EFFECT OF COMPACTIONCONDITIONS AND STATE VARIABLES ON ENGINEERING PROPERTIES OF THE LIME STABILIZED SOIL

A Thesis

by

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ABSTRACT

To observe the effect of state variables on the strength of the lime stabilized soil, test samples were prepared by varying lime content, moisture content, compactive effort, compaction delay time, mixing method and soil type.

Three types of soil collected from three different areas of Bangladesh were selected for the research work. Two types of soil was CL and other type was ML, according to the Unified Soil Classification System. According to AASHTO Soil Classification System the three types of soils are A-6, A-7-6 and A-4 type. Slaked lime was used for stabilization. Unconfined compressive strength test and by CBR test was performed on the stabilized soil.

Strength increase of the lime stabilized soil is highly dependent upon the soil type. In general, fine grain soils are suitable for lime stabilization.

Addition of 3% lime produces significant increase in strength of the stabilized soil. Due to the increase of lime content from 3% to 5%, the increase in strength is not significant.

The maximum dry density of the soil is reduced due to the addition of lime. The maximum dry density decreases with the increase of lime content.

Optimum moisture content of the stabilized soil increases due to the addition of lime and the optimum moisture content continues to increase with the increase of lime percent. The maximum strength of the stabilized soil occurs at a moisture content higher than the optimum moisture content of the soil at which maximum dry density is achieved. When wet soil is difficult to compact, lime stabilization may be helpful.

The strength of the stabilized soil increases rapidly with the increase of compaction energy upto a certain level. After that the rate of increases of strength is gradual. The increase of strength is rapid upto about half of the Standard Proctor energy.

There is practically no change in strength due to delay in compaction of 6 hours after mixing. About 5% decrease in strength occurs due to the compaction delay time of 24 hours and about 12% decrease in strength occurs due to the compaction delay time of 48 hours.

Unconfined compressive strength of the lime stabilized soil increases with the increase of age of the stabilized soil. For clay soils, seven days strength is about 70% of 28 days strength.

CONTENTS

CHAPTER ONE

Introduction

CHAPTER TWO

Literature Review

CHAPTER THREE

Laboratory Investigation

CHAPTER FOUR

Results and Discussions

CHAPTER FIVE

Conclusions and Recommendations for the Future Research

List of Figures

List of Tables

NOTATION

ASTM	American Society of Testing Materials
AASHTO	American Association of State Highways and Transportation Officials
CBR	California Bearing Ratio
UCS	Unconfined Compressive Strength
OMC	Optimum Moisture Content
c	Cohesion
ø	Angle of internal friction
Mpa	Mega pascals
Kpa	Kilo pascals
EDTA	Ethylene Diamine Tetra Acetic Acid
W_p	Plastic limit
W_l	liquid limit

CHAPTER ONE

Introduction

1.1 General

Engineering properties of soil in many areas does not fulfil the requirements of construction. So improvement of its properties may be essential to meet the required soil condition for construction. Improvement of the engineering properties of the soil can be done by different ways. Soil improvement is one of the techniques among the procedures of improving its engineering properties. Cement, lime, bitumen or calcium chloride can be used as stabilizing material for soil. Improving the strength and other properties of the soil by the addition of lime to the soil as admixture is simply referred to as lime stabilization.

It is evident that earth structures, such as embankment, highways, airport runways, dams or reclamation appurtences require soils with sufficiently good engineering properties such as low plasticity, high bearing capacity, low settlements etc. In early days, engineers could avoid unsuitable site or unsuitable construction material source whenever the required condition did not fulfilled. Now a days, as engineers are concerned about the stability of the soil under construction, they are also worried about the economics of the stabilization method. In the developing countries, considering the conventional construction materials that are being adopted in the country today, there appears to be an ample scope for exercising further economy by the way of incorporating locally available materials and adopting the soil stabilization technique to the maximum extent possible (Khan, 1989).

Natural soil is a complex and variable material which exhibit differing behaviour under different conditions. Because of the universal availability of soil and the cost of obtaining such materials, it offers greater opportunities for skilful use with engineering technology.

Many procedures have been developed to improve the physical behaviour of soil in which a wide range of stabilizing agents, additives, and conditioners have been incorporated with soils. Undoubtedly, the most widely applied methods involve the use of inorganic cementative bonds between the particles in the soil system (Khan, 1989).

Two most common cementetive agents are normal portland cement and hydrated lime. Due to the application of each of these materials, the soil components and the stabilizing agents are both involved in the chemical changes responsible for the soil stabilization.

Faisal et al. (1992) pointed out that soil stabilization has been widely used for developing countries for the construction of various elements of pavements. The reason usually put forward is that the use of locally available materials will lead to lower cost. For proper lime stabilization, the principle of soil stabilization and understanding of local conditions is of paramount importance.

Locally available soil may differ in important aspect from soil tested in other places. Climatic conditions can affect the behaviour of stabilized soil materials as well as construction procedure. For example, the rate of curing may be more rapid at higher temperatures and rain may affect the compaction and strength properties of the stabilized soil (Faisal et al., 1992).

Utpal & Palit (1960) stated that lime stabilization was not extensively used since second world war. Many of the early lime stabilized roads were disappointing due to the lack of field control, over mixing, compaction and curing. Since then, the need for field and laboratory research has been evident .

Ingles and Metcalf (1972) noted that soil stabilization using lime alters the natural properties of unsuitable soil and thus provide sound geotechnical engineering requirements for design purpose. Lime, as an additive, has been recognized to bring several beneficial changes in the engineering properties of the fine grained soils. Treatment with lime is observed to decrease the soil plasticity and shrink-swell potential and improve the strength characteristics of soil.

Thompson (1966) stated that reaction between soil and lime involve interaction between soil silica (or alumina) and lime to form various types of cementing agents which are regarded as major sources of strength. Lime is very effective in stabilizing soft clay and is widely used for subgrade, subbase and base stabilization as a construction expedient on the wet sites. The necessary measures to accomplish soil stabilization are the addition of lime to the stabilizing soil and the application of optimum condition, various methods exists depending on the degree of mixing and compaction desired.

1.2 History of lime stabilization

Soil stabilization with lime seems to be the most ancient method of improving soil. There is evidence that the Alpine way, the access route to ancient Rome was constructed using this technique. Romans used it in their roads and buildings back to several hundred years B.C. They used naturally occurring reactive material such as pumice or volcanic ash which react much more readily with lime than ordinary soil and are known as pozzolans. The name pozzolana came after a volcanic ash obtained near pozzolana in Italy (Yang, 1988).

The favourable effects exerted by lime on soil have long been known. Silt and clay mixed with lime were used for construction of Shensi Pyramid in Tibet and some of Roman roads. At several points along the Great Wall in China, a calcium-hydrate reinforced earth wall was sandwiched between the two and one meter brick walls. Lime stabilized soil had also been used for other construction purposes in China at that time (Kezdi, 1979).

In India, many masonry dams were constructed using lime-surky, which was a mixture of lime, burnt clay and sand. Many of the old structures constructed with lime have been giving satisfactory service even to this days (Indian Road Congress, 1976).

As an experiment on the construction of a road using hydrated lime, the first attempt to improve soil using lime for roads was perhaps made in 1924 in the state of Missouri. In spite of some initial success in the stabilization of soil with lime, no serious attempt was made to extent the technique to a large scale construction till the second world war when gravel stabilized with lime was used in the construction of runways and taxiway in Texas (Utpal and Palit, 1960).

Laboratory as well as field experiment on lime stabilization was started by the Taxas Highway Dept. in 1948. Development of theory for the mechanism of lime stabilization started from 1950.

In 1954, the National Lime Association of America issued a booklet describing the method of determining the lime requirement for the stabilization of soil in the laboratory and the technique of constructing stabilized soil road (Uppal & Palit, 1960)

Extensive study on mechanism of lime stabilization was started from 1960. The major work in this regard was done by Eades and Grim (1960), Mateos (1964), Diamond and Kinter(1965), Thompson (1966), Ingles and Metcalf (1972), Kezdi (1979) Broms (1984) Locat et al. (1990) and others.

Though lime is being used as stabilizing material from long ago, the scientific knowledge regarding the use of lime-soil mix as engineering practice has newly developed one. More study regarding scientific knowledge can lead economically favourable results. Many researchers, even now a days are working with the theory of lime stabilization.

1.3 Necessity of soil stabilization in Bangladesh

Unconsolidated flood plain sediments occupy the greater part of Bangladesh (roughly 70% of land areas of Bangladesh, Bangladesh Transport Survey, 1974). These sediments are far from homogeneous in age and texture.

Most floodplain sediments have a high silt content. This is particularly so in case of Brahamaputra/Jamuna and Meghna sediments. Sandy sediments occur extensively in the substratum of soils in the North of Tista flood plain and West of Ganges floodplain. Clay deposits occur on surface over most of the Ganges flood plain (Geological Survey of Bangladesh, 1993). These soil are difficult to compact to the desirable limit without additive due to the present of high water content.

Most of the present national highways and urban and non-urban roads of different grades exist in the above mentioned silty and clayey alluvial deposited area and future national highways and road will have to be constructed in such areas of developing Bangladesh (Serajuddin, 1992). Most of the land remains under water table for 4 to 5 months in a year during the rainy season and ground water table remains near the surface for the rest of the years.

Because of periodic flooding, natural flood surface of the soil is generally soft and obviously create problem for the construction of earth dam, embankment and also heavy structures. In most of the cases due to low topography, earth filling is necessary. Most of the fill materials have inadequate shear strength to support load applied to them, sometimes it is necessary to improve their strength properties.

Lime is prepared in Bangladesh by heating lime stone (calcium carbonate) until they lose carbon dioxide and becomes calcium oxide. Lime stone mainly comes from Sylhet. Limestone of this region is not at the surface but at a shallow depth from the surface (source: Geological Survey of Bangladesh).

Limestone is available at Joypurhat but no action has yet taken to bring it out. Limestone, specially coral limestone is available at St. Martine Island. But its removal from the ground will lower the top surface of the land with respect to The Bay of Bengal. So these limestone is not used to prepare lime. Chemical analysis of the limestone indicate that impurities are mainly Sio_2, Al_2O_3, Feo, Mgo etc. (source: Geological Survey of Bangladesh). Lime stone is also imported from India and lime is prepared from that imported lime stone in Bangladesh.

1.4 Advantage of lime stabilization

Various method used to improve soil can be divided in to two categories.

1. Stabilization by compaction
2. Chemical stabilization

Soil stabilization by compaction is done by maximizing dry density at optimum moisture content. Such a compaction can bring about an increase in strength to a limited extent. Very often such a compaction technique alone will suffice for use in the subbase and base course.

Stabilization by compaction has limitations on the clay with high water content or on marsh land. If there is an overly plastic clay and in quantities more than the desired amount can give rise to a weak material and slippery surface when used in the surfacing. So these type of soil is not suitable for compaction (E-Rawi et al., 1968).

On the other hand, adding chemical stabilizer with the wet soil and using the conventional compaction method may improve the properties of the soil to an economical way. Cement and lime are mostly used as a chemical stabilizer. Lime has the following advantage over cement as a stabilizing material.

1. Lime is non-industrialized material

2. It is usually more effective in reducing the plasticity index of heavy clays, thus making it more workable even with low concentration

3. Cement stabilization is suitable for sandy soil

4. The need for compaction immediate after mixing is less critical for lime.

1.5 Objective of the study

The effective stabilization of lime depends upon a number of factors; such as: soil type, lime percent, moisture content, compaction effort, compaction delay time, mixing etc. To get the maximum benefit from lime stabilization, it is important to know how these factors influence the strength and other properties of the stabilized soil. Prime importance of this research is to study of the important factors that affect the lime stabilization.

The main objective of the research is to investigate the followings:

a) The effect of soil type on lime stabilization for some selected soil samples of Bangladesh

b) The effect of lime percent on the soil stabilization

c) The effect of moisture content on the lime stabilized soil

d) The effect of compaction energy on the lime stabilized soil

e) The effect of time on the strength of the lime stabilized soil

f) The effect of compaction delay time (time interval between mixing and compaction) and the effect of the stage mixing (i.e one stage and two stage mixing) on the lime stabilized soil.

Realizing the extreme variation in properties that exist among soils, research was limited to the laboratory investigation on three kinds of soil and using commercially available lime from the open market.

Unconfined compressive strength was performed on the test specimen to determine the strength of the untreated and the stabilized soil to observe the effect of moisture content, soil type, lime percent, compaction effort, age, compaction delay time and mixing on the lime stabilized soil. CBR test was also done on the specimen to observe the effect of soil type, lime percent and compaction effort.

CHAPTER TWO

Literature Review

2.1 General

The objective of soil stabilization is to increase strength, reduce deformability, reduce permeability, reduce erodibility, increase durability, and to provide volume stability of the soil. Lime, cement, bitumen, calcium chloride or others can be used as stabilizing material for soil stabilization, depending upon the required strength of the soil, existing soil condition, economy and considering the existing situation.

The use of lime to improve the engineering characteristics of soil have been recognised since the day of Roman empire. Yu Kuen, (1975) stated that Mccorssaland pointed out the extensive use of lime in building industry in 1925. Lime is extensively used in the civil construction. There is evidence that lime was also used for improvement of the soil at that time. At that time, lime was used without any theoretical background.

Laboratory as well as field experiment on lime stabilization was started by the Taxas Highway Dept. in 1948. Development of theory for the mechanism of lime stabilization started from 1950 and the extensive study on mechanism of lime stabilization was done in 1960. The major work in this regard was done by Eades and Grims (1960), Hilt and Davidson (1960), Mateos (1964), Diamond and Kinter(1965), Thompson (1966), Ingles and Metcalf (1972), Kezdi (1979), Mitchell (1961, 1981), Broms (1984) Locat et al. (1990) and others. Many researchers, even now a days are working with the theory of lime stabilization.

Mateos (1964) worked with various factors which affect soil stabilization using lime. As the time passed, researchers found out several factors that affect lime stabilization. Locat et al. (1990) worked extensively with the factors that affect the strength of lime stabilized soil. He developed mathematical relationship with the shear strength development and various factor that affect the strength of the lime stabilized soil.

Addition of lime alters various physical properties of the soil, such as plasticity, permeability, compressibility, swelling potential (volume change). Extensive work regarding physical properties of the soil was performed by Hilt & Davidson (1960), Kezdi (1979), Broms (1984) and others.

Application of lime on soil also affects stress-strain and deformation characteristics of soil. Research on the stress-strain and deformation characteristics of the stabilized soil was done by Thompson (1966), Brandl (1981) and others.

Soil stabilization with lime are used to improve the engineering properties of the soil. It is extensively used in the civil construction. Lime is used in building and lime stabilized soil is used in the field of highway, rail roads and airport construction to improve roadbeds and bearing layers. Lime stabilization is also used to improve soil below the foundation to prepare the foundation bed suitable for shallow foundation. H. Brandl (1981) stated that lime is also used to the construction of embankments, in sliding slopes by improving the soil shear strength and/ or by changing the failure geometry, in the backfill of bridge abutments and retaining walls.

2.2 Mechanism of lime stabilization

Eades and Grims (1960), Thompson (1966), Compendium (1987) and other researchers worked with the mechanism of lime stabilization. They observed that mechanism of lime stabilization can be classified into four phases, such as:

1. Cation exchange
2. Flocculation/ Agglomeration
3. Carbonation
4. Pozzolanic reaction

Cation Exchange

The type and the amount of different cations in a clay water electrolyte system have a major influence on the interactions of the water double layer that surrounds the clay particles. According to Thompson (1966), the most common cation found in soil are Calcium (Ca^{++}), Magnesium (Mg^{++}), Aluminum (Al^{+3}), and Potassium (k^{+}). The general order of replaceability of the common cations associated with soil is given by the lyotropic series as: $Na^{+} < K^{+} < Mg^{++} < Ca^{++}$. Any cation will tend to replace another cations to the left of it, and monovalent cations are usually replaceable by multivalent cations.

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 sheet with Ca^{++}. Following the saturation of inter layer positions with Ca^{++}, the whole clay minerals deteriorate without the formation of substantial new crystalline phases.

Many natural soils are largely cation saturated. Addition of lime to the soil supplies to an excess of Ca^{++} and the cation exchange will occur, with Ca^{++} replacing dissimilar cations from the exchange complex of the soil. In some cases the exchange complex is particularly Ca^{++} saturated before the lime addition but cation exchange may still take place because the cation exchange capacity will increase as the p^H of the soil increase.

Flocculation/ Agglomeration

Flocculation of the soil particles occurs due to the mixing of soil with lime in presence of water. After cation exchange of soil and lime take place, agglomeration of the flocculated particles occurs. Diamond and Kinter (1965) suggested that the rapid formation of calcium aluminate hydrate cementing materials are significant in the development of flocculation agglomeration tendencies in the soil lime mixture.

Thompson (1966) indicated that flocculation and agglomeration are primarily responsible for the change in plasticity, shrinkage, and workability characteristics of soil lime mixture.

Mitchell et al. (1961) demonstrated that the introduction of lime in moist soil increase the p^H of the soil and release Ca^{++} ions into the pore water. This phenomenon reduces the forces of repulsion and encourages compression of the diffused double layers.

Mitchell (1981) stated that increased thickness of double layer creates less tendency of repulsion for particles in double layer resulting in a flocculated structure. These reactions results to an apparent change in texture-the clay particles "clumping" into larger sized "Aggregate".

Lime carbonation

Lime reacts with carbon dioxide in atmosphere or in the soil to form relatively weak cementing agents such as calcium carbonate or magnesium carbonate. Eades et al. (1962) demonstrated that although carbonation does take place, the strength gain is said to be occur by virtue of cementation of soil grains with calcium carbonate is negligible.

Diamond and Kinter (1965) indicated that the carbonation is probably a deteriorate rather than a helpful phenomenon in soil stabilization. The strength of calcium carbonates which are formed by this process can generally be discounted.

Yu Kuen (1975) stated that carbonation is normally confined to the surface exposed to the air and involve the conversion of lime to the Calcium carbonate by carbon dioxide absorbed from the air.

Soil lime pozzolanic reaction

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.

Lime, water, soil silica and alumina react to form various cementetious compounds. Possible source of silica and alumina in a typical soil include clay minerals, are quartz, feldspars, mica and similar silicate or alumino-silicate minerals either in crystalline or amphorous in nature. The minerals that react with lime to produce a cementing material are known as pozzolans (Khan, 1989).

Addition of lime to the soil in presence of water causes a instantaneous rise in p^H of the moulding water due to the dissociation of the $Ca(oH)_2$ in the water. Eades and Grims (1960) showed that high p^H causes silica and alumina to be dissolved out of the structure of the clay minerals and it combines with the calcium to form calcium silicate and calcium aluminates. The calcium ions combine with reactive hydrous silica and alumina and form gradually hardening cementetious material. This reaction will continue as long as $Ca(OH)_2$ exist in the soil and there is available silica. These mechanism may be referred as "Through-Solution".

Herzog & 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.

The diffuse cementation theory proposed by Stocker (1975) does not postulate lime adsorption of clay surface but that lime reacts directly with clay crystal edge, generating accumulations of cementetious material.

Asserson et al. (1974) worked with red tropical soils suggested that after the initial 7 days of curing, strength increases are the result of hydration and increase in crystallizing of reaction products rather than from the continued formation of additional pozzolanic compounds.

Ramle, (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.

2.3 Effects of lime on the physical properties of soil

Lime react with soil silica or alumina in presence of water. New materials is produced by the chemical reaction of soil with lime and thus causes the change of the physical properties of soil. Some of the important properties of the soil which are changed due to the stabilization of soil with lime are plasticity, permeability, strength, compressibility, stress-strain character, volume change, shear strength parameter etc.

Plasticity

Plasticity is the property of the soil which allows to be deformed rapidly without rupture, without elastic rebound and without volume change. Due to the addition of lime to the soil, plasticity of the soil will reduce.

Hilt & Davidson (1960), Pietsch & Davidson (1962) reported that the plastic limit of soil generally increases with the addition of small amount of lime until a certain critical lime content. The amount of lime at the point of critical lime content is referred to as the "Lime fixation point".

Mateos (1964) observed that for the lime content beyond the optimum lime content, no further increase in plastic limit can be observed.

Rodriguez et al. (1988) stated that lime generally reduce the plasticity index of high plastic soil. but has little influence on the plasticity index of the low plastic soils.

Mateos (1964), Rodriguez et al. (1988) stated that the net effect of adding lime on soil is always a marked decrease in plasticity index of soil. The increase in plasticity index may be due to the increase in plastic limit or may be due to the decrease in liquid limit.

Permeability

Permeability is the property of the soil which permits the passage of water through its interconnecting void. Permeability of the soil increase due to the addition of lime to it.

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, they observed a marked increase in permeability but for silty clay soil, erratic or no change of permeability was observed.

Broms & Boman (1977), 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.

Compressibility

Compressibility is the property of the soil of resisting deformation due to the applied load upon it. Broms & Boman (1977) told that the lime stabilized soil is normally firm to hard and the texture is grainy. It has a low compressibility compared with unstabilized soil.

Broms (1984) observed that the deformation and strength properties of the lime stabilized soil have been found to be similar as those of a heavily over consolidated stiff fissured clay in the desiccated dry zone.

Strength

Shear strength of the soil increases due to the addition of lime to it. Assarson et al. (1974) stated that the increase of strength is lowest immediately after mixing of lime with soil but after 28 days the increase in strength can be reached up to 30 times to the initial strength. They also found that the increase of shear strength due to stabilization is dependent upon lime content and other factors.

Yu Kuen (1975) stated that the part of the initial strength increase in stabilized soil is due to the formation of crystalline calcium hydroxide (or gel phase), which posses cementing properties due to the super saturation of the soil solution. Kezdi (1979) noted that the gel phase can be clearly discerned through microscope though its chemical and crystalline composition could not be determined experimentally. Broms (1984) also reported that another part of increase in shear strength is caused by flocculation of clay because of cation exchange and by a reduction of water content, owing to the flocculation of clay particles into larger size aggregates. A significant decrease in the plasticity index of the clay is also observed at this stage.

Among the strength parameter of the soil (c & ø), the increase in cohesion may be due to formation of cementetious products resulting from pozzolanic action and the increase of the angle of internal friction may be the effect of aggregation which results in greater interlocking and rough surface.

Broms & Bomans (1977) noted that the ultimate strength of lime stabilized soils is not uniform, even when the mixing of lime with clay has been done very carefully.

Broms (1984) pointed that the physical and chemical reactions brought about by lime stabilized soil results to a corresponding increase in shear strength for the treated soil mass. The shear strength of clay stabilized soil with lime will normally be higher than that of the undisturbed clay for about one or two hours after mixing. Thereafter, the shear strength of stabilized soil gradually increases with time through pozzolanic reactions, which take place for larger period.

Broms (1984) pointed that the carbonation also results when lime reacts with carbon dioxide present in the soil and air. However, the strength of calcium carbonate thus formed is low. The calcium carbonate has been to retard pozzolanic reaction

Volume change

Herrin and Mitchell (1961) indicated that lime has a great influence on reducing volume change in soils that readily change its volume when added with water.

Thompson (1966) stated that the addition of lime restricts the volume change on wetting. Swelling potential of the soil is also reduced due to the addition of lime. The change in swelling potential can be related to an increase in shrinkage limit which occur when the soil mixed with lime.

Stress-strain characteristics of the soil and the shear strength parameter of the soil is also changed due to the addition of lime to the soil.

2.4 Factors affecting lime stabilization

For the effective stabilization of soil with lime, it is essential to be aware about the factors that effect the properties of the lime stabilized soil.

The effectiveness of lime stabilization depends upon a number of factors. According to Eades and Grims (1960), Thomson (1966), Mitchel (1981) and other researchers, the strength of the lime stabilized soil depends upon factors. Such as:

Lime

i) lime content
ii) Lime type

Soil

i) Soil type
ii) Organic matter in the soil
iii) Clay content of the soil

Compaction conditions

i) Compaction method
ii) Compactive effort
iii) Moisture content during compaction
iv) Compaction delay time
v) Mixing of lime and soil before compaction

Aging

i) Age effect on the stabilized soil
II) Curing temperature
III) Relative humidity during curing
iv) Warped vs unwarped

2.4.1 Lime

Lime content

Lime react with soil silica and thus improve the engineering properties of the soil. So lime percent is an important factor for strength gain of the lime stabilized soil.

Nassara (1970) presented a table showing the usual content of hydrated lime for different types of soil and are shown in table 2.1. From the Table, it can be seen that the usual content of lime for silty soil is 3%- 6% but for the plastic clay this figure is 3% to 9%

Ingles et al. (1972) presented a similar table showing the usual contents of hydrated lime for different types of soil to stabilize soil and is shown in Table 2.2. From the table, it can be observed that the higher limit of lime content for stabilization is 8% for plastic and highly plastic clay. The table also shows that the usual content of lime for silty clay is 2%-4%.

TABLE 2.1 USUAL CONTENTS OF HYDRATED LIME IN DIFFERENT SOIL (PERCENTAGE BY WEIGHT OF DRY SOIL TO LIME) AFTER NASSARA (1970).

soil type	stabilization (lime percent)
crushed rock	not recommended
well-graded clayey gravels	2
pure sands	not recommended
silty sandy	not recommended
clayey sand	2-4
clay silt	2-4
silty clay	2-6
plastic clay	3-9
highly plastic clay	3-9
organic soil	not recommended

TABLE 2.2 USUAL CONTENTS OF HYDRATED LIME IN DIFFERENT SOILS (PERCENTAGE BY WEIGHT OF DRY SOIL) BY INGLES ET AL. (1972).

soil type	stabilization (lime percent)
crushed rock	not recommended
well-graded clayey gravels	3
sands	not recommended
sandy clay	5
silty clay	2-4
plastic clay	3-8
highly plastic clay	3-8
organic soils	not recommended

Variation of unconfined compressive strength of the lime stabilized soil with the variation of lime percent for different curing period as presented by Kezdi (1979) is shown in fig. 2.1. From the figure, it is observed that the unconfined compressive strength of the lime stabilized soil initially increase and reduces after certain amount of lime content. The figure also shows that no appreciable change of unconfined compressive strength after 8% lime content. From the figure, it can be also observed that the increase of unconfined compressive strength due to the variation of lime content is also dependent on the age.

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 Hansmann (1990) is shown in fig. 2.2. From the figure, it can be observed that the unconfined compressive strength of the lime stabilized soil increase with the increase of lime content. But it can also be observed that the unconfined compressive strength of the lime stabilized soil decrease due to the increase of lime percent above 8% lime.

Optimum lime content

From the present study, it can be concluded that lime percent varies from soil to soil to achieve the maximum strength of the lime stabilized soil. 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 & 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.

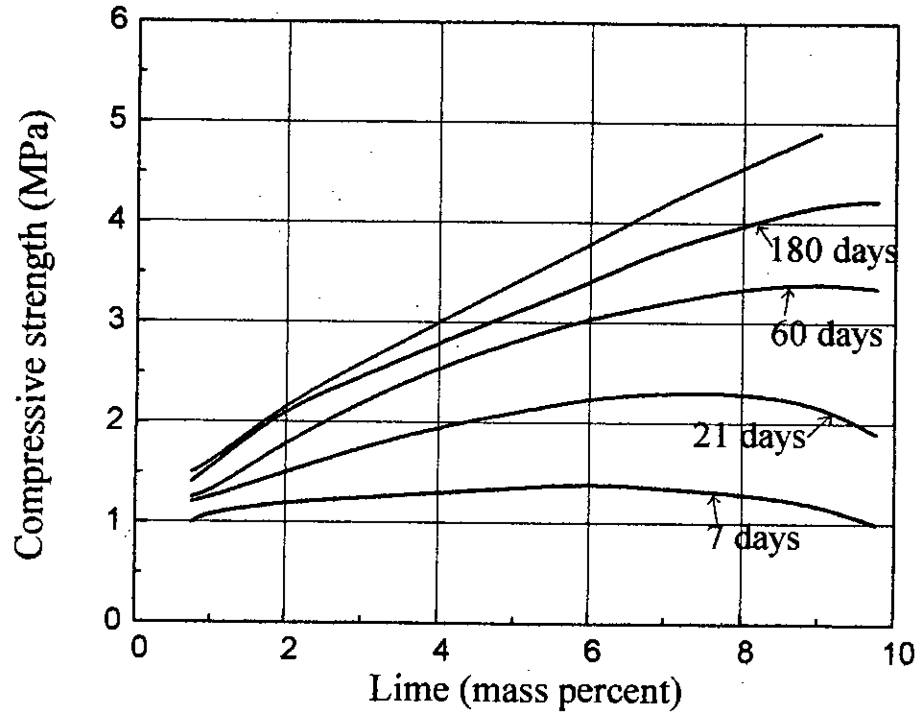


FIG 2.1 VARIATION OF UNCONFINED COMPRESSIVE STRENGTH WITH LIME CONTENT (CL SOIL) AFTER KEZDI (1979)

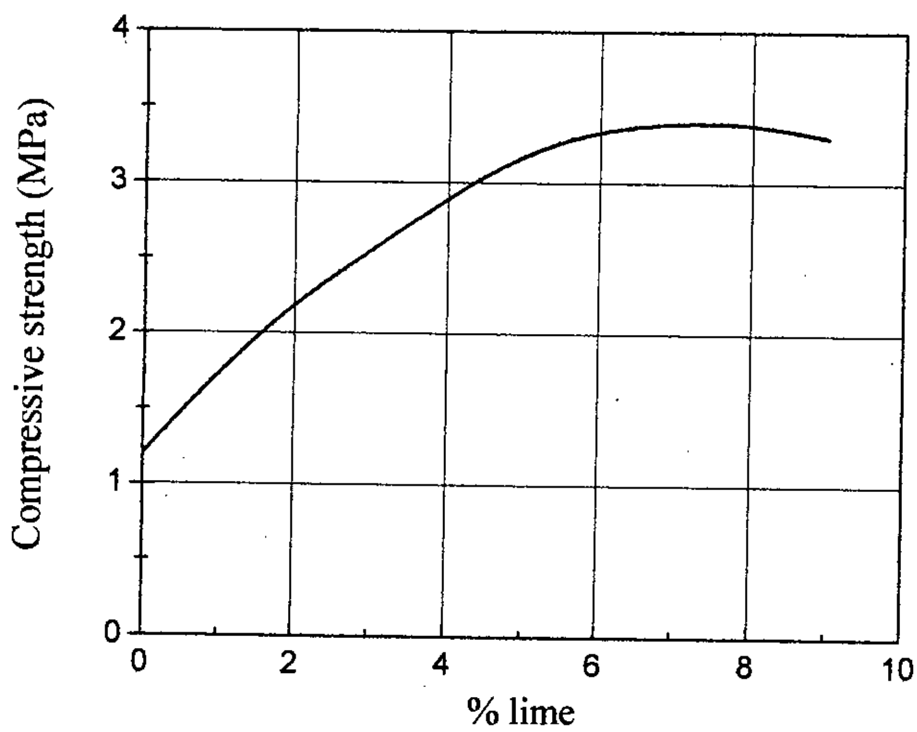


FIG 2.2 VARIATION OF UNCONFINED COMPRESSIVE STRENGTH WITH LIME CONTENT (ML SOIL) AFTER HANSMANN (1990)

Based on intensive investigation at the Iowa State University, Diamond & 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 & 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. They presented a correlation with optimum moisture content and clay content which show that of optimum lime content is proportional to the type and amount of clay present and is independent of the adsorbed cation present in the clays. The relationship is given as: optimum lime content = % of clay/35 +1.25

Eades & Grim (1960) suggested that the amount of lime consumed by soil after one hour period, a quick method of determining the percentage of lime required for stabilization. The percentage of lime consumed by one hour can be determined by the chemical test of the stabilized soil after one hour of mixing soil with lime (ref. "Lime content of the uncured soil-lime mixture", D 3155-83, ASTM, 1989).

Mcdowell (1966) pointed out that short term or quick tests probably will not identify optimum lime content, although are essential in checking against the use of non-reactive soils for treatment with lime. Although long term strength tests would do a better job of identifying optimum lime contents, this may be impractical from the standpoint of time and may even suggest the use of insufficient amount of lime due to the ideal conditions under which they run.

Lime type

Lime type is one of the factors that affect the strength of the lime stabilized soil. Mateos (1964) shown, lime can be divided chemically into two categories:

1) quick lime
2) hydrated lime

Quicklime are of two types,

1) calcitic (CaO) and
2) dolomitic (CaO+MgO)

Hydrated lime are of three types,

1) calcitic $Ca(OH)_2$
2) dolometic monohydrate $[Ca(OH)_2+MgO]$ and
3) dolomitic dihydrate $[Ca(OH)_2+Mg(OH)_2]$.

Chemical analysis of lime shows that at calcitic hydrated lime has 75.67% quicklime and 24.33% water. Dolomitic monohydrate lime has 15.79% water and 84.21% Dolomitic quicklime. Dolomitic hydrated lime has 27.27% water and 72.73% dolomitic quicklime.

Mateos (1964) demonstrated that quick lime lower the plasticity more than the equivalent amount of corresponding hydrated lime. These are more effective in improving the shrinkage properties (reducing the plasticity index) of soil than the corresponding hydrated lime. Among the hydrated limes, calcitic lime is slightly better in improving the strength of the stabilized soil at low amount of addition.

Wang et al. (1962), Mateos (1964), stated that for some kaolinite soil both type of lime gave approximately the same strength. They told that at higher temperature calcitic lime was found to be more effective in increasing strength.

Wang et al. (1962), Mateos (1964), investigated the effect of lime manufacturing and properties of various commercial lime on the unconfined compressive strength of soil lime mixtures. They found that strength of the test specimens containing calcitic hydrated lime were lower and less variable than strength of specimens containing dolomitic monohydrated lime. They observed that small amount of calcitic quicklime were found to be more effective in increasing strength than calcitic hydrated lime but at higher percent of lime no conclusion was reached among these two types of lime.

Holm et al. (1983) have found that quicklime and calcitic hydrated lime is superior in stiffening soft, wet soil, on the other hand dolomitic lime has been found to produce higher strength in plastic soil-lime mixture than calcitic lime at normal curing temperature.

Ingles (1972) presented a table showing the maximum and minimum amount of different constituents on quick lime and hydrated lime and are shown in Table 2.3. The table shows that the requirement for the calcium magnesium oxide and carbon dioxide in lime for stabilization with soil differs for the two types of lime. The reason behind the difference in constituents is due to the presence of water in the hydrated lime.

According to Metcalf (1977) quicklime will corrode equipment and will cause burn to unprotected skin, quick lime can be used only where these problem is outweighed by the advantage. Gypsum or pozzolanic materials such as, pulverized blast furnace slag, fly ash or expended shale can be used in combination with lime.

Kezdi (1979) pointed that quicklime is more effective to create a favourable condition for lime stabilization and thus increase strength than hydrated lime but generally needs care in handling for soils with high moisture.

Broms (1984) stated that unslaked lime or quick lime are also more effective since water will be absorbed from the soil and more importantly the hydration will cause an increase in temperature which is favourable to strength gain.

TABLE 2.3 REQUIREMENTS THAT MUST BE MET BY LIME STONES ON NATURAL CALCIUM CARBONATES IN ORDER TO PROVIDE STABILIZING LIME BY INGLES ET AL. (1972).

Property	quicklime (CaO)	hydrated lime $Ca(OH)_2$)
calcium magnesium oxides	not less than 92%	not less than 95%
carbon dioxide in the oven out of the oven	 not more than 3% not more than 5%	 not more than 5% not more than 7%
fineness	–	not more than 12% retained on No. 180 sieve

TABLE 2.4 PROPERTIES OF THE THEORETICALLY PURE LIME AFTER GHOS (1987)

chemical name	quicklime		hydrated lime	
	calcia or calcium oxide	magnesia or magnesium oxide	calcium hydroxide	magnisium hydroxide
chemical formula	CaO	MgO	$Ca(OH)_2$	$Mg(OH)_2$
crystalline formula	cubic	cubic	hexagonal	hexagonal
melting point	2570°c	2800°c	-	-
decomposition point	-	-	580°c	745°c
boiling point	2850°c	3600°c	-	-
molecular weight	36.09	40.32	74.1	58.34
specific gravity	3.40	3.65	2.34	2.40

Ghos et al., (1987) experimentally found different properties of the pure lime for stabilization and are shown in Table 2.4

2.4.2 Soil

Type of soil

Strength of soil is improved by the addition of cement, lime or bitumen as admixture to the soil. Selection of admixture depends upon the soil type. For a selected soil, one type of admixture will be suitable but for the other types of soil another admixtures may be better.

Thompson (1966) 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.

