COEFFICIENT OF PERMEABILITY OF TREATED SOILS

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DECLARATION

It is hereby declared that except for the contents where specific references have been made to the work of others, the studies contained in this thesis are the result of investigation carried out by the author under the supervision of Dr. Mohammad Shariful Islam, Professor, Department of Civil Engineering, Bangladesh University of Engineering and Technology.

No part of this thesis has been submitted to any other university or other educational establishments for a degree, diploma or other qualifications (except for publication).

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ABSTRACT

Effects of lime and fly ash content on the permeability of treated soils were investigated. Two types of soils (clay with low plasticity and fine sand) were selected. One of the samples was collected from Porsha of Rajshahi (clay) and other one was collected from Mirpur of Dhaka (fine sand). After collecting these soil-mass from the field, they were dried and cleaned. Samples were prepared by mixing additives with a percentage of 1%, 3% and 5% (by weight). Water was mixed with it for reaction to occur and then a sufficient time was allowed to complete hydration. Afterwards, the sample was subjected to permeability test. Permeability of sandy soils was determined by performing constant head permeability tests on the respective samples, while for clay soil falling head permeability tests were performed. Index tests indicated that plastic limit increased with increasing lime content and fly ash content while liquid limit and plasticity index reduced with the increase in lime content and fly ash content. The results are in agreement with those reported in Ahmed (1984) and IRC (1976). In the research scheme, lime and fly ash content was varied up to 5% and the reaction time was varied up to 14 days, i.e., after mixing a particular soil with a definite percent of lime, fly ash and sufficient water, the mixture was allowed for complete hydration at room temperature for 3, 7 and 14 days before testing. Test results show that decrease of permeability for both sandy and clayey soil with the increase of lime and fly ash content. Void ratio of treated soil was influenced by the addition of lime and fly ash content. For soils having substantial amount of fines were observed that void ratio decreased with the increase of lime and fly ash content. The permeability characteristic also changed accordingly. Aging had no affect on the permeability and void ratio of untreated soils. But the void ratio and hence permeability of lime and fly ash treated soil were slightly changed with the reaction time.

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NOTATIONS

ASTM = American Association of State Highways and Transportation Officials

C_s = Shape factor

D = Effective diameter of the particle

e = Void ratio

G_s = Specific gravity of the soil solid

k = Coefficient of permeability

MIT = Massachusetts Institute of Technology

n = Porosity

S = Specific surface area of the soil solids

S_s = Surface area per unit volume

T= Tortuosity

V = Volume of soil mass

W_s = Dry weight of soil grain

w_p = Plastic limit

w_L= Liquid limit

γ_w = Unit weight of soil mass

γ_d = Dry density of soil

$\boldsymbol{\eta}$ = Viscosity

Chapter One

INTRODUCTION

1.1 General

The soil improvement technique, stabilization, is applied when there is a particular and obvious deficiency in the potential a material property. The usual deficiencies are associated with strength and stiffness, excessive sensitivity to change in moisture content, high permeability, poor workability, tendency to erode, etc. By stabilization or treatment of soil, one or more of such deficiencies are improved up to the desired level by altering the associated properties. In the past, soil stabilization with lime was used in the field of highway, railroads and airport construction to improve rail beds and bearing layers. Meanwhile this method is employed also to the construction of embankments, soil exchange in sliding slopes, the backfill of bridge abutments and retaining walls, soil improvement under foundation slabs and for lime piles (for foundation, excavation pits and slope stabilization).

A lots of works on cement and lime stabilization were reported in literature, such as for general behavior of lime-treated soils (Brandl, 1981), for construction of roads (Ingles and Metcalf, 1972; Naasra, 1986; Haunsmann, 1990), for agricultural road network (Kezdi, 1979; Ahmed, 1984; Rajbonshi, 1997), for sub-base and base construction of roads on non-plastic alluvial soils of floods plains (Bangladesh Transport Survey, 1974), etc. In these works, the major engineering benefits are expected to increase strength, stiffness, durability and volumetric stability.

At present Bangladesh is moving forward with large development projects including construction of high rise building, bridges, oil storage tank, harbor and port structures, pond constructions, haor and beel development structures such as fish-pass, regulator, etc. Slope and settlement failures are not unusual in Bangladesh. So, during this stage of infrastructure development in Bangladesh, a detailed knowledge and sound understanding of flow rate of water through soil and its effects on the deformation behavior of soil are of utmost importance.

In order to improve the behaviour of such soils, attempts are made to utilize low cost local materials including waste products. Those studies deal stabilizing agents such as lime -fly ash and cement-fly ash as in the ground modification of soft compressible clays. The proper use of fly ash can reduce the cost of stabilization with pure cement or pure lime, as fly ash is a waste material. Lime during stabilization contributes to initial rapid improvement of the undrained soil strength due to cation exchange, whereas fly ash contributes mainly to the long-term gain in strength and stiffness as a result of its time-dependent pozzolanic reactivity. A small quantity of cement accelerates this process of pozzolanic hardening.

Bangladesh is a land of rivers and canals. Its ground formation mainly consists of alluvial deposits. There are many natural depressions known as beel, haor, baor and many natural or man-made ponds. The hydraulic properties of naturally deposited soils in many areas do not fulfill the requirement of construction especially related to aquaculture. Due to lower table in dry seasons, water flows from storage reservoir, ponds, etc. through the soil by percolation or seepage. This problem is severe in case of projects constructed on sandy and silty types of soils. Improvement of strength and other properties of the soil by the addition of lime as admixture is simply referred to as lime stabilization. Soil improvement in some locality is, therefore, essential particularly for water retention purpose (especially, in Tangail, Manikgong, Thakurgaon, Serajgong, Rangpur, Dinajpur, Cox's Bazar area). Now a day, different mechanical, chemical or electrical stabilization technique of soil has been developed. Depending upon the availability of equipment and technology, generally mechanical technique (compaction) is used to reduce the permeability. However, this procedure is limited to clay and silty soils. On the other hand, cement, lime, cow-dung, bitumen, fly ash, etc. can be used as stabilizing materials for sandy soil. Of them, cow-dung, fly ash, rice-husks, lime, etc. are used for solving the water retention problem at locations where sandy soils dominate the permeability characteristics.

Although a number of researches were carried out to investigate the strength and deformation characteristics of stabilized soils, little attempt has been to assess the hydraulic characteristics of such treated soils in Bangladesh. The characterization of hydraulic properties of such soil is important for permeability and seepage analysis of highways, barrage, dyke, foundation, ponds, etc. if the underlying or the protecting soil is stabilized with a method mentioned earlier. This has special importance in fishery sector, where the ponds or dykes for aquaculture consist of sandy or silty soil (e.g. the northern part of Bangladesh). In such places, water retention is ensured usually by providing clay lining on the high permeable deep-seated soil-mass. On the other hand, lime is periodically added to pond water for better production of fish as stated above. Therefore, lime is always mixing up with soil, which after some time may behave just like treated soil.

To support the proposition, Brandl (1981) showed that the permeability of lime treated soil may be changed over one or two decimal exponent. Therefore, a possibility exists that lime treated sand and silt can be used as a lining instead of clay lining in the aquaculture projects facing water retention problem.

In early day, ponds/water reservoir is generally constructed in region, where the water retention capacity is more and other hydraulic properties are favorable to the aquaculture. There was an ample opportunity for the engineers to avoid unsuitable site or unsuitable construction material source whenever the required condition did not fulfilled. But now a days, this scope has been limited. In the developing countries, considering the conventional construction materials that are adopted today, there appears to be an ample scope for exercising by the way of incorporating locally available materials and adopting the soil stabilization technique to the maximum extent possible (Khan, 1989).

1.2 Background and Present Status of the Problem

Due to lower water table position in dry season, water flows from storage reservoir, ponds, etc. through the soil by percolation or seepage. This problem is severe in case of water reservoir constructed on sandy and silty types of soils. Properties of a soil can be improved by additives. Soil improvement in some locality is, therefore, essential particularly for water retention purpose. Cow-dung, fly ash, rice-husks, lime, etc. are used for solving the water retention problem at locations where sandy soils dominate the permeability characteristics. Large-scale utilization of fly ash in geotechnical construction like embankments, road sub-bases, and structural landfill, as a replacement to the conventional earth material solves two main problems: elimination of solid waste and provision for needed construction material by conserving the soil. Use of industrials wastes have drawn the attention of researchers recently because of the low price of material cost in compare with the cost of other material. Fly ash being the most common pozzolanic material encountered in construction is a by-product of coal burning power plants. For soil improvement (i.e. strength, permeability characteristics) lime and cement are traditionally used. The unused fly ash is disposed into holding ponds, lagoons, landfills and slag heaps. Use of fly ash as a ground improvement soil admixture, when found technically viable constitutes a cost effective and environmentally beneficial alternative with considerably less capital investment. Therefore, a study needs to be carried out to evaluate the effectiveness of different additives on the coefficient of permeability of soils.

1.3 Objectives of the Present Research

The objectives of this study are:

1) To investigate permeability characteristics of additives treated soils.
2) To evaluate the effect of additives content and curing age (reaction time) on the permeability of treated soils.

1.4 The Research Scheme

The research was conducted using the following steps:

1) Two types of soils (clay of low plasticity and fine sand) were selected for this research work. One of them was collected from Porsha of Rajshahi (clay of low plasticity) and another from Mirpur of Dhaka (fine sand).

2) Index properties (specific gravity, grain size distribution and atterberg limits of the collected soil samples were determined using ASTM standards.

3) Commercially available lime and fly ash were used as additives in this study.

4) Samples were prepared by mixing soil with additives in percentage of 1%, 3% and 5% (by weight). Water was mixed thoroughly. After that the mixture were kept at room temperature for a period of 7 days and 14 days for hydration. Permeability tests were conducted on these hydrated soil samples to study the effect of curing age.

5) For the constant head arrangement, the specimen was connected through the top inlet to the constant head reservoir. The bottom valve was opened and allowed the water to flow into the permeameter. A reasonable steady flow of water was established. The quantity of flow was collected and elapsed time was measured. This process was repeated thrice.

6) In the constant-head permeability test laboratory set-up, the water supply at the inlet was adjusted in such a way that the difference of head between the inlet and the outlet was remaining constant during the test period. After establishment of a constant rate of flow, water was collected in a graduated cylinder for a known duration. This process was repeated thrice.

7) Falling head permeability tests were performed for soil Type-1(clay of low plasticity) and constant head tests were performed for soil Type-2(fine sand). Samples were prepared using both type of the soils adding lime and fly ash for an arbitrary selected moisture content. At first, permeability of the collected soil samples was determined. After that permeability of six types of treated soils were determined.

1.5 Thesis Layout

The complete research work for achieving the stated objectives is divided in number of chapters so that it becomes easier to understand the chronological development of the work. Briefly the contents of each chapter are presented below:

Chapter One describes the background of this study, objectives, methodology of the research. Finally, the organization of the thesis is summarized in this chapter. The dissertation is written in the following sequence:

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

Chapter Three presents the details of laboratory testing procedures and equipment used for determining the effects of lime and fly ash stabilization on the physical and engineering characteristics of the soils studied.

Physical and engineering characteristics of the untreated soils and soils stabilized with different cement and lime contents, as obtained from the laboratory investigation, are presented and discussed in Chapter Four.

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

Chapter Two

LITERATURE REVIEW

2.1 General

Considering the existing atmospheric condition and economy, lime stabilization is performed to increase strength, to improve permeability and erodibility of the side slope of embankment, to increase overall durability of the pond and earthen dyke so that the overall aquatic production would be increased. It has been proved that the use of lime is favorable for fish production. The aqua scientists use it extensively to improve the water quality. Liming increases the P^H of bottom mud and thereby increase the availability of phosphorus added in fertilizer (Boyed and Scarsbrook, 1974).

The objective of mixing additives with soil is to improve volume stability, strength and stress-strain properties, permeability, and durability. The development of high strength is achieved by reduction of void space, by bonding particles and aggregates together, by maintenance of flocculent structures, and by prevention of swelling. Good mixing of stabilizers with soil is the most important factors affecting the quality of results. Most commonly used stabilizers for improving the physical and engineering properties of soil are fly ash, lime, cement and foundry sand, bitumen and chemicals like calcium chloride, sodium chloride and sodium silicate.

In Civil Engineering purposes, field as well as laboratory experiments were stated by the Texas Highway Dept. in 1948. Development of theory, for the mechanism of lime stabilization was started from 1950 and the extensive study on mechanism of lime stabilization was done from 1960. In this regard, Eades and Grimes (1960), Kezdi (1979), Broms (1984) did the major work.

Lime stabilized soil are used to improve the engineering properties of soil in the field of highway, railway, airport construction. Brandl (1981) showed that lime could also be used for the construction of embankment slopes, in the fill of bridge abutments and retaining wall. He also showed that specific gravity of lime treated soil takes longer duration to get stabilized thus indicating long-term transformations of the chemical bound water and the gel. By adding lime, permeability of treated soil can be changed one or two decimal exponents.

Kezdi (1979) pointed out that lime stabilization is the addition of calcium in the form of CaO or $Ca(OH)_2$ which will reduce soil plasticity, increase strength and durability, decrease water absorption and swelling.

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

2.2 Mechanism of Lime Stabilization

Many researchers even now a day is working with the theory of lime stabilization. Indian Road Congress (IRC, 1973b) and Haunsmann (1990) have described the basic mechanism of soil-lime interactions. The basic mechanism of lime stabilization can be classified as follows.

- 2.2.1 Cat-ion Exchange
- 2.2.2 Flocculation or Agglomeration
- 2.2.3 Carbonation
- 2.2.4 Cementation

2.2.1 Cat-ion Exchange

Cat ion exchange capacity of soil depends upon the p^H value of the soil. Clay soils composed of different mineral and have different cation. Replace-ability of cation primarily depends on diffused double layers. The general order of replace-ability of common cation is $Na^+ < K^+ < Ma^{++} < Ca^{++}$. Mono-valent cations are usually replaced by divalent or multivalent cations.

The reaction of lime with three layers materials, which are montmorllinite, Kaolinite and illite, begin by the replacement of existing cat-ions between the silicate sheets with

Ca^{++}. Following the saturation of inter layer positions with Ca^{++}, the whole clay minerals deteriorate without the formation of substantial new crystalline phases 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. Cat-ion exchange capacity increases as the p^H of the soil increases.

2.2.2 Flocculation and Agglomeration

Flocculation of the soil particle occurs due to the mixing of soil with lime in presence of water. After cation exchange of soils and lime take place, agglomeration of the flocculated particle occurs. Kezdi (1979) pointed out that immediately after mixing, the soil structure starts to undergo a transformation. Flocculation and coagulation begin, and then the clay particles form much larger grains in the silt fraction. This, in turn, will modify the Atterberg limits and the compaction properties and so in practice the soil becomes much easier to handle in the course of earthwork. Diamond and Kinter (1965) suggested that the rapid formation of hydrated calcium aluminate (which is a cementing material) is responsible in the development of flocculation/agglomeration tendencies in the soil lime mixture.

2.2.3 Carbonation

When soil lime mixture is exposed to air, lime react with atmosphere carbon dioxide to form relatively weak cementing agents such as calcium carbonate or magnesium carbonate (Haunsmann, 1990). This reaction is the slowest of all the reactions involved in a soil-lime system and as in pozzolanic reaction, requires that the mixture must be thoroughly compacted. Eades et al. (1962) demonstrated that although carbonation takes place, the strength gain by virtue of cementation of soil grains with calcium carbonate is negligible.

2.2.4 Cementation

Long-term chemical reaction of lime with certain clay minerals (silicate and aluminates) of soil, in presence of water is referred to pozzolanic reaction. The minerals that react with lime to produce a cementing material are known as pozzolans. Possible source of silica and alumina in a typical soil include clay minerals are quartz, feldspars, mica and similar silicate-silicate minerals either in crystalline or amphorous in nature. When lime is added to the soil in presence of water causes an instantaneous rise in p^H of the molding water due to the dissociation of the $Ca(OH)_2$ in water.

Eades and Grims (1960) observed 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 cementitious material. This reaction will continue as long as $Ca\ (OH)_2$ exists in the soil and there is available silica. This mechanism may be referred as "Through Solution". Soil lime pozzolanic reaction usually does not appear until after long curing period and then only in cases where a high percentage of lime was added pozzolanics possess little or no cementitious value in finely divided form, but in the presence of moisture, it chemically reacts with calcium hydroxide at ordinary temperature to form components cementitious properties.

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

$$Ca^{++} + 2(OH)^{-} + SiO_2 \text{ (soil silica)} \longrightarrow CSH \qquad (2.1)$$

$$Ca^{++} + 2(OH)^{-} + Al_2O_3 \text{ (soil alumina)} \longrightarrow CAH \qquad (2.2)$$

In equations 2.1 and 2.2, CSH and CAH are cementitious products. The above reactions represented by Equations 2.1 and 2.2 are slow and long-term in nature. Moreover, these are more effective when the soil-lime mixture is adequately compacted. Cementation is, however, limited by the amount of an available silica. Increasing the quantity of lime added will increase strength only up to the point where all the silica of the clay is used up; adding too much lime can actually be counterproductive. This contrasts with cement stabilization, where strength continues to improve with the amount of admixture. Cementation on the surface of clay lumps causes a rapid initial strength gain, but further diffusion of the lime in the soil will bring about continued improvement in the longer term, measured in weeks or months.

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

2.3 Factor Affecting Permeability of Lime Stabilized Soils

Permeability of lime-stabilized soil depends upon various factors. In the following section some of the factors are discussed.

2.3.1 Index Properties of Soil

(1) Soil Types i.e. Shape and Size of the Soil Particle

Permeability varies approximately as the square of the grain size since soils consists of many different sized grains. Based on the experimental report on sand filter, (0.1mm and 0.3mm). Hazen (1892) suggested the following formula for permeability determination.

