EFFECTIVENESS OF USING POLYMERS AND CEMENT FOR SOIL STABILIZATION

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in partial fulfillment of the requirements for

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MASTERS OF ENGINEERING

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STRUCTURAL ENGINEERING

Submitted by

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DECLARATION

I hereby declare that the Theisis entitled **"Effectiveness of using polymers and cement for soil stabilization"** is an authentic record of my own work carried out as requirements for the award of degree M.E. Structures at Thapar University, Patiala, under the guidance of Dr. Shruti Sharma, Asstt Professor (CED) and Mr. Tanuj Chopra, Asstt Professor (CED) during the Year 2013.

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CERTIFICATE

This is to certify that the thesis entitled "**Effectiveness of using polymers and cement for soil stabilization**" being submitted by **Neeraj Verma, Roll No.820922001** in partial fulfillment for the award of degree of **Masters of Engineering in Structural Engineering** at **Thapar University,Patiala** is a bonofide work carried out by him under our guidance and that no part of this theisis has been submitted for the award of any other degree.

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ABSTRACT

The aim of the study was to determine the value of Unconfined compressive strength and CBR values of Soil after stabilizing it with Cement and Polymer. Soil stabilization has been widely used as an alternative to substitute the lack of suitable material on site. The use of nontraditional chemical stabilizers in soil improvement is growing daily. In this study a laboratory experiment was conducted to evaluate the effects of waterborne polymer on unconfined compression strength on sandy soil and CBR Test on clayey soil .The laboratory tests were performed including grain size of sandy soil, unit weight, and unconfined compressive strength test. The sand and various amounts of polymer (2%, 3%, and 4%) and cement (20%, 30%, and 40%) were mixed with all of them into dough using hand mixing in laboratory conditions. The samples were subjected to unconfined compression tests to determine their strength after 7 days of curing. The results of the tests indicated that the waterborne polymer significantly improved the unconfined compression strength of sandy soils which have susceptibility of liquefaction. Polymer modified the engineering properties of soil through physical bonding. The amount of polymer required to modify the engineering properties was directly related to specific surface and soil particle coating thickness. Polymer amended soils displayed a reduced performance compared to cement amended soils.

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SOIL STABILISATION

1. INTRODUCTION

1.1 General

Soil stabilisation is a regulated process to improve the soil by using additives in order to use it as base or sub base courses and carry the expected traffic and pavement loads. There are several methods by which soils can be stabilised.

There are two methods to enhance the properties of sandy soils, one of them is the mechanical stabilization which is mixed the natural soil and stabilizing material together for obtaining a homogeneous mixture and the second one is adding stabilizing material into un-disturbed soils to obtain interaction by letting it permeate through soil voids . Chemical stabilization is the modification of properties of a locally available soil to improve its engineering performance. The two most commonly used chemical stabilization methods are lime stabilization and cement stabilization.

Methods of Soil Stabilisation:

1) **Lime Stabilisation**

Lime stabilization is one of the oldest methods used to increase strength over the long term. There are two major objectives of the lime stabilization process, with respect to the improvement of clayey subgrade soils, to improve workability and increase strength. The first objective is attained by decreasing the Plasticity Index (PI) and volume change characteristics of the subgrade soil. The second objective is to increase the strength of the subgrade soil over the long term.

Physical and chemical events occur in lime stabilization. Hydrated lime is calcium hydroxide, designated in chemical form as Ca(OH)2. Hydrated lime is produced by reacting quicklime with sufficient water to form a white powder. Hydrated lime is the form of lime used in the majority of lime stabilization procedures. This process is referred to as slaking.

I

n words and in chemical form, this reaction is denoted as:

High Calcium Quicklime + Water $\longrightarrow$ Hydrated Lime + Heat

$CaO + H_2O \longrightarrow Ca(OH)_2$ + Heat.

Almost all fine grained soils display some cation exchange when treated with lime. The reaction is quite rapid on soils that are finely pulverized and intimately mixed with lime. The cations can be listed in approximate order of their replacement ability. In order of increasing replacement power, the ions are as follows.

$Li^+ < Na^+ < H^+ < K^+ < NH_4^+ << Mg^{++} < Ca^{++} << Al^{+++}$. Monovalent cations are more easily replaced by multivalent cations. The addition of lime to soil in adequate quantities provides an excess of Ca^{++} to trigger a cation exchange. Typically, lime [$Ca(OH)_2$] stabilizes a sodium clay soil by replacing the sodium ions (Na^+) in the clay's exchange complex, since calcium has a greater replacement power than sodium. The other important reaction is the pozzolanic reaction in soil-lime stabilization. With the addition of lime, aluminous and silicious minerals in clay react with the lime to produce calcium silicates and aluminates that bond the particles together. Cement, however, provides its own pozzolans and, therefore, only requires a supply of water. Pozzolanic reactions are time and temperature dependant, with lime hydration requiring more hydration time than cement.

2) **Cement Stabilisation**

Strength gain in soils using cement stabilization occurs through the same type of pozzolanic found using lime stabilization. Both lime and cement contain the calcium required for the pozzolanic reactions to occur; however, the origin of the silica required for the pozzolanic reactions to occur differs. With lime stabilization, the silica is provided when the clay particle is broken down. With cement stabilization, the cement already contains the silica without needing to break down the clay mineral. Thus, unlike lime stabilization, cement stabilization is fairly independent of the soil properties; the only requirement is that the soil contains some water for the hydration process to begin.

Similar to lime stabilization, carbonation can also occur when using cement stabilization. When cement is exposed to air, the cement will react with carbon dioxide from the atmosphere to produce a relatively insoluble calcium carbonate. Thus, similar to lime, proper handling methods and expedited construction procedures should be employed to avoid premature carbonation of cement through exposure to air.

3)**Polymer Stabilisation**

A variety of natural polymers, such as lignosulfonate, and synthetic polymers such as Polypropylene(PP),polyester(PET),polyethylene(PE),Glass fibres, etc.are available.It is known that the polymers consist of hydrocarbon chains, and it is thought that these chains become interwined within the soil particles thus producing a stabilizing effect. In effect, the polymers act as a binder to glue the soil particles together reducing dust, and even stabilizing the entire soil matrix.

Polymer amendment for improvement of soils is a growing industry and has been of particular interest in recent field applications. Polymers improve the soil by providing physical stabilization through the use of binding agents. Polymers are easily modified; therefore, a range of polymer combinations can be prepared to modify soils.

1.2.Various tests for determination of Subgrade Quality

In the past, the strength quality of the subgrade soil used in pavement construction had been determined by various laboratory tests such as the California bearing ratio (CBR), Hveem stabilometer and cohesiometer test (The Hveem method), and R-value tests in order to characterize the pavement materials. However, neither of these methods considers the effect of cyclic loading of the vehicular load on the pavement due to static nature of their loading conditions.

The recent development in pavement design includes the introduction of stiffness based modulus, called the resilient modulus, which deals with the repeated loading condition on the materials to be tested, thus simulating the actual vehicular loading in the field. The repeated loading triaxial test is performed within the elastic range of the soil in order to determine the resilient modulus. On the other hand, the permanent deformation deals with the cyclic loading of materials beyond the elastic limit or sometimes up to failure of the specimens in order to evaluate the rutting performance (single-stage tests) and different shakedown stages (or limits) of the materials (multi-stage tests). Despite the more precise results from resilient modulus and permanent deformation tests, some designers and contractors still prefer using CBR value or any other conventional method in the design of pavement rather than the use of resilient modulus due to associated low cost and lesser time compared to the repeated loading triaxial tests.

1.3 Objectives of Study

The adverse effect of increase in moisture content on the soil behavior has been a major concern among the geotechnical as well as pavement engineers. Soil possesses excellent performance at the optimum moisture content or below the optimum moisture content (dry side of optimum); however the strength and stiffness of soils reduces drastically as the moisture content increases beyond the optimum (wet side of optimum). Due to soft nature of soil in some regions and with the presence of high water table strength/stiffness of subgrade soil is too weak to support the pavement loads. In addition, some soils have great tendency to shrink/swell with moisture content and often creates serviceability problems during or after construction of the foundations or pavement layers. The replacement of such soil with better quality of borrow soil fill is not always a good option especially in pavements due to associated cost of excavation and hauling of the materials.

In order to cope with this problem, various techniques have been applied by engineers depending upon the types of the soil. For example, mechanical stabilization is preferred to coarse grained soils. But, in some regions, with soft clay subgrade and high water table, it is customary to treat the soils with some chemical stabilizers or calcium rich stabilizers. These stabilizers not only provide the working platform for construction through enhancing the strength of treated subgrade layer; but also can give the relatively stable subbase for pavement. The shrink/swell characteristic of the soils is a function of in situ moisture content.

Most of the soils have in situ moisture content higher than the optimum, and therefore the prediction of subgrade behavior based only on the property around the optimum or near the optimum on either side is not enough. The use of different stabilizers based on the properties of the raw to treated/stabilized subgrade soil has made it easier to construct pavement on high moisture contents and weak soil subgrade.

1.4. Methodology

In this Study, cement and Polymer is used for soil stabilization. Cement and Polymer content is varied in two types of soil viz. Sandy and Clayey Soil. The effect of Unconfined compression strength and CBR values are studied in the Experiment with the variation in contents of cement and Polymer. Firstly mechanical analysis of two types of soil are done,then the soil is mixed with different contents of cement and polymer and their variation are studied.

1.5. Closing Remarks

The use of cement and polymers has good effect on stabilisation of soil which can be observed from the experiments. More the quantities of additives are used, less will be the variation in values. The use of Additives has good impact on soil Stabilisation but at the optimum value, its effects are more pronounced.

Chapter 2

BACKGROUND & LITERATURE REVIEW

2. 1 General

Extensive research has been completed pertaining to the use of traditional stabilizers, namely lime and cement. The stabilization mechanisms for lime and cement are well documented, and the effectiveness of these traditional stabilizers has been demonstrated in many applications. However, relatively little research documenting the use of nontraditional stabilizers such as synthetic polymers, and magnesium chloride is available, and their performance record is varied. Although much promotional material exists attesting to the effectiveness of nontraditional stabilizers, such materials often lack documentation of measured engineering properties, and often they do not explain the stabilization mechanism involved. This literature review focuses on the known properties of both traditional and nontraditional stabilizers. The literature review also discusses factors influencing development of the laboratory test procedures used for this research.

2.2 Lime Stabilization and review of Works

2.2.1 Effect of freezing and thawing on strength and permeability of lime-stabilized clays (Yıldız et al,2012) examined thin sections of frozen sedimented silt and clay and found horizontal ice lenses perpendicular to the direction of freezing, and vertical ice-filled shrinkage cracks that were linked to form columns with polygonal cross sections. Othman and Benson investigated the effect of compaction conditions (molding water content and compactive effort) and external conditions (temperature gradient, ultimate temperature, dimensionality of freezing, number of freeze-thaw cycles and state of stress) on the hydraulic conductivity of three compacted clays of different properties. Laboratory studies indicate that the number of freeze-thaw cycles, rates of freezing and states of stress have the largest effect on the change in hydraulic conductivity. The hydraulic conductivity increases as the rate of freezing and number of freeze-thaw cycles are increased and as the overburden pressure is decreased.

Other factors, such as the ultimate temperature, dimensionality of freezing, and availability of an external supply of water, do not appear to have a significant effect on the change in hydraulic conductivity. The effect of freezing and thawing on the strength characteristics of clay soils has not been researched by these researchers. It has been reported that freeze/thaw cycles detrimentally affect strength gain for several types of peat with moisture contents in the range of 200%. In their investigation, stabilized peat samples exposed to eight freeze/thaw cycles (−10 °C/ + 20 °) lost 30% of their strength as compared to those that were not exposed to these cycles. If adequate lime is available, pozzolanic reactions will continue to occur under favorable conditions.

In this study, two types of soil that have different plasticity were used. While Soil-1 from Doğanhisar (Konya) had low plasticity (CL), Soil-2 from Ortaköy (Aksaray) had high plasticity (CH) Hydrated high-calcium lime [Ca(OH)2] was used as a stabilizer. In their study, strength and permeability alterations of two types of clay sample during curing were studied. These samples were also subjected to the freezing-thawing process during curing.

The type of preparation of pure clay samples is different from samples stabilized with lime. For pure clay samples, distilled water was lightly sprayed onto the material. For lime stabilized clay samples, firstly, 6% lime was mixed with dry clay samples, then, water was added.

Both clay and lime were compacted at water content at about 2% of optimum water content using standard Proctor compaction, because soil liners have traditionally been compacted in the field over a specified range of water content. Lime stabilized samples were cured for 1, 3, 7, 21 and 28 days at room temperature. Before curing, samples were wrapped with nylon film, and then covered with aluminum foil and put into nylon bags so as not to lose their water content. When the curing times ended, a series of unconfined compression tests and permeability tests were conducted on the samples.

The permeability results of Aksaray and Doğanhisar clays are presented in Tables 2.1 and 2.2. The effects of curing time and freeze-thaw are summarized in Table 2.3 for the two types of clay.

Table 2.1 Results of permeability tests for Aksaray clay

Specimen	Curing time (day)	k (cm/s) (without freeze-thaw)	k (cm/s) (with freeze-thaw)	% Change of permeability
Without lime	0	7×10^{-7}	7×10^{-7}	0
%6 lime	1	1×10^{-6}	1×10^{-5}	900
%6 lime	3	5×10^{-6}	7×10^{-5}	1300
%6 lime	7	1×10^{-5}	8×10^{-5}	700
%6 lime	21	5×10^{-5}	1×10^{-4}	100
%6 lime	28	9×10^{-5}	2×10^{-4}	1222

Table 2.2 Results of permeability tests for Doğanhisar clay

Specimen	Curing time (day)	k (cm/s) (without freeze-thaw)	k (cm/s) (with freeze-thaw)	% Change of permeability
Without lime	0	5×10^{-9}	5×10^{-9}	0
%6 lime	1	1×10^{-6}	1×10^{-5}	900
%6 lime	3	5×10^{-6}	5×10^{-5}	900
%6 lime	7	6×10^{-6}	8×10^{-5}	12,333
%6 lime	21	7×10^{-6}	9×10^{-5}	11,857
%6 lime	28	8×10^{-6}	8×10^{-5}	900

Table 2.3 Unconfined compression test results for Aksaray and Doğanhisar clays treated with 6% lime

Specimen	Curing time (day)	Unconfined compressive strength q_u (kN/m^2) (without freeze-thaw testing)	Unconfined compressive strength q_u (kN/m^2) (with freeze-thaw testing)	% Change of strength
Doğanhisar clay	1	428	375	123
	3	542	448	173
	7	623	562	98
	21	828	723	127
	28	995	902	93
Aksaray clay	1	548	461	159
	3	1138	989	131
	7	1676	1474	121
	21	3422	2856	165
	28	3878	3326	142

Permeability-curing time relationships for Doğanhisar and Aksaray clays are plotted in Figures 2.1 and 2.2.

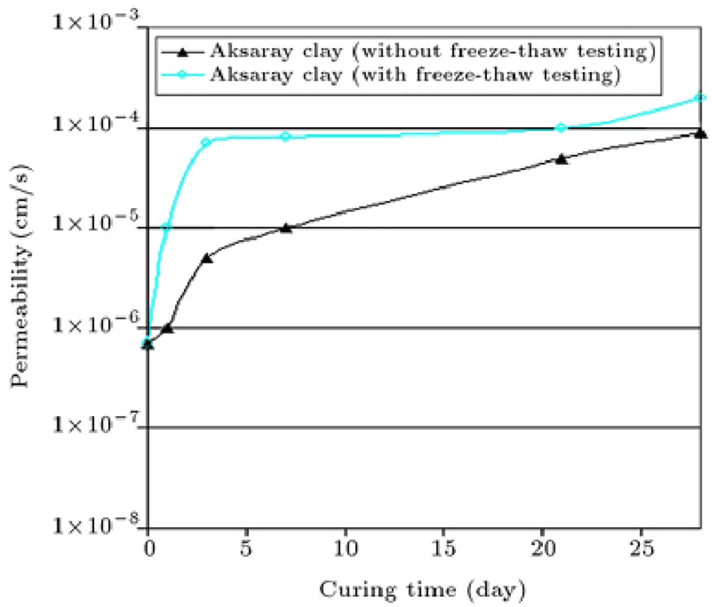


Fig.2.1 Permeability-curing time relationship for Aksaray clay treated with 6% lime.

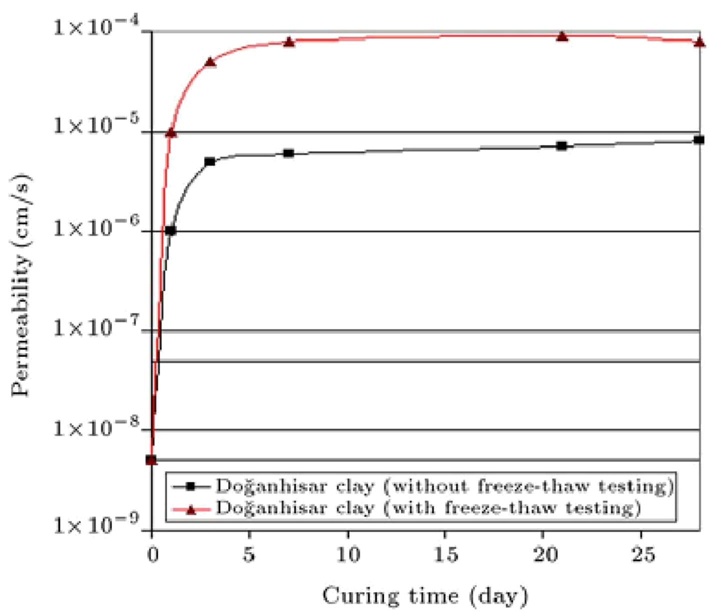


Fig.2.2 Permeability-curing time relationship for Doğanhisar clay treated with 6% lime.