Thompson (1966) has divided the soils in to two types, on the basis of their reactivity:

1) Non-reactive soil
2) Reactive soil

Thompson (1966) stated that after treatment, a non reactive soil is not a cemented system, although many properties including strength and durability are improved. CBR test are usually performed on these type of soil.

Thompson (1966) also stated that stabilization of reactive soil with lime always produce a cemented system and therefore CBR test and unconfined compressive strength test can be done to evaluate the strength of these types of soil.

Nassra (1970) stated that highly plastic soils are more effective to gain in strength. He 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 et al. (1972) studied the effect of the unconfined compressive strength on different types of soil stabilized using lime are presented in fig. 2.3. The figure shows 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.

Mitchell (1981) stated that lime is commonly used to stabilize cohesive soil while stabilization of sandy soil usually done with portland cement.

Broms (1984) reported that the increase in strength of the soil stabilized with lime is in general maximum for silty clays with a low plasticity index.

Ahmed (1984) pointed that due to the addition of lime to a fine grained soil, smaller clay particles aggregates to a bigger ones. This important change makes the grains coarser. Clay particles aggregates markedly because of the negative charge and the base exchange, flocculation and pozzolanic reaction. For the same reason, silt and sandy soils do not aggregate to that extent. Thus, for successful stabilization of soil, a certain minimum clay fraction in soil is always looked for.

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 type of soil is such that the stabilized soil becomes stronger than those that would be obtained with cement. He also added 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.

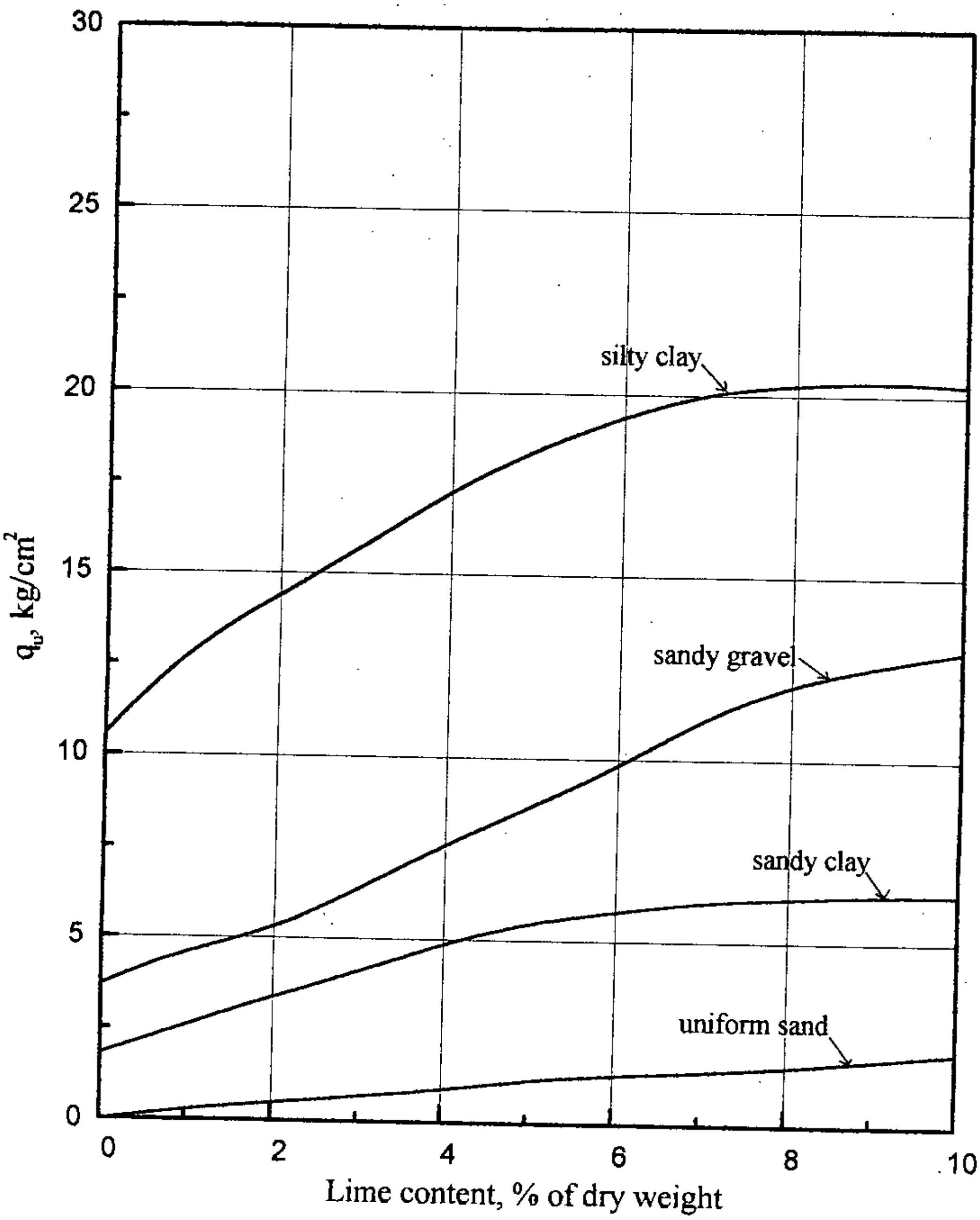


FIG 2.3 EFFECT OF THE LIME CONTENT ON THE UNCONFINED COMPRESSIVE STRENGTH (7 DAYS) OF DIFFERENT TYPES OF SOIL AFTER INGLES ET. AL (1972)

Locat et al. (1990) studied with four types of soil of Canada and observed that the unconfined compressive strength of the silty clay soil is higher than the other types of soil. Fig. 2.4 shows the variation of unconfined compressive strength with lime content for four types of soil. It can be observed from the figure that the maximum strength is gained by the soil with higher clay content.

Serajuddin (1991) stabilized three types of soil of the South-West region of Bangladesh. He used silt and clay types of soil for investigation. The results of his investigation is shown in fig. 2.5. It can be observed from the figure that silty soil has much lower strength than the clay types of soil. The reason for high strength of the clay type of soil is due to the presence of large amount of 2 micron particles in clay soil.

Organic matter present in the soil

Organic matter present in the soil act in reducing the reactivity of lime with soil. So the presence of organic matter reduce the strength of the lime stabilized soil.

Nassra (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 et al. (1972) studied the effect of the percent of organic matter on the unconfined compressive strength of the lime stabilized soil. Their findings are presented in fig 2.6. From the figure, it can be observed 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, the strength continue to decrease.

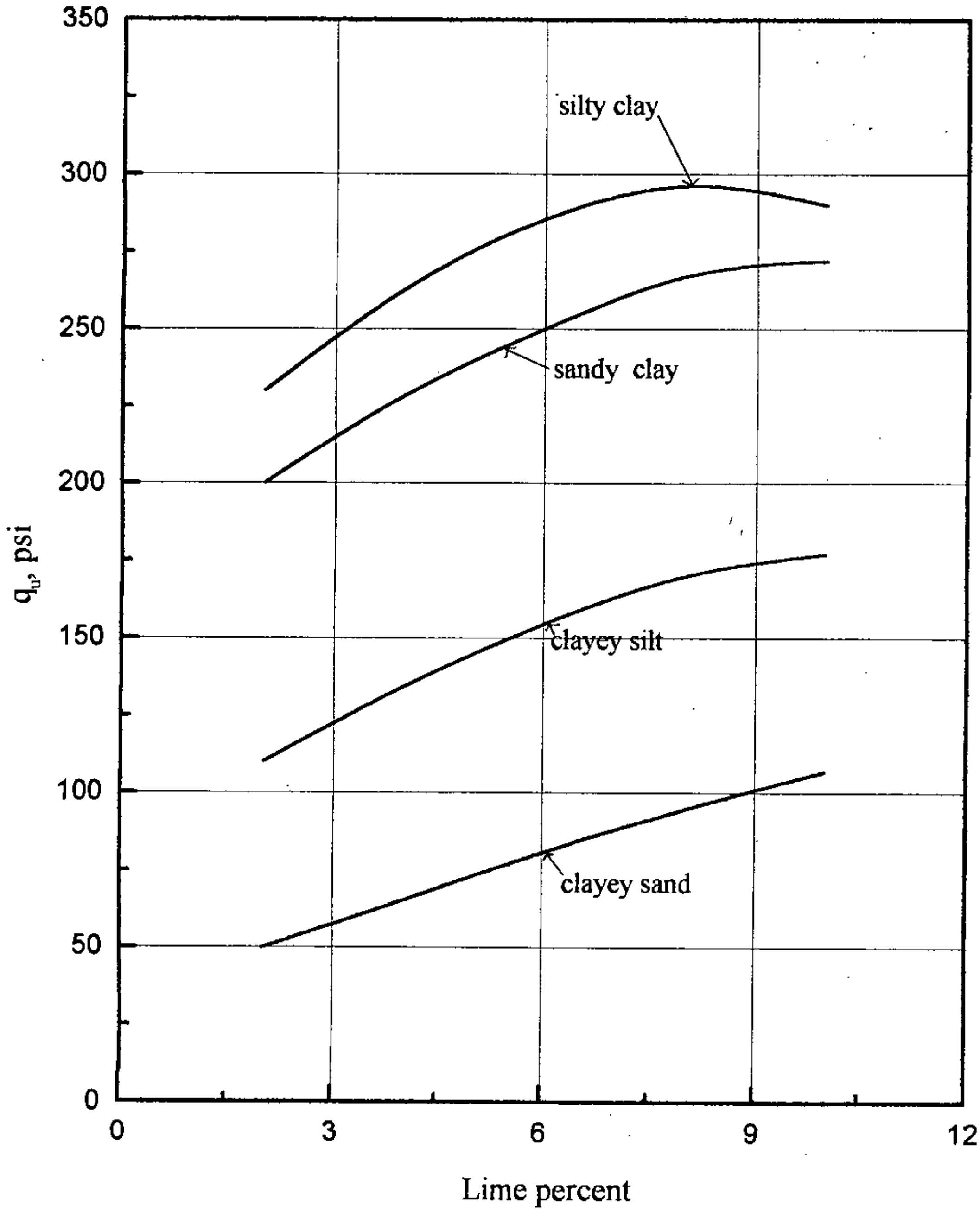


FIG 2.4 VARIATION OF UNCONFINED COMPRESSIVE STRENGTH WITH LIME FOR DIFFERENT TYPES OF SOIL AFTER LOCAT ET. AL (1990)

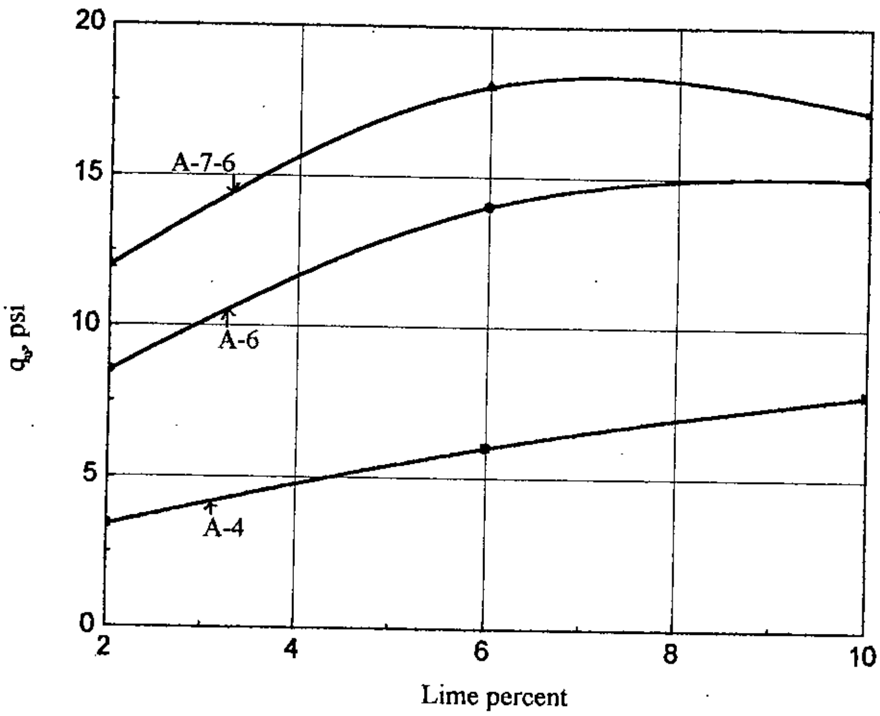


FIG 2.5 VARIATION OF UNCONFINED COMPRESSIVE STRENGTH WITH LIME FOR VARIOUS TYPES OF SOIL AFTER SERAJUDDIN (1992)

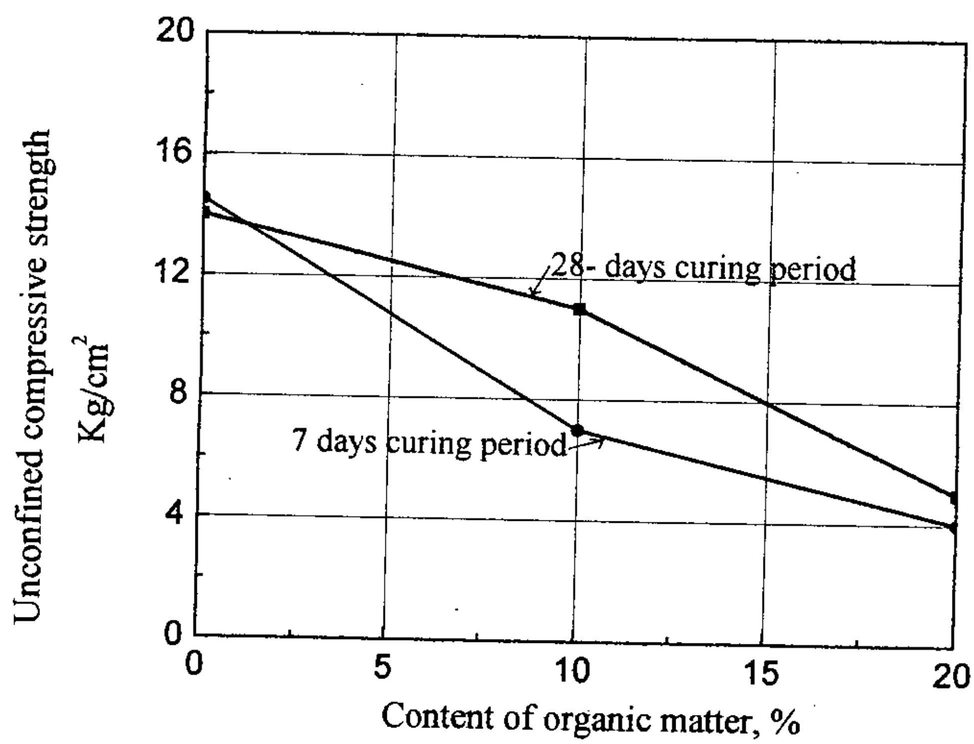


FIG 2.6 EFFECT OF ORGANIC MATTER ON THE UNCONFINED COMPRESSIVE STRENGTH AFTER ARMAN ET. AL (1972)

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 reason is that organic matter has high base exchange capacity. When lime is added to organic soils some of the Ca^{++} ions are used to satisfy the exchange capacity of organic matter, thus depriving the clay minerals of calcium ions for pozzalanic reactions. Even a small amount of organic content can have a large effect on the strength.

Rodriguez et al. (1988) noted that lime has little effect on highly organic soils or soils without clay.

Clay content of the soil

Clay minerals present in the soil plays an important role for the soil lime reaction in the lime stabilized soil. Eades & Grim (1960) pointed that the quantity of lime required for effective treatment is dependents on the type of clay minerals present in the soil. Clay minerals fall into three groups, namely, kaolinite minerals, the montimorillonite groups and the illites.

Hilt & Davidson (1960) found from unconfined compressive strength tests, that kaolinic and montmorillonitic clayey are effectively stabilized with lime, whereas illitic (and chlorotic) are not. Both montmorillonite and kaolinite were found to be effective pozzolanic agents as compared to clays containing illite, chlorite or vermiculite.

Dewet (1967) examined the clay lime reaction with DTA and X-ray diffraction. He reported that the reaction products of montmorillinite and lime were mainly calcium silicate hydrates and of kaolinite lime mixture were either calcium silicate hydrates or calcium silicate, depending on the clay lime ratio.

Eades & Grim (1960) observed that although kaolinite, illite, montmorillonites and other mixed layered clays, react with lime to give greater strength, the quantity of lime needed to treat certain amount of clay is dependent on the type of mineral present. For kaolinite clay the increase in strength began with the addition of the first increment of lime, since the strength begins to increase, some of the calcium attacks on the edge of the kaolinite particle. For illite and montmorillonite strength developed only after the clay was saturated with Ca^{++} ions and minerals began to be destroyed.

Lee et al. (1982) has found that lime has a greater effect in kaolinic soil than the montmorillonite soil. This may be due to the observation that the reaction of lime with minerals began by a replacement of existing cations between the silicate sheets with cation ions. Following the saturation of the interlayer positions with Ca^{++}, the whole clay mineral structure deteriorates and thus low strength develops.

2.4.3 Compaction conditions

Compaction is the process by which soil particles are rearranged and packed together in a close state of contact by mechanical means in order to decrease the porosity (or void ratio) of the soil and thus increase the dry density. As the water content of the soil increases, the compacted density goes on increasing till a maximum dry density is achieved after which further addition of water decreases the density. The moisture content at which the maximum dry density is achieved are termed as optimum moisture content. Typical moisture density relationship is shown in fig 2.7 by Punmia (1990).

Similar phenomenon of the increase of dry density is applicable for the lime stabilized soil. Fig. 2.8 shows the typical moisture density relationship for both the stabilized and the untreated soil as found by Compendium (1987). The figure also indicate that when compacted with a given effort, soil-lime mixtures have a lower maximum dry density than the original untreated soil and higher optimum moisture content than the untreated soil.

Hansmann (1990) studied moisture density relationship for both the untreated and stabilized soil. The relationship with moisture content and dry density of both the untreated and lime stabilized soil as found by Terrel et al. (1985) as shown in Fig. 2.9.

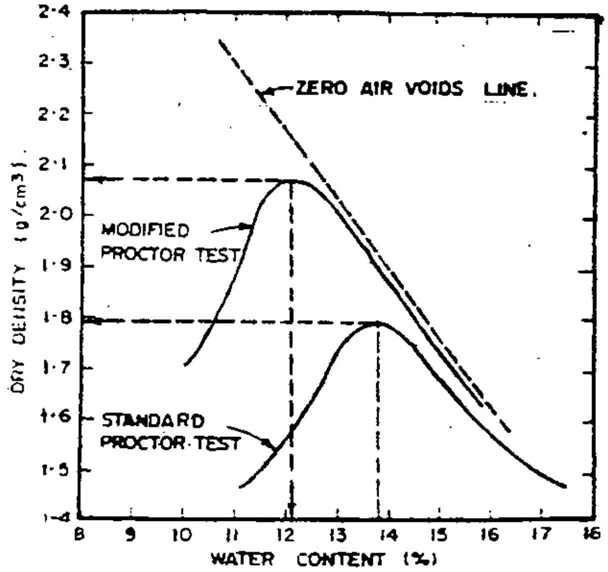


FIG. 2.7 TYPICAL MOISTURE- DENSITY RELATIONSHIP OF SOIL BY PUNMIA (1990)

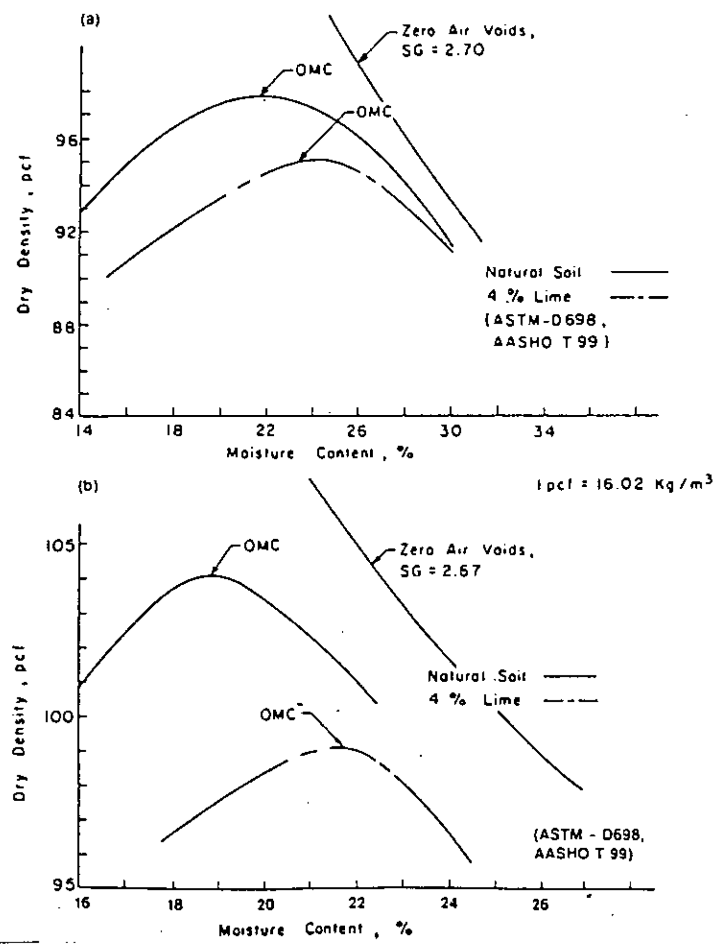


FIG. 2.8 TYPICAL MOISTURE- DENSITY RELATIONSHIP ON TWO TYPES OF SOIL AFTER COMPENDIUM (1987)

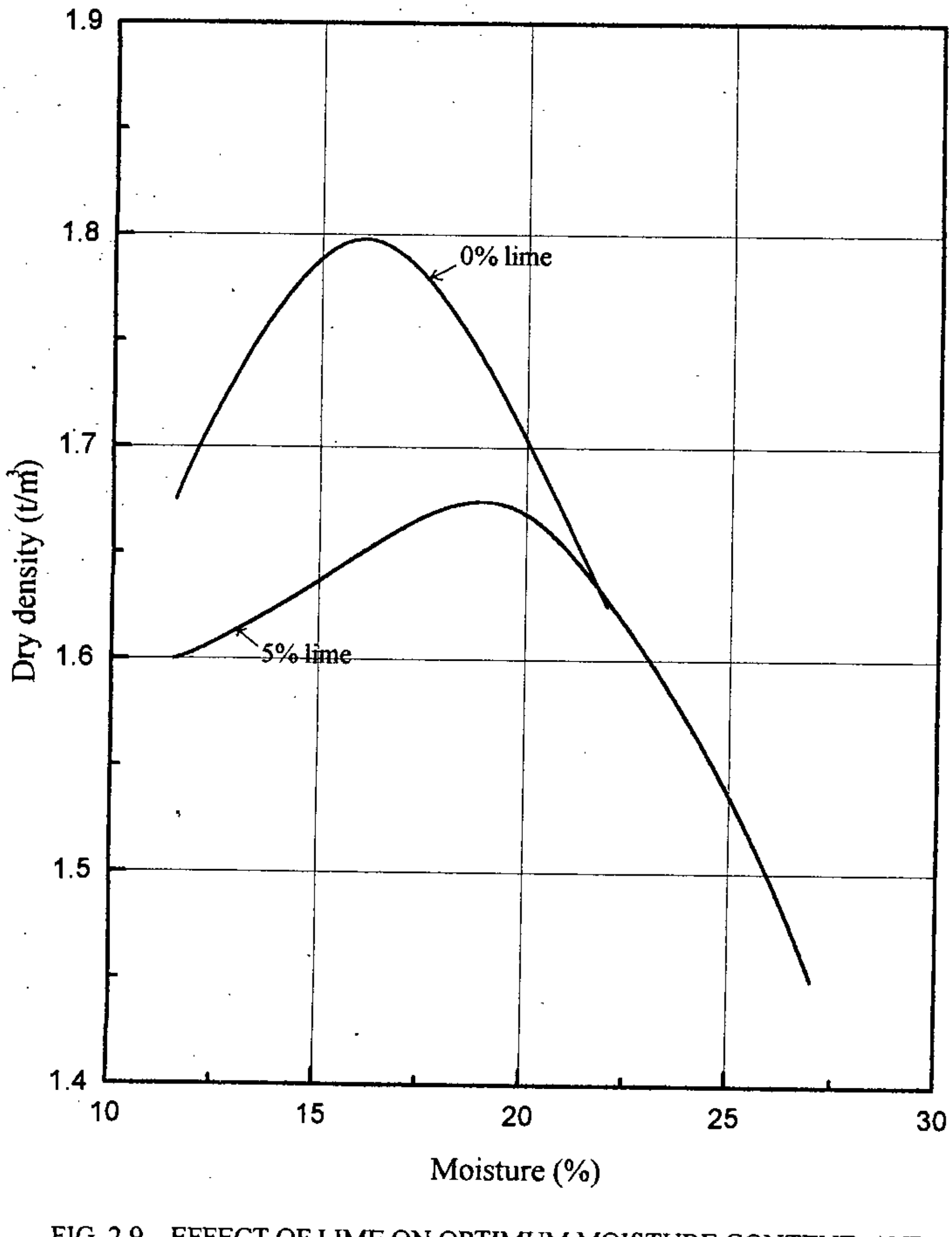


FIG. 2.9 EFFECT OF LIME ON OPTIMUM MOISTURE CONTENT AND DRY DENSITY OF SOIL AFTER TERREL ET. AL (1985)

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.

Hansmann (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. He 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.

Compactive effort

Compaction on the lime stabilized soil as well as untreated soil can be done by different methods in the field and in the laboratory. Three recognized method of compaction in the laboratory are Standard Proctor test method, which is normally used, Modified Proctor test method which is used for the higher energy in the compaction and Harvard miniature compaction in which compaction is applied by kneading action.

The amount of compaction energy greatly effect the maximum dry density and the optimum moisture content of the soil. The effect of increasing the compactive effort results in an increase in the maximum dry density and decrease in optimum moisture content as found by Singh and Punmia (1965) is shown in fig. 2.10.

In laboratory, compactive effort is applied by dropping a standard weight from a standard height on the standard compaction mould. The standard weight are dropped in different layer in 25 blows per layer. Compaction effort can be varied in a specified method of compaction by adjusting the different variables that affect the total compactive effort.

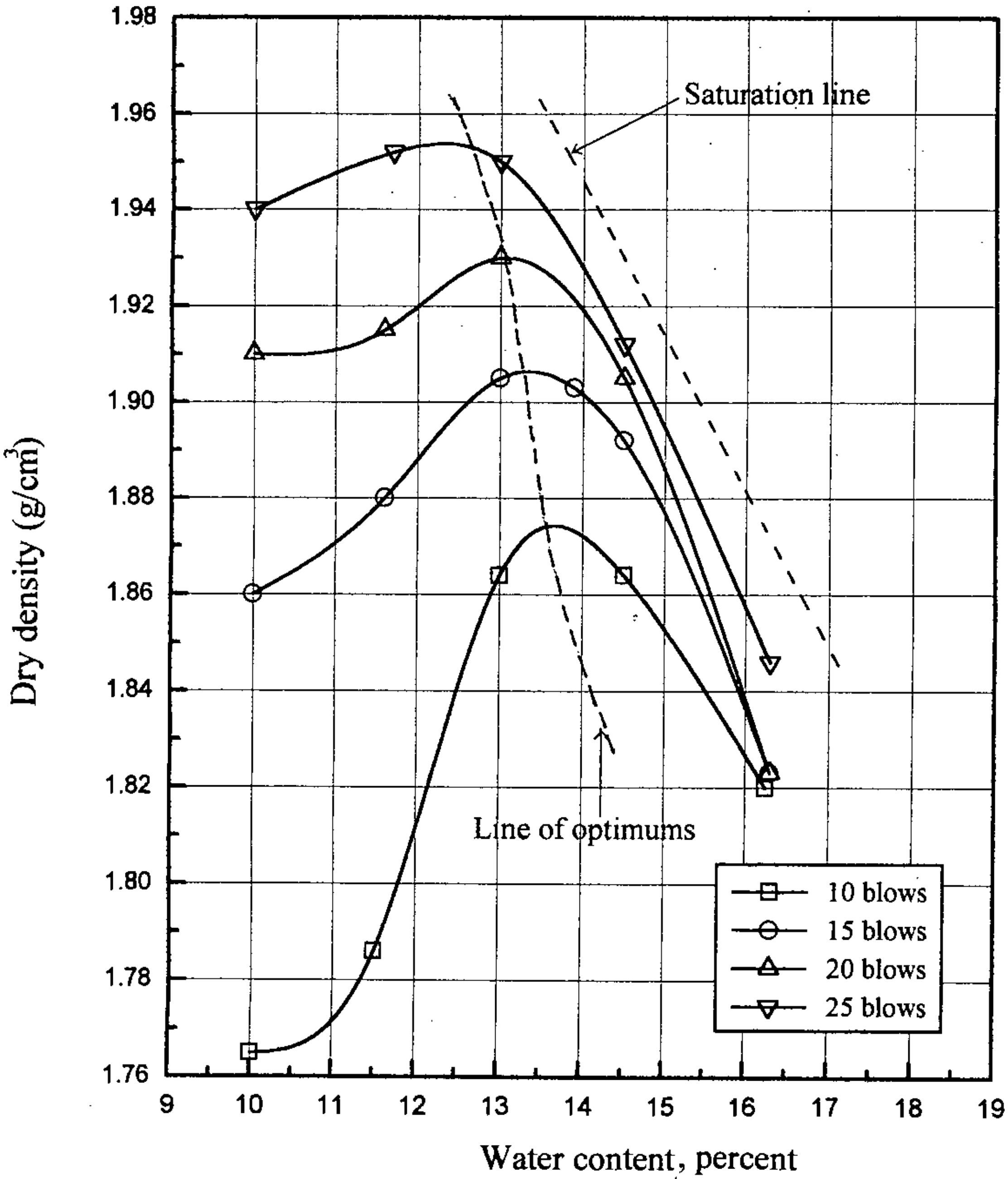


FIG. 2.10 EFFECT OF COMPACTIVE EFFORT ON COMPACTION OF NATURAL SOIL AFTER SINGH & PUNMIA (1990)

Dunlop (1977) observed the effect of compactive effort on the lime stabilized soil by varying the no. of blows per layer, dropping height and falling weight in the Standard Proctor test method.

Serajuddin (1991) observed the effect of reduced compactive effort by reducing the no. of blows per layer in the Standard Proctor test method. They observed lower strength of the stabilised soil at lower compactive effort.

Herrin and Mitchel (1961) pointed that a minimum amount of energy is essential for properly stabilize soil with lime. Without a certain minimum amount of energy, a very low strength of the lime stabilized soil is observed. They also mentioned that the required compactive effort depends upon the soil type. For the fine grained soil required compactive effort is lower than the other types of soil. They stated that about 85% compaction can be achieved at 75% of the total compactive effort in the Standard Proctor test method.

Croft (1964) investigated that compaction is considered to be necessary to bring the clay minerals into close and intimate contact so that the inter growth of reaction products is facilitated. The particle orientation of the specimens is changed due to the different compaction.

Ei-Rawi (1968) found that the kneading compaction produce stronger specimen. He also stated that lime reacts slowly with soil particles and this phenomenon is more evident for specimens at reduced compaction. Very low density specimens are not well stabilized.

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) observed that higher strength and density in Modified Proctor test method than the Standard Proctor test method. He also observed that the compactive effort has a large effect on the CBR value of the lime stabilized soil. He observed that the CBR value of the stabilized soil is as twice in the Modified Proctor test method than the Standard Proctor test method. He observed 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. Fig 2.11 shows the variation of CBR value with moisture content for different compactive effort as found by him.

Moisture content for compaction

For untreated soil water is essential for proper compaction. Soil attain maximum dry density at the optimum moisture content of the soil for a specified compactive effort. Although optimum moisture content is highly dependent upon soil type, it is also dependent upon the compactive effort, At different compactive effort different optimum moisture content of the soil can be obtained. Besides compaction, water is also essential for the reaction of the lime with soil in lime stabilized soil. So the optimum moisture content of the lime stabilized soil differs form the optimum moisture content of the untreated soil for the same compactive effort.

Chu et al. (1955), Mateos et al. (1962) indicated that the Unconfined compressive strength of the lime stabilized soil is affected considerably by the moisture content of the mixture during the time of compaction, using Standard Proctor effort. They also suggested that the essential moisture content for compaction to get the maximum strength is slightly on the wet side of clay soil.

Feft (1965) also observed similar phenomenon when stabilized soil using different moisture content. He observed that the strength of the stabilized soil initially increase with the increase of water content. After a certain water content, the strength decreases.

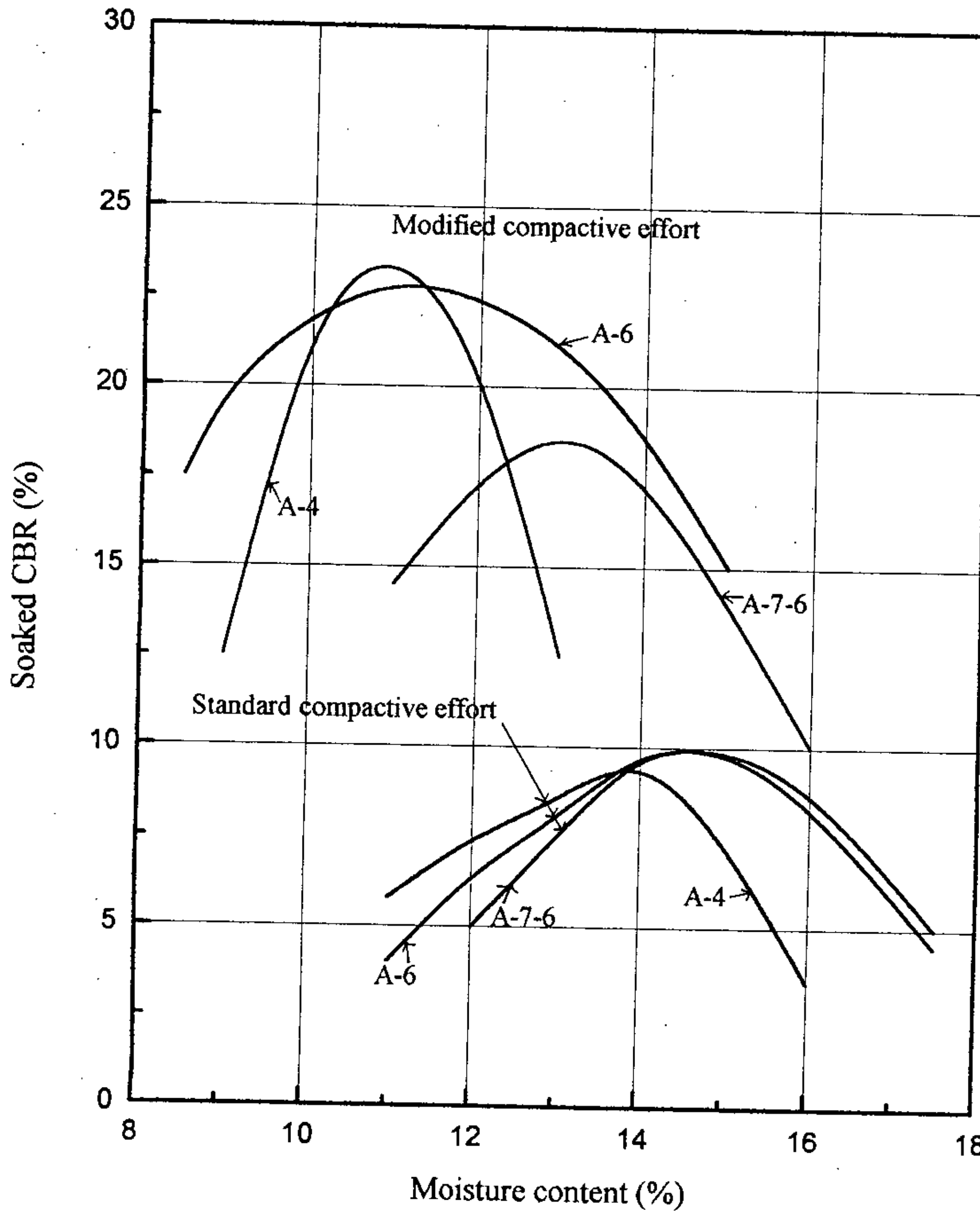


FIG. 2.11 EFFECT OF MOISTURE CONTENT ON CBR VALUE AT MODIFIED AND STANDARD EFFOER AFTER SERAJUDDIN (1990)

Choquette (1987) noted that the cementation process between lime and soil is responsible for the increase in strength and acts primarily on the cohesion component of the shear strength parameters of the soil. On the long term, high water content may even perform better than stabilized soil of low water content, likely because of the movement of solutes is eased within the pore space. Cementation is also related to structure of soil.

