PERFORMANCE STUDIES ON PAVEMENTS USING CHEMICALLY STABILIZED SOILS

Thesis

Submitted in partial fulfillment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

by

LEKHA B M

DEPARTMENT OF CIVIL ENGINEERING

NATIONAL INSTITUTE OF TECHNOLOGY KARNATAKA

SURATHKAL, MANGALORE - 575 025

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DECLARATION

By the Ph.D Scholar

I hereby declare that the Research Thesis entitled **“Performance Studies on Pavements Using Chemically Stabilized Soils”** which is being submitted to the National Institute of Technology Karnataka, Surathkal in partial fulfillment of the requirements for the award of the degree of **Doctor of Philosophy** in **Civil Engineering**, *is a bona fide report of the research work carried out by me*. The material contained in this Research Thesis has not been submitted to any University or Institution for the award of any degree.

(LEKHA B. M)
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Place: NITK, Surathkal
Date: 13-10-2015

CERTIFICATE

This is to certify that the Research Thesis entitled **"Performance Studies on Pavements Using Chemically Stabilized Soils"** *submitted by* **Lekha B M** (Register Number: **112012CV11F09**) as the record of research work carried out by her, is accepted as the Research Thesis submission in partial fulfillment of the requirements for the award of the degree of **Doctor of Philosophy.**

Dr. A.U Ravi Shankar
Research Guide
(Signature with date and seal)

Dr. K. N. Lokesh
Chairman-DRPC
(Signature with date and seal)

DEDICATED

TO

MY PARENTS,

FAMILY MEMBERS, FRIENDS

AND

TEACHERS

ACKNOWLEDGEMENT

I would like to express my sincere gratitude to my research supervisor Prof. A.U Ravi Shankar for his motivation and invaluable guidance throughout my research work. I am grateful to him for his keen interest in the preparation of this thesis. It has been my pleasure to work with him.

I acknowledge my sincere thanks to Prof. Nityananda Shetty A, Dept. of Chemistry and Prof. Sitaram Nayak, Dept. of Civil Engineering for being the members of Research Progress Assessment Committee and giving valuable suggestions and the encouragement provided at various stages of this work.

I wish to thank Director NITK Surathkal, Prof. R. Shivashankar, Prof. Katta Venkataramana, former Heads of the Civil Engineering Department and Prof. Lokesh K N, Head of the Civil Engineering for their support and encouragement throughout my stay at the NITK campus.

I appreciate the co-operation and help rendered by the staff of laboratories and the office of the Civil Engineering Department. My special thanks to Mr. Sadanada Kadri and Mr. Yathish S for their help in completing my experimental work.

I am also thankful to the suppliers of chemicals, Dr. Ajay Ranka, (Chief Executive Officer, Zydex Industries), Mr. Ganesh Hegde, (Deputy General Manager, Zydex Industries), Mr. Samir Das Gupta, (Proprietor, Terra Nova Technologies) and M/s. Alchemist Technology Limited for providing material and technical suggestions.

I sincerely thank Prof. G L S Babu, Professor, IISc Bangalore for giving permission to carry out the resilient modulus experiment in their laboratory and Mr. Anthony Raj, Technician, Geotechnical laboratory, Civil Department, IISc Bangalore for providing help during the experiments.

I am fortunate to have friends, Mr. Goutham Sarang, Mrs. Lekshmi J Nair, Ms. Priyanka B A, Ms. Shourie Ranjana J and Ms. Vajreshwari U, whose contributions and encouragements have taken me this far. The informal support and encouragement of many friends has been indispensable. I also acknowledge the good company and help received by of PG students Ms. Chaitali Nare, Ms. Aswathi, Ms. Saswathi and Mr. Avinash during laboratory works.

I am especially grateful to my father Sri. Manjappa and mother Smt. Sunanda B N, who provided me the best available education and encouraged in all my endeavors. I am grateful to my fiance Mr. Vikram A V, for his cooperation and moral support, and I lovingly acknowledge the support and help extended from my family members during this research work. They have always been a source of inspiration for me.

Finally I am grateful to everybody who helped and encouraged me during this research work.