For both clays, lime addition flocculated the clay soil particles and increased the value of the hydraulic conductivity of specimens 1000 times. These studies showed that the hydraulic conductivity of compacted clay increased by 10 to 20 times after only 3 freeze-thaw cycles.

The unconfined compressive strength of the two clays exhibited different results. The strength of the high plasticity clay (Aksaray clay) increased approximately 15 times at the end of 28 day curing, while that of the low plasticity clay (Doğanhisar clay) increased approximately 3 times after the same curing period.

Conclusions

1. Calcium is the most important ingredient in the stabilization of clay. Lime provides calcium through the dissolution of calcium hydroxide in the presence of water.

2. Lime changes the properties of clay through a series of physiochemical modifications, including cation exchange, flocculation and agglomeration, and pozzolanic reaction.

3. Lime stabilization was more efficient on Aksaray clay. This effect could be justified by the high value of silica and calcium components used in that clay.

4. The effect of freezing-thawing does not stop the pozzolanic reaction; however, it retards the reaction in the clay soil stabilized with lime. This situation shows that clay soils can be stabilized with lime in cold seasons.

5. Determining the levels of freezing effects of pozzolanic reactions occurring in two different clay types, having different plasticities and stabilized with lime, on permeability and strength is the significant objective of this study. In this study, differing from previous researchers, two types of clay having different pozzolanic reactions were used and compared with each other.

2.2.2 Impact of cyclic wetting–drying on swelling behavior of lime-stabilized soil (Guney et al,2005) The stabilization, especially with lime, is a common applied method among the others due to its effective and economic usage. The findings from the previous studies show that when lime is added to clay soils in the presence of water, reactions including cation exchange, flocculation and pozzolanic reaction take place. It is stated that, flocculation is primarily responsible for the modification of the engineering properties of clay soils when treated with even a small amount of lime. The studies reported in the literature showed that the addition of lime increased the optimum water content, shrinkage limit and strength, and reduced the swelling potential, liquid limit, plasticity index and maximum dry density of the soil. The optimum addition of lime needed for the stabilization of the soils is between 1% and 3%, while the other researchers suggested the use of lime between 2% and 8% lime by weight. It is also indicated that further additions of lime do not change the swelling potentials, but increase the engineering properties of expansive soil.

Three different types of clayey soils and a lime have been adopted for this study. The soils consist of Na-bentonite and kaolinite mixtures and a natural soil called Turkmen clay. **Preparation of samples:** In this study, Na-bentonite and kaolinite clays are prepared by mixing in two different proportions to ascertain two types of high expansive clay soil. The first mixture, named as SoilA, is in the form of 70% of bentonite and 30% of kaolinite, by dry weight, whereas in the second mixture, SoilB, constitutes of 30% of bentonite and 70% of kaolinite, by dry weight. In addition, as a natural high plasticity clayey soil, Turkmen clay is used, which is named as SoilC. The lime stabilization of the soil samples is conducted with addition of two different proportions of 3% and 6% lime by weight of soil.

Table 2.4 The sieve analysis and the Atterberg limits of the samples are given in Table 2.4

Table 2.4 sieve analysis and the Atterberg limits

Sieve analysis and Atterberg limits of the samples

Samples	Lime rate (%)	Sieve analysis (%)			Atterberg limits (%)			
		Sand	Silt	Clay	Liquid limit	Plastic limit	Plasticity index	Shrinkage limit
SoilA	—	4	15	81	385	35	350	23
SoilB	—	9	18	73	168	28	315	27
SoilC	—	7	18	75	115	45	208	25
SoilA + 3L	3	—	17	83	360	45	140	26
SoilA + 6L	6	—	20	80	255	57	123	29
SoilB + 3L	3	—	20	80	160	37	95	30
SoilB + 6L	6	—	24	76	140	45	70	35
SoilC + 3L	3	7	19	74	104	49	55	41
SoilC + 6L	6	6	21	73	103	50	53	58

3 L: 3% lime, 6 L: 6% lime.

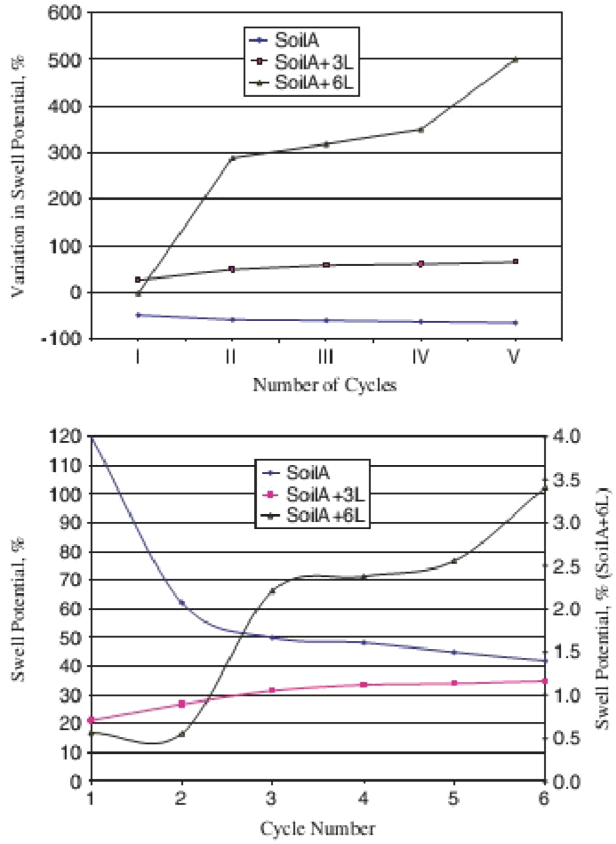
As seen from the above table, the stabilization of the samples with lime resulted in decreasing in liquid limit, and increasing in plastic limits.

Swelling potential test

The swell percent of the specimen was measured using the loaded-swell method. It was carried out by the standard one-dimensional oedometer. The sample, which is in the ring, was placed between two porous stone which has a number of small holes. The ring with the specimen was placed on the lower porous stone, followed by placement of the upper porous stone. The specimen was then loaded to a seating pressure of 0.7kPa and it was maintained with water to cover the top porous plate. The displacement measurement, dial gauge, was initialized. The specimen was allowed to swell under the initial seating load. The dial gauge readings were recorded periodically until no further changes in expansion were observed, which the swelling potential was fully reached. The swell percent (e), which is defined as the percentage increase in height in relation to the original height and was calculated as $e=ð/h0$, where ð is the axial expansion in mm, and ho is the original thickness of the specimen in mm.

Swelling pressure

This test is intended to measure the axial stress necessary to constrain radially confined soil specimen at constant thickness when immersed in water within in the cell. In other words, the swell pressure of test specimen was measured using the constant volume method. The pre-paration of the specimen and the consolidation cell were the same as in the axial swell percent test. It was then flooded with water and the volume was kept constant by continuous addition of loads at each axial expansion of the specimen, observing the dial gauge displacement keeping at zero level. The load was maintained by using loading arm of the oedometer. The addition of load via the loading arm was continued until deformation of the specimen could not be observed. The swelling pressure was calculated from the sum of the load increments divided by the cross-sectional area of the specimen and was calculated as $\alpha = N/A$,Wheres α is the axial stress in kPa, N is the total load in kN, And A is the area of cross section of the specimen in m2.



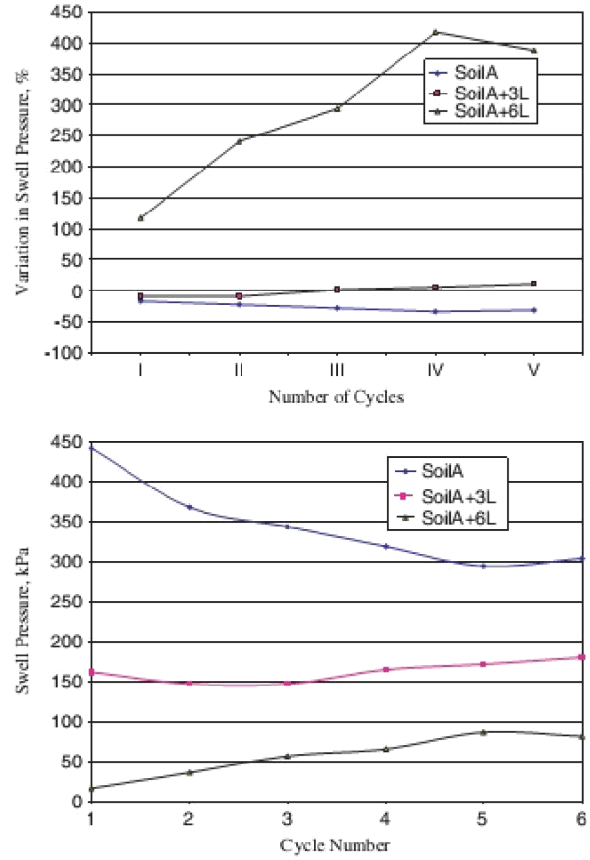
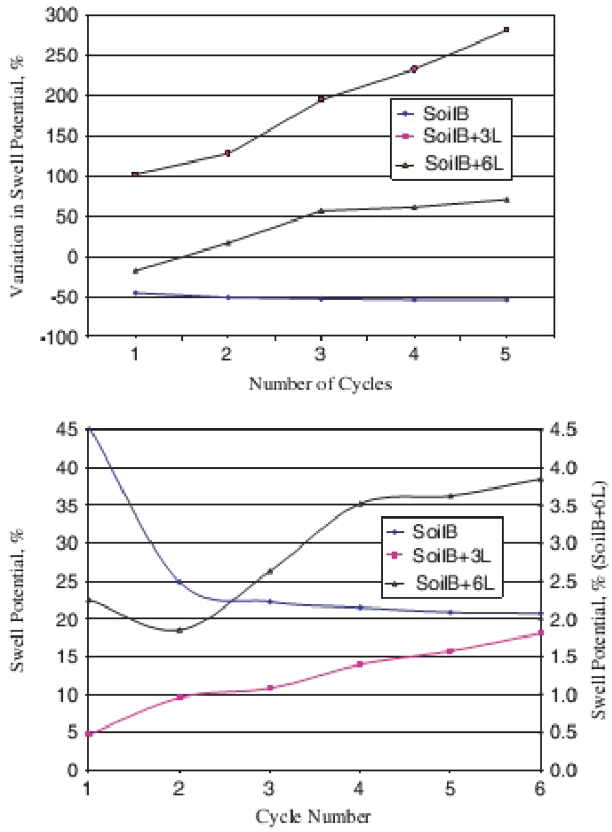


Fig.2.3 Change of swell percent with no of cycles for soil A type Fig.2.4 Change of swell pressure with no of cycles for soil A type

The wetting–drying cycling effect on swelling potential and swelling pressure for all SoilA type of samples are given in Figs.2.3 and 2.4, respectively. From these figures, it can be seen that both of the swelling potential and swelling pressure decrease with increasing number of cycles, for Soil A. On the other side, the lime-stabilized soil samples, SoilA+3L and SoilA+6L, show a gradual increase in swelling percent and pressure from the first cycle to the fourth cycle. One of the reasons of the low swell potentials of lime-treated samples, observed initially, can be due to their higher optimum water content and lower maximum dry unit weight. The increment of clay content for the lime-treated clayey soil samples, probably suggests partial breakdown of cemented soil aggregates due to cyclic wetting and drying process. This could also be responsible for the small to moderate swelling.



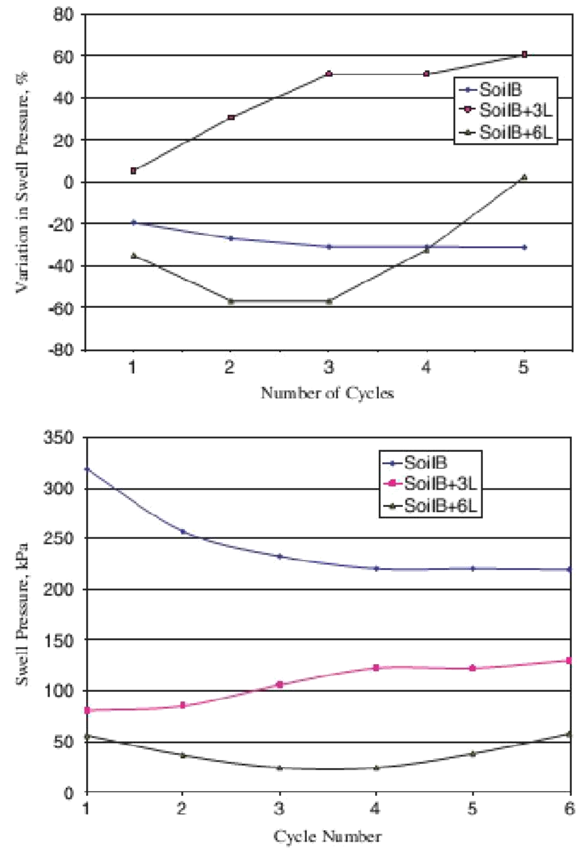
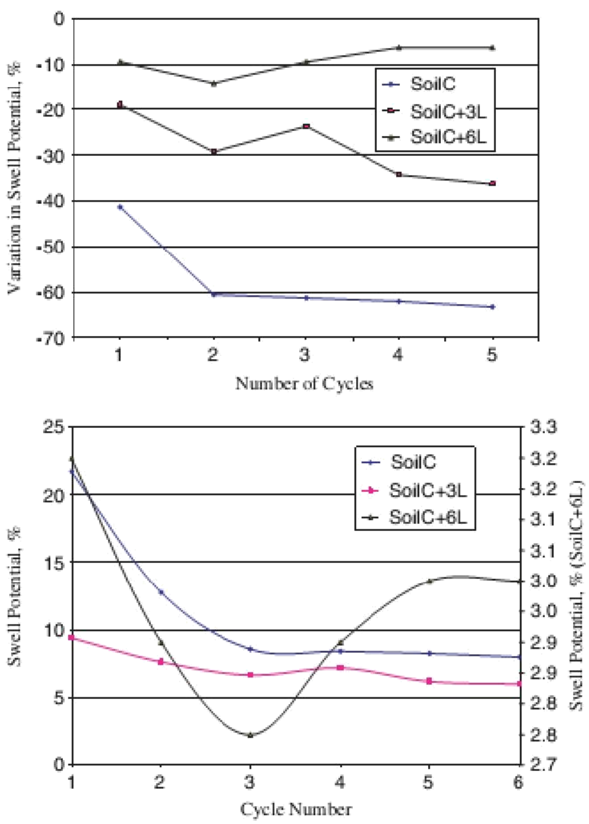


Fig.2.5 Change of swell percent with no of cycles for soil B type Fig.2.6 Change of swell pressure with no of cycles for soil B type

The results of the swelling potential and swelling pressure tests for Soil B and lime-stabilized Soil B are presented in Figs. 2.5 and 2.6. An examination of the plots in these figures shows that the variation in swelling potential and swelling pressures corresponding to the number of cycles follows similar trends as those observed in the Soil A tests. For untreated soils a decrease in swell behavior is observed as the number of cycles increase whereas an increase in swelling percent is noted for lime-stabilized Soil B samples.



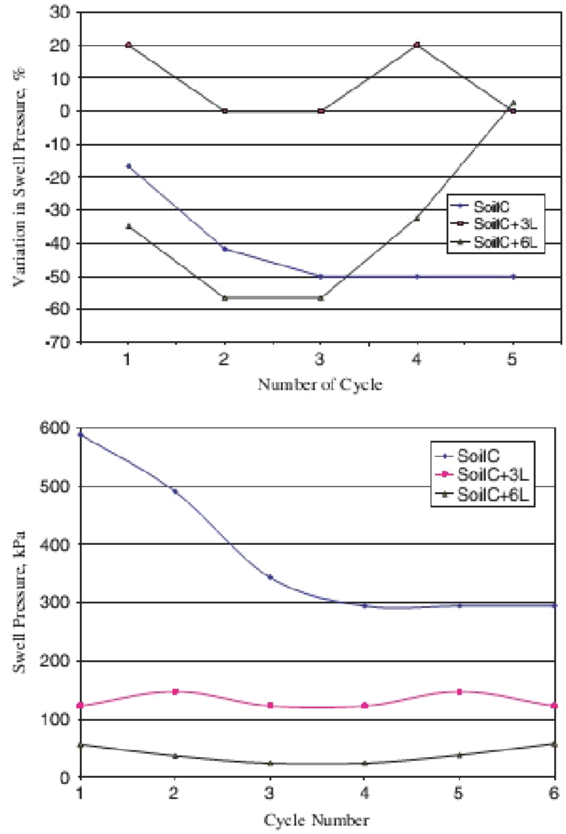


Fig.2.7 Change of swell percent with no of cycles for soil C type Fig.2.8 Change of swell pressure with no of cycles for soil C type

Figs 2.7 and 2.8 show the swelling behavior of the SoilC,SoilC+3L, and SoilC+6L samples subjected to six cycles. of wetting and drying. The SoilC samples show a drop in both of the swelling percent and the swelling pressure. The behavior of the curve, SoilC curve, is similar to the other tested untreated soil samples. Addition of 3% and 6% lime causes significant reduction in both the swell percent and the pressure, initially. Then the swelling percent decreases subsequent wetting and drying cycles, but swelling pressure increases a little bit at the period of the first two cycles. Both of the swelling percent and swelling pressure curve follow gradual increase or decrease, rather than abrupt behavior. On the other hand, the SoilC shows abrupt decrease in the first cycle and follow gradual decrease in the subsequent cycles.