Locat et al. (1990) observed that the initial controlling reaction parameter are grain size and specific surface area. But with the development of the pozzolanic reactions, the mineralogy becomes the only parameter that is positively related to the strength development. For soils with high water content, lower strength development could be partly associated with fewer lime particles per unit volume. On the other hand, for high water content, the easier the mixing and the better the dispersion should be. The lower strength development was observed at high water content is mostly attributed the fact that more cementing products between the more distant soil particles before there is any significant strength increase.

Serajuddin (1992) pointed out that at the equal level of compactive effort, CBR value vary appreciably on variation of the moisture content during compaction. He also pointed out that the maximum strength of the stabilized soil occur at the 2% to 3% higher water content than the optimum moisture content of the soil depending upon soil type.

Compaction delay time

Compaction delay time is the time interval between mixing of lime with soil and compaction. Compaction delay time is critical for cement stabilization rather than lime stabilization. As the reaction of lime with soil occurs slowly, it is not essential to compact the specimen immediately after mixing the soil with lime.

Mitchell and Hopper (1961) observed that the strength of the lime stabilized soil decreases due to compaction delay time. Working with an organic clay mixed with 4% dolomitic hydrated lime, they found that at a delay of 24 hours between wet mixing and compaction results a loss of 8 Lb/cu.ft from the maximum dry density of 98.5 lb/ cu.ft as was found by compacting the specimen immediately after mixing. A loss of unconfined compressive strength of 30% was observed.

Mateos and davidson (1962) inferred from a laboratory study that delay of 24 hours had no significant effect on the density and strength for clay type of soil. But for dune sand, the decrease in dry density and strength was significant. They observed 7% decrease in strength for the clay soil and 30% reduction of strength for dune sand as compare to the specimen compacted after one hour and 24 hours of mixing. They suggested that when larger delays (two weeks or more) cannot be avoided, it may be necessary to incorporate a small amount of additional lime into the mixture (.5%) to compensate for loss due to carbonation and erosion.

Metcalf, (1977) stated that as lime stabilized soil has no rapid cementing action, the effect of delay in compaction is less pronounced. Thompson (1966) and Mitchell (1981) however, revealed that the delay between wet mixing and compaction of soil lime mix was of considerable importance in achieving high strength and durability.

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.

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. Fig 2.12 shows the variation of unconfined compressive strength of the lime stabilized soil due to the variation of compaction delay time. By an independent study they 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 upto 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 upto 4 days are not detrimental.

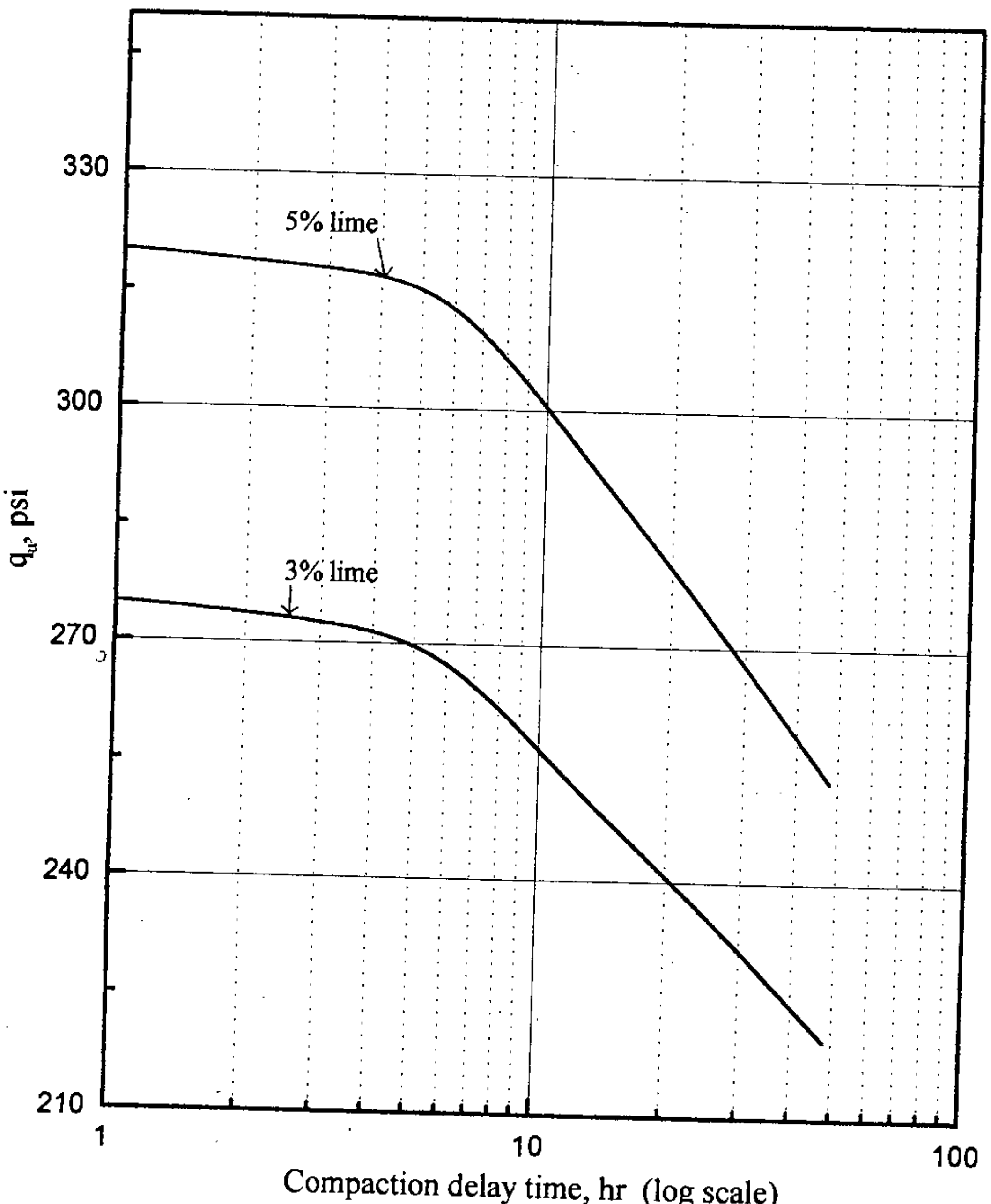


FIG. 2.12 VARIATION OF UNCONFINED COMPRESSIVE STRENGTH WITH COMPACTION DELAY TIME AFTER SASTRY ET. AL (1987)

Rodriguez et al. (1988) observed that compaction delay is not so critical for lime as critical for cement. They observed that only 3% decrease of unconfined compressive strength and about 2% decrease in dry density for a compaction delay period of 6 hours compared with the lime stabilized soil was compacted immediately after mixing. They also observed that after a few hours, lime mixture require more water for compaction. They explained that some water react with soil lime mixture so the water content of the mixture is reduced before compaction. Fig 2.13 shows the variation of dry unit weight due to the variation of compaction delay time as found by them.

Boominathan and Prasad (1992) stated that compaction delay of 24 hours can decrease the strength from 30% to 70%. In their statement, they pointed out that the reduction in strength and density are attributed to granulation of lose soil particles by week cementation, as the soil mellows.

There are controversial statement on the effect of strength due to the compaction delay time. Most of the authors stated that the delay of 24 hours has no marked effect on the strength of the lime stabilized soil, while some authors stated that compaction delay time of 24 hours has marked effect on the strength of the stabilized soil. However, fine grained soil has less effect on the compaction delay time. Lime stabilization with coarse grained soil showed marked effect on strength due to compaction delay time.

Mixing of lime and soil before compaction

In laboratory, for preparing the test specimen, lime is mixed with soil by hand for 5 to 10 minutes until a uniform mix is ensured. In the field soil lime mixture is mechanically mixed with different types of mechanical mixtures. Normal practice of mixing lime with soil in the field is of two ways, 1) In place mixing in which mixing and compaction is done in the same operation in the mechanical mixture. 2) Plant mixing, in which soil lime is mixed in a central plant and than transported to the site. Water is added to the mixture by the stream or by a fine spray while the mixture is remixed. After 2.5 minutes the mixture machine is stopped for few second to break the pieces and then again mixing is done for 2.5 minutes.

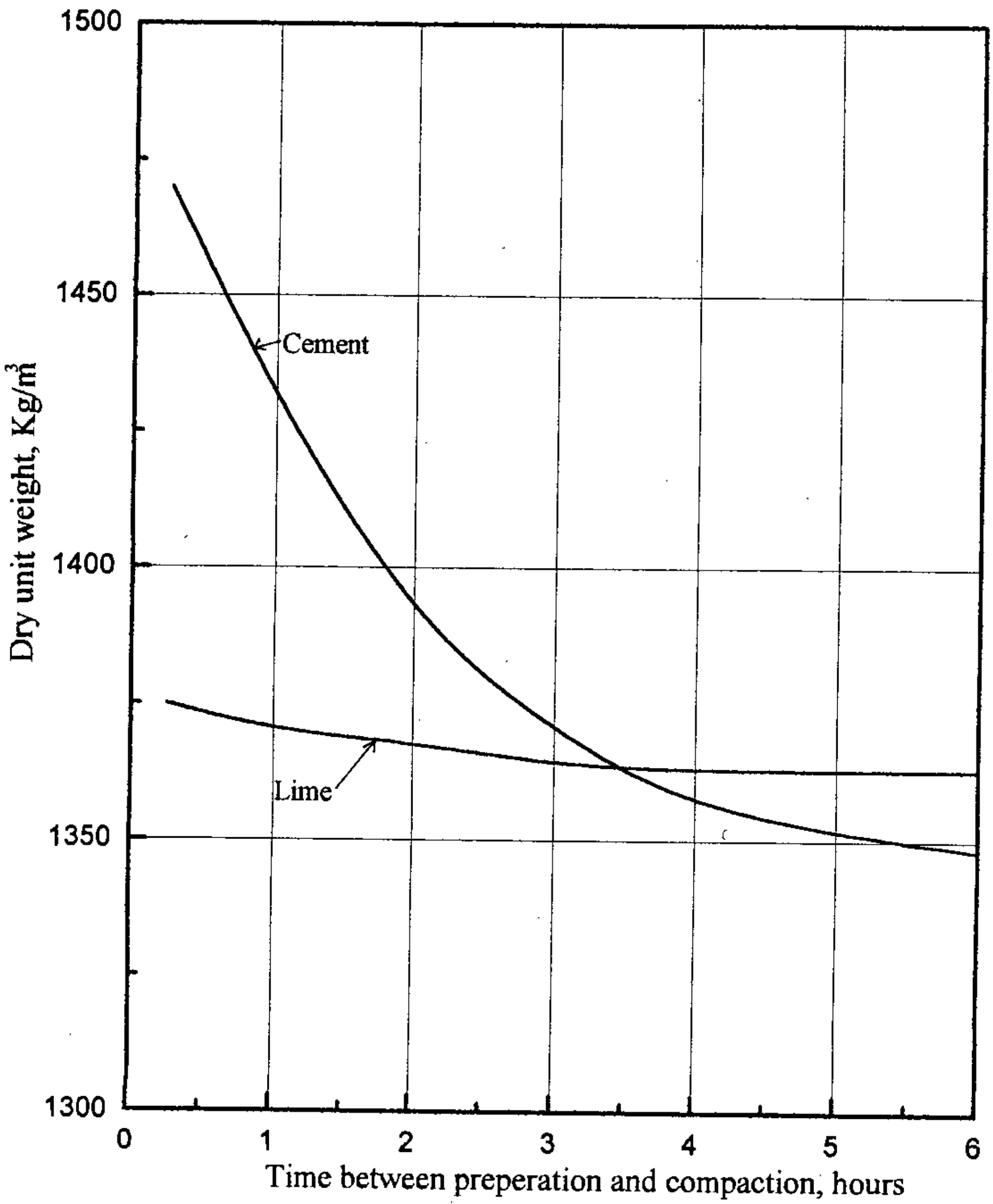


FIG. 2.13 EFFECT OF COMPACTION DELAY TIME ON DRY WEIGHT OF AN ACTIVE CLAY AFTER INGLES ET. AL (1987)

Chew et al. (1955) indicated that the increasing the time of mixing in a mechanical mixer at constant speed gives increased unconfined compressive strength. Khan (1989) pointed that it seems logical that an intimate mix of water and soils is necessary for maximum utilization of cementing properties of lime. Since lime reaction is slow prolonged mixing is not objectionable.

Compendium (1987) stated that compaction can be done by one and two stage of mixing. In two stage of mixing half of the total water and half of the total lime is mixed with soil and the rest water and lime is mixed after 18 hrs. They observed that due to two stage of mixing, unconfined compressive strength of the stabilized soil increases. They explained the reason for the increase of unconfined compressive strength is that after first stage of mixing the soil becomes more friable and the improved soil becomes more effective in lime stabilization.

2.4.4 Aging

Age effect on the stabilized soil

Shear strength of the lime stabilized soil increases with time in a similar manner to concrete or soil-cement mixtures as observed by many researchers from the beginning of lime stabilization. The rate of increase is generally rapid at the early stage of curing time, thereafter the rate of increase in strength decreases with increase of time.

Arman et al. (1972) stated that lime has an initial reaction with soil taking place during first 48-72 hours after mixing and secondary reaction starts after that period and continues.

Hilt et al. (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.

Lambe (1962) studied the change of unconfined compressive strength with age. The effect of time on the lime stabilized soil are presented in fig. 2.14. He mentioned the pozzalanic reaction is responsible for shear strength development of the lime stabilized soil. From the figure, it is seen that the increase of strength with time is also dependent upon lime type and lime content.

Ingles et al. (1972) studied the effect of initial time on unconfined compressive strength of the lime stabilized soil. The variation of strength gain of the lime stabilized soil for different curing period for the first few hours is shown in fig. 2.15. From the figure, it can be observed that unconfined compressive strength of the lime stabilized soil increase at a slow but constant rate at the first few hours but this phenomenon is not applicable for cement.

Ingles et al. (1972) also studied the effect of time on the unconfined compressive strength. The variation of strength for the different time as observed by them is presented in fig. 2.16. From the figure, it can be observed 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.

Ingles and Metcalf (1972) studied the change of physical condition of the soil due to the addition of lime to the soil. The physical conceptual model for lime stabilized sensitive clays is presented in fig. 2.17. The figure shows the particle orientation of the stabilized soil due to curing with low water content and with high water content. It can be seen from the figure that soil with high water content has particle orientation such that the particles come in contact with lime produces highs strength.

Townsend et al. (1970) observed that longer curing period is necessary for specimen compacted at reduced level of compaction.

Brandl (1981), however found that the time dependent increase in shear strength is approximately linear with the logarithm of time.

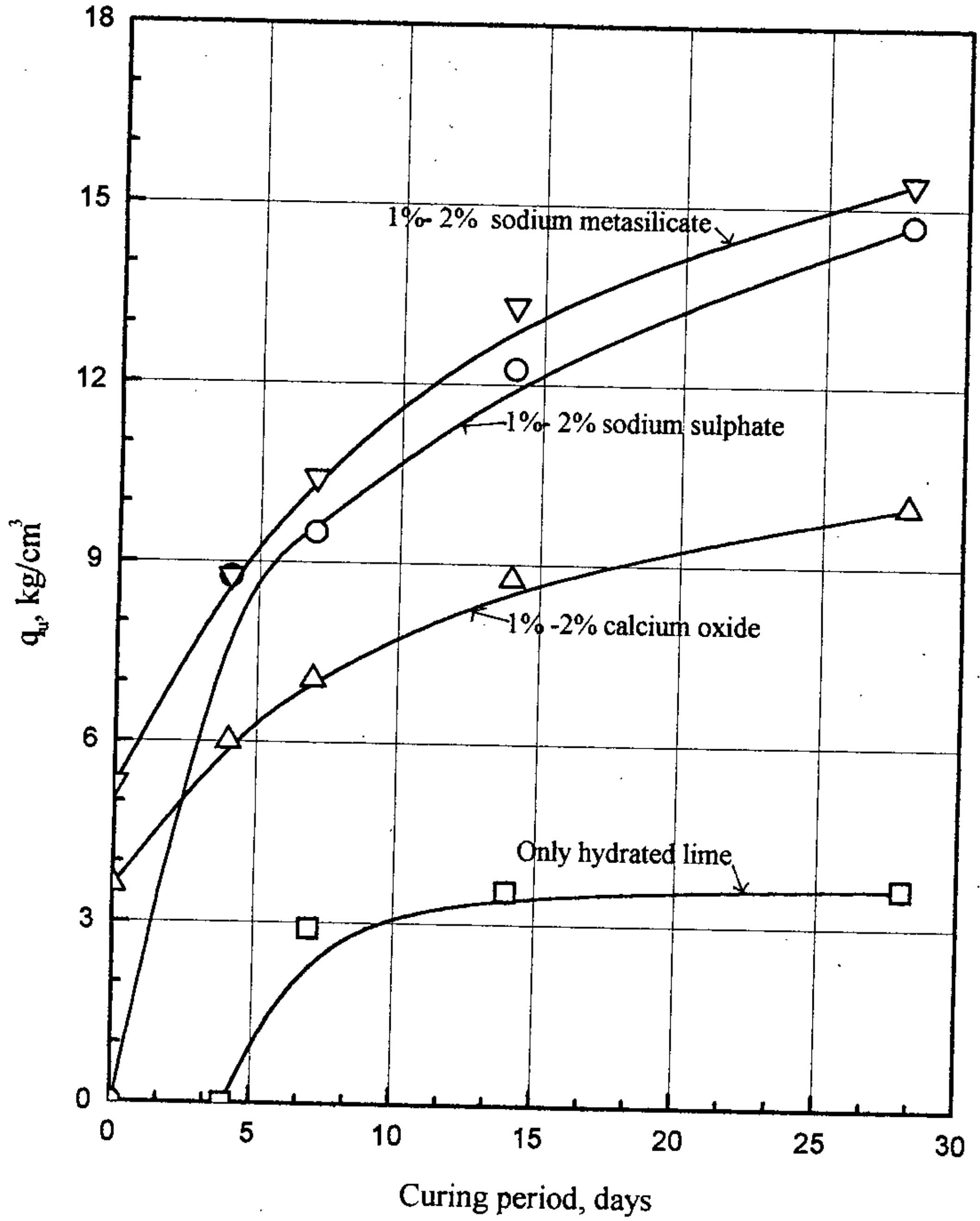


FIG 2.14 UNCONFINED COMPRESSIVE STRENGTH WITH CURING TIME AFTER LAMBE T.W. (1962), SILTY SOIL WITH ADDITIVE AS SHOWN USING 10% LIME

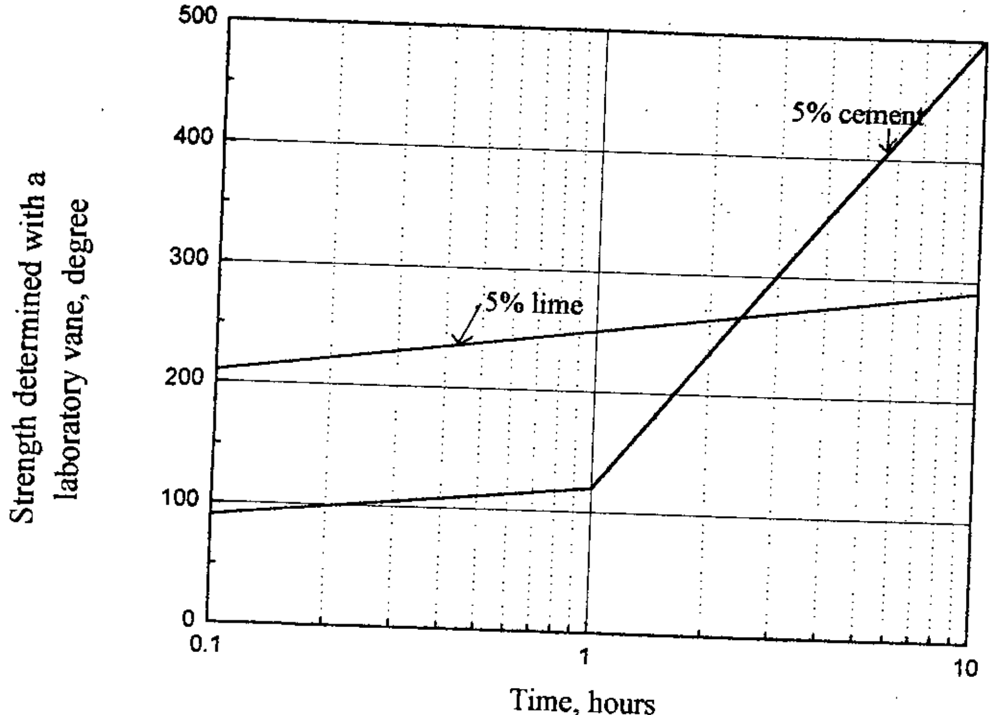


FIG. 2.15 CHANGE INTO THE STRENGTH OF STABILIZED CLAY DURING THE FIRST FEW HOURS AFTER STABILIZATION BY INGLES ET. AL. (1972)

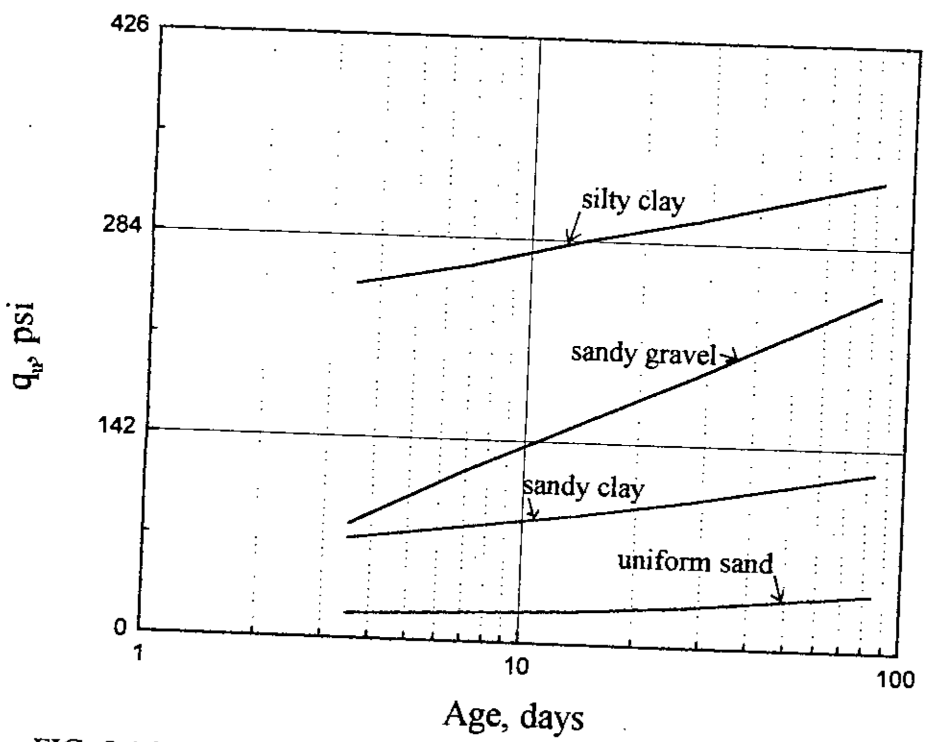


FIG. 2.16 EFFECT OF THE AGE OF A MIXTURE OF 5% LIME BY WEIGHT ON DIFFERENT TYPES OF SOILS (INGLES ET. AL 1972)

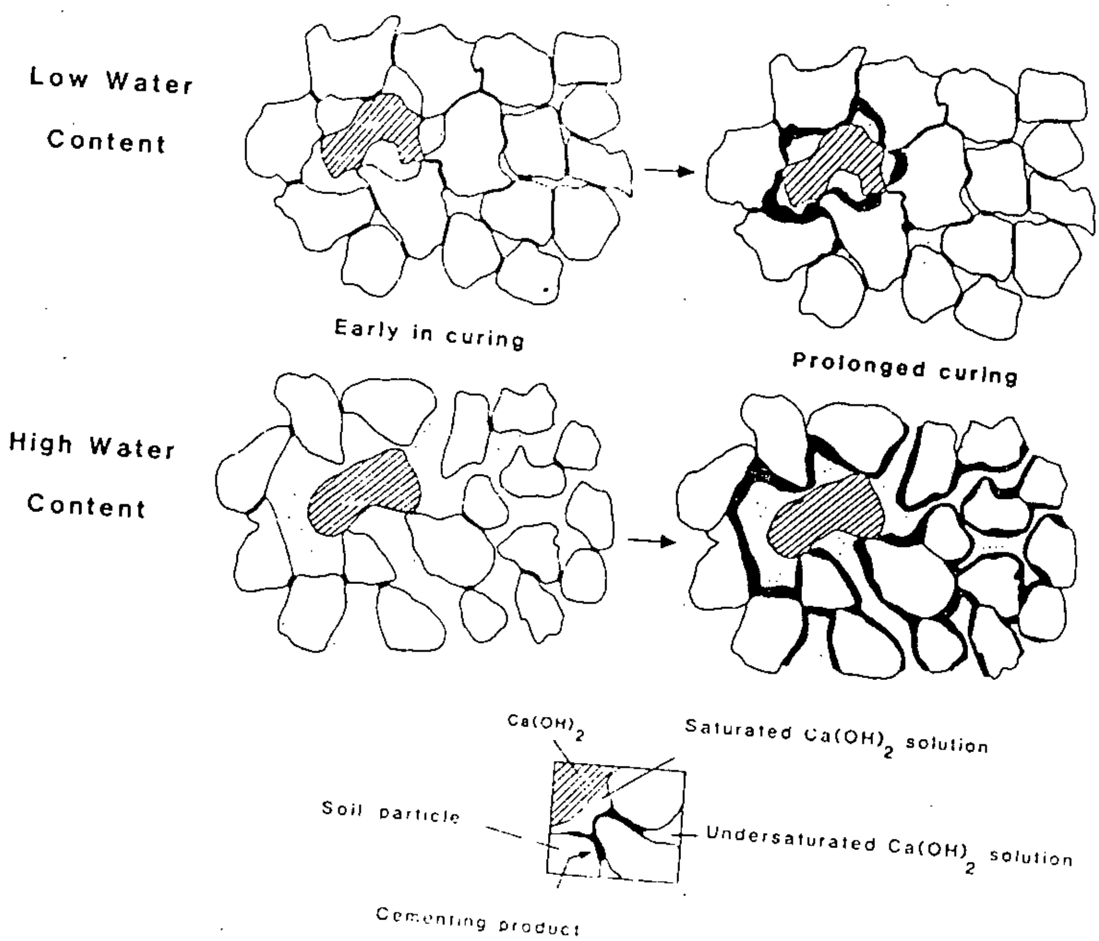


FIG. 2.17 PHYSICAL CONCEPTUAL MODEL FOR LIME STABILIZATION OF SENSITIVE CLAYS BY INGLES AND METCALF (1972)

Broms (1984) reported that the shear strength of the lime stabilized clay will normally be higher than that of untreated clay just after the mixing, even for the sensitive clays. The subsequent increase in strength which continues for years is mainly due to pozzolanic reactions. He also found that the strength of stabilized soil as determined by unconfined compressive strength test increases linearly with time when plotted on a log-log scale.

Sastry et al. (1987) stated that the compaction delay time of 48 hours will reduce the strength upto 20% which can be compensated by curing for 90 days instead of 28 days.

Curing temperature

Increase of curing temperature on lime soil mixture of the same age increases the strength. Broms (1984) observed that the temperature effects the reactions between lime and soil. High temperature is favourable to long term chemical reaction, since the solubility of silicates and alluminates (pozzalans) increases with temperature.

Warped & unwarped curing

Curing of specimen can be done by warped and unwarped condition. Ramle (1987) has pointed out that the warped specimen maintain a slightly higher moisture content than duplicate unwarped specimens. There was little difference between the strength of warped and unwarped specimens after two months of curing but after six month and one year of curing, the unconfined compressive strength of the warped specimens were greater than that of unwarped specimens by as much as 100%. This greater strength may have been due to a more uniform and slightly higher moisture content during curing or to the exclusion of carbon dioxide which reacted with lime to form calcium carbonate.

2.4.5 Summary

From the present discussion, it can be observed that the strength of the lime stabilized soil depends upon a number of factors. Some of the factors have marked effect on the lime stabilized soil while the others has minor effect. Field condition vary from place to place. For some selected areas, some factors always prevails which can not be avoided. The effect of one factor can be minimized by the other. Although the effect of the factors of lime stabilization are shown independently, actually they are interrelated.

According to Locat et al. (1990), the gain in shear strength of the lime stabilized soil (S_u), at a given time will be a function of many variables, These are:

$$S_u = f(S_{uo}, A, A_w, W_o, c, t_a, t)$$

where,

S_{uo}	=	the undrained shear strength immediately after compaction;
A	=	the mineralogical parameter that includes mineralogy, grain size, specific surface area, and cation exchange capacity,
A_w	=	a parameter related to the initial pore water chemistry,
W_o (%)	=	the moulding water content,
c (%)	=	the additive concentration
ta (days)	=	time of mellowing
t (days)	=	is the time of curing

Locat et al. (1990) expressed the relation of the shear strength and moisture content as power-law relationship are as follow: $S_u = a*w^b$,

where,

a and b are constants which depends upon the soil nature, curing time and lime concentration.

Too many variables affect the lime stabilization process. Field condition vary from place to place. So it is extremely important to study different variables and to adjust these variables for a certain area so that an economical and effective lime stabilization process can be established. Many researchers tried to find out an optimum and rational method for the lime stabilization considering the local condition and adjusting the variables that effect lime stabilization.

Locat et al. (1990) stated that planning of a lime stabilization project is not trivial They studied deliberately to find out an optimum condition for the lime stabilization process. They observed that although too many factors affect lime stabilization process, it is possible to find out rational method so that practically, lime stabilization process will be easy and will be able to get the maximum benefit from the stabilization.

Locat et al. (1990) from their experimental work in the laboratory proposed a chart for planning of the lime stabilization projects, presented in fig 2.18. For preparing the chart, water content, strength and lime content was considered. Similar chart can be prepare for other variables for planning of lime stabilization.

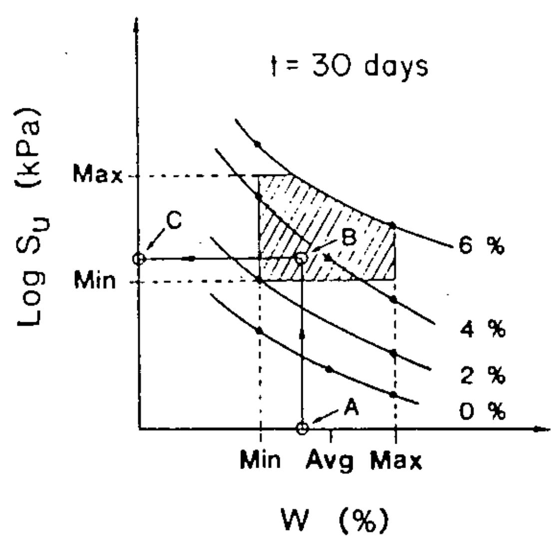


FIG. 2.18 LABORATORY CHART PROPOSED FOR PLANNING LIME STABILIZATION PROJECT AFTER LOCAT ET AL. (1990)

CHAPTER THREE

Laboratory Investigation

3.1 General

Shear strength of soil increases due to the addition of lime on it. The strength gain of the lime stabilized soil depends upon a large number of variables. The main objective of this research is to observe the change of strength of the lime stabilized soil due to the change of these variables. Among too many variables, tests were limited to some important variables. A short description of the total work is presented.

Three types of soil samples were used throughout the experimental work. The soils were collected from Jamuna Bridge site, Sirajgong (soil type-1), from BUET campus of Dhaka (soil type-2) and from Dhaleshari bridge site of Dhaka- Mowa road, Munshigonj (soil type-3). The laboratory investigations on the three types of soil were performed to determine optimum moisture content, particle size distribution, Atterberg limit, maximum dry density.

Lime was collected from open market and the properties of lime was determined by standard laboratory test (ASTM C 25-83 (1984).

Test specimens were prepared varying different state variables. The variables that were changed are soil type, lime percent, moisture content, compaction energy, compaction delay time and stage mixing.

Unconfined compressive strength test and CBR test was performed to evaluate the strength of the untreated soil specimen and the lime stabilized soil specimen.

3.2 Test programme

A test programme was undertaken to achieve the objectives of the study. A short description of the experimental work are be summarized:

i) Routine tests on the three types of soil were performed in the laboratory to classify the soils.

ii) Optimum moisture contents of the three types of soil were determined at the maximum dry density of the soil using Standard Proctor test method and Modified Proctor test method.

iii) The lime constituents as collected from open market was found out through standard chemical test of lime (ASTM C 25-83 (1984).

