quick evaluation technique of compacted subgrade.

It is found that the construction of pavement on a stabilized subgrade will be economical than that on an untreated subgrade.

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ABBREBIATIONS

CBR	-	California Bearing Ratio
PCP	-	Portland Cement Association
DCP	-	Dynamic Cone Penetrometer
IRC	-	Indian Road Congress
ASTM	-	American Society for Testing Materials
AASHO	-	American Association for SState Highway Officials
AASHTO	-	American Association for State Highway and Transportation Officials
BCL	-	Bangladesh Consultants Limited
BRRL	-	BangladeshRoad Research Laboratory
RHD	-	Roads and Highways Department
TRRL	-	Transport Road Research Laboratory

CHAPTER 1

INTRODUCTION

1.1 General

A soil exhibiting a marked and sustained resistance to deformation under repeated or continuing load application, whether in dry or wet state, is said to be a stable soil. When a less stable soil is treated to improve its strength and its resistance to change in volume and moisture content, it is said to be stabilized. Thus stabilization infers improvement in both strength and durability. In its earlier usage, the term stabilization used to signify improvement in a qualitative sense only. More recently stabilization has become associated with quantitative values of strength and durability, which are related to performance. These quantitative values are expressed in terms of compressive strength, shearing strength or some measure of load bearing value. These in turn indicate the load bearing quality of the stabilized construction. Again the durability indicates its resistance to freezing and thawing and wetting and drying.

Stabilization as used in road construction is a method of processing available materials for the production of low-cost roads. In this type of design and construction, emphasis is usually placed upon the effective utilization of local materials with a view to decreasing the construction cost. In some areas naturally occuring soils require a minimum of processing for successful stabilization, while in other places the natural soils are of unfavorable character and require modification through the use of suitable stabilizer such as cement, lime, bitumen etc.

According to Winterkorn (1975), soil stabilization is a collective term for any physical, chemical or biological methods, employed to improve certain properties of a natural soil to make it serve adequately for an intended engineering purpose.

1.2 Soil Stabilization Techniques

There are several methods of soil stabilization in use. The degree of improvement of in situ soil may differ within a particular method and also between the other methods. The reason behind is that soils exist in a broad range of types and different soils react differently to a stabilizer.

The available important methods are:

i) Mechanical stabilization

ii) Cement stabilization

iii) Lime stabilization

iv) Bitumen stabilization

v) Electro-osmosis

vi) Thermal stabilization

vii) Chemical stabilization.

Mechanical stabilization is sometimes termed as granular stabilization. In this process, gradation of soil-aggregate mixture is the only factor which controls the stability of the resulting construction. The basic principles involved in mechanical stabilization are `proportioning´ and `compaction´. Stability and strength of granular materials having negligible fines when mixed with clay and compacted, can be improved by this technique. Similarly, the stability of the clayey soil can be improved by mixing a proper proportion of granular materials in it.

Cement stabilization has been used successfully to stabilize granular soils, sands, silts and medium plastic clays. Details of cement stabilization will be discussed later.

Lime stabilization has been in use to stabilize clayey soils. Lime depends for its action on pozzolanic materials in the soils. These normally consist of clay minerals and amorphous compounds. Lack of these materials in pure sands and granular soil, makes lime stabilization ineffective for them. Addition of lime to a soil generally results in decreased soil density, changed plasticity properties and increased soil strength. This method is discussed latter.

Bitumen when mixed with soil imparts binding property and makes it waterproof. Water proofing property imparted to the soil helps in retaining its strength even in the presence of water. In the case of fine grained soil, bituminous materials seal the voids between the small soil clods and keep soil away from coming in direct contact with water and thus inherent properties of the soil are retained. In the case of soils like sand and gravel, individual particles get coated with a very thin film of bituminous materials and thus impart binding property in the soil.

The electrical stabilization technique is also known as electro-osmosis. The process involves sending a direct electric current through a saturated soil. This flow of current results in movement of water towards the cathode end from where it is pumped out. Thus the soil is consolidated with decrease in volume. This consolidation increases the strength of the soil appreciably.

By thermal treatment, soil can be stabilized for expediting construction facility. A reliable temporary expedient to facilitate construction of open and underground excavation is stabilizing the soil by freezing the pore water. When a clayey soil is heated, there is a progressive hardening. The resultant effect is improvement of certain properties of soil like plasticity index, swelling properties, strength compressibility and durability. The method is uneconomical for stabilizing in-situ soils.

By chemical grouting, it is possible to stabilize fine sands and silts. Groute fill the pores of these soils resulting in stabilized material.

1.3 Soil-Cement Stabilization

Soil-cement stabilization is the process in which cement is used as an admixture. The strength of the soil is increased and it becomes resistant to softening by water. This improvement in the quality and bearing capacity of the soil at a reasonable cost make it more desirable and efficient in comparision to other methods of stabilization.

Though history of stabilization using admixture dates back to early civilization of Mesopotamia and Babylon and more recent Roman civilization, in modern times, it was in South Carolina, USA in the 1935, that a highway engineer innovated this method of stabilization. Since then about 50 millions sq. yds. of soil-cement pavement including roads, runways, car-parks and similar construction have been made in U.S.A. alone. Soil-cement construction in Britain exceeded 6,60,000 sq. yds. in 1950, half of which had been constructed since the second World war. These include building blocks, foundation for houses, housing roads and sub-base of major roads (Road Research Laboratory, 1952). Today soil-cement stabilization is used in many developed and developing countries in the tropical and arctic regions of the world (Kezdi, 1979).

Cement stabilized soil road has always been a topic of discussion in road construction system but tried limitedly in practice in this country to check its suitability in adaptation and performance.

1.4 Soil-Lime Stabilization

Lime stabilization has been successfully used to stabilise clayey soils. The clay minerals carry a negative charge on the surface which has adsorbed cations of sodium, magnesium, potassium or hydrogen, and to a large extent responsible for imparting plasticity to the soil sample. Lime brings changes in the plasticity properties of a soil, it increases soil contact bonds and hence the strength.

In India, many masonry dams were constructed by using lime-surki-mortar, which was a mixture of lime, burnt clay and sand. Many of these old structures have been giving satisfactory service even to this days (IRC, 1976).

In spite of man's early used of lime as a stabilizing material, the scientific knowledge regarding the use of lime-soil as a significant engineering practices is only of recent origin. The pioneering work in this connection was done in the U.S.A. (IRC, 1976).

1.5 Need of Soil Stabilization for Road Construction in Bangladesh

Flood plain occupies roughly seventy percent of the total land area in Bangladesh (Bangladesh Transport Survey 1974). The flood plain deposits are of recent origin. These deposits consist of soils of alternate repeated layers of clays, silt and sands. Major portion of this deposit is inundated by seasonal flooding every year. As a result, the sub-soil becomes soft and has low density and shear strength. Presence of ground water table close to the surface in other times of the year except flood time also contributes to lower the density and bearing value of the sub-soil. Due to low topography, during road construction in most of the land surface, earth fillings are necessary. Fill soils are generally excavated from nearby borrow pits. Most fill soils have inadequate shear strength to support the traffic loads applied on them. Also, prolonged rainfall seriously impairs the stability of these soils. In order to serve adequately, it is essential to improve their strength characteristics.

The conventional practice of constructing earth roads in rural areas is to dump the loose soil over the road formation and to render a nominal compaction. This road is subsequently exposed to rain and monsoon flood. This together with inadequate compaction seriously impair the durability of earth roads.

For uplifting rual masses, communication is a must. If the rural masses are to join the main stream of the more previleged urbanities, the most essential pre-requisite would be to provide an adequate network of roads. With limitations, it is essentil that roads are to be constructed in stages. The way is to be found out to proivide low cost roads in rural areas.

Soil is normlly the foundation material for any road, if the load bearing capacity of soil is improved by any suitable means then a lower thickness of road structure is needed, and eventually road construction would be economical. Cement and lime treatments of the in-situ soil are some of the effective methods to cope with the above problem.

1.6 Objective of the Research

Cement stabilized soil road has always been a topic of discussion throughout the world, but a very little effort was exercised in practice in Bangladesh to check its suitability in adaptation and performance characteristics. Stabilization of subgrade soil is not in practice in our country. Review of literatures shows a deficiency of knowledge with regard to the application of stabilization technique in this country. The objectives of the research therefore as follows:

i) to find out the effect of stabilizer such as cement and lime in order to have an improved subgrade.

ii) to develop a quick evalaution techniques of compacted subgrade.

iii) to have comparative economic analysis of pavement using untreated and treated subgrade soils.

CHAPTER 2

LITERATURE REVIEW

2.1 General

The properties of the stabilized soils are influenced by a number of important factors, such as mineralogy of the soil, quality and amount of admixtures, soil properties, compactive effort, condition following addition of admixture and curing period. In this chapter, a brief review is made on the mechanism of cement and lime stabilization, important aspects of properties of stabilizsed soil, factors influencing the mechanism of stabilization and probable effect of stabilization on the properties of soil. A summary at the end of the chapter briefs the detailed discussion of the chapter.

2.2 Basic Principles of Soil-Cement Stabilization

Addition of inorganic stabilizers like cement and lime have two fold effect on soil-acceleration of flocculation and promotion of chemical bonding. Due to flocculation, the clay particles are electrically attracted and aggregated with each other. This results in an increase in the effective size of the clay aggregation (Jha, 1977). Ingles (1968) asserted that such aggregation converts clay into the mechanical equivalent of a fine silt. Also a strong chemical bonding force develops between the individual particles in such aggregation. The chemical bonding depends upon the type of stabilizer employed.

When water is added to cement the major hydration products are basic calcium silicate hydrates, calcium aluminate hydrates, and hydrated lime. The first two of these products constitute cementitious compounds, while the lime is deposited as a separate crystaline solid phase. They are also responsible for strength gain of soil-cement mix (O'Flaherty, 1974).

The interaction between cement and soil differs somewhat for the two principal types of soil, granular and cohesive.

In granular soils, the cementation effect is similar to that in concrete, the only difference being that the cement paste does not fill the voids of the additives, so that the latter is only cemented at contact points.

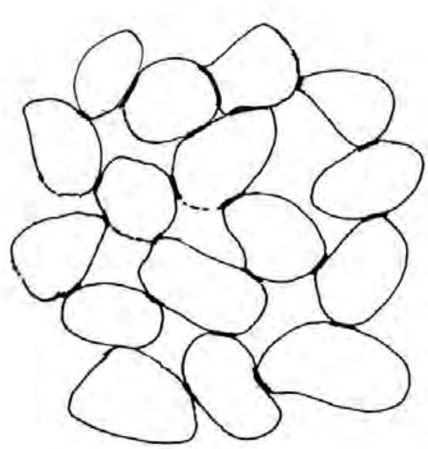


Fig. 2.1 Cementation effect around the contact points of the coarse grains (after Kezdi, 1979).

Thus no continuous matrix is formed and the fracture type depends on whether the interparticle bond or the natural strength of the particles themselves is stronger. The better graded the grain distribution of a soil, the smaller the voids and the greater the number and the larger the interparticle contact surfaces, the stronger the effect of cementation (Kezdi, 1979).

In fine grained silts and clays, the cement stabilization creates rather strong bonds between the various mineral substances and form a matrix which efficiently encloses the non-bonded soil particles. This matrix develops a cellular structure on whose strength that of the entire construction depends. This happens due to the fact that the strength of the clay particles within the matrix is rather low. Since this matrix pins the particles, the cement reduces plasticity and increases shear strength. The chemical surface effect of the cement reduces the water affinity of the clay and in turn, the water retention capacity of the clay. Together with a strength increase, this results in the enclosure of the larger unstabilized grain aggregates which, therefore, cannot expand and will have improved durability. The cement clay interaction is significantly affected by the interaction of lime, produced during hydration of Portland Cement and the clay minerals.

2.3 Soil-Lime Stabilization

Addition of lime of soil changes the plasticity properties of soil. Calcium ions reduce plasticity of cohesive soils so that they become more friable and more easily worked. Changes in grain size distribution are observed almost immediately following the addition of lime to a clayey soil. The major change occurs within the first hour. The new grains produced as a result of lime treatment are mostly silt or sand sized though these are relatively weakly bonded. Aggregation is caused by addition of 1 or 2 percent lime. This reaction is due to alteration of the water film surrounding the clay minerals. The strength of the linkage between two clay minerals is dependent on the charge, size, and hydration of attracted ions. The lime is divalent and serves to bind the soil particles close together. This in turn decreases plasticity and results in a more open and granular structure (Sharma, 1985).

Lime results in decreased soil density. The moisture content needed to achieve maximum density for a given compactive effort usually increases sometime rather significantly. Lime in excess of about 5 percent by weight of soil, generally produces little additional increase in optimum moisture content.

Lime increases soil strength due to reaction of lime with soil components to form new chemicals. The two principal components of soil which react with lime are alumina and silica. This reaction known as pozzolanic action is a long term one and results in a slower long-term cementation of compacted soil lime mixtures. This reaction is somewhat similar to hydration of cement. Soil lime mixes show increased strength over relative long periods of time. This slow setting provides more flexibility in road construction (Sharma, 1985).

Lime mixed with fine grained cohesive soils causes cation exchange, flocculation, agglomeration of the soil or all of these, and thus a lime modified soil layer is created. These immediate reactions produce an improvement in workability and an increase in stability (Thomson, 1970).

2.4 Characteristics and Composition of Admixtures.

In this research, Portland Cement Type - I and quick lime were used as admixtures.

2.4.1 Portland Cement Type - I

Type - I is designated as Ordinary Portland Cement for use in general construction.

Composition of ordinary portland cement according to Mindness and Young (1981) is shown in Table 2.1

Table 2.1 Composition of Portland Cement

Chemical name	Chemical Formula	Weight percent
Tricalcium Silicate	3 CaO, SiO_2	50
Dicalcium Silicate	2 CaO, SiO_2	25
Tricalcium Aluminate	3 CaO, Al_2O_3	12
Tetracalcium aluminoferrite	4 CaO, H_2O_3, Fe_2O_3	8
Calcium sulphate dihydrate	$CaSO_4$, $2H_2O$	5

Calcium trisilicate sets fast and is responsible for immediate strength gain. Calcium disilicate is responsible for long term strength due to hydration reaction. Free lime, a product of hydration reaction, brings about base exchange capacity and changes the texture of the soil (Jha, 1977).

On hydration of this two calcium silicate which constitute about 75 percent of the portland cement, new compound lime and tobermorite gel (calcium-silicate hydrate) are formed. Tobermorite gel plays the leading role as regards strength (Kezdi, 1979).

On an average 23 percent of water by weight of cement is required for chemical reaction. This water chemically combines with cement. A certain quantity of water is trapped within the pores of tobermorite gel. It has been estimated that about 15 percent water by weight of cement is required to fill up the gel-pores. Therefore, a total 38 percent of water is required for the complete chemical reactions and for occupying the space within gel pores (Shetty, 1982).

2.4.2 Air Slaked Lime

According to the ASTM-C-51-47, lime is defined as a general term which includes the various chemical and physical forms of quick lime, hydrated lime, and hydraulic lime. It may be high-calcium, magnesium or dolomitic. The quick lime is a calcined lime stone, the major part of which is calcium oxide or calcium oxide in association with magnesium oxide, capable of slaking with water. Air slaked lime is the product containing various properties of the oxides, hydroxides, and carbonates of calcium and magnesium which results from the exposure of quick lime to the air in sufficient quantity to show physical signs of hydration (difficult to determine visually in pulverized quick lime).

The calcined lime-stones contain the oxides of calcium and magnesium in varying proportions. These oxides show a great affinity for water. The chemical reaction is

$$CaO + H_2O = Ca(OH)_2$$

Kulkarni (1977) showed that for every 56 parts of calcium oxide, 18 prts of water by weight combine to form 74 parts of calcium hydroxide. He also pointed out that for the hydration of lime, 47 percent of water by weight of lime was required.

2.5 Properties of Cement Stabilized Soil Mixtures

The properties of soil-cement mixtures vary with several factors that is soil type, cement content, degree of compaction, degree of pulverisation of soil, mixing methods and environmental condition. Because of the variations in properties due to these factors, it is not possible to list specific values representative of the several properties. However, since moisture content, compaction energy, amount of additives and conditions of curing are closely controlled in accordance with standard methods, it is possible to present laboratory values of the several properties for different soils. Accordingly, the strength characteristics, durability, volume and moisture change characteristics, plasticity, moisture-density relationship of the treated soil will be discussed in a limited range in the following articles.

2.5.1 Compressive Strength

Evaluation of stabilized soil with admixture like cement is widely made with the help of compressive strength of stabilized mix. It serves as an indicator of the degree of reaction of the soil-cement water mixture as well as an indicator of setting time and 'rate of hardening'. For normally reacting granular soils, it serves as a criterion for determining cement requirements for the construction of soil-cement. In Britain, usual practice is to specify the desired stabilities of most soil-cement mix in terms of minimum unconfined compressive strengths (Mustaque, 1986). The most recent specification for soil-cement require a minimum 7 day value of 400 psi for moist-cured cylindrical specimens having a height/diameter ratio of 2:1 and 500 psi for cubical specimens (Ministry of Transport, 1960). Portland Cement Association (1956) established the range of compressive strength of cement treated soils under three broad textural soils groups - sandy and gravelly soils, silty soils and clayey soils as shown in Table 2.2.

Table 2.2 Range of Compressive Strength of Soil Cement (PCA, 1956)

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

Balmer (1958) and Christensen (1969) found that addition of cement increases both the angle of friction and the cohesion. At lower cement contents, the strength increase is mainly due to increase in angle of internal friction whereas the same at higher cement content is due to increase in cohesion. However, the rate of increase of cohesion and internal friction depends on soil type and curing period.

2.5.2 Durability

Durability of soil-cement mixture is its resistance to repeated drying and wetting or freezing and thawing.

In the United States, the desired cement content is normally selected to meet durability. Portland Cement Association (1956) reported maximum soil-cement loss in the wet-dry or freeze-thaw test as shown in Table 2.3.

Table 2.3 Soil Cement Loss Criteria (After PCA, 1956)

AASHO Soil Group	Freeze-thaw and wet-dry losses (%)
A-1-a, A-1-b, A-3,A-2-4 and A-2-5	14
A-2-6, A-2-7, A-4 and A-5	10
A-6, A-7-5 and A-7-6	7

PCA (1959) requirs that the stabilized material should be evaluated using the compressive strength given in Figs. 2.2 and 2.3.