Conclusion: The beneficial effect of lime stabilization in controlling the swelling potential of lime-treated samples is partially lost, on subjecting them to cycles of wetting and drying. The clay content of the cycled samples increases that, in turn, affected the liquid limit,plastic limit, shrinkage limit and swell potential of the lime-treated expansive soil.

According to these results, it has been found out that lime stabilized expansive clayey soil, must not be used at the regions where wetting and drying cycles are significantly effective.

2.2.3 Lime treatment of Laterite Soils (N.A. Attoh- Okine,1995):

Laterite as a soil group, rather than a well-defined material, is found in leached soils of humid tropics. They are highly weathered reddish tropical soils that have concentrated oxides of iron and aluminium with kaolin as the predominantly clay mineral; gravel nodules are cemented by iron and aluminium sesquioxides, and precipitation of the oxides with increasing crystallinity and dehydration as the soil becomes weathered. Variation of UCS with lime is shown in fig.2.9.

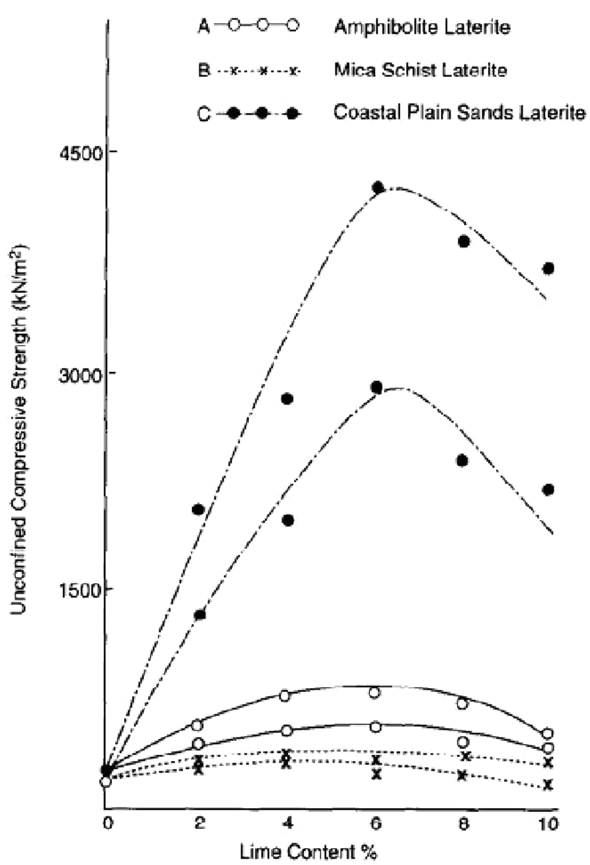


Fig.2.9 Variation of UCS with lime

Unconfined compression

In all lime-treatment lateritic soil investigations, the unconfined compressive strengths were determined from samples. The unconfined compression strength increases as the curing time increases. Balogun reported that the addition of lime (Above figure) increases the compressive strength to a maximum value; thereafter, the strength drops with additional lime.

This research attempted to investigate the consistency of results of lime-treated lateritic soils and gravels in an engineering context. At this point it is very difficult to generalize results. However, one can speculate that, as in 'temperature soils', the characteristics of lime-treated soils depend on the quantity of lime, curing time, environmental conditions and the method of testing. Statistically valid data needs to be collected from various testing results to help in preparing a guide for lime-treated laterites and developing a criterion that should be used to assess the suitability of the lime- treated laterite soil as a pavement material.

2.3 Cement Stabilization and review of Works

Cement stabilization is ideally suited for well graded aggregates with a sufficient amount of fines to effectively fill the available voids space and float the coarse aggregate particles

2.3.1 Stabilization of residual soil with rice husk ash and cement (Basha et al,2004):

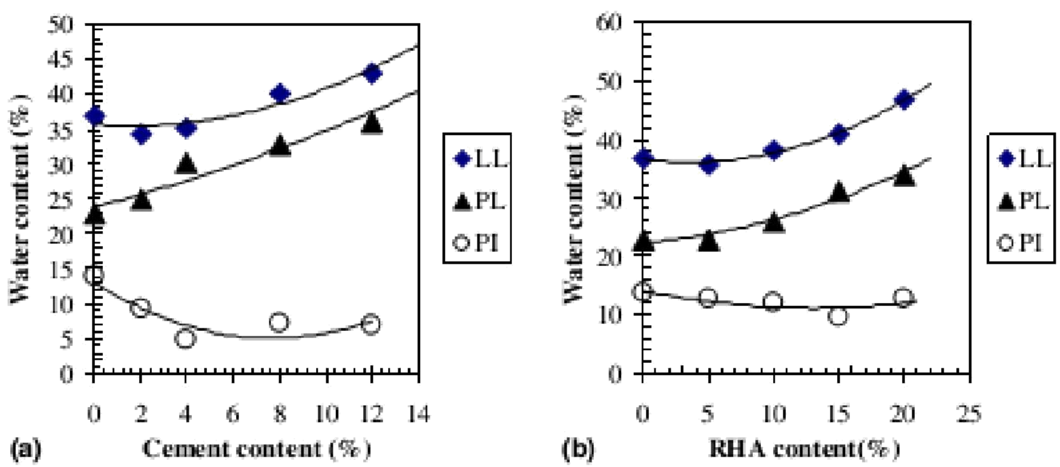
Stabilized soil is, in general, a composite material that results from combination and optimization of properties in individual constituent materials. Well-established techniques of soil stabilization are often used to obtain geotechnical materials improved through the addition into soil of such cementing agents as Portland cement,lime, asphalt, etc.

Replacement of natural soils, aggregates, and cement with solid industrial by-product is highly desirable. In some cases, a by-product is inferior to traditional earthen materials. Due to its lower cost, however, it makes an attractive alternative if adequate performance can be obtained. In other cases, a by-product may have attributes superior to those of traditional earthen materials. Often selected materials are added to industrial by-products to generate a material with well-controlled and superior properties.

The primary objective of this study is to examine the potential of burnt agricultural by-product, rice husk, as a material for stabilising soil. The effects on the consistency, density, and strength of residual soil are studied..

Atterbergs limits, compaction tests, unconfined compressive strength and durability Tests and CBR test were conducted. The tests were carried out on the soils with different proportion of cement and rice husk ash (RHA).

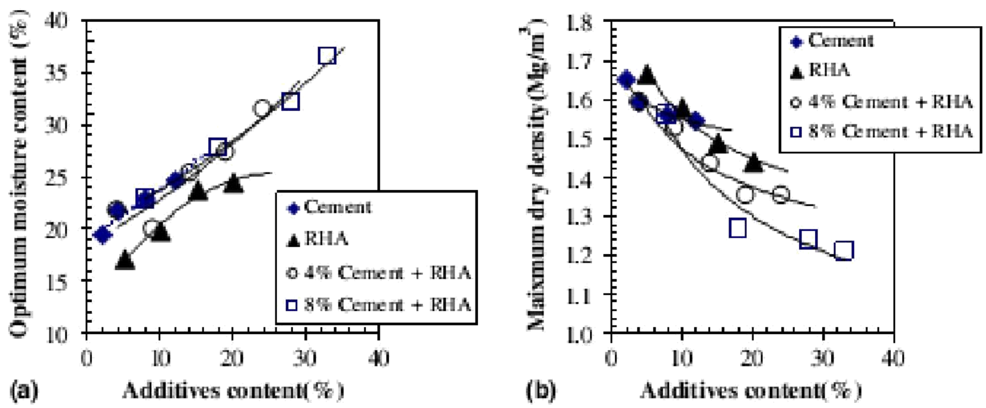
Effect on the consistency limits:The effect of cement and RHA stabilized soils on the liquid limit (LL) and plasticity index (PI) on the different soils are shown in Fig.2.10. It can be observed that cement and RHA reduce the plasticity of soils. In general, 6–8% of cement and 10–15% RHA show the optimum amount to reduce the plasticity of soil. Reduce in the PI indicate an improvement.



Variation of consistency limits: (a) cement-stabilized residual soil; (b) RHA-stabilized residual soil.

Fig. 2.10 Variation of consistency limits

Effect on the compactability: Fig. 2.11 shows the effect of the addition of cement, RHA, and cement–RHA mixtures on the compaction characteristics of the soils tested. The figure depicts that adding cement and RHA increased the OMC and diminish amount of the MDD correspond to increasing of cement and RHA percentage. The increase in OMC is probably a consequence of two reasons: (1) the additional water held with the flocculant soil structure resulting from cement interaction, and (2) exceeding water absorption by RHA as a result of its porous properties. It was revealed that the change-down in dry density occurs because of both the particles size and specific gravity of the soil and stabilizer. Decreasing dry density indicates that it need low compactive energy(CE) to attain its MDD. As a result, the cost of compaction becomes economical.



Variation of compaction characteristics: (a) optimum moisture content; (b) maximum dry density.

Fig.2.11 Variation of compaction characteristics

Effect on the compressive strength: The effect of the addition RHA and cement on the unconfined compressive strength is shown in Fig.2.12.Cement shows undoubtedly a very effective additive to enhance the strength of tested soils. In Fig.2.12,it can be observed that the optimum cement content is 8%. It corresponds with the optimum cement content that reaches to the consistency limit. In contrast with RHA–soil mixtures, the RHA slightly increases the strength because of the lack of cementitious properties in RHA. This investigation shows that cement-stabilized soils can be intensified by adding between 15–20% of RHA as shown in Fig. 2.12. The figure either shows that 4% cement mixed with residual soil and 20% RHA, kaolin with 4% cement and 15% RHA, and bentonite with 4% cement and 15% RHA have a strength, respectively, almost 4, 2, and 1.4 times that of a sample with 8% cement. A lesser amount of cement is required to achieve a given strength as compared to cement-stabilized soils. Since cement is more costly than RHA this results in lower construction cost.

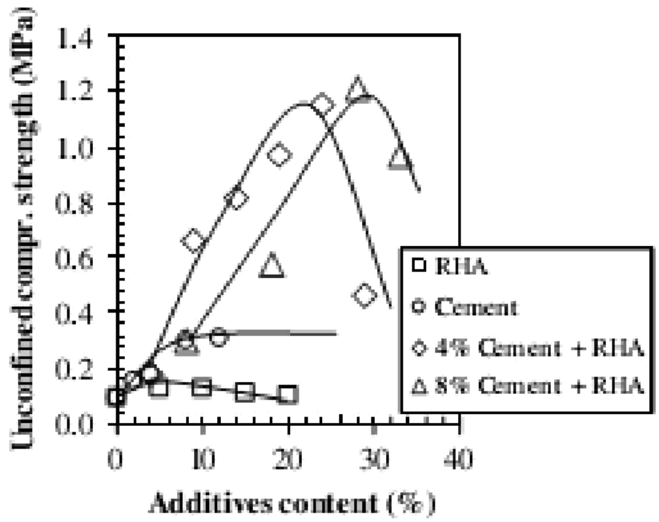


Fig. 2.12 Effect of the addition of RHA and cement of unconfined compressive strength.

Effect on California bearing ratio: The laboratory determination of the CBR of a compacted specimen was obtained by measuring the forces required to cause a cylindrical plunger of a specified size to penetrate the specimen at specified rate. As with the unconfined compressive strength, the CBR increases with addition of lime Fig. 2.13a but, however, the RHA-treated residual soils decrease the CBR value. This, again, alludes that RHA alone is not suitable as stabilizer. Combination between RHA and cement yields a significant enhancing of strength as well as CBR (Fig. 2.13(b)). This result confirms that 4% cement–5% RHA mixtures, and 8% cement–20% RHA mixtures attain the maximum CBR value, respectively, 60% and 53%. Multiple enhancement of CBR value is reached when lesser cement content and RHA is mixed. Further, this is an benefit for road construction.

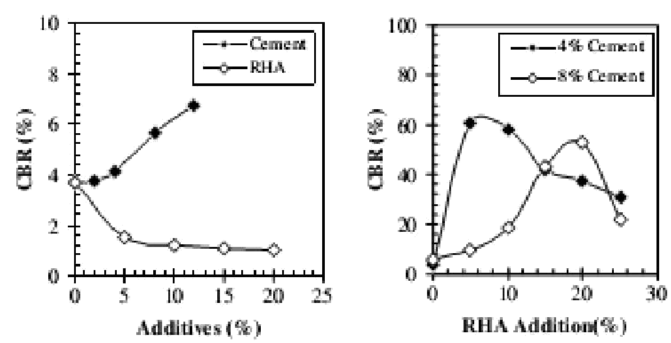


Fig.2.13 Effect of cement and RHA addition on CBR

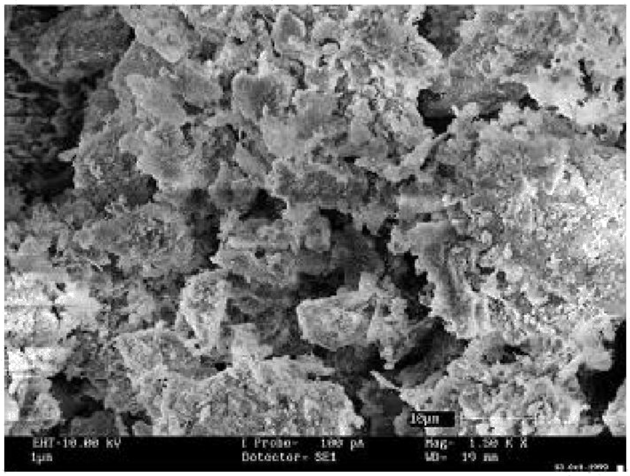


Fig.2.14 Scanning electron micrograph of stabilized soil with 4% cement and 20% RHA.

Conclusions: 1. Cement and RHA reduce the plasticity of residual soil. A considerable reducing is attained by cement-stabilized soils.

2. The MDD of cement-stabilized residual soil slightly decreases with the increase in cement content. Adding RHA and cement, the OMC is increased steeply.

3. The unconfined compressive strengths of cement-stabilized soils increase with addition of RHA. Addition of RHA needs a lesser amount of cement to achieve a given strength as compared to cement-stabilized soils. The resistance to immersion shows a better improvement. Since cement is more costly than RHA this can result in lower construction cost.

4. The increase in CBR value corresponds to the increase in cement content. Adding RHA into cement-treated residual soil, the CBR value increase multiply. The maximum CBR, as much as 60% is found at combination of 4% cement and 5% RHA.

5. In general, 6–8% of cement and 15–20% RHA show the optimum amount to improve the properties of soils. Reduce in PI and increase in strength and resistance to immersion indicate an improvement.

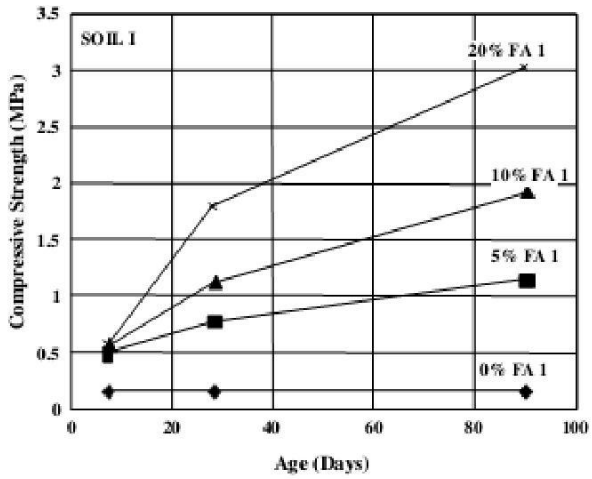
6. RHA can potentially stabilize the residual soil, either solely or mixed with cement. Utilizing is an alternative, it is available to reduce construction cost, particularly in the rural area of developing countries.

2.3.2 Stabilisation of clayey soils with high calcium fly ash and cement (Kolias et al,2005): High calcium fly ash is produced in large quantities in electric power thermal plants using lignite as the main combustion material. The fly ashes before being distributed for use are usually homogenized and processed in order to slack all the contained free lime. In this work fly ash without the latter process is used in order to take advantage of the free lime in stabilising the fine-grained plastic soils. Cement was also used as a second additive to enhance the strength, especially at early ages.

All specimens were prepared with the static compaction method at the optimum moisture content and maximum density determined by the standard compaction test. The specimens were demoulded 1 min after completion of the compaction, were wrapped with thin plastic film and were stored in the curing room until testing at 7, 28 and 90 days.

Atterberg limits:All the materials (Soils I, II and III) became non-plastic 24 h after mixing with 5%, 10%, 20% FA by mass. In addition, all materials became friable 24 h after the initial mixing with FA and the lumps were minimised after remixing.

Figs.2.16-2.18 show the development of the unconfined compressive strength in relation to curing time for Soils I, II and III respectively. It can be seen that considerably higher compressive strengths are obtained with Soil I than with Soils II and III. The effect of hydrated lime on the strength gain of the mixes is shown in Fig.2.19. Since the strength values obtained with lime are considerably lower than the values obtained with FA it is inferred that the effect of fly ash on strength is not due to its free lime content alone but also to hydraulic and pozzolanic reactions.