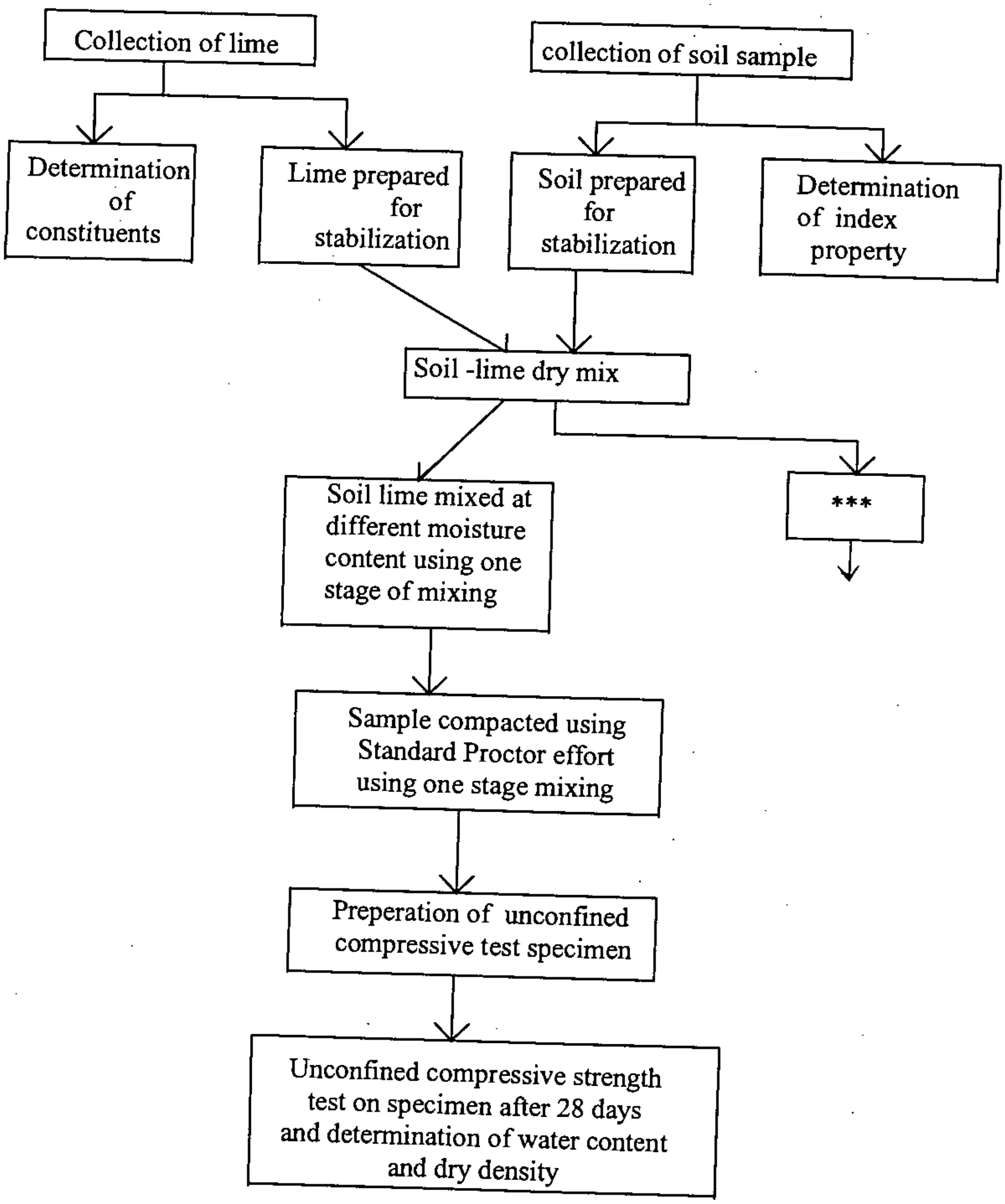
iv) Optimum moisture contents of the three types of stabilized soil were determined at the maximum dry density of the soil lime mix using Standard Proctor test method at different lime content and by Modified Proctor test method using 3% lime content.

v) Unconfined Compressive strength test specimen was prepared by compacting the untreated soil using Standard Proctor testing method at different moisture content.

vi) Unconfined Compressive strength test specimen was prepared by compacting the soil lime mix using Standard Proctor testing method at different moisture content for different lime percent.

vii) Test specimen was prepared by compacting the soil lime mix using Standard Proctor testing method near optimum moisture content by varying:

FLOW CHART OF THE EXPERIMENTAL WORK



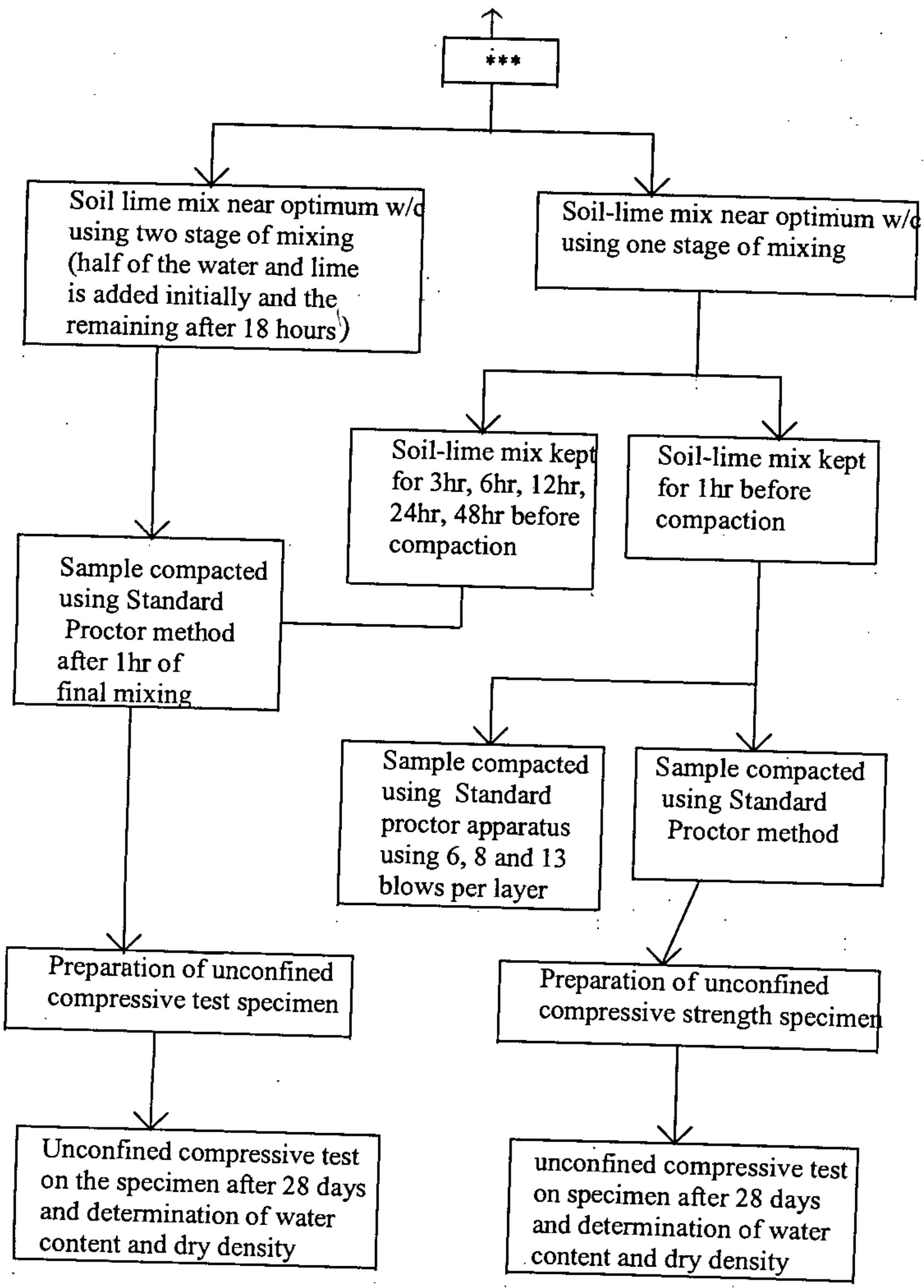


FIG. 3.1 FLOW CHART OF EXPERIMENTAL WORK

i. compactive effort
ii. compaction delay time
iii different age
iv mixing method such as, one stage and two stage mixing.

viii) CBR test was performed on the stabilized soil compacted near the optimum moisture content.

A flow chart of the laboratory investigation is shown in fig. 3.1.

3.3 Materials used

For the stabilization of soil with lime, soil was mixed with lime in presence of water. Distilled water was used.

In addition to these primary materials, some chemicals were used to determine the properties of the soil and to identify the constituents of lime.

3.3.1 Soil type

To observe the effect of soil type upon the strength of the lime stabilized soil three types of soil was selected for investigation.

Soil type-1

This type of soil was obtained from the Jamuna Bridge site, Sirajgong at a depth of about 1.5 meters below the existing ground level. This type of soil was gray in colour and having medium plasticity.

Soil type-2

This type of soil was obtained from Polashi at the site of BUET Girls High School at a depth of about 1.5 meters below the existing ground level. The colour of this type of soil is reddish gray having high plasticity.

Soil type-3

This type of soil was obtained from the site of Dhaka-Mowa road of the Dhaleshari bridge of site of Dhaka-mowa road, Munshigonj at a depth of about 1.5 meters below the existing ground level. The colour of this type of soil is gray having low plasticity.

After the soil samples were air dried, it was broken with a mallet and was sieved through the mesh using the sieve no. 10 of U.S. standard. The portions passing through no. 10 sieve was thoroughly mixed and separated in small batches and was kept in plastic bags. The soil was then ready for experimental use.

The following tests on the three types of soils were performed using standard methods as follows:

i)	Gradation using wet sieve & hydrometer analysis	ASTM D 2487-85 (1989
ii)	Liquid limit and plastic limit	ASTM D 4318-84 (1989)
iii)	Optimum moisture content	ASTM D 698-78 (1989)

The test results of the three types of soil is presented in Table 3.1. According to Unified Soil Classification System (ASTM, 1978), soil type-1 is CL, soil type-2 is CL and the soil type-3 is ML. According to AASHTO (1993) System of Soil Classification, soil type-1, soil type-2 and soil type-3 are A-6(11), A-7-6(20) and A-4(9) respectively. The test results also indicate that three type of soil has different clay content.

TABLE 3.1 PHYSICAL PROPERTIES OF THE SOIL USED FOR LIME STABILIZATION

Properties of the soil	soil type-1	soil type-2	soil type-3
Textural composition:			
sand %	12	19	10
silt %	81	60	86
clay %	7	21	4
Atterberg limits:			
Liquid limit %	34	47	37
Plastic limit %	21	21	28
plasticity index %	13	26	9
Classification:			
Unified (ASTM, 1976)	CL	CL	ML
AASHTO (1993)	A-6(11)	A-7-6(20)	A-4(9)

To determined the grain size distribution of the three types of soil, wet sieve analysis and hydrometer analysis (combined analysis) was performed on the soil sample. The results of the grain size analysis for the three types of soil are presented in the fig. 3.2.

3.3.2 Lime

One kind of lime was used for the experimental work. After collection of lime from open market, it was kept sealed to prevent carbonation until immediately before use. The test of the lime was done on the laboratory according to ASTM standard procedure ASTM C 25-83 (1984). The results of the chemical analysis of lime are shown in Table 3.2.

The quantity of calcium and magnesium were determined by Volumetric Method (complexometric titration with EDTA). For total calcium and magnesium titration Eriochrome black-T indicator was used. For calcium titration Patton and Reeders indicator was used. Impure silicon or acid insoluble matter was determined by Gravimetric Method. Loss on ignition was determined by heating sample in muffle furnace and weighting.

3.3.3 *Water*

Distilled water was used throughout the test in all the mixture of soil and lime to reduce the effect of impurities in the mixing water.

3.4 State variables

The strength of the lime stabilized soil changes with the change of state variables. A number of unconfined compressive strength test and CBR test were carried out on the soil specimen to observe the effect of variables. Those of the different variables used in these research are presented below:

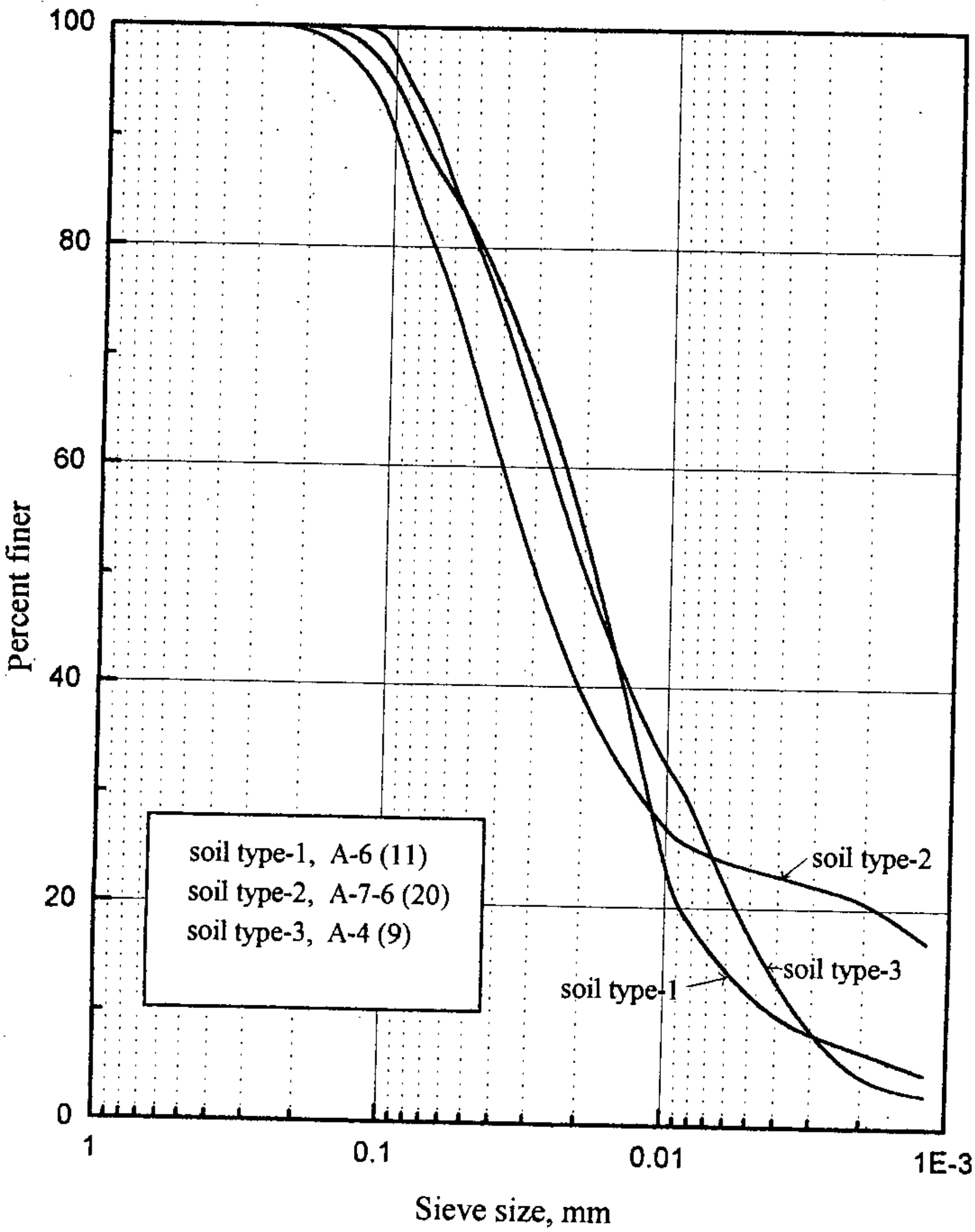


FIG. 3.2 GRAIN SIZE DISTRIBUTION CURVE

TABLE 3.2 CONSTITUENTS OF LIME FROM CHEMICAL ANALYSIS

name of the ingredients	quantity of ingredient in percent
CaO	48.72
MgO	17.60
Fe, Al, Na, K etc.	8.18
SiO_2	1.80
* loss due to ignition	23.7

*The loss in ignition is due to the removal of water from $Ca(OH)_2$ and $Mg(OH)_2$

Soil

Three types of soil as mentioned earlier was used throughout the test to observe the effect of soil type on the lime stabilization.

Moisture content

The optimum water content for the three types of soil was determined using ASTM D 698-78 (1989) procedure. Water was mixed with the soil on the basis of the percent of the moisture content to dry weight of the soil. To observe the effect of moisture content on the stabilized soil, unconfined compressive strength test specimen were prepared using different water content.

Lime content

Conventional lime as available at the open market was used throughout the test. Strength tests were performed in stabilized soil using different percentage of lime.

Compaction delay time

Compaction delay time is the time interval between wet mixing and compaction of the soil mix. After thorough mixing, soil-lime mix was placed in a plastic bag with a seal and stored in a humid chamber. The mixture was kept for a compaction delay time of 1 hr, 3 hr, 6 hr, 12 hr, 24 hr and 48 hrs. After the delay time, the soil-lime mix was again mixed before compaction to ensure uniform distribution of moisture.

Compaction energy

Moisture-density relationship was found for the three types of soil, using Standard Proctor test method (ASTM D 698-78, 1989) i.e 25 blows of 2.49 kg mass hammer was applied dropping from a height of 30.48 cm on each of three successive layers of soil sample in a compaction mould. The size of the mould was 10.16 cm diameter and 11.46 cm height. Moisture-density relationship was also determined using Modified Proctor test method (ASTM D 698-78, 1989) i.e 4.52 kg hammer dropped from a height of fall of 45.72 cm.

Test specimen for unconfined compressive strength test was prepared by trimming a specimen from a compacted soil lime mix using Standard Proctor test method (ASTM D 698-78, 1989).

To observe the effect of higher compaction energy on the lime stabilized soil, test specimen was also prepared by compacting the specimen using Modified Proctor test method (ASTM D 698-78, 1989).

To observe the effect of lower compaction energy on the lime stabilized soil, specimen was prepared using reduced compactive effort by reducing the no. of blows per layer (i.e. 13 blows, 8 blows, 6 blows per layer) but keeping other variables similar to the standard proctor test method (i.e weight of hammer, height of fall, number of layers).

Total compactive effort was 12.375 k-ft/cu.ft for 25 blows per layer, 6.435 k-ft/cu.ft for 13 blows per layer, 3.96 k-ft/cu.ft for 8 blows per layer and 2.97 k-ft/cu.ft for 6 blows per layer at Standard Proctor test method. Compactive effort was 56.25 k-ft/ cu.ft for Modified Proctor test method.

Stage mixing

After adding water on the soil lime mixture, it was thoroughly mixed with water. Total water was applied on the soil lime mix at a time. This type of mixing procedure is called as one stage of mixing. To observe the effect of stage mixing, another type of mixing procedure was followed which is called two stage of mixing. In this type of mixing procedure, half of the total water was added and then the mixture is allowed to stay for 18 hours, remaining water and lime was then added and remixed.

3.5 Preparation of the stabilized soil specimen for strength test

In order to evaluate the strength of both the untreated and stabilized soil, test specimen was prepared for unconfined compressive strength and CBR test.

Preparation of soil specimen for unconfined compressive strength test

To perform the unconfined compressive strength test on the lime stabilized soil, specimen was prepared with 1.5 inch diameter and height of 3 inch. After compacting the soil lime mixture in the compaction mould, it was divided into two parts. Each part of soil was trimmed off. After making the soil sample of about a size of 1.5 in dia, it was inserted in a block of size 3 in hight and 1.5 in dia and again the specimen was trimmed to get the specimen of block size. The preparation of the test specimen can be described by the flow chart as presented in the fig. 3.3.

After preparing the test specimen of 1.5 inch dia and 3 inch height, the specimens were completely warped with plastic bag and kept in a desiccator sealed with paraffin wax and kept in humid condition for a period of 7, 14, 28 days.

FLOW CHART FOR THE PREPERATION OF SPECIMEN FOR UNCONFINED COMPRESSIVE STRENGTH TEST

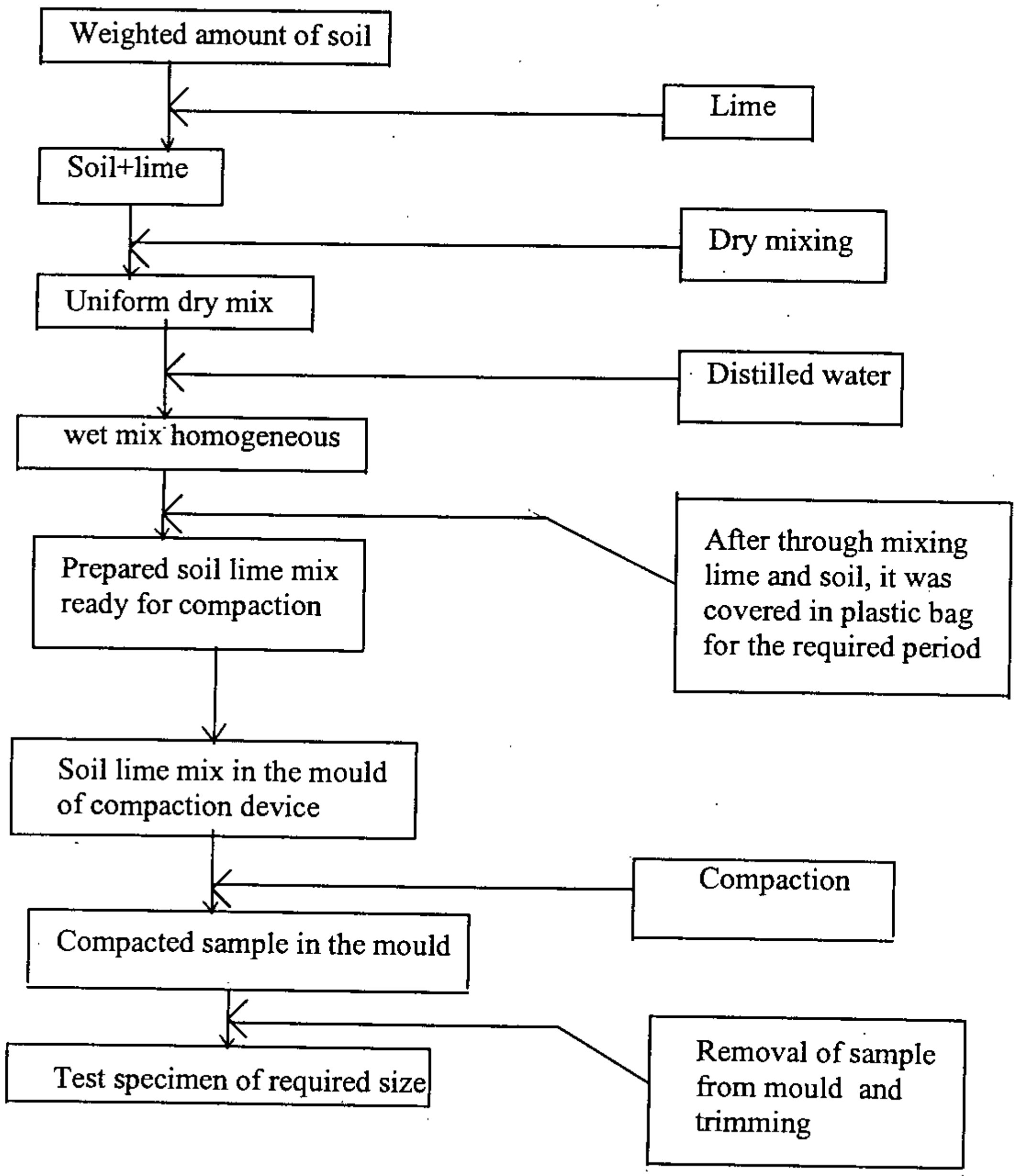


FIG. 3.3 FLOW CHART SHOWING THE DIFFERENT STAGE FOR THE PREPERATION OF SOIL SPECIMEN FOR UNCONFINED COMPRESSIVE STRENGTH TEST

Preparation of the soil sample for CBR test

To determine the strength under soaked and confinement of the lime stabilized soil, CBR test specimen was prepared to perform CBR test according to AASHTO T-193 (1993) procedure. Weighted amount of soil was thoroughly mixed with lime near the optimum water content of soil lime mix The soil sample in the mould were compacted by a 5.5 lb hammer applied in three layers with 65 blows per layer, having a height of fall of 18 inch. Moisture content of the prepared sample was measured.

To observe the effect of compactive effort specimen was also prepared using 10 blows and 30 blows per layer in three layers with a height of fall of 18 inch. For each type of compactive effort three test specimen were taken for testing. Same moisture content was used for 10, 30, 65 blows per layer at a given lime content and average moisture content was determined.

The test specimen were kept in water for 4 days, keeping a surcharge weight of 10 lbs on the top of the mould. During soaking, the water in the mould and soaking tank was maintained approximately 1.0 inch above the top of the specimen. After 4 days, the penetration test was performed.

3.6 Strength tests on the stabilized soil specimen

To evaluate the strength of stabilized and untreated soil, strength test on specimen was performed by two recognized and widely used testing methods. These are unconfined compressive strength test and CBR test.

Unconfined compressive strength test

Unconfined compressive strength was perform according to ASTM D 2166-85 (1989) test procedure. The prepared specimen was placed on a plate of a loading device. Loading device was adjusted carefully such that the upper platoon just make contact with the specimen. Reading of deformation indicator was made zero.

The rate of deformation during testing for stabilized soil was chosen to be 0.15 mm per minute. The rate was calculated taking 0.2% of the height of the stabilized specimen per minute.

The axial deformation of the specimen and the corresponding load applied were recorded at frequent intervals to define stress-strain curve. The maximum load causing failure of the specimen was taken as the unconfined compressive strength of the corresponding soil mix.

California Bearing Ratio (CBR) test

California Bearing Ratio test was performed on both the untreated and stabilized soil according to AASHTO T-193 (1993) procedure.

After compacting the soil-lime mix block on the penetration device, penetration piston was set with smallest possible load. The initial load is required to ensure satisfactory setting of the piston and was considered to zero when determining the load penetration piston. Both the stress and strain gauge was made zero.

Load was applied on the penetration piston such that the rate of penetration was 0.05 in/min. Penetration load was calculated in lb/sq.in and the load-penetration curve was plotted. CBR value were determined from load penetration curve.

CHAPTER FOUR

Results and Discussions

4.1 General

The properties of the soil collected from the three different areas of Bangladesh were determined at the laboratory to identify the soil and to ascertain its different characteristics. Chemical analysis of lime was done in the laboratory. Lime stabilized soil specimen was prepared and effect of changing lime percent, moisture content, compactive effort, compaction delay time, age, stage mixing was studied. To evaluate the strength of the stabilized soil, unconfined compressive strength and California Bearing Ratio (CBR) test was performed on the prepared specimen. All the test results are presented and discussed in this chapter.

4.2 Properties of the materials used

Soil

The physical properties of the three types of soil was determined according to standard methods. Soil was classified according to AASHTO (1993) and Unified Soil Classification System ASTM (1978). The results of the different physical properties of the soil has shown in Table 3.1.

According to Unified Soil Classification System (ASTM, 1978), soil type-1 and soil type-2 are CL type and soil type-3 is ML type. According to AASHTO (1993) soil type-1, soil type-2 and soil type-3 are A-6, A-7-6 and A-4 type respectively. All of the three types of soil has different clay content which is considered as an important controlling factor for lime stabilization.

Lime

The results of the chemical analysis of lime collected from open market is shown in Table 3.2. From the table, it can be observed that MgO and CaO content together is 66.32% of the total lime. On ignition, 23.7% water was removed from the lime. The lime used had 87.62% of the pure hydrated lime.

4.3 Effect of the state variables on the lime stabilized soil

A large number of variables affect the lime stabilization process. Effect of the change of moisture content, lime percent, compactive effort, soil type, compaction delay time, age, stage mixing on the lime stabilized soil have been studied.

4.3.1 Effect on optimum moisture content due to the addition of lime to soil

Lime react with soil silica in presence of water. So water is an important factor for lime stabilization. Dry density of the soil was determined for all the three types of soil at different moisture content using Standard and Modified Proctor test method. The results are presented in Table annex-1. The relationship between the dry density and water content for three types of soil is shown in fig 4.1.A, fig 4.1.B and fig. 4.1.C. It can be seen from the figures that dry density of the soils depend upon the moisture content. Maximum dry density attains at a certain moisture content at a given compactive effort, which is known as the optimum moisture content of the soil.

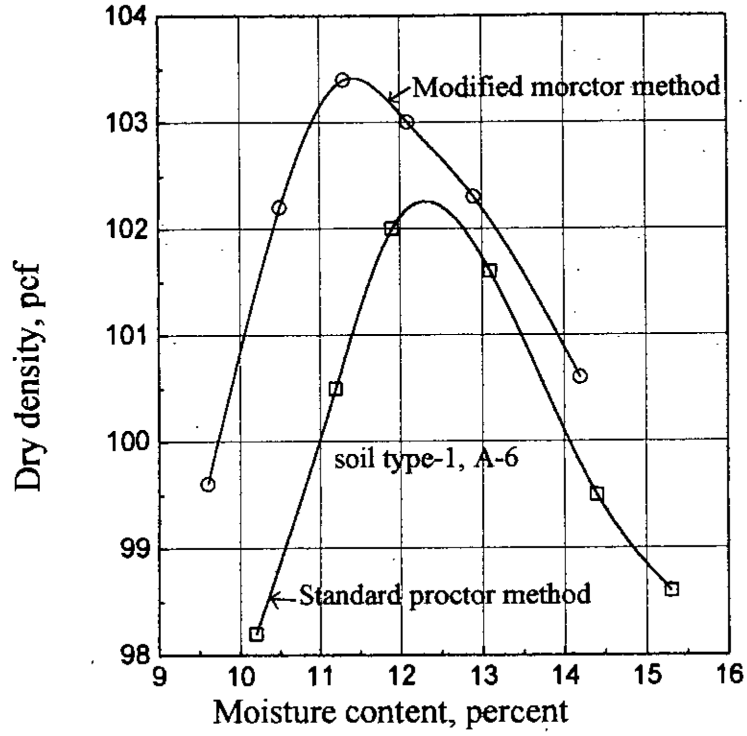


FIG. 4.1A MOISTURE-DENSITY RELATIOPNSHIP FOR SOIL TYPE-I

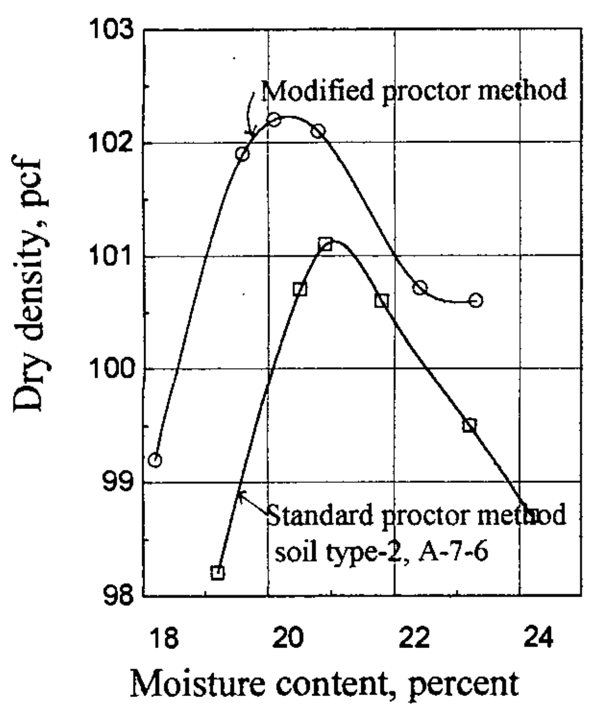


FIG. 4.1B MOISTURE-DENSITY RELATIOPNSHIP FOR SOIL TYPE-2

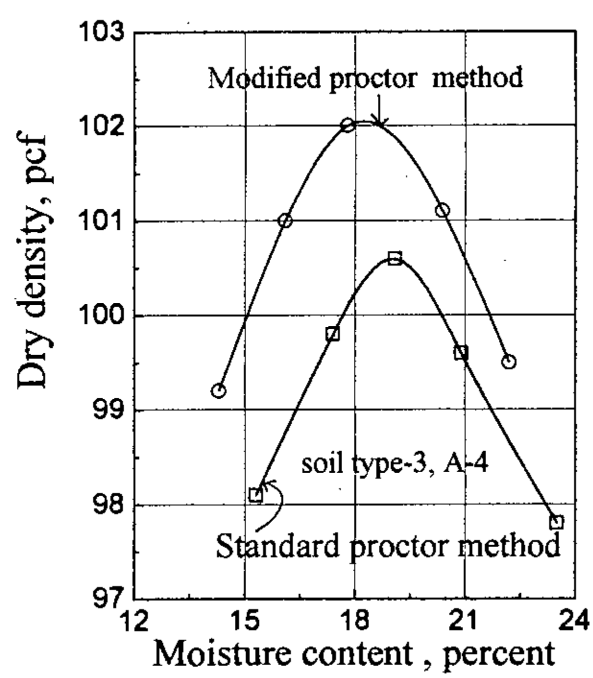


FIG.4.1C MOISTURE- DENSITY RELATIONSHIP FOR SOIL TYPE-3

It can also be seen from the fig 4.1.A, fig 4.1.B and fig. 4.1.C that the optimum moisture content depends on compactive effort. Optimum water content decreases with the increase of compactive effort. For soil type-1, the optimum moisture content using Standard Proctor test method is 12.5 percent and by Modified Proctor method is 11.5 percent. Optimum moisture content for soil type-2, using Standard Proctor method is 21.0 percent and by Modified Proctor method is 20.1 percent. Values of the optimum moisture content, using Standard and Modified proctor method are 18.8 and 18.0 respectively, for soil type-3.

From the figures, it can be also seen that the maximum dry density is a function of compactive effort. For soil type-1, the value of the maximum dry density is 102.4 lb/cu.ft by Standard Proctor test method and 103.5 lb/cu.ft by Modified Proctor test method. For soil type-2, the value of the maximum dry density is 101.2 lb/cu.ft by Standard Proctor test method and 102.3 lb/cu.ft by Modified Proctor test method. Values of maximum dry density is 100.7 and 102.1 lb/cu.ft by standard proctor and by Modified Proctor test method respectively for soil type-3.

Dry density of the lime treated soil was determined for all the three types of soil at different moisture content using Standard Proctor test method. The relationship between dry density and the moisture content of the stabilized soil as well as untreated soil are shown in fig. 4.2A, fig. 4.2B and fig. 4.2C. From the figures, it can be seen that the dry density of the lime stabilized soil reduces due to the addition of lime on the soil. For soil type-1, maximum dry density of untreated soil is 102.4 lb/cu.ft and maximum dry density of stabilized soil using 3% lime is 98.2 lb/cu.ft. For soil type-2, dry density is 101.3 lb/cu.ft for untreated soil and dry density is 98.0 lb/cu.ft for lime stabilized soil with 3% lime content.