Permeability, $k = CD^2_{10}$ (2.3)

where, C is constant approximately equal to 100 when effective diameter D_{10} is in cm. and the formula is also useful for permeability determination of clean sand and Gravel.

Cassagrande (1937) stated the empirical relation for permeability determination of fine or clean sand with bulky grains as

$k = 1.40\ k_{0.85}\ e^2$ (2.4)

where, $k_{0.85}$ is the permeability at void ratio = 0.85

Different attempt has been taken to correlate permeability with the specific surface area of the particle. Kozeny (1927) gives one such relationship

$$k = \frac{1}{k_k \eta S_s^2} \frac{n^2}{1-n^2} \qquad (2.5)$$

where, k = Coefficient of permeability in cm/sec per unit hydraulic gradient n = Porosity

S_s = Specific surface of particle (cm^2/cm^3)

η = Viscosity (y-sec/cm^2)

k_k = Constant, equal to 5 for spherical particle

Carman (1956) developed the formula as

$$k = \frac{1}{C_{sT^2 S_s^2}} \times \frac{e^2}{1-e} \tag{2.6}$$

where, k = Absolute permeability

T= Tortuosity

S_s = Surface area per unit volume of soil solid's

C_s = Shape factor

This formula is worked well for coarse-grained soils such as some sand and silts and has serious discrepancies for clay soil. The discrepancies between the theoretical and experimental values are shown in Fig. 2.1 and Fig 2.2 based on consolidation permeability tests.

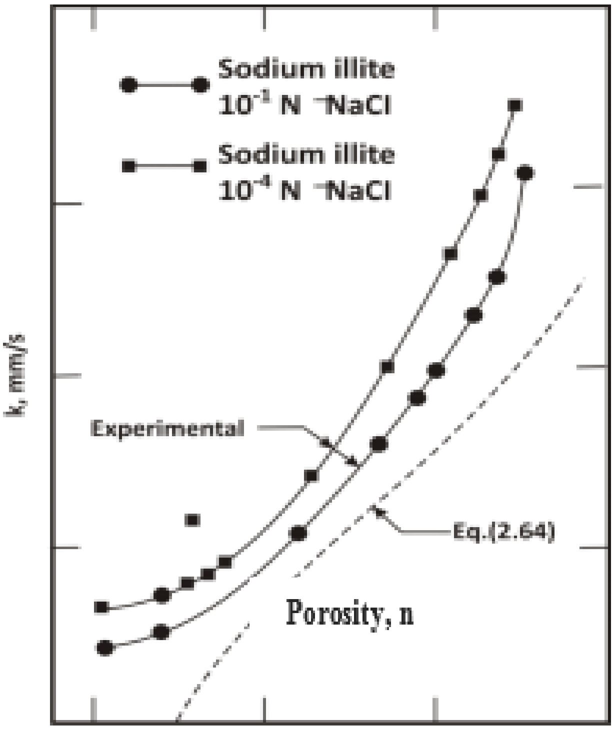

Fig. 2.1 Coefficient of permeability for sodium illite after Olsen (1961).

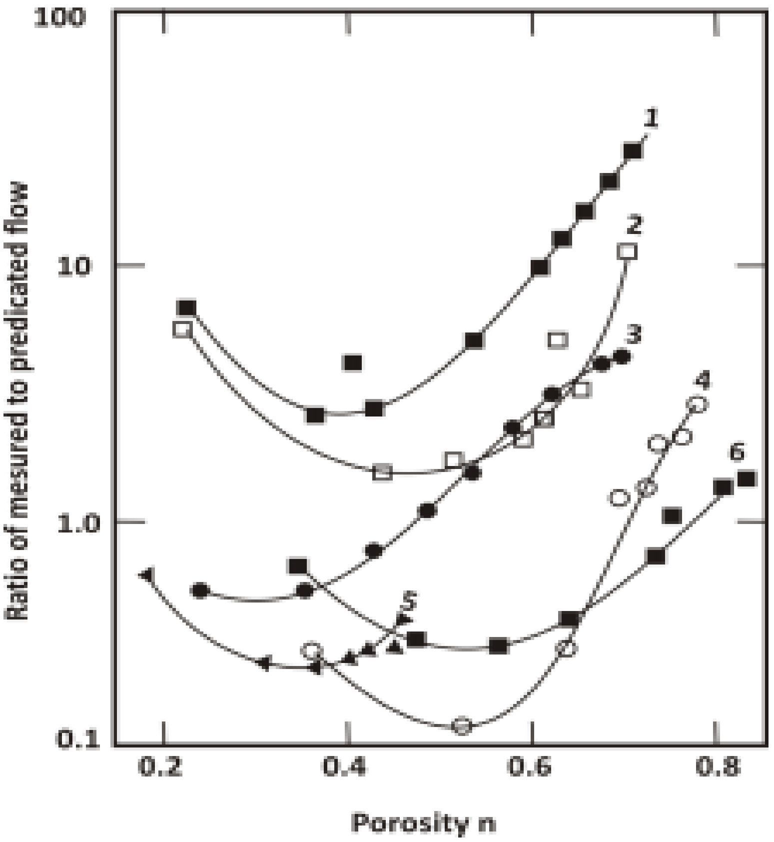

Fig. 2.2 Ratio of the measured flow rate to that predicted by the Kozeny-Carman equation for several clays. Curve I: Sodium illite, 10^{-1}N NaCI. Curve 2: Sodium illite, $1O^{-4}$N NaCl. Curve 3: Natural Kaolinite, Distilled water H_2O Curve 4: Sodium Boston blue clay, 10^{-1}N NaCI. Curve 5: Sodium Kaolinite, 1% (by Wt.) sodium tetraphospate. Curve 6: Calcium Boston blue clay, 10^{-4}N NaCl, after Olsen (1961).

(2) Void Ratio of the Soil

Permeability increases with of void ratio. For course grained soil

$$\frac{k_1}{k_2} = \frac{C_1 e_1^3}{1+e_1} \times \frac{1+e_2}{C_2 e_2^2} \qquad (2.7)$$

For coarse-grained soil C changes a little and can be

where, k_1 and k_2 are the coefficient of permeability write

$\frac{k_1}{k_2} = \frac{e_1^3}{1+e_1} \times \frac{1+e_2}{e_2^2}$ of a given soil at void ratio e_1 and e_2 respectively. Based on the mean hydraulic radius concept for the soil, the following relationship is obtained

$$\frac{k_1}{k_2} = \frac{e_1^2}{e_2^2} \qquad (2.8)$$

It has been found that a semi-logarithmic plot of void ratio versus permeability is approximately straight line for fine grained as well as coarse-grained soil. A typical plot worked on uniform Madison sand is presented in Fig. 2.3 based on constant head test.

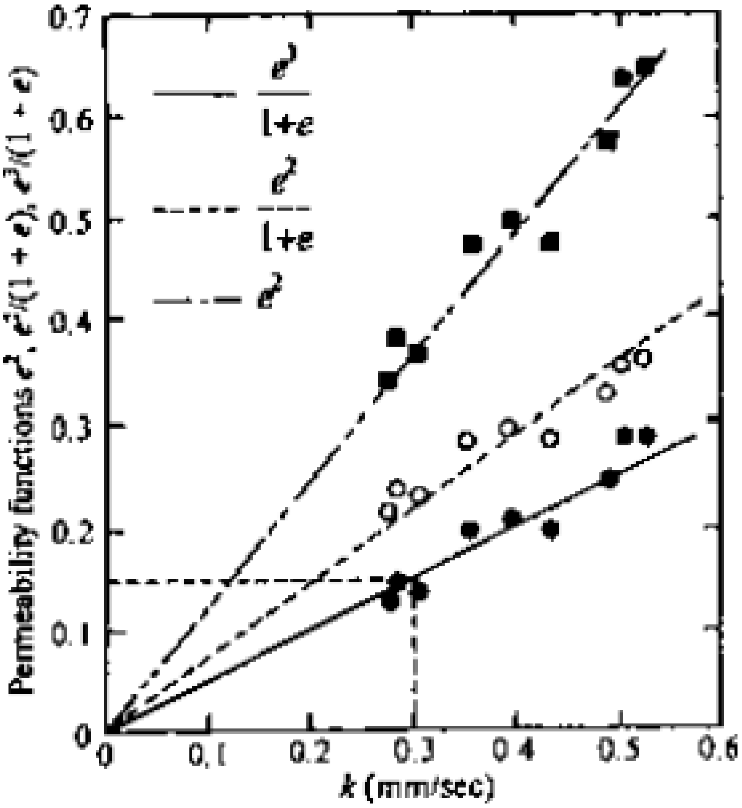

Fig. 2.3 Plot of k against permeability function after Das (1983)

(3) Degree of Saturation

When air is entrained in the voids, it reduces the degree of saturation and permeability decreases. Water contains dissolved air and it may get liberated while changing the permeability. So, permeability increases with the increase of degree of saturation. The variation of the value of permeability (k) with degree of saturation Madison sand as shown in Fig. 2.4 and Fig. 2.5.

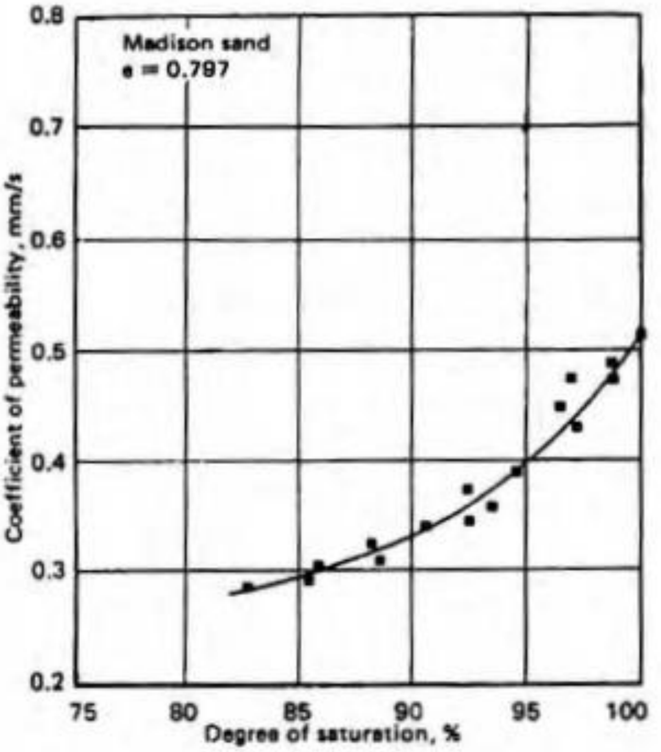

Fig. 2.4 Influence of degree of saturation on permeability of Madison sands after Mitchell et a1. (1965)

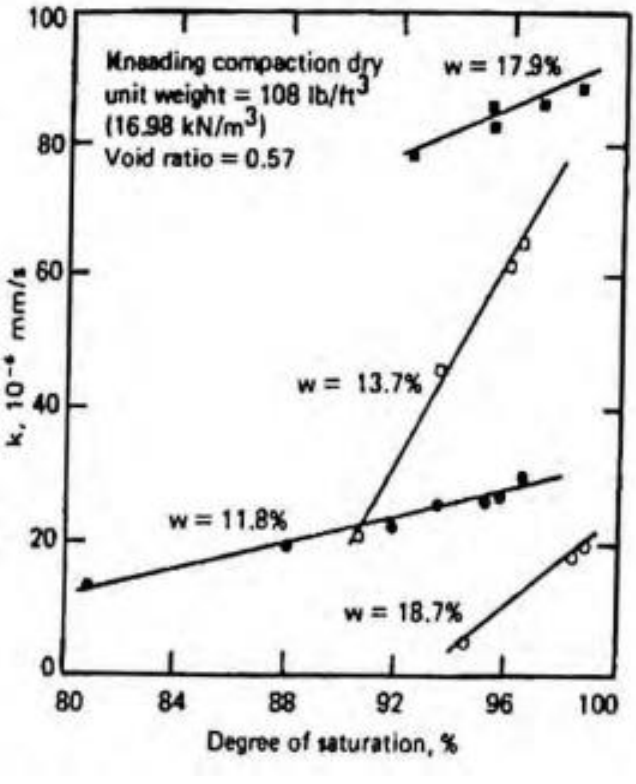

Fig. 2.5 Influence of degree of saturation on permeability of compacted silty clay. (Note: Samples aged 21 days at constant water content and unit weight after compaction prior to test) Redrawn after Mitchell et al. (1965).

(4) Compaction of Soil Particles

For sands and silts this is not important; however for soils with clay minerals, this is one of the most important factors. Permeability in this case depends on the thickness of water held to the soil particles, which is a function of cat ion exchange capacity, valence of the cat ions, etc. Other factors remaining the same, the coefficient of permeability decreases with increasing thickness of the diffuse double layer.

(5) Soil Structure

Fine-grained soils with a flocculated structure have a higher permeability then those with a dispersed structure. This is demonstrated in Fig. 2.6 for the case of silty clay. The test specimen was prepared to constant dry unit weight by kneading compaction. When moisture content increases the soil becomes more dispersed. With increasing degree of dispersion, the permeability decreases.

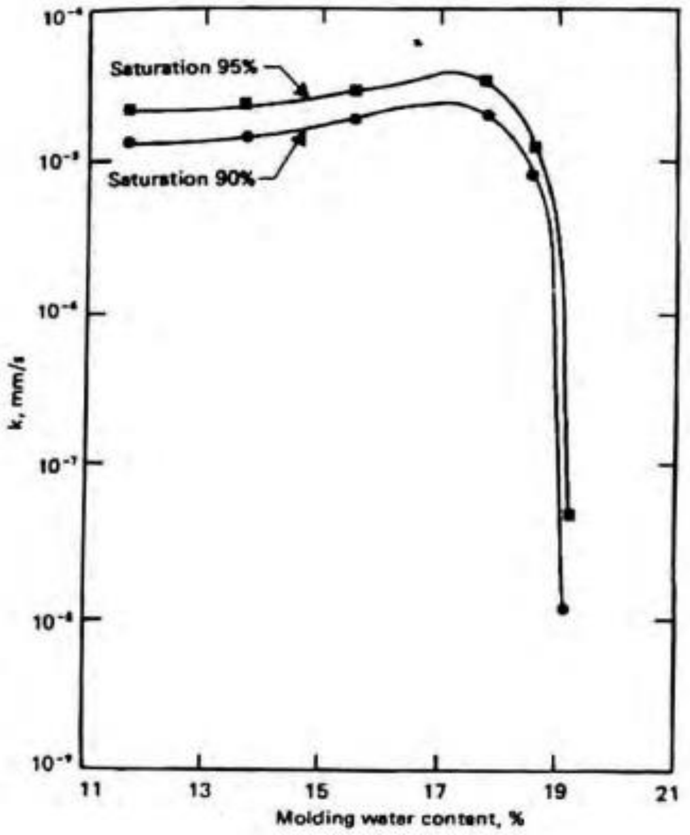

Fig. 2.6 Dependence of permeability on the structure of silty clay. Redrawn after Mitchell et al. (1965).

(6) Property of the Permeate

Permeability is directly proportional to the unit weight of water and inversely proportional to its viscosity. Through the unit weight of water does not change much with the change in temperature there is a great variation in viscosity with temperature. When other factor remaining same, permeability at a given temperature (k_T) is given by

$$k_{20} = k_T \frac{\eta_T}{\eta_{20}} \tag{2.9}$$

η =Viscosity of water

Muskat (1937) pointed out that a more general co-efficient of permeability called the physical permeability k_p is related to the Darcy's co-efficient of permeability k as follows:

$$k_p = k \frac{\eta}{\lambda_w} \tag{2.10}$$

In any soil, kp has the same value at temperature as long as the void ratio and the structure of the soil skeleton are not changed for all fluids.

(7) Organic Matters Present in the Soil

Presence of organic matter influences the permeability. Organic matter has the tendency to move towards critical flow channels and choke them up and thus decreasing the permeability. Rodriguez et al. (1988) noted that lime has effect on highly organic soil or soils without clays.

(8) Adsorbed Water with Soil Particle

Adsorbed water surrounding the fine soil particles are not free to move thus reduces the effective pore apace available for the passages of water. Casagrande's crude approximation is to take 0.1 as the void ratio occupied by the adsorbed water hence it is the square of the net void ratio (i.e., void ratio = e-0.1).

(9) Effect of Stratified Soil Layer

In general, natural soil layer is stratified. Their bedding planes may be horizontal, inclined or vertical. Assuming each layer homogeneous and isotropic, it has own value of co-efficient of permeability. The average permeability to the whole deposit will depends upon the direction of flow with relation to the direction of the bedding planes. Range of permeability for various soils are given in Table 2.1.

Table 2.1 Range of Permeability for Various Soils after Islam (2001)

Soil	Permeability Coefficient, k (cm/sec)	Relative Permeability
Coarse gravel	Exceeds 10^{-1}	High
Sand, clean	10^{-1} to 10^{-3}	Medium
Sand, dirty	10^{-3} to 10^{-5}	Low
Silt	10^{-5} to 10^{-7}	Very low
Clay	Less than 10^{-7}	Impervious

2.3.2 Lime

2.3.2.1 Lime Content

Since lime reacts with soil to form some new compounds, which improve the engineering properties of soil, lime content is an important factor. The usual content of hydrated lime for different types of soil are given in Table 2.2 and 2.3.