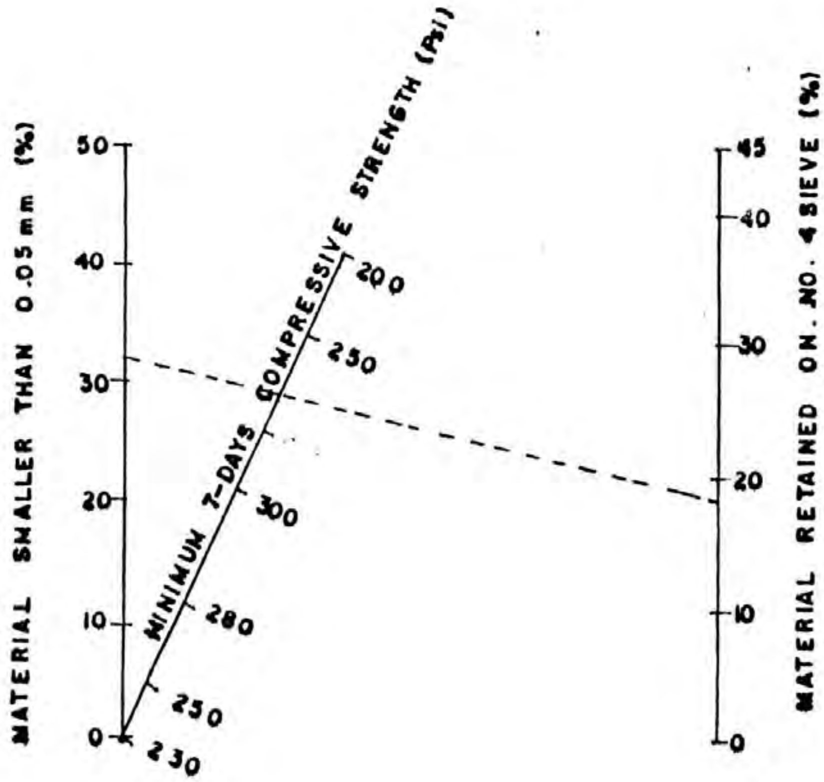


Fig. 2.2 MININUM 7 — DAY COMPRESSIVE STRENGTHS REQUIRED FOR SOIL — CEMENT MIXTURES CONTAINING MATERIAL RETAINED ON THE NO. 4 SIEVE (AFTER PCA , 1959).

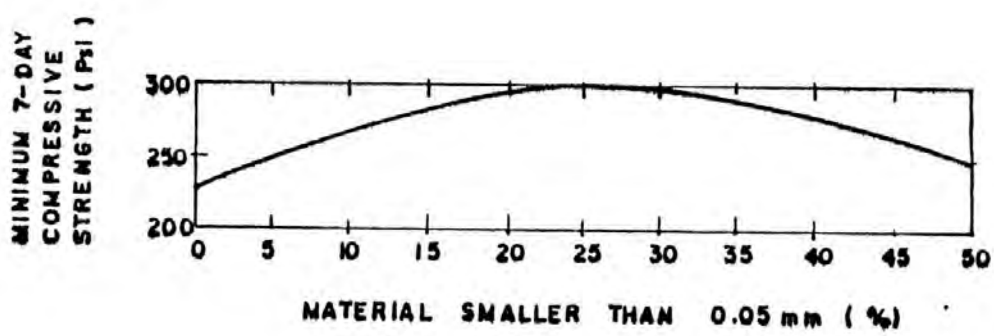


Fig. 2.3 MINIMUM 7-DAY COMPRESSIVE STRENGTH REQUIRED FOR SOIL-CEMENT MIXTURES NOT CONTAINING MATERIAL ON THE NO. 4 SIEVE (AFTER PCA , 1959) ,

Kemahliglu et al (1967) concluded that a minimum compressive strength requirement would not necessarily result in the most economical cement requirement due to the fact that different soil-cement mixtures exhibit different strengths at similar degree of durability. Interesting conclusion by them was the minimum compressive strength required for various AASHTO soil groups to meet PCA criteria when applied through wet-dry test (i.e. satisfying maximum soil-cement loss criteria of PCA) is not a constant but probably varies as a function of other parameters (physical and chemical properties).

Mustaque (1986) showed that the local silty soils satisfied the durability criteria recommended by the Portland Cement Association (PCA) at about 8 percent cement contents.

2.5.3 Volume and Moisture Change

The volume and moisture change of soil-cement mixtures are of particular importance with respect to pavement cracking. Cracking formation is a natural characteristics of soil-cement mixes whose tendency to crack is related to strength, although this relation is not yet fully understood.

Apart from fractures due to loading, cracks are caused by volume changes which may be due to three effects: water content, temperature changes and freezing. If a cohesive soil is treated with cement, then the shrinkage due to water content variation of the soil-cement thus obtained will certainly be less than that of the original soil. Shrinkage decreases with increased cement content, owing to the development of a soil-cement matrix (Willis, 1947 and Jones, 1958). With the increase in cement content, the soil-cement matrix assumes more stable configureation resulting in decreased shrinkage. If on the otherhand, cement is added to a soil which is not liable to volume change by itself, the volume change of the product will be greater. This happens because of the shrinkage during the cement hydration (Kezdi, 1979).

The volume change of soil cement is determined by the usual wetting and drying test methods through direct volume measurement or linear measurement of height. Cement addition has been seen to reduce the specific volume variation upto 33 or even 50 percent.

Another reason for the volume change of cement soils is temperature variation. According to measurements performed in India, the thermal expansion co-efficient depends on the cement content and density (Kezdi, 1979).

2.5.4 Plasticity

If a plastic soil is treated with cement, its plasticity index decreases. This effect is reflected by the different types of failure encountered in such cases. Felt (1953) showed that plasticity index of the granular soil decreases when treated with cement.

For cement stablized soil, it is seen that the plastic limit and liquid limit increases with icnreasing cement content. But increase in plastic limit is appreciable resulting in decrease in plasticity index at higher cement content (Mustaque, 1986).

Redus (1958) found that with increase in cement content and for longer curing period, plasticity index reduces. Ahmed (1984) also showed that for sandy soil and silty clay plastic limit increases on addition of cement.

2.5.5 Moisture - Density Relation

The optimum moisture content and maximum dry density influence the compaction characteristics of cement treated soils. Generally, for cement treated soils, these two data can be said to vary slightly from those obtained from untreated soils. However, there is exception of this behaviour (Mustaque, 1986).

With the addition of cement, maximum dry density of sand increases. Little or no change is observed for light to medium clays, but decrease in density may occur in silts. Decrease in optimum moisture content occur for clays and it increases for the silts but little or no change takes place for sands and sandy soils. Mustaque (1986) showed that for silty soil density decreased with increasing cement content. He also found for an A-

4 silty soil that density decreased upto 4% cement content and after that, almost no change occured. For sandy silt, there had been decreased in density with the increase in cement content from 1/2 percent upto 10 percent by Ahmed (1984).

2.5.6 Strength in Terms of CBR Value

The California Bearing Ratio abbreviated as CBR is the most widely used method of evaluation of subgrade. The method was first developed by California Division of Highways and then adopted and modified by U.S. Corps of Engineers, in 1961. The American Association of State Highway and Transprotation Officials, AASHTO accepted this test in 1963 with designation T 193 - 63 for determining the bearing values of subgrade soils and some sub-base and base course materials containing only a small amount of material retained on the 3/4 inch sieve.

The CBR value is an important parrameter for evaluating the subgrade and bases. It is used to rate the performance of soils primarily for use as bases and subgrades beneath pavements of roads and airfields. The Table 2.4 gives typical ratings (AASHTO, 1966).

Table 2.4 Ratings of Performance of Soil

CBR value	General rating	Uses	AASHTO classification
0-3	Very poor	Subgrade	A5,A6,A7,
3-7	Poor to fair	Subgrade	A4,A5,A6,
7-20	Fair	Subase	A2,A4,A6,A7,
20-50	Good	Base,Subbase	A1b,A2-5,A3,A2-6
50	Excellent	Base	A1a, A2-4,A3

Very few published information is available about stabilisation of subgrade soil of Bangladesh. Local government Engineering Bureau (1985) had taken a project as trial basis in Rajbari district. They constructed a trial roads using 4% cement stabilized base course and found its field CBR as 95% and 103% for optimum Moisture contents 15% and 16% respectively. Using different percent of cement they showed that CBR value increased with the increment of cement (BCL, 1986).

2.6 Properties of Lime-treated Soil Mixture

2.6.1 Compressive Strength

Compressive strength is one of the methods for evaluating soil-lime mixture. The percentage of lime for a given project generally are determined by testing lime soil mixtures using the unconfined compression test. AASHTO T220 recommends that generally, an unconfined compressive strength of 7 kg/cm^2 is satisfactory for final course of base construction. It further recommends that various soil materials may be treated for subbase and the minimum suggested unconfined strength is 3.5 kg/cm^2 (Sharma, 1985).

It is generally found that beyond a certain percentage of lime, the increase in strength ceases and in fact a lowering of the strength may result due to the presence of unreacted free lime. The similar findings were drawn by Thomson and Neubour (1968). They tabulated the value of compressive strength for 2,3 and 4 percent lime are 312 psi, 507 psi and 497 psi respectively. They uncluded that as the lime contents increased above an optimum amount, the reduction in strengths was occured.

For clayey soil with the addition of lime, Croft (1964) suggested that increased optimum moisture content can possibly be attributed to the increase in hydroxyl ion concentration which modifies the surface of clay particles and increases the strength.

Ahmed (1984) found that a silty clay soil (contain 10 percent of clay) provides an increase in strength when stabilized with lime admixtures. He also showed that for silty sand there was very small change in strength due to the increase in lime content and for sandy silt soil there is a reduction in strength due to the addition of lime and curing period. At the time of curing the specimen drew water by capillary suction and the sample became soft. This probably reduced the compressive strength of the soil.

2.6.2 Durability

A little published information is available regarding durability characteristics of lime stabilised soil. But it is the major requirement in freezing climates. One method for measuring durability is to measure the decrease in unconfined compressive strength after cycles of freez-thaw. A durability ratio, defined by British Road Research Laboratory, is the strength after weathering divided by the strength obtained by curing for the same length of time. A ratio of 80 percent after 74 cycles freezing and thawing is regarded satisfactory (Sharma, 1985).

2.6.3 Plasticity

Significant changes take place in the plasticity properties with the addition of lime. The liquid limit is generally seen to decrease with increasing quantity of lime as observed by Uppal and Bhatia (1958) and Jan and Walker (1963). This observation is particularly true for clayey soils. Clare and Cruchly (1957) reported on the basis of work done in the U.K. that immediate effect of small addition of lime such as 1 percent is to raise the liquid limit of the soil (P.I. 50) considerably, and further addition of lime upto 10 percent steadily reduces this value. It is concluded by Harrin and Metchell (1961) that the liquid limit decreases in the more plastic soil and increases in the less plastic soil.

Irrespective of the decrease or increase in the liquid limit of the mixture, there is a general unanimity of view that the plastic limit increases with the addition of greater percentage of lime. Hilt and Davidson (1960) experimentally found that the plastic limit increases with the addition of lime upto some limiting lime content and any increase thereafter causes insignificant or no change.

As a result of the general decrease in liquid limit and a good rise in the plastic limit, the plasticity index drop down very considerably. Johnson (1948) showed that the plasticity index of highly plastic clays is reduced considerably with only a small amount of lime, where as the less plastic soils are slightly reduced by the addition of even large amount of lime.

2.6.4 Moisture-Density Relation

It has been generally found that lime-soil mixture has a lower standard miximum dry density than the raw soil without lime. Ladd and et al (1960) showed that as the lime content increases, the density tends to fall. The reduction in density is probably due to the flocculated particles present in the lime soil mixture. Andrews and O'Flaherty (1968) showed that with a sandy type of soil, if a semi-hydrated lime is added, the maximum dry weight can actually increase as a result of the additive. Another trend that can be found is the increase in the optimum moisture content of the soil with addition of lime was shown by Croft (1964). The increased optimum content can possibly be attributed to the increase in CH-ion concentration which modifies the surface of clay particles and increases the water associated by them.

2.6.5 Bearing Strengths by CBR Value

Lime treatment of a wide range of typical fine grained Illinois soils compacted at their optimum water contents showed that CBR value was increased due to increment of lime (Thomson and Beubaur, 1968). Arman and Munfakh (1968) found that the CBR value for inorganic soils increased due to the addition of lime upto lime fixation point and then decreased.

No published information is available regarding this test result of lime stabilized soil of this country.

2.7 Dynamic Cone Penetrometer Value and its Relationship with CBR

The dynamic Cone penetrometer abbreviated as DCP is a simple equipment which can be used for evaluating the strength of pavement structure and for controlling the quality of work. It can also be used to design the new road pavements.

A typical diamgram of the dynamic cone penetrometer is shown in Fig. 2.4. In order to obtain the pavement thickness it is necessary to find the strength of the subgrade. The DCP of the subgrade should be found in the worst condition, that is soaked condition, during the wet season. If it is necessary to determine the DCP in the dry season then an area of subgrade should be soaked by ponding for at least a week before the DCP is taken (BRRL, 1985).

A corelation can be establized between the laboratory DCP and CBR value. During the construction period, the field DCP can be measured sand its corresponding CBR value is evaluated from the correlation. A relationship between DCP and CBR of subgrade soil was developed by Bangladesh Road Research Laboratory (BRRL, 1985) which is shown in Fig. 2.5.

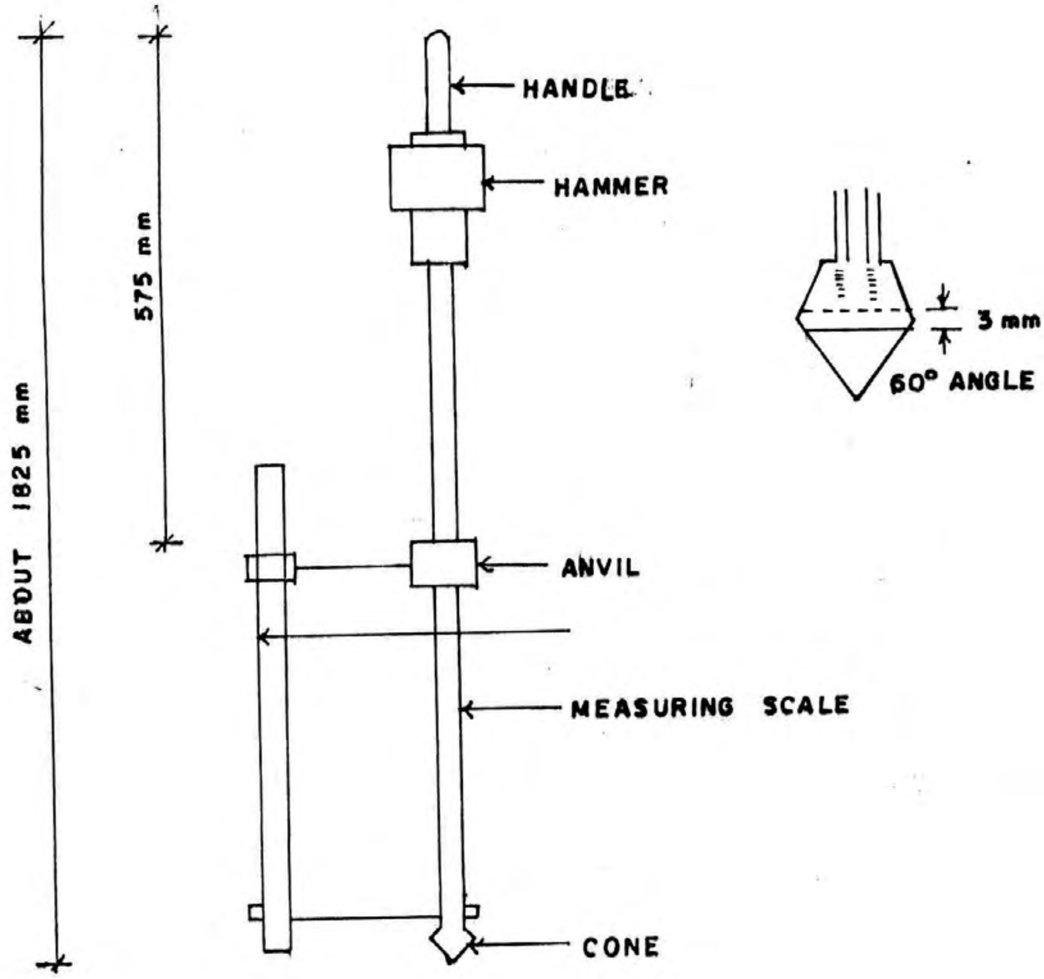


Fig. 2.4 DYNAMIC CONE PENETROMETER

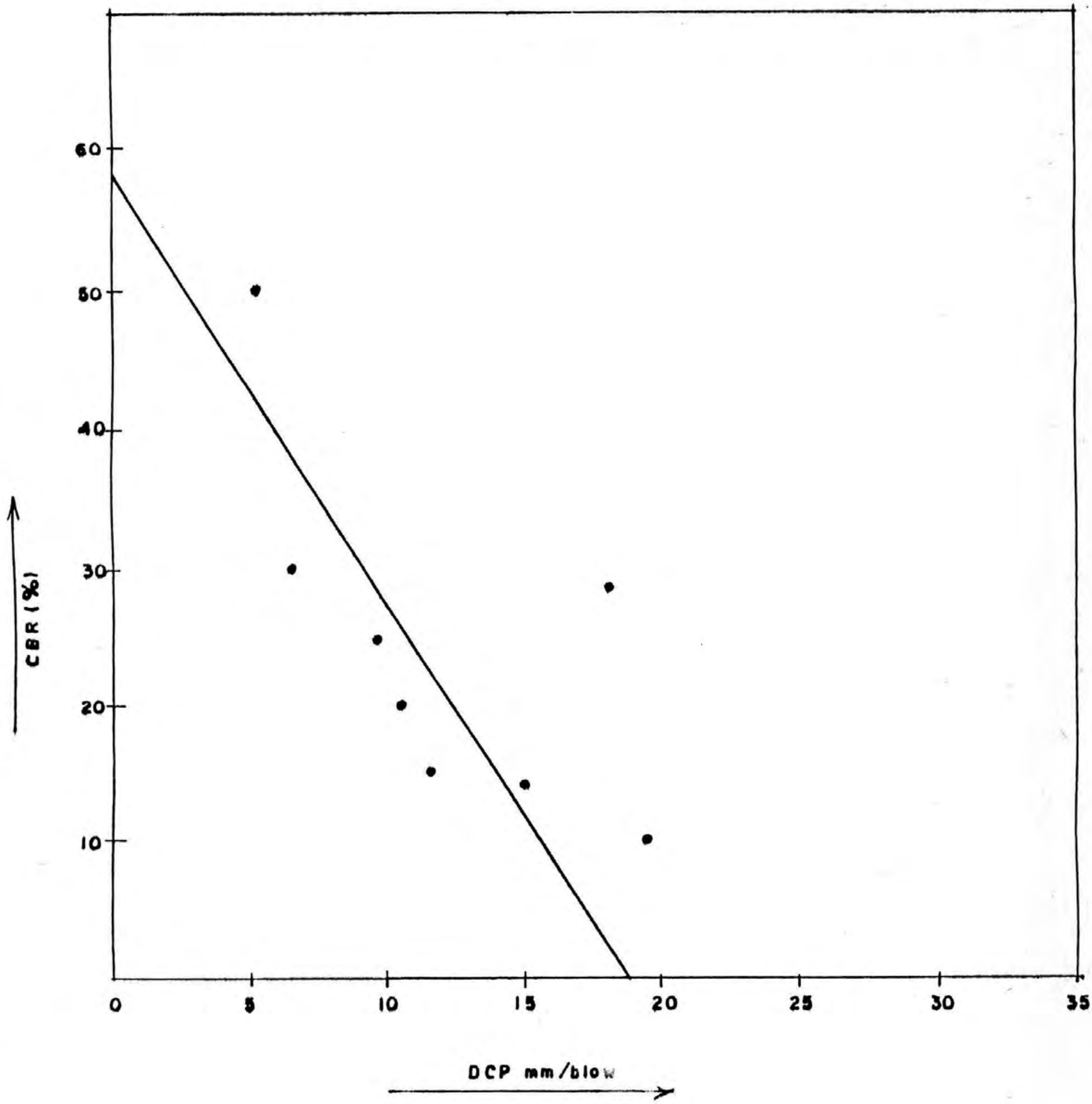


Fig. 2.5 CBR – DCP RELATIONSHIP.