NITK Surathkal LEKHA B M

Date: 13-10-2015

ABSTRACT

Pavements constructed on weak soils can cause significant distress due to moisture-induced volume changes and low strength, thereby reducing the pavement life. Soil stabilization is the alteration of one or more soil properties, by mechanical or chemical means, to obtain an improved soil material possessing the desired engineering properties. Subgrade soils may be stabilized to increase the strength and durability or to prevent erosion and dust generation. In the present study two types of soils, Lateritic Soils (LS1 and LS2) and Black cotton soil and were stabilized with five different stabilizers viz. Terrasil, Terrabind, Cement, Road Building International grade 81, and marginal materials like Fly ash, Arecanut coir and aggregates. These additives can be used with a variety of soils to improve their native engineering properties, but their effectiveness depends on the amount of additive and the nature of soil. The laboratory investigations were conducted for different curing days to determine the basic and engineering properties of soil such as Atterberg's limits, grain-size distribution, Maximum Dry Density (MDD), Optimum Moisture Content (OMC), California Bearing Ratio (CBR), Unconfined Compressive Strength (UCS), Indirect Tensile (IDT) Strength, Durability, Fatigue and Resilient Modulus (E). The investigations are also carried out to study the effect of addition of 12.5 mm down aggregates to the soil with optimum content of Cement and RBI 81 to evaluate the extent of modification in the Compaction, CBR, IDT strength and resilient modulus tests. The experimental investigations indicate that there is a good improvement in the engineering properties of the soils treated with different stabilizers. KENPAVE software was used for stress strain and damage analyses of both natural and stabilized soils and also to prepare pavement design sections for low and high volume pavements. For low volume pavements, CBR 3% and traffic T4 to T7 conditions were considered as per IRC-SP-72:2007. For high volume pavements, analyses were carried out for CBR 8% and traffic 2 to 150 million standard axles, using the standard design thickness as per IRC-37:2012 guidelines. Trial and error method was adopted to determine the thickness for treated soil aggregate mixture, by keeping the strain value within permissible limits. For stabilized soil, rutting and fatigue lives and damage ratio were also observed to be significantly improved. From the results of the experimental research and KENPAVE analysis, it has been observed that modified soil can be effectively used as a modified subgrade and base layers. Analysis was also performed in IITPAVE for high volume roads under dual wheel loading. Cost analysis was carried out as per the Schedule of Rates (SOR) 2014-2015 for stabilized and unstabilized materials.

Key words: Lateritic soil, Black Cotton soil, Stabilization, UCS, CBR, IDT, Fatigue, Durability, KENPAVE, IITPAVE, Cost analysis.

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NOMENCLATURE

Symbol	Abbreviation
A	Aggregate
BC soil	Black Cotton Soil
C	Cement
CBR	California Bearing Ratio
DBM	Dense Bituminous Macadam
DispZ	Displacement
DR	Damage Ratio
E	Resilient Modulus
epT	Tangential Strain
epZ	Vertical Strain
FA	Fly Ash
FSI	Free Swell Index
FT	Freeze Thaw
GB	Gravel Base
GSB	Granular Sub Base
H	Depth
IDT	Indirect Tensile Strength
IRC	Indian Road Congress
LS1	Lateritic Soil 1
LS2	Lateritic Soil 2
MDD	Maximum Dry Density
msa	million standard axles

N_F	Fatigue Life
N_R	Rutting Life
OMC	Optimum Moisture Content
RBI 81	Road Building International Grade 81
SADW	Single Axle Dual Wheel
SASW	Single Axle Single Wheel
SigmaZ	Vertical Stress
UCS	Unconfined Compressive Strength
Vert. Displ.	Vertical Displacement
Vert. Stress	Vertical Stress
WBM	Water Bound Macadam
WD	Wet Dry

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Highways provide the most reliable means of transport under different topographical conditions. India has over 4.885 million kilometers of roads in which less than 40% are surfaced and around 90% of these are flexible pavements. The latest statistics present that nearly 92851 km of National Highways (NH), 1,42,687 km of State Highways (SH) and 46,49,462 km of Rural roads cater to the mobility requirements in addition to roads of other categories (MoRTH, Annual report 2013-14). Pavement construction technology is being improved with rapid changes in the field of infrastructural development taking place all round the world. India has proven its engineering excellence in various fields of Civil Engineering including road construction, and our engineers have demonstrated their capability in adopting scientific approaches to solving challenging problems in providing accessibility to the people.

Subgrade is an important foundation layer in any pavement generally formed by natural or borrowed soil, above which other layers including sub-base, base and surface courses are constructed. The quality and stability of subgrade is a major factor responsible for the adequate performance and service of any pavement during its life span. The design of pavement should be focused on the most economical and effective use of existing subgrade materials to optimize their performance. In case of soft and wet subgrades, necessary treatment might be needed in order to make the subgrade workable for overlying layers for pavement construction.

Lateritic soils (LS) have been found in the coastal region, along the Konkan belt of India. High rainfall, temperature and humidity with alternative wet and dry period, which are ideal conditions for laterization, makes nearly 40 per cent of the soils in the area laterites. Its colour ranges from red to yellowish red and depth from 30 to 150 cm. The laterites have been mostly originated from igneous rocks and are well drained residues with the presence of excessive Iron and Aluminium. Black Cotton (BC) soil, being expansive in nature, exhibits large volumetric variations caused by moisture fluctuations from seasonal changes, and considered as one of the problematic soils by the highway engineers. Damage due to swelling action is common in BC soil in the form of cracking and breakup of pavements and other structures. The removal of expansive soils and replacement with suitable fill material is a commonly adopted solution to this problem. However, feasibility of this method depends on the availability of preferred fill material within a suitable distance, the thickness of the weak sub-grade soil to be replaced and ultimately the cost and time involved.