Table 2.2 Usual content of hydrated lime in different soil (% by weight of dry soil to lime) after Naasra (1987)

Soil Type	Stabilization (lime %)
Crushed rock	Not recommended
Well-graded clayey gravel's	2
Pure sand	Not recommended
Silty sand	Not recommended
Clayey sand	2-4
Clayey silt	2-4
Silty Clay	2-6
Plastic clay	3-9
Highly plastic clay	3-9
Organic soil	Not recommended

Table 2.3 Usual content of hydrated lime in different soil (% by weight of dry soil) Ingles and Metcalf (1972)

Soil Type	Stabilization (lime %)
Crushed rock	Not recommended
Well-graded clayey gravel's	3
Sands	Not recommended

Sandy clay	5
Silty clay	2-4
Plastic clay	3-8
Highly plastic clay	3-8
Organic soil	Not recommended

From the above table, it can be seen that the usual content for lime for silty sand is 2-4% and for sandy clay it is 5%. It can be also observed that lime percent increase as the soil become coarser to fine grained from stabilization viewpoint. Brandl (1981) pointed out that the more cohesive and reactive the untreated soil is, the more increases the permeability of the mixture according to the immediate flocculation. The maxima are gained at lime amounts between 1% for inactive silt to 10% for active clays. Kezdi (1979) classify the soil from lime stabilization viewpoint into three categories that is represented in figure 2.7.

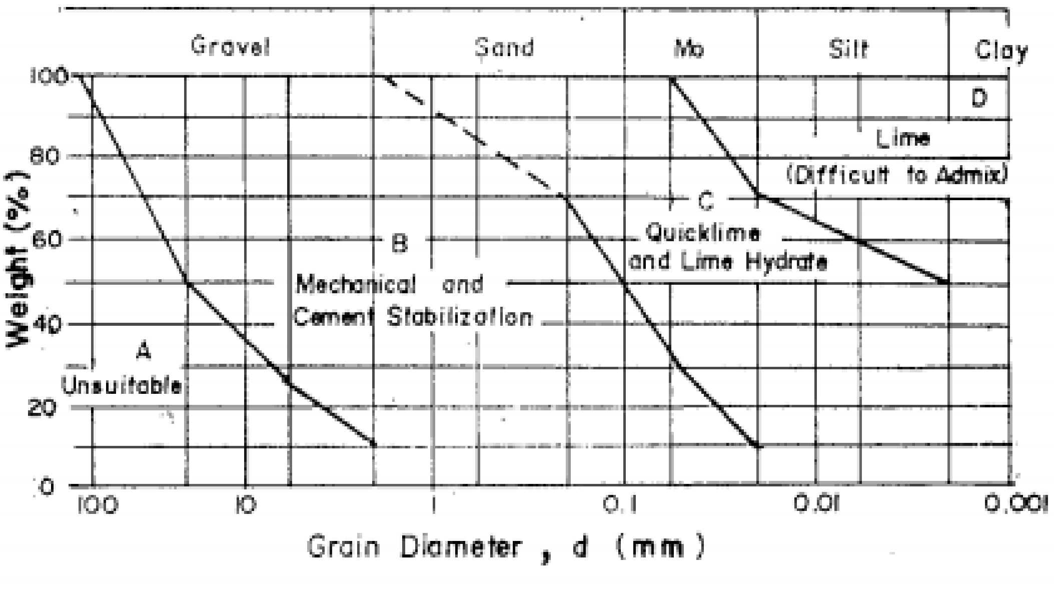

Fig. 2.7 Lime stabilization ranges of grain distribution after Kezdi (1979)

From the figure, it is observes that in range-**A**, no stabilization is possible since the available equipment simply cannot work these course materials. In range **B**, the soil behavior is governed mainly by the grain distribution itself. In range **C**, mechanical stabilization is not fruitful by hydraulic binders, cement, etc. It would only be economical in the case of clay soil.

Optimum lime content: May be defined as the lime content at which the percentage of such additional lime increment will not produce appreciable increase in the plastic limit. According to Diamond and Kinter (1965), 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 increases of strength.

From the literature review, it can be concluded that lime percent varies soil to soil. To achieve the minimum and the maximum permeability, strength and specific gravity (i. e. overall durability of the structure) optimum lime contents is the lime content by which the maximum strength and the maximum or the minimum permeability of soil can be achieved.

2.3.2.2 Fat Lime

Lime may be divided into three categories as follows:

1) Fat Lime: This lime known as fat lime because it increases 2 to 2.5 times in volume, when slaked. It contains about 95% calcium oxide and about 5% other materials inform of impurities. This lime is also sometime known as pure lime, white lime or high calcium lime. It is obtained by burning lime stone which containing mostly calcium carbonate in atmosphere, carbon dioxide is driven out, leaving back calcium oxide (CaO), known as quick lime. Fat lime is obtained by slacking quick lime. Setting of this lime is entirely dependent upon the atmosphere oxygen. For setting, this lime absorbs carbon dioxide (CO_2) from atmosphere and after chemical reaction gets converted into calcium carbonate ($CaCo_3$), which is quite hard substance, insoluble in water. Setting and hardening actions of this lime are very slow.

2) Hydraulic Lime: This lime has the property of setting under water. It is obtained by burning limestone, containing lot of clay and other substances that develop hydraulicity. Hydraulicity of this lime depends upon the amount of clay and type of clay present in it. Silica, alumina and or iron oxide are present in chemical combination with calcium oxide. Depending upon the amount of clay (silica and alumina) present, hydraulic limes may be further divided into following three categories.

Feebly Hydraulic Lime: It contains clay (silica, alumina or iron oxide) less than 15%. The usual percentage of these constituents varies between 5% to 10%. On slaking, it increases in volume by very small amount. It slacks slowly.

Moderately Hydraulic Lime: This lime contains 15 to 30% silica and alumina. On slacking, it increases in volume by very small amount. It slacks slowly.

Eminently Hydraulic Lime: This lime contains 25 to 30% silica and alumina. It resembles very much to Portland cement in chemical composition. Slaking of this lime is hardly noticeable. Its initial setting starts after two hours and the final setting within 48 hours.

3) Poor Lime: This lime contains 30% of clay. It slacks slowly. It does not dissolve in water. It forms a thin plastic paste with water. This lime is also known as lean lime or improves lime and hardens and sets very weak slowly.

Boyed (1990) pointed out that liming material most frequently used is agricultural limestone, which is prepared by finely crushing limestone. Agricultural lime (Calcium carbonate) is not suitable for stabilization. Dolomite lime is usually not as effective as calcium lime. In order to give a common quantitative base, lime content is expressed as equivalent 100% pure hydrated lime. On a mass basis pure quick lime is equivalent to 1.32 unit of hydrated lime. Limestone is calcite ($CaCO_3$), dolomite [$CaMg(CO_3)_2$] or blend of these two substances. There is some confusion to the actual composition of locally available liming material. Basic slag, a by-product of steel making, contains calcium carbonate and phosphorous, so it is both a liming material and fertilizer. Blast furnace slag, also a by-product of steel making, is comprised largely of calcium silicate, but these calcium silicate slag's are not suitable for fish culture in pond.

Lime rapidly reacts with any available water producing hydrated lime, releasing considerable amount of heat. The water content of common slurry lime can range from 80-100%. Table 2.4 represents the property of hydrated, quick and slurry lime.

Table 2.4 Properties of lime after Naasra (1986)

Parameters	Hydrated lime	Quick lime	Slurry lime
Composition	$Ca(OH)_2$	CaO	$Ca(OH)_2$
Form	Fine Power	Granular	Slurry
Equivalent $Ca(OH)_2$/ Unit Mass	1.00	1.32	0.56 to 0.33
Bulk Density (kg/m_3)	450 to 560	1050	1250

According to Mateos (1964), lime can be divided chemically into two categories.

Hydrated lime: It is divided into three groups

Calcite [$Ca(OH)_2$], commonly known as hydrated lime, slaked lime or builder's lime. Dolometric monohydrated, Ca $(OH)_2$+MgO and Dolomitic dehydrate $Ca(OH)_2$ + $Mg(OH)_2$.

Quick lime: It is divided into two groups

Calcite (CaO) and Dolomitic [CuO+MgO]

On chemical analysis of calcite hydrated lime has found 75.677% quick lime and 24.33% water. Dolometric monohydrate lime has 15.79% water and 84.21% dolomitic quicklime. Dolomitic monohydrate lime has 27.27% water and 72.73% dolomitic quicklime. Table 2.5 shows the different constituents of quicklime and hydrate lime after Ingles and Metcalf (1972) and Table 2.6 shows different properties for soil stabilization after Ghos (1987).

Table 2.5 Requirements that must be meet by limestones on natural calcium carbonates in order to provide stabilizing lime after Ingles and Metcalf (1972)

Property	Quicklime (CaO)	Hydrate 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 2.6 Properties of the Theoretically Pure Lime after Ghos (1987)

Chemical name	Quicklime		Hydrated Lime	
	Calcium or Calcium oxide	Magnesia or magnesium oxide	Calcium hydroxide	Magnesium hydroxide
Chemical formula	CaO	MgO	$Ca(OH)_2$	$Mg(OH)_2$
Crystalline formula	cubic	cubic	Hexagonal	Hexagonal
Melting point	2570^0C	2800^0C	-	-
Decomposition point	-	-	580^0C	745^0C
Boiling point	2850^0C	3600^0C	-	-

Molecular weight	36.09	40.32	74.1	58.34
Specific gravity	3.40	3.65	2.34	2.40

It shows from the above discussion that for soil stabilization .Hydraulic or Poor lime will be best variety to work. If they are used in the hydrated form, duration of effective curing time will be small.

2.3.3 Water

Distilled water is preferred for any research work. When requirement of the water is high, portable water is used for lime stabilization. Water containing organic matter should be avoided. Seawater may be used, where bituminous seal do not exist.

2.3.4 Age Effect on Lime Stabilized Soil

It has been stated earlier that effect on lime stabilized soil occur slowly. Lime reacts with soil and the gain of strength is higher at initial stage of curing and the rate of gain of strength reduces as the goes on. Arman and Muhfakh (1972) stated that lime has an initial reaction with soil taking place during first 48-72 hours after mixing and the secondary reaction starts after that period and continues.

The rate of gain of strength is time dependent and depends on soil types. For some types of soil, the rate of gain of strength with curing time is high but for others the rate is slow. Brandl (1981) observed that the time dependent increase in shear strength is approximately linear with the logarithm of time. From his study, it is seen that the permeability decrease with curing time. With increasing curing time, the mineral particles are cemented within the soil-lime-mixture; the fine skeleton is embedded partly within a gelatinous intermediate mass, hardening products of the binder grow into the voids of soil aggregate changing the void structure.

2.4 Effect of Lime Treatment on the Physical and Engineering Properties of Soil

Lime reacts with soil silica or alumina in the presence of water causing the change of the physical properties of soil. The chemical-physical reactions in the soil are rather complex and can be generalized only in some cases. Some change of the soil parameters being interesting for practical application. Some of the important properties of soil, which are changed due to the stabilization of soil with lime, are atterberg limits, permeability, strength, compressibility, stress strain character, volume change, shear strength etc. In the following sections the various physical and engineering properties of lime stabilized soils are reviewed.

2.4.1 Effect of Lime on Atterberg Limits

Plasticity is the property of the soil, which allows to be deformed rapidly without ruptures, without elastic rebound and without volume change. Due to the addition of lime, structural transformation and flocculation begin immediately. This amount of colloidal clay and the chemo-physical activity of the soil, the more likely is the decrease of liquid limit (Figs. 2.8 a, b and c). Silts reaches in natural calcium show an increase. The plasticity limit without exception, only with silts and sand reach in calcium and dolomite the value in almost constant. Too much lime causes the transgression of the point of reversal (liquid limit, w_L and plastic limit, w_P). Generally both liquid and plastic limits increase with time, because the attractive forces between the soil particle increase and the absorbed water film is influenced.

Thus the plasticity of reactive soils is reduced considerably easing the workability on construction site. Only inactive soil becomes even more plastic.

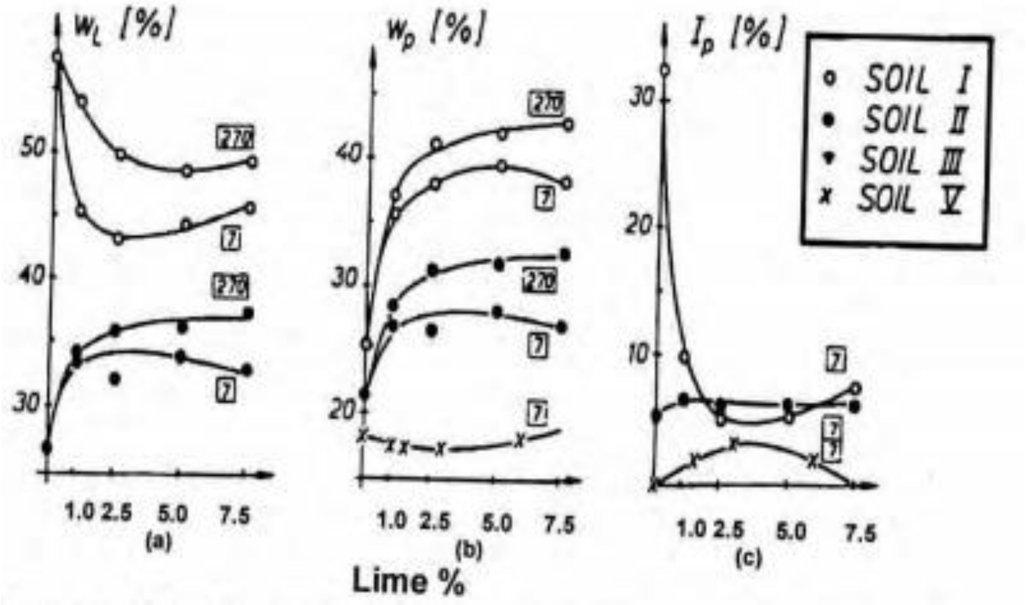

Fig. 2.8 Alteration of atterberg limit by adding lime; Reaction time (curing age) in days as parameter (drawn in rectangle): period from stabilization till test performance after Brandl (1981).

Generally soils with high clay content or soils exhibiting a high initial plasticity index require greater quantity of lime for achieving the non-plastic condition. If it can be achieved at all, the amount of reductions in plasticity index varies with the quantity and types of lime and also of soil (IRC, 1976).

Ahmed (1984) pointed out that plastic limit of a silty clay increases with the increase of lime content, while liquid limit and plasticity index decreases with increasing lime content (Fig 2.9). Shrinkage limit and linear shrinkage of a clayey soil are also affected by addition of lime. The shrinkage limit increases while linear shrinkage reduces as the lime content increases (IRC, 1976). Hilt and Davidson (1960), Pietsch and Davidson (1962) pointed out that the plastic limit of soil generally increases with the addition of small amount of lime until certain critical lime content called “lime fixation point”. Rodriguez et al. (1988) stated lime generally reduce the plasticity index of high plastic soil but has little influence on the plasticity index of the low plastic soils.

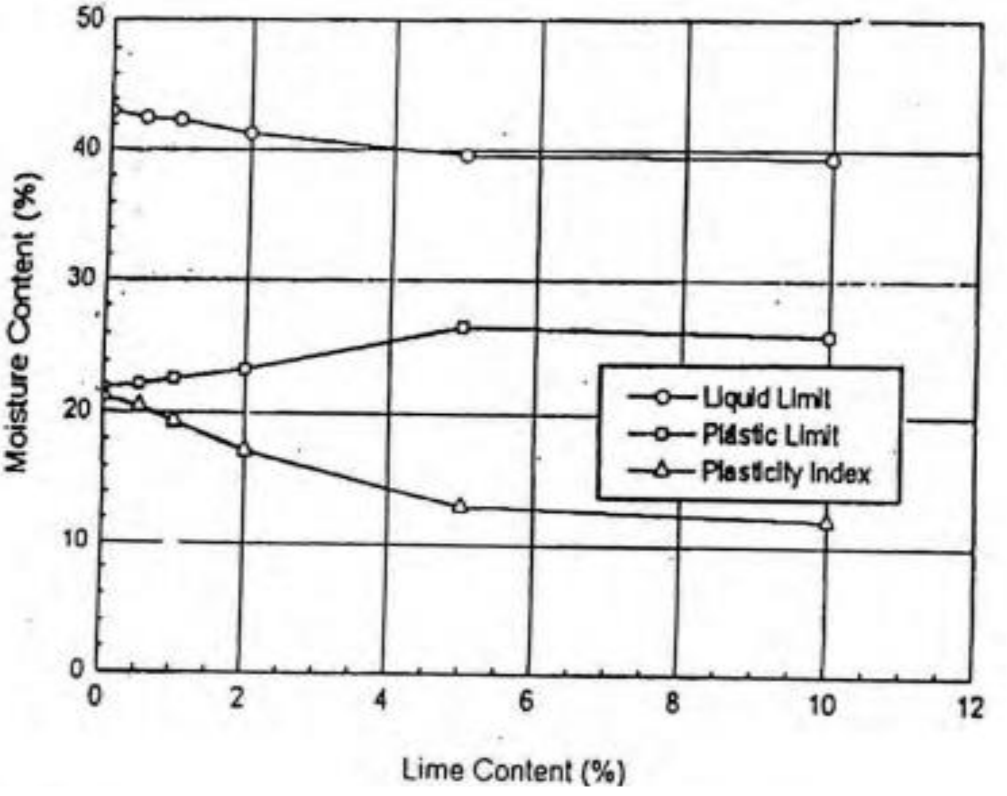

Fig. 2.9 Effect of lime content on Atterberg limits of a silty clay soil reported from Ahmed (1984).

2.4.2 Effect of Lime on Specific Gravity

Specific gravity of any substance is defined as the unit weight of the material divided by the unit of distilled water at 4^0C.Due to addition of lime, specific gravity changes. Brandl (1981) stated that changes of specific gravity are time dependent, which indicates long-term transformations of the chemical bound water and the gel; the formation of new minerals is responsible too. The changes of specific gravity are presented for in Fig 2.10.