From the fig. 4.2A, fig. 4.2B and fig. 4.2C, it can be seen that optimum moisture content of the soil increases due to the addition of lime on it, i.e. lime treated soil get the maximum dry density at a higher moisture content than the optimum moisture content of the untreated soil. Optimum moisture content of the untreated soil type-1 is 12.5 percent and the optimum moisture content of the lime treated soil using 3% lime is 13.2 percent. Optimum moisture content of the lime treated soil type-2 using 3% lime is 22.7 percent and for untreated

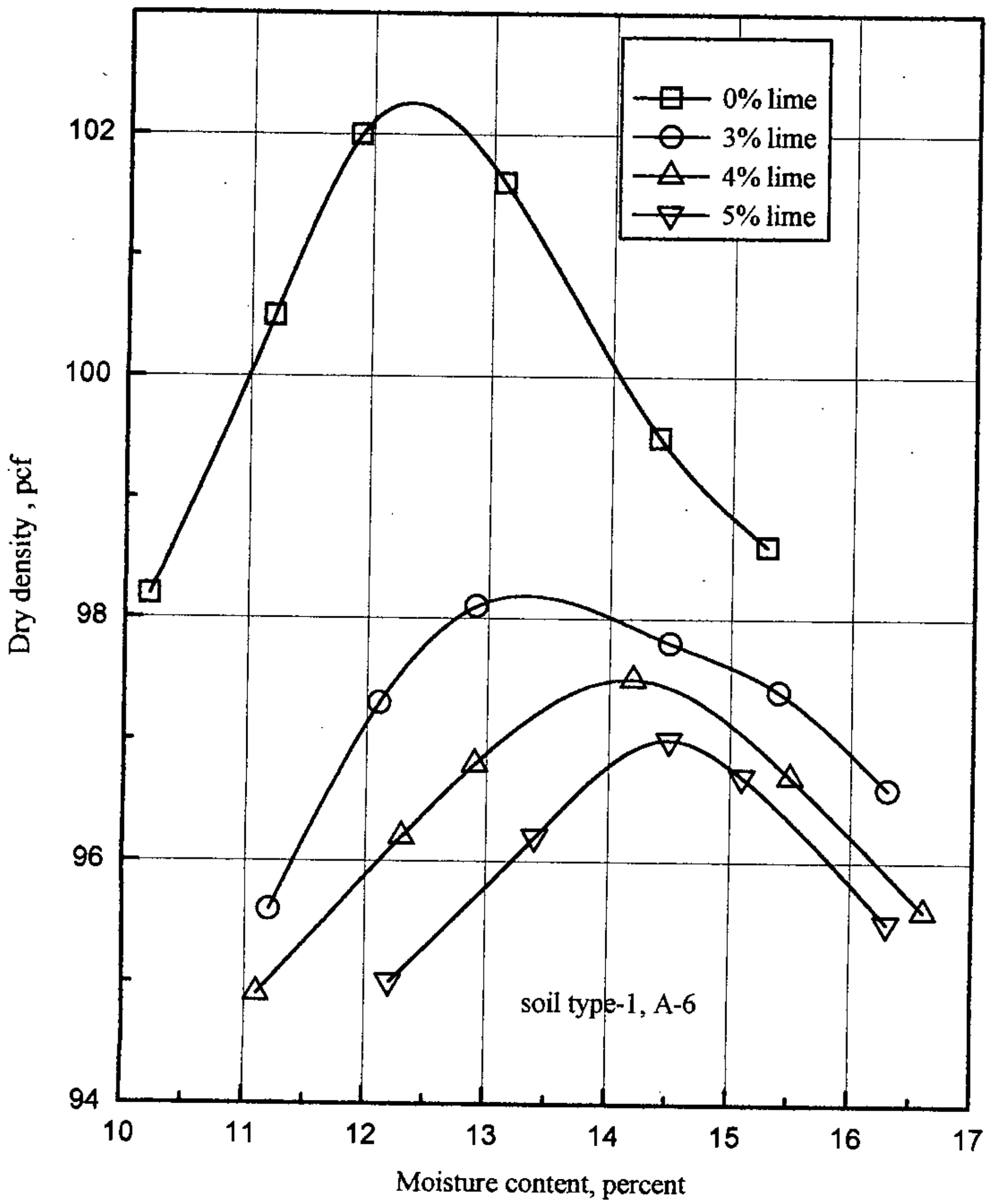


FIG. 4.2A MOISTURE- DENSITY RELATIONSHIP FOR STABILIZED SOIL TYPE-1 AT DIFFERENT LIME CONTENT USING STANDARD PROCTOR METHOD

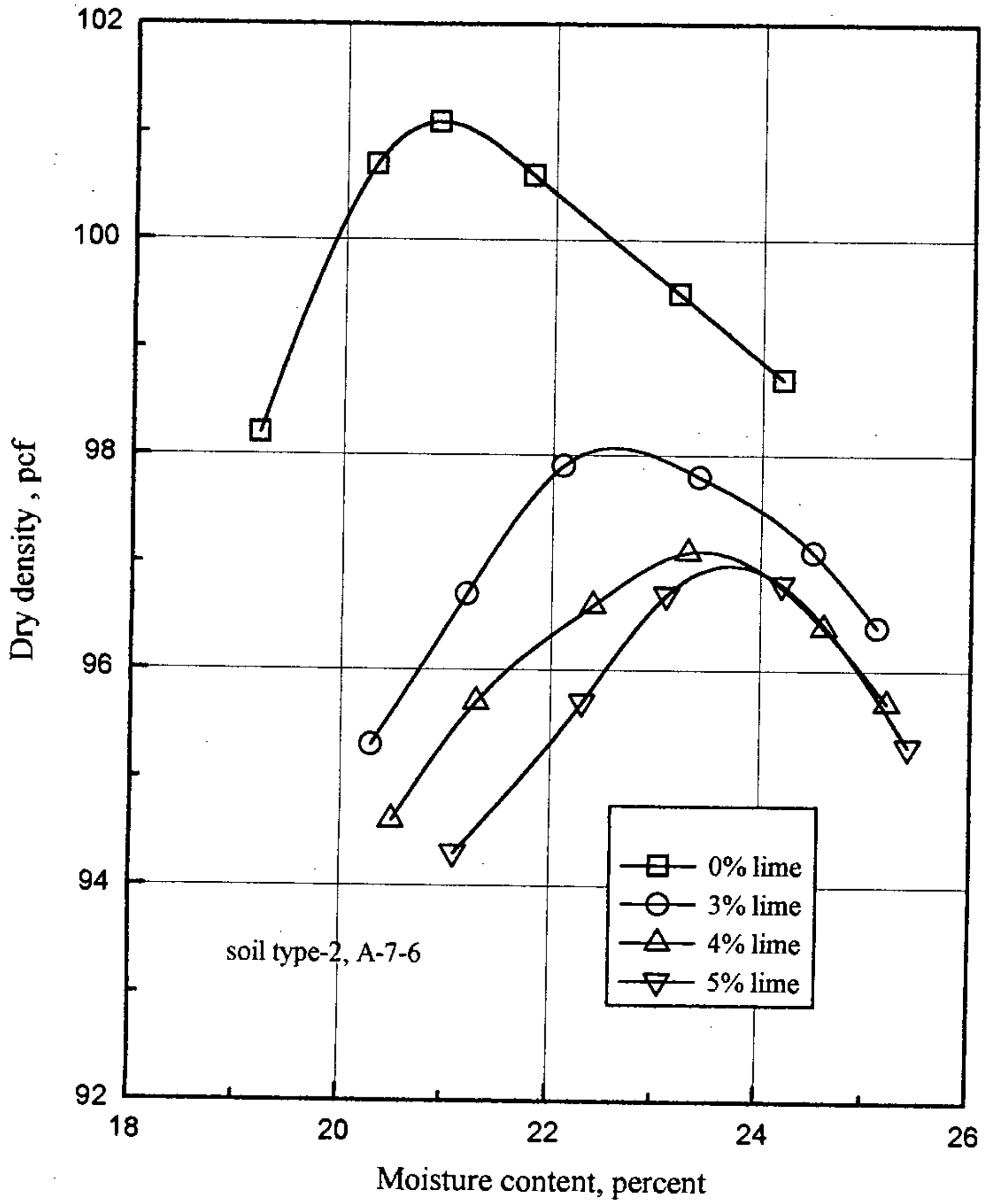


FIG. 4.2B MOISTURE- DENSITY RELATIONSHIP FOR STABILIZED SOIL TYPE-2 AT DIFFERENT LIME CONTENT USING STANDARD PROCTOR METHOD

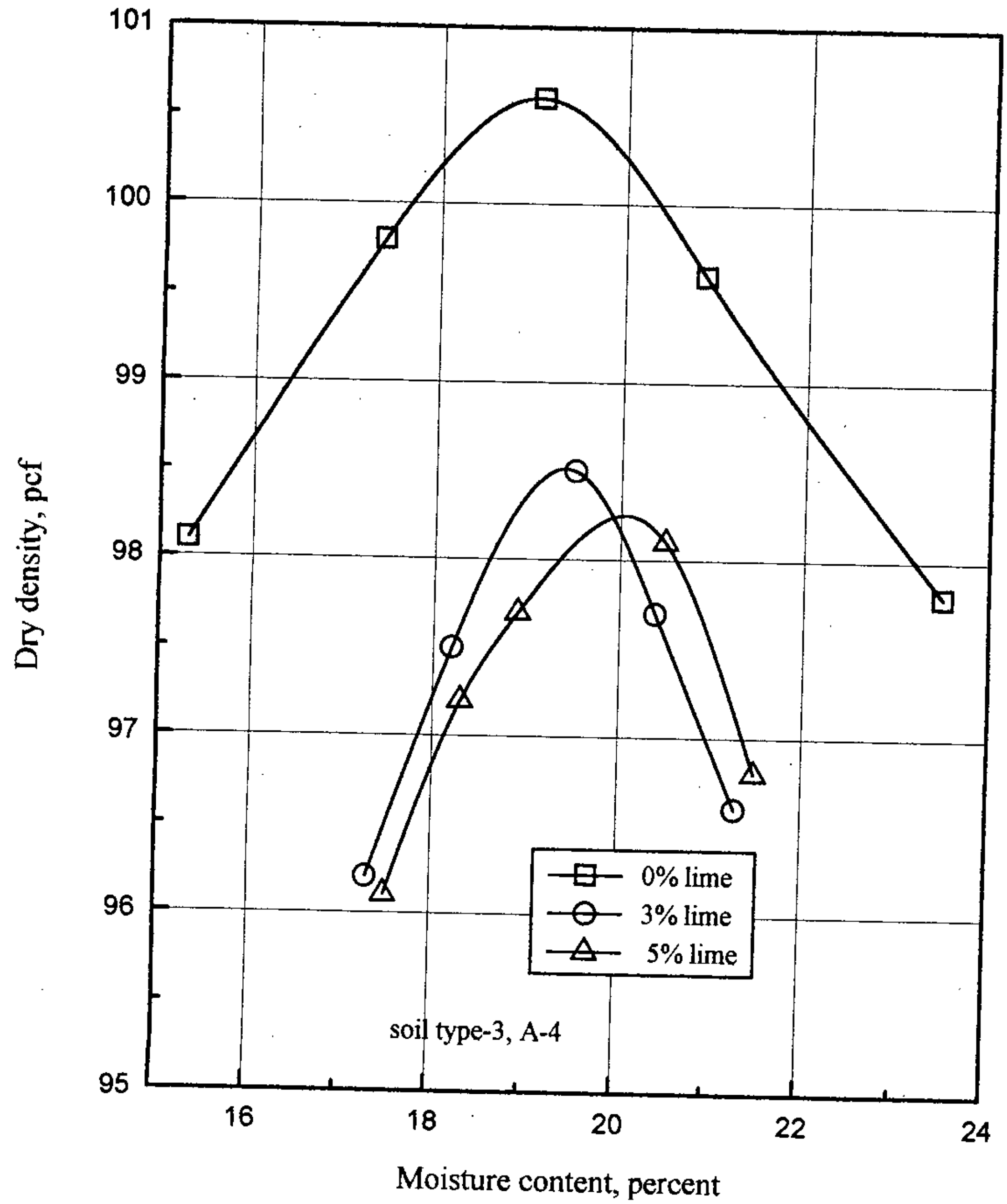


FIG. 4.2C MOISTURE DENSITY RELATIONSHIP FOR THE STABILIZED SOIL TYPE- 3 AT DIFFERENT LIME CONTENT USING STANDARD PROCTOR TEST METHOD

soil the value is 21.0 percent. Optimum moisture content of the lime treated soil type-3 using 3% lime is 19.4 percent and for untreated soil this value is 18.8 percent. Therefore, it can be concluded that the optimum moisture content of the lime treated soil is higher than the optimum moisture content of the untreated soil.

It can be observed from the fig. 4.2A, fig. 4.2B and fig. 4.2C that the optimum moisture content increases with the increase of lime content in lime stabilized soil. Optimum moisture content for 3% lime is 13.2 percent and for 5% lime content is 14.3 percent for soil type-1. Similar trend of results was also observed for the soil type-2 and soil type-3.

The value of maximum dry density and corresponding optimum moisture content determined from fig. 4.2A, fig. 4.2B and fig. 4.2C of both untreated and lime stabilized soil with different percent of lime using Standard Proctor effort is presented in Table 4.1.

It can also be observed from the table 4.1 and fig. 4.2A, fig. 4.2B and fig. 4.2C that the maximum dry density decrease with the increase of lime content for all the three types of soil.

Density and water content relationship of the soil was also determined by Modified Proctor test method using 3% lime content. Relationship between the dry density and the water content using Standard Proctor and Modified Proctor energy for three types of stabilized soil are shown in fig. 4.3A, fig. 4.3B and fig. 4.3C. From the figures, it can also be seen that the dry density of the lime stabilized soil follow the similar trend as untreated soil i.e. dry density is higher in Modified Proctor test method than the dry density in the Standard Proctor test method. The maximum dry density of the lime stabilized soil type-1 using 3% lime in Modified and Standard Proctor test methods are 99.0 lb/cu.ft and 98.2 lb/cu.ft respectively.

From the fig. 4.3A, fig. 4.3B and fig. 4.3C, it can be seen that the optimum moisture content of lime stabilized soil is dependent upon the compactive effort. Optimum moisture content decreases with the increase of compactive effort. Optimum moisture content for soil type-1, using Standard Proctor test method is 13.2 percent and for Modified Proctor test method is

TABLE 4.1 OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY OF LIME STABILIZED SOIL AT DIFFERENT LIME CONTENTS USING STANDARD COMPACTIVE EFFORT.

soil type	nominal lime percent	optimum moisture content (%)	maximum dry density lb/cu.ft. (kN/cu.m)
soil type-1 A-6	0	12.5	102.4 (16.09)
	3	13.2	98.2 (15.46)
	4	13.8	97.6 (15.39)
	5	14.3	97.1 (15.25)
soil type-2 A-7 -6	0	21.0	101.2 (15.91)
	3	22.7	98.0 (15.46)
	4	23.1	97.4 (15.30)
	5	23.6	96.8 (15.21)
soil type-3 A-4	0	18.8	100.7 (15.82)
	3	19.4	98.6 (15.48)
	5	19.8	98.3 (15.44)

Note: Values in parentheses represents the value in SI unit

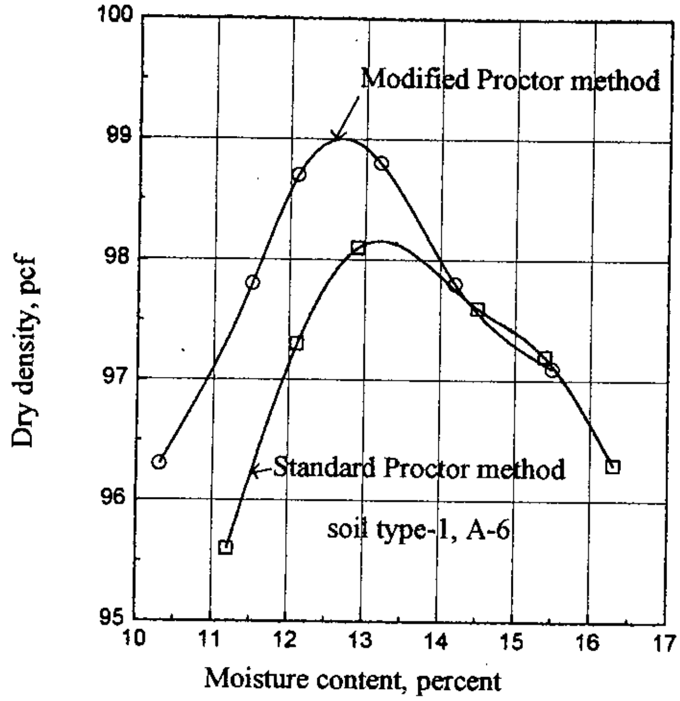


FIG. 4.3A MOISTURE-DENSITY RELATIOPNSHIP FOR STABILIZED SOIL TYPE-1 AT DIFFERENT COMPACTION ENERGY (3% LIME)

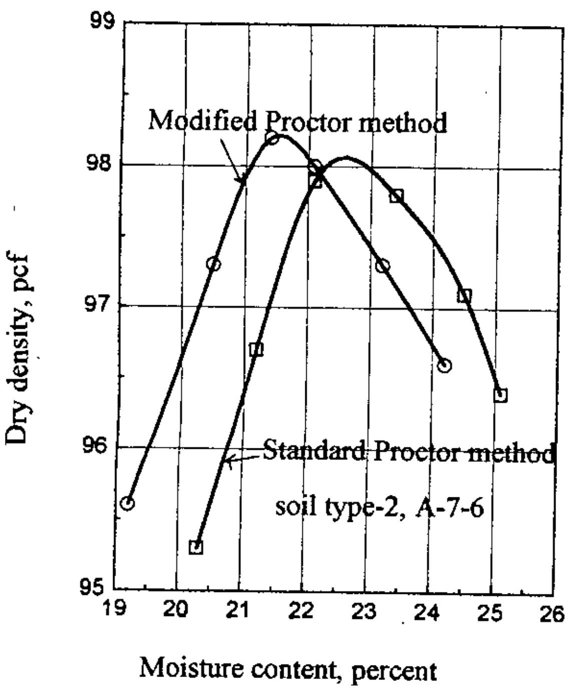


FIG. 4.3B MOISTURE-DENSITY RELATIOPNSHIP FOR STABILIZED SOIL TYPE-2 AT DIFFERENT COMPACTION ENERGY (3% LIME)

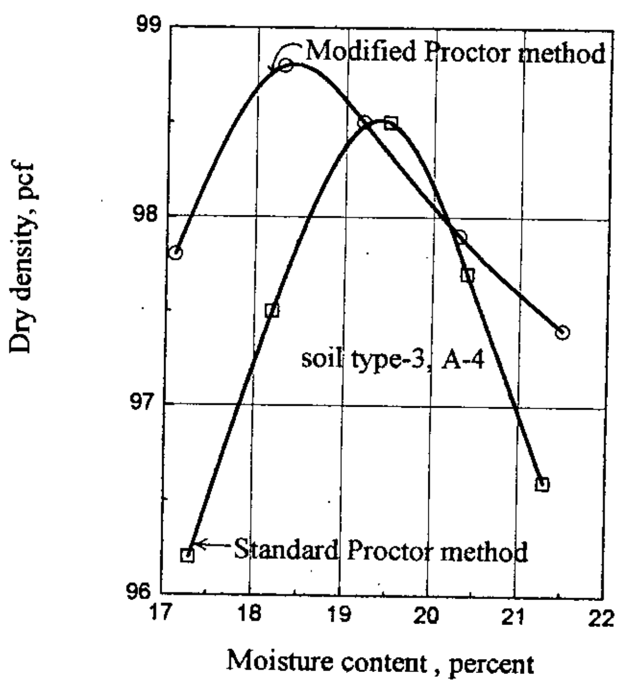


FIG.4.3C MOISTURE- DENSITY RELATIONSHIP FOR STABILIZED SOIL TYPE-3 AT DIFFERENT COMPACTION ENERGY (3% LIME)

12.6 percent. Similar results were obtained for soil type-2 and soil type-3 i.e. optimum moisture content decreases with the increase of compactive effort.

According to Thompson (1966), Compendium (1987), the reason behind the decrease in dry density and increase in optimum moisture content is that due to the addition of lime, structure of the soil changes. The change in structure of the soil causes the change on its other properties.

According to Faisal et al. (1992), the reason behind the decrease in maximum dry density of the lime stabilized soil is the reflection of increased resistance offered by the flocculated soil structure. The increase in optimum moisture content is probably the consequence of the additional water held within the fluctuated soil structure resulting from lime interaction with soil.

4.3.2 Effect of moulding water content on lime stabilized soil

To study the effect of moulding water content on the strength of lime treated soil, a number of unconfined compressive strength test specimen was prepared at different water content and at different lime content. Standard Proctor test energy was applied to prepare the specimen. The unconfined compressive strength and dry density at different water contents with varying lime content for three types of soil is shown in Table 4.2A, Table 4.2B and Table 4.2C.

The results of the unconfined compressive strength at different moisture content for untreated soil is shown in fig 4.4A, fig 4.4B and fig 4.4C. Unconfined compressive strength on untreated soil was performed on soil specimen compacted at three moisture content near to the optimum moisture content of soil. From the figures, it can be observed that the unconfined compressive strength of the sample with minimum water content is least.

Unconfined compressive strength of the stabilized soil using different moisture content are plotted in fig 4.5A, fig 4.5B and fig 4.5C. From the figures, it can be observed that the unconfined compressive strength of the stabilized soil increases with the moisture content to a certain maximum value after that the strength starts decreasing.

TABLE 4.2A DRY DENSITY AND UNCONFINED COMPRESSIVE STRENGTH (q_u)OF THE LIME STABILIZED SOIL AT DIFFERENT MOISTURE CONTENTS USING STANDARD COMPACTIVE EFFORT (SOIL TYPE-1, A-6)

lime content percent	water content percent	dry density lb/cu.ft (kN/cu.m)	qu, psi (kPa)
0	10.2	98.2 (15.43)	
	11.2	100.5 (15.80)	10.5 (72.4)
	11.9	102.0 (16.03)	10.8 (74.47)
	13.1	101.6 (15.96)	11.0 (775.85)
	14.4	99.5 (15.63)	
	15.3	98.6 (15.49)	-
3	11.2	95.6 (15.02)	-
	12.1	97.3 (15.28)	48.3 (335.8)
	12.9	98.1 (15.41)	49.7 (342.7)
	14.5	97.8 (15.37)	51.2 (353.05)
	15.4	97.4 (15.30)	49.7 (349.56)
	16.3	96.6 (15.19)	48.2 (332.36)
4	11.1	94.9 (14.91)	-
	12.3	96.2 (15.1)	50.3 (346.84)
	12.9	96.8 (15.21)	-
	14.2	97.5 (15.32)	52.8 (364.08)
	15.5	96.7 (15.19)	52.0 (358.86)
	16.6	95.6 (15.02)	48.8 (336.5)
5	12.2	95.0 (14.93)	53.7 (370.28)
	13.4	96.2 (15.11)	-
	14.5	97.0 (15.24)	55.2 (380.63)
	15.1	96.7 (15.19)	54.9 (378.55)
	16.3	95.5 (15.0)	53.5 (368.2)

Note: Value in parentheses represents the value in SI unit

TABLE 4.2B DRY DENSITY AND UNCONFINED COMPRESSIVE STRENGTH (q_u) OF THE LIME STABILIZED SOIL AT DIFFERENT MOISTURE CONTENTS USING STANDARD COMPACTIVE EFFORT (SOIL TYPE-2, A-7-6)

lime content percent	water content percent	dry density lb/cu.ft (kN/cu.m)	q_u, psi (kPa)
0	19.2	98.2 (15.43)	
	20.3	100.7 (15.82)	15.9 (109.63)
	20.9	101.1 (15.88)	16.5 (114.77)
	21.8	100.6 (15.80)	16.8 (115.84)
	23.2	99.5 (15.63)	
	24.2	98.7 (15.51)	-
3	20.3	95.3 (14.97)	-
	21.2	96.7 (15.19)	83.3 (574.4)
	22.1	97.9 (15.38)	84.6 (583.4)
	23.4	97.8 (15.37)	85.5 (589.56)
	24.5	97.1 (15.26)	83.8 (577.78)
	25.1	96.4 (15.15)	82.7 (570.25)
4	20.5	94.6 (14.86)	-
	21.3	95.7 (15.04)	87.3 (601.97)
	22.4	96.6 (15.18)	-
	23.3	97.1 (15.26)	89.4 (616.45)
	24.6	96.4 (15.15)	88.8 (612.3)
	25.2	95.7 (15.04)	87.4 (602.66)
5	21.1	94.3 (14.82)	91.3 (629.55)
	22.3	95.7 (15.04)	-
	23.1	96.7 (15.19)	94.0 (648.17)
	24.2	96.8 (15.21)	94.3 (650.24)
	25.4	95.3 (14.97)	90.5 (624.04)

Note: Value in parentheses represents the value in SI unit

TABLE 4.2C DRY DENSITY AND UNCONFINED COMPRESSIVE STRENGTH (q_u) OF THE LIME STABILIZED SOIL AT DIFFERENT MOISTURE CONTENTS USING STANDARD COMPACTIVE EFFORT (SOIL TYPE-3, A-4)

lime content percent	water content percent	dry density lb/cu.ft (kN/cu.m)	q_u, psi (kPa)
0	15.3	98.1 (15.41)	-
	17.4	99.8 (15.68)	4.8 (33.01)
	19.1	100.6 (15.81)	5.6 (38.61)
	20.9	99.6 (15.65)	5.7 (39.07)
	23.5	97.8 (15.37)	
3	17.3	96.2 (15.11)	-
	18.2	97.5 (15.32)	14.75 (101.71)
	19.5	98.5 (15.47)	15.2 (104.81)
	20.4	97.7 (15.35)	15.6 (107.57)
	21.3	96.6 (15.18)	14.7 (101.35)
4	17.5	96.1 (15.1)	-
	18.3	97.2 (15.27)	16.32 (112.53)
	18.9	97.5 (15.32)	16.8 (115.84)
	20.5	98.1 (15.41)	17.1 (117.91)
	21.5	96.8 (15.2)	16.4 (113.08)

Note: Value in parentheses represents the value in SI unit

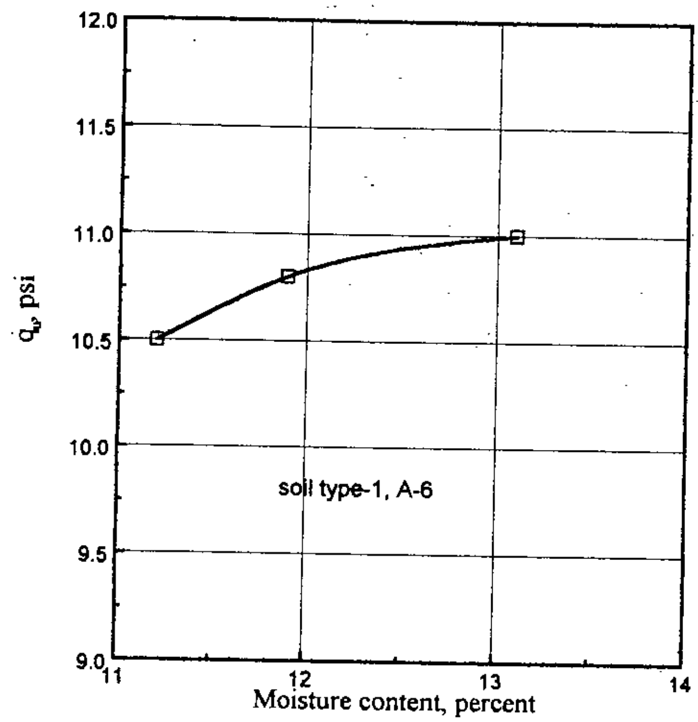


FIG.4.4A UNCONFINED COMPRESSIVE STRENGTH (q_u) OF SOIL TYPE-1 AT DIFFERENT MOISTURE CONTENT USING STANDARD PROCTOR MEHOD

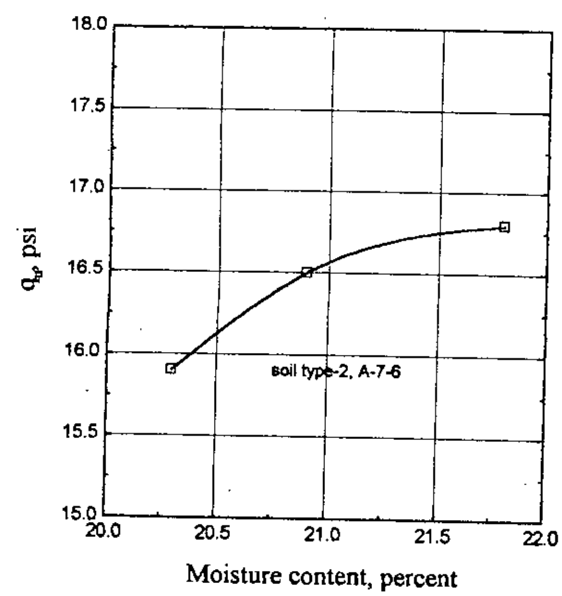


FIG.4.4B UNCONFINED COMPRESSIVE STRENGTH (q_u) OF SOIL TYPE-2 AT DIFFERENT MOISTURE CONTENT USING STANDARD PROCTOR METHOD

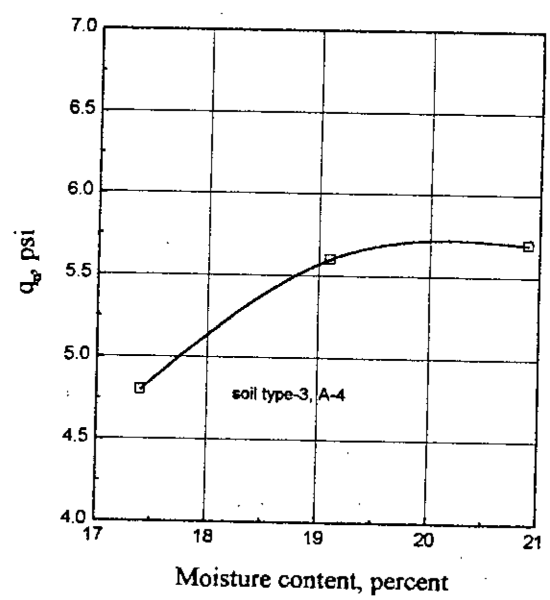


FIG.4.4C UNCONFINED COMPRESSIVE STRENGTH (q_u) OF SOIL TYPE-3 AT DIFFERENT MOISTURE CONTENT USING STANDARD PROCTOR METHOD

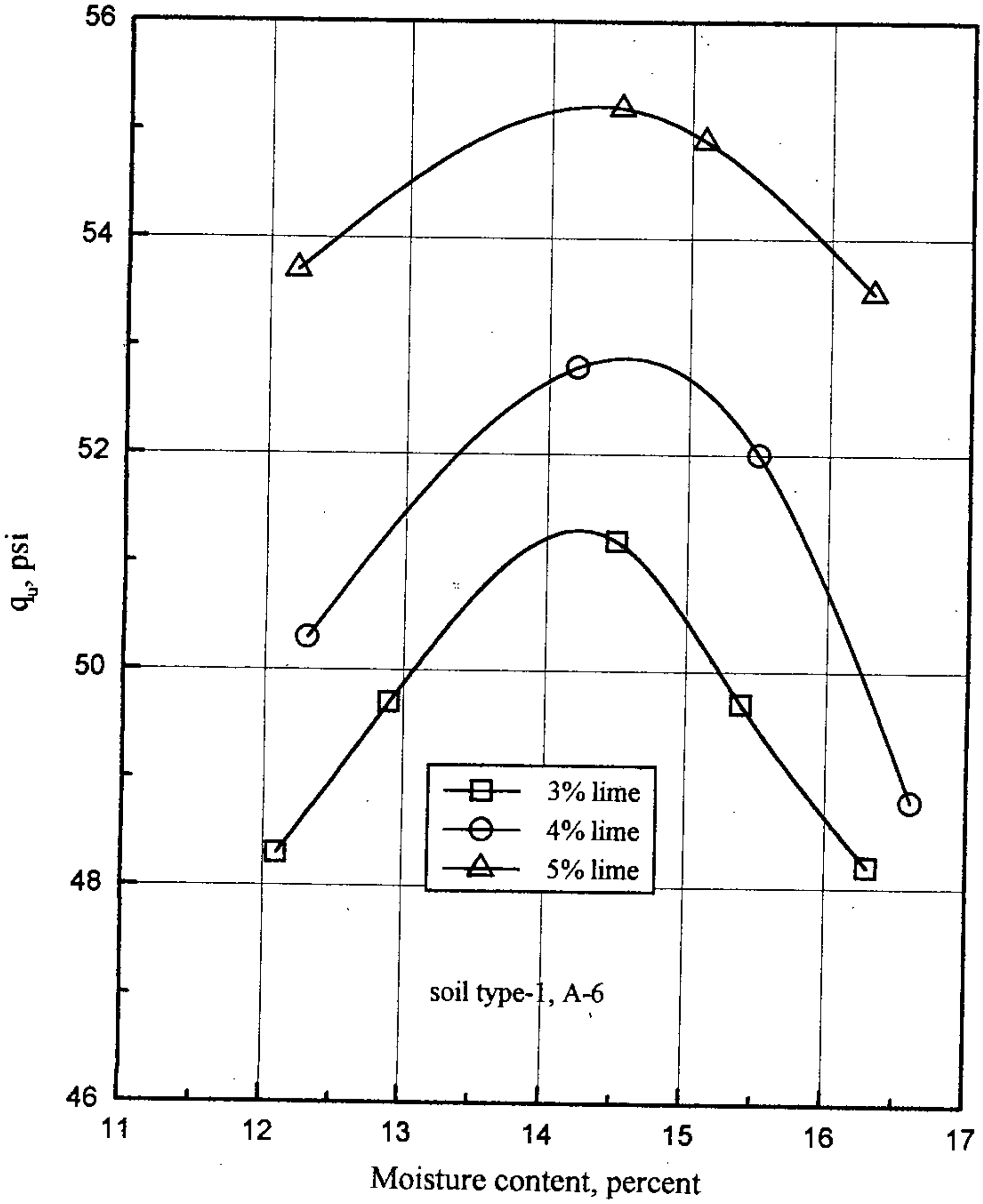


FIG. 4.5A MOISTURE -UNCONFINED COMPRESSIVE STRENGTH RELATIONSHIP FOR THE STABILIZED SOIL TYPE-1 AT DIFFERENT LIME CONTENT (COMPACTED USING STANDARD PROCTOR METHOD)

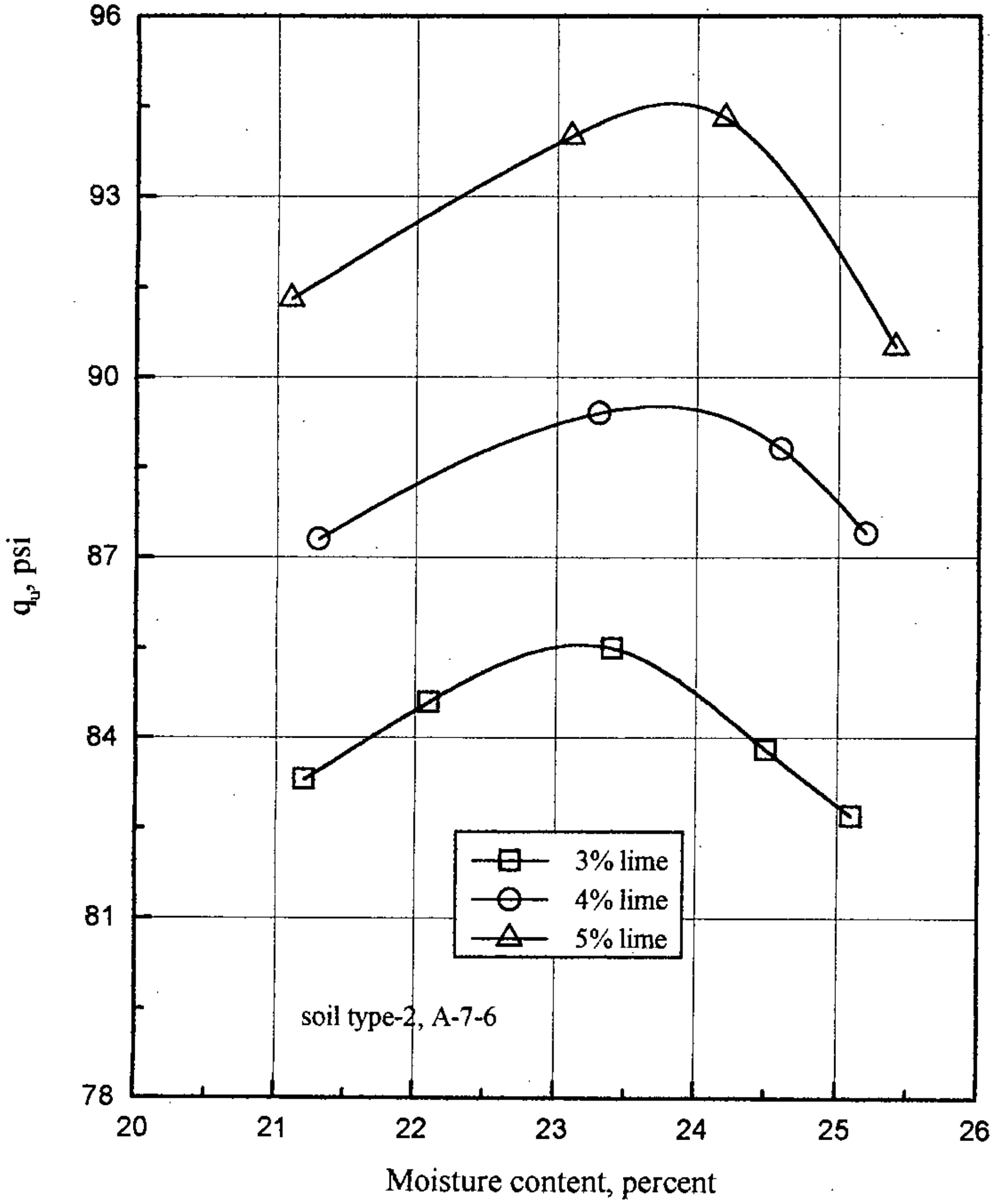


FIG. 4.5B MOISTURE -UNCONFINED COMPRESSIVE STRENGTH (q_u) RELATIONSHIP FOR THE STABILIZED SOIL TYPE-2 AT DIFFERENT LIME CONTENT (COMPACTED USING STANDARD PROCTOR METHOD)

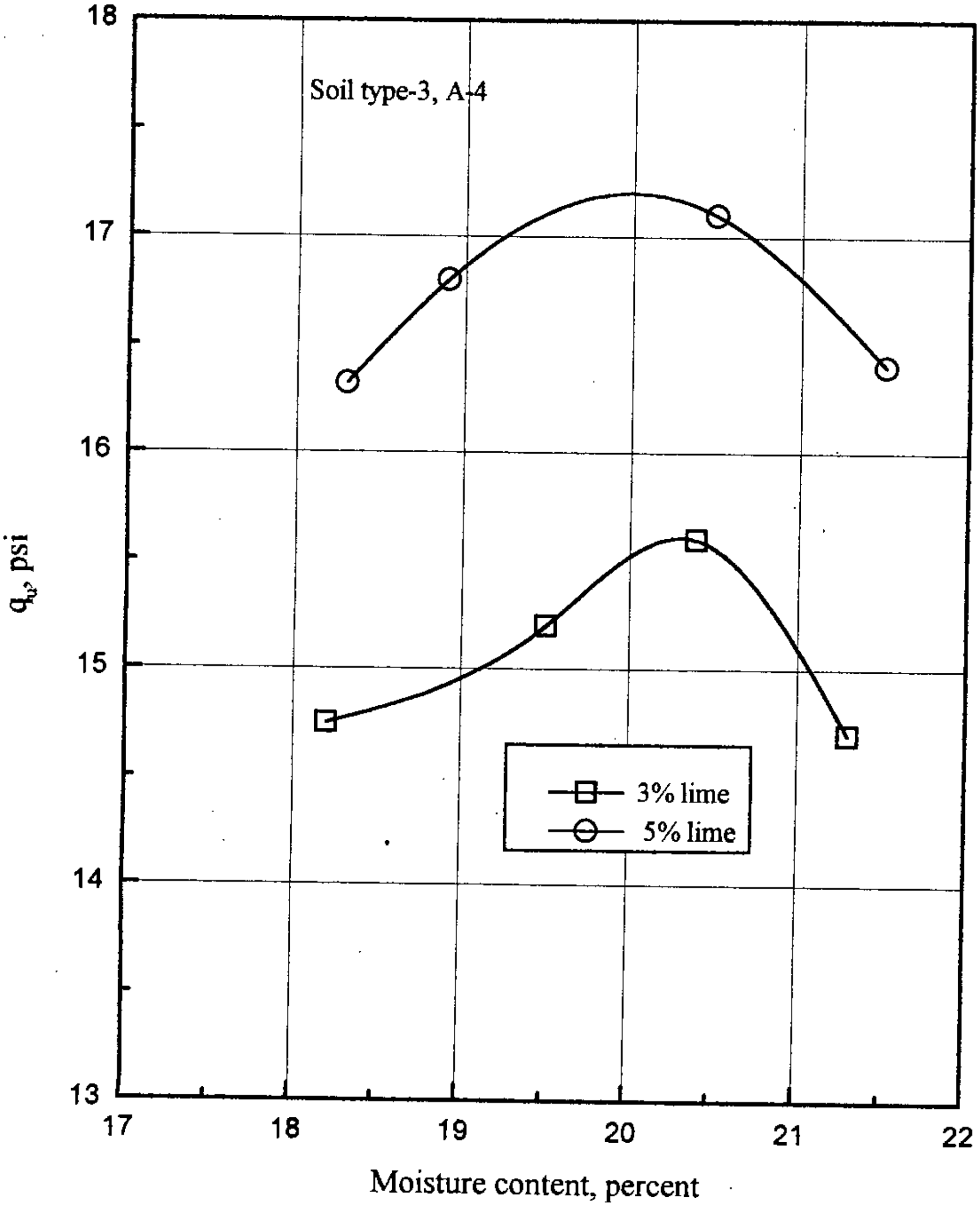


FIG. 4.5C MOISTURE-UNCONFINED COMPRESSIVE STRENGTH RELATIONSHIP FOR THE STABILIZED SOIL TYPE-3 AT DIFFERENT LIME CONTENT (COMPACTED USING STANDARD PROCTOR METHOD)

To observe the effect of moisture content on unconfined compressive strength and dry density, the value of dry density and unconfined compressive strength are plotted simultaneously with respect to moisture content and are shown in fig 4.6A, fig 4.6B and fig 4.6C. It can be observed from figures that the required moisture content to attain maximum dry density and maximum unconfined compressive strength of a soil is different.. Higher moisture content is necessary to gain maximum unconfined compressive strength than the moisture content needed to gain maximum dry density. However, the difference is not that significant.