2.8 Summary of the Literature Review

From above literature review the important points may be summarized below:

i) Cement can be used successfully for stabilizing sands and silty soils whereas for increasing clay content in the soil excessive cement is warranted.

ii) Silty clay soil provides an increase in strength when stabilized with lime admixture.

iii) Different soil-cement mixtures at the similar degree of durability may exhibit different strengths.

iv) In cement treated soil mixtures, the plasticity index reduces with increase in cement content. Lime treatment is suitable for plastic soil.

v) California Bearing Ratio (CBR) value of soil cement mixture increases as the cement content and compactive effort increases.

vi) For lime treated soils california bearing ratio generally increases due to the addition of lime upto a limit and then decreases.

vii) Dynamic Cone Penetrometer value is important for evaluating the subgrade, subbase and base courses.

CHAPTER 3

THE RESEARCH SCHEME

3.1 Introduction

For efficient and economic application of stabilization technique it is essential to understand the basic mechanism of the process. The broad objective of this research is to experimentally review various aspects of soil-cement and soil-lime stabilization of two selected subgrade soils.

3.2 The Test Programme

The whole research was divided into the following phases:

i) In first phase, index properties of the soil samples were determined in order to classify them.

ii) In the 2nd phase, the moisture density relationship of the soils were established. Then durability and strength of stabilized soil were evaluated by wetting drying test, unconfined compressive strength test and California Bearing Ratio Test.

iii) In the third phase, dynamic cone penetrometer test was conducted on stabilized soil to have a relationship between DCP and CBR.

iv) In the final phase, a cost analysis was developed on treated and untreated subgrade soil.

The experimental program followed is illustrated by a flow chart as shown in Fig. 3.1.

3.3 Materials

3.3.1 Soils

In this research, two soil samples were collected from flood protection embankment of Greater Dhaka city and Dhaka Aricha Highway. Both the samples have low CBR values. The soils were designaated as follows:

Soil A - Collected from Embankment at Katasur, Mohammadpur, Dhaka.

Soil B - Collected from Chandgaon on Dhaka - Aricha Highway 13 miles away from Zero point, Dhaka.

The properties of untreated soils are presented in Table 3.1 and grain size distribution curves are shown in Fig. 3.2.

3.3.2 Cement

For this research, ordinary Portland Cement Type - 1 was used. ASTM Standards 1979 (b) C 187-79, C191-77, and C109-77 were followed for determination of normal consistency, time of setting and compressive strength of the cement respectively.

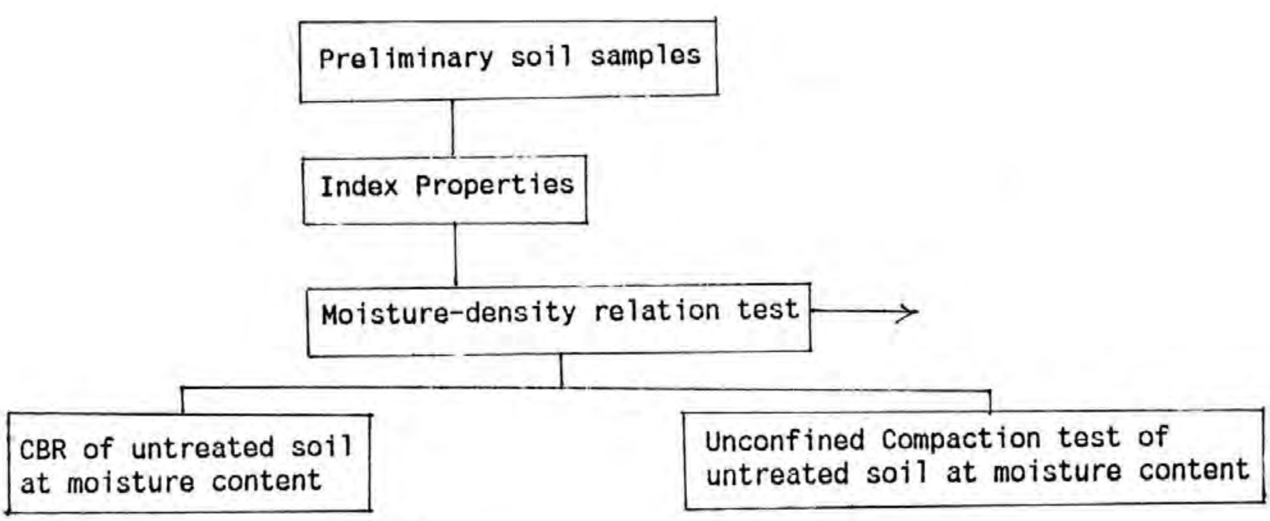
The results are presented below:

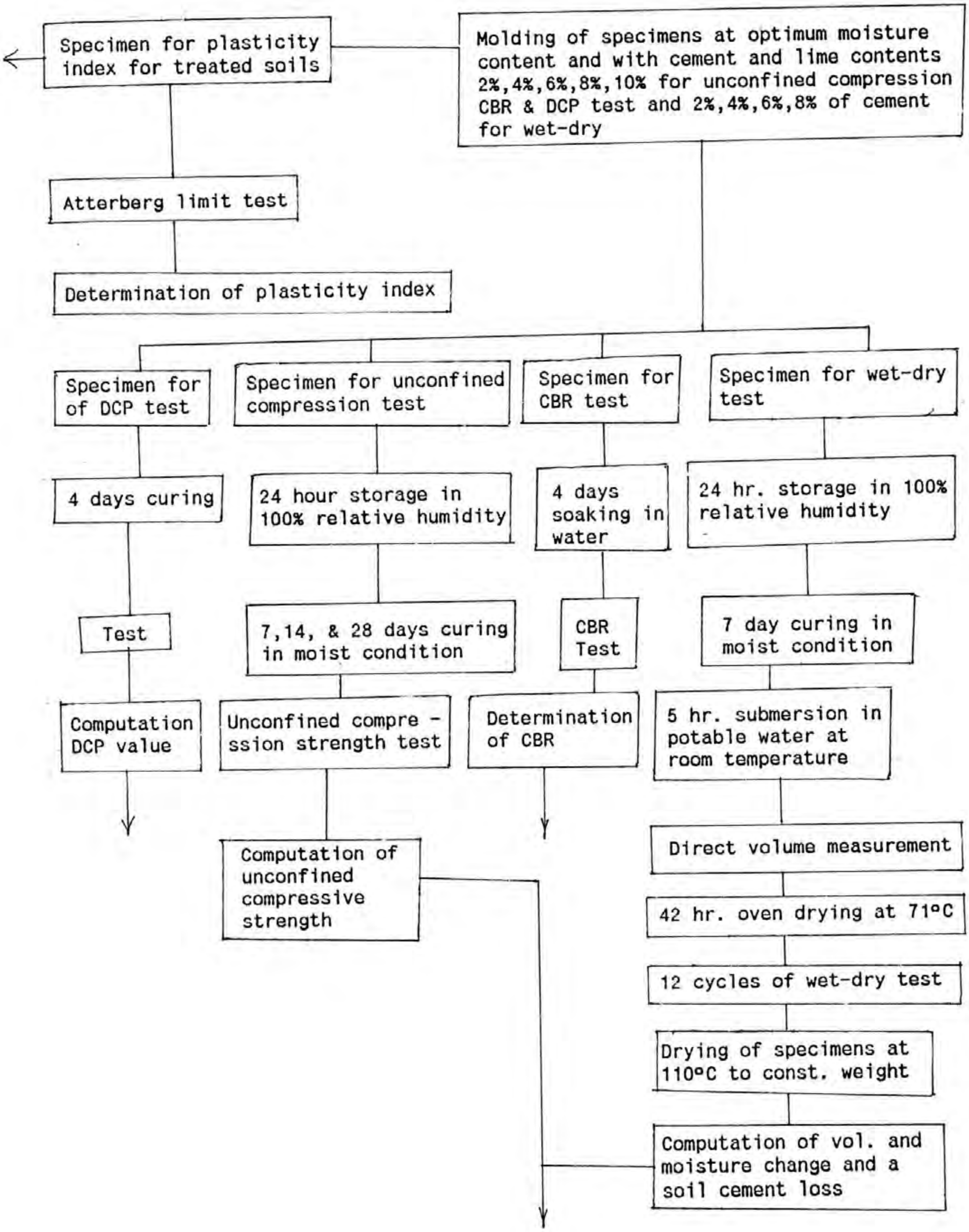
i) Normal consistency is 25 percent

ii) Initial setting time is 2 hours 10 minutes

iii) Final setting time is 3 hours 10 minutes

iv) Compressive strength of 2 inch standard cube specimens are 2,580, 4,535 and 5,195 psi for 7, 14 and 28 days respectively.




Specimen for plasticity index for treated soils
Molding of specimens at optimum moisture content and with cement and lime contents 2%,4%,6%,8%,10% for unconfined compression CBR & DCP test and 2%,4%,6%,8% of cement for wet-dry
Atterberg limit test
Determination of plasticity index
Specimen for of DCP test
Specimen for unconfined compression test
Specimen for CBR test
Specimen for wet-dry test
4 days curing
24 hour storage in 100% relative humidity
4 days soaking in water
24 hr. storage in 100% relative humidity
Test
7,14, & 28 days curing in moist condition
CBR Test
7 day curing in moist condition
Computation DCP value
Unconfined compre - ssion strength test
Determination of CBR
5 hr. submersion in potable water at room temperature
Computation of unconfined compressive strength
Direct volume measurement
42 hr. oven drying at 71°C
12 cycles of wet-dry test
Drying of specimens at 110°C to const. weight
Computation of vol. and moisture change and a soil cement loss

Develop a CBR-DCP relationship

Conclusion about volume and moisture change characteristics, strength and durability to stabilized soil

Fig. 3.1 Flow Chart of Test Programme.

Table 3.1 Properties of Untreated Soils

Soil Property	Soil-A	Soil-B
Textural composition (MIT classification)		
Sand, % (2 mm - 0.06 mm)	8	13
Silt, % (0.06 mm - 0.002 mm)	69.23	67
Clay, % (<.002 mm)	22.77	20
Percent passing # 200 sieve	95.4	89.0
Atterberg limits & indices:		
i) Liquid limit, %	25	42
ii) Plastic limit, %	13	22
iii) Plasticity index, %	12	20
iv) Specific gravity	2.67	2.7
Engineering Properties:		
Optimum moisture content, % (Standard proctor)	14	20
Maximum dry density, pcf	101.5	95
Unconfined compressive strength, psi	15.73	11.91
California Bearing ratio, percent	3.0	2.0
Classification:		
AASHTO/AASHO	A - 6(8)	A-7-6(8)
General rating as subgrade:		
AASHTO/AASHO	Very poor	Very poor

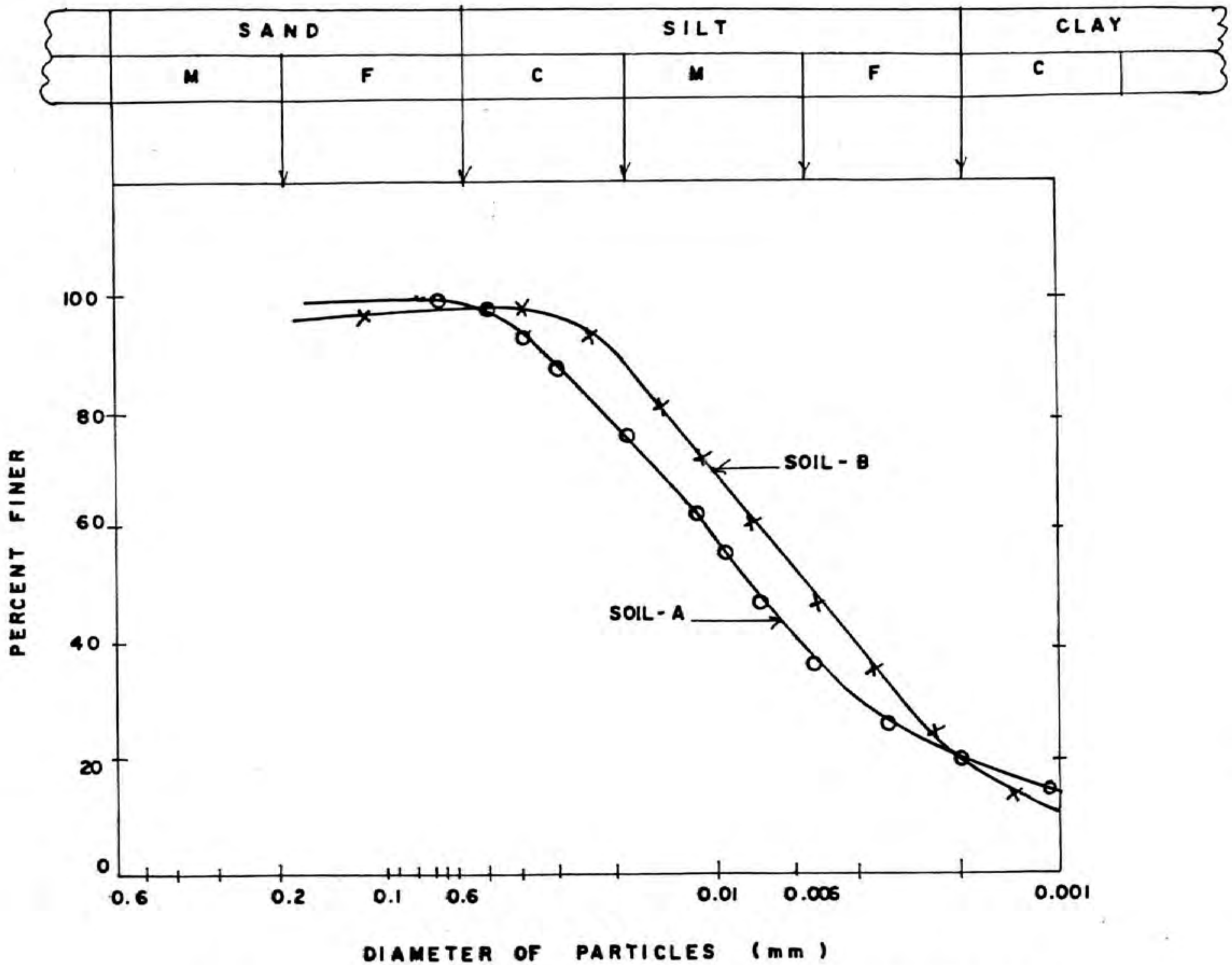


Fig. 3.2 GRAIN SIZE DISTRIBUTION OF SOILS.

CHAPTER 4

LABORATORY INVESTIGATION

4.1 Introduction

The investigation in the laboratory were conducted in accordance with the programme outlined in Art. 3.2. The details of the experimental procedure are discussed in this chapter.

4.2 Test for Index Properties

Test for index properties of the soils were determined according to procedures specified by the American Association of State Highway and Transportation Officials (AASHTO) and the American Society for Testing Material (ASTM). Table 4.1 shows the standard methods followed:

Table 4.1 Standards Method followed for Testing to Classify Soil

Property of Soil	AASHTO standard followed
Liquid limit	T89
Plastic limit and plasticity index	T90
Grain size distribution	T88
Amount of materials finer than no.200 sieve	T11

In addition AASHTO T100 was followed for determination of specific gravities of the soils.

The soils were then classified according to AASHTO M145 - 49 standard. The test results along with their classification and grain size distribution are presented in Table 3.1 and Fig. 3.2.

4.3 Moisture - Density Relation

Moisture - density relationships for the soils were determined according to AASHTO method T99. For compaction of the soils, cylindrical mold of 4 inch diameter and 4.6 inch height was used.

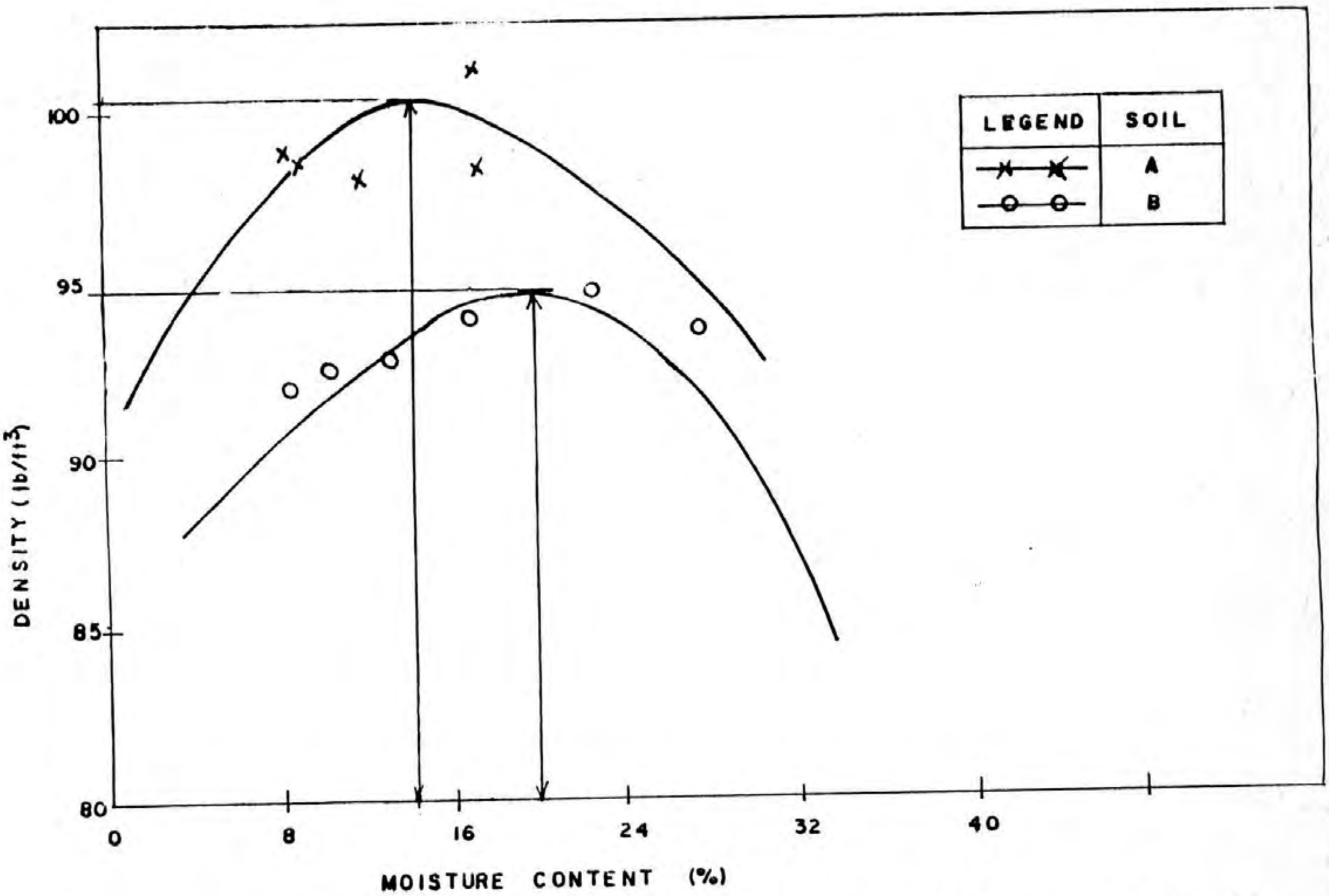


Fig.4.1 MOISTURE DENSITY RELATION OF UNTREATED SOILS.