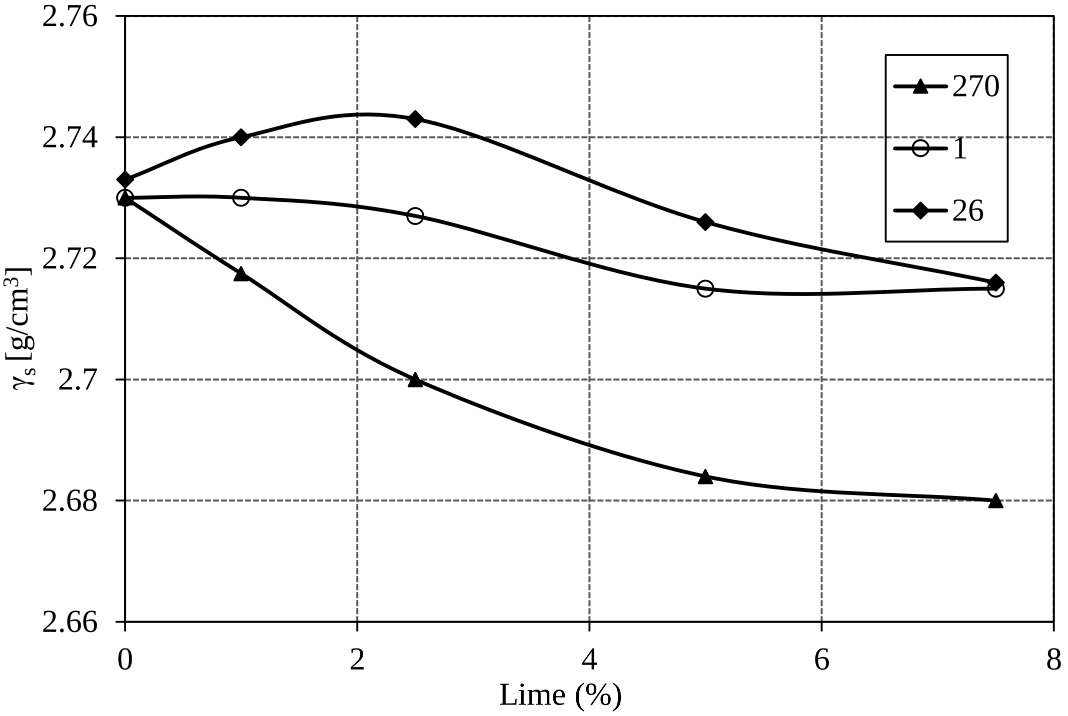

Fig. 2.10 Variation of Specific gravity with lime content after Brandl (1981); Reaction time (curing age) in days as parameter (drawn in rectangle): period from stabilization till test performance. Where γ_s is the specific gravity.

2.4.3 Effect of Lime on Permeability

Permeability is the property of soil, which permits the passage of water through its interconnected void space. Townsend and Klyn (1970) pointed out that the permeability of soil increases due to addition of lime. While conducting the experiment with heavy clay, they observed a marked increase in permeability, while for silty clay, erratic or no change of permeability was observed. Broms and Boman (1977) show 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 enables the fluid to flow more readily in between the clay and corresponding change in grain size distribution.

Variation of permeability with lime stabilized soil with lime percent for different curing period as present by Brandl (1981) is shown in Fig 2.11. From the figure, it can be seen that the permeability for silt A-4 (symbol used were, solid circle (-•-), (sand 15%, silt 73%, clay 12%) increases with lime upto 1% lime content and then decreases.

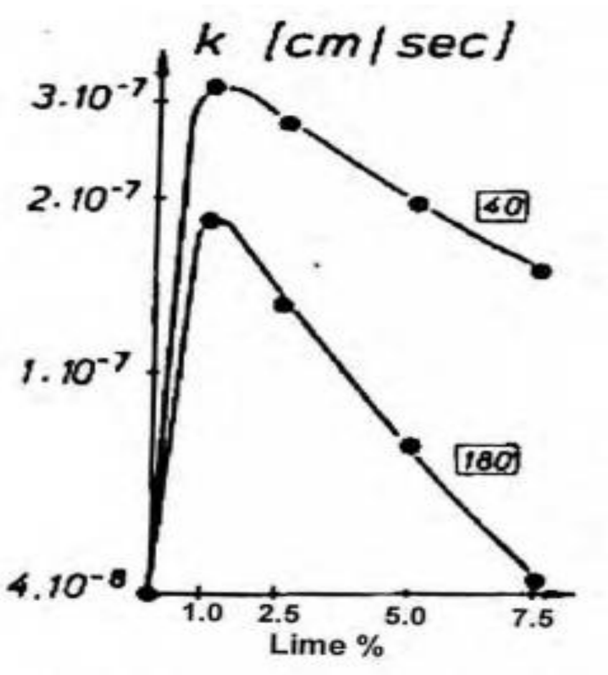

Fig 2.11 Alteration of Permeability coefficients by adding lime; reaction lime (curing age) in days as parameter (drawn in rectangle): period from stabilization till test performance after Brandl (1981). Where, k = water permeability, cm/sec.

Brandl (1981) pointed out (Fig. 2.11) that the more cohesive and more reactive the untreated soil is, the more increase the permeability of the mixtures according to the immediate flocculation. The maxima are gained at lime content between 1% for inactive silts to 10% for active clays. He also pointed that the change of permeability over one or two decimal exponent is easily possible. He also pointed out that with increasing curing time the mineral particles are cemented within the soil lime mixtures. The fine skeleton is embedded partly within a gelatinous intermediate mass; hardening products of the binders grows into the voids of the soil aggregates the void structure. Additionally, the stabilized soil grains surround themselves with a wider film of bound water in a way that finally, the remaining void space becomes smaller and thus the permeability decreases with curing time marching on.

2.5 Fly Ash

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

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

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

1n 1986 some 65 to 70 million metric tons fly ash were produced in the United States alone, only 15 to 20% of this massive amount was used constructively: less than half of that was used in the manufacture of concrete. The rest is pumped in slurry form into lagoons or is conditioned by the addition of 10 to 15% water and disposed of as more or less engineered landfills.

Now a day’s coal is more and more frequently adopted as fuel for electric power plants. On the basis drawbacks of its use is the large quantity of produced ashes (up to 15% of the weight of coal). 1n the past, the coal ashes were disposed into abandoned open-pit mines or stream valleys; at present, it is becoming more and more necessary to use them for embankments and hydraulic or compacted fills.

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

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

Add cement or lime to stabilize the fly ash.

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

Use fly ash in the containment of toxic wastes.

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

2.6 Properties of Fly Ash

2.6.1 Chemical Composition and Reactivity

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

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

(1) Coal type and purity.

(2) Degree of pulverization.

(3) Boiler type and operation.

(4) Collection and stockpiling methods.

(b) There is no single chemical or physical property which gives a reliable indication of the pozzolanic reactivity)"of fly ash. Cementitious calcium silicate and calcium aluminosilicate hydrates are formed when the glassy components of the fly ash ($3A_2lO_2$) SiO_2 or "mullite") react with water and lime. Critical to the pozzolanity of fly ash are conditions such as

(1) Amount of silica and alumina in the fly ash.

(2) Presence of moisture and lime.

(3) Fineness of the fly ash (surface area).

(4) Low carbon content.

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

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

2.6.2 Engineering Properties

The specific gravity of the ash particle ranges from 1.9 to 2.5, which is below that normally measured for soil solids. Some of the ash particles may actually float if they consist of hollow glass spheres (cenospheres); these have numerous industrial applications. The average grain size D_{50} of fly ash is likely to be in the range of 0.02 to 0.06mm. Fly ash is nonplastic and in a dry state as collected, completely cohesionless. This lack of cohesion makes no hardening fly ash highly erodible. In a moist, unsaturated state, surface tension of the pore water gives fly ash an apparent cohesion; if and when pozzolanic reaction occurs, considerable unconfined compressive strength is observed, increasing with age. The friction angle as measured in consolidated drained triaxial tests is typically on the order of 30^0, but values as low as 20^0 and as high as 40^0 have been reported. As a guide, compacted ash may have a dry density anywhere between 1.2 and 1.9t/m^3 and a corresponding optimum moisture content ranging from 30 down to 15%; however, more extreme values are also reported in the literature, such as γ_{dmax} = 700 kg/m3 and w_{opt} = 60%.

Low compacted density points to a potential advantage in the use of fly ash as backfill or embankment material; Low unit weight means low overburden pressures and combined with a high friction angle, also low earth pressures.

EPRI (1986) reports that the compression index Cc of fly ash can range from 0.05 to 0.37 for initial loading in recompression, these values are much lower: 0.006 to 0.04. The compressibility of compacted ash must rate as small when compared with clayey soils. Compacted dry fly ash may swell upon wetting if subjected to vertical pressures less than that equivalent to 0.5 to 1 m or fly ash fill. It was also reported that 11 to 14.5% free swell for a particular ash tested.

Table 2.7 Permeability of a fly ash treated with different coal type (EPRI, 1986).

Type of treated Fly Ash	Coefficient of Permeability, cm/s
Fly ash treated with Bituminous	10^{-4} to 10^{-3}
Fly ash treated with Subbituminous	10^{-5} to 3×10^{-6}
Fly ash treated with Lignite	9×10^{-6} to 10^{-7}

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

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

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

2.7 Fly Ash Stabilized with Lime, Cement and Aggregate

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

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

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

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

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

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

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

Chapter Three

METHODOLOGY

3.1 Introduction

As mentioned earlier, the permeability of soil depends upon various factors, such as size and shape of the particle, particle orientation, temperature of liquid, soil compaction, etc. In addition to this, permeability of treated soil depends on additive types, additive content, procedure of mixing, curing time (reaction time) and procedure of curing, etc. The main objective of this research is to investigate the influence on the permeability of treated soil of lime and fly content, reaction time, etc.

The laboratory investigations carried out on the untreated two soil samples collected one of them from Porsha of Rajshahi (clay of low plasticity) and another from Mirpur of Dhaka (fine sand) have been described in details in this chapter.

3.2 Sampling and Collection of Soil Samples

The study area of Porsha is located in the Barind region, which lies within Naogaon district .It lies between 24º55´-25º15´N latitude and 88º22´-88º40´E longitude. At present many parts of the Barind Tract suffer from problems of water scarcity. Once before 1960 there was no tube wells and even dug wells in many of the villages of the area, particularly in the north-western part of the Barind Tract and the water demand was met up by the surface reservoirs like Ponds, Beels etc.

Disturbed soils from two selected sites, namely Porsha of Rajshahi and Mirpur of Dhaka were collected for the present investigation. The site locations are shown in Fig 3.1. Soil sampling was carried out according to the procedure outlined in ASTM 0420-87. For each location approximately 2 m by 2 m area was excavated to a depth of 2 m to 3 m using hand shovels. Proper care was taken to remove any loose material, debris, coarse aggregates and vegetation from the bottom of the excavated pit. Disturbed samples were collected from the bottom of the borrow pit through excavation by hand shovels. All samples were packed in large polyphone bags covered by gunny bags and were eventually transported to the Geotechnical Engineering Laboratory of Bangladesh University of Engineering and Technology, Dhaka.

Fig. 3.1. Map of Bangladesh showing study locations

3.3 Laboratory Testing Programme

Embankment failures happen continuously throughout Bangladesh. Coastal area, a comprehensive laboratory investigation programme was undertaken in order to examine the physical, index and engineering characteristics of the base soils (i.e., untreated soils). Fly ash and lime were used as additives. Both Soil Type-1 and Soil

Type -2 were treated with fly ash and lime content in percentages of 1, 3 and 5 (by weight). Detail of laboratory test performed on the two types of soils are shown in Table 3.1.

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

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

(iii) The following tests on soil type-1 and soil type-2 without any treatment. Soil type-1 and soil type-2 treated with three different lime and fly ash contents (1%, 3% and 5%) were carried out.

a) Constant head permeability test

b) Falling head permeability test

Table 3.1 Detail of laboratory test performed on the two types of soils.

Types of Test	Sample	No. of tests	
		Soil Type-1	Soil Type-2
Specific Gravity	Untreated	1	1
Liquid Limit and Plastic limit	Untreated	1	
	Treated	1	
Grain size distribution	Untreated	1	1
Permeability test	Untreated	1	1
	Treated	9	9

3.4 Physical and Index Properties of Untreated Soils

The samples collected from the field were disturbed samples. These samples were then air-dried and the soil lumps were broken carefully with a wooden hammer so as to avoid breakage of soil particle. The required quantities of soil were then sieved through sieve No.40. Table 3.2 presents the standard test procedures were followed in determining the physical and index properties of the untreated soils. The soils were classified according to MIT Soil Classification System.

Table 3.2 List of conducted laboratory tests with their ASTM designations.

Name of Test	ASTM Standards
Specific Gravity	ASTM D854
Liquid Limit, plastic limit and plasticity index	ASTM D4318
Grain size distribution	ASTM D422
Permeability test	ASTM D2434-68

3.5 Index Property Tests on Stabilised Soil Samples

Liquid limit, plastic limit, plasticity index of samples of the two soils (from Porsha of Rajshahi and another from Mirpur of Dhaka) treated with fly ash and lime were determined. Fly ash and lime were used as additives. Fly ash and lime were used in percentages of 1, 3 and 5. Liquid limit, plastic limit and plasticity index of the samples were carried out on air-dried pulverized samples. The required quantities of pulverized soil were sieved through sieve. No. 40 (0.425 mm). The fly ash and lime treated soils were compacted following ASTM D558 method. The compacted samples were cured in moist environment for 7 days and air-dried. The air-dried samples were pulverized to pass through no. 40 sieve. Liquid limit, plastic limit and plasticity indexes of the stabilized samples were determined following the standard procedure outline in ASTM 4318.

3.6 Constant Head Permeability Test

The following initial measurements in centimetres or square centimetres was taken and recorded on the data sheet; the inside diameter (D) of the permeameter; the length (L) between manometer outlets; the depth (H_1), measured at four symmetrically spaced points from the upper surface of the top plate of the permeability cylinder to the top of the upper porous stone or screen temporarily placed on the lower porous plate or screen. This automatically deducted the thickness of the upper porous plate or screen from the height measurements used to determine the volume of soil placed in the permeability cylinder. A duplicate top plate was used containing four large symmetrically spaced openings through which the necessary measurements were made to determine the average value for H_1. The cross-sectional area, A, of the specimen were calculated.

The chamber cap was removed of the permeameter and put one porous stone in the base of the chamber. A scoop or funnel was used to pour the sand specimen into the chamber. A technique was used to ensure that the soil was placed in a uniform manner. Sample weight were determined, a pan with dry sand was weighed. The desired sample was removed and the pan and sand was weighed again. The difference in weights was the sample weight. The upper porous stone on the specimen and the compression spring on the porous stone was placed. Then the chamber cap and sealing gasket in place were put and secured it firmly with the cap nuts. The length of the specimen was measured and recorded it. The constant-head reservoir was assembled and it was adjusted to the desired height above the outlet of the permeameter to create the desired head. The inlet valve was closed to the permeameter. The constant-head reservoir with desired water was filled. The outlet valve of the permeameter was opened and the valves were closed to the piezometers. The inlet valve was opened to saturate the specimen. The valve was opened slowly to prevent a sudden inrush of water that liquefied the soil sample. The height between the outlet tube and the water in the constant head reservoir was measured. The inlet and outlet valves were opened. When an equilibrium flow condition was established, a graduated cylinder was placed to receive the outflow and a timer was started. When a sufficient quantity of water was obtained in the graduated cylinder, graduated cylinder was removed from the stream of water and the timer was stopped. The quantity of water obtained and the time required to obtain it were record. The temperature of the water was also recorded. During the time that the water was being collected in the previous step, the piezometers were observed and a set of average readings were recorded. This process was repeated thrice. Constant head permeability is calculated using the following equation:

$$k_T = \frac{QL}{Ath} \tag{3.1}$$

where, k_T= Coefficient of permeability at temperature T, cm/sec.

L = Length of specimen in centimeters.

t = Time for discharge in seconds.

Q = Volume of discharge in cm^3 (assume 1 mL = 1 cm^3).

A = Cross-sectional area of permeameter (= $\frac{\pi D^2}{4}$, D= inside diameter of the permeameter).

h = hydraulic head difference across length L, in cm of water; or it is equal to the vertical distance between the constant funnel head level and the chamber overflow level.

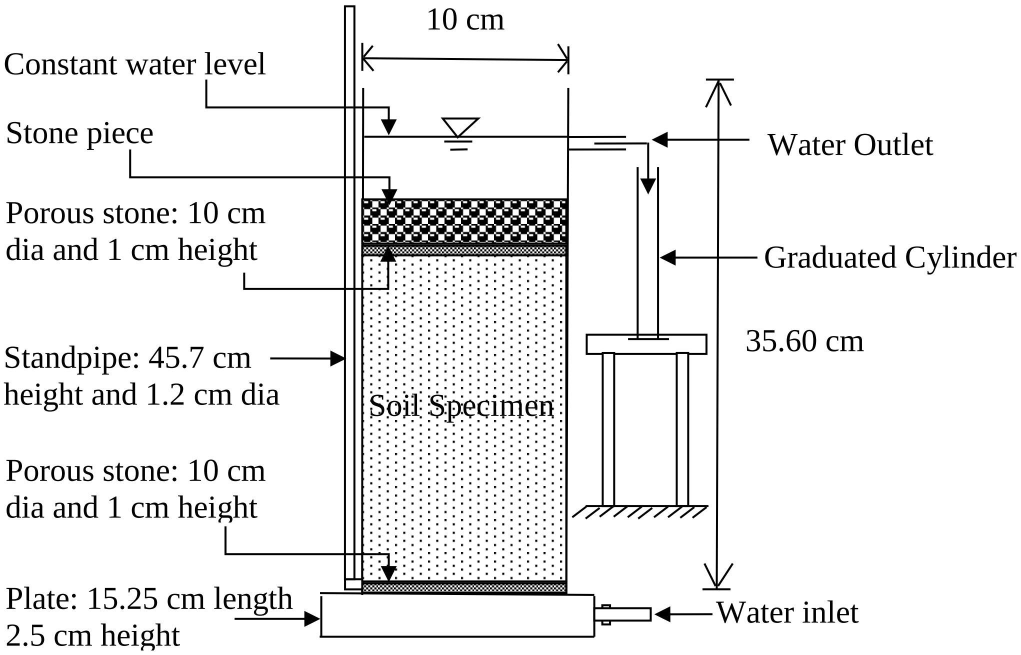

Fig. 3.2 Constant head experimental set-up used in this study.