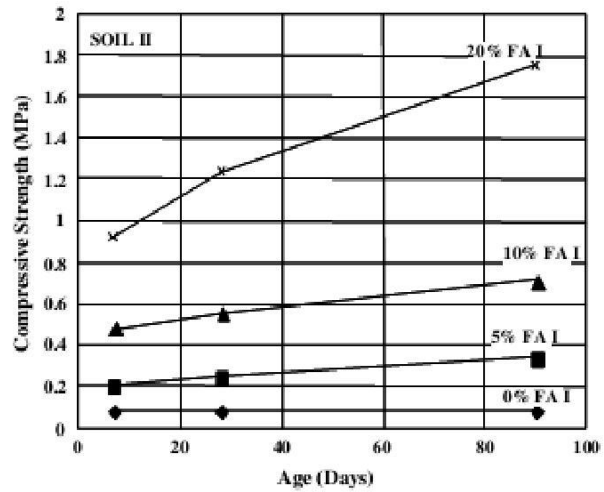
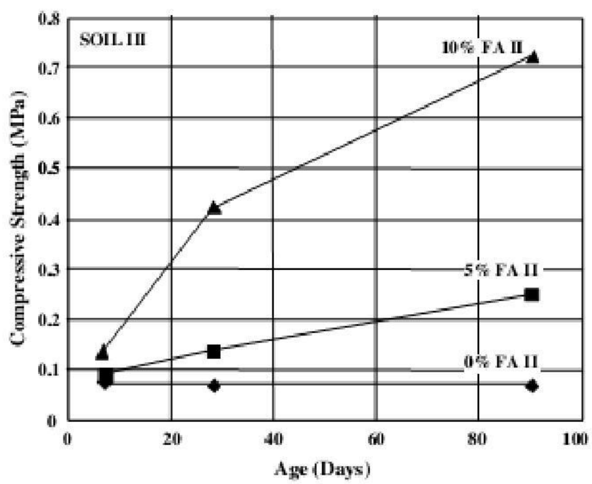


Fig.2.16 Effect of fly ash addition on uniaxial compressive strength–Clay I

Fig2.17 Effect of fly ash addition on uniaxial compressive strength––Clay II



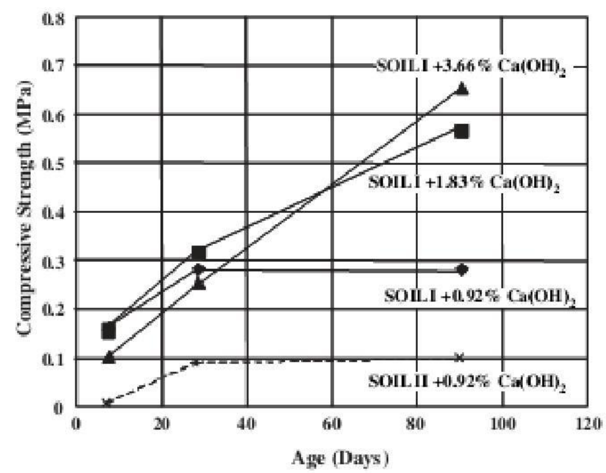
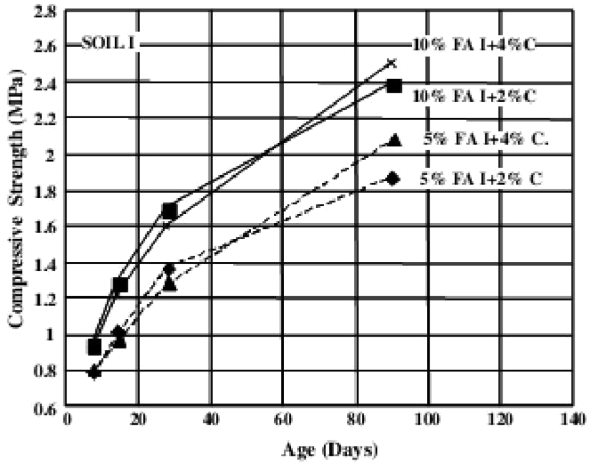


Fig.2.18 Effect of fly ash addition on uniaxial compressive. strength–Clay III

Fig.2.19 Effect of Ca(OH)2 content on uniaxial compressive strength––Clays I and II.



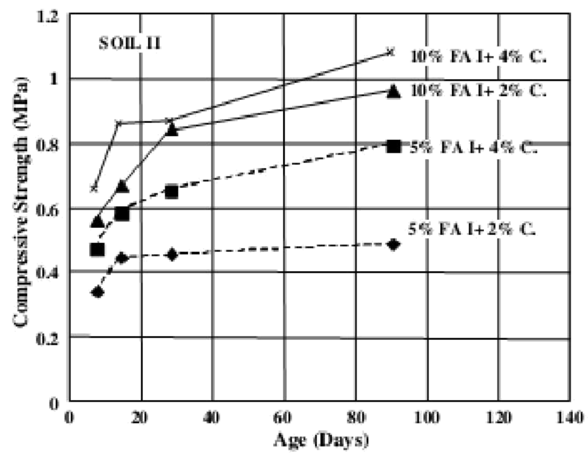
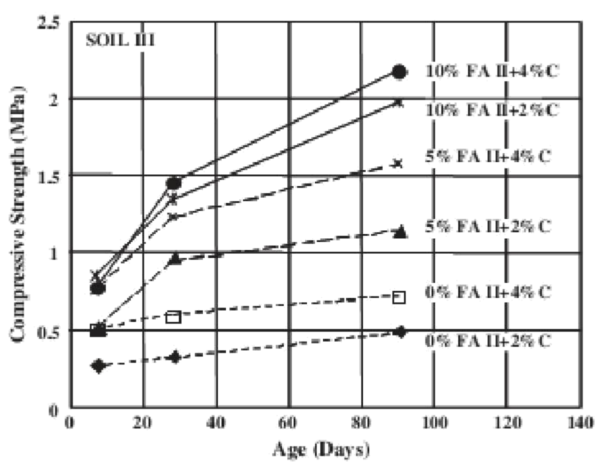


Fig.2.20 Effect of combination of fly ash and cement addition on uniaxial compressive strength––Clay I.

Fig.2.21 Effect of combination of fly ash and cement uniaxial compressive strength––Clay II

The effect of combining 2% and 4% cement with fly ash is shown in Figs.2.20-2.23 for Soils I, II and Soil III respectively. The early strengths are higher for cement. It should be noted that the 90-day strength of both Soil I, and Soil II are higher when 20% of fly ash is used than with the combination of 10% fly ash and 2% or 4% cement. However, in the case of Soil III the effect of combining cement with FA is more pronounced since the 90-day strengths are increased by two to six times. It is evident therefore, that the soil type greatly influences the results.



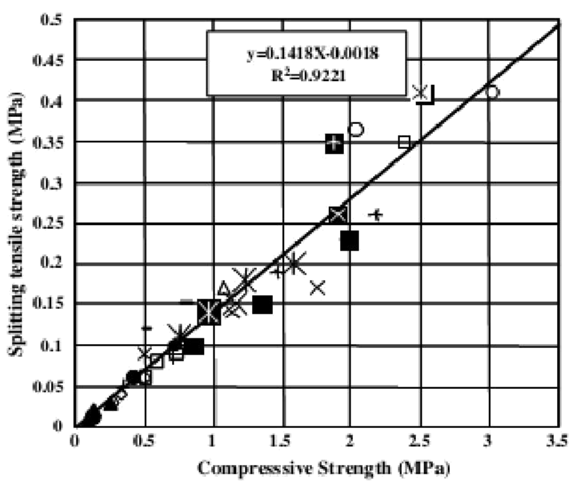


Fig.2.22 Effect of combination of fly ash and cement addition on compressive strength––Clay III

Fig.2.23 Relationship between compressive uniaxial and splitting tensile strength of Clays I, II and III.

CBR tests:Fig.2.24 shows the 90-day old (24-h soaked) CBR values in relation to soil type and fly ash content. It can be seen that as in the case of strength, much-improved CBR values are obtained in case of Soil I while in case of Soils II and III the increases are not as high. It should be noted however, that the 15% minimum CBR value usually required by many specifications is by far attained by the three soils with only 5% fly ash. The relation of CBR vs compressive strength is shown in Fig. 2.25. It can be seen that, for the three fine-grained soils examined, a linear relationship between CBR and strength exists although this applies strictly to the soils examined.

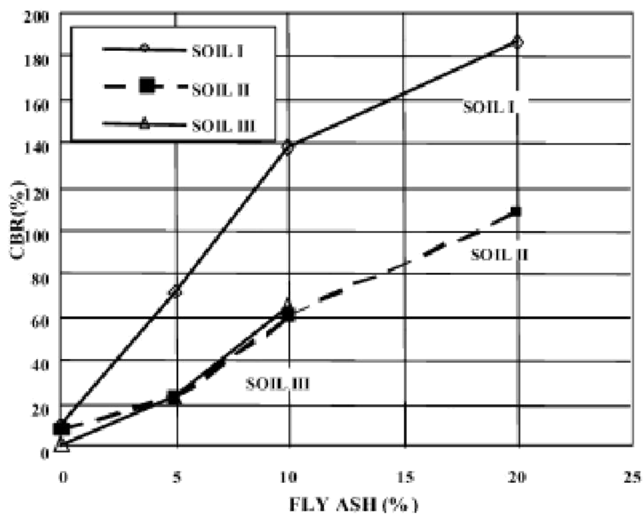


Fig.2.24 Variation of Californian Bearing Ratio for Clays I, II and

III stabilised with fly ash.

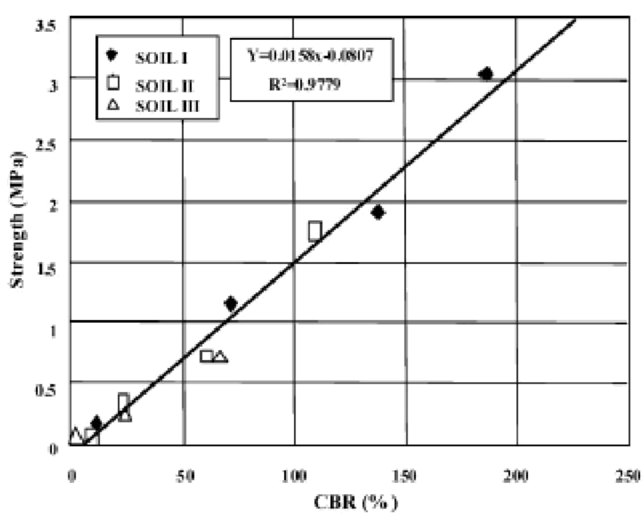


Fig. 2.25 Relation between CBR and strength in uniaxial compression

Conclusions: This work shows that the potential benefit of stabilising clayey soils with high calcium fly ash but this depends on the type of soil, the amount of stabilising agent and the age. The study of the formation of the hydraulic products during the curing of clay containing as a stabilising agent high calcium fly ash shows that a significant amount of tobermorite is formed leading to a denser and more stable structure of the samples. A further addition of cement provides better setting and hardening and the combination of these two binders can increase the early as well the final strength of the stabilised material. The free CaO of fly ash reacts with the clay constituents(SiO_2 and the other aluminium silicates) leading to the formation of tobermorites and calcium aluminium silicate hydrates as well.

The mechanical properties such as strength (compressive, tensile and flexural), modulus of elasticity and CBR are considerably increased. If suitable measures are taken in order to avoid or minimise cracking of the stabilised layer and maintain the high modulus values,substantial reductions of the total pavement thickness,specially the asphalt or Bitumen layer can be achieved.

2.4 Stabilization Using Polymers

A variety of natural polymers, such as lignosulfonate, and synthetic polymers such as Polypropylene(PP),polyester(PET),polyethylene(PE),Glass fibres, etc. are available. It is known that the polymers consist of hydrocarbon chains, and it is thought that these chains become entwined within the soil particles thus producing a stabilizing effect. In effect, the polymers act as a binder to glue the soil particles together reducing dust, and even stabilizing the entire soil matrix.

2.4.1 A simple review of soil reinforcement by using natural and synthetic fibers(Hejazi et al,2011): The stabilisation of soil using polymers provide an improvement in the mechanical behavior of the soil composite. It comprises the oriented fibrous materials, it not only in optimizing fiber properties, fiber diameter, length, surface texture, etc., but also in reinforcing mechanism.

McGown et al. classified soil reinforcement into two major categories including ideally inextensible versus ideally extensible inclusions. The former includes high modulus metal strips that strengthens soil and inhibits both internal and boundary deformations.

Catastrophic failure and collapse of soil can occur if reinforcement breaks. Ideally extensible inclusions include relatively low modulus natural and/or synthetic fibers, plant roots; and geosynthetics. They provide some strengthening but more importantly they present greater extensibility (ductility); and smaller loss of post-peak strength compared to the neat soil.

Firstly, polyester filaments before staple fibers entered to the geotechnical engineering market under the traditional brand of ''Texsol''. This product was used in retaining walls and for slope protections. However, randomly distributed fiber-reinforced soils, known as short fiber soil composites, have recently attracted increasing attention in many geotechnical engineering applications, not only in scientific research environment, but also at executive real field. Synthetic staple fibers have been used in soil since the late 1980s, when the initial studies using polymeric fibers were conducted.

Finally, it can be concluded that the concept of reinforcing soil with natural fibers was originated in ancient times. However, short natural and synthetic fiber soil composites have recently attracted increasing attention in geotechnical engineering for the second time. Therefore, they are still a relatively new technique in geotechnical projects.

Significance of Polymers:

Polypropylene (PP) fiber is the most widely used inclusion in the laboratory testing of soil stabilization. Currently, PP fibers are used to enhance the soil strength properties, to reduce the shrinkage properties and to overcome chemical and biological degradation. PP fiber reinforcement also enhance the unconfined compressive strength (UCS) of the soil and reduced both volumetric shrinkage strains and swell pressures of the expansive clays.

From the experiments on field test sections in which a sandy soil was stabilized with PP fibers, it was concluded that the technique showed great potential for military airfield and road applications and that a 203-mm thick sand fiber layer was sufficient to support substantial amounts of military truck traffic. Field experiments also indicated that it was necessary to fix the surface using emulsion binder to prevent fiber pullout under traffic.

The PP-reinforced specimens showed a marked hardening behavior up to the end of the tests, at axial strains larger than 20%, whereas the non-reinforced specimens demonstrated an almost perfectly plastic behavior at large strain. This improvement suggests the potential application of fiber reinforcement in shallow foundations, embankments over soft soils, and other earthworks that may suffer excessive deformation.

The effects of PP fiber inclusions on the soil behavior could be visually observed during the triaxial testing and/or UCS testing shown in Fig. 2.26. Axial deformation of the unreinforced specimen resulted in the development of a failure plane, while PP reinforced specimens tended to bulge, indicating an increase in the ductility of fiber–soil mixture.

The efficacy of combination of fly ash and PP fibers in reducing swelling and shrinkage characteristics has been also reported. The available reports show that PP fiber reinforcements reduce the swelling potential of expansive clays.

Fig.2.26 Effects of PP fiber inclusions on the soil

Tang et al. investigated the micromechanical interaction behavior between soil particles and reinforcing PP fibers. They concluded that the interfacial shear resistance of fiber/soil depends primarily on the rearrangement resistance of soil particles, effective interface contact area, fiber surface roughness and soil composition.

Polyester Fibres: The study on soil fly ash mixture reinforced with 0.5% and 1% polyester fibers 20 mm in length was conducted in India by Kaniraj, which indicated the combined effect of fly ash and fiber on soil..

Maheshwari mixed polyester fibers of 12 mm in length with highly compressible clayey soil vary from 0% to 1%. The results indicated that reinforcement of highly compressible clayey soil with randomly distributed fibers caused an increase in the ultimate bearing capacity and decrease in settlement at the ultimate load. They concluded that the soil bearing capacity and the safe bearing pressure (SBP) both increase with increase in fiber content up to 0.50% and then it decreases with further inclusion of fibers.

Kumar et al. tested highly compressible clay in UCS test with 0%, 0.5%, 1.0%, 1.5% and 2.0% flat and crimped polyester fibers. Three lengths of 3 mm, 6 mm and 12 mm were chosen for flat fibers, while crimped fibers were cut to 3 mm long. The results indicate that as the fiber length and/or fiber content increases, the UCS value will improve. Crimping of fibers leads to increase of UCS slightly. These results are well comparable to those found by Tang et al.

Polyethylene (PE) fibers:The feasibility of reinforcing soil with polyethylene (PE) strips and/or fibers has been also investigated to a limited extent. It has been reported that the presence of a small fraction of high density PE (HDPE) fibers can increase the fracture energy of the soil. Nowadays, GEOFIBERS typically 1–2 in long discrete PP and/or PE fibrillated or tape strands, are mixed or blended into sand or clay soils. But, it is important to know that some researchers have applied the term ''Geofiber'' for PP fibers used in soil reinforcement.

Sobhan and Mashand demonstrated the importance of using toughness as a measure of performance. These studies showed that increases in tensile strength with added HDPE strips were not realized but large increases in toughness resulting from increased strain capacity was observed. With increasing toughness, much of the expected performance benefits due to fiber inclusion are in the post-peak load portion of the stress–strain behavior. Thus, as the fibers develop tension, an improved stress–strain response is the result. However, improvements in fatigue behavior were not noted.

Choudhary et al. reported that the addition of reclaimed HDPE strips to local sand increases the CBR value and secant modulus. The maximum improvement in CBR and secant modulus is obtained when the strip content is 4% with the aspect ratio of 3, approximately three times that of an unreinforced system. As well, base course thickness can be significantly reduced if HDPE strip reinforced sand is used as sub-grade material in pavement engineering.

As it can be seen environmental purposes are the main reason of using PE fibers and/or strips in geotechnical engineering to land-fill the waste PE-based materials.

Glass fibers: Consoli et al. indicated that inclusion of glass fibers in silty sand effectively improves peak strength. In another work, Consoli et al. examined the effect of PP, PET and glass fibers on the mechanical behavior of fiber-reinforced cemented soils. Their results showed that the inclusion of PP fiber significantly improved the brittle behavior of cemented soils, whereas the deviatoric stresses at failure slightly decreased. Unlike the case of PP fiber, the inclusion of PET and glass fibers slightly increased the deviatoric stresses at failure and slightly reduced the brittleness.