The maximum value of the unconfined compressive strength, the value of the unconfined compressive strength at the optimum moisture content of the soil lime mix and value of the unconfined compressive strength at the optimum moisture of soil was as obtained from fig 4.5A, fig 4.5B and fig 4.5C, are tabulated in table 4.3. From the table, it can be observed that the value of the unconfined compressive strength vary for the three types of moisture content but the differences are not so significant.

Ali (1980) performed undrained triaxial test on compacted soil at lower and upper side of optimum moisture content of soil. He observed that at low confining pressure, axial strain at failure is higher on the wet side of the optimum moisture of soil rather than the moisture content below the optimum moisture content. However, he found that the reduction in ultimate strength do take place in wet side sample but it is small compared to the reduction in the dry side specimen of untreated soil

Hansmann (1990) reported that maximum strength occurs at higher moisture content than the optimum moisture content of soil when soil is stabilized with cement.

Rainy season exist at about half of the total year of Bangladesh. As a result, soil of the most of the area remain wet. Wet soil having clay particle are very difficult to work with. The optimum moisture content of the soil increases due to the addition of lime and also the strength corresponding to the maximum strength is higher than the optimum moisture content of the stabilized soil. So it may be easier to compact the soil with lime in wet season.

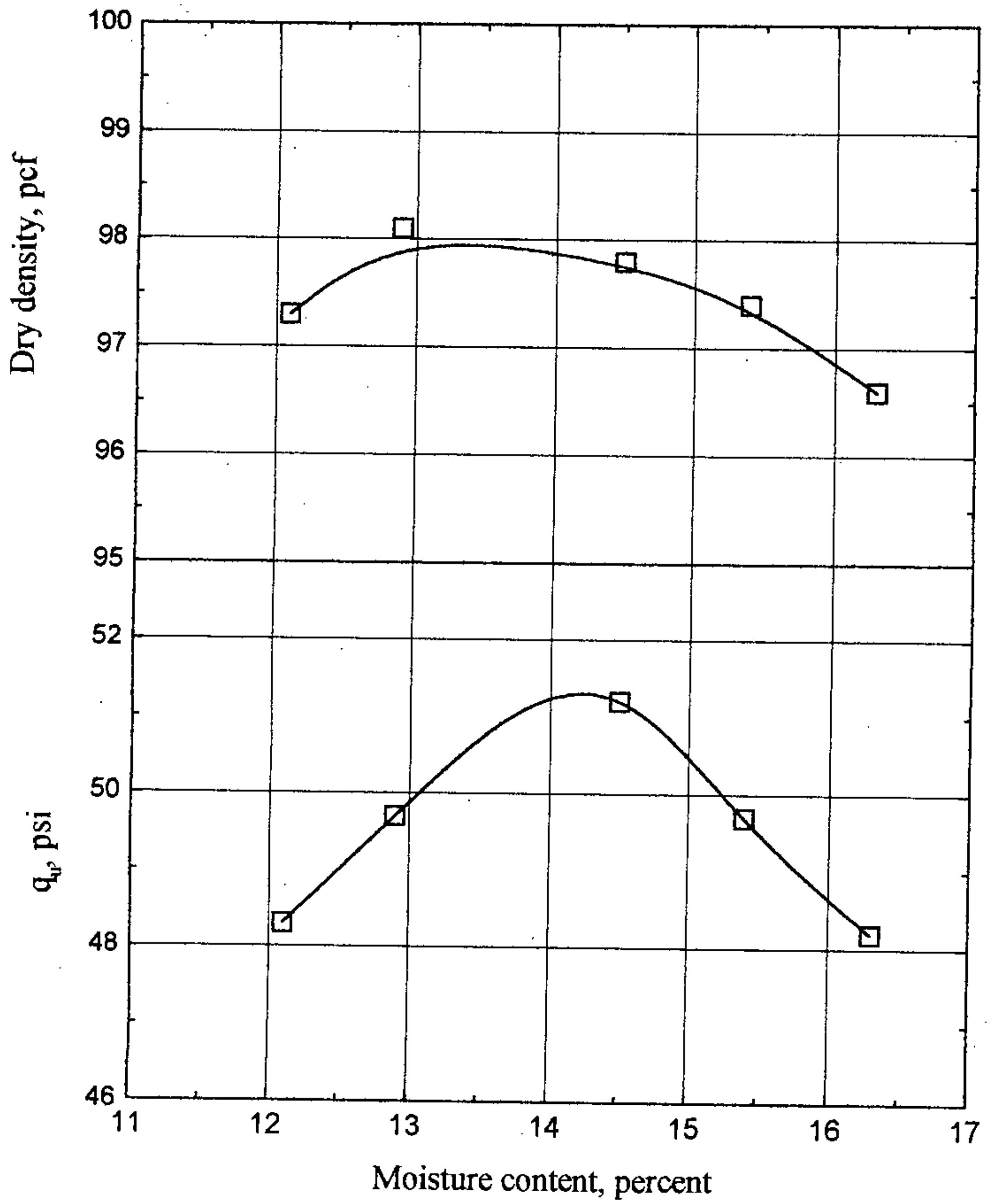


FIG. 4.6A UNCONFINED COMPRESSIVE STRENGTH (q_u) AND DRY DENSITY AT DIFFERENT MOISTURE CONTENT FOR STABILIZED SOIL TYPE-1 (3% LIME, COMPACTED BY STANDARD PROCTOR METHOD)

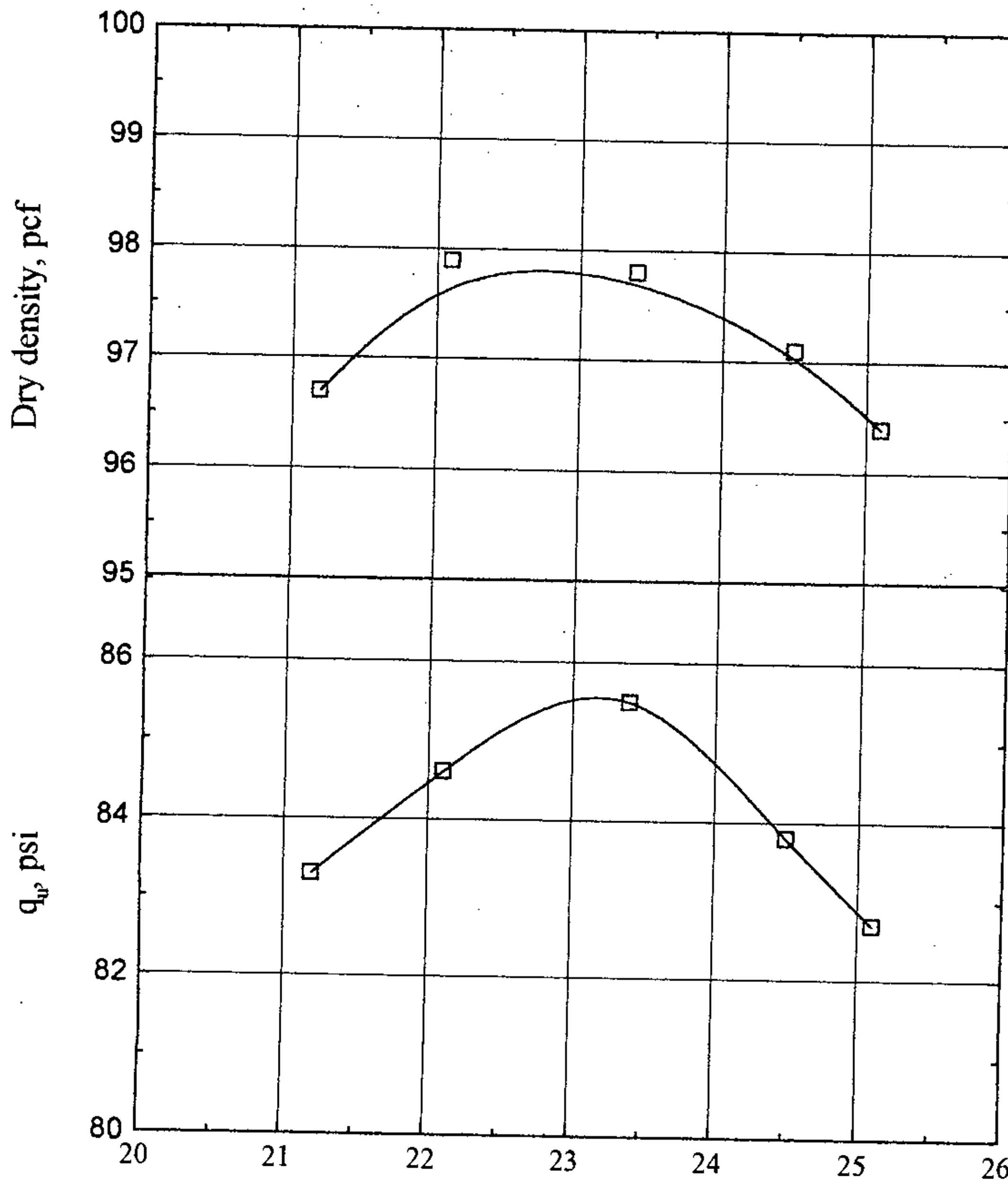


FIG. 4.6B UNCONFINED COMPRESSIVE STRENGTH AND DRY DENSITY AT DIFFERENT MOISTURE CONTENT FOR STABILIZED SOIL TYPE-2 (3% LIME) (COMPACTED BY STANDARD PROCTOR METHOD)

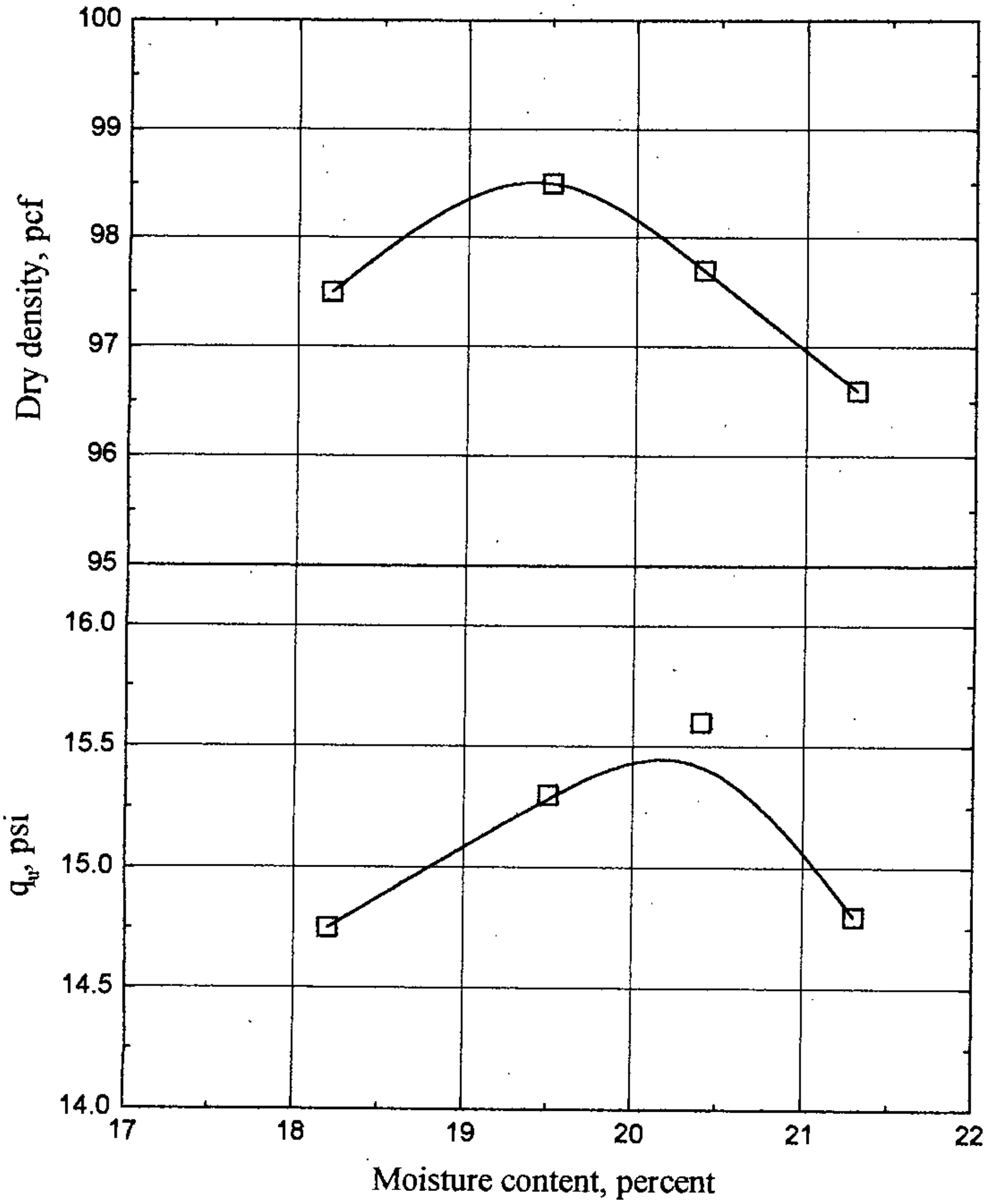


FIG. 4.6C UNCONFINED COMPRESSIVE STRENGTH AND DRY DENSITY AT DIFFERENT MOISTURE CONTENT FOR STABILIZED (3% LIME) SOIL TYPE-3 (COMPACTED BY STANDARD PROCTOR METHOD)

TABLE 4.3 UNCONFINED COMPRESSIVE STRENGTH (q_u) AND DRY DENSITY AT OPTIMUM MOISTURE CONTENT OF SOIL, OPTIMUM MOISTURE CONTENT OF SOIL LIME MIX, AND AT THE MAXIMUM VALUE OF q_u

soil type	lime %	at ω_{opt} of soil			at ω_{op} of soil-lime mix			at the maximum value of q_u		
		water content %	* q_u, psi (kPa)	dry density lb/cu.ft (kN/cu.m)	water cont. %	* q_u, psi (kPa)	dry density lb/cu.ft (kN/cu.m)	water cont. %	* q_u, psi (kPa)	dry density lb/cu.ft (kN/cu.m)
soil type-1 A-6	3	12.5	48.8 (336.5)	97.8 (15.38)	13.2	50.2 (347.5)	98.2 (15.43)	14.1	51.4 (354.43)	97.9 (15.38)
	4	12.5	50.4 (347.5)	96.6 (15.18)	13.8	52.5 (363.4)	97.6 (15.33)	14.4	52.8 (364.26)	97.3 (15.29)
	5	12.5	53.7 (370.3)	95.5 (15.0)	14.3	55.0 (379.25)	97.0 (15.24)	14.7	55.2 (380.63)	96.7 (15.19)
soil type-2 A-7-6	3	21.0	82.9 (571.63)	94.2 (14.8)	22.7	85.2 (587.5)	97.7 (15.35)	23.3	85.6 (590.25)	97.5 (15.32)
	4	21.0	86.5 (598.8)	95.4 (14.99)	23.1	89.7 (618.5)	97.2 (15.27)	23.6	90.0 (620.6)	96.8 (15.21)
	5	21.0	90.9 (626.8)	96.3 (15.13)	23.6	94.6 (654.95)	96.6 (15.8)	23.8	95.0 (655.06)	96.5 (15.16)
soil type-3 A-4	3	18.8	15.3 (105.5)	97.6 (15.33)	19.4	15.5 (106.88)	98.7 (15.51)	19.9	15.7 (108.08)	98.4 (15.46)
	5	18.8	16.7 (115.62)	98.1 (15.41)	19.8	17.2 (118.6)	98.3 (15.44)	20.3	17.4 (120.0)	98.1 (15.41)

Notes: value in parentheses represents the value in SI unit

* value of q_u was determined from fig. 4.5A, fig. 4.5B and fig. 4.5C

The stress-strain curves of both the untreated and stabilized soil are plotted in fig. 4.7A, fig. 4.7B and fig 4.7C for different moisture contents.

From the figures, it is observed that the lime stabilized soil is much more stiffer than untreated soil. Due to the addition of lime to the soil, structure of the soil is changed. It can also be observed from the figures that the stress-strain curve of the stabilized soil is not significantly different for different moisture contents.

The modulus of elasticity of the soil corresponding to 50% peak stress (E_{50}) on the three type of soil using different lime content are presented in Table 4.4. From the table, it can be observed that the modulus of elasticity increases significantly with the addition of lime. The variation of (E_{50}) for soil stabilized with 3% , 4%, and 5% lime content is very small.

4.3.3 Effect of lime content *on lime stabilized soil*

Lime percent is considered as an important factor for lime stabilization from the very beginning of the history of lime stabilization and investigators have been working on it since the beginning of the theory of lime stabilization. Hilt and Davidsion (1960), Eades and Grim (1960) suggested to use 2% to 8% lime for stabilization.

Optimum moisture content increases due to the addition of lime to the soil. Optimum moisture content continues to increase due to further addition of lime. The relationship between optimum water content and lime content for three different soil is shown in fig 4.8. The relationship between maximum dry density and lime content are presented by fig 4.9. It can be observed from the figure that the maximum dry density of the lime stabilized soil reduces due to the addition of lime and the dry density continues to decrease due to the increase of lime content for the three types of soil. Compendium (1987) found similar result from their study. Due to the increase of lime content flocculation of the soil particles increases, resulting decrease of dry density.

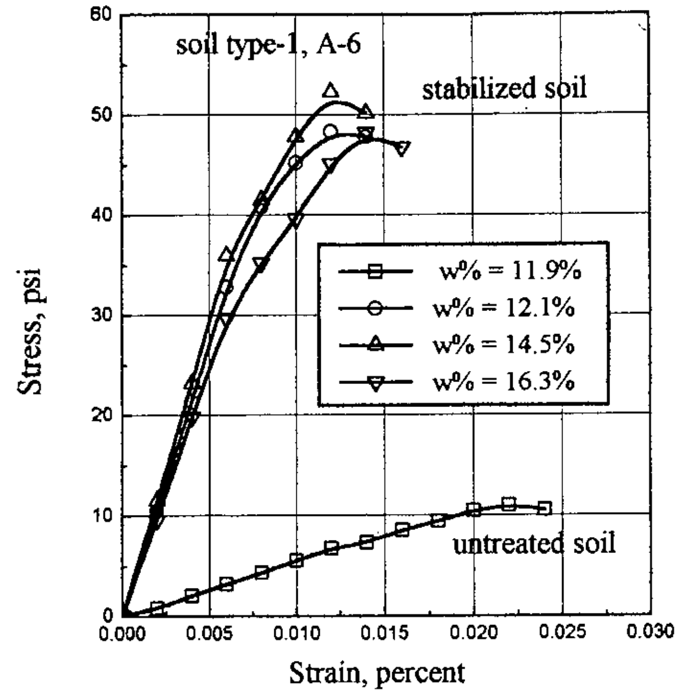


FIG 4.7A STRESS-STRAIN RELATIONSHIP OF UNTREATED AND STABILIZED (3% LIME) SOIL TYPE-1

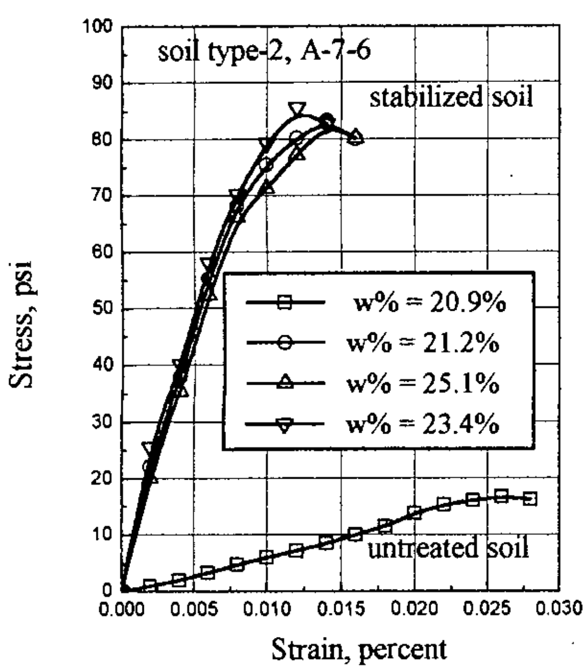


FIG 4.7B STRESS-STRAIN RELATIONSHIP OF UNTREATED AND STABILIZED (3% LIME) SOIL TYPE-2

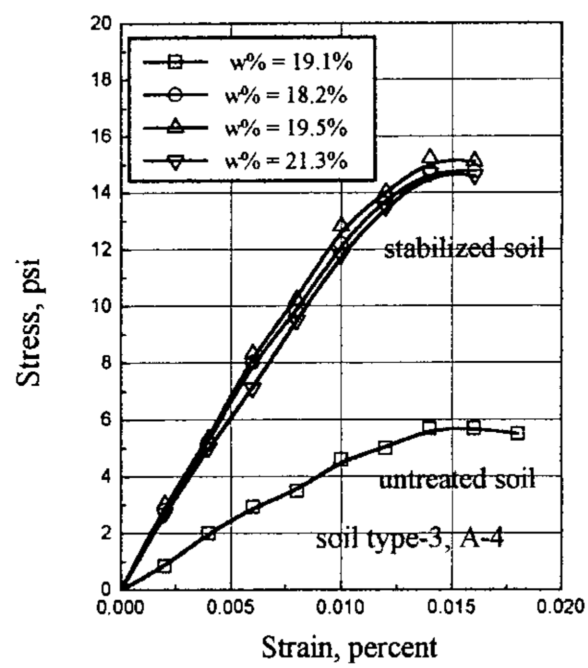


FIG 4.7C STRESS-STRAIN RELATIONSHIP OF UNTREATED AND STABILIZED (3% LIME) SOIL TYPE-3

TABLE 4.4 SECOND MODULOUS OF ELASTICITY (E_{50}) OF THE STABILIZED SOIL

soil type	lime percent	water content percent	E_{50} psi (kPa)	q_u, psi (kPa)	maximum value of q_u, psi (kPa) at corrosponding lime content*
soil type-1 A-6	0	11.9	550 ($3.79x10^3$)	10.8 (74.47)	11.1 (76.45)
	3	14.5	5,550 ($38.29x10^3$)	51.2 (353.05)	51.4 (354.43)
	4	14.2	5,750 ($39.63x10^3$)	52.8 (570.94)	52.8 (364.26)
	5	14.5	6,000 ($41.35x10^3$)	55.1 (379.93)	55.2 (380.63)
soil type-2 A-7-6	0	20.9	560 ($3.85x10^3$)	16.8 (115.15)	16.7 (115.15)
	3	23.4	8,500 ($58.57x10^3$)	85.5 (589.56)	85.6 (590.25)
	4	23.3	8,900 ($61.34x10^3$)	89.7 (618.52)	90.0 (620.6)
	5	23.1	9,350 ($64.45x10^3$)	94.0 (648.14)	95.0 (655.06)
soil type-3 A-4	0	19.1	407 ($2.8x10^3$)	5.6 (38.61)	5.7 (39.3)
	3	19.2	1250 ($8.61x10^3$)	15.4 (106.2)	15.5 (106.88)
	5	18.9	1380 ($9.51x10^3$)	16.8 (115.84)	17.4 (120.0)

Notes: Value in parentheses represents the value in SI unit

* Value of q_u was calculated from fig 4.5A, fig. 4.5B and fig. 4.5C

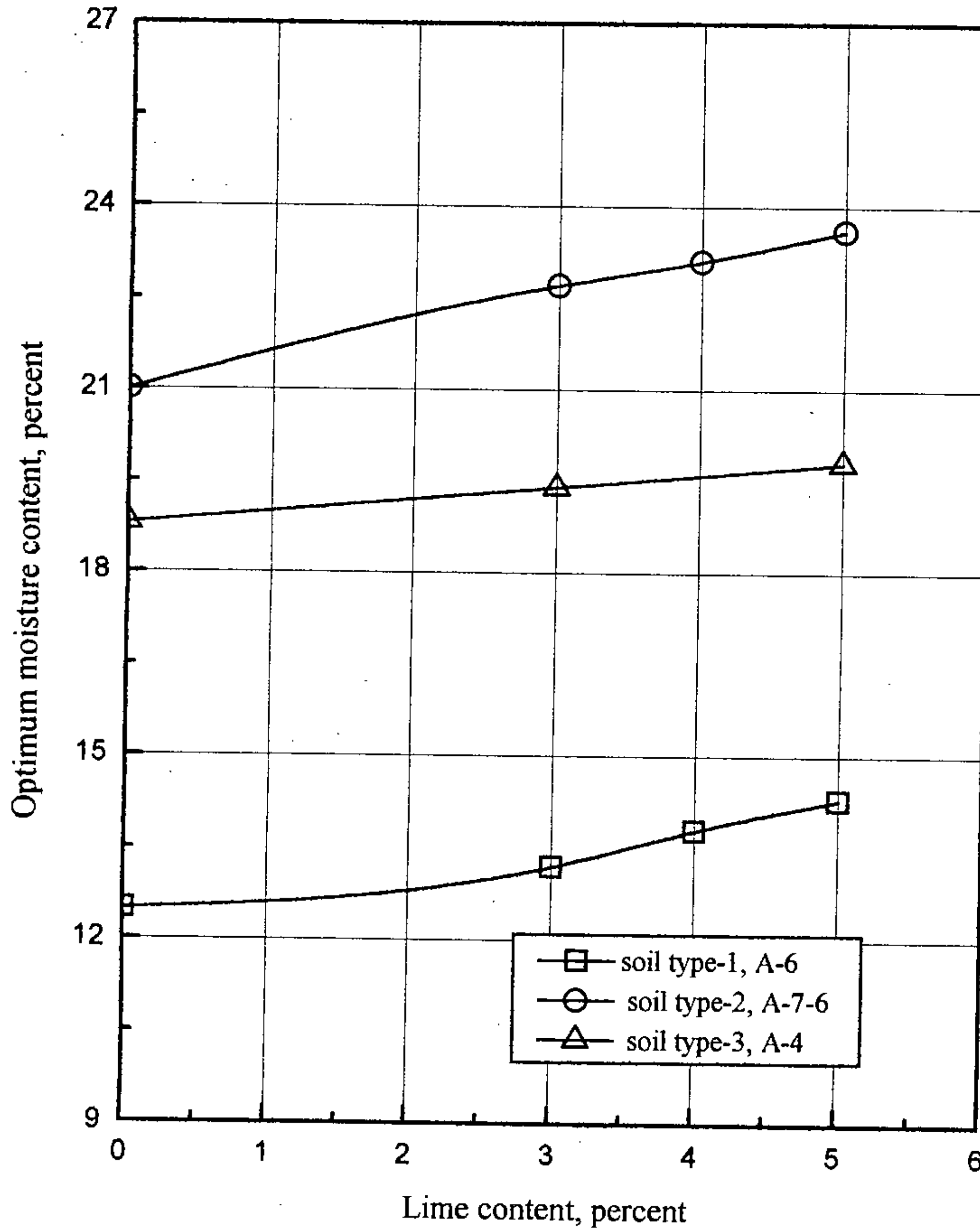


FIG 4.8 RELATIONSHIP BETWEEN OPTIMUM MOISTURE CONTENT AND LIME CONTENT (COMPACTING BY STANDARD PROCTOR METHOD)

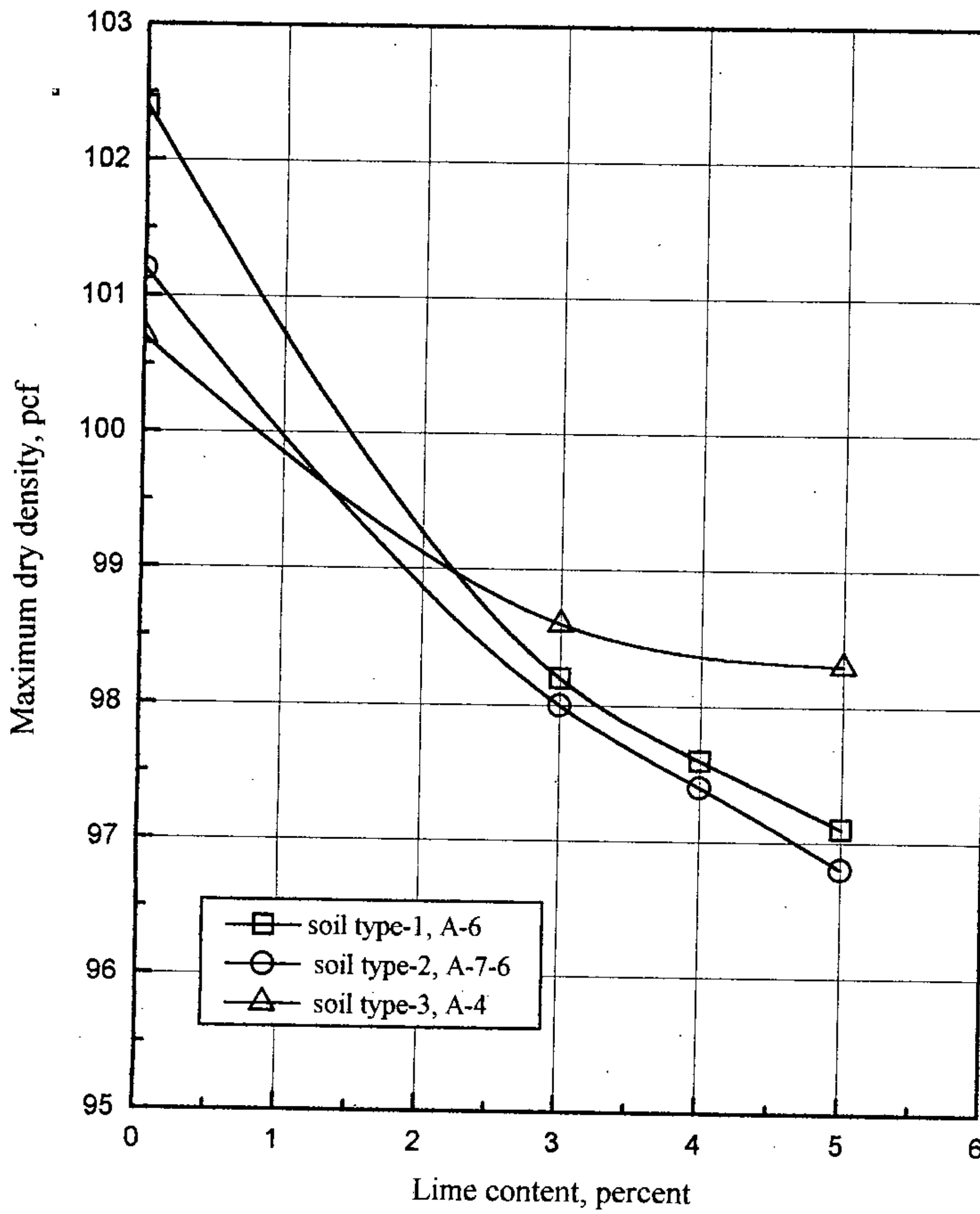


FIG 4.9 RELATIONSHIP BETWEEN MAXIMUM DRY DENSITY AND LIME CONTENT (COMPACTING BY STANDARD PROCTOR METHOD)

To observe the effect of lime content on the unconfined compressive strength of the soil, test specimen was prepared compacting the soil sample using Standard Proctor test method at different moisture contents using 3%, 4% and 5% lime. Values of the unconfined compressive strength at different moisture content is shown in figure fig 4.5A, fig 4.5B and fig 4.5C. The value of q_u at the optimum moisture content of soil lime mix was estimated from these figures and are tabulated in table 4.5. The unconfined compressive strengths due to the addition of 3%, 4% and 5% lime, at the optimum moisture content of the soil are shown in the fig. 4.10.

From the fig. 4.10, it can be seen that there is a marked increase of unconfined compressive strength with the addition of the 3% lime for all the three type of soil. However, it should be kept in mind that the dry density decreases due to the addition of lime. Unconfined compressive strength value is about 4.6 times higher than the untreated soil sample due to the addition of three percent lime for soil type-1. The value of unconfined compressive strength of the untreated soil is 11 psi. For the soil type-2, unconfined compressive strength increased about 5.2 times higher than the untreated soil using 3% lime, the value of the unconfined compressive strength of the untreated soil is 16.7 psi. The increase of strength is about 2.7 times higher from the untreated value of 5.7 psi for the soil type-3.

From the fig. 4.10, it can be seen that the increase of unconfined compressive strength due to the addition of lime has marked effect upon the soil type. At 3% lime content, the unconfined compressive strength of soil type-2 is 85.2 psi at the optimum moisture content of the soil lime mix, for soil type-1 is 49.7 psi at the optimum moisture content of the soil lime mix and for soil type-3 the value is 15.2 psi which indicates that the soil type-1 is more suitable for lime stabilization than soil type-2 and soil type-3.

According to Thompson (1966), Eades and Grim (1960), Broms (1984), the reason behind higher increase in strength for soil type-2 is that large amount of clay particle in soil type-2 than the soil type-1 or soil type-3. Additionally, it can be observed that the soil type-3 has very low strength increase which indicates that ML type of soil is not effective for lime stabilization.