3.7 Falling Head Permeability Test

The chamber cap was removed of the permeameter and put one porous stone in the base of the chamber. Mold measurements were taken to compute the area of the mold and void ratio, e. A scoop or funnel was used to pour the sand specimen into the chamber. A technique was used to ensure that the soil was placed in a uniform manner. Sample weight were determined, a pan with dry sand was weighed. The desired sample was removed and the pan and sand was weighed again. The difference in weights was the sample weight. The upper porous stone on the specimen and the compression spring on the porous stone was placed. Then the chamber cap and sealing gasket in place were put and secured it firmly with the cap nuts. The length of the specimen was measured and recorded it. The burette was attached to the inlet of the permeameter. The desired water was used from the constant-head reservoir to fill the burette to the top by opening both inlet valves at the base of the permeameter. The elevation of the constant head reservoir had increased to do this. The outlet valve of the permeameter was opened and the valves were closed to the piezometers. The inlet valve was opened to saturate the specimen. The valve was opened slowly to prevent a sudden inrush of water that liquefied the soil sample. Two elevations along the burette, one near the top and the other near the bottom were chosen. A grease pencil or other means was used to mark them. The height from each of these marks to the outlet tube of the permeameter was measured. These was recorded on the data sheet as h_1 and h_2, respectively. The inlet valve from the constant-head reservoir was closed if it was still open. The inlet valve from the burette was opened and watch the water level in the burette. When it was reached the higher of the two marks, started the timer. When the water level was dropped past the lower mark on the burette, stopped the timer and removed the graduated cylinder from the outflow tube. The time, the volume of the water input (difference in burette readings) and the water outflow were recorded. The temperature of the water are also recorded. This process was repeated thrice.

Fig. 3. 3 Photograph showing the set-up for falling head test.

Falling head permeability is calculated using the following equation:

$$k = \frac{aL}{At} \ln \frac{h_1}{h_2} \tag{3.2}$$

where, a= Cross sectional area of the stand pipe.

A= Cross sectional area of the soil sample.

h_1= Hydraulic head across sample at beginning of the test.

h_2= Hydraulic head across sample at end of the test.

L= Length of the soil sample.

Viscosity of water is corrected from the following equation

$$k_{20} = k_T \frac{\eta_T}{\eta_{20}} \tag{3.3}$$

η =Viscosity of fluid (water)

The entire work is shown sequentially in a flow diagram in Fig. 3.2. The whole laboratory testing programme consisted of carrying out the following tests on samples of the two soils (Table 3.1).

For the calculation of void ratio of soil

$$e = \frac{G_S \lambda_W V_r}{W_S} - 1 \tag{3.4}$$

Where W_s is the dry weight of the soil grain and is calculated by the difference between air-dry sample weight and moisture content at the stage of testing and V_r is the total volume of soil.

Chapter Four

RESULTS AND DISCUSSIONS

4.1 Introduction

The main objectives of this chapter is to present detail results of laboratory investigations obtained in the study. Permeability of two different soils (sandy and clay soil) was treated in the laboratory to investigate the effect of lime and fly ash treatment. The soils were treated by using lime and fly ash of different percentages (e.g. 1%, 3%, and 5%). Soil type-1 was clayey soil, so permeability of these soil was determined by using falling head method. For the case of soil type-2 (sandy soil), constant head test was performed to evaluate its permeability. The tests were performed on the treated samples at 3 day, 7 day and 14 day (for soil type-1 and 2) after mixing and subsequently cured for reaction to occur in air-dry state (the curing period was defined earlier as the reaction time).

4.2 Soil Properties

The index properties of the collected soil samples obtained from laboratory investigation. Index properties tests were conducted according to ASTM Standard as mentioned in chapter Three. Index properties of the selected soil samples are presented in Table 4.1. Their grain size distribution curves are presented in Fig. 4.1.

Table 4.1 Index Properties of the soil samples collected from study areas

Index Properties and classification		Soil Type-1	Soil Type-2
Specific Gravity		2.65	2.72
Atterberg Limits	Liquid Limit	35	-
	Plastic Limit	19	-
	Plastic Index	16	-
Grain Size*	Sand %(0.06 mm to 2 mm)	3	100
	Silt % (0.002 mm to 0.06 mm)	79	-
	Clay %(< 0.002 mm)	18	-
Group Classification (ASTM D2487)		CL	SP

* Classification based on MIT Classification

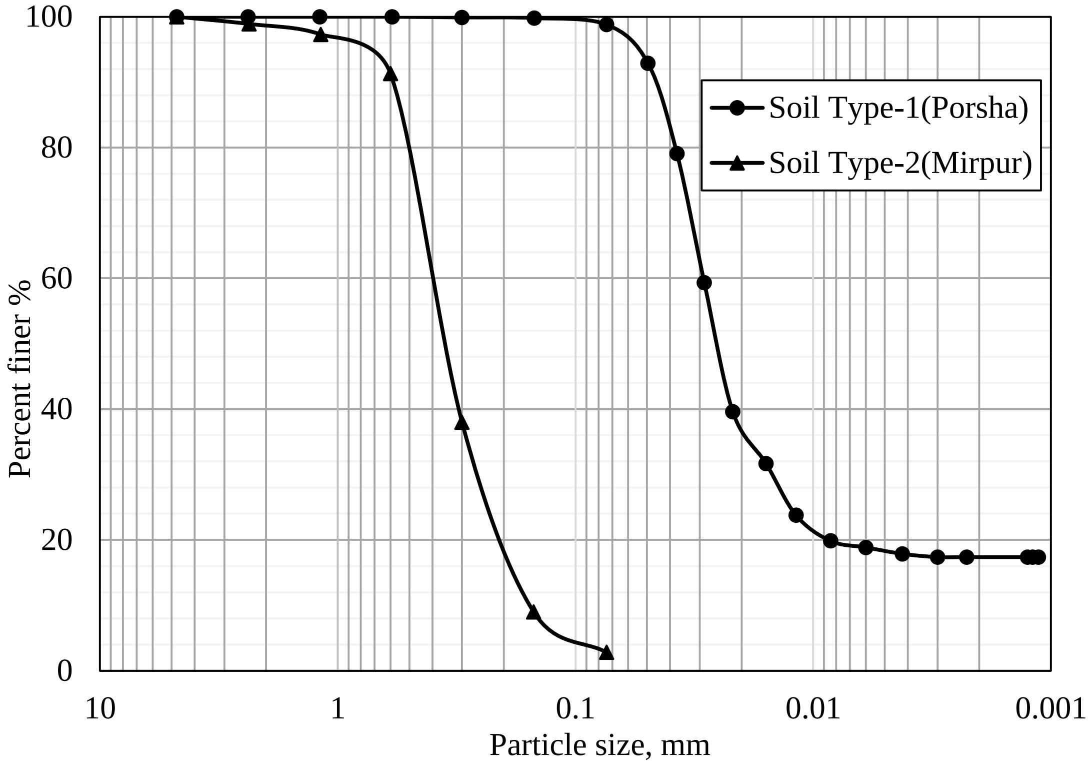

Fig. 4.1 Grain size distribution curves of Soil Type-1 and Soil Type-2.

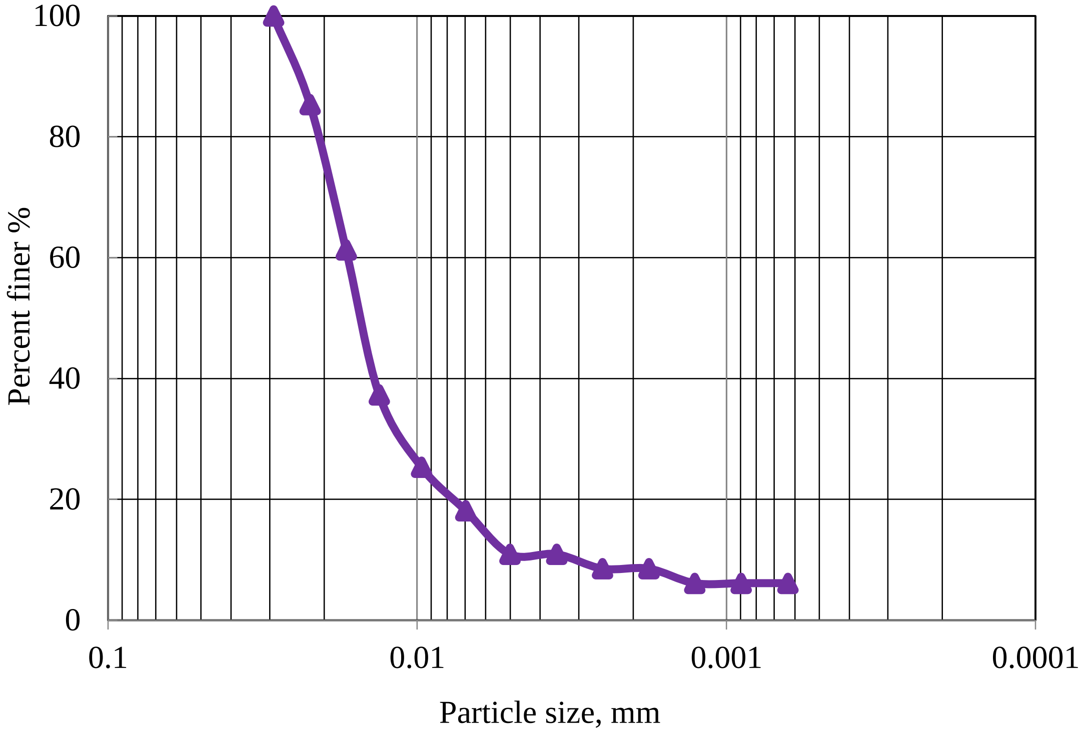

Fig. 4.2 Grain size distribution curve of fly ash

Table 4.2 Index properties of treated Soil Type-1

Index Properties	Lime Content (%)				Fly ash Content (%)			
	0	1	3	5	0	1	3	5
Liquid Limit (%)	35.0	34.0	33.5	33.0	35.0	34.5	34.0	33.5
Plastic Limit (%)	19.0	21.0	23.0	25.0	19.0	20.5	22.0	24.0
Plasticity Index (%)	16.0	13.0	10.5	8.0	16.0	14.0	12.0	9.5

The values of plasticity properties of the untreated and treated soil samples are shown in Table 4.2. Fig. 4.2 and Fig. 4.3 show the variation of liquid limit, plastic limit and plasticity index with the increment of additives addition. It can be seen from Fig 4.2 and Fig. 4.3 that plastic limit increased with increasing lime content and fly ash content while liquid limit and plasticity index reduced with the increase in lime content and fly ash content. The results are in agreement with those reported Ahmed (1984) and IRC (1976).

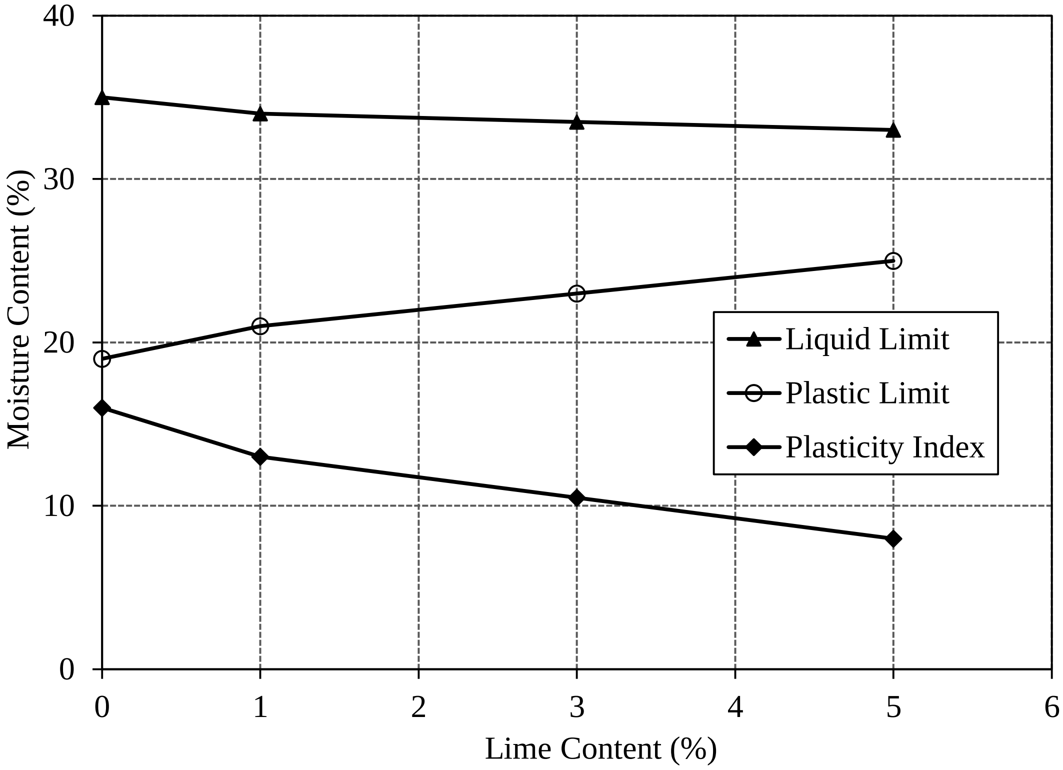

Fig 4.3 Effect of lime content on Atterberg limits of Soil Type-1

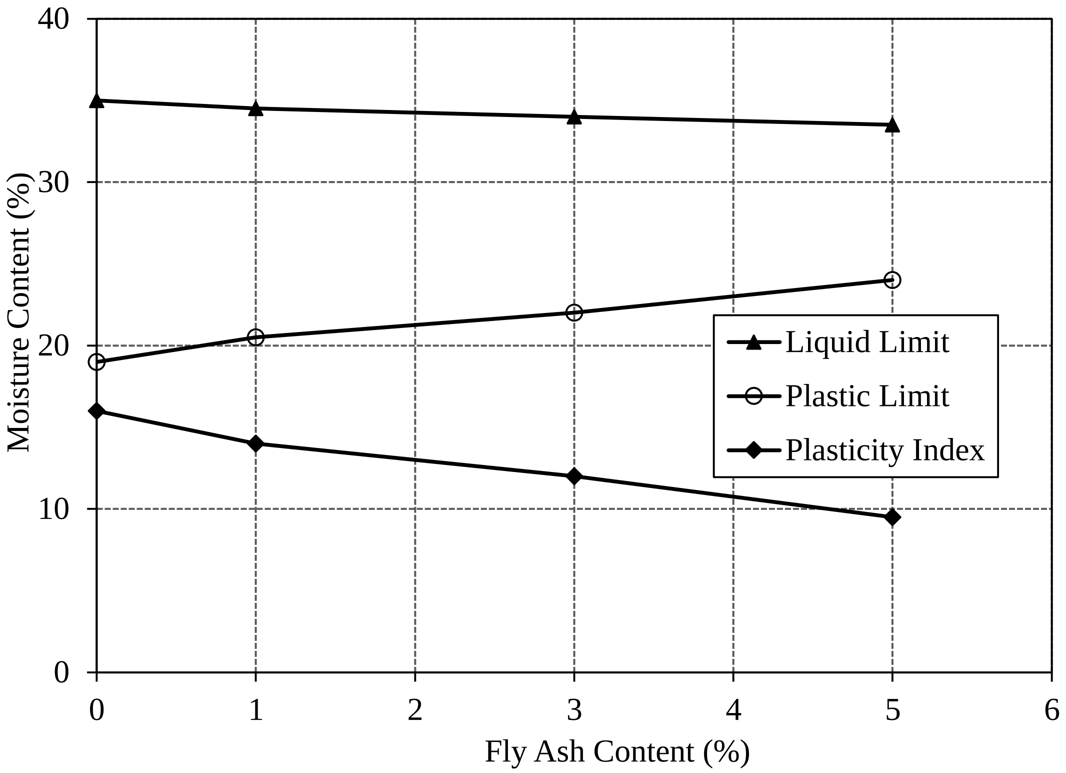

Fig 4.4 Effect of fly ash content on Atterberg limits of Soil Type-1

4.3 Test Results and Discussions

Laboratory tests were performed to investigate the effects of lime content, fly ash content and reaction time (after mixing) on the permeability of lime and fly ash treated two sandy and a clayey soils. The samples were reconstituted in the laboratory. Since permeability of treated soil is very sensitive to void ratio, soil fabric and structure, aging after reconstitution, stress level, etc., attention was given to maintain these controlling factors unchanged among the tests except the lime content, fly ash content and aging (which was termed as the reaction time). To check the repeatability of test samples, two samples were prepared with the similar testing conditions (i.e. with the same lime content, moisture content during mixing and the same reaction time). After testing, it was observed that although testing conditions were similar, both void ratio and permeability were varied noticeably.

Permeability and void ratio were calculated from the test results of samples subjected to similar test conditions and are summarized in Table 4.1. Other results related to this table will be discussed later. In the following discussion, the average values of the test results for soil type 1 and 2 were used. It was already mentioned that for a given soil, various percentages of lime and fly ash were used for the investigation and besides, for a given percentage of lime content and fly ash content, the reaction time was also varied (up to 14 days).