Maher and Ho studied the behavior of kaolinite–fiber (PP and glass fibers) composites, and found that the increase in the UCS was more pronounced in the glass fiber-reinforced specimens.

Conversely, Al-Refeai reported that PP fiber outperformed glass fiber[126]. Maher and Ho found that the inclusion of 1% glass fiber to 4% cemented sand resulted in an increase of 1.5 times in the UCS when compared to non-fiber-reinforced cemented sand.

Nowadays, fiberglass threads termed ‘‘roving’’ can be used to reinforce cohesionless soils.

The volume of fiberglass fibers is generally between 0.10% and 0.20% of the weight of the soil mixture by weight. Experimental studies have indicated that embedded roving increases soil cohesion between 100 and 300 kN/m2. It is interesting to know that the fiberglass roving is an effective promoting seed adhesion and root penetration.

Nylon fiber: Kumar and Tabor studied the strength behavior of nylon fiber reinforced silty clay with different degree of compaction. The study indicates that peak and residual strength of the samples for 93% compaction are significantly more than the samples compacted at the higher densities.

Gosavi et al. reported that by mixing nylon fibers and jute fibers, the CBR value of soil is enhanced by about 50% of that of unreinforced soil, whereas coconut fiber increases the value by as high as 96%. The optimum quantity of fiber to be mixed with soil is found to be 0.75% and any addition of fiber beyond this quantity does not have any significant increase in the CBR value.

Murray et al. conducted a laboratory test program to evaluate the properties of nylon carpet waste fiber reinforced sandy silt soil. As well, field trials have showed that shredded carpet waste fibers (to 70 mm long) can be blended into soil with conventional equipment. The availability of low cost fibers from carpet waste could lead to wider use of fiber reinforced soil and more cost-effective construction.

2.4.2 Stabilisation of soil using hybrid needle punched nonwoven geotextiles(Rawal et al,2010): Geotextiles are required to be used for uniform and rapid growth of vegetation in addition to fulfilment of reinforcement function. In general, geo-textiles can be made from synthetic and natural fibres but the former fibre type has been widely used in civil engineering applications primarily due to their superior mechanical properties and long-term durability. Nevertheless, natural fibre based geotextiles are environment friendly, less costly, easily available, and ecologically compatible as they are degraded within the soil. Several researchers have demonstrated the use of natural fibres including jute,flax, coir, wood and bamboo in various applications of geotextiles such as soil erosion control, vertical drains, road bases, bank protection and slope stabilization.

In applications where natural fibres are exposed to microbiological agents and solar radiation, the effectiveness of these fibres is expected to reduce. The effect of solar radiation is not limited to natural fibres but synthetic fibres such as polypropylene also has a poor resistance to ultra-violet radiation. Cellulosic regenerated fibre such as viscose rayon is highly suitable for soil stabilisation applications as it is biodegradable, capable of holding water and has uniform inherent properties. However, it has low strength and stiffness in comparison to the synthetic fibres namely, polyester and polypropylene. Thus, the overall objective of the present work is to compare and analyse the properties of hybrid needle punched nonwoven geotextiles produced from regenerated cellulosic and synthetic fibres (poly-ester and polypropylene) that can be potentially used for soil stabilisation applications. Furthermore, the changes in porosity of geotextiles are computed by determining the reduction in thickness at a range of pressures.

Sample preparation: Polypropylene/Viscose (PP/V) and Polyester/Viscose (PET/V) combinations were used in varying weight proportions (0%, 20%, 40%, 60%, 80% and 100%) to produce twenty-two sets of hybrid Needle punched nonwoven geotextiles. The constituent fibre properties are shown in Table 2.4 below.

Table 2.4

Constituent fibre properties used in the production of nonwoven geotextiles.

Type of fibre	Linear density (dtex)	Length (mm)	Diameter (μm)	Modulus (GPa)
Polyester	5.3	50	22	2
Polypropylene	6.6	60	30	0.7
Viscose	4.2	60	18	0.2

Twenty-two sets of needle punched nonwoven geotextiles were tested for various physical and mechanical properties.

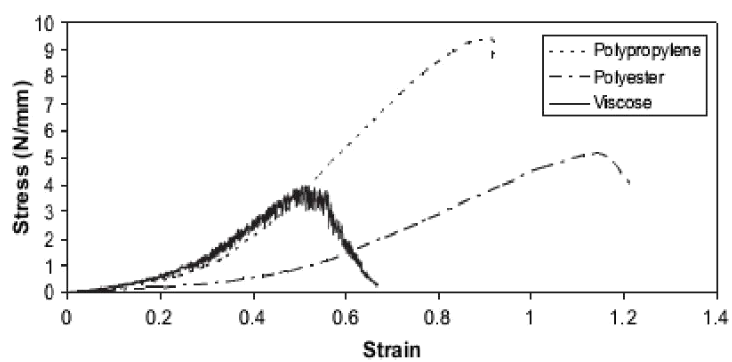


Fig. 2.27

A typical stress strain curve of 100% polyester, 100% polypropylene and 100% viscose needle-punched nonwoven geotextile is shown in above fig.2.27.

The fig.2.28 shows a typical stress strain curve revealing the extent of"stick slip"effect in 80/20 PET/V and 20/80 PET/V hybrid nonwoven geotextiles. It is observed that the extent of"stick slip"effect is dependent upon the viscose weight content in hybrid geotextiles.

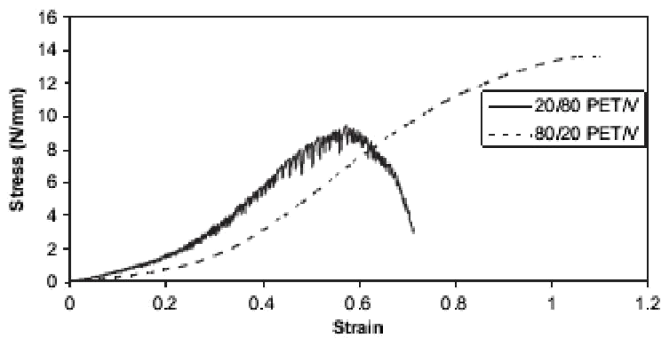


Fig.2.28

Fig. 2.29 illustrates the tensile strengths of hybrid nonwoven geo-textiles produced from different proportions of viscose and polyester fibres. The 100/0 PET/V geotextiles of 200 and 400 g/m2 can be replaced by corresponding hybrid geotextiles, 60/40 PET/V, as the latter is having highest tensile strength in both machine and cross-machine directions amongst the hybrid geotextiles.

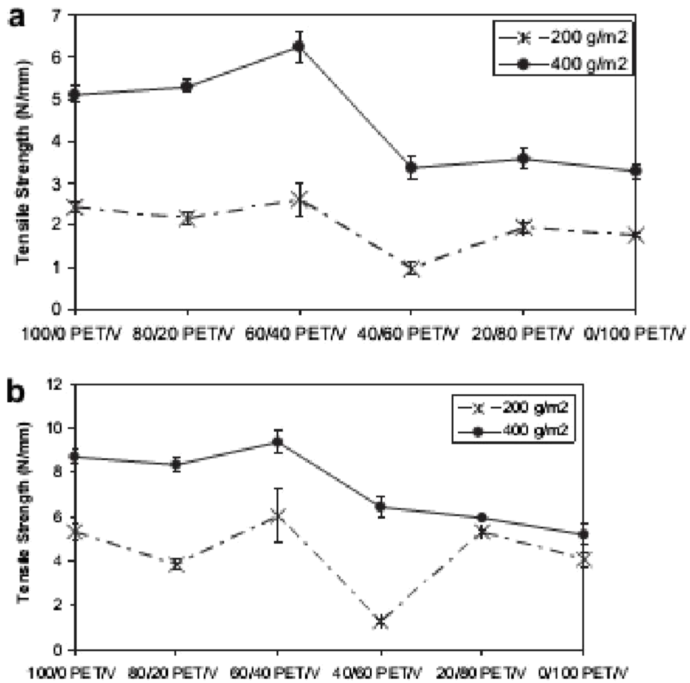


Fig.2.29 Tensile strengths of hybrid nonwoven geo-textiles

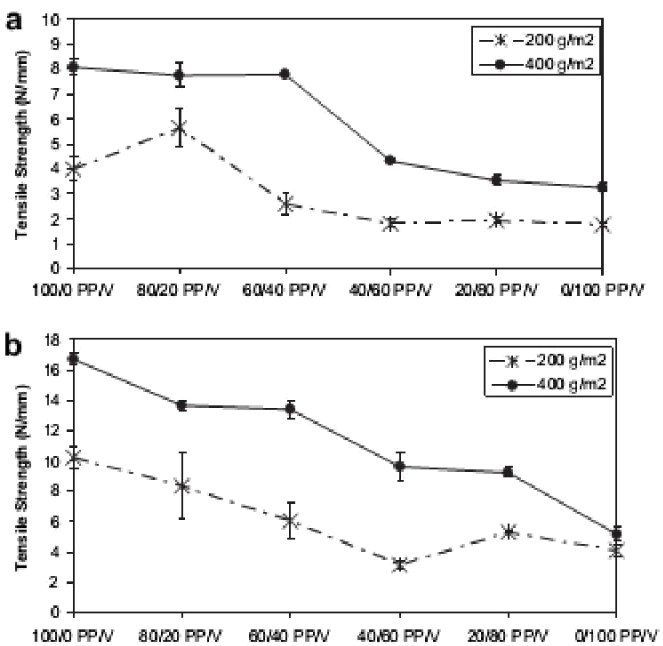


Fig.2.30 Tensile Strength in the machine direction

Similarly, the tensile strength in the machine direction of 100/0 PP/V geotextiles of 200 g/m2 is lower than corresponding hybrid geotextiles, 80/20PP/V, as shown in Fig.2.30.

Conclusions:

Hybrid needle punched nonwoven geotextiles were produced from polypropylene/viscose and polyester/viscosefibres in various weight proportions. The porosity reductions in needle punched nonwoven geotextiles were computed at defined normal pressures of 2, 20 and 200 kPa. For 200 g/m2, the minimum reduction in porosity was found in 20/80 PET/V and 20/80 PP/V. However, 20/80 PP/V and 60/40 PET/V of mass per unit area of 400 g/m2 under high normal pressures (200 kPa) have yielded minimum reduction in porosity. It has also been revealed that “stick slip” characteristics are present in stress strain curve of hybrid geotextiles having higher viscose weight content. For 200 and 400 g/m2, 60/40 PET/V is found to have highest tensile strength in both machine and cross-machine directions amongst the hybrid geotextiles. The tensile strength in the machine direction of 100/0 PP/V geotextiles of 200 g/m2 is lower than corresponding hybrid geotextiles with blend of 80/20 PP/V.

2.4.3 Soil Stabilization Using Nontraditional Additives:

Santoni et al. (2003) stabilized silty sand with several nontraditional stabilizers, including acids, enzymes, lignosulfonates, petroleum emulsions, polymers, and tree resins. UC tests were used as an index performance test for all samples. Samples were prepared in moist and dry test conditions. A total of six control samples, twelve nontraditional samples, and three traditional stabilizer samples were tested. The results indicated three polymers have the potential to increase the strength of silty sand in wet and dry conditions. For the traditional stabilizers, only cement provided significant strength improvement. Both the traditional and nontraditional stabilizers lost strength under wet conditions. The optimum additive dosage for the polymer emulsion ranged from 2.5% to 5% by weight of dry soil.

Tingle et al. (2003) looked at the stabilization of clay soils using several nontraditional additives including several polymer emulsions. The purpose of this study was to develop a compare effectiveness of several different liquid stabilizers. Low- and high-plasticity clays were used in this study. Samples were subjected to wet and dry test conditions and were tested using unconfined compression. The nontraditional stabilizers were compared to more traditional ones, such as cement and lime. The unconfined compression results showed the polymer emulsions to have variable improvements in the dry condition with minimal loss of unconfined compressive strength in the wet conditions with both soil types. The optimum amount of fluid for polymer emulsions was in the range of 2-5% by dry soil weight. Overall, the products used in this study proved to be promising for use in low-volume roads.

Newman and Tingle (2004) investigated the use of four polymer emulsions on silty sand specifically manufactured for their study. The level of 2.75% polymer emulsion by dry mass of the soil was chosen as a basis of comparison for all of the polymer emulsions. This was compared to Portland cement used at concentrations of 2.75%, 6%, and 9%. All samples were subjected to unconfined compression testing. The toughness was used as an index property to measure the effectiveness of the mix designs. The toughness is a measure of the energy absorbed by the system per unit volume to the yield point. Three separate cure periods were investigated: 24 hours, 7 days, and 28 days. Samples showed similar strength in the 24-hour time period compared to the 7-day cure time, with the Portland cement samples seeing the greatest increase in strength. Samples treated with polymer emulsions showed marked improvement in Unconfined Compressive Strength (UCS) and toughness after a 28-day curing period, with polymers showing significantly higher toughness values than the soil-cement mixtures.

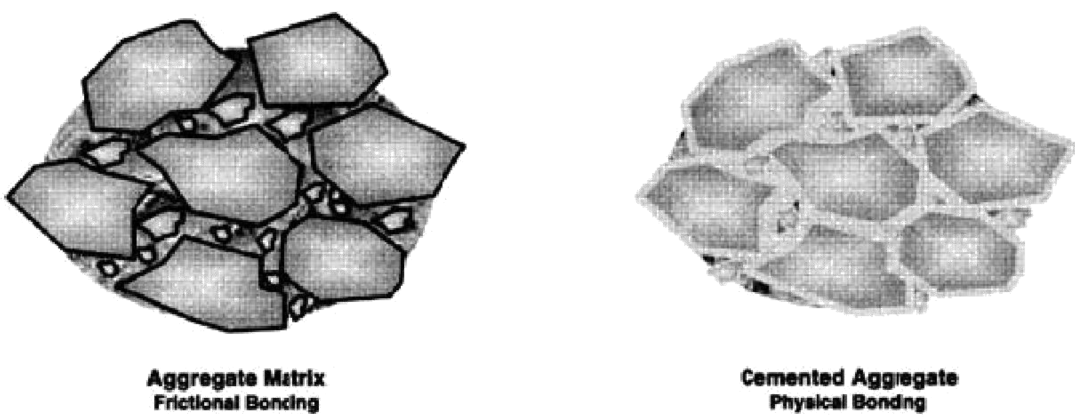


Fig.2.31 Cemented bond between Soil particles

Santoni et al. (2002) conducted a test program on silty sand (SM) with 28-day curing periods, and unconfined compressive strength as the engineering parameter of comparison. Three polymers were tested at application rates of 0.1% to 5% by dry weight. The polymer additives gained significant strength with time over their 28-day curing duration. Polymer amended specimens had an average of 57% increase in strength in the dry test condition, and 221% in the wet test condition relative to control. An optimum polymer addition rate to obtain maximum unconfined compressive stress was identified. Finally, it was concluded that nontraditional stabilizers gained strength over a shorter time duration than traditional stabilizers. A summary of test results is shown in figure 2.32.

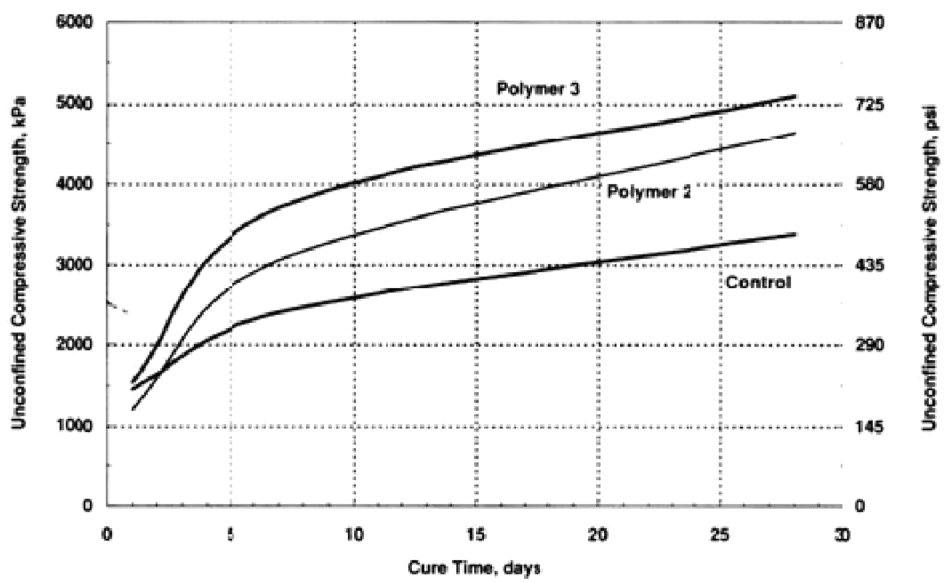


Fig 2.32. Increase in unconfined compression strength as a function of polymer addition rate and time.

However **Jones(2007)** noted that the criteria for determining enhanced engineering performance were: resistance to abrasion, resistance to erosion, resistance to leaching, increased shear strength, long term durability, and maintainability.

2.5. Closing Remarks

All types of soil stabilization shows an improvement in the soil which can be seen with the improvement of UCS, Toughness, Tensile strength, CBR values.

Chapter 3

EXPERIMENTAL INVESTIGATIONS

3.1 General

A laboratory test program was undertaken to evaluate the engineering properties of polymer amended soil viz. Unconfined compression Strength and CBR values. The objectives were to determine the influence of soil types, moisture content, and polymer addition rates on these engineering properties. In addition, comparisons were made between polymer modified soil and soil modified with traditional chemical stabilizer cement. Two types of soil-Sandy and Clayey Soil are used for the investigation.