The weight of the hammer was 5.5 lbs and the height of the drop was 12 inches. The mold was then filled with soil in three approximately equal layers. Each layer was compacted by 25 blows of the hammer. Air-dried samples passing through No. 4 sieve were used for compaction.

The test results are shown in Fig. 4.1. From the moisture density curves of Fig. 4.1, optimum moisture contents and corresponding maximum dry densities for the soils were determined.

4.4 Unconfined Compressive Strength Test

This test was done to determine the unconfined compressive strength of the soils.

The soils are air dried first and then broken down to pass No. 4 sieve. Air dry moisture content was calculated. For cement stabilized soil. cement contents used were 2%, 4%, 6%, 8% and 10% by weight of air dried soil. For lime stabilization, cement was taken as 2%,4%,6% and 8% by weight of air dried soil.

The molding moisture content for treated soil was calculated summing the water required in addition to air dried state and that required for hydration of cement and lime. For hydration of cement, water required is 38 percent by weight of cement used (Shetty, 1982). For hydration of lime additional water required is 47%. For each batch, 8 lbs of soil samples were taken and mixed manually with the required amount of moisture and admixture. Immediately after mixing, the mixture was compacted following AASHTO standard T99. After compaction, density was determined by weighing the mold with the compacted soil. This is the molding density. For molding moisture content determination, around 50 gms of sample from the mixture was taken. The compacted sample was then extracted from the mold by a jack. For each compacted sample. 3 cylindrical samples of 1.4 inch diameter and 2.8 inch height were trimmed off by a piano wire.

These samples were then transferred to a dessicator to store in moist environment for 24 hours and then cured for 7,14 and 28 days. Curing was done by placing the samples on a filter paper placed on the porous plate in the dessicator. Water was added so that the filter paper became saturated and water level was always maintained just in touch with the filter. It was expected that the samples would draw water from the dessicator by capillary rise and got cured.

The unconfined compressive strengths for 7,14 and 28 days were then determined following ASTM standard D2166 - 66 (1972).

Moisture contents at failure also determined. The results are shown and discussed in the 5th chapter.

4.5 Durability Test (Wetting and Drying Test)

This test is aimed at testing the reaction of the stabilized soil to the effect of repeated drying and wetting.

The samples were prepared by compaction following AASHO Method T99. Dimensions of the samples tested were identical to those of the standard proctor molds. The air dried soils were passed through No. 4 sieve. Air dry moisture contents was calcualted. For cement stabilized samples, cement contents of percentages of 2,4,6 and 8 by weight of air dried soil were used.

In order to attain the required moisture content, the water required in addition to air dried state and for hydration was calculated. 8 lbs of soil sample were taken and the required amount of water and admixture were added. The mixture was compacted according to AASHO standard T99 except that the surface of each compacted layer was roughened prior to the application of the next by scratching a square grid lines 8 inch wide and 1/8 inch deep having approximately 1/4 inch spacing. During compaction the water content of a representative sample was determined. After compaction, the mold was weighed for determination of density. The compacted sample was then extracted from the mold by an extruder.

Each test required two samples : One for testing the volume and moisture change. While the second was used for soil-cement loss determination. The ready made samples were weighed and stored for 24 hours in humid surrounding. Then the samples were cured for 7 days in dessicator, keeping the samples over a filter paper just touching the water below. Weight and dimensions are checked in curing period. Following 7 day treatment, the samples were submerged in tap water for 5 hours at room temperature, leaving a water layer of 1 inch above them. After removal the weight and dimensions of specimen no. 1 were checked, then both samples were placed into an oven at 71°C for 42 hours. This was followed by another weight check, then specimen no. 2 was brushed by standard ASTM brush by eighteen to twenty strokes on sides and four on each end. Finally a third weighing was performed to determine the weight loss. The operations enumerated represent a single durability or wetting - drying test cycle, 12 cycles for each sample performed.

Therefore, volume and moisture change were calculated as a percentage of original volume and moisture content. The soil-cement loss was expressed as a percentage of the original oven dry weight.

The results of the wet - dry test for soil-cement have been shown and discussed in the 5th Chapter.

4.6 California Bearing Ratio Test

CBR test is a penetration test wherein, a standardized piston, having an end area of 3 sq. inch is caused to penetrate the sample at a standard rate of 0.05 inch per minute. The unit load required to penetrate the sample at 0.1 inch and 0.2 inch penetration is then compared with a value of 1000 lb per sq. inch and 1500 lb per sq. inch respectively required to effect the same penetration in standard crushed rock. For design purpose the CBR value of the subgrade, base or subbase course at worst condition is required which can be obtained by testing the sample, after being saturated.

California Bearing Ratio (CBR) test was performed according to AASHO T193 and ASTM D1883 - 73. With the variation of cement and lime from 2 percent to 10 percent, samples were prepared at their optimum moisture contents. Molds were compacted by 5.5 lbs hammer applying 65, 35 and 10 blows per layer. For each type of compactive effort three samples were taken for testing. The samples were cured in water for 4 days keeping a surcharge weight of 10 lbs on the top of the mold. During soaking, the water level in the mold and the soaking tank was maintained approximately 1.0 inch above the top of the specimen. After being saturated, the penetration test was done. Due to swelling of the specimen, the top surface may be loose to some extent. Therefore the strees-strain curve obtained from the penetration test some times become upward which required correction by moving it to the right. By CBR value it means corrected value when this correction has been applied to the curve.

The test results are shown in Chapter 5.

4.7 Dynamic Cone Penetrometer Test

The dynamic cone penetrometer used for this investigation was described in literature review having the vertical steel rod 16 mm diameter instead of 10 mm diameter as specified by BRRL (1985). In use the cone at the bottom of the rod is placed on the bed under test and the hammer is lifted to the stop at the top of the handle and allowed to fall freely a fixed distance onto the anvil in the middle of the rod. This drives the cone into the sample and its depth of penetration is read on the measuring scale. This process is repeated until the cone has penetrated the full depth of the sample. The rate of penetration in millimetres per below is termed as the DCP value.

The samples prepared for this test were identical to those of CBR test. The DCP apparatus was placed on the CBR mould and initial reading from the scale was recorded. Then the cone was driven into the mould by allowing the hammer to fall freely. During the process the depth of penetration was recorded for each below. The penetration in mm per below was considered as the DCP value.

Three operators were required to work the dynamic cone penetrometer, one to hold the instrument vertically, a second to lift the hammer and a third to control the procedure and record the penetration.

The test result are shown in Chapter 5.

CHAPTER 5

RESULTS AND DISCUSSIONS

5.1 Introduction

In this chapter, test results are presented and discussed in details. These results would demonstrate the effect of admixture on the plasticity, durability, strength, volume and moisture change characteristics of the stabilized soils. The quick evaluation technique of compacted subgrade is also discussed. A comprative cost analysis of pavement using untreated and treated subgrade soil is also presented in this chapter.

5.2 Wetting and Drying Test

The results of the wetting and drying test are presented in the following articles:

5.2.1 Minimum Cement Content

Minimum cement content required for soil-cement mixture was ascertained from the results of the wet-dry test as per PCA recommendation.

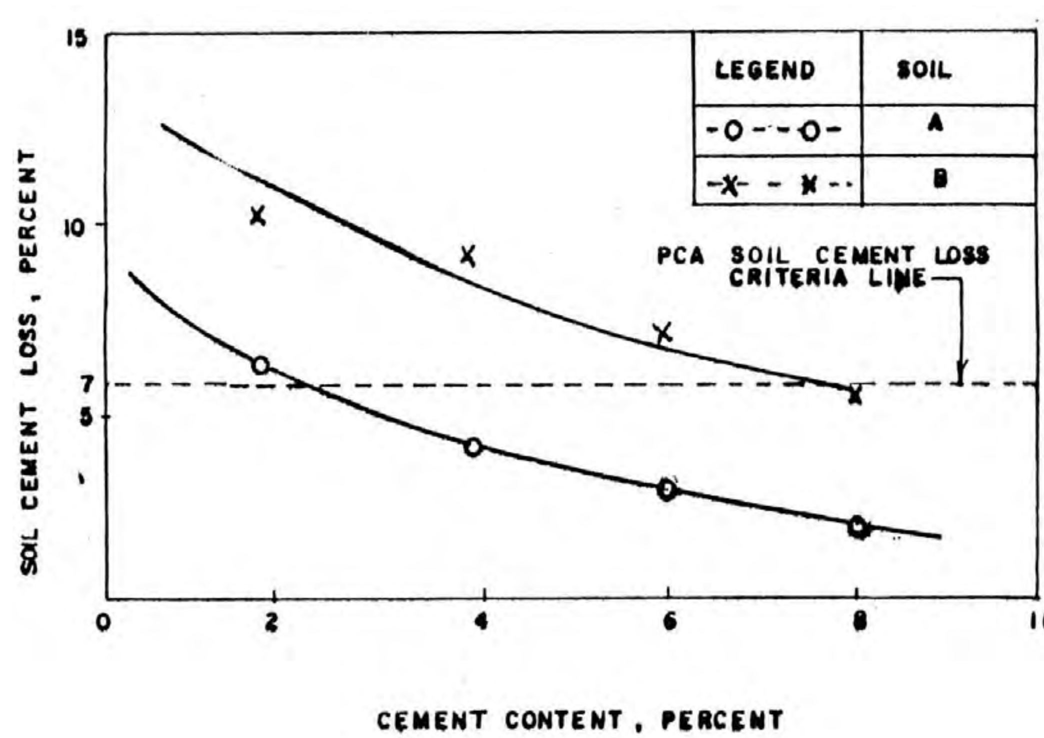


Fig. 5.1 EFFECT OF CEMENT CONTENT ON SOIL-CEMENT LOSS OF THE SOIL IN WET-DRY TEST.

Figure 5.1 shows the relationships between soil-cement loss and cement content for Soil - A and Soil - B. It indicates that higher the cement content, the lower the soil-cement loss in wet-dry test. Similar findings were reported by Mustaque (1986). According to AASHO classification, Soil - A is a soil of group A-6(8). The Portland Cement Association (PCA, 1956) suggested that a maximum of 7 percent loss of soil cement in the wet-dry test is allowable for this type of soil. Fig. 5.1 shows that addition of 2.1 percent cement in this soil would result in a durable soil-cement mixture satisfying PCA criteria.

From Table 3.1, it is seen that Soil - B is a soil of group A-7-6(8). Cement requirement satisfying PCA (1956) soil cement loss criteria in wet-dry test is 6.7 percent which is quite below the maximum limit of cement content that is 16% by weight recommended by Cotton (1940).

It is seen that Soil - A and Soil - B require different amount of cement for stabilization to satisfy durability criteria. This confirms the reporting of Catton (1940) that different AASHO soil groups requrie different amount of cement.

5.2.2 Moisture Change

Maximum moisture content is the highest amount of water heldup in the soil sample during its cycles of wetting in wet-dry test.

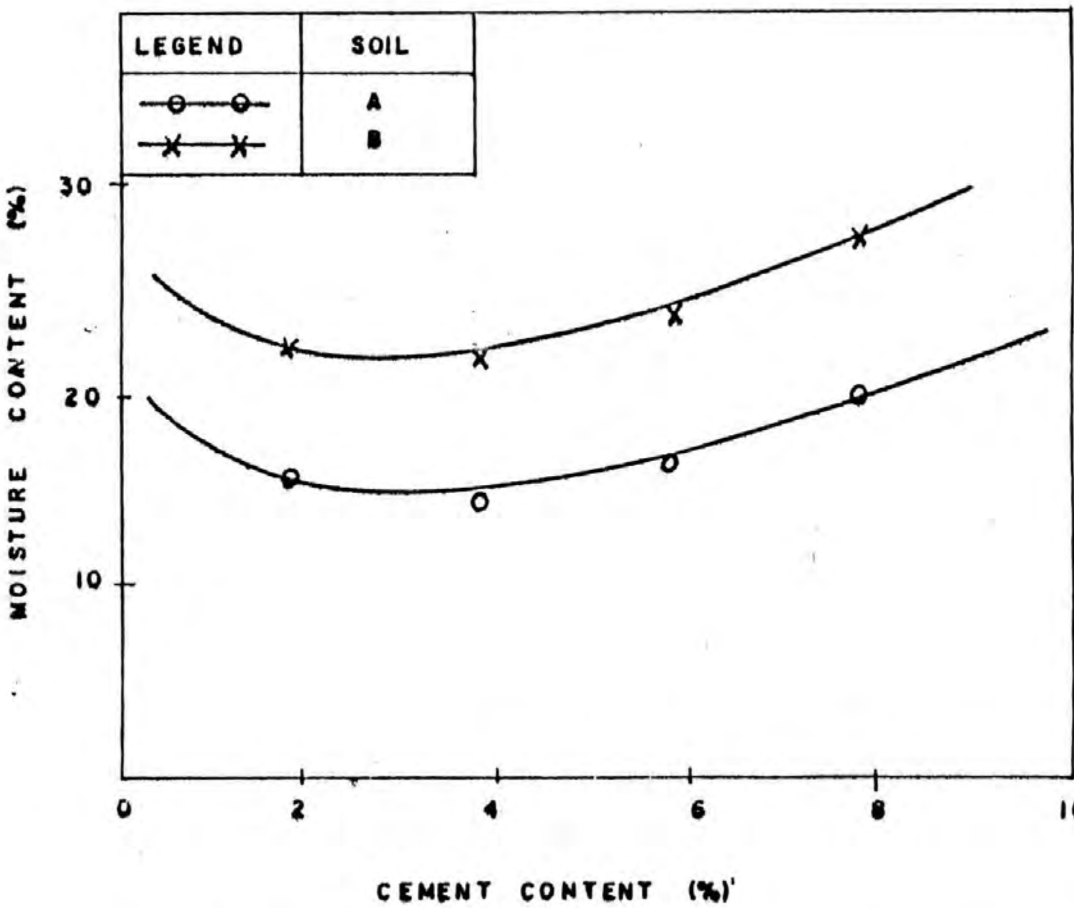


Fig. 5.2 MOISTURE CONTENT IN CEMENT STABILISED SOILS DURING WET-CYCLE OF WET-DRY TEST.

Fig. 5.2 shows the maximum moisture contents for Soil A and B against cement contents. It is seen that moisture content increases for higher cement. Similar results were reported by Mustaque (1986). It is seen that the maximum moisture content in wet dry test occurs for 8 percent cement contents for soil A and B which are 18.57 and 20.72 percent respectively (Appendix A-5). On the otherhand, moisture content at failure of unconfined compressive strength of the above soils are 27.35 and 27.70 percent respectively. Soil confirms the result of Mostaque (1986) where he showed that moisture contents at failure in unconfined compressive strength were well above the maximum moisture content in wet-dry test which also confirmed that the strength test result were representative for a situation when road subgrade or sub-base were completely submerged.

5.2.3 Volume Change

The relationships between volume change and percent of cement content for soils have been shown in Fig. 5.3. It is found that with the increase in cement content shrinkage occurs in both the soil. This occurs due to shrinkage during the cement hydration (Kezdi, 1979). Similar conclusion was drawn by Mustaque (1986). The volume change in both cases is well below 2 percent reported as requirement by Kezdi (1979).

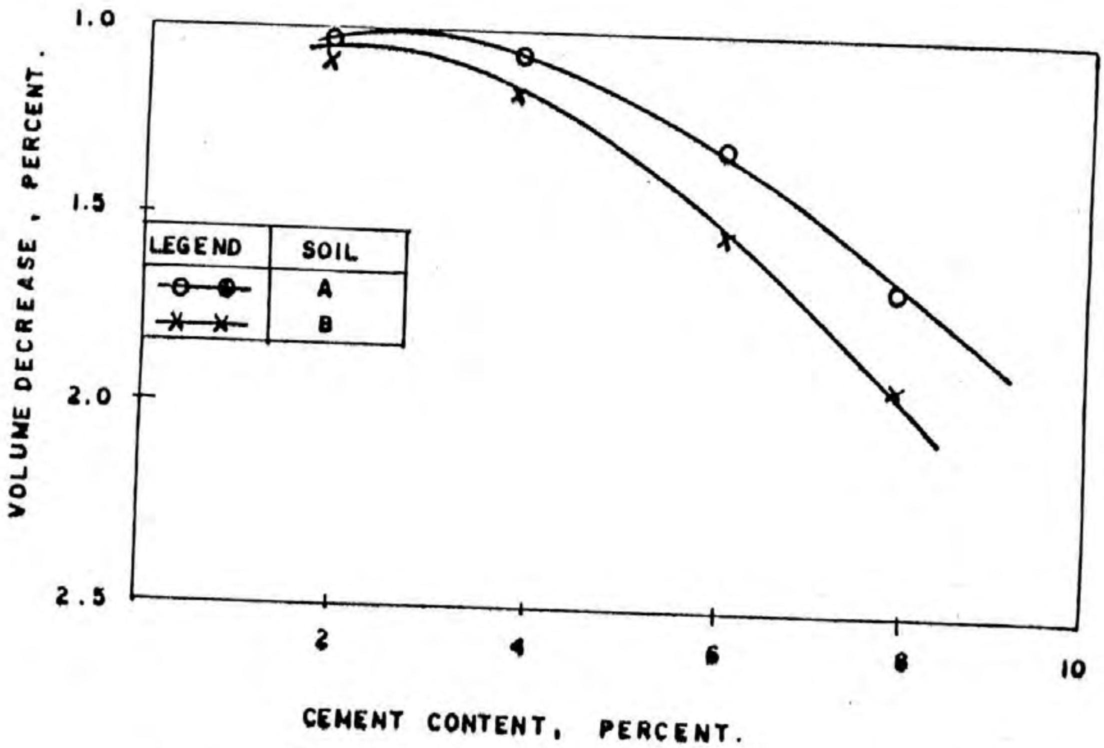


Fig. 5·3 VOLUME CHANGE OF THE CEMENT TREATED SOILS DURING DRY CYCLE OF WET-DRY TEST.

5.3 Unconfined Compressive Strength

5.3.1 The relationship between unconfined compressive strength and cement content cured for 7 days, 14 days and 28 days are presented in Fig. 5.4 and 5.5.