Table 4.3 Test results from Soil Type-1 and Soil Type-2

Stabilizer	Content	Soil Type-1			Soil Type-2		
		Reaction time (day)	Void ratio, e	Permeability, k (× 10^{-5}) cm/sec	Reaction time (day)	Void ratio, e	Permeability, k (× 10^{-3}) cm/sec
-	0%		0.981	9.712		1.082	4.457
Lime	1%	3	0.916	6.783	3	1.013	3.925
		7	0.905	5.473	7	0.987	3.528
		14	0.893	4.081	14	0.927	3.102
	2%	3	0.826	3.762	3	0.905	2.071
		7	0.812	3.315	7	0.892	1.819
		14	0.804	2.683	14	0.831	1.458
	3%	3	0.791	1.827	3	0.807	1.04
		7	0.782	1.038	7	0.772	0.928
		14	0.773	0.801	14	0.743	0.716
Fly ash	1%	3	0.895	3.861	3	1.058	2.983
		7	0.884	3.265	7	1.014	2.412
		14	0.843	3.081	14	0.980	2.013
	2%	3	0.786	2.673	3	0.953	1.184
		7	0.775	2.357	7	0.934	0.982
		14	0.763	2.016	14	0.893	0.897
	3%	3	0.759	1.392	3	0.871	0.837
		7	0.746	1.012	7	0.843	0.534
		14	0.732	0.534	14	0.822	0.218