The test program is outlined in this chapter. First, the materials used in the program, including the test soils and the soil amendments are described. The tests conducted included: unconfined Compressive strength, CBR tests.

3.2 Experimental Program

Testing was performed in accordance with all applicable Indian Standard Codes IS: 2720 (Part 16) 1979.The index properties tests are conducted first, then the soil optimum moisture content and dry density tests are conducted and at OMC, maximum dry density, the tests are performed. Soil index properties such as maximum dry unit weight, optimum moisture content, and specific gravity were used to classify soils. These tests were performed in accordance with their respective standards.

3.3 Material Used

3.3.1 Sandy Soil:

Table 3.1: Engineering properties of sandy soil.

Property	Sample
Specific gravity	2.75
Grain size:	
(4.75-20) mm (%)	2.5
(< 4.75mm) (%)	97.5
Max. void ratio (emax)	0.8

Min. void ratio (emin)	0.42
Void ratio, (%)	1.024
Optimum moisture content (%)	15
Maximum dry unit weight (g/cm^3)	1.86
Soil classification (USCS)	S

3.3.2 Clayey Soil:

Index property:

The result of index properties such as liquid limit, plastic limit, PI value are

presented in Table below:

Table 3.2 Index properties

Description of Index properties	Experimental Value
Liquid limit	30%
Plastic limit	18.50%
Plastic Index	11.50%
Shrinkage limit	14.65%

Particle size distribution - The grain size distribution of this soil sample has been shown in Table 3.3 below:

Table 3.3 Grain size distribution of soil

IS sieve no	Wt. retained in gm	% wt. retained in gm	% wt. passing
4.75 mm	18.84	1.884	98.51
2.36 mm	17.2	1.72	95.25
1.18 mm	15.56	1.556	93.21
425 µm	12.51	1.251	92.13
300 µm	3.12	0.312	91.15
150 µm	22.3	2.23	89.91
75 µm	42.45	4.245	86.43

Based on the above properties the IS Soil Classification for the soil sample under test is ‘CL’

Modified Proctor Compaction Test

The result of modified proctor compaction test are represented in figure 3.1

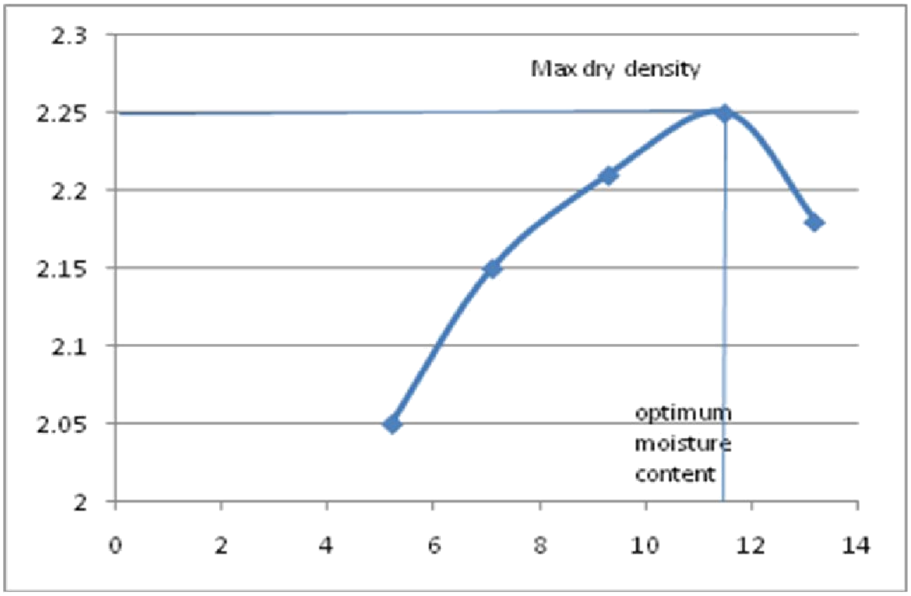


Fig.3.1 result of modified proctor compaction test

From the figure 3.1 it is clear that,

MDD = 2.25 g/cc

OMC = 11.5 %

3.3.3 Cement

The cement used for the study is Portland cement 43 grade and the properties of which are given in Table 3.4

Table 3.4 Properties of cement

PROPERTIES OF 43 GRADE OPC							
Fineness (sq.m/kg) min	**Soundness by**	**Setting time**		**Compressive strength**			
	Lechatlier (mm) max	Initial (mts) min	Final (mts)Max	1 day Min Mpa	3 day Min Mpa	7 day Min Mpa	28 day Min Mpa
225	10	30	600 specified	NS	23	33	43

3.3.4 Polymer

A commercial product of Acrylic Polymer was used, which is an emulsion synthetic elastic chemical substance that increases the bound with the substrate as additive in optimum moisture as well as the cohesion and the strength. Some important properties were given in Table 3.5 below:

Table 3.5: Important physicochemical properties of as-received emulsion.

Name	Acrylic-Copolymer watered solution
Physical state	Liquid-white colour
Solvability in water	Solution
Boiling point	100°
Water Absorption	1% max
Non-self-burning	Nonexplosive
Applicable temperature	Not less than 10°
Density (g/cm^3)	1.11 (20°)
Toxicity	Non Toxic

3.4 Tests for Investigation:

3.4.1 California Bearing Ratio Testing

The CBR is the only test which can figure out the strength of a subgrade. By this test we can compare the strength of different subgrade materials .The CBR test is done in a standard manner by which one can find out or design the strength or thickness of subgrade layer. CBR value is inversely proportional to thickness of the pavement layer. If the subgrade is stronger, the higher is the CBR value, so lesser thickness is required and vice-versa.

The strength of a soil or subgrade can be determined by using a test known as California Bearing Ratio Test which was developed in California in the year 1930's and it is way to determine the standard soil properties such as density. It is graph showing the values for aspect of design of road pavement. Mostly all the design charts are based on the value of CBR for the subgrade.

The CBR test was first introduced or developed by O.J. Porter at California Highway Department in 1920. It is otherwise called as load-deformation test which is conducted in the laboratory or in the fields and these results are generally used to find the thickness of pavement layers, base course and other layers of a given traffic loading by the use of empirical design chart. First it was adopted by the US Army Corps of Engineers (USACE) for the design of flexible airfield pavements. Initially it practiced for the design of surfaced and un-surfaced airfields which is still based upon CBR today. The CBR determination is performed in the laboratory mainly on recompacted soil or in the field and the field CBR is normally used by the military for contingency roads and design of airfields.

The CBR determines the thickness of different elements constituting the pavement.The CBR test is the ratio of force per unit area required to penetrate soil mass by a circular plunger of 50mm at the rate of 1.25mm/min. Observations are carried out between the load resistances (penetration) vs. plunger penetration.. The California bearing ratio, CBR is expressed as the ratio of the load resistance (test load) of a given soil sample to the standard load at 2.5mm or 5mm penetration, expressed in percentage .

CBR = (Test load/Standard load)×100

The standard load for 2.5mm and 5mm penetrations are 1370 kg and 2055 kg respectively. The CBR test is carried out on a small scale penetration of dial reading with probing ring divisions. Initially experiments were conducted to find out different properties of soil such as index properties, grain size distribution etc. Later on heavy compaction tests were conducted to find out the optimum moisture content & corresponding maximum dry density. Then CBR tests were made at OMC and analysis made to investigate the variation of CBR with respect to different days of soaking, i.e. from unsoaked (day 0) to soaked (day 4). Using the moisture content and corresponding dry density the amount of soil used for CBR was calculated. The sample was tested using the CBR instruments and each soil sample was soaked for 1 day, 2 day, 3 day, 4 day, and corresponding CBR values was found out.

Fig.3.2 CBR Testing Machine, Lab

Sample Preparation

All CBR samples are prepared by first measuring the necessary amount of soil to fill a standard CBR mold into a 4 kg plastic bag. The amount of water required to bring the soil to optimum moisture content is added and blended until homogenously distributed. Chemical additives are used they are added at the desired content to the moist soil mixture. The chemical added is then blended with the soil-water mixture.

Unsoaked

The unsoaked condition is the fastest conditioning method used. Unsoaked samples are soil samples that are subjected to CBR testing immediately after compaction.

Soaked.

The soaked condition is CBR samples that have been submerged in water for 96 hours. Swell measurements are recorded before and after submerging the sample in water. The soaked condition typically causes CBR values to decrease compared to unsoaked samples.

3.4.2 Unconfined Compression Testing

Unconfined Compression (UC) testing was used for fine-grained material treated with polymer emulsion. The Soil test loading frame used in CBR testing was modified by exchanging the penetration piston with a 4 inch plate. Figure provides an illustration of the Unconfined Compression apparatus.

Samples were compacted in a 4 inch by 8 inch mold using modified proctor compaction. After compaction samples were extruded from the mold and wrapped in arubber membrane to prevent excess air from curing the samples. A strain rate of 0.6 inches per minute was used with measurements taken every 0.01 inches to a total strain of 15% (up to 1.23 inches). Sandy soils with a different rate of cement mixing were used and various percentages of waterborne polymer were added to soils to investigate the compressive strength of stabilized samples. The soils were dried before using in the mixtures. First, the required amounts of polymer as a percentage of dry weight of sample and cement were blended and then added to dry soils. The amount of aqueous polymer was chosen as 2, 3, and 4% by total weight of dry sample and the amount of cement was chosen as 20, 30, 40% by weight of dry sample, respectively. The mixing sample was placed into the mould. After 24 hours later, the specimens were taken out of the moulds and specimens were stored in the curing room at the temperature ranging from 21 to 25 centigrade and then tested at 7 days.

The polymer mixture was developed in to dough using proper Kneading by hand. The uniformly mixed dough was subsequently placed into a steel mold measuring 150 mm in height and 300 mm in diameter

Following Table 3.6 presents the consistency classification for fine-grained soils (Terzaghi et al.,1996).

Table 3.6 consistency classification for fine-grained soils

Classification	Description	Unconfined Compressive Strength, q_u (lb/in^2)
Very soft	Thumb penetrates easily; extrudes between fingers when squeezed	<3.5
Soft	Thumb will penetrate soil about 1 inch; molds with light finger pressure	3.7-6.9
Medium	Thumb will penetrate about 0.25 inches with moderate effort; molds with strong finger pressure	6.9-13.9
Stiff	Thumb indents easily, and will penetrate 0.5 inches with great effort	13.9-27.8
Hard	Thumb will not indent soil, but thumbnail readily indents it	27.8-55.5
Very hard	Thumbnail will not indent soil or will indent it only with difficulty	>55.5

Fig.3.3 UNCONFINED COMPRESSION TESTING MACHINE

(From Google images)

Unconfined compressive strength testing was performed on all extracted specimens with aconstant stress rate by manually controlled test machine (Figure 3.3). A data acquisition system was used to record the applied load. Each specimen was loaded until peak load was obtained.

Chapter 4

RESULTS AND DISCUSSIONS

4.1 Unconfined Compression Test Results

The results of 7 days curing on unconfined compression strength results were shown in Figures 4.1and 4.2.The unconfined compression strength of stabilized samples increases with curing time. Both specimens containing polymer content of 2–4% by wt.% and cement content of 20–40 wt.% were cured in air during 7 days. So, by increasing the polymer contents, cross-linking between polymer network increased and the strength of soil increased.It is clear from Figures 4.1 that compressive strength of the stabilized soils was increased while increasing the curing time in air curing conditions.

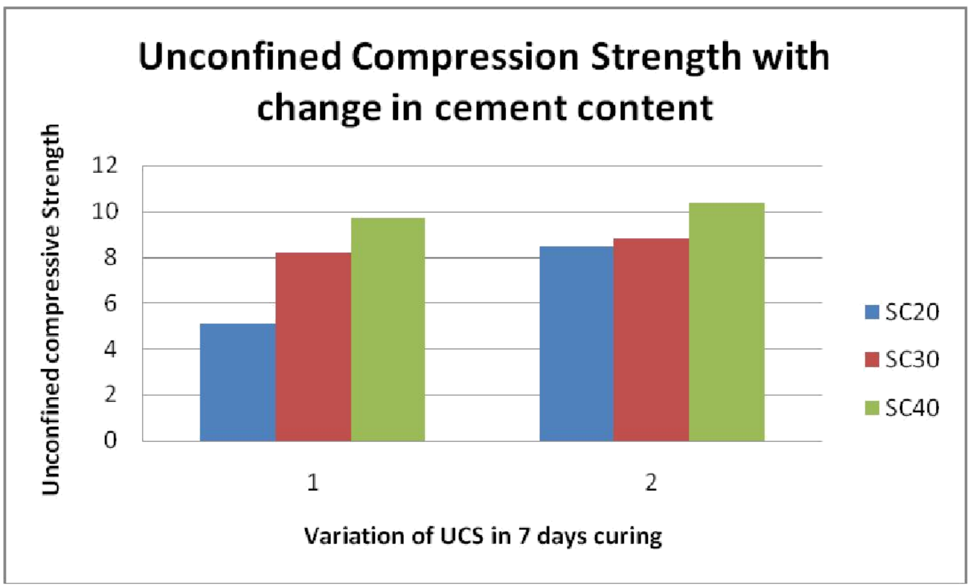


Fig.4.1

Mix ID's

SC20=Soil+20% cement

SC30=Soil+30% cement

SC40=Soil+ 40% cement

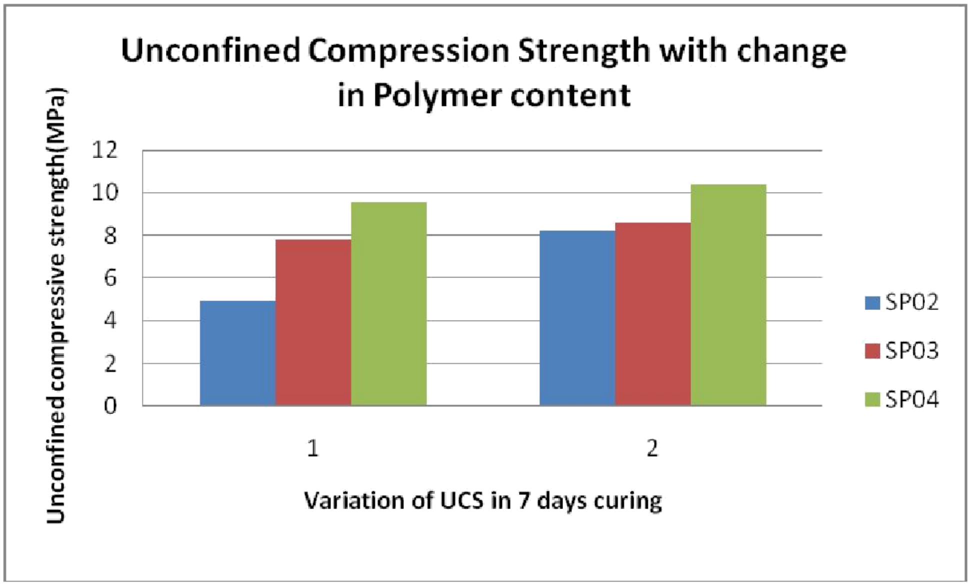


Fig.4.2

Mix Id's

SP02=Soil+2% Polymer

SP03=Soil+3% Polymer

SP04=Soil+4% Polymer

Table 4.1

	Variation in cement content			Variation in Polymer Content		
Unconfined Compression Strength(Mpa)	20%	30%	40%	2%	3%	4%
	5.1	8.2	9.7	4.9	7.8	9.56
	8.5	8.8	10.4	8.2	8.6	10.35

4.2 CBR Test Results

The result of CBR test of soil sample taken at 20% cement content and 2% polymer content under different times of soaking are presented in

1) Figure – 4.3, Un-Soaked (0 day)

2) Figure – 4.4, Soaked (1 day)

3) Figure – 4.4, Soaked (2 days)

4) Figure – 4.5, Soaked (3 days)

5) Figure – 4.6, Soaked (4 days)

Figure – 4.3, Unsoaked (0 day)

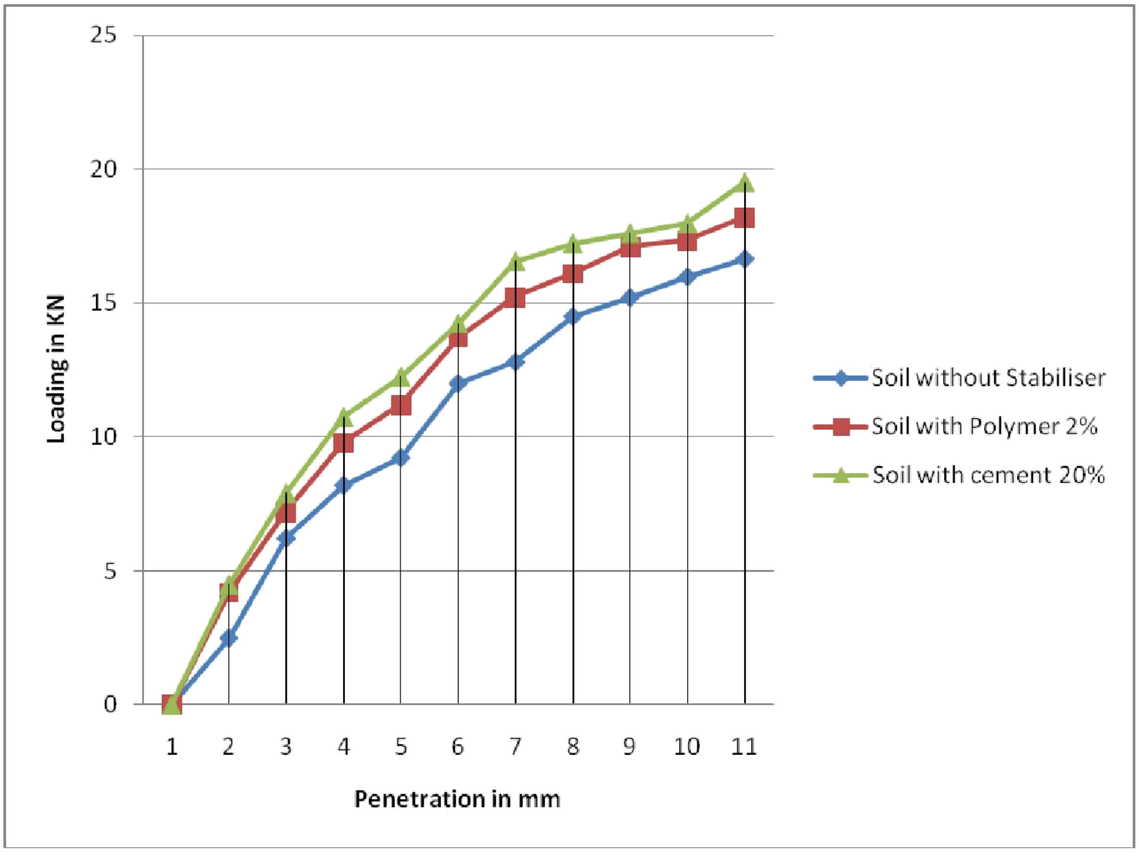


Table.4.2

CBR Values:

Soil with different Stabilisers	Penetration at	
	2.5 mm	5 mm
Soil without Stabilisers	45.40	44.91
Soil with Polymer 3%	52.55	54.55
Soil with cement 20%	57.66	59.56

Figure – 4.4, Soaked (1 day)

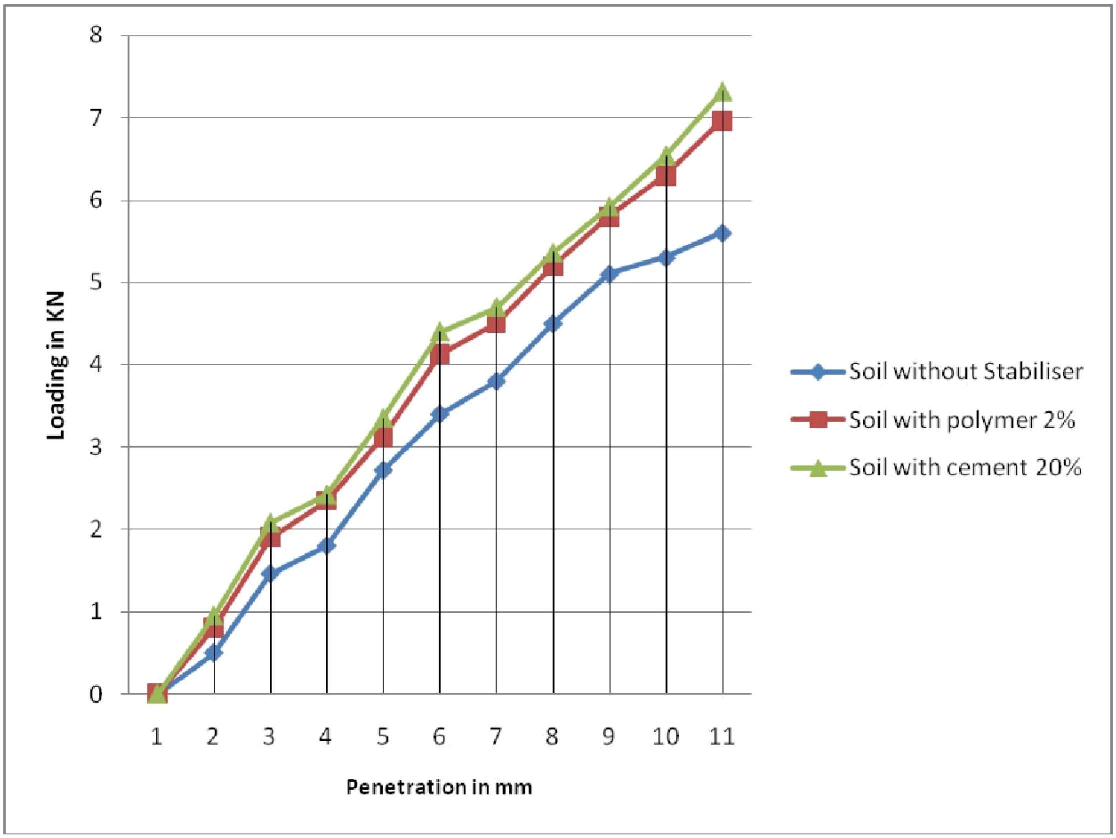
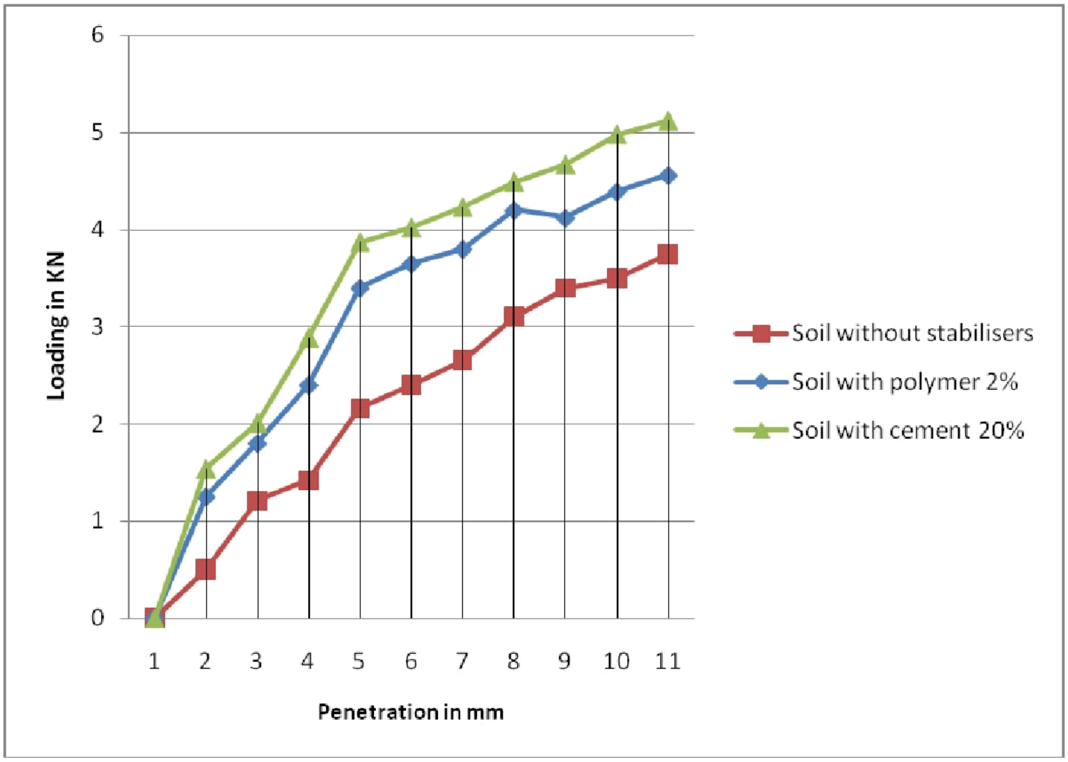


Table 4.3

CBR Values:

Soil with different Stabilisers			Penetration at	
			2.5 mm	5 mm
Soil without Stabilisers			10.66	13.24
Soil with Polymer 3%			13.87	15.18
Soil with cement 20%			15.18	16.30

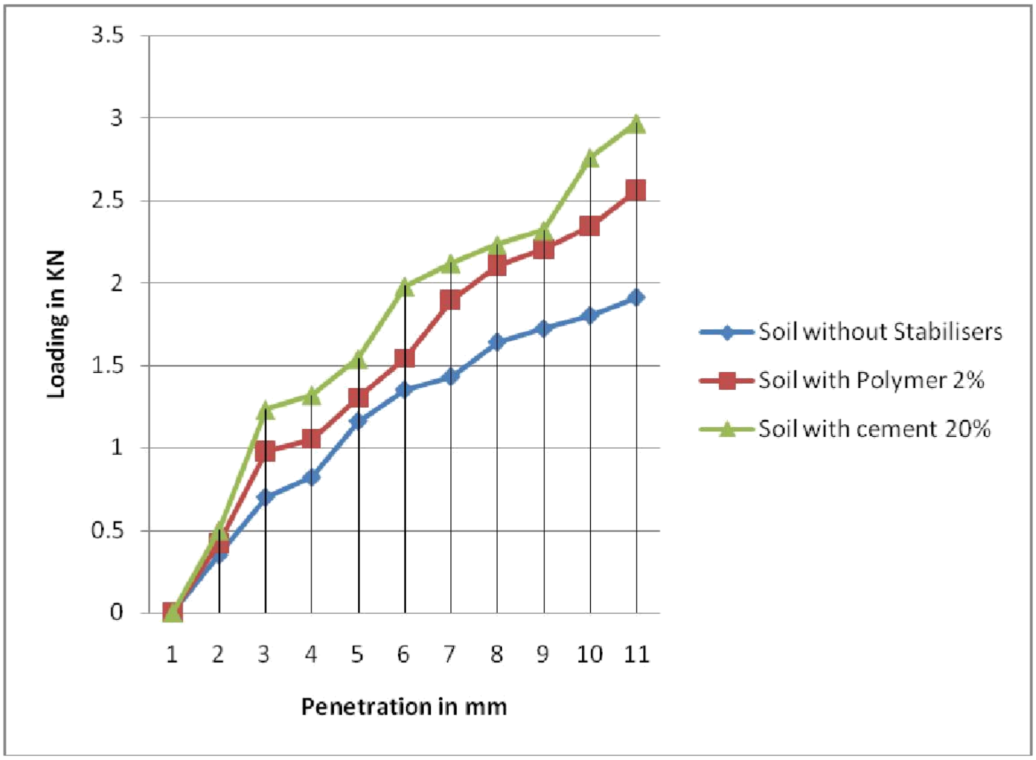
Figure – 4.5, Soaked (2 days)



CBR Values: Table 4.4

Soil with different Stabilisers			Penetration at	
			2.5 mm	5 mm
Soil without Stabilisers			8.83	10.51
Soil with Polymer 3%			13.14	16.55
Soil with cement 20%			14.67	18.83

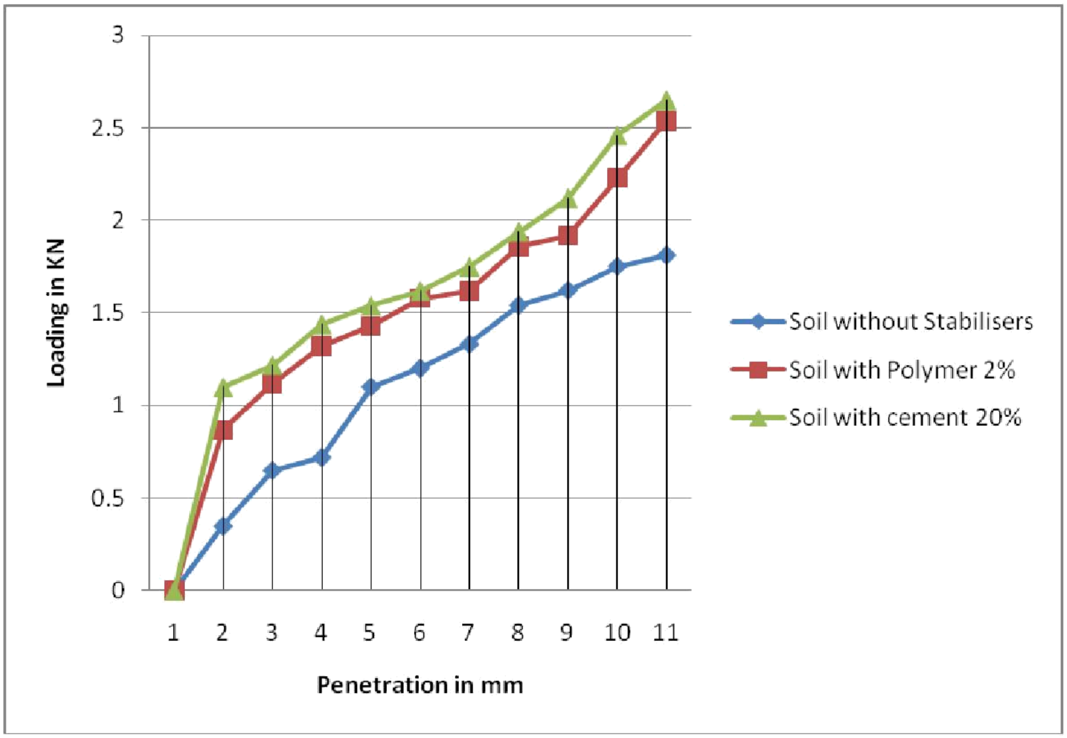
Figure – 4.6, Soaked (3 days)



CBR Values: Table 4.5

Soil with different Stabilisers			Penetration at	
			2.5 mm	5 mm
Soil without Stabilisers			5.11	5.64
Soil with Polymer 3%			7.15	6.33
Soil with cement 20%			8.98	7.49

Figure – 4.7, Soaked (4 days)



CBR Values: Table 4.6

Soil with different Stabilisers			Penetration at	
			2.5 mm	5 mm
Soil without Stabilisers			4.74	5.35
Soil with Polymer 3%			5.25	6.03
Soil with cement 20%			5.77	6.28

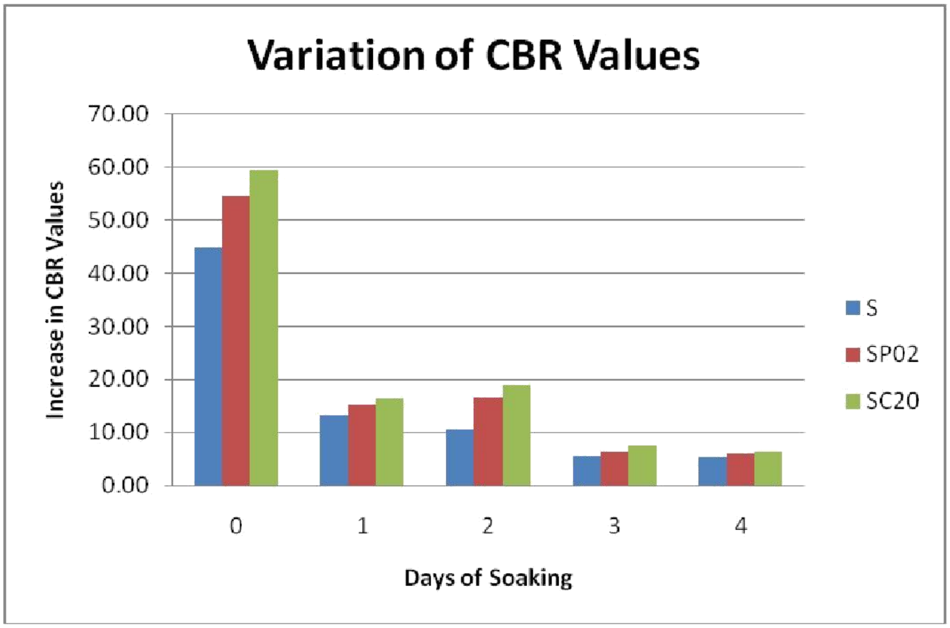


Fig.4.8 Variation in CBR Values with days of Soaking

Table 4.7

Increase in CBR Values

Days of Soaking	0	1	2	3	4
S	44.91	13.24	10.51	5.64	5.35
SP02	54.55	15.18	16.55	6.33	6.03
SC20	59.56	16.30	18.83	7.49	6.28

Mix Id's:

S=Soil without Stabilisation

SP02=Soil with polymer 2%

SC20=Soil with cement 20%

It has been observed from the Fig.4.8 that the variation of CBR values is more pronounced in starting days of soaking but at the fourth day, CBR values is not increased much as expected.

From table 4.6,as the CBR values at 5mm are more compared to 2.5mm, tests are repeated but values of CBR at 5mm came more again, we will follow the maximum values of 5mm for the design of Flexible pavement.