Here it is seen that the increasing of cement content and curing period, the compressive strength increases. The results confirm the experimental findings of Leadbrand (1955) where he showed that the soil-cement continues to increase in strength with age. Ramaswamy et al (1984) showed for silty soils in Singapore that with increasing cement content and curing period, cement stabilized soil continues to increase in strength with age. Ahmed (1984) showed for a silty clay of Bangladesh that addition of cement from 2% to 15% by weight, strength increased appreciably.

Table 5.1 shows the ratio of unconfined compressive strength of cement stabilized (UCCS) Soil-A and Soil-B at 7, 14 and 28 days to that of untreated soils (UC) respectively.

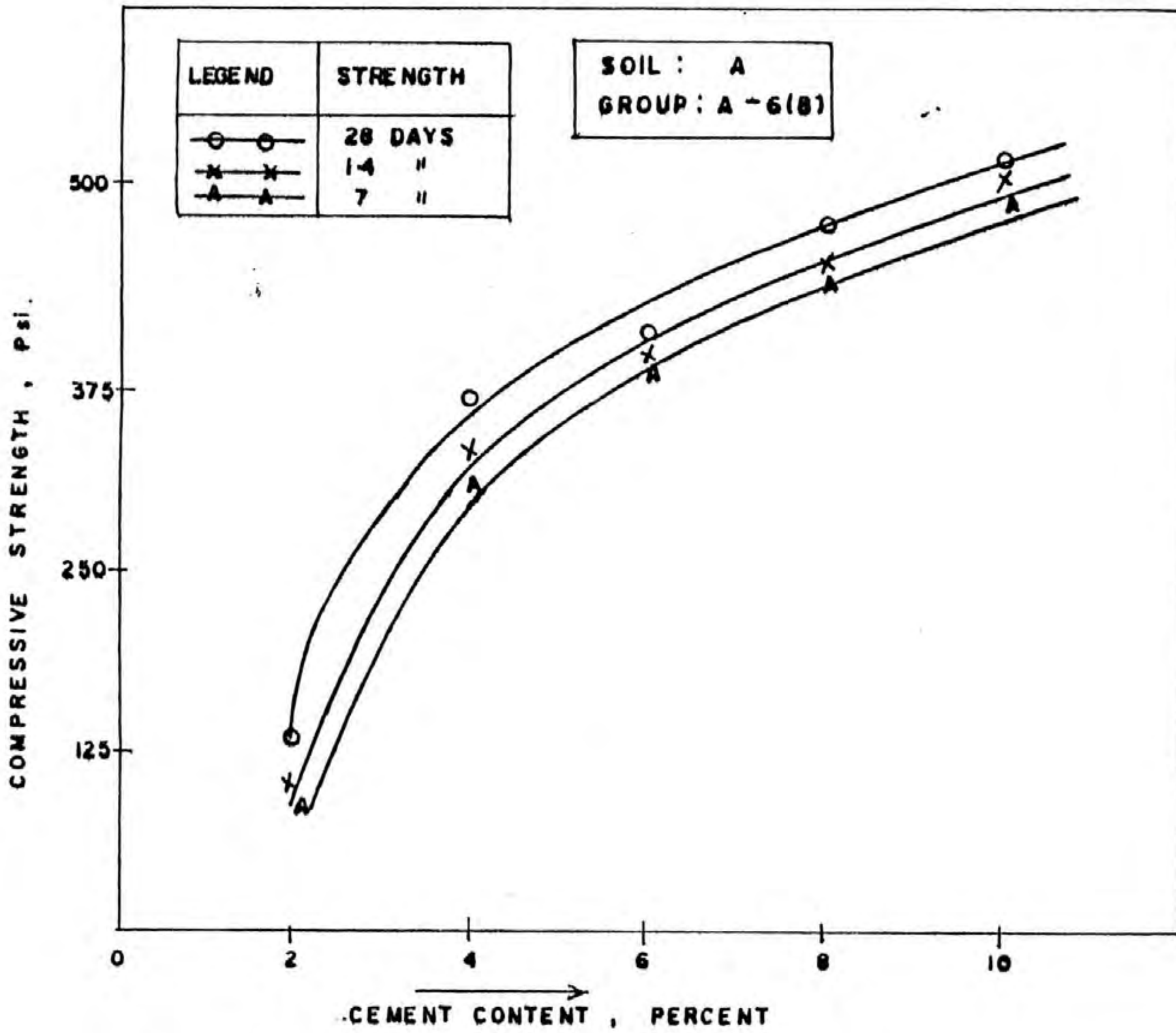


Fig. 5.4 EFFECT OF CEMENT CONTENT ON COMPRESSIVE STRENGTH.

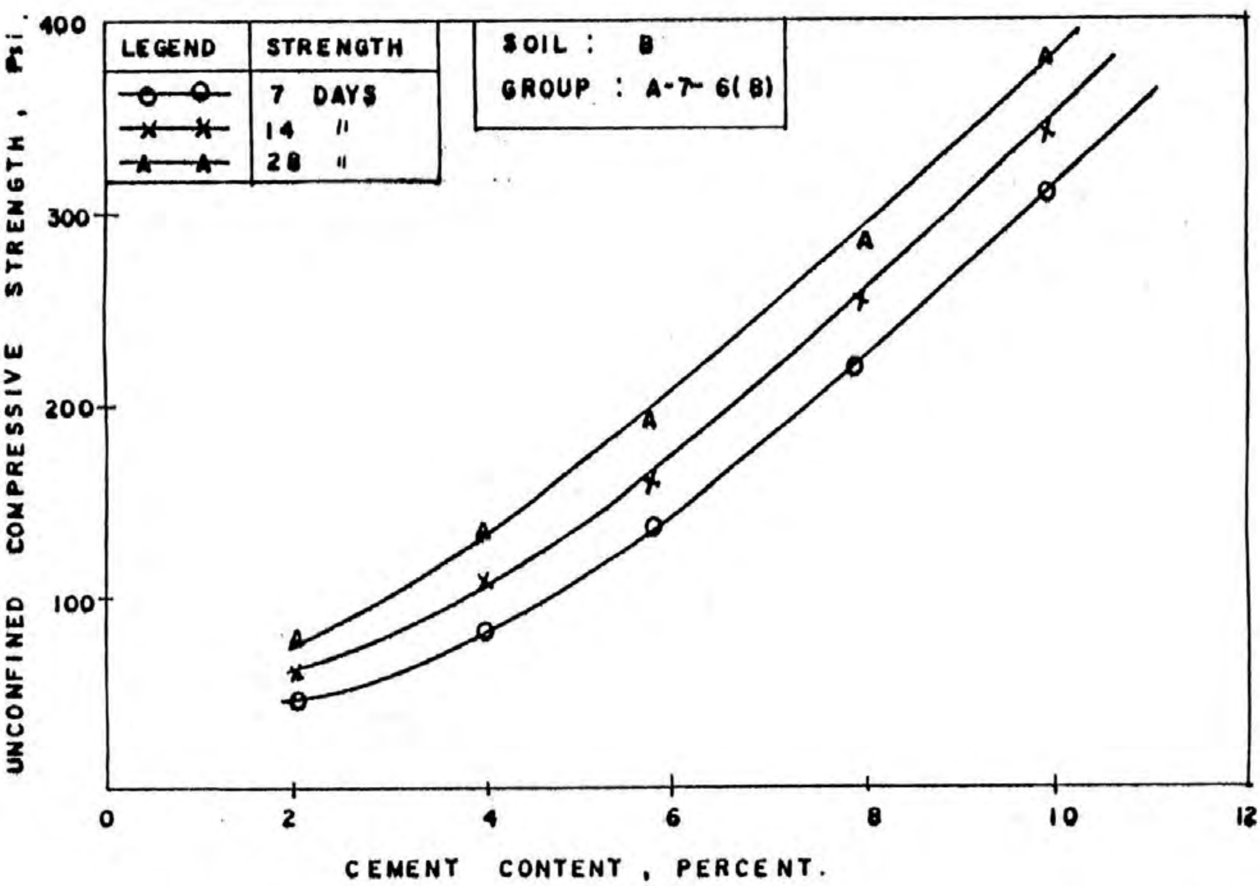


Fig · 5·5 EFFECT OF CEMENT CONTENT ON COMPRESSIVE STRENGTH OF SOIL – B.

Table 5.1 Strength gain cement stabilized soil voer untreated soil

Soil sample	Cement content, (%)	UCCS/UC		
		7 days	14 days	28 days
	0			
	2	5.08	6.34	8.37
	4	19.88	21.15	23.78
A	6	24.34	25.39	26.54
	8	28.78	29.32	29.83
	10	31.12	31.94	32.61
	2	4.05	5.01	6.37
	4	7.07	8.49	10.37
B	6	11.04	13.48	15.99
	8	17.06	21.45	22.88
	10	24.47	26.00	29.92

For Soil-A, it is seen that with the addition of a small amount of cement (i.e 2%) this strength ratio is 5.08 for 7 day curing period and 8.37 for 28 day curing period. For 10 percent cement content, this ratio is 31.12 for 7 day curing and 32.61 with 28 day curing. So it is clear that for this soil, 5 times increasing of cement does not produce that much increase in strength for higher curing period. Soil-B also shows the same relationship.

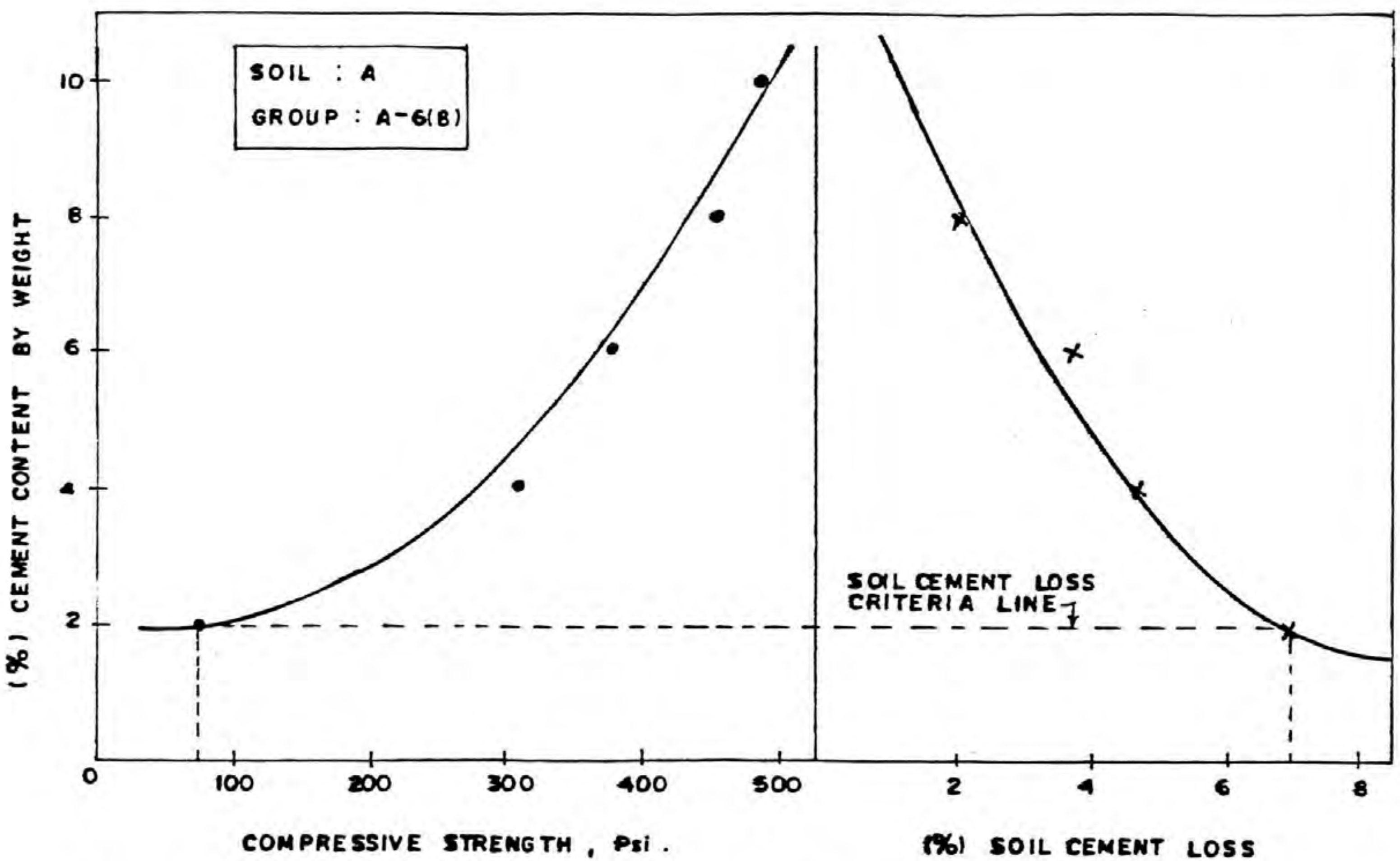


Fig. 5.6 ESTIMATION OF MINIMUM CEMENT CONTENT AND CORRESPONDING UNCONFINED COMPRESSIVE STRENGTH.

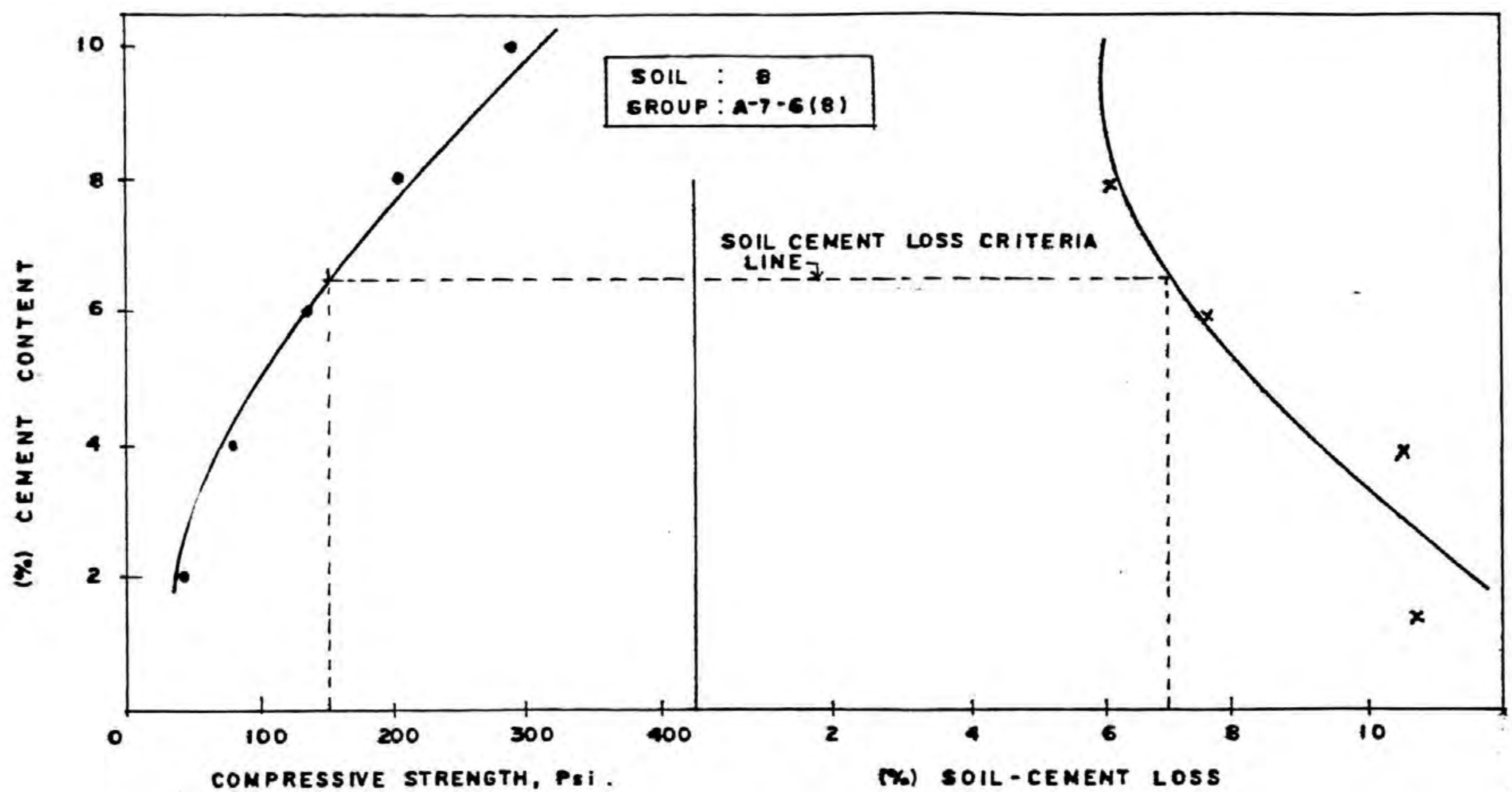


Fig. 5.7 ESTIMATION OF MINIMUM CEMENT CONTENT AND CORRESPONDING COMPRESSIVE STRENGTH.

It is observed that for Soil-A strength values are greater than those for Soil-B at similar cements and curing periods. From textural composition as shown in Table 3.1 it is seen that Soil-A contains 8% fine sand, 69.23% silt and 22.77% clay, and Soil-B contains 13 fine sand, 67% silt and 20% clay. Presence of clay in greater proportion in Soil-A may contribute to its higher strength development compared to Soil-B. Similar findings were reported by early researchers, Mustaque (1986) and Ahmed (1984).

Figure 5.6 shows that the 7 day unconfined compressive strength of Soil-A for 2.0% cement content at which the PCA criteria of soil cement loss is satisfied as shown in Art. 5.2.1 is only 65 psi. From Fig. 5.7 for Soil-B, that unconfined compressive strength is 150 psi for 6.7% cement content at which the PCA criteria is satisfied as shown in Art. 5.2.1. Thus the unconfined compressive strengths for the soil-A is much below the range of strength mentioned in Table 2.0 by PCA. But if we use 4% cement content, then the 7 day strength satisfied the PCA criteria. For Soil-B, addition of 7% cement gives the strength which is closer to the strength by PCA (Table 2.0).