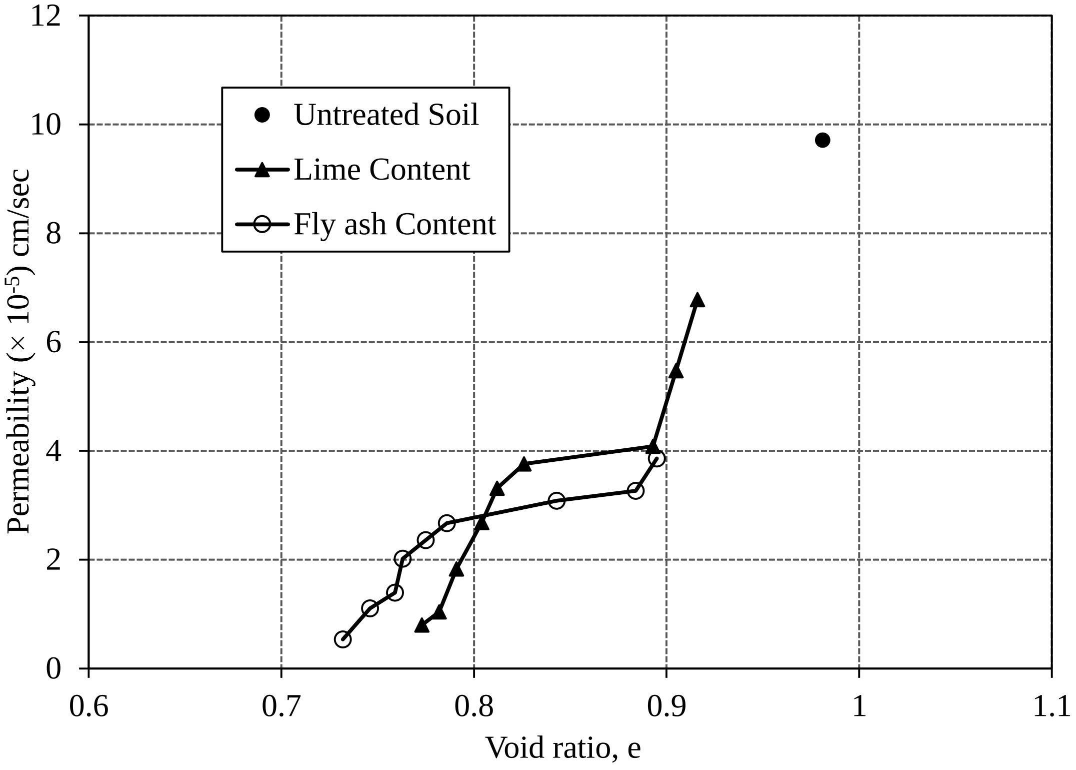

Fig 4.5 Relationship between permeability and void ratio of Soil Type-1

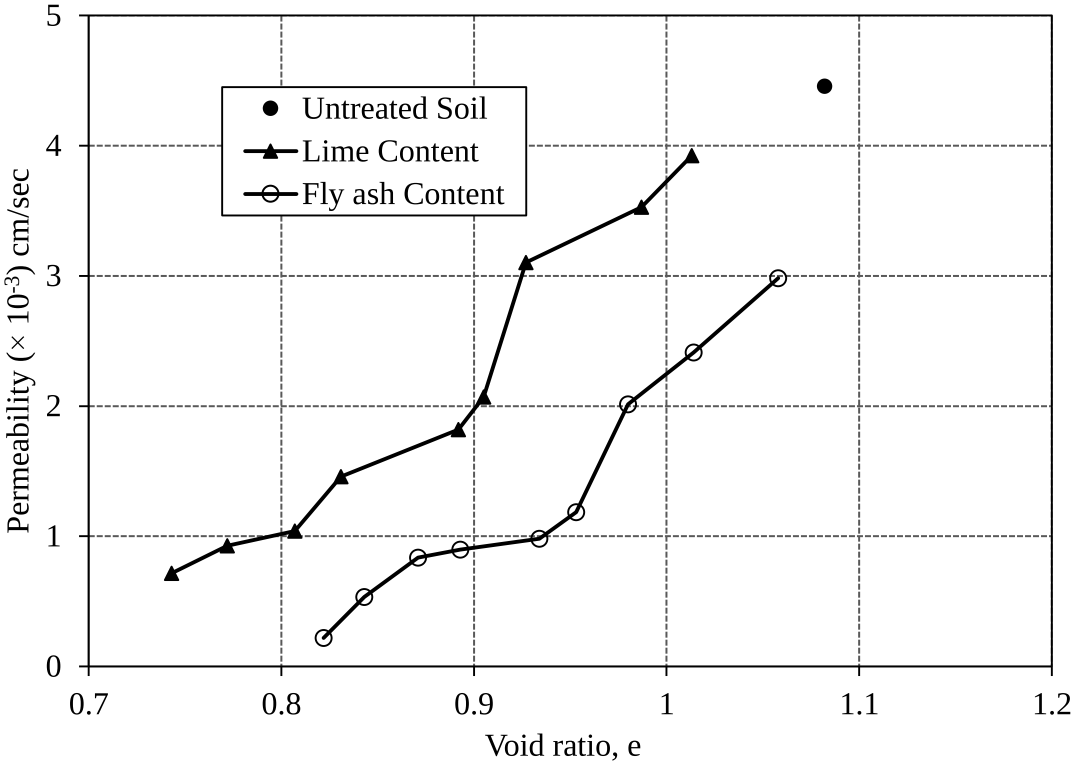

Fig 4.6 Relationship between permeability and void ratio of Soil Type-2

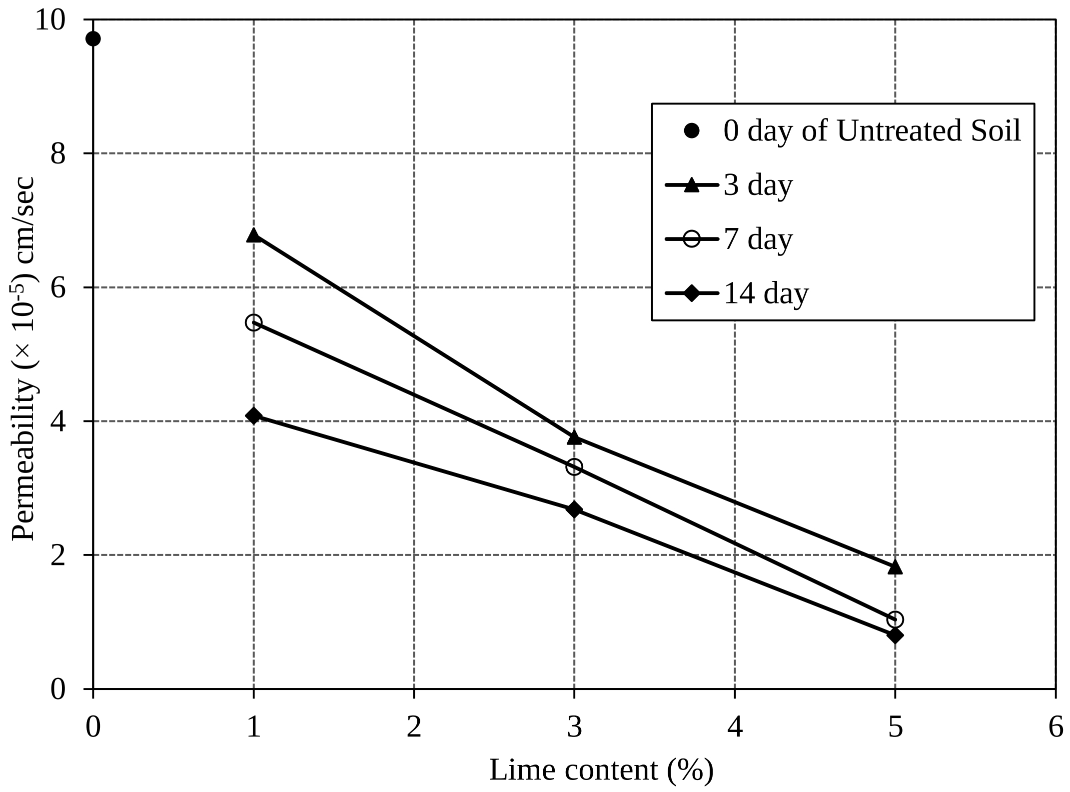

Fig 4.7 Effect of lime content on permeability of Soil Type-1

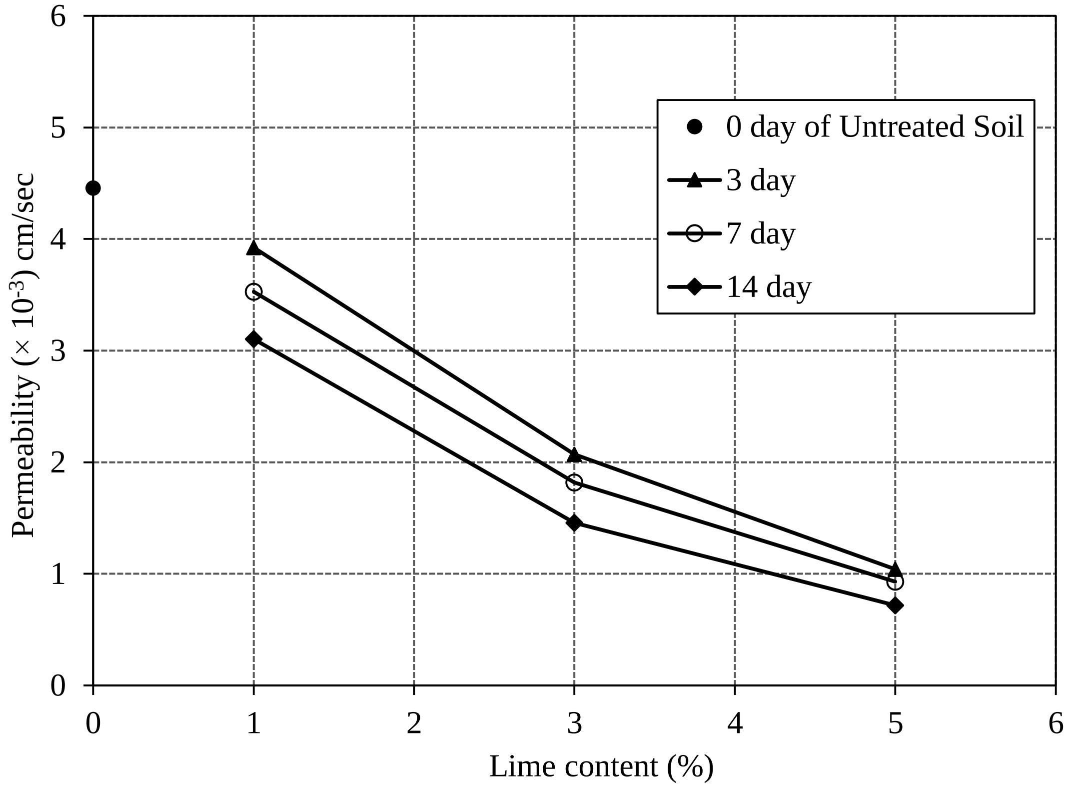

Fig 4.8 Effect of lime content on permeability of Soil Type-2

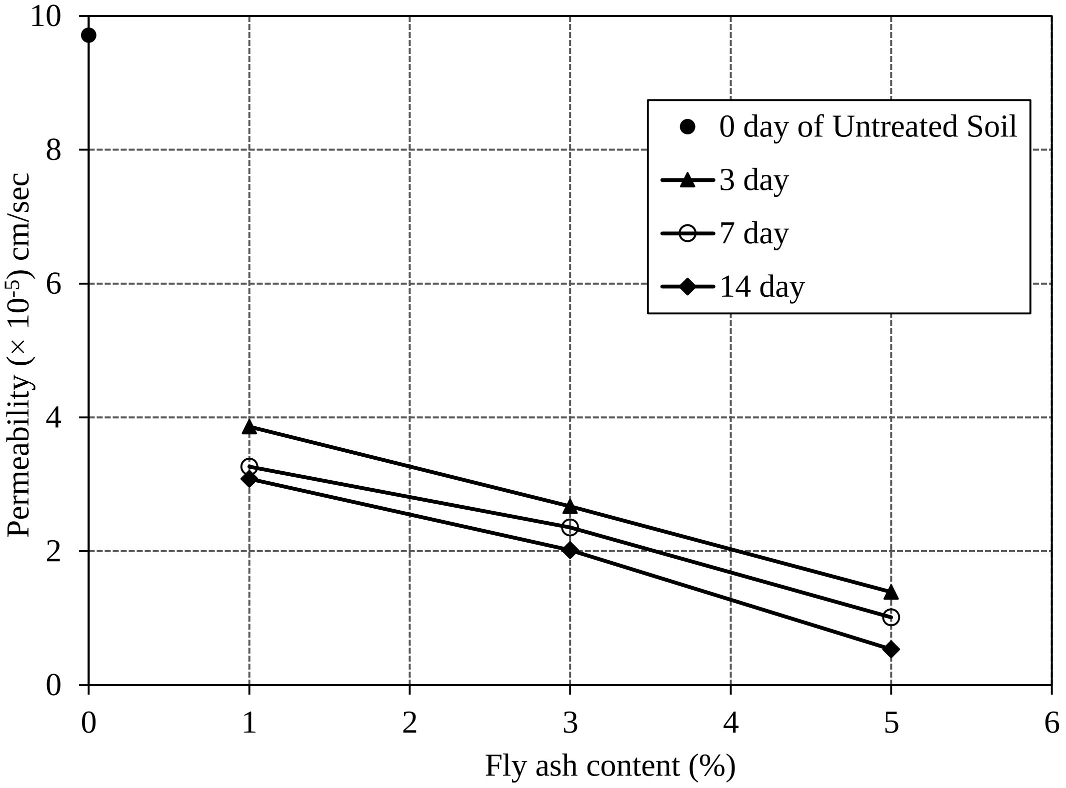

Fig 4.9 Effect of fly ash content on permeability of Soil Type-1

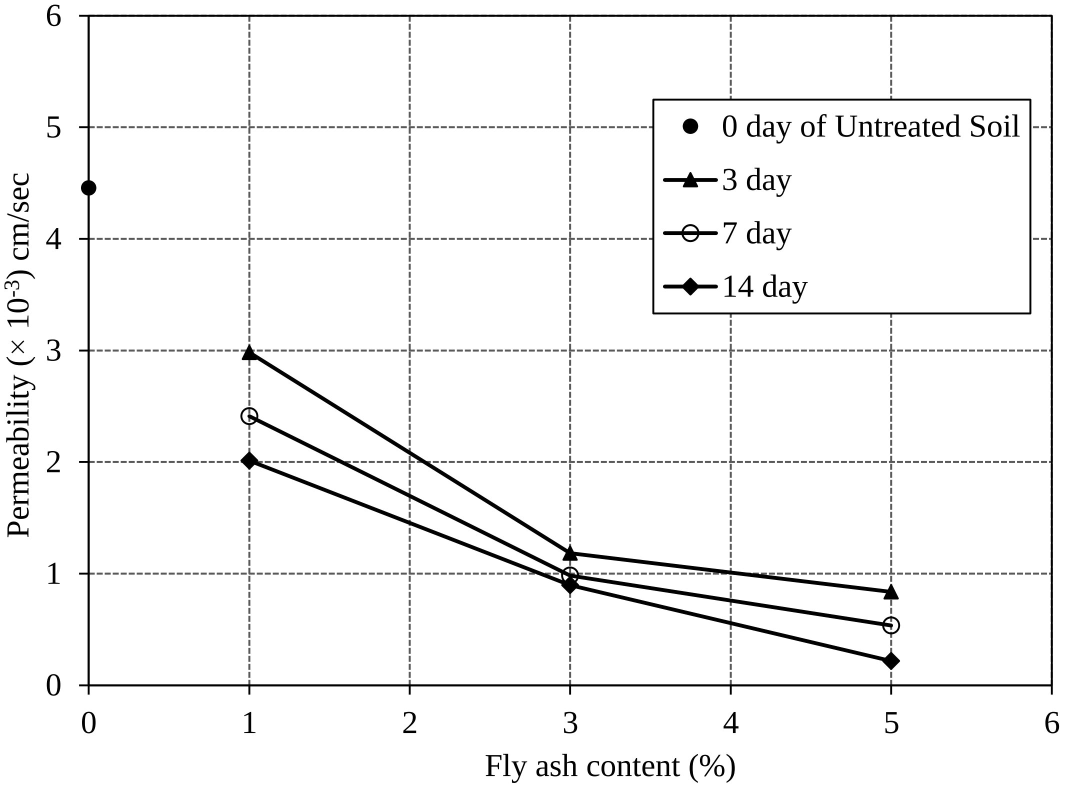

Fig 4.10 Effect of fly ash content on permeability of Soil Type-2

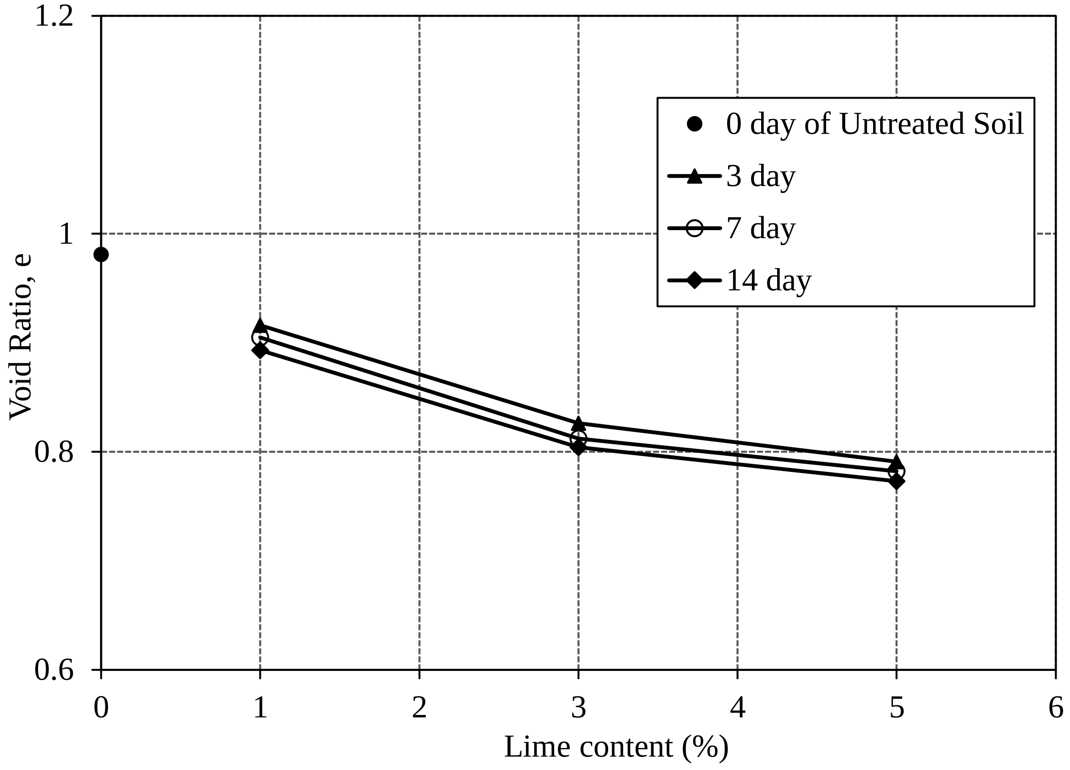

Fig 4.11 Relationship between void ratio and lime content of Soil Type-1

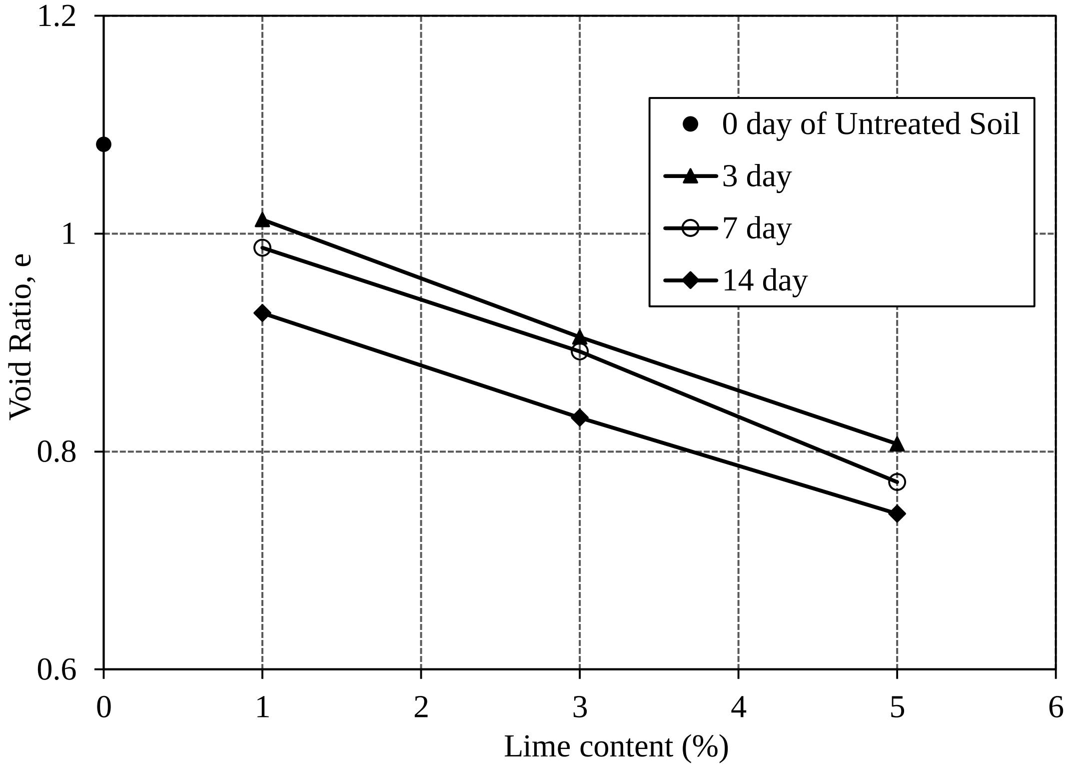

Fig 4.12 Relationship between void ratio and lime content of Soil Type-2

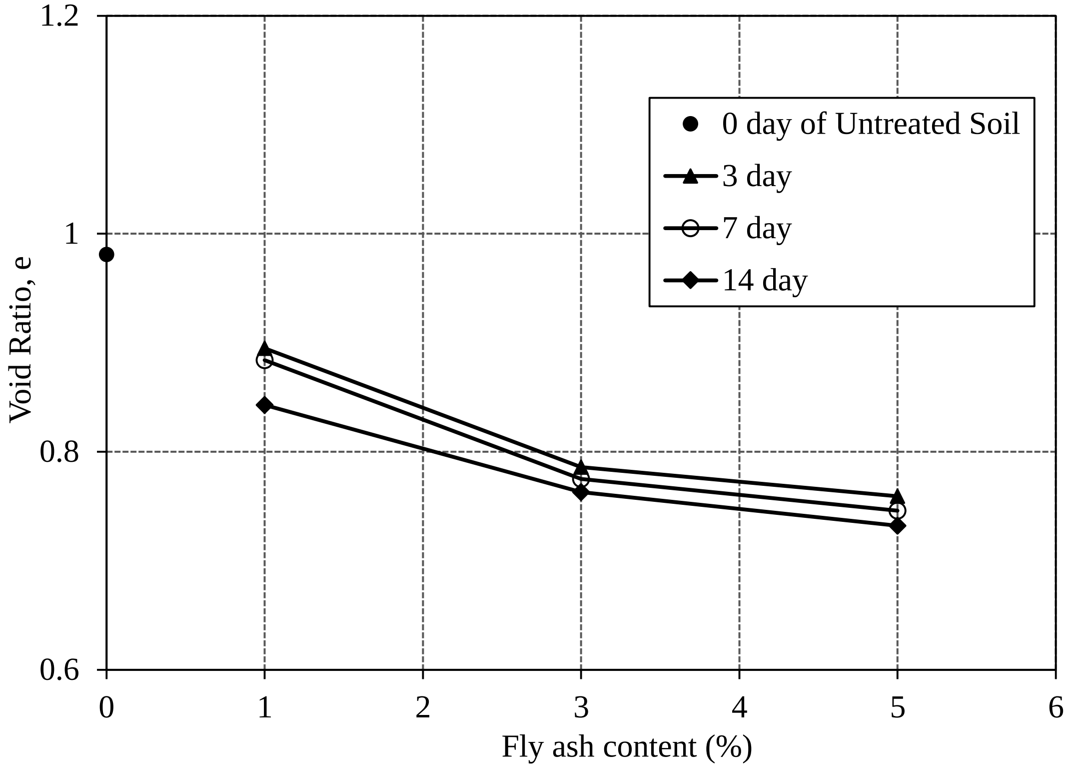

Fig 4.13 Relationship between void ratio and fly ash content of Soil Type-1

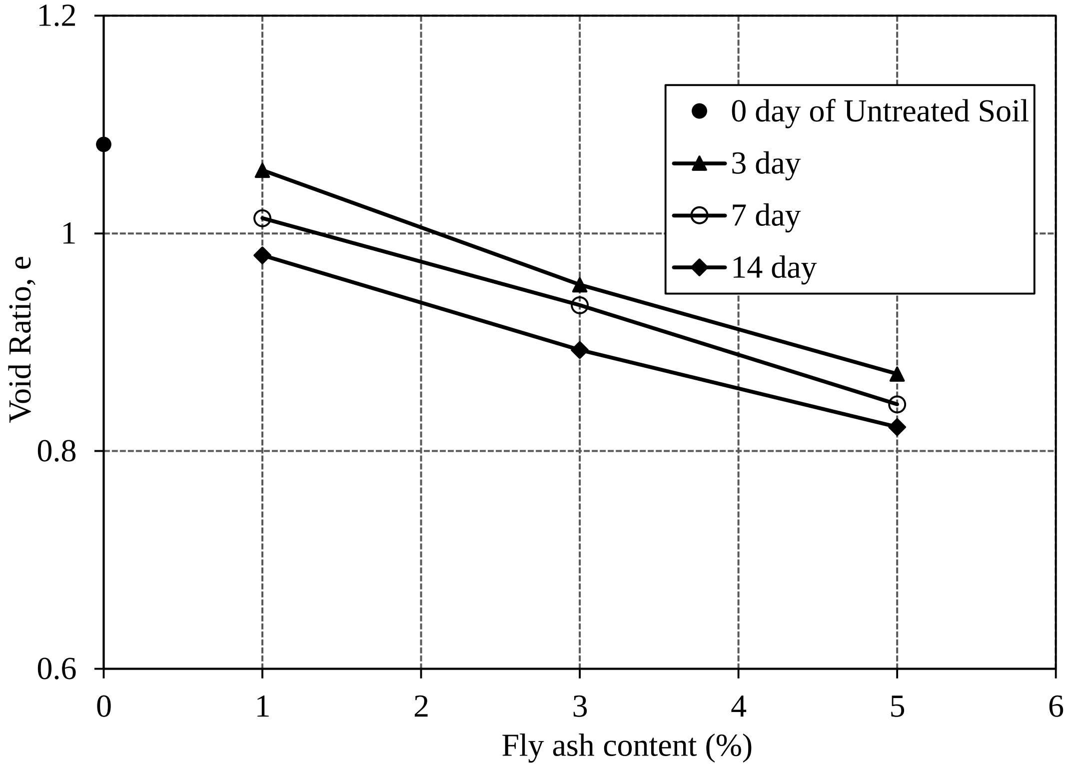

Fig 4.14 Relationship between void ratio and fly ash content of Soil Type-2

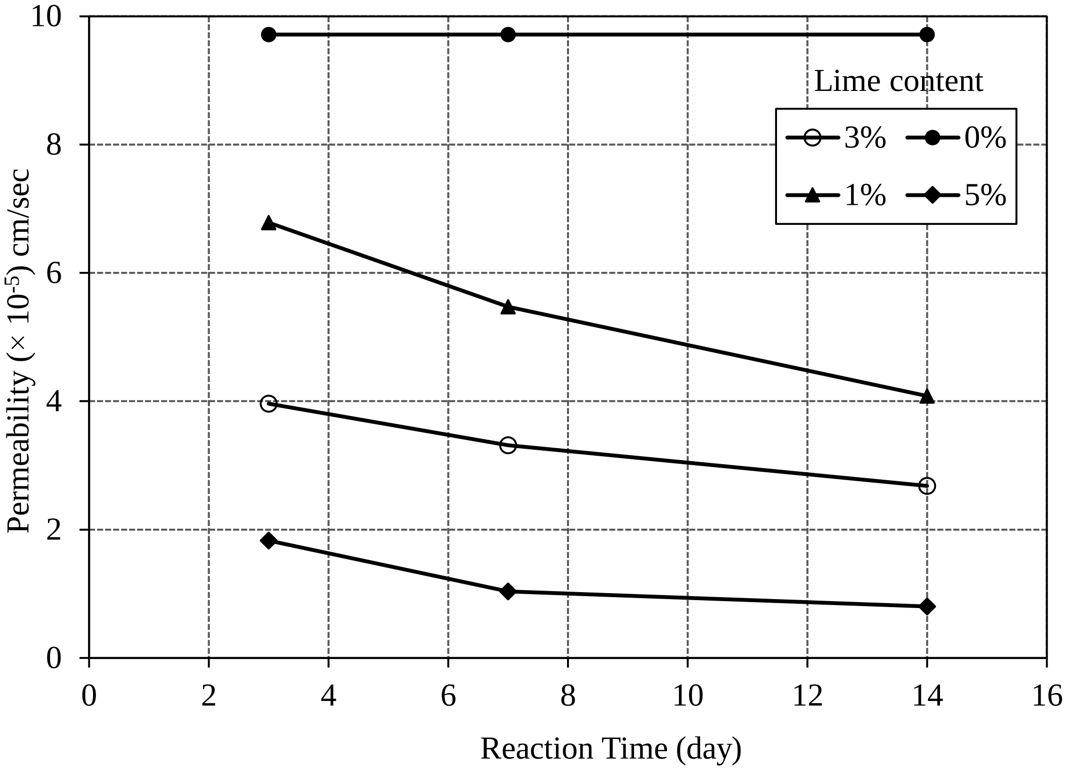

Fig 4.15 Effect of reaction time on permeability of lime treated Soil Type-1

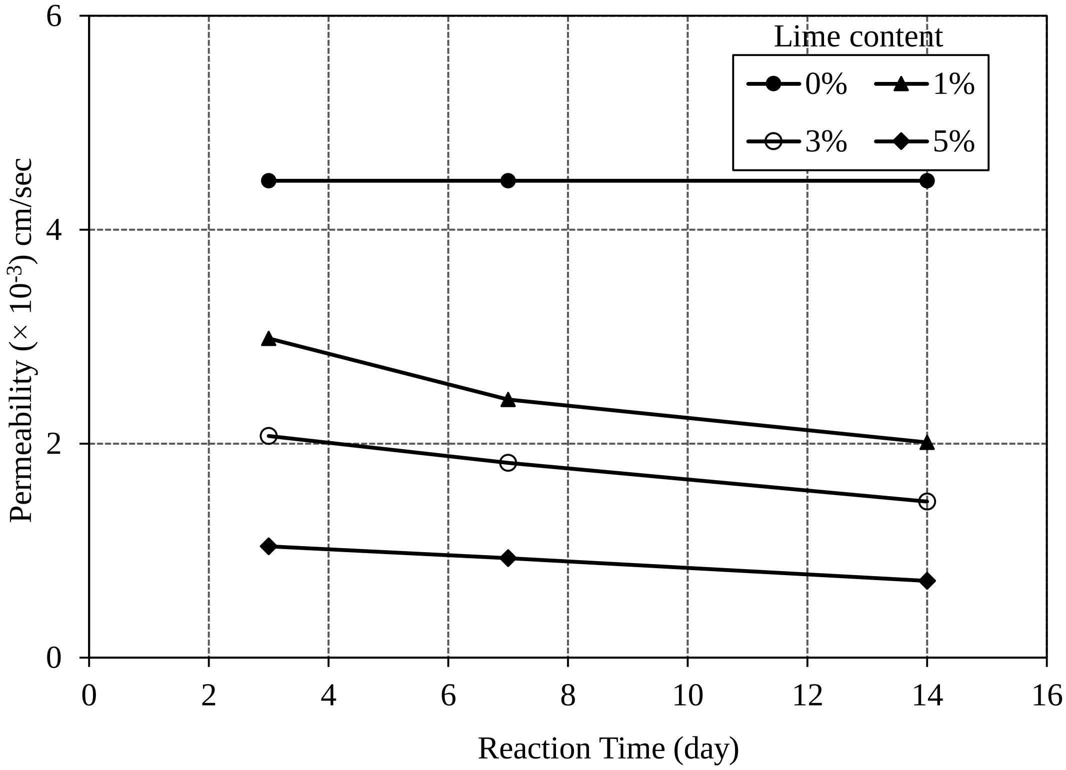

Fig 4.16 Effect of reaction time on permeability of lime treated Soil Type-2

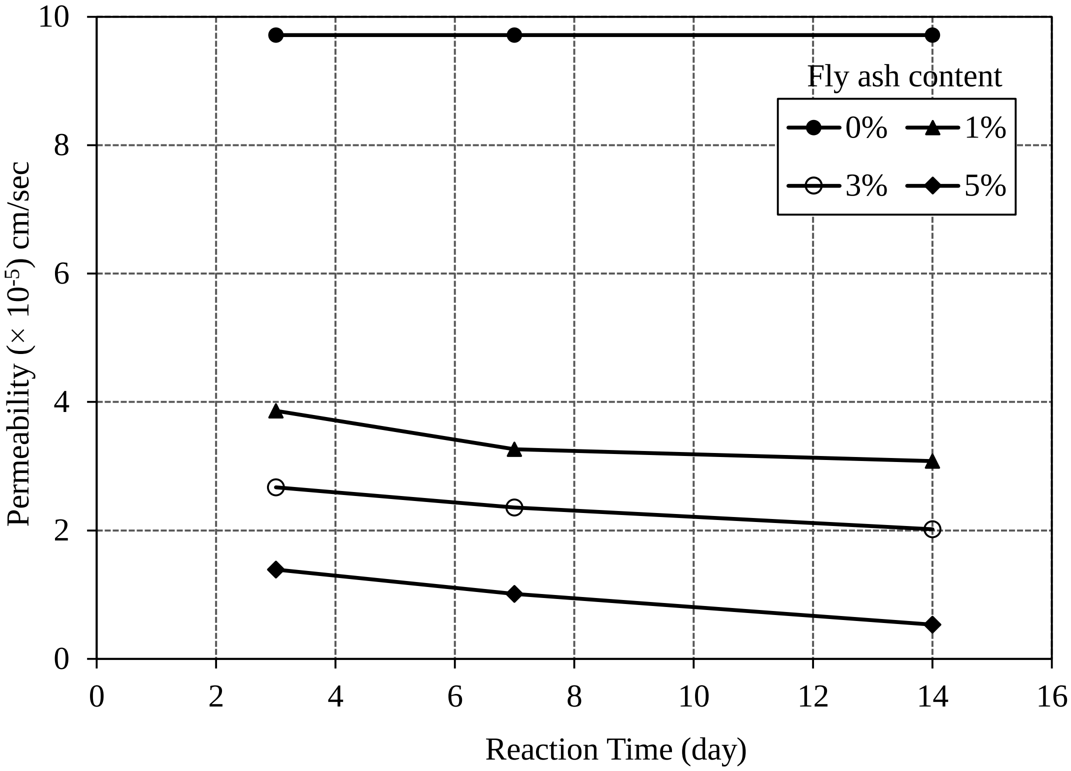

Fig 4.17 Effect of reaction time on permeability of lime treated Soil Type-1

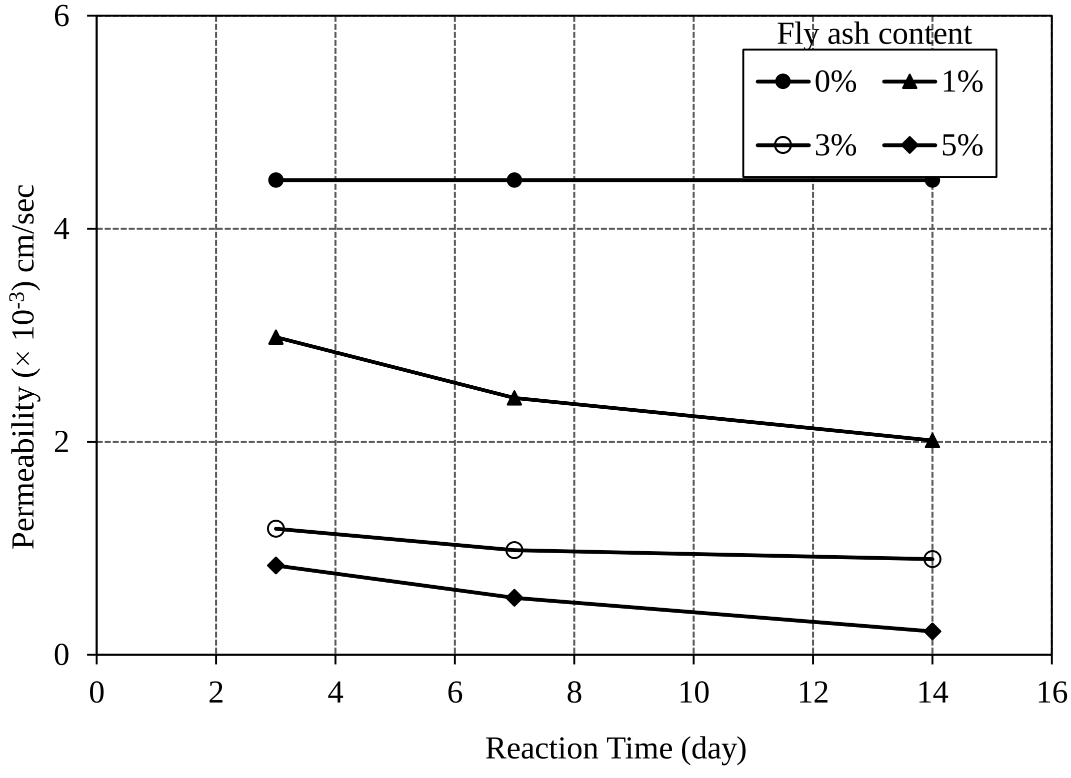

Fig 4.18 Effect of reaction time on permeability of fly ash treated Soil Type-2

Chapter Five

CONCLUSIONS AND RECOMMENDATIONS

5.1 Introduction

In this research work, two soils were collected from two different locations to investigate the effect of fly ash content and lime content on permeability characteristics. Soils were treated with lime and fly ash at different percentages of 1 %, 3%, and 5% (by weight). Tests were performed on air-dry sample after allowing different reaction time (curing age). Soil Type-1 was clayey soil and its permeability was determined by using falling head method. On the other hand, Soil Type-2 was sandy soil and constant head test was performed to determine its permeability characteristics. After mixing soil with additives at a definite proportion, a specific time (denoted earlier as reaction time) was allowed for lime and fly ash to react with water and soil particles. The reaction time was varied from 3 to 14 days.

5.2 Conclusions

The following conclusions can be drawn based on the results obtained after investigating various aspects related to permeability characteristics of lime and fly ash treated soils:

1) Overall decrease in permeability of Soil Type-1 and Soil Type-2 were observed with the increase in additives (lime and fly ash content).

2) The permeability of treated soils was also changed consistently with the void ratio pattern. Due to the decrease in void ratio, permeability of the treated soils decreased.

3) Aging had no affect on the permeability and void ratio of untreated soils. But the void ratio and hence permeability of lime and fly ash treated soils were slightly changed with the reaction time.

5.3 Recommendation for Future Study

It is recommended to extent the research work in the following field to have a better understanding about treated soils:

1) More similar soils should be investigated to obtain a general conclusion based on the present findings.
2) Coefficient of permeability can be investigated using the additives such as cow-dung, rice husk, lime-cow-dung mix, lime-fly ash mix, fly ash-cow-dung mix.
3) Various engineering properties including permeability of stabilized soils can be investigated after the application of different compaction efforts such as kneading, preloading, vibration, etc.

REFERENCES

Ahmed, B., Alim, M.A. and Sayeed, M.A., “Improvement of soil strength using cement and lime admixtures”, *Earth Science*, Vol.2, pp. 139-144(2013).

Ahmed, N.U. (1984), “Geotechnical properties of selected local soils stabilized with lime and cement”, M.Sc. Engineering Thesis, Department of Civil Engineering, BUET, M-51, BUET, Dhaka, Bangladesh.

Al-Dahlaki, M.H., “Effect of Fly Ash on the Engineering Properties of Swelling Soils”, *Journal of Engineering and Development*, Vol. 11, pp. 1-11 (2007).

Ansary M.A. and Hasan K.A., “Lime Stabilization on Soil of a Selected Reclaimed Site of Dhaka City”, *Journal of Geotechnical Engineering,* Vol. 1, pp.1-6 (2011).

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

ASTM (1976) Standard Method for Classification of Soils for Engineering Purposes, ASTM Designation D2487-69 ASTM Standard Part 19. pp. 30-39.

Arman, A.H. and Muhfakh, G.A. (1972), “Lime stabilization of organic soils”, Highway Research Board, Washington, D.C. HRB bul. No381.

Arce, R.G. and Boyed, C.E (1975), “Effects of Agricultural limestone on water chemistry, phytoplankton productivity and fish production in soft-water ponds”, Trans. Amer. Fish Soc, 104; 308-312.

Barenberg, E.J. (1974). "Lime Fly Ash Aggregate Mixtures in Pavement Construction", Process and Technical Data Publication, National Ash Association.

BS 1377 (1975), Method of tests for soils in Civil Engineering Purposes, British Standard Institution, Gaylard and Son Ltd, London.

Bell, F.G. (1988), Lime stabilization of clay soils, Part-1, Basic properties, Ground Engineering, vol. 21, No.1 pp. 10-15.

Boominathan, A. and Prasad, S.R. (1992), “Laboratory investigation of effectiveness of lime treatment in polluted soils”, Proceedings of the Indian Geotechnical Conference, pp. 497-500.

Bowles, E.J. (1978), Engineering properties of soils and their measurements, Second Edition, McGraw-Hill, Inc.213p

Brooks, D.R.M., “Soil Stabilization with Fly Ash and Rice Husk Ash”, *International Journal of Research and Reviews in Applied Sciences,* Vol. 1, pp.209-217 (2009).

Broms, B. (1984),"Lime-Clay mineral reaction products", Ontario Ministry of Transportation and Research Report, RR243.

Brooms, B. and Boman, P. (1977), "Stabilization of soil with lime columns", Design Hand Book, Royal Institute of Technology".

Boyed, C.E. (1990), Water quality in ponds for Aquaculture, Birmingham Publishing Co. Birmingham, Alabama.

Boyed, C.E and Scarsbrook, E. (1974), "Effects of Agriculture limestone on phytoplankton communities of fish ponds", Arch. Hydrology, pp336-349.

Bowling, M.L. (1962), "The effects of lime treatments on Benthos production in Georgia farm pond", Proc. Annual Conf. S. E. Assoc. Game and Fish Comm., 16; 418-424.

Brandl, H. (1981), "Alteration of soil parameters by stabilization with lime", Proceeding of the Tenth ICSMFE vol.3, pp. 587-594.

Bangladesh Transport Survey, (1974), "Basic Data- The Soils of Bangladesh, Part 8, The Government of the People's Republic of Bangladesh.

Compendium (1987), "Lime stabilization, reactions, properties of design and construction", State of the art Report-5, U.S. Transportation Research Board, Washington D.C.

Casagrande, A. (1937), "Seepage through Dams, in "Contribution to soil mechanics 1925-1940", Boston Society of Civil Engineers, Boston 295p.

Carman, P.E. (1956), "Flow of Gasses through Porous Media", Academic, New York.