At 4 days Soaking, variation in CBR values are given as

Days of Soaking	4
S	5.35
SP02	6.03
SC20	6.28

Design of Flexible Pavement (IRC: 37-2012)

Assume Traffic Volume=100 msa

i.e,N=100 msa

From IRC: 37-2012, At CBR=5.35, Thickness of pavement=714

mm At CBR=6.03, Thickness of pavement=684 mm

At CBR=6.28, Thickness of pavement=675 mm

Decrease in Pavement thickness is as:

With polymer 2%=30mm

With cement 20%=39mm

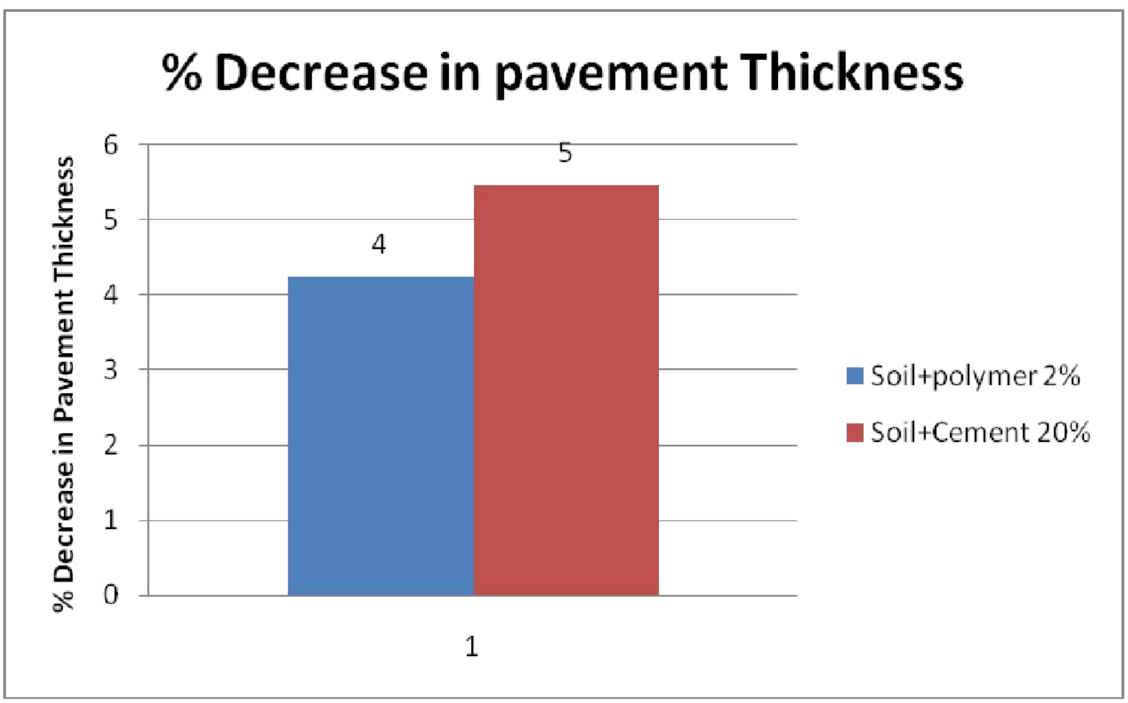


Fig 4.9 % Decrease in Pavement thickness

Analysis for cost of Pavement per km road length:

Polymer and cement addition to the clayey soil has some effect in stabilization of soil and improvement in the thickness required i.e., less thickness is required for pavement design. Here cost analysis is done to see the effect of addition of polymer and cement to the clayey soil by taking 1km stretch of road.

S.No	Particulars	Amount. (Lacs)
1	Compacting original ground supporting Sub-Grade loosening of the ground upto a level of 300mm below the sub-grade level watered graded & compacted in layers for Sub-Grade construction as per Technical specification clause 303.5.2 1000x0.286x3.60 = 1029.6cum Total = 1029.6 cum @ 29	**0.29**
2	Construction of Granular Sub Base by providing well graded material spreading in uniform layers with moter grader on prepared surface by mix in place method with rotavator at OMC and compacting with smooth wheel ruler to achieve desired density complete as per technical specification Clause 401. For grading 3rd material. 1000x3.60x0.286 = 1029.6cum. Add for passing zones = 3 x 22.5x 2.40 x 0.286 = 46.33cum Total :- 1075.93 Cum @ 709/cum	**7.63**
3.	**WBM Grade-III** Providing and laying, spreading compacting stone aggregates of specified size to water bound macadam specification including spreading in uniform thickness hand packing rolling with three wheel 80-100 Kn static roller in stages to proper grade and camber applying and brooming stone screening binding material to fill up the interstices of course aggregates watering and compacting to required density grading II as per technical specification clause 405. By mechanical means. 1000x3.30x0.25 = 825. Cum Add for passing zones = 3 x 2.40 x 0.25 x 22.50 = 40.5 cum Total :- 865.5Cum @ 1198/cum	**10.36**
4	**BM**	

.	Providing and laying bituminous macadam with hot mix plant using crushed aggregates of grading as per Table 500.4 premixed with bituminous binder, transported to site upto a lead of 1000 m laid over a previously prepared surface with paver finisher to the required grade, level and alignment and rolled to achieve the desired compaction as per Technical Specification Clause 504. a. 1000 X 3 X 0.128 = 384Cum Add for passing zone = 3 x 2.70x 22.50 x 0.128 = 23.32cum Total:- 407.32 @ 6932/Cum	**28.23**
5	Providing and applying primer coat with bitumen emulsion (SS-1) on prepared surface of granular base including cleaning of road surface and spraying primer at the rate of 0.70 – 1.0kg/sqm. Using mechanical means as per technical specifications clause 502 (low porosity) 1000x 3 = 3000 Sqm Add for passing zone = 3 x 22.50 x 2.70 = 182.25 sqm = 3182.25 @ 43/sqm	**1.36**
6	Providing and applying tack coat with Bitumen emulsion (RS-1) using emulsion distributed at the rate of 0.20-0.30 Kg/sqm on the prepared/primer coat as per technical specification clause 503. Quantity as per item No. 5 =3182.25 sqm @ 16/sqm	**0.51**
7 .	20mm thick open graded premix carpet using bituminous (penetration grade/ modified bitumen) binder and rolling with smooth wheel 80-100 KN static roller capacity, finished to required level and grade to be followed by seal coat of Type B as per Technical Specification Clause 508 by mechanical means (Bitumen S-90) Quantity as per item No. 5 = 3182.25 sqm @141/sqm	**4.49**
8	Providing and laying seal coat for sealing the voids in a bituminous surface laid to the specified levels, grade and Cress fall type b as per technical specification clause 510 Quantity as per item No. 6= 3182.25 sqm @ 54/ sqm	**1.72**
9	Berm Filling with approved material obtained from borrow pits with a lift upto 1.50 m transporting to site, spreading grading to required slope and compacting within a lead of 1000m as per technical specification 301.5 2x 1000 x 0.90+1.20 x 0.286 = 1800 Cum @ 258/Cum	**4.64**

Total cost per km=59.23 lac

Cost Assessment of polymer(2%) Stabilised Soil

S.No	Particulars	Amount. (Lacs)
1	Compacting original ground supporting Sub-Grade loosening of the ground upto a level of 300mm below the sub-grade level watered graded & compacted in layers for Sub-Grade construction as per Technical specification clause 303.5.2 1000x0.259x3.60 = 932.4cum Total = 932.4 cum @ 29	0.27
2	Construction of Granular Sub Base by providing well graded material spreading in uniform layers with moter grader on prepared surface by mix in place method with rotavator at OMC and compacting with smooth wheel ruler to achieve desired density complete as per technical specification Clause 401. For grading 3rd material. 1000x3.60x0.259 = 932.4cum. Add for passing zones = 3 x 22.5x 2.40 x 0.259 = 46.33cum Total :- 978.73 Cum @ 709/cum	**0.69**
3.	**WBM Grade-III** Providing and laying, spreading compacting stone aggregates of specified size to water bound macadam specification including spreading in uniform thickness hand packing rolling with three wheel 80-100 Kn static roller in stages to proper grade and camber applying and brooming stone screening binding material to fill up the interstices of course aggregates watering and compacting to required density grading II as per technical specification clause 405. By mechanical means. 1000x3.30x0.25 = 825. Cum Add for passing zones = 3 x 2.40 x 0.25 x 22.50 = 40.5 cum Total :- 865.5Cum @ 1198/cum	**10.36**
4.	**BM** Providing and laying bituminous macadam with hot mix plant using crushed aggregates of grading as per Table 500.4 premixed with bituminous binder, transported to site upto a lead of 1000 m laid over a previously prepared surface with paver finisher to the required grade,	**27.57**

	level and alignment and rolled to achieve the desired compaction as per Technical Specification Clause 504. a. 1000 X 3 X 0.125 = 375Cum Add for passing zone = 3 x 2.70x 22.50 x 0.125 = 22.78cum Total:- 397.78 @ 6932/Cum	
5	Providing and applying primer coat with bitumen emulsion (SS-1) on prepared surface of granular base including cleaning of road surface and spraying primer at the rate of 0.70 – 1.0kg/sqm. Using mechanical means as per technical specifications clause 502 (low porosity) 1000x 3 = 3000 Sqm Add for passing zone = 3 x 22.50 x 2.70 = 182.25 sqm = 3182.25 @ 43/sqm	**1.36**
6	Providing and applying tack coat with Bitumen emulsion (RS-1) using emulsion distributed at the rate of 0.20-0.30 Kg/sqm on the prepared/primer coat as per technical specification clause 503. Quantity as per item No. 5 =3182.25 sqm @ 16/sqm	**0.51**
7.	20mm thick open graded premix carpet using bituminous (penetration grade/ modified bitumen) binder and rolling with smooth wheel 80-100 KN static roller capacity, finished to required level and grade to be followed by seal coat of Type B as per Technical Specification Clause 508 by mechanical means (Bitumen S-90) Quantity as per item No. 5 = 3182.25 sqm @141/sqm	**4.49**
8	Providing and laying seal coat for sealing the voids in a bituminous surface laid to the specified levels, grade and Cress fall type b as per technical specification clause 510 Quantity as per item No. 6= 3182.25 sqm @ 54/ sqm	**1.72**
9	Berm Filling with approved material obtained from borrow pits with a lift upto 1.50 m transporting to site, spreading grading to required slope and compacting within a lead of 1000m as per technical specification 301.5 2x 1000 x 0.90+1.20 x 0.259 = 1800.31 Cum @ 258/Cum	**4.64**

Total cost per km=51.61 lac

Cost Assessment of cement(20%) Stabilised Soil

S.No	Particulars	Amount. (Lacs)
1	Compacting original ground supporting Sub-Grade loosening of the ground upto a level of 300mm below the sub-grade level watered graded & compacted in layers for Sub-Grade construction as per Technical specification clause 303.5.2 1000x0.252x3.60 = 907.2cum Total = 907.2 cum @ 29	**0.26**
2	Construction of Granular Sub Base by providing well graded material spreading in uniform layers with moter grader on prepared surface by mix in place method with rotavator at OMC and compacting with smooth wheel ruler to achieve desired density complete as per technical specification Clause 401. For grading 3rd material. 1000x3.60x0.252 = 907.2cum. Add for passing zones = 3 x 22.5x 2.40 x 0.252 = 40.82cum Total :- 948.02 Cum @ 709/cum	**0.67**
3.	**WBM Grade-III** Providing and laying, spreading compacting stone aggregates of specified size to water bound macadam specification including spreading in uniform thickness hand packing rolling with three wheel 80-100 Kn static roller in stages to proper grade and camber applying and brooming stone screening binding material to fill up the interstices of course aggregates watering and compacting to required density grading II as per technical specification clause 405. By mechanical means. 1000x3.30x0.25 = 825. Cum Add for passing zones = 3 x 2.40 x 0.25 x 22.50 = 40.5 cum Total :- 865.5Cum @ 1198/cum	**10.36**
4.	**BM** Providing and laying bituminous macadam with hot mix plant using crushed aggregates of grading as per Table 500.4 premixed with bituminous binder, transported to site upto a lead of 1000 m laid over a previously prepared surface with paver finisher to the required grade,	**27.13**

	level and alignment and rolled to achieve the desired compaction as per Technical Specification Clause 504. a. 1000 X 3 X 0.123 = 369 Cum Add for passing zone = 3 x 2.70x 22.50 x 0.123 = 22.41cum Total:- 391.41 @ 6932/Cum	
5	Providing and applying primer coat with bitumen emulsion (SS-1) on prepared surface of granular base including cleaning of road surface and spraying primer at the rate of 0.70 – 1.0kg/sqm. Using mechanical means as per technical specifications clause 502 (low porosity) 1000x 3 = 3000 Sqm Add for passing zone = 3 x 22.50 x 2.70 = 182.25 sqm = 3182.25 @ 43/sqm	**1.36**
6	Providing and applying tack coat with Bitumen emulsion (RS-1) using emulsion distributed at the rate of 0.20-0.30 Kg/sqm on the prepared/primer coat as per technical specification clause 503. Quantity as per item No. 5 =3182.25 sqm @ 16/sqm	**0.51**
7.	20mm thick open graded premix carpet using bituminous (penetration grade/ modified bitumen) binder and rolling with smooth wheel 80-100 KN static roller capacity, finished to required level and grade to be followed by seal coat of Type B as per Technical Specification Clause 508 by mechanical means (Bitumen S-90) Quantity as per item No. 5 = 3182.25 sqm @141/sqm	**4.49**
8	Providing and laying seal coat for sealing the voids in a bituminous surface laid to the specified levels, grade and Cress fall type b as per technical specification clause 510 Quantity as per item No. 6= 3182.25 sqm @ 54/ sqm	**1.72**
9	Berm Filling with approved material obtained from borrow pits with a lift upto 1.50 m transporting to site, spreading grading to required slope and compacting within a lead of 1000m as per technical specification 301.5 2x 1000 x 0.90+1.20 x 0.252 = 1800.3 Cum @ 258/Cum	**4.64**

Total cost per km=51.14 lac

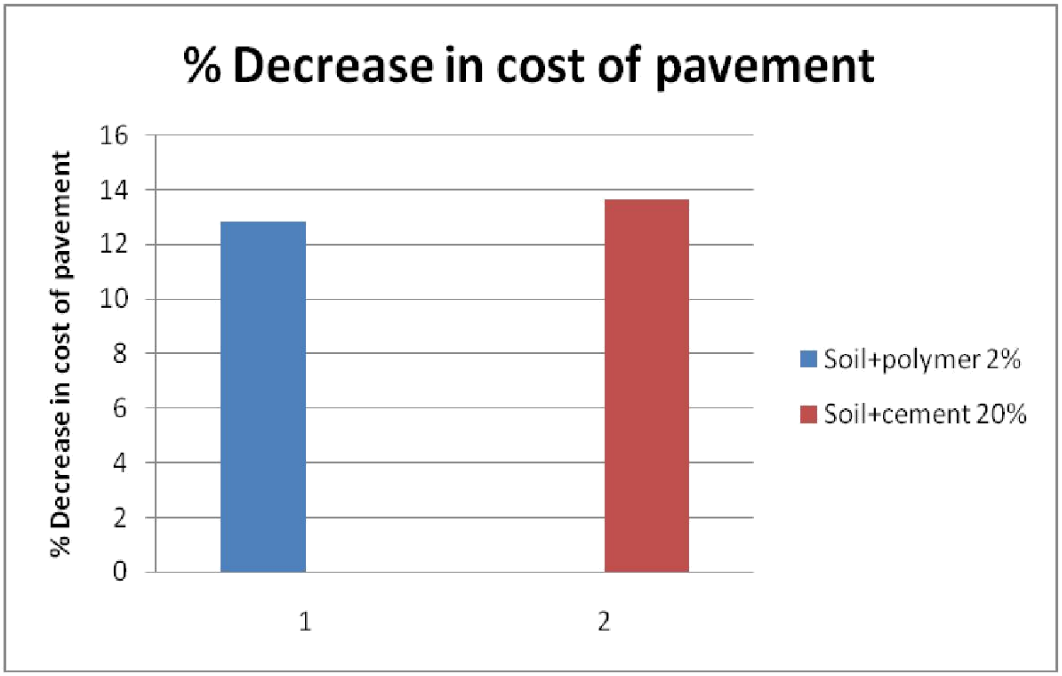


Fig.4.10 % Decrease in cost of Pavement

SANDY SOIL

EFFECT OF CEMENT STABILISATION

The unconfined compression strength of stabilized samples increases with curing time. Specimen containing cement content of 20–40 wt.% were cured in air during 7 days. It is clear from Figures 4.1 that compressive strength of the stabilized soils was increased while increasing the curing time in air curing conditions

EFFECT OF POLYMER STABILISATION

Specimens containing polymer content of 2–4% by wt and were cured in air during 7 days. So, by increasing the polymer contents, cross-linking between polymer network increased and the strength of soil increased.

CLAYEY SOIL

EFFECT OF CEMENT STABILISATION

From Fig.4.8,CBR values shows an increasing trend with the increase in cement content from Unsoaked day to day 4 of soaking. The variation is seen much in the first two days of soaking but at the 3rd and 4th day, the increase is not much.

EFFECT OF POLYMER STABILISATION

From Fig.4.8, it has been observed that both the cement and polymer content increases the values of CBR. The polymer has also good impact on first two days of soaking then the observation is same as that of cement.

The polymer and cement incorporation into the soil has the same effect but cement impact is more comparative to polymer.

As seen in Fig 4.9, there is decrease in pavement thickness to 4% and 5% after addition of polymer and cement into the clayey soil.

In fig.4.10, the addition of polymer and cement shows an decrease in cost of pavement to nearly 13% and 14% respectively.

GSB and DBM can be decreased to a certain extent which reduces the total cost of pavement.

Chapter 4

CONCLUSIONS

The results of the study were presented in following conclusions.

(i)The addition of polymer to the natural soil produced an improvement in its mechanical capacities that were determined by unconfined compression tests, from the first period of curing examination. From the strength aspect of liquefiable sandy soils, the optimum polymer content estimated polymer at 2%.

.(ii)The strength of sandy soil mixtures has increased with increment of cement contents up to about 30% and above 30% cement content; the strength of the soil almost becomes constant. This phenomenon is explained by the fact that the fine grains of cement were covered and positioned around and among the sand grains.

(iii) From Fig.4.1, it is clear that the increase in polymer content also increases Unconfined compressive strength of soil if it is maintained less than 4 %, this phenomenon is explained by the fact that increment of polymer and the polymer cover all of sample's area and increases cross-links And the impact on strength with variation in cement content and polymer content is not much.

.(iv) The increase in Unconfined compressive strength is more at start of 20% cement addition in the sand, then its increase is not much when cement content is increased.

(v)It has been observed that the CBR values increases with increase in cement and Polymer content, CBR values is much increased in the first and second days of Soaking but its values not increases much with increase in days of Soaking.

(vi)CBR values have much impact when soil is stabilized with cement and polymer but cement and polymer content does not give much variations when their impact is observed closely.

(vii) Polymer and cement addition into the clayey soil reduces the pavement thickness and hence the cost of pavement to a good extent.

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