5.3.2 Unconfined Compressive Strength for Soil-Lime

The relation between unconfined compressive strength and percentage of lime, cured for different periods are presented in Figure 5.8 for Soil-A and Soil-B. From Fig. 5.8 and Table A-3, it is seen that the compressive strength gradually inecreased with

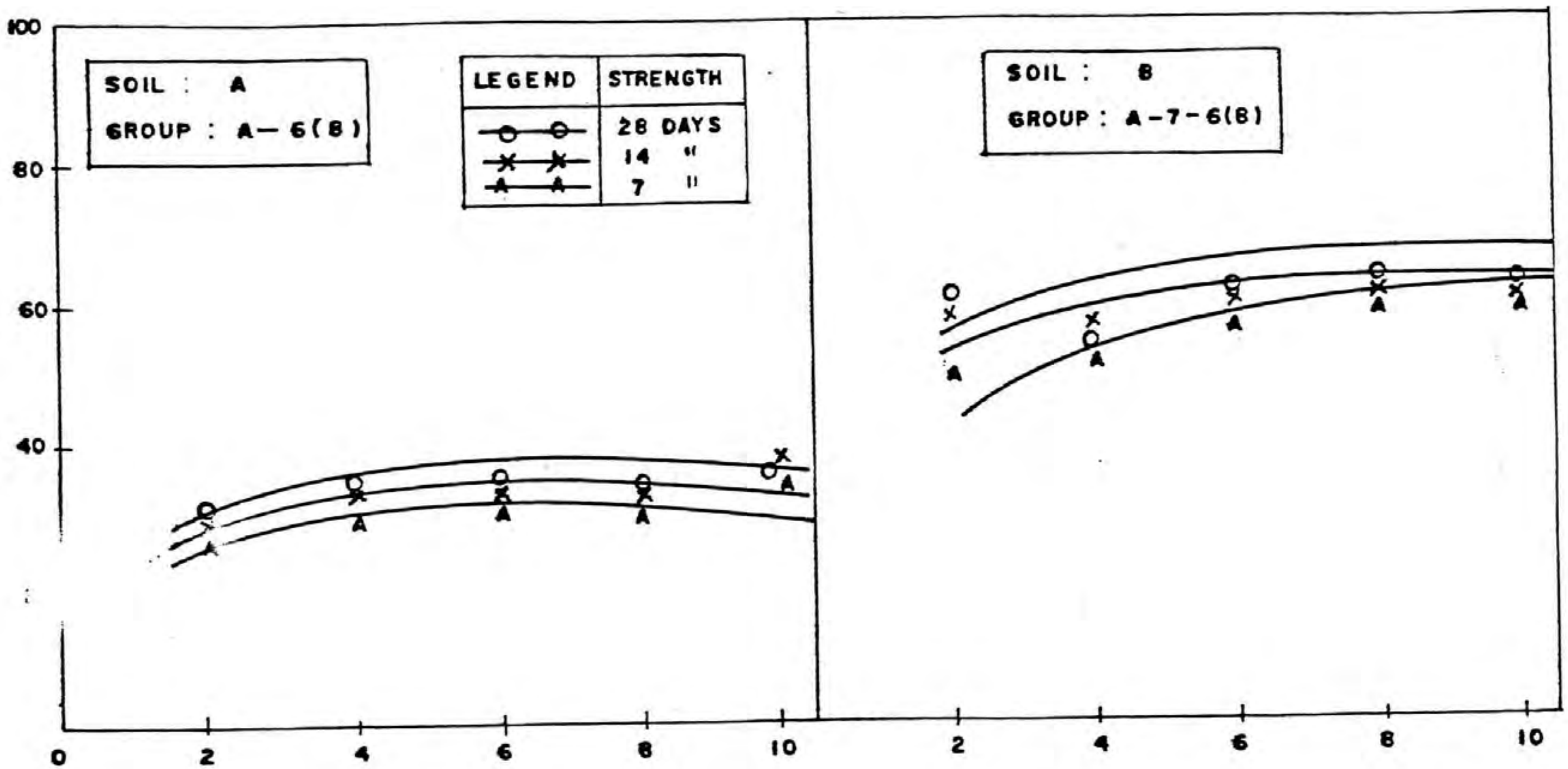


Fig. 5.8 VARIATION OF COMPRESSION STRENGTH AS A FUNCTION OF THE LIME CONTENT.

the addition of lime. Similar findings were reported by Thomson and Neubaur (1965). They concluded that as the lime contents were increased above an optimum amount, the reduction in strengths was occured. From Fig. 5.8 for Soil-B, it is seen that due to addition of lime the strength first increases and then decreases. This result also confirms the findings of Thomson and Neubaur (1965). Ahmed (1984) showed that a silty clay soil provides an increase in strength when stabilised with lime admixtures.

The optimum moisture content for Soil-B is greater than that of Soil-A. This may lead to increase of unconfined compressive strength for lime stabilized Soil-B. Croft (1964) suggested that the increased optimum moisture content can possibly be attributed to the increase in hydroxide ion concentration which modifies the surface of clay particles and increase the strength.

5.4 Plasticity Indices

5.4.1 Cement Treated Mixtures

The variation of Atterberg Limits and the Plastic Limits with the increments of cement contents is shown in Appendix. For cement stabilized soil, it is seen that the plastic limit and liquid limit increase with the increasing of cement content. For both the soils, increase in plastic limit is appreciable resulting in decrease in plasticity index at higher cement content. Similar conclusion was drawn by Mustaque (1986). From Fig. 5.9, it is

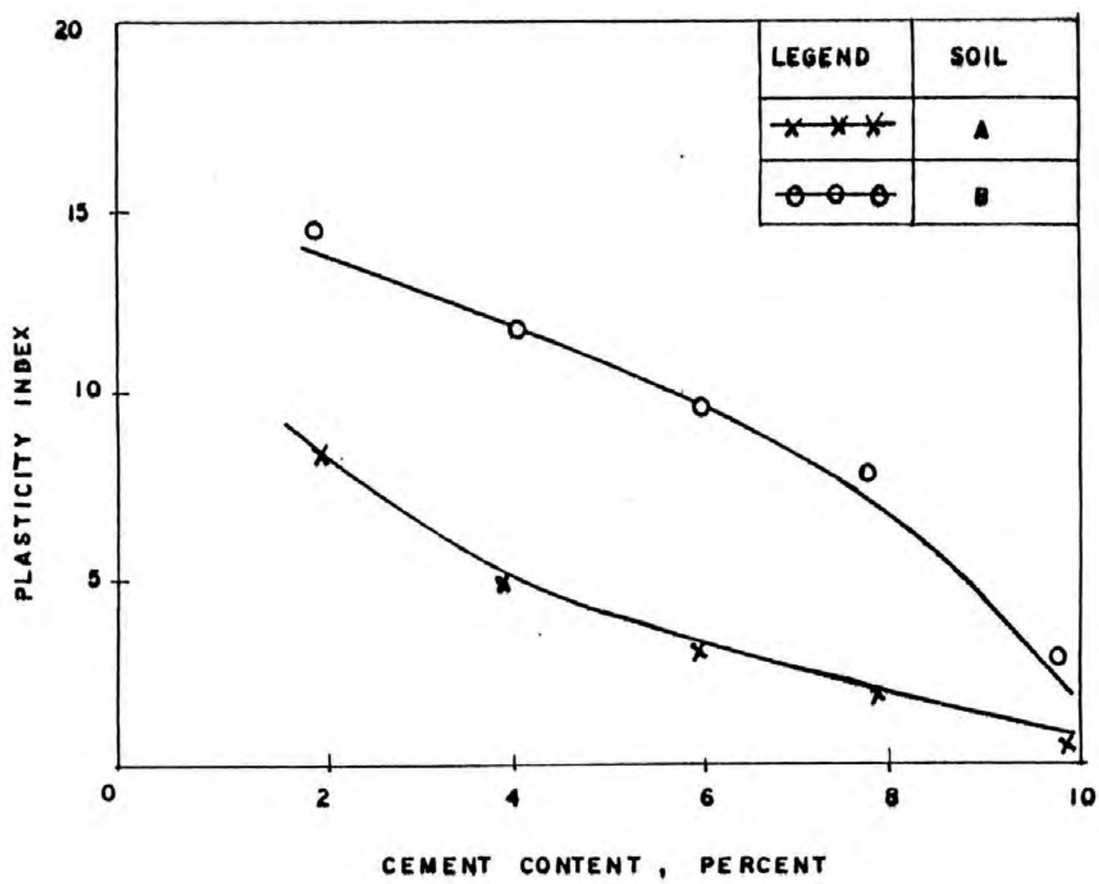


Fig. 5.9 EFFECT OF CEMENT ADDITION ON PLASTICITY INDEX OF SOIL.

seen for both the soils that the plasticity index decreases as the cement content increases. Similar findings were reported by Redus (1958).

5.4.2 Lime Treated Mixture

The effect of addition of lime on plasticity index of soils is presented in Table A-8. From the table it is seen that after addition of 4 and 6 percent lime to Soil-A and Soil-B respectively, the soils become non-plastic. As the Soil-B is more palstic than Soil-A, it needs more lime to be non-plastic. Thomson and Neubaner (1969) carried out an experiment on two soils. They found that A-7(2) and A-6(10) soils became non-plastic after addition of lime 2% and 6% respectively. It is observed for both the soils that plastic limit increases with the addition of lime. Similar findings were reported by Hilt and Davidson (1960). Liquid limit for Soil-A and Soil-B increases with the increment of lime content. But this result does not conform with the findings of Uppal and Bhatia (1958). they found that liquid limit decreased with increasing quantity of lime.

5.5 Strength in Terms of CBR Value

5.5.1 Cement Treated Mixture

Figure 5.10 and Figure 5.11 show that the variation of California Bearing Ratio value with the variation of cement content for Soil-A and Soil-B respectively. From Fig. 5.10 and Table A-9 it

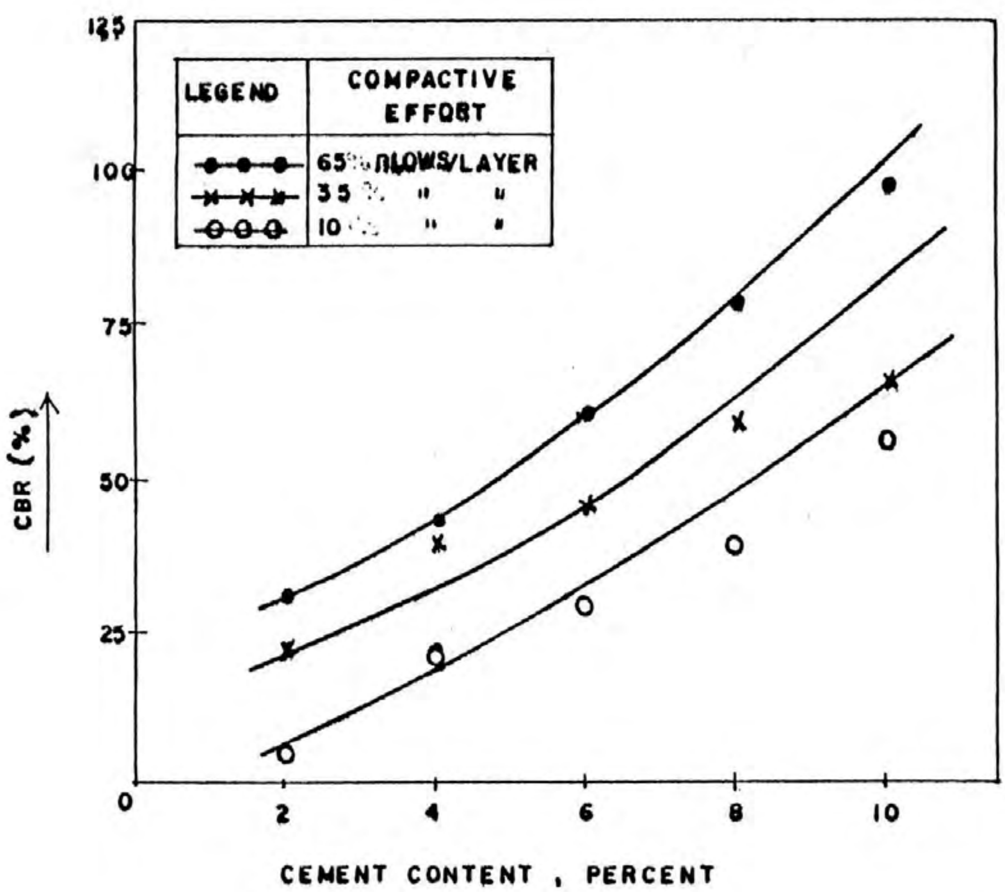


Fig. 5.10 EFFECT OF CEMENT TREATMENT ON CBR OF SOIL-A ·

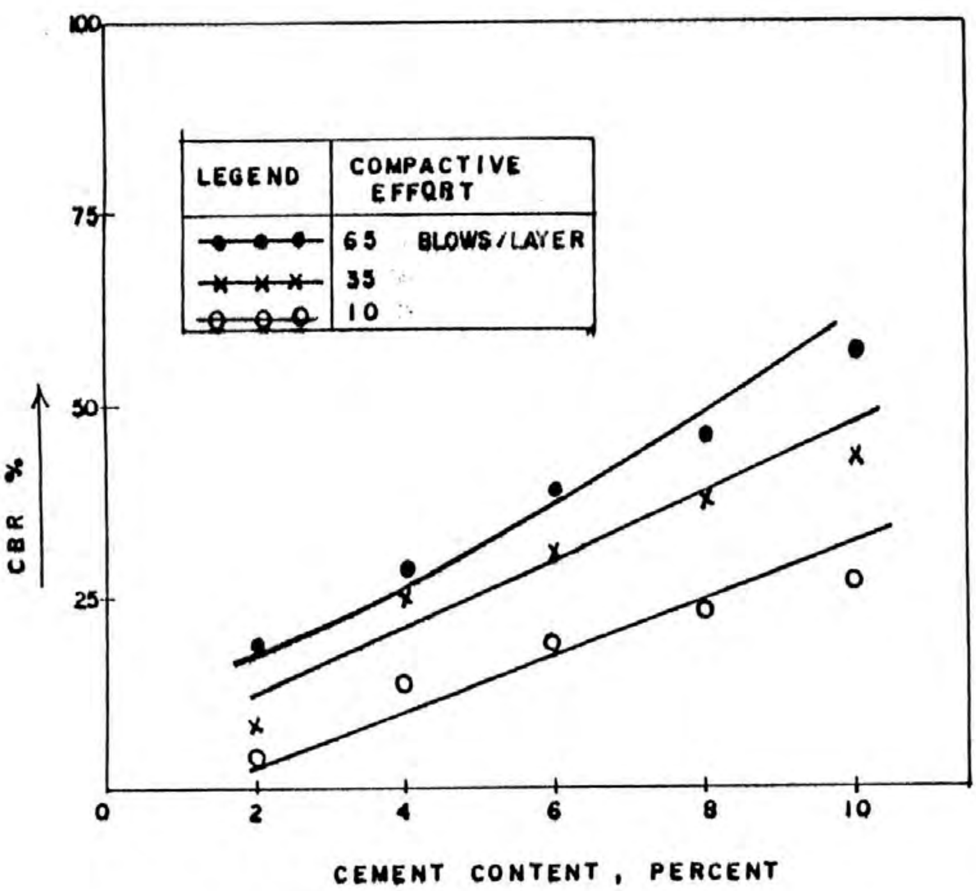


Fig. 5.11 EFFECT OF CEMENT TREATMENT ON CBR OF SOIL-B.

-is seen that the higher percent of cement content increases the CBR values at an increasing rate. Similar findings were reported by Bangladesh Consultant Limited (1986). The CBR value also depends on the compaction. It increases with the increasing of compactive effort. The CBR value of stabilized soil having 2 percent cement content at which the durability criteria is satisfied is 3.1.

From literature review, we find that this value belongs to general rating "good" when used as subgrade, base, subbases. Fig. 5.11 also shows the increment of CBR value with the increment of cement content. For Soil-B the CBR values are lower than that of Soil-A. BCL (1986) reported that for the addition of 4 percent cement on two soils having 15 percent and 16 percent optimum moisture contents, the latter one gave the greater CBR value. This is contrary of this research. The Soil-B having greater optimum moisture contents 20 percent than that of Soil-A when stabilised with 4 percent cement results lower CBR value than Soil-A.

5.5.2 Lime Stabilized Mixture

Figure 5.12 and Figure 5.13 show the relationship between lime contents and California Bearing Ratio value. It is seen from both the figures that there is no regular increment of CBR value with the increment of lime contents. Fig. 5.12 and Fig. 5.13 reveal that the CBR value increases with increasing of lime. Similar

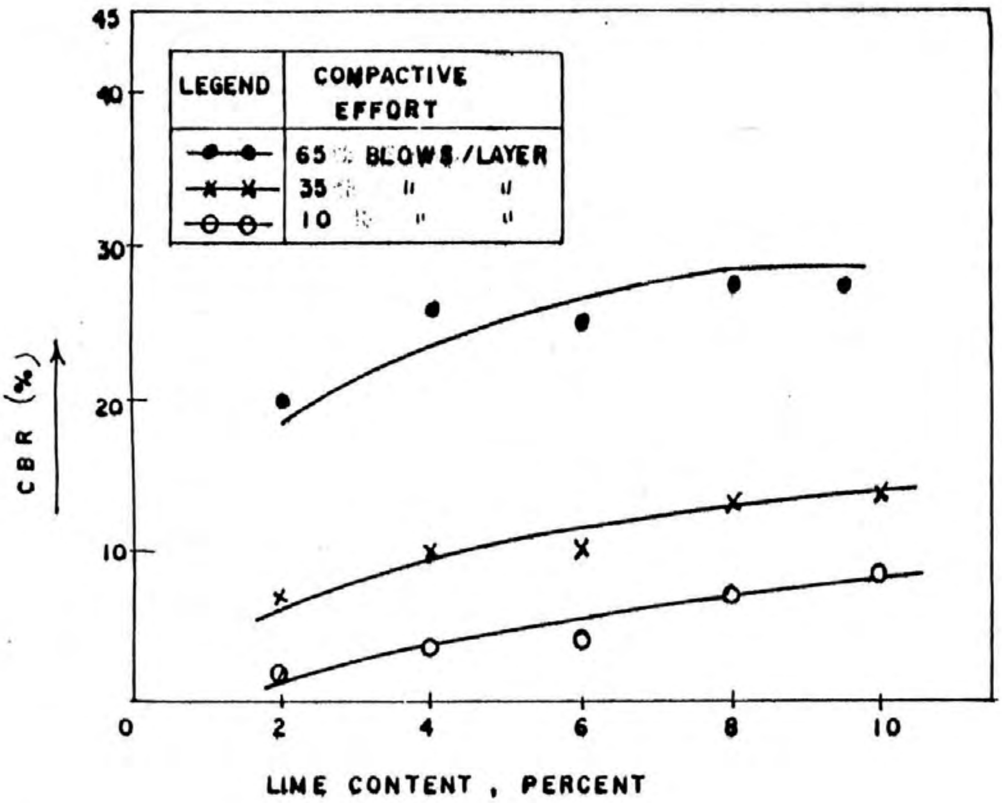


Fig. 5.12 EFFECT OF LIME ON CBR OF SOIL-A.