TABLE 4.5 RATIOS OF UNCONFINED COMPRESSIVE STRENGTHS (q_u) OF LIME STABILIZED AND UNTREATED SOILS AT OPTIMUM MOISTURE CONTENT OF SOIL

soil type	nominal lime percent	q_u, psi (kPa) at optimum water content	q_u (soil+lime) / q_u (soil)
soil type-1 A-6	0	11.1 (75.85)	-
	3	50.2 (345.46)	4.55
	4	52.5 (363.47)	4.73
	5	55.0 (379.25)	4.95
soil type-2 A-7-6	0	16.7 (115.15)	-
	3	85.2 (587.55)	5.10
	4	89.7 (612.52)	5.37
	5	94.6 (652.3)	5.66
soil type-3 A-4	0	5.7 (39.3)	-
	3	15.5 (106.88)	2.71
	5	17.2 (118.6)	3.01

Notes: Value in parentheses represents the value in SI unit

* The value of q_u was determined from fig. 4.5A, 4.5B and 4.5C

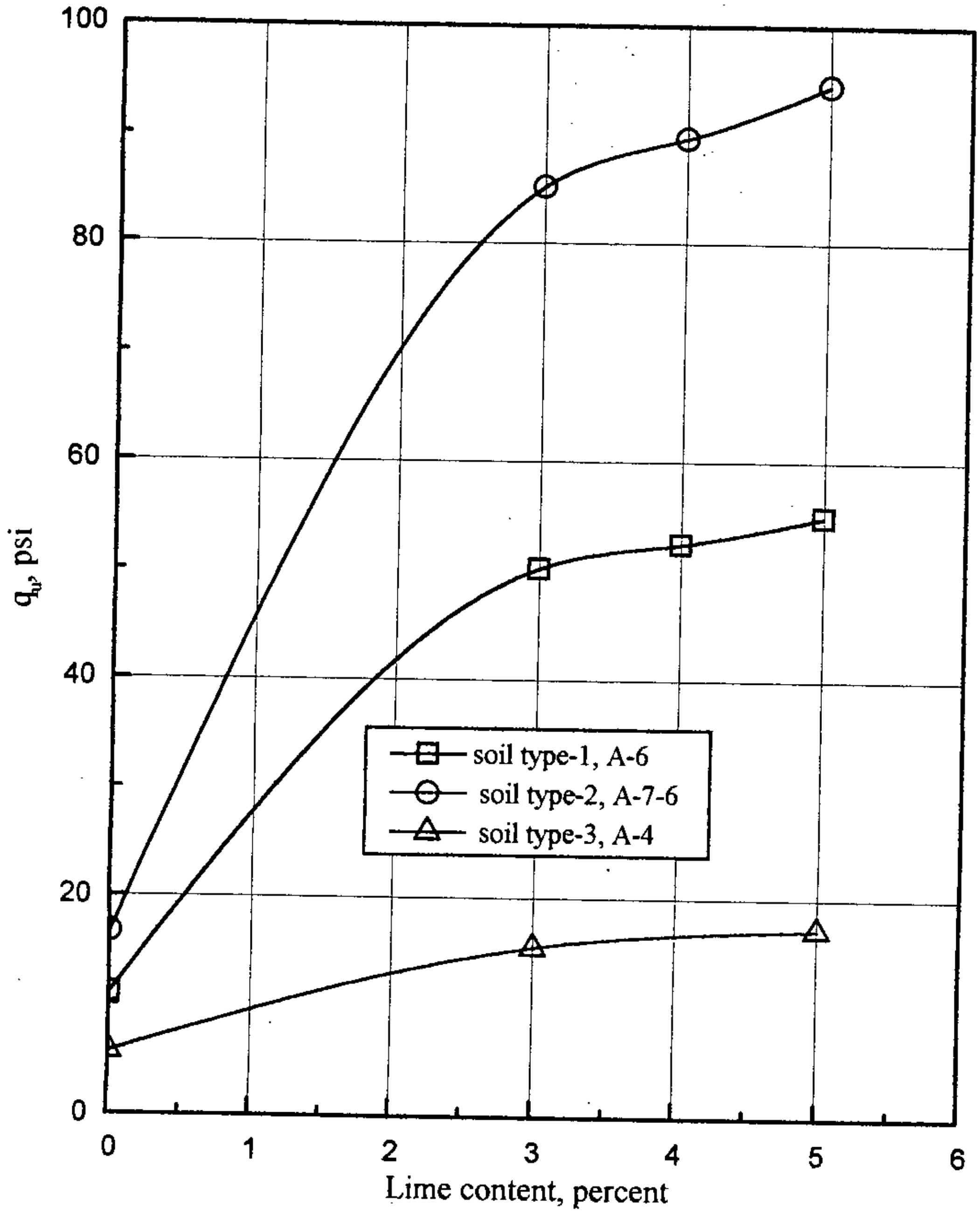


FIG 4.10 RELATIONSHIP BETWEEN UNCONFINED COMPRESSIVE STRENGTH (q_u) AND LIME CONTENT (COMPACTING USING STANDARD PROCTOR METHOD)

The maximum value of unconfined compressive strength, unconfined compressive strength at optimum moisture content of the soil, unconfined compressive strength at optimum moisture content of soil-lime mix are presented in Table 4.3. The increase of unconfined compressive strength due to lime content at three different moisture contents (i.e. at optimum moisture content of soil, optimum moisture content of soil lime mix and the moisture content at which the maximum q_u occur) are presented in fig 4.11. From the figure, it is seen that the difference between maximum strength, strength at optimum moisture content of soil-lime mix and strength at the optimum moisture content of the soil is insignificant.

Effect of lime content on CBR value

To observe the effect of lime content on the strength of the lime stabilized soil in soaked and confined condition, CBR test was performed. For the CBR test, specimen were prepared near the optimum moisture content of the soil lime mix having 3%, 4% and 5% lime content. The lime soil mix was compacted in the CBR compaction mould and CBR test was performed after keeping the sample in soaked condition for four days. The results of the CBR value for different lime contents are shown in Table 4.6. The variation of CBR value due to variation of lime content are also plotted in fig. 4.12 for the three types of soil.

From fig. 4.12, it can be seen that the CBR value of the stabilized soil increases due to the addition of lime similar to the unconfined compressive strengths. Table 4.6 shows that the CBR value is 12 using 3% lime while the value is 3 for untreated soil type-1. For soil type-2, CBR value is 18 using 3% lime content and the value is 2 for untreated soil. For soil type-3, CBR value for the untreated and the stabilized soil are 4 and 7 respectively using 3% lime. The results indicate a marked increase in CBR value for soil type-1 and soil type-2, but for soil type-3, the increase of CBR value is not remarkable.

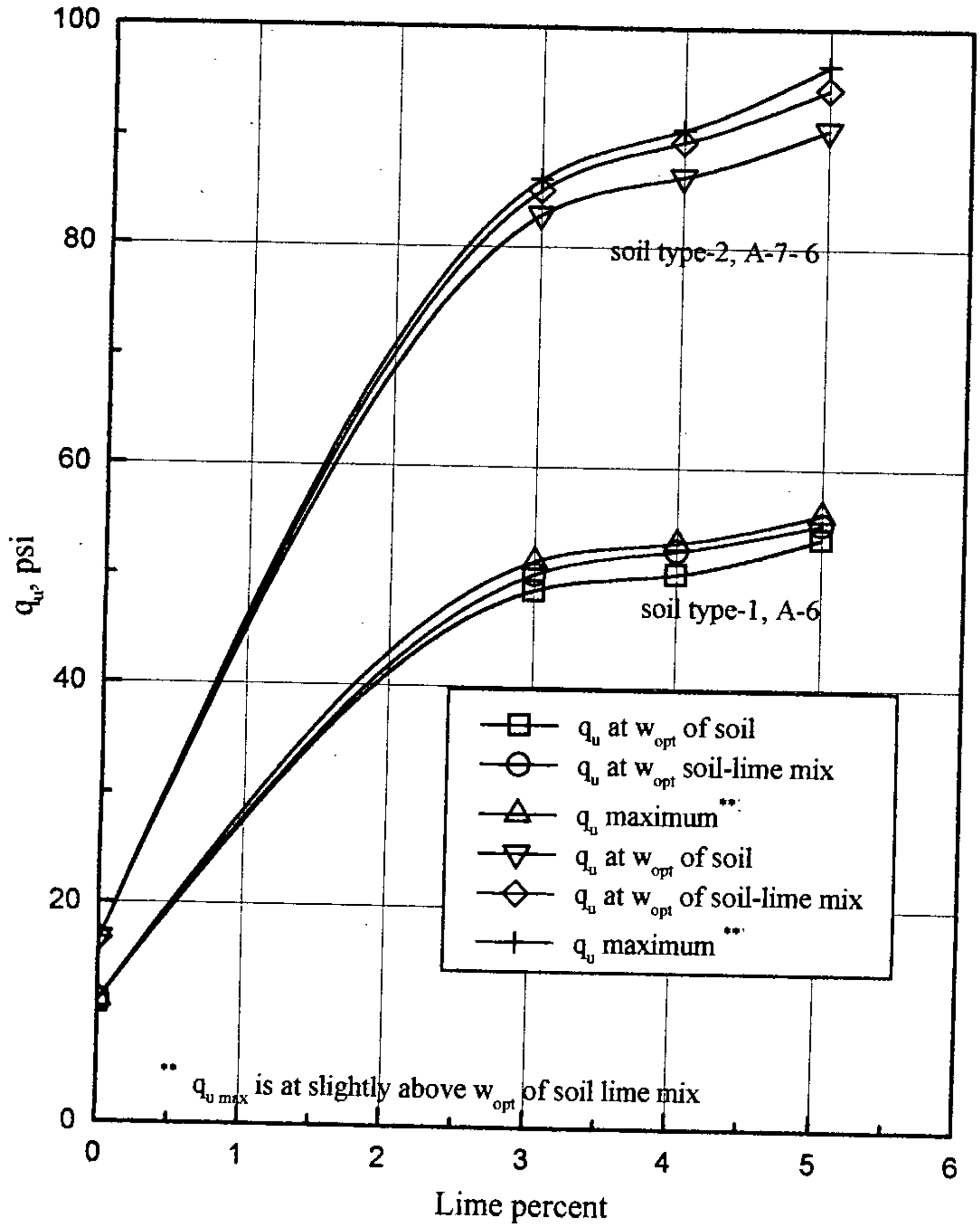


FIG4.11 UNCONFINED COMPRESSIVE STRENGTH (q_u) OF THE SOIL AT DIFFERENT LIME CONTENT WITH THE VARIATION OF MOISTURE CONTENT

TABLE 4.6 CALIFORNIA BEARING RATIO (CBR) VALUE OF THE STABILIZED SOIL AT DIFFERENT LIME CONTENTS

soil type	lime content percent	water content percent	CBR value (4 days)	q_u, psi (kPa) (28 days)*	maximum q_u, psi (kPa) (28 days)*
soil type-1 A-6	0	12.1	3	10.8 (74.42)	11.1 (76.85)
	3	12.9	12	49.8 (343.4)	51.4 (354.43)
	4	13.5	13	52.3 (360.63)	53.0 (365.46)
	5	14.0	14	55.0 (370.25)	55.2 (380.63)
soil type-2 A-7-6	0	21.2	2	16.6 (114.92)	16.7 (115.15)
	3	23.2	18	85.2 (587.5)	85.6 (590.25)
	4	23.4	20	89.0 (613.69)	90.0 (620.65)
	5	23.4	21	94.2 (649.55)	95.0 (655.06)
soil type-3 A-4	0	18.5	4	5.3 (36.54)	5.7 (39.3)
	3	19.3	7	15.4 (106.19)	15.7 (108.25)
	5	19.5	9	17.0 (117.22)	17.4 (120.0)

Notes: Value in parentheses represents the value in SI unit
* Value of q_u was calculated from fig 4.2A, fig. 4.2B and fig. 4.2C

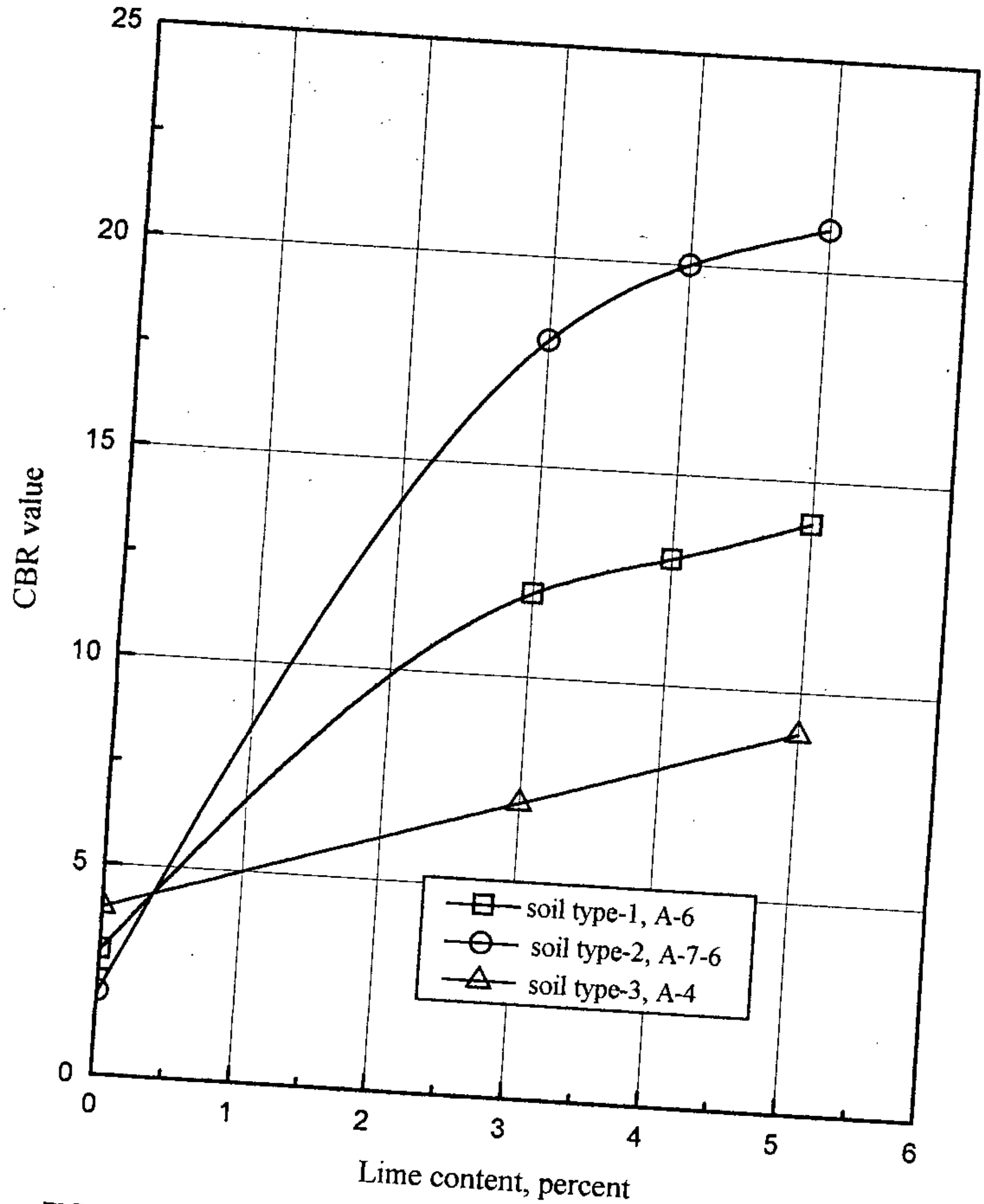


FIG 4.12 RELATIONSHIP BETWEEN CBR VALUE AND LIME CONTENT (AFTER FOUR DAYS SOAKING)

The CBR value using 3%, 4%, 5% lime contents are 12, 13, 14 respectively for soil type-1. For soil type-2 these value are 18, 20, 21 respectively. For soil type-3 the values are 7 and 9 for 3% and 5% lime content respectively. The results shows that the increase of CBR value is less significant for the increase of lime content between 3% and 5%. It may be mentioned that CBR test was performed after 4 days soaking. However, its value will be different after 28 days soaking.

4.3.4 *Effect of Compactive effort on the lime stabilized soil*

To study the effect of compactive effort on the lime stabilized soil, unconfined compressive strength and CBR test was performed on the stabilized soil at different compactive level.

Unconfined compressive strength

To observe the effect of compactive effort on the lime stabilized soil, test specimen were prepared near the optimum moisture content of the soil with different lime content using Modified Proctor energy. Unconfined compressive strength and water content of these sample was determined. Unconfined compressive strength of the stabilized soil compacting by Standard Proctor method corresponding to these moisture contents was determined from fig, 4.5. The results of the unconfined compressive strength of the lime stabilized soil using this two types of compaction energy are presented in the Table 4.7.

From table 4.7, it can be observed that the unconfined compressive strength of the lime stabilized soil is higher when compacted using Modified Proctor method than the strength using Standard Proctor test method. It is also observed from the table that the strength increase due to the use of Modified Proctor test method is not that significant for lime stabilized soil.

TABLE 4.7 UNCONFINED COMPRESSIVE STRENGTHS (q_u) OF STABILIZED SOIL COMPACTING BY STANDARD AND MODIFIED PROCTOR METHOD

soil type	lime percent	moisture content	q_u, psi (kPa) (28 days)		q_u (modified) / q_u (standard)
			modified proctor method	* standard proctor method	
soil type-1 A-6	0	11.6	13.4 (92.4)	10.6 (73.09)	1.28
	3	13.3	58.0 (399.94)	49.7 (342.7)	1.16
	4	13.0	58.9 (406.14)	51.4 (354.42)	1.14
	5	13.5	60.8 (419.24)	54.1 (373.04)	1.12
soil type-2 A-7-6	0	20.8	19.3 (133.08)	16.5 (113.77)	1.17
	3	21.8	95.0 (655.07)	84.1 (579.9)	1.13
	4	21.5	95.6 (659.2)	87.5 (603.35)	1.09
	5	21.7	100.1 (690.23)	91.8 (633.0)	1.09
soil type-3 A-4	0	18.3	7.7 (53.1)	5.2 (35.85)	1.48
	3	18.3	18.0 (124.11)	14.7 (101.36)	1.22
	5	18.7	20.33 (140.17)	16.8 (115.84)	1.21

Notes: Value in parentheses represents the value in SI unit

* The value of q_u was determined from fig. 4.5A, 4.5B and 4.5C

It can be also observed from table 4.7 that the ratio of q_u (modified)/q_u (standard) is dependent on lime percent. The ratio is low for high lime content. The ratio of q_u (modified)/q_u (standard) is also dependent on soil type. The ratio is higher for ML type of soil than CL type of soil. The value of q_u (modified)/q_u (standard) is 1.16 for soil type-1, 1.13 for soil type-2 and 1.24 for soil type-3 at 3% lime content. However, it is worth mentioning that the total applied energy by Modified Proctor method is about four and a half times than the total applied energy by the Standard Proctor method.

According to Croft (1964), the reason behind the increase in unconfined compressive strength due to the addition of lime to the soil is that due to higher compaction energy in Modified Proctor test method, both the soil particles and the lime come close together and thus expedite the reaction of lime with soil. Particle orientation of the soil is changed due to the application of higher compaction energy.

In order to observe the effect of the application of lower compaction energy on the lime stabilized soil, total applied energy was adjusted using Standard Proctor test method but reducing the no of blows per layer. The number of blows per layer was 13, 8, and 6 instead of 25 blows per layer, the other variables (height of fall, weight of hammer, number of layers) remained the same as Standard Proctor test method.

Results of the unconfined compressive strength of the specimen prepared at different compaction energy are shown in Table 4.8. Unconfined compressive strength corresponding to different blows per layer using Standard Proctor method are shown in fig. 4.13. The unconfined compressive strength of soil specimen compacting using different compactive efforts are plotted in fig 4.14.

It can be observed from fig 4.13, that the strength of the lime stabilized soil increases sharply due to the increase of compaction energy upto a certain level for CL type of soil (soil type-1, soil type-2). After certain specific energy, the increase of strength is not that significant. It is also observed from the fig. 4.13 that the rate of increase of unconfined compressive strength is sharp upto about half of the Standard Proctor compactive effort and after that the increase

TABLE 4.8 UNCONFINED COMPRESSIVE STRENGTH (q_u) OF THE STABILIZED SOIL CORRESPONDING TO DIFFERENT BLOWS PER LAYER USING STANDARD PROCTOR METHOD

soil type	lime (%)	water content percent	q_u, psi (kPa)			
			* at 25 blows per layer	at 13 blows per layer	at 8 blows per layer	at 6 blows per layer
soil type-1 A-6	3	12.7	49.3 (335.78)	40.3 (277.87)	35.7 (246.15)	23.5 (162.03)
	4	13.4	52.1 (353.71)	42.1 (290.28)	35.8 (246.84)	24.7 (170.31)
	5	13.6	54.8 (379.4)	45.7 (315.10)	38.2 (263.40)	24.3 (167.55)
soil type-2 A-7-6	3	22.1	84.7 (574.35)	72.5 (499.88)	68.3 (470.93)	46.1 (317.86)
	4	21.9	88.1 (601.93)	77.1 (531.6)	70.9 (488.85)	49.3 (339.92)
	5	22.4	93.2 (632.27)	80.2 (552.98)	72.4 (499.20)	52.3 (360.61)
soil type-3 A-4	3	18.4	14.95 (103.08)	10.7 (73.78)	7.1 (48.96)	5.8 (40.0)
	5	18.4	16.3 (112.4)	11.9 (82.05)	7.3 (50.33)	5.9 (40.68)

Notes: Value in parentheses represents the value in SI unit

* The value of q_u was determined from fig. 4.5A, 4.5B and 4.5C

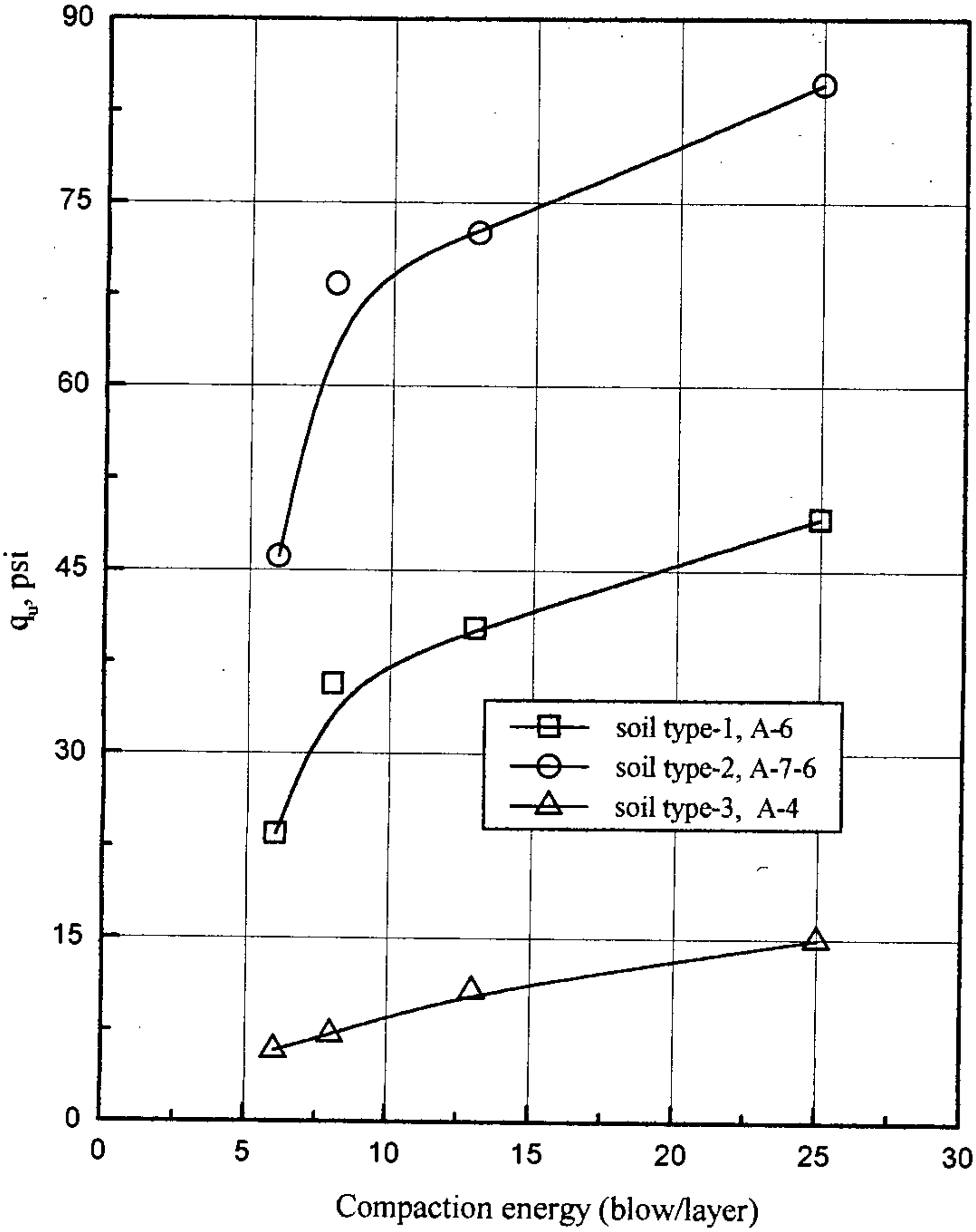


FIG 4.13 UNCONFINED COMPRESSIVE STRENGTH (q_u) AT DIFFERENT BLOWS PER LAYER AT 3% LIME USING STANDARD PROCTOR EQUIPMENTS

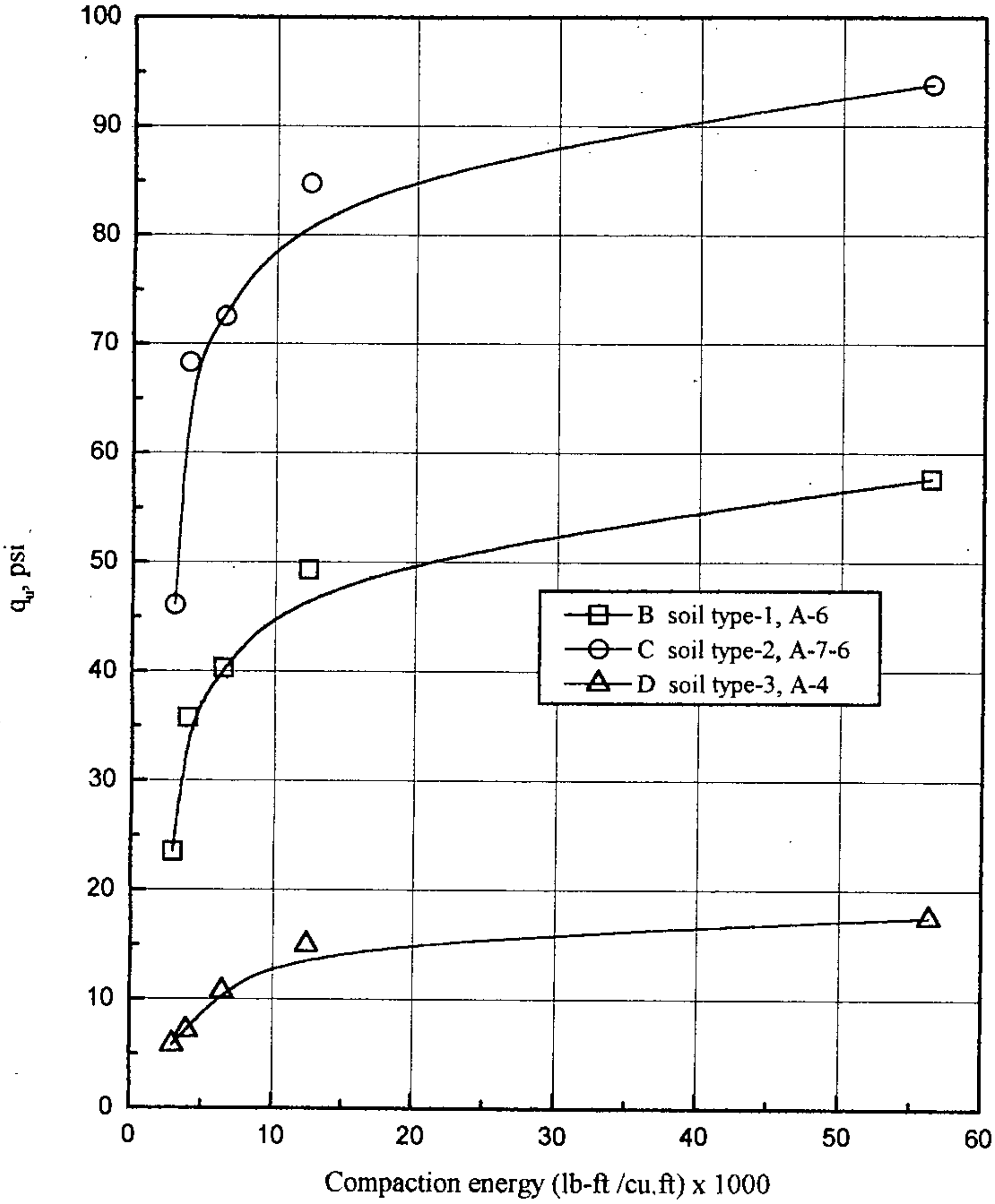


FIG. 4.14 UNCONFINED COMPRESSIVE STRENGTH (q_u) AT DIFFERENT COMPACTIVE EFFORT FOR STABILIZED SOIL USING 3% LIME

is gradual. For soil type-3, the increase in strength is low and gradual with increase of compactive effort.

From fig 4.13, it is also important to note that unconfined compressive strength value is very low when compacted with very small energy (e.i at 6 blows per layer). It was stated previously that lime react slowly with soil particles and this phenomenon is more evident for specimens at reduced level of compaction. From the present study, it may be concluded that the optimum energy for lime stabilization is about half of the standard compactive effort.

California Bearing Ratio (CBR) Test

To observe the effect of compaction effort on the lime stabilized soil in soaked condition, CBR test was performed on the test specimen prepared near the optimum moisture content of the soil. The CBR value of lime stabilized soil using different blows per layer are shown in Table 4.9. The CBR value and corresponding compactive effort are plotted in fig 4.15. From the figure, it can be observed that CBR value increases with the increase of compactive effort. Similar results was also observed for unconfined compressive strength.

It is interesting to note that the CBR value of the soil without lime, using 65 blows per layer is higher than the lime stabilized soil with 3%, 4% and 5% lime using 10 blows per layer. So it can be concluded that lime itself will not increase the strength but a certain level of compaction energy is required for proper lime stabilization.

For proper stabilization of lime, a minimum amount of energy is essential. Herrin and Mitchell (1961) and Mateos (1964) have suggested that successful stabilization can be achieved without full compaction but a minimum amount of energy is essential for proper stabilization.

TABLE 4.9 CALIFORNIA BEARING RATIO (CBR) VALUE OF THE LIME STABILIZED SOIL CORROSPONDING TO DIFFERENT BLOWS PER LAYER USING STANDARD PROCTOR METHODS

soil type	lime percent	Initial mixing moisture content	CBR value		
			at 10 blows per layer (standard proctor apparatus)	at 30 blows per layer (standard proctor apparatus)	at 65 blows per layer (standard proctor apparatus)
soil type-1 A-6	0	12.5	2	3	6
	3	13.2	3	12	16
	4	13.8	3	13	17
	5	14.3	4	14	20
soil type-2 A-7-6	0	21.0	1	2	4
	3	23.1	4	18	28
	5	23.5	4	20	30
	5	23.8	5	21	33
soil type-3 A-4	0	18.8	3	4	8
	3	19.4	4	7	11
	5	19.8	4	9	14

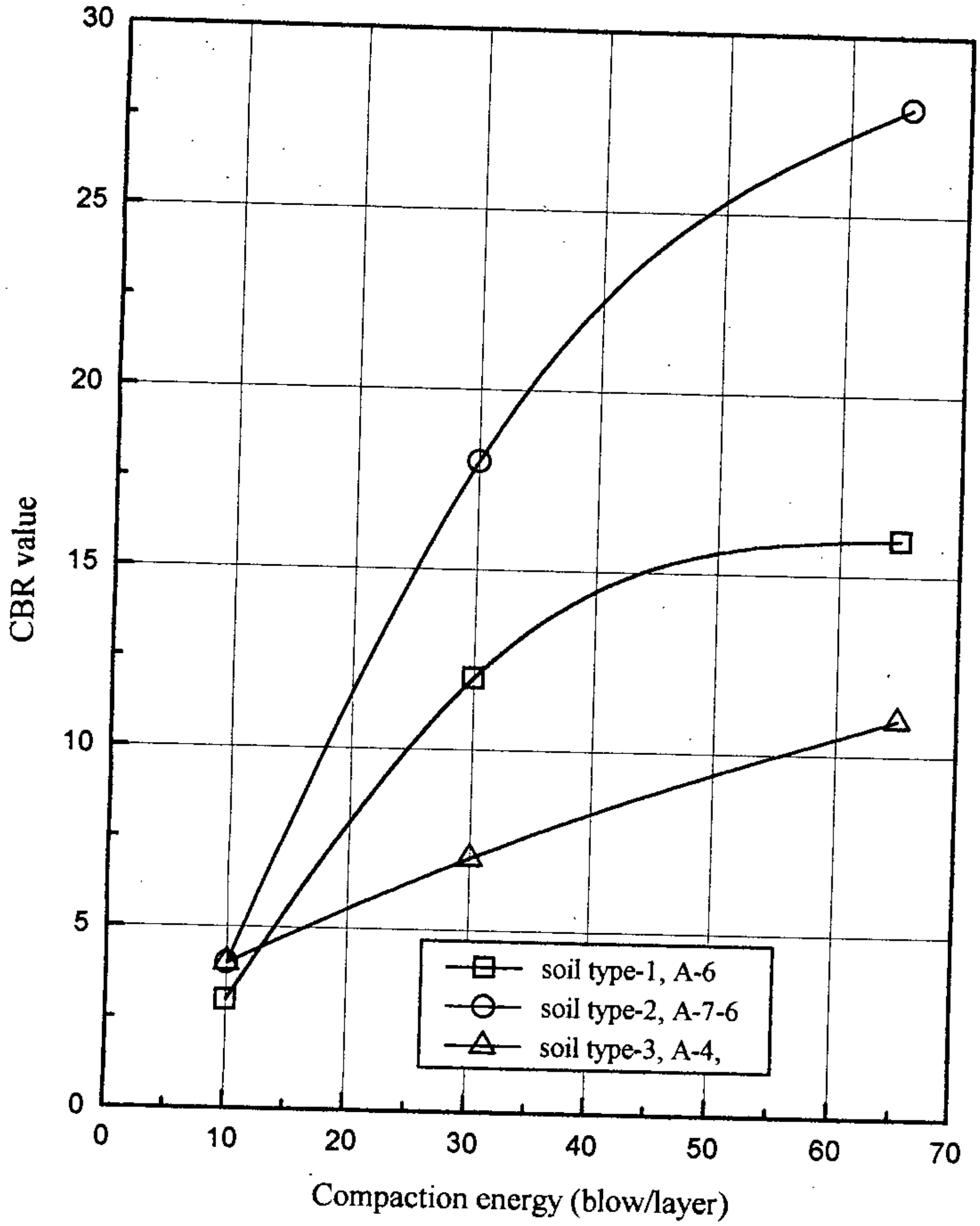


FIG 4.15 VARIATION OF CBR VALUE WITH COMPACTION ENERGY FOR STABILIZED SOIL USING 3% LIME (SOAKED)

4.3.5 Effect of soil type on strength

Three types of soil was used for stabilization throughout the research work. Three types of soil has different clay content. According to Unified (ASTM, 1978) Soil Classification System, soil type-1 and soil type-2 are CL type of soil and soil type-3 is of ML type of soil. According to the AASHTO (1993), system of soil classification soil type-1 is A-6, soil type-2 is A-7-6 and soil type-3 is A-4. The variation of the maximum unconfined compressive Strength with lime content for the three types of soil are shown in Table 4.10.

For a given lime content, laboratory test result shows a marked difference in variation of strength for the three types of soil due to the stabilization with lime. For 3% lime content, soil type- 2, have unconfined compressive strength of 85.6 psi while for soil type-1 and soil type-3, the value are 51.4 psi and 15.7 psi respectively. The difference in increase of strength for the three types of soil is very significant. Similar difference in unconfined compressive strength was also observed for 4% and 5% lime content.

CBR value of the lime stabilized soil also vary for the three types of soil for a given lime content. The variation of CBR value for the three types of soil are tabulated in Table 4.11. CBR test was performed at the laboratory in the soaked condition after 4 days of soaking. Sample was prepared near the optimum moisture content of the soil lime mix. From the table 4.11, it can be observed that CBR value is 3 for untreated soil type-1 soil and the value is 12 when stabilized with 3% lime. The CBR value is 2 for untreated soil type-2 and the value is 18 when 3% lime is used. For soil type-3, the value of CBR for untreated soil and stabilized soil (3% lime) are 4 and 7 respectively. CBR value at the age of 28 days would be higher than the values presented in Table 4.11.

The reason behind the higher strength for soil type-2 is that the soil type-2 has large amount of clay content. On the other hand soil type-3 has low amount of clay content, which is the controlling factor for the strength gain of the lime stabilized soil. Compendium (1987) observed that the clay type of soil is very effective in lime stabilization.

TABLE 4.10 RATIO BETWEEN UNCONFINED COMPRESSIVE STRENGTHS (q_u) OF STABILIZED AND UNTREATED SOILS (COMPACTED USING STANDARD PROCTOR METHOD)

soil type	lime percent	* q_u, psi (kPa)	$\frac{q_u \max (\text{soil+lime})}{q_u \max (\text{soil})}$ (28 days) strength)
soil type-1 A-6	0	11.1 (76.53)	-
	3	51.4 (355.8)	4.63
	4	52.8 (364.41)	4.76
	5	55.2 (380.63)	4.97
soil type-2 A-7-6	0	16.7 (115.15)	-
	3	85.6 (590.25)	5.12
	4	90.0 (620.6)	5.39
	5	95.0 (655.06)	5.68
soil type-3 A-4	0	5.7 (39.3)	-
	3	15.7 (108.25)	2.75
	5	17.4 (120.4)	3.05

Notes: Value in parentheses represents the value in SI unit

* The maximum value of q_u was determined from fig. 4.5A, 4.5B and 4.5C

TABLE 4.11 RATIO OF CBR VALUE BETWEEN STABILIZED AND UNTREATED SOILS

soil type	lime percent	moisture content	CBR value	CBR (soil+lime) / CBR (soil)
soil type-1 A-6	0	13.2	3	-
	3	12.9	12	4.0
	4	13.5	13	4.33
	5	14.0	14	4.66
soil type-2 A-7-6	0	22.8	2	-
	3	23.2	18	9
	4	23.4	20	10
	5	23.4	21	10.5
soil type-3 A-4	0	19.7	4	-
	3	19.3	7	1.75
	5	19.5	9	2.25

Ingles et al. (1972) stabilized different types of soil with lime and observed a wide variation of strength increase for different types of soil. For silty clay, they observed higher unconfined compressive strength than clayey silt. They explained that as the fine particles of the soil increases, the negative charges of soil increases. Negative charge of the soil is responsible for base exchange, flocculation and pozzolanic reaction.