Diamond, S. and Kawamura, M. (1974), "Soils stabilization for erosion control", Joint Highway Research project 74-12, Purdue University, West Lafayette, Indiana.

Diamond, S. and Kinter, E.F. (1965), "Mechanism of soil lime stabilization", An Interpretive Review, High way Research Board, bul. No. 92, pp. 83-96.

Das, B.M. (1983), Advanced soil mechanics, International Edition, Hemisphere Publishing Corporation, New York, pp. 65-166.

Eades, J.L. and Grim, R.E. (1960), "Research of hydrated lime with pure clay minerals in soil stabilization", Highway Research Board, Washington D. C., bull. No. 262, pp. 51-63.

Eades, J.L., Nichols, F.P. and Grim, R.H. (1962), "Foundation of new materials with lime stabilization as proven by field experiments in Virginia", Highway Research Board, Washington D.C., pp. 3-39, bull. 335.

EPRI (Electrical Power Research Institute), (1986). Fly Ash Design Manual for Road and Site Application", Vol. 1: Dry or Conditioned Placement, CS-4419, Research Project 2422-2, Interim Report.

Ghos, R.M. (1987), "Properties of the lime for stabilization", Journal of the Institution of Engineers (India), vol. 68, part C12, pp. 307-313.

Hasan, M., Khan, M.A. and Alam, J., "Morphological and Geotechnical Properties of Dadri Fly Ash",*International Journal of Structural and Civil Engineering Research,* Vol. 3,pp. 3 (2014).

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

Hilt, G.H. and Davidson, D.T. (1960), "Soil lime mixture", Highway Research Board, Washington D.C. bull no. 304, pp. 99-138.

Hickling, C.F. (1962), Fish culture, Faber and Faber, London. 295p.

Hazen, A. (1892), "Some physical properties of sands and gravels with special reference to their use in filtration", 24th Annual Report, Massachusetts State Board of Health.

Ishtiak, I., Hoque, E. and Islam, M.S., "Use of carbide lime as a potential stabilizing agent of a selected silty soil*", Proceedings of the 12th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering,* Singapore, Vol. 1, pp. 431-434 (2003).

Islam, M.S.,"*Permeability Characteristics of Lime Treated Soil",* Master of Engineering Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology, 2001.

Indian Road Congress (IRC) 1973b, "Recommended design criteria for the use of soil-lime mixes in road construction", IRC Highway Research Board: 51-1973, New Delhi.

IRC (1976), "State of the Art: Lime-soil stabilization", Special report' IRC highway research board, New Delhi.

Kezdi (1979), "Stabilization Earth Board with Lime", Elsvier Scientific Publishing Company, New York. pp. 163-174.

Khan, M.J. (1989), "Effect of Fly Ash with lime and cement additive on the engineering behavior of soft Bangkok clay", M. Sc. Engineering, Thesis no. GT 89-lb, Asian Institute of Technology, Bangkok.

Ingles, O.G. and Metcalf, J.B. (1972), “Soil stabilization-principles and Practice”, Butterworths Pty. Limited, Australia.

Kulkarni G.J. (1927), “Ueber Kapillare leitung des wassers in Boden”, Wien, Akad. Wiss. Vol. 136 part 2a, p.271.

Kozeny, J. (1927), "Ueber kapillare leitung des wassers in Boden", Wien, Akad,Wiss. vol. 136 part 2a, p.271.

Kozeny, J. (1933), “Theorie and Berechung der Brunnen”, Wasserkr, Wasserkwritsch, vol. 28, p.104.

Lambe, T.W. (1993), Soil testing for engineers, Fifth Wiley Eastern Reprint, 4835/24 Ansari road, Darya Ganj, New Delhi 110 002. 165p.

Mateos, M. (1964), “Soil lime research at lowa state University”, Journal of the Soil Mechanics and Foundation Engineering Division, ASCE vol. 90, pp.127-153.

Mateos, M. and Davidson, D.T. (1962), “Lime and fly ash mixes and some aspect of soil lime stabilization”, Highway Research Board, Washington D.C., bull. No. 335, pp. 40-64.

Metcalf, J.B. (1977), Soil stabilization principles and practice, Willy and sons, New York.

Mitchell, J.K. and Hooper D.R. (1961), “Influence of time between mixing and compaction on properties of lime stabilized expensive clay”, Highway Research Board, Washington D.C., bull. No. 304, pp. 14-31.

Mitchell, J.K. and Hooper D.R. and Campanella, R.G. (1965), “Permeability of compacted clay”, J. soil Mech. Found. Div., ASCE, vol. 91, SM4, pp. 41-65.

Molla A.A. (1997), “Effects of compaction condition and state variable on engineering properties of the lime stabilized soil”, M.Sc Engineering Thesis, BUET Library, BUET, Dhaka, Bangladesh.

Muskat, M. (1937), The flow of homogeneous fluids through porous media, McGraw-Hill, New York.

Naasra (1986), A Guide to stabilization in road work, Sydney, Australia.

Naasra (1987), Pavement design, ‘A guide to the structural design of road pavement, Fine press offset printing, Pty. Ltd, Australia.

Olsen, H.W. (1961), “Hydraulic flow through saturated clay”, Sc. D. Thesis, Massachusetts Institute of Technology.

Olsen, H.W. (1962), “Hydraulic flow through saturated clay”, Proc. 9th Nat. Conf. on clay and clay Min. pp. 131-161.

Pietsch, P.I. and Davidson, D.T. (1962), “Effect of lime on plasticity and compressive strength of representative lowa soils”, Highway Research Board, Washington D.C., bull. No. 335, pp.11-30.

Punmia, B.C. (1991), Soil Mechanics and Foundations, Laxmi Publications, 7/21, Ansri Road, Daryaganj, New Delhi.pp. 171-198.

Pamatmat, M.M. (1960), “The effects of basic slag and Agricultural lime stone on the chemistry and productivity of fertilized ponds”, M.S. Thesis, Auburn, Aia. 113p.

Rajbongshi, B. (1997), “A Study on cement and lime stabilized Chittagong coastal soils for use in road construction”, M.Sc. Engineering Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology,

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

Reyes, A., and Pando, M., “Evaluation of CFBC fly ash for Improvement of Soft Clays”, World of Coal Ash (WOCA), Northem Kentucky, USA, May 7-10, 2007.

Roy, A., “Soil Stabilization using Rice Husk Ash and Cement”,*International Journal of Civil Engineering Research,* Vol. 5, pp. 49-54 (2014).

Sastry, M.V.B. and Rao, A.S. (1987), “Cinder Ash as soil stabilization”, Journal of the Institution of Engineers (India), vol.67; part-C16. Pp 316-321.

Serjuddin, M. (1992), “Chemical stabilization of Bangladesh fine grains soils for improvement of road construction”, Proceeding of the Indian Geotechnical Conference, Calcutta vol.1. pp 223-226.

Singh, A. and Punmia, B.C. (1965), “A new laboratory compaction device and its comparison with the proctor test” Highway Research News, No. 17, HRB, Washinton.

Takhelmayum, G., Savitha, A.L. and Gudi, K., “Laboratory Study on Soil Stabilization Using Fly ash Mixtures”, *International Journal of Engineering Science and Innovative Technology (IJESIT),* Vol. 2, pp. 477-482 (2013).

Thompson, R.M. (1966), “Shear strength and elastic properties of lime soil mixtures”, Highway Research Board, Washington D.C., vol. No. 139, pp.61-72.

Thompson, R.M. (1968), “Lime treated soil for pavement construction”, Proceeding of the American Society of Civil Engineering, pp. 191-213.

Townsend D,L. and Klyn, T.W. (1970), “Durability of lime stabilized soil”, Highway Research Board, Washington D.C., vol. No.315, pp.81-89.

Uddin, M.K., “The Role of Fly Ash as an Additive with Cement and Lime on the Behaviour of Soft Clay”, *Journal of Geotechnical Engineering,* Vol. 1, pp.38-51(2011).