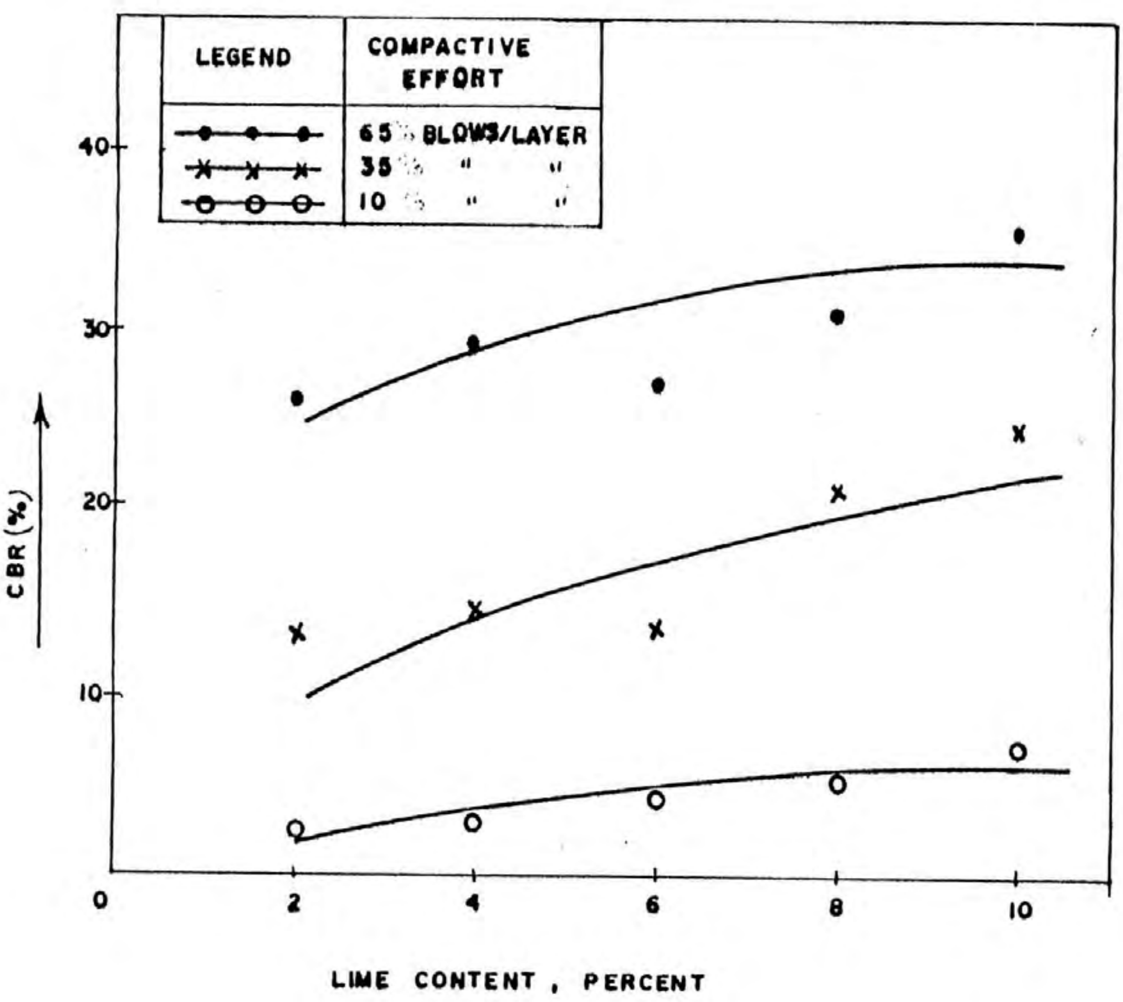


Fig. 5.13 EFFECT OF LIME TREATMENT ON CBR OF SOIL B .

results was reported by Arman and Manfakh (1968) where they concluded that the CBR value increased due to addition of lime upto lime fixation pointand then decreased. But in this research lime fixation point was not ascertained. More tests should be done to established this above fact.

5.6 Evaluation of Subgrade by DCP Value

Dynamic Cone Penetrometer test results of stabilized Soil-B having different amount of cement contents are depicted in Appendix and Fig. 5.14 show the relationship between DCP and CBR value of stabilized subgrade. This developed relationship reveals a quick evaluation technique of compacted stabilized subgrade. If we know the field DCP value of the soil then we can readily find out the subgrade strength of that soil. From Fig. 5.14 it is seen that the CBR value is 23 and 11 corresponding to DCP value of 8 and 49 respectively. Figure 1.4 from literature review shows the CBR value as 25 corresponding to DCP value 21. Here it is seen that for the same DCP value, the CBR value is higher for the stabilised soil than that of untreated one.

The DCP of the subgrade should be found in the worst condition, that is in soaked condition, during the wet season. If it is necessary to determine the field DCP in the dry season, then an area of subgrade should be soaked by ponding for at least a week before the DCP is taken. Several DCP readings should be taken; there will be quite a lot of variation in these readings.

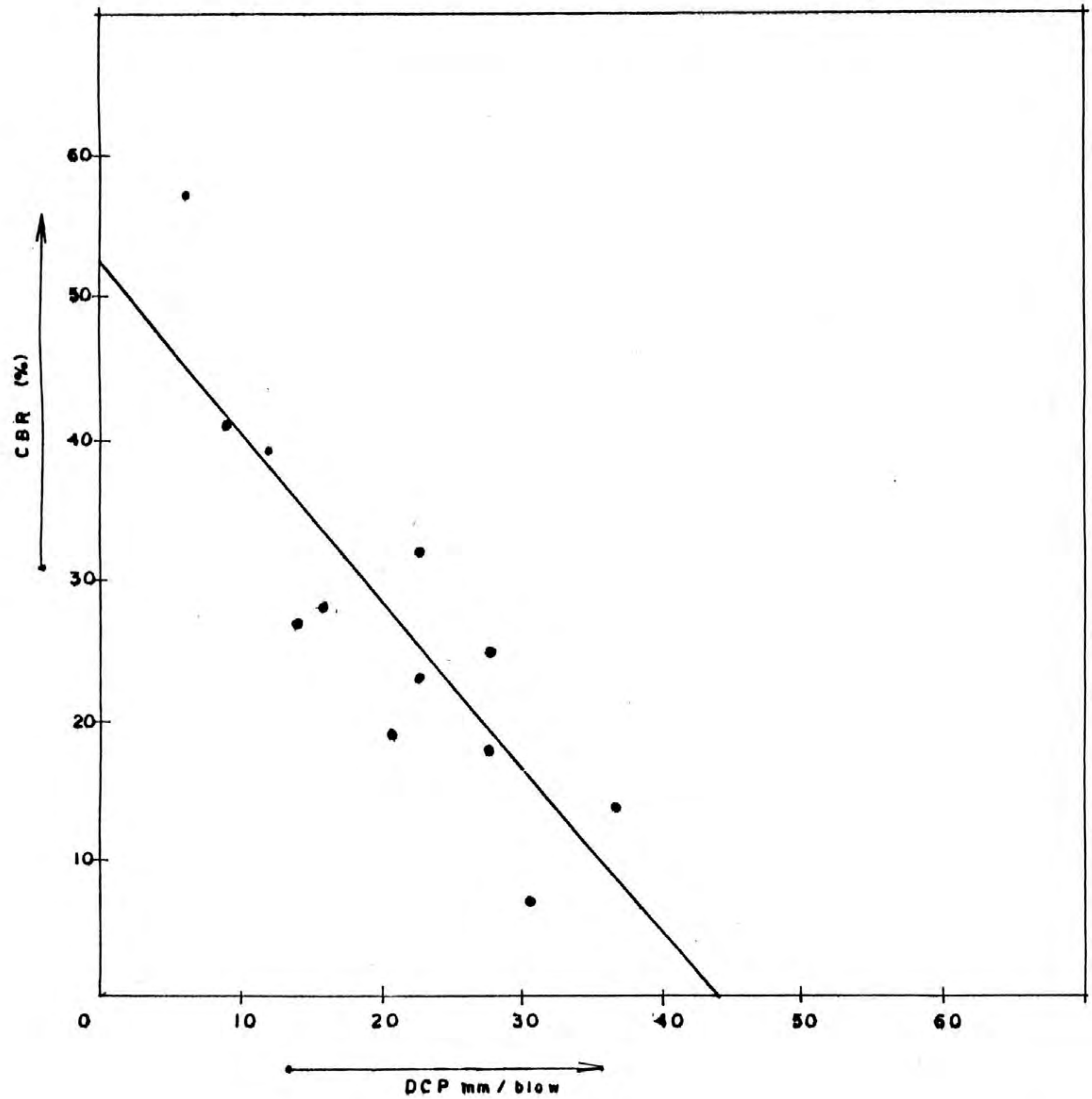


Fig. 5.14 CBR – DCP RELATIONSHIP FOR STABILISED SOIL

From Fig. 5.14 it is seen that there is a negative linear correlation between DCP and CBR values of this investigation which is represented as

CBR = 52 - 1.18 DCP

with a co-efficient of correlation 92 for standard deviation 14.

Bangladesh Road Research Laboratory (1985) reported a linear correlation for untreated soil which showed that

CBR = 58 - 3.05 DCP

So it is possible to correlate the findings of more CBR and DCP test to have a more accurate linear equation.

5.7 Comparison of Cost

The economic analysis is done according to Roads & Highways Department Schedule (RHD, 1990) and BCL schedule assuming a section of length 100 metre, width 10 metre and having 150 mm compacted subgrade. The trial design of pavement is done according to Road Note 31 (TRRL, 1977) and Asphalt Institute Method (Wright & Paquette, 1979). The trial design was done according to following conditions:

A) Road Note 31

Commercial vehicles = 300/day

Growth rate = 6%

Cumulative no. of vehicles = 0.78 million

Design life = 10 years

CBR = 3

The thicknesses of pavement structure becomes

Untreated (CBR=3)	: Subbase	= 340 mm (13")
	Base	= 150 mm (6")
	Surfacing	= 50 mm (2")
Treated with 2% Cement (CBR=31)	: Base	= 150 mm (6")
	Surfacing	= 50 mm (2")

B) Asphalt Institute Method:

Initial daily traffic = 300/day

% heavy trucks = 16

Design life = 10 years

Growth rate = 6%

Thickness:

Untreated:	Base	= 150 mm (6")
	Surfacing	= 50 mm (2")
Treated :	Base	= 75 mm (3")
	Surfacing	= 50 mm (2")

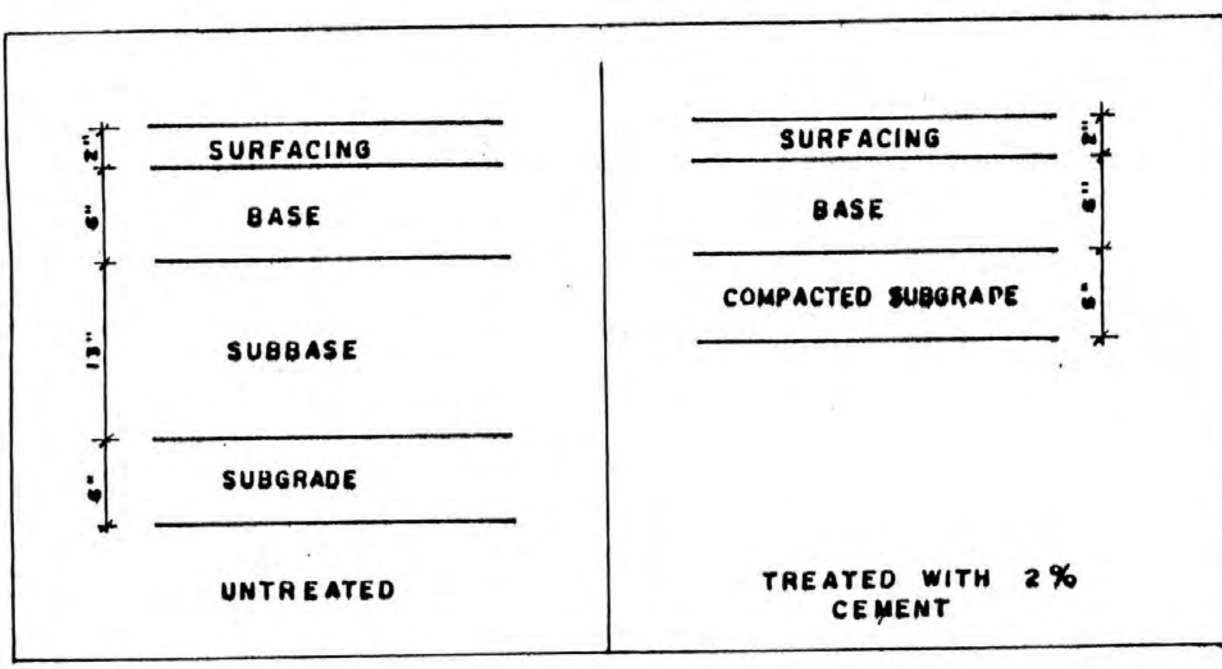


Fig. 5.15 TRIAL SECTION, ROAD Note 31

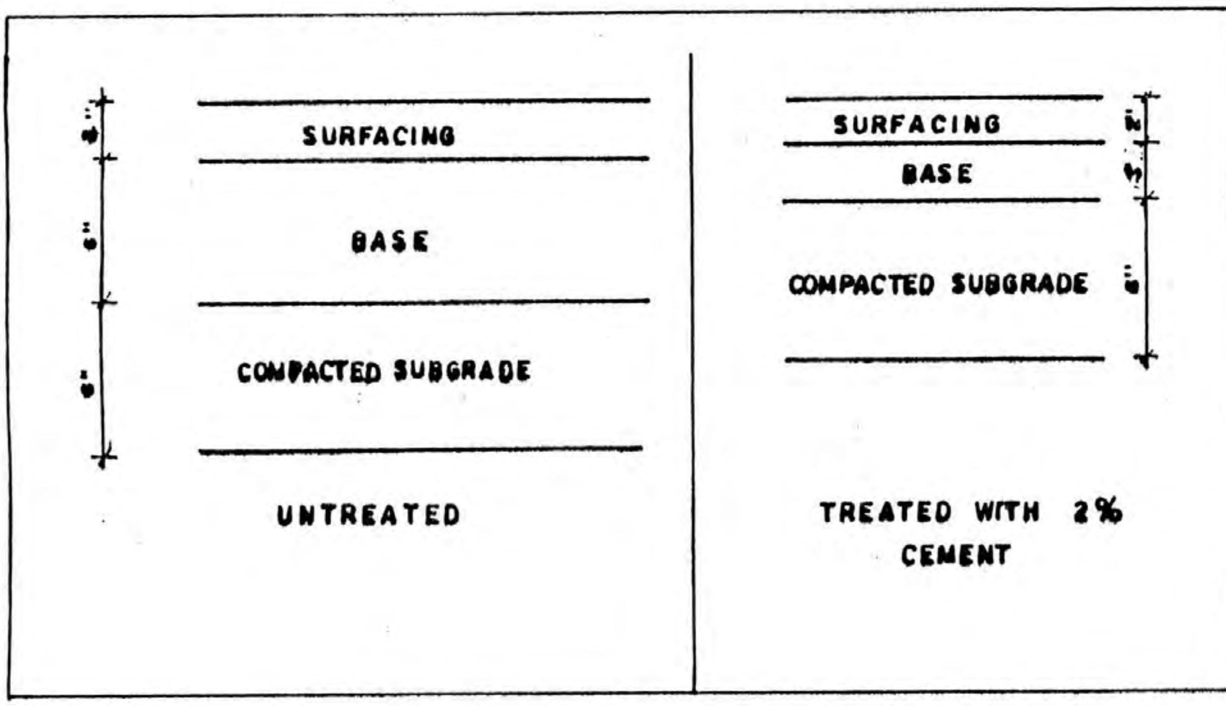


Fig. 5.16 TRIAL SECTION, ASPHALT INSTITUTE METHOD

COSTING

The detail costing of the above sections are given as follows:

Base Course:

Labor for compacted base course requires Tk. 51.00 per cu.m.

Now, say compaction is 40 percent

Therefore, materials requried for 1 cu.m compacted base course = 1.67 cu.m which costs = Tk. 1.67 x 1055.00 = Tk. 1761.85

Therefore, Total cost (Labor + material) = Tk. 1812.85 per cu.m.

Therefore,

1) Compacted subgrade

= 100x10x0.15 = 150 cu.m @ Tk. 90.00 per cu.m = Tk. 13500.00

2) Compacted stabilized subgrade

= 150 cu.m @ Tk. 310.00 per cu.m = Tk. 46500.00

3) Subbase

= 100x10x0.340 = 340 cu.m. @ Tk. 180.00 per cu.m

= Tk. 61200.00

4) Base (150 mm)

= 100x10x0.15 = 150 cu.m @ Tk. 1812.85 per cu.m

= Tk. 271927.50

Base (75 mm)

= 100x10x0.075 = 75 cu.mx1812.85 = Tk. 135963.75

5) Surfacing (50 mm)

= 100x10 = 1000 sq.m @ Tk. 119.50 per sq.m = Tk. 119500.00

A) Road Note 31

	Untreated	Treated
a) Subgrade	= Tk. 13500.00	= Tk. 46500.00
b) Subase	= Tk. 61200.00	= Tk. --------
c) Base	= Tk. 271927.00	= Tk. 271927.00
d) Surfacing	= Tk. 119500.00	= Tk. 119500.00
	Tk. 466127.00	= Tk. 437927.00

B) Asphalt Institute Method

a) Subgrade	= Tk. 13500.00	= Tk. 46500.00
b) Base	= Tk. 271927.00	= Tk. 135964.00
c) Surfacing	= Tk. 119500.00	= Tk.1199500.00
	Tk. 404927.00	Tk. 301964.00

The above cost analysis reveals that stabilization of subgrade soil reduces the thickness of base course and surfacing in both methods followed. This reduction in pavement thickness obviously decreases the construction cost. Similar conclusion was drawn by LGEB (1988) for a cement stabilized soil road constructed as a road base over sand-soil sub-base. But the most difficult aspect in this type they observed and experienced was not its construction but its protection against damage by local traffic during the post construction curing period.

The cost analysis also points out that the saving of construction cost for stabilization over untreated condition using Road Note 31 and Asphalt Institute Method are respectively 6 percent and 25 percent.

5.8 Implementation of the Study

The conventional methods for construction of road pavement structure in Bangladesh is quite expensive both from construction and maintenance point of view. In most developing countries like Bangladesh there is a continuous increase in the cost of construction materials. The main parameters those influence the construction of low cost roads are the cost and availability of suitable materials. Soil is normally the foundation material for any road and is available in Bangladesh.

If the load bearing capacity of the subgrade soil is improved by stabilization, then a lower thickness of road structure is needed and eventually road construction would be economical as shown in article 5.7.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATION FOR FUTURE RESEARCH

6.1 Conclusions

The important findings and conclusions drawn on the various aspects of this research may be summarized with this limited study as follows:

1. The two subgrade soils satisfy the durability criteria recommended by the Portland Cement Association (PCA) at about 2 percent and 7 percent cement contents.

2. The silty clay soils fail to satisfy the minimum unconfined compressive strength criteria of PCA for cement content at which the durability criteria is satisfied. For the type of soils used, higher cement content would be required to satisfy the strength criteria. However the soils showed appreciable strength gain over untreated soil with addition of only 2 percent cement by weight.

3. The unconfined compressive strengths of lime treated soils increase due to the addition of lime.

4. Curing period and proportion of cement and lime significantly influence the strength characteristics of soil-cement and soil-lime mixtures.

5. Due to the addition of lime, the subgrade soils become non plastic. This treatment is suitable for highly plastic soils.

6. There is a sharp increase of CBR value after addition of 2 percent cement over untreated soil. The value then increase at an increasing rate for higher cement content. The CBR value increases slightly after addition of lime.