Serajuddin (1992) performed experiment with three types of soils. He observed that the strength of A-4 (silt loam) type soil is much less compared to those of A-6 (silty clay loam) and A-7-6 (silty clay loam) type of soil. According to him, the higher strength with A-6 and A-7-6 types of soil is due to the presence of larger quantity of clay content, which is the basic element of soil when added with lime in the formation of calcium silicate due to the chemical reaction between lime and clay minerals producing cementing compounds.

4.3.6 Effect of compaction delay time on strength

For lime stabilization, the effect of delay in compaction after mixing with time is less important than cement stabilization. The delay in compaction after mixing is critical for cement but it is not so critical for lime. Because cement react with soil very fast. To observe the effect of compaction delay time on the lime stabilized soil, soil-lime mixture was left for 1 hr, 3 hr, 6 hr, 12 hr, 24 hr, 48 hr before compaction. 3%, 4% and 5% lime was used for preparing soil sample. The specimen was prepared near the optimum moisture content of the soil lime mix. After compacting the soil sample using Standard Proctor test method, test specimen were prepared.

The value of the unconfined compressive strength at different compaction delay time is presented in Table 4.12. Unconfined compressive strength at different compaction delay time of the lime stabilized soil type-1 and soil type-2 are shown in fig. 4.16a and fig. 4.16b.

TABLE 4.12 UNCONFINED COMPRESSIVE STRENGTH (q_u) OF THE LIME STABILIZED SOILS FOR DIFFERENT COMPACTION DELAY TIME.

soil type	lime (%)	water cont.	q_u, psi (kpa)						$q_{u(48hr)}$ / $q_{u(1hr)}$
			1 hr compaction delay*	3 hr compaction delay	6 hr compaction delay	12 hr compaction delay	24 hr compact-ion delay	48 hr compact-ion delay	
soil type -1 A-6	3	12.3	48.7 (335.78)	48.2 (332.34)	47.4 (326.82)	45.7 (315.10)	44.0 (303.38)	40.4 (278.56)	0.83
	4	12.8	51.3 (353.71)	49.8 (343.37)	49.0 (337.85)	47.5 (327.51)	46.1 (317.86)	43.7 (301.31)	0.85
	5	12.5	53.7 (370.26)	53.2 (366.81)	52.3 (360.61)	51.0 (351.64)	49.0 (337.85)	47.1 (324.75)	0.87
soil type -2 A-7-6	3	20.9	83.3 (574.35)	82.4 (568.15)	81.5 (561.94)	80.0 (551.6)	78.3 (539.88)	76.1 (524.71)	0.91
	4	21.6	87.3 (601.93)	86.7 (597.80)	85.6 (590.21)	84.1 (579.87)	82.5 (568.84)	80.3 (553.67)	0.92
	5	21.6	91.7 (632.27)	90.4 (623.31)	88.9 (612.96)	87.3 (601.93)	85.8 (591.59)	84.2 (580.56)	0.92

Notes: Value in parentheses represents the value in SI unit

* Value of q_u was determined from fig. 4.5A, fig. 4.5B and fig 4.5C

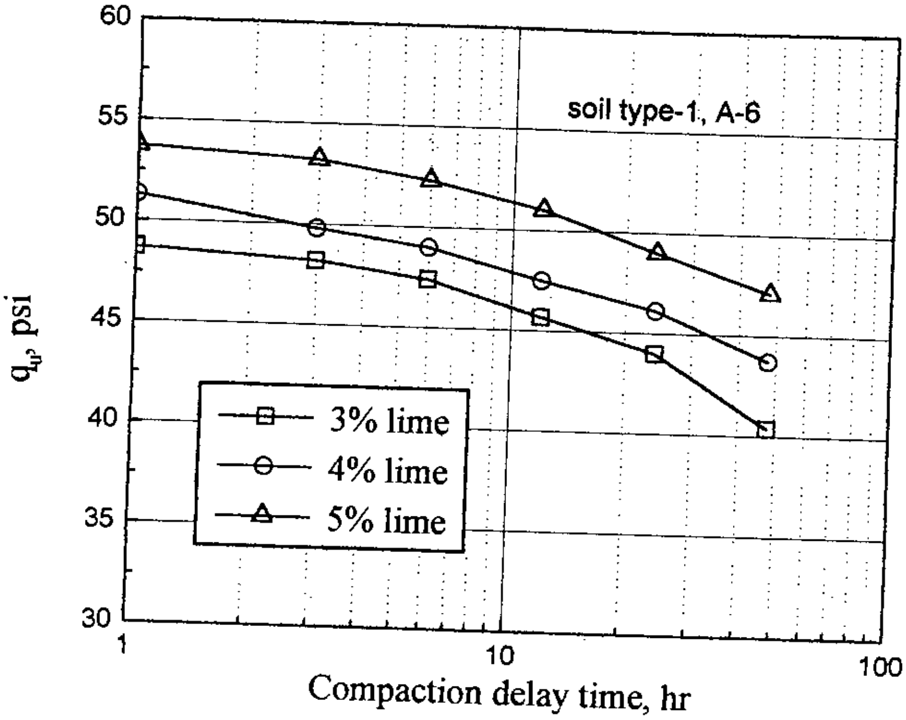


FIG 4.16A UNCONFINED COMPRESSIVE STRENGTH AT DIFFERENT COMPACTION DELAY TIME FOR SOIL TYPE-1 (USING STANDARD PROCTOR METHOD)

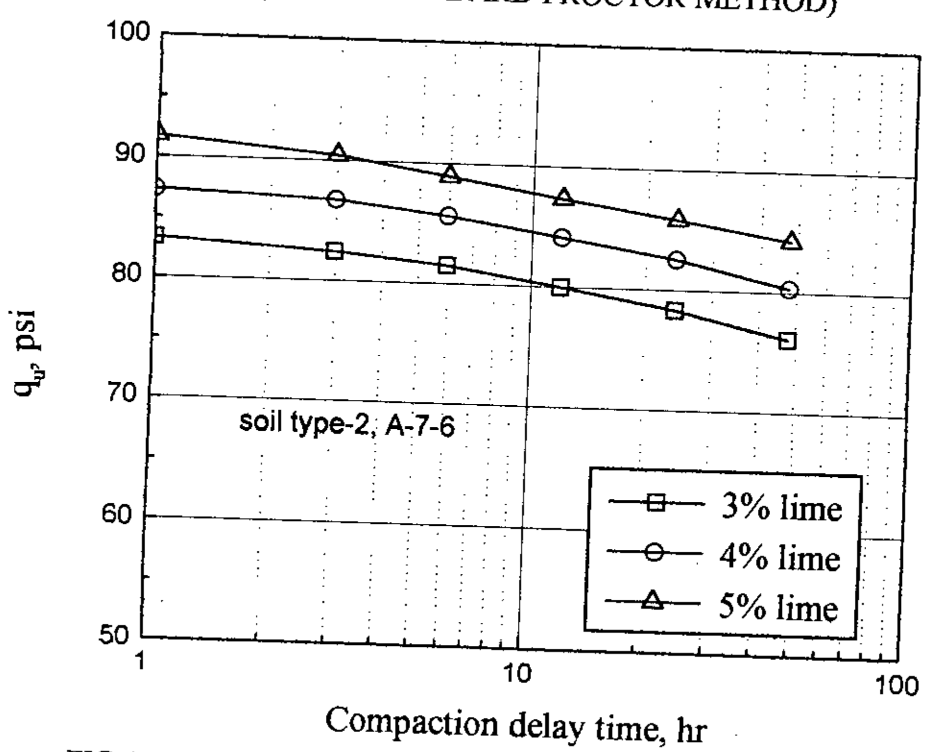


FIG 4.16B UNCONFINED COMPRESSIVE STRENGTH AT DIFFERENT COMPACTION DELAY TIME FOR SOIL TYPE-2 (USING STANDARD PROCTOR METHOD)

From figures, it can be observed that the delays in compaction period decreases, the strength of the stabilized soil. It can also be observed from the figures that at a delay time of six hours between mixing and compaction, the unconfined compressive strength remains almost same, but with further delay, the strength of the soil lime mix continues to fall at a small rate. Sastry (1987) observed that for a delay period of two hours, there is no appreciable decrease in strength of the lime stabilized soil.

For a time lag of 48 hrs, the resultant fall in strength is between 8 to 17 percent. For silty soil with 5% lime content, Mitchell and Hopper (1961) found that a 24 hr compaction delay time causes 30% strength decrease. But Metcalf (1977), Compendium (1987) observed that the compaction delay time for two days has no marked effect on the strength of the lime stabilized soil.

Mitchel and Hopper (1961) suggested that the decrease in strength for delay in compaction after mixing is due to the formation coagulation between soil and lime particles.

Townsend et al. (1970) noted that if the specimen were compacted to the same densities, approximately equal strength can be obtained for the specimen with different mellowing times up to 72 hr. Hence compaction delay is not detrimental except for additional costs to provide extra compaction.

It can be concluded that some researchers observed compaction delay time has marked effect on the strength of the lime stabilization while the others observed that it has no appreciable effect on strength. From the present study, it can be observed that upto a delay period of 6 hours there is practically no change of strength at all. But after 48 hours of delay period, there is 8% to 17% decrease in strength depending upon the soil type and lime content.

From the table 4.12, it can be noted that the decrease in strength due to compaction delay time is less sensitive for soil type-2 than soil type-1. For 3% lime content, the ratio between the unconfined compressive strength at a delay time of 48 hrs ($q_{u\,(48\ hr)}$) and unconfined compressive strength of at the delay time of 1 hrs ($q_{u\,(1\ hr)}$) is 0.83 for soil type-1 and 0.91 for soil type-2.

It can also be observed that as the lime content increases, the compaction delay time has less effect on strength of the stabilized soil type-1.

4.4.7 Effect of age on strength

Effect of age is considered as an important factor for lime stabilization and a large number of study was performed on it. For the present study, unconfined compressive test specimen were prepared near the optimum moisture content of the soil lime mix by Standard Proctor test method using different lime content. Sample were kept in desiccator for 7 days, 14 days and 28 days after preparation. Samples were prepared near the optimum moisture content of soil. The value of unconfined compressive strengths for different ages are presented in Table 4.13. The percentage of strength gain at 7 days and 14 days with respect to 28 days are shown in Table 4.14. The effect of age on the strength of the lime stabilized soil are also presented in fig. 4.17a, fig. 4.17b and fig. 4.17c.

From table 4.13 and the figures, it can be seen that the unconfined compressive strength of the stabilized soil increases with the increase of age similar to cement concrete.

From table 4.14, it can be observed that the value of shows that the value of q_u (7 days) /q_u (28 days) is 0.67, 0.72 and 0.80 for soil type-2, soil type-1 and soil type-3 respectively using 3% lime. The ratio of q_u (7 days) /q_u (28 days) is higher for silty soils than clayey soils.

From fig. 4.17a, fig. 4.17b and fig. 4.17c, it can be also observed that the rate of increase in strength is not uniform with time. Initially, the rate of increase in strength is high. The rate of increase of unconfined compressive strength of the stabilized soil changes with the increase of age. Hilt and Davidson (1962) had found similar behaviour from their research. Their findings can be explained by their statements " Apparently strength develops at a rate that parallel the rate of reaction. As the lime combines with soils, the amount of free lime decreases, the rate of strength increase gradually slows".

The ratio of the unconfined compressive strength between 7 days and 28 days using 3%, 4% and 5% lime are 0.67, 0.70 and 0.71 respectively for soil

TABLE 4.13 GAIN OF STRENGTH WITH TIME OF LIME STABILIZED SOIL

soil type	lime percent	moisture content	q_u, psi (kPa) at 28 days *	q_u, psi (kPa) at 14 days	q_u, psi (kPa) at 7 days
soil type-1 A-6	3	13.1	50.0 (344.77)	42.5 (293.04)	36.0 (248.2)
	4	13.0	51.5 (355.1)	44.8 (308.31)	40.17 (277.0)
	5	12.8	54.2 (373.73)	50.4 (347.57)	42.82 (293.24)
soil type-2 A-7-6	3	21.5	83.6 (576.46)	76.24 (525.43)	56.0 (386.0)
	4	21.8	87.5 (603.35)	81.3 (561.11)	61.25 (422.3)
	5	21.5	91.5 (630.97)	84.1 (579.97)	64.96 (447.96)
soil type-3 A-4	3	18.6	15.2 (104.81)	13.7 (94.48)	12.2 (84.12)
	5	18.9	16.8 (115.84)	14.5 (99.98)	13.0 (89.64)

Notes: Value in parentheses represents the value in SI unit
The value of q_u was determined from fig. 4.5A, 4.5B and 4.5C

TABLE 4.14 RELATIVE GAIN OF STRENGTH WITH TIME OF LIME STABILIZED SOIL

soil type	lime percent	moisture content	q_u, psi (kPa) at 28 days	q_u (14 days) / q_u (28 days)	q_u (7 days) / q_u (28 days)
soil type-1 A-6	3	13.1	50.0 (344.77)	0.85	0.72
	4	13.0	51.5 (355.1)	0.87	0.78
	5	12.8	54.2 (373.73)	0.93	0.79
soil type-2 A-7-6	3	21.5	83.6 (576.46)	0.90	0.67
	4	21.8	87.5 (603.35)	0.93	0.70
	5	21.5	91.7 (632.27)	0.90	0.71
soil type-3 A-4	3	18.6	15.2 (104.81)	0.90	0.80
	5	18.9	16.8 (115.84)	0.86	0.77

Note: Value in parentheses represents the value in SI unit

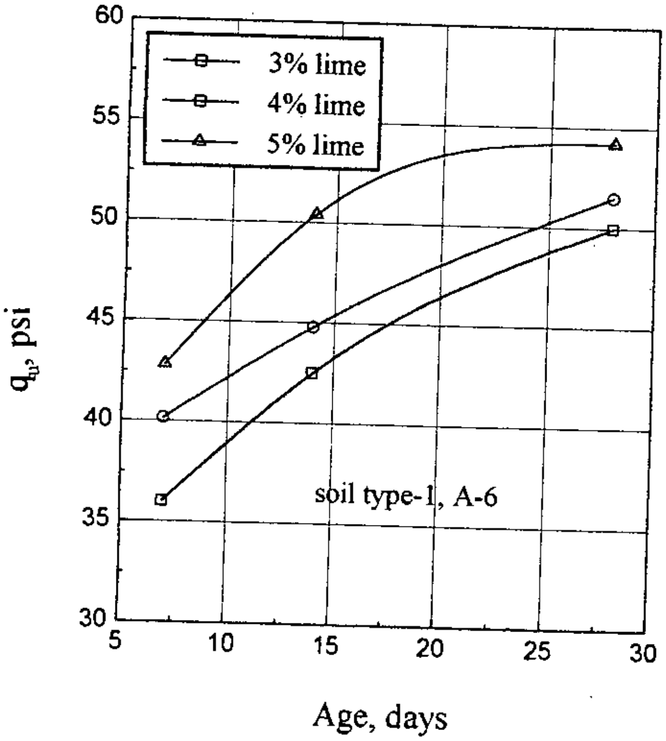


FIG 4.17A UNCONFINED COMPRESSIVE STRENGTH (q_u) WITH AGE FOR SOIL TYPE-1

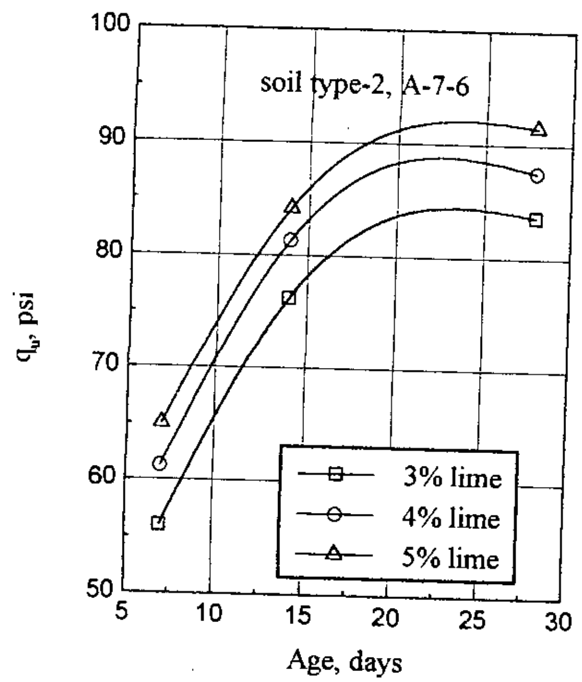


FIG 4.17B UNCONFINED COMPRESSIVE STRENGTH(q_u) WITH AGE FOR SOIL TYPE-2

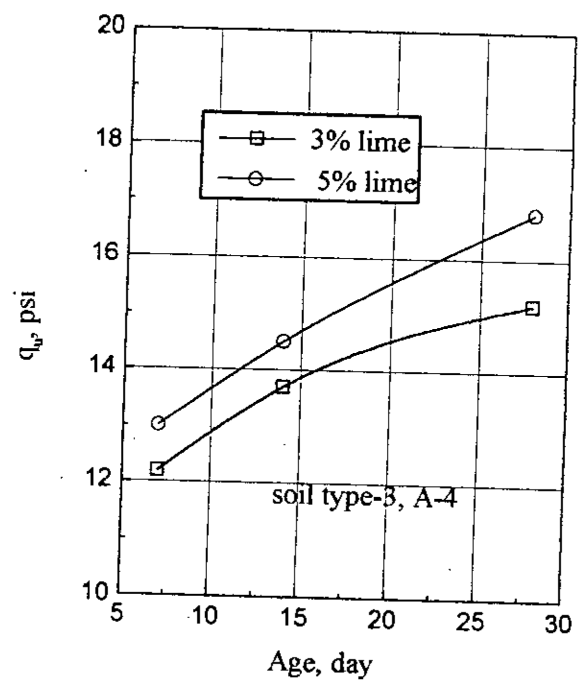


FIG 4.17C UNCONFINED COMPRESSIVE STRENGTH (q_u) WITH AGE FOR SOIL TYPE-3

type-2. For soil type-1, the ratio of the unconfined compressive strength between 7 days and 28 days are 0.72, 0.76, 0.78 respectively. So it can be observed that the ratio $q_{u\,(7\ days)}/q_{u\,(28\ days)}$ varies with lime content. For clayey soils the ratio increases with lime content.

Assarson et al. (1974) pointed that the pozzolanic reaction is a slow reaction and from the test result they confirmed that at the early stage after stabilization the reaction is not completed. Fine grained soil takes longer time for reaction to be completed.

Serajuddin (1991) measured the unconfined compressive strength at 7 days, 14 days and 28 days with different types of soil and found that the unconfined compressive strength of 7 days vs. unconfined compressive strength of 28 days varies between 0.6 to 0.9 depending upon the type of the soil.

4.4.8 Effect of stage Mixing on strength

To investigate the effect of strength on the mixing procedure, unconfined compressive test specimen was prepared near the optimum moisture content of soil by two stage of mixing. Specimen was prepared by compacting the soil sample using Standard Proctor test method. In two stage of mixing, half of the lime and half of the water was mixed initially followed by another half after 18 hours. The results of the unconfined compressive strength are shown in Table 4.15. The effect of the stage mixing on the strength of the stabilized soil type-1 and soil type-2 are also plotted in fig. 4.18a and 4.18b.

From the figures, it can be seen that the unconfined compressive strength is higher in two stage mixing than the strength at one stage of mixing for soil type-1 and soil type-2. But the increase of strength is not remarkable. The reason behind the increase in strength as Yu kuen (1975) stated that due to two stage mixing soil become more friable after first stage of mixing.

TABLE 4.15 RELATIVE STRENGTH OF LIME STABILIZED SOIL FOR ONE AND TWO STAGE MIXING.

soil type	lime percent	water content	q_u, psi (kPa)		q_u (two stage) / q_u (one stage)
			two stage of mixing*	one stage of mixing (from graph**	
soil type-1 A-6	3	13.0	51.3 (353.73)	50.0 (344.77)	1.02
	4	13.4	53.7 (370.28)	52.1 (359.23)	1.03
	5	12.7	55.6 (383.38)	54.2 (373.73)	1.02
soil type-2 A-7-6	3	22.1	87.88 (605.97)	84.5 (582.66)	1.04
	4	21.3	90.37 (623.18)	86.9 (599.23)	1.04
	5	21.8	96.81 (667.5)	92.2 (635.76)	1.05

Notes: Value in parentheses represents the value in SI unit

* In two stage of mixing half of the total water and lime is mixed initially and the remaining water and lime is mixed after 18 hrs

** The value of q_u was determined from fig. 4.5A, 4.5B and 4.5C

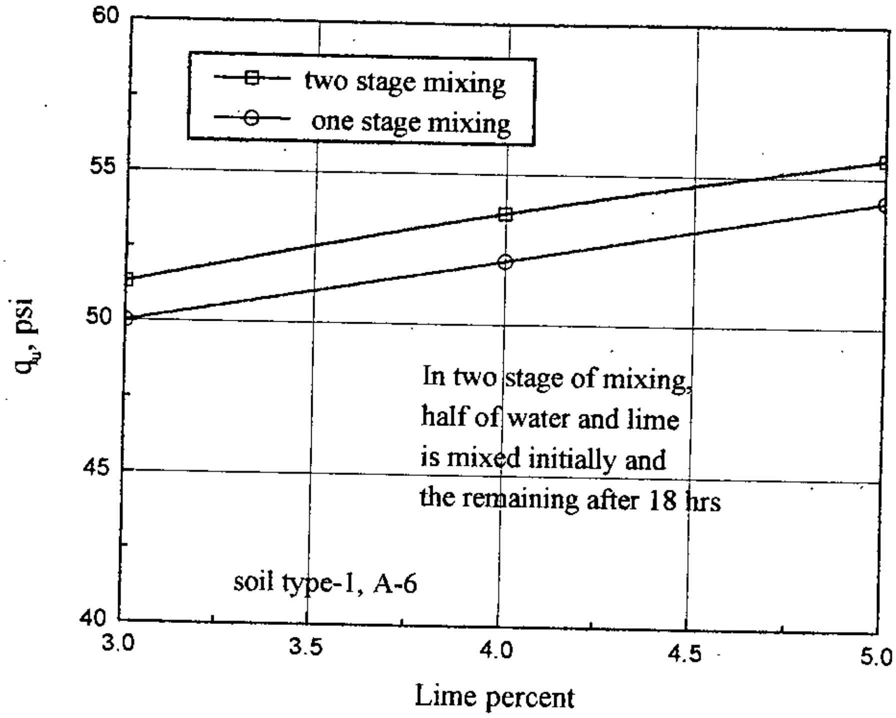


FIG 4.18A UNCONFINED COMPRESSIVE STRENGTH (q_u) OF STABILIZED SOIL TYPE-1 FOR STAGE MIXING

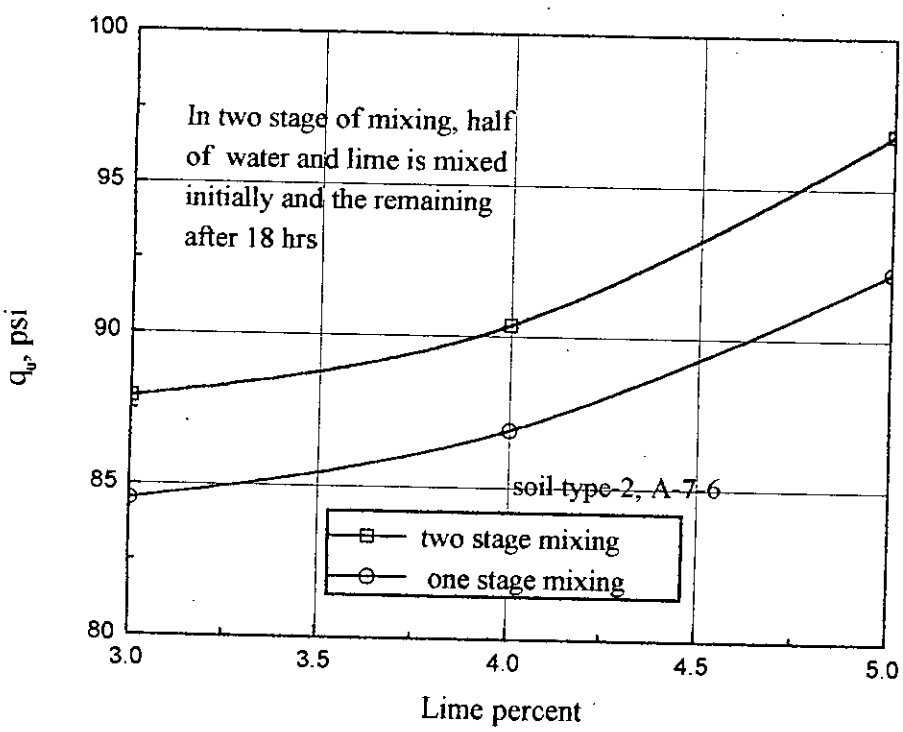


FIG 4.18B UNCONFINED COMPRESSIVE STRENGTH (q_u) OF STABILIZED SOIL TYPE-2 FOR STAGE MIXING

CHAPTER FIVE

Conclusions and Recommendations for the Future Research

5.1 General

It is observed from the present study that the lime stabilization changes the physical properties of the soil and increases the strength of the soil. A number of variables affect the strength of the lime stabilized soil in different ways. Some of the factors have significant effect on the strength of the stabilized soil. The major factors those affect the strength of lime stabilized soil are soil type, lime percent, compaction energy, water content and compaction delay time.

5.2 Conclusions

Different aspects of lime stabilized soil has been studied in the present research work. From the present study, the following conclusions can be drawn:

(i) The increase of strength of the lime stabilized soil is highly dependent on the soil type. Strength of the lime stabilized soil increases due to the increase of clay content in the soil. The soil type-2 [according to Unified Soil Classification System CL and according to AASHTO A-7(6)] is more suitable for lime stabilization than soil type-1 (according to Unified Soil Classification System CL and according to AASHTO A-6) and soil type-3 (according to Unified Soil Classification System ML and according to AASHTO A-4). Soil type-3 is not suitable for lime stabilization.

ii) For a given compactive effort, the unconfined compressive strength and CBR value of the lime stabilized soil (CL type) is much higher than the untreated soil. Soil stabilization using 3% lime produces significant increase in strength. The strength of the lime stabilized soil (CL type) using 3% lime is about 4.5 to 5.7 times higher than the untreated soil when compacted by the Standard Proctor method. Due to the increase of lime content from 3% to 5%, the increase in strength is not significant.

iii) Secant modulus of elasticity corresponding to 50% peak stress (E_{50}) increases significantly with the addition of lime to the soil. For a given lime content and compactive effort the E_{50} of CL type soil is higher than the E_{50} of ML type soil.

iv) The maximum dry density of the soil reduces due to the addition of lime. However, addition of lime increases the strength significantly. For field control of compaction, dry density is normally measured for natural soils. For lime stabilized soil, strength should be used as indicator for field control.

v) Optimum moisture content of the soil increases due to the addition of lime to the soil and the optimum moisture content continues to increase with the increase of lime percent. The optimum moisture content of soil type-2 [according to Unified Soil Classification System CL and according to AASHTO A-7-6] is increased to about 23 percent from 21 percent due to the addition of 3% lime. Maximum strength of the stabilized soil occurs at the moisture content higher than the optimum moisture content of the stabilized soil at which maximum dry density is achieved. However the difference is not significant. When wet soil is difficult to compact, lime stabilization may be helpful.

vi) The strength of the stabilized soil increases rapidly with the increase of compaction energy upto a certain level. After that the rate of increase is gradual. Upto about the half of the standard proctor energy, the increase of strength is rapid. For proper lime stabilization, high compaction energy is not required, at the same time very low compaction energy is not desirable.

vii) There is practically no change in strength due to delay in compaction of 6 hours after mixing. About 4% to 7% decrease in strength occurs due to the compaction delay time of 12 hours and about 8% to 17% decrease in strength occurs due to the compaction delay time of 48 hours.

viii) Unconfined compressive strength of the lime stabilized soil increases with time. The ratio between q_u (7 days) and q_u (28 days) of the stabilized soil is about 67% to 80% depending upon the soil type. For clay soil, the ratio between q_u (7 days) and q_u (28 days) is lower than silty soil. This ratio increases with the increase of lime content.

ix) Increase of unconfined compressive strength is insignificant for two stage of mixing with respect to one stage of mixing for the CL type of soil. In two stage of mixing half of the total water and lime were added initially and the remaining water and lime were added after 18 hours.

5.3 Recommendations for future research

The present study covers the strength gain due to lime stabilization considering different variables that affect lime stabilization. It is recommended to extend the research in the following field to have a better understanding about lime stabilization:

1) As the soil type is one of the most important parameter for lime stabilization, investigation using different types of natural soil and mixing two types of soil togather can be performed.

2) Applying compaction effort using other methods i.e kneading compaction, compaction with preloading, compaction by vibration on the lime stabilized soil can be investigated.

3) Detailed investigation can be performed on the permeability and compressibility characteristics of the lime stabilized soil.

4) The effect of age for longer period on the stabilized soil can be investigated.

5) Investigation may be carried out on the field of the lime stabilized soil.

TABLE ANNEX-1 DRY DENSITY AND CORROSPONDING MOISTURE CONTENT OF SOIL, COMPACTED BY STANDARD PROCTOR METHOD

soil type	Standard Proctor method		Modified Proctor method	
	moisture content percent	dry density lb/cu.ft (kN/cu.m)	moisture content percent	dry density lb/cu.ft (kN/cu.m)
soil type-1 A-6	10.2	98.2 (15.43)	9.6	99.6 (15.65)
	11.2	100.5 (15.79)	10.5	102.2 (15.90)
	11.9	102.0 (16.03)	11.3	103.4 (16.25)
	13.1	101.6 (15.96)	12.1	103 (16.18)
	14.4	99.5 (15.63)	12.9	102.3 (16.07)
	15.3	98.6 (15.49)	14.2	100.6 (15.81)
soil type-2 A-7-6	19.2	98.2 (15.43)	18.2	99.2 (15.59)
	20.5	100.7 (15.82)	19.6	101.9 (16.01)
	20.9	101.1 (15.88)	20.1	102.2 (16.06)
	21.8	100.6 (15.81)	20.8	102.1 (16.04)
	23.2	99.5 (15.63)	22.4	100.7 (15.82)
	24.2	98.7 (15.51)	23.3	100 (15.71)
soil type-3 A-4	15.3	98.1 (15.41)	14.3	99.2 (15.59)
	17.4	99.8 (15.68)	16.1	101.0 (15.87)
	19.1	100.6 (15.81)	17.8	102.0 (16.02)
	20.9	99.6 (15.65)	20.4	101.1 (15.88)
	23.5	97.8 (15.37)	22.2	99.5 (15.63)

TABLE ANNEX-2 DRY DENSITY AND UNCONFINED COMPRESSIVE STRENGTH OF LIME STABILIZED SOIL COMPACTED BY MODIFIED PROCTOR METHOD

soil type	lime percent	moisture content percent	dry density lb/cu.ft (kN/cu.m)	unconfined compressive strength psi (kpa)
soil type-1 A-6	3	10.3	96.3 (15.13)	-
	3	11.5	97.8 (15.37)	-
	3	12.1	98.7 (15.51)	53.8 (370.97)
	3	13.2	98.8 (15.52)	58.0 (399.93)
	3	14.2	97.7 (15.35)	56.1 (386.83)
	3	15.5	97.3 (15.29)	52.8 (364.08)
soil type-2 A-7-6	3	19.2	95.6 (15.02)	-
	3	20.5	97.3 (15.29)	90.5 (624.04)
	3	21.4	98.2 (15.43)	95.0 (655.07)
	3	22.1	98.0 (15.40)	93.70 (646.07)
	3	23.2	97.3 (15.29)	91.3 (628.86)
	3	24.2	96.6 (15.18)	
soil type-3 A-4	3	17.1	97.8 (15.36)	16.8 (115.84)
	3	18.3	98.8 (15.52)	17.1 (117.9)
	3	19.2	98.5 (15.47)	16.2 (117.7)
	3	20.3	97.9 (15.38)	15.5 (106.87)
	3	21.5	97.4 (15.3)	-

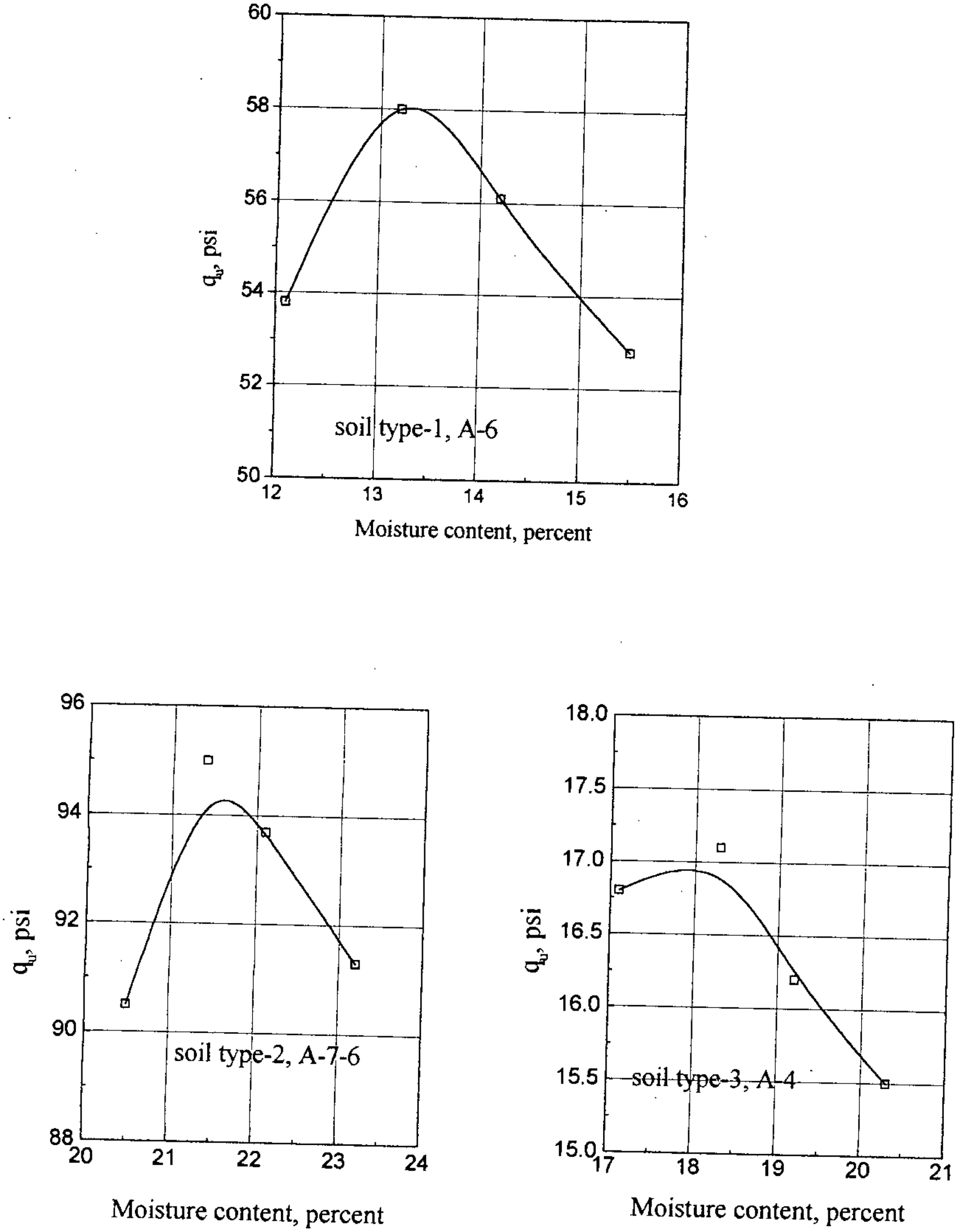


FIG. ANNEX-1 MOISTURE - STRENGTH RELATION SHIP FOR STABILIZED SOIL USING MODIFIED PROCTOR METHOD AT 3% LIME

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