7. Development of DCP - CBR relationship provides a quick evaluation technique of compacted subgrade.

8. The construction of pavement on a stabilized subgrade is economical than that of an untreated subgrade.

6.2 Recommendation for Further Study

New studies are required for investigation various aspects of soil-cement and soil-lime stablization which cannot be covered fully in this research. These may be listed as below:

1. Only two silty clay soil samples were used in this study. So conclusions based on relatively little data need more study to be confirmed. Also, other types of soil must be investigated on treatment with cement and lime.

2. In this research, investigation were carried out in the laboratory. Field tests and trial constructions are to be studied to check the suitability and adaptation in performance of this method in road construction of Bangladesh.

3. Permeability characteristics, consolidation characteristics and erosion resistance of stabilised mix need to be evaluated to get a through knowledge of moisture change, volume change, and durability of stabilized construction.

REFERENCES

1. Jha, J. and Sinha, S.K. (1977): Construction and Foundation Engineering Handbook, Khanna Publishers, New Delhi.

2. Ingles, D.G. (1968): "Soil Chemistry Relevant to the Engineering Behavior of Soils", Soil Mechanics Selected Topics Edited by I.K, Lee.

3. O'Flaherty, C.A. (174): Highway Engineering Vol. 2, Edward Arnold Publishers, Great Britain.

4. Kezdi, A. (1979): Stabilized Earth Roads, Elsvier Research Board, Washington D.C.

5. Sharma, S.K. (1985): Principles, Practice and Design of Highway Engineering, S. Chand & Company Ltd., New Delhi.

6. Thomson, M.R. (1970): Lime Reactivity of Illiois Soils. Jour. Soil Mech. and Found., Divi, Proct, ASCE, Vol. 92. No. SM 5.

7. Snetty, M. S. (1982): Concrete Technology, S. Chand Company Ltd., New Delhi.

8. ASTM (1979a): Annual Book of ASTM (American Society for Testing Materials) Standards, Part 19, Stones, Soil and Rock.

9. ASTM (1979b): Annual Book of ASTM Standards, Part 13, Cement, Lime, Ceiling and Walls.

10. Mustaque A (1986): Stabilization of Local Alluvial Soils, with Cement and Cement Rice Hush", M.Sc.Engg. Thesis, Dept. of CE, BUET, Dhaka.

11. Balmer, G.G. (1958): "Shear Strength and Elastic Properties of Soil-Cement Mixtures under Triaxial Loading", ASTM Proc. Vol. 58.

12. Christensen, A.P. (1969): Specification for Road and Bridge Works, London, H. M.S.U.

13. Ministry of Transport (U>K) (1969): Specification for Road and Bridge Works, London, H. M.S.U.

14. Portland Cement Association (PCA) (1956): Soil-Cement Construction Handbook, Chicago, Illinois, USA.

15. Portland Cement Association (PCA) (1959): Soil-Cement Laboratory Handbook, Chicago, Illinois, USA.

16. Kemahlioglu, A., Higgns, C.M. and Adam, V. (1967): "A rapid method for Soil-Cement design; Louisiana Slope Value Method", Highway Research Record 198, Highway Research Board, Washington D.C.

17. Redus, J.F. (1958): "Study of Soil-Cement Base Course on Military Airfields", Paper Presented at the 37th Annual Meeting of the Highway Research Board.

18. Ahmed N.U. (1984): "Geotechnical Properties of Selected Local Soils Stabilized with Lime and Cement", M.Sc. Engg. Thesis, Dept. of Civil Engg., BUET, Dhaka.

19. AASHO (1966): American Association of State Highway Officials Standard Specifications for Highway Materials and Methods of Sampling and Testing - Part - II, Washington D.C., U.S.A.

20. LGEB (1985): Road Construction Trial Project, Faridpur.

21. Croft, J.B> (1964): "The Process Involved in the Lime Stabilization of Clay Soils", Proceedings of the Australian Road Research Board, Vol. 2, Part 2, Melbourne.

22. Uppal, H.L. and Bhatia, H.S. (1958): "Stabilization of Black Cotton Soilfor Use in Road Construction" RR Bulletin No. 5, IRC, New Delhi.

23. Jan, M.A. and Walker, R.D. (1963): "Effect of Lime, Moisture and Compaction of a Clay Soil", Highway R, Reocrd, No. 29, Highway R. Board, Washinton.

24. Clare, K.E. and Pollard, A.E. (1953): "The Process Involved in the Lime Stabilization of Clay Soils", Proc. of the Australian Road Research Board, Vol. 2, Part - 2, Melbourne.

25. Hilt, e.H. and Davidson, D.T. (1960): Lime Fixation in Clayey Soils", Bulletin 262, Highway Research Board, W.S.

26. Johnson, A.H. (1948): "Lab. Exp. with Lime Soil Mixture", Proceedings, 38th Annual Meeting Highway Research Board, W.S.

27. Ladd, e.e, Moh. Z.e. and Lamb T.W. (1960): "Recent Soil, Lime Research at the Massachussetts I of T" Bullatin 262, Highway Research Board, Washington D.C.

28. Andrews, D.C. and O'Flaterty, C.A. (1968): "Lime Type and Quality in Relation to the Stabilization of Soils with Gradation and Clay Minerals", Australian Road Research Board, Proceedings, of the Conference, Melbourne.

29. Neubauer, e.H. and Thomson, M.R. (1968): "Stability Properties of Uncured Lime Treated Fine Grained Soils", Proceedings of the Committee on Lime and Lime Flyash Stabilization, Vol. 2, Part-1, Washington.

30. Arman Ara and Manfakh, G.A (1968) " Lime stabilisation of Organic Soils", -- Proceedings on the 30th Annual Convention of or Highway Engineering Sponsored by Lime and Lime Flyash Stabilization Committee.

31. Bangladesh road Researdh Laboratory (1985) a Booklet on Dynamic Cone Penetrometer Test, Dhaka.

32. Roads and Hihghways Department (1990): Schedule of Rates for Road and Bridge Works.

33. Transport and Road Research Laboratory (1977): A Guide to the Structural Design of Bituminous Surface Roads in Tropical and Subtropical Countries, Minsitry of Transport, Road Note No. 31, 3rd Edition, U.K.

34 Wright, A. Paul and Paquette, J. Rander (1979): "Highway Engineering Ae" 4th Edition, U.K.

35. Catton, M.D. (1940): "Researfh on the Physical Relations of Soil and Soil-Cement Mixtures", Proc. Highway Research Board, Vol. 20, pp. 821 - 855.

36. Bangladesh Transport Survey (1974): Geology of Bangladesh.

APPENDIX

Table A-1 Unconfined Compressive Strength Test Results for Cement Treated Soil-A

Cement content, (% by wt.)	7 days			14 days			28 days		
	Unconfined compressive strength, psi, UCCS	Failure strain	Specimen moisture content of unconfined test	UCCS (psi)	Failure strain	Specimen moisture content of unconfined test	UCCS (psi)	Failure strain	Specimen moisture content of unconfined test
2	80.01	1.75	20.56	99.71	2.00	22.32	131.74	1.75	23.91
4	312.66	2.30	20.20	332.67	1.50	21.51	374.02	2.00	23.00
6	382.85	1.50	19.56	399.36	2.10	20.20	417.52	2.00	24.10
8	452.76	2.10	23.25	461.16	2.00	24.52	469.23	2.15	27.35
10	489.51	4.15	18.32	502.50	4.30	21.16	513.00	4.45	26.97

Table A-2 Unconfined Compressive Strength Test Results for Cement Treated Soil-B

Cement content, (% by wt.)	7 days			14 days			28 days		
	Unconfined compressive strength, psi, UCCS	Failure strain	Specimen moisture content of unconfined test	UCCS (psi)	Failure strain	Specimen moisture content of unconfined test	UCCS (psi)	Failure strain	Specimen moisture content of unconfined test
2	48.29	1.75	20.17	59.62	1.75	23.47	75.90	2.00	25.52
4	84.21	1.25	23.59	101.15	1.50	24.92	123.48	1.75	24.52
6	131.52	1.50	21.60	160.52	1.50	23.20	190.50	2.00	25.19
8	203.17	1.25	25.70	255.50	1.25	26.50	272.60	1.25	27.70
10	291.50	1.00	24.92	309.69	1.50	25.15	356.32	1.25	26.50

Table A-3 Unconfined Compressive Strength Test Results for Lime Treated Soil-A

Cement content, (% by wt.)	7 days			14 days			28 days		
	Unconfined compressive strength, psi, UCLS	Failure strain	Specimen moisture content of unconfined test	UCLS (psi)	Failure strain	Specimen moisture content of unconfined test	UCLS (psi)	Failure strain	Specimen moisture content of unconfined test
2	82.79	2.90	13.55	82.90	2.55	13.10	84.15	2.90	13.25
4	32.67	3.20	12.90	32.50	3.00	13.20	32.40	3.00	12.55
6	33.55	3.56	12.75	34.10	3.70	12.55	34.59	3.40	12.10
8	31.90	5.35	13.65	32.55	4.10	13.22	33.00	4.00	13.22
10	28.51	4.53	16.00	31.50	4.21	16.75	31.45	4.11	16.00

Table A-4 Unconfined Compressive Strength Test Results for Lime Treated Soil-B

Cement content, (% by wt.)	7 days			14 days			28 days		
	Unconfined compressive strength, psi, UCLS	Failure strain	Specimen moisture content of unconfined test	UCLS (psi)	Failure strain	Specimen moisture content of unconfined test	UCLS (psi)	Failure strain	Specimen moisture content of unconfined test
2	50.52	2.20	20.57	58.23	2.10	19.23	60.51	2.20	20.95
4	52.55	2.80	24.33	52.90	2.85	23.50	50.00	2.55	22.00
6	57.60	2.95	26.53	59.20	2.95	24.00	60.15	2.60	22.55
8	59.95	4.10	23.50	60.25	3.96	20.05	60.86	3.23	20.00
10	58.66	3.57	22.70	59.25	3.10	20.72	60.00	3.00	20.25

Table A-5 Maximum Volume Change and Maximum Moisture Content During Wet-dry Test of Cement Stabilized Soils

Soil	Cement content %	Maximum Volume Change, %		Maximum moisture content, %
		Increase	Decrease	
	2	-	1.05	16.72
	4	-	1.08	15.45
A	6	-	1.26	18.55
	8	-	1.52	18.57
	2	-	1.07	23.57
	4	-	1.15	23.62
B	6	-	1.51	25.58
	8	-	1.83	26.72

Table A-6 Effect of Addition of Cement on Soil-Cement Loss

Soil	Cement content, %	Soil-Cement Loss, Percent
	2	6.90
	4	4.75
A	6	3.97
	8	2.12
	2	10.62
	4	10.51
B	6	7.82
	8	6.25

Table A-7 Effect of Addition of Cement on Plasticity Index of Soils

Soil	Cement content percent	Atterberg Limits		Plasticity Index
		Liquid limit (%)	Plastic Limit (%)	(%)
A	2	33.0	24.5	8.5
	4	34.0	29.0	5.0
	6	35.5	32.0	3.5
	8	36.0	33.5	2.5
	10	38.0	37.0	1.0
B	2	36.5	22.0	14.5
	4	38.0	25.5	12.5
	6	39.5	29.0	10.5
	8	43.0	35.0	8.0
	10	44.5	41.5	3.0

Table A-8 Effect of Addition of Lime on Plasticity Index of Soils

Soil	Lime content,	Atterberg Limits		Plasticity Index
	%	Liquid Limit %	Plastic Limit %	%
A	2	31.0	30.5	0.5
	4	33.0	33.0	0
	6	-	-	-
	8	-	-	-
B	2	39.5	37.0	2.5
	4	41.5	40.0	0.5
	6	45.0	45.0	0
	8	-	-	-

Table A-9 Effect of Cement Treatment on CBR of Soil A.

Cement content, percent	Dry density (lb/ft³) at compactive efforts			CBR values in percent at compactive efforts		
	14,300 lb-ft/ft³	7,700 lb-ft/ft³	2,200 lb-ft/ft³	14,300 lb-ft/ft³	7,700 lb-ft/ft³	2,200 lb-ft/ft³
2	108.62	107.72	95.50	31	23	4
4	116.89	114.37	98.40	44	35	21
6	118.51	115.4	99.00	61	46	29
8	122.0	119.47	100.63	78	59	39
10	128.57	122.97	103.70	97	62	51

Table A-10 Effect of Cement Treatment on CBR of Soil B

Cement content, percent	Dry density (lb/ft³) at compactive efforts			CBR values in percent at compactive efforts		
	14,300 lb-ft/ft³	7,700 lb-ft/ft³	2,200 lb-ft/ft³	14,300 lb-ft/ft³	7,700 lb-ft/ft³	2,200 lb-ft/ft³
2	103.03	100.38	88.97	19	87	3
4	99.93	94.82	84.00	28	25	14
6	99.84	94.10	84.20	39	32	18
8	100.28	95.40	89.38	46	38	23
10	103.10	99.92	94.50	57	44	27

Table A-11 Effect of Lime Treatment on CBR of Soil A

Cement content, percent	Dry density (lb/ft³) at compactive efforts			CBR values in percent at compactive efforts		
	14,300 lb-ft/ft³	7,700 lb-ft/ft³	2,200 lb-ft/ft³	14,300 lb-ft/ft³	7,700 lb-ft/ft³	2,200 lb-ft/ft³
2	102.09	96.05	84.1	20	8	1.5
4	103.10	96.9	85.22	26	10	3
6	103	97	85	25	10	3
8	105.67	96.90	85.30	27	13	7
10	105.90	99.50	87.40	26.5	13.50	8

Table A-12 Effect of Lime Treatment on CBR of Soil B

Cement content, percent	Dry density (lb/ft³) at compactive efforts			CBR values in percent at compactive efforts		
	14,300 lb-ft/ft³	7,700 lb-ft/ft³	2,200 lb-ft/ft³	14,300 lb-ft/ft³	7,700 lb-ft/ft³	2,200 lb-ft/ft³
2	104.34	95.13	84.78	26	13	2
4	93.12	92.19	80.10	29	14	2
6	97.30	93.9	84.23	23	8	3
8	101.51	98.5	89.30	31	21	5
10	104.0	95.35	91.70	35.5	24	7

Table A-13 Dynamic Cone Penetrometer Test Data on Treated Soil B

Cement content	No. of blows	Compactive efforts 14,300 lb-ft/ft^3			Compactive Efforts 7,700 lb-ft/ft^3			Compactive Efforts 2,200 lb-ft/ft^3		
		Penetration in mm			Penetration in mm			Penetration in mm		
		1	2	3	1	2	3	1	2	3
	Initial	31	32	32	31	31	31	51	51	50
	1	57	57	58	69	68	67	150	150	150
	2	75	80	80	93	94	93			
	3	92	93	92	122	122	125			
2%	4	122	125	121						
	5	137	137	138						
	6	151.0								
	7	153								

Table A-14 Dynamic Cone Penetrometer Test on Treated Soil B

Cement content	No. of blows	Compactive efforts 14,300 lb-ft/ft^3			Compactive Efforts 7,700 lb-ft/ft^3			Compactive Efforts 2,200 lb-ft/ft^3		
		Penetration in mm			Penetration in mm			Penetration in mm		
		1	2	3	1	2	3	1	2	3
	Initial	34	34	34	37	37	57	39	41	41
	1	64	65	64	71	72	70	90	92	90
	2	78	78	79	97	97	94	123	122	126
4%	3	91	91	90	120	121	120	150	150	149
	4	111	110	111						
	5	119	120	119						
	6	129	128	128						

Table A-15 Dynamic Cone Penetrometer Test Data on Treated Soil B.

Cement content	No. of blows	Compactive efforts 14,300 lb-ft/ft^3			Compactive Efforts 7,700 lb-ft/ft^3			Compactive Efforts 2,200 lb-ft/ft^3		
		Penetration in mm			Penetration in mm			Penetration in mm		
		1	2	3	1	2	3	1	2	3
	Initial	34	34	34	37	38	37	41	40	41
	1	54	54	53	71	72	71	78	76	80
	2	65	64	64	89	90	89	100	102	111
	3	74	75	74	119	119	120	122	123	129
6%	4	84	84	84	129	129	129	150	150	150
	5	96	95	96						
	6	105	105	105						
	7	115	116	118						
	8	127	128	131						

Table A-16 Dynamic Cone Penetrometer Test Data on Treated Soil B.

Cement content	No. of blows	Compactive efforts 14,300 lb-ft/ft³			Compactive Efforts 7,700 lb-ft/ft³			Compactive Efforts 2,200 lb-ft/ft³		
		Penetration in mm			Penetration in mm			Penetration in mm		
		1	2	3	1	2	3	1	2	3
	Initial	27	27	28	27	27	27	37	37	37
	1	50	53	50	50	50	50	62	65	61
	2	58	59	57	62	60	60	89	89	85
	3	66	69	66	69	69	69	102	111	106
8%	4	74	78	75	84	80	81	123	124	121
	5	80	84	80	95	93	93	146	149	146
	6	86	87	87	106	105	104			
	7	94	97	94	117	114	114			
	8	101	101	100	123	123	124			
	9	108	108	109						
	10	116	119	115						
	11	122	123	122						

Table A-17 Dynamic Cone Penetrometer Test Data on Treated Soil B.

Cement content	No. of blows	Compactive efforts 14,300 lb-ft/ft³			Compactive Efforts 7,700 lb-ft/ft³			Compactive Efforts 2,200 lb-ft/ft³		
		Penetration in mm			Penetration in mm			Penetration in mm		
		1	2	3	1	2	3	1	2	3
	Initial	27	27	27	27	28	27	31	30	30
	1	42	40	42	43	43	42	48	47	48
	2	46	45	46	53	53	53	60	60	61
	3	51	49	51	64	64	64	73	72	73
	4	55	53	54	73	74	75	86	85	85
	5	61	59	60	82	84	84	100	101	100
	6	65	64	64	91	92	92	114	113	114
	7	70	70	70	100	102	101	125	123	124
10%	8	74	75	73	109	110	111			
	9	78	79	76	118	118	118			
	10	83	84	82	127	126	126			
	11	86	88	87						
	12	91	92	91						
	13	97	97	95						
	14	103	102	100						
	15	109	106	105						
	16	115	111	110						
	17	120	116	115						

Table A-18 Dynamic Cone Penetrometer Test Data on Treated Soil B.

Cement content	Compactive efforts 14,300 lb-ft/ft^3	Compactive Efforts 7,700 lb-ft/ft^3	Compactive Efforts 2,200 lb-ft/ft^3
	Penetration in mm /blow	Penetration in mm /blow	Penetration in mm /blow
2	21	31	50
4	16	28	37
6	12	23	28
8	9	12	22
10	6	10	14

Table A-19 DCP - CBR Relationship

Cement content %	Compactive efforts 14,300 lb-ft/ft^3		Compactive Efforts 7,700 lb-ft/ft^3		Compactive Efforts 2,200 lb-ft/ft^3	
	CBR (%)	DCP (mm/blow)	CBR (%)	DCP (mm/blow)	CBR (%)	DCP (mm/blow)
2	19	21	7	31	3	50
4	28	16	25	28	14	37
6	39	12	32	23	18	28
8	46	9	38	12	23	22
10	57	6	44	10	27	14