1.2 FLEXIBLE PAVEMENTS

Based on the materials and layers used, pavements are mainly classified as flexible, rigid and composite types. Flexible pavements are constructed with bituminous surfacing and are generally preferred over the other types because of their less initial cost, suitability for stage construction and easy to repair. Flexible pavements are layer system constituting surface, base, sub base, and sub-grade layers with better quality materials at top and locally available or marginal materials in the bottom layers. Full depth bituminous pavements, though popular are more expensive. In the multi-layer system, classical design procedures are based on limiting the vertical compressive strain at the top of the sub-grade and the horizontal tensile strain at the bottom of the surface layer. Stresses and strains are limited by increasing the layer thickness and stiffness of the top layers.

1.2.1 Low Volume Roads

Low volume roads are constructed in areas where lesser traffic is expected and this traffic criterion varies for different agencies. Generally roads to carry an average daily traffic of less than 400 vehicles per day are considered in this category and in India, the limit is less than 450 commercial vehicles per day (Ramulu et al. 2012). The construction of low volume roads connecting villages has enormously increased with the introduction of Pradhan Mantri Gram Sadak Yojana (PMGSY) in 2000. Over the past decades, clay soils present unique problems to engineers in the construction of long lasting pavements. Low volume roads constructed on clay subgrade are a major challenge to engineers because the moisture changes cause uneven pavement surface, cracking, premature deterioration and replacement.

1.3 PROBLEMS IN FLEXIBLE PAVEMENTS

The state of art of planning, design, construction and maintenance of various existing road networks in India could be more appropriately described as traditional in character and quite a bit of empiricism is involved. Maintaining the same pavement thickness, without properly characterizing the sub-grade soils, improper selection of materials and construction methods and inadequate compaction due to non-availability of compacting equipment pose many problems. In addition, the increased traffic volume, truck loading and additional overlays as maintenance cause heavy stresses on the pavement layers resulting in early distresses. Conventional flexible pavements have Water Bound Macadam (WBM) or Wet Mix Macadam (WMM) as the base course and gravel as the sub-base material, both of which are good load distributing layers with sufficient material properties to transfer the loads coming from the top layers. They do not need any replacement or improvement, but require strict quality control during construction.

1.4 SOIL STABILIZATION

Soil stabilization is the process of improving the engineering properties of soil and thus making it more stable. It is required when the available soil for construction is not suitable for the intended purpose. In the broad sense, stabilization includes compaction, pre-consolidation, drainage and many other such processes. The process may include blending of soils to attain a required gradation or incorporation of commercially available additives to modify the gradation, texture or plasticity, or to act as a binder for cementation of the soil. Along with mechanical methods, chemical stabilization is also being applied worldwide even if the method is at a judging stage in India. Geotechnical properties of poor subgrade soil can be improved by various methods and it can be replaced by good quality of subgrade material. But this method may be expensive and hence overall economy cannot be achieved (Patel and Patel 2012). Soil stabilization techniques are considered to be an economic solution at places where granular materials are not easily available (Portelinha et al. 2012). These techniques are expected to provide good quality structural stability for long term performance of pavement (McConnell 2009).

1.5 MARGINAL MATERIALS

Construction costs of the upper pavement layers like base and sub base are typically about 30 to 40 per cent of the total road construction cost. The Indian standards provide limiting criteria on compaction, plasticity and grading for base and sub base materials, which can be applied to all traffic levels. Those materials, which do not meet these requirements, are considered as “sub-standard”. But these materials can be made suitable for pavement construction after proper modification. Similarly, some waste materials and by-products from industries, including fly ash, slag etc., can also be used as pavement materials and stabilizers to improve soil properties. These materials are generally encouraged to utilize in pavements in this era with scarcity of good quality construction materials, and they are commonly known as marginal materials. In India, marginal materials have been used to a very limited extent in the construction of pavements and embankments. It is finding difficulty to procure materials to satisfy base and sub base requirements in most of the places. In such cases, marginal materials can be used by improving their characteristics or by using them along with other construction materials.

1.6 PAVEMENT PERFORMANCE

Among the three layers in the conventional flexible pavements, the surface course is meant to provide better riding quality, imperviousness, friction and visibility. When the base course acts as load distributing layer, the sub-base layer has an additional role as a drainage media. The sub-grade acts as the foundation, receiving the entire load transferred from the wheel loads through the component layers. Well-designed flexible pavements over poor sub-grades experience distress in a shorter period. Though distress is an important component in pavement design, many of the structural distresses are caused by the deficiencies in construction, material properties and maintenance. A typical pattern of structural failure in flexible pavements is rutting, which develops rapidly during the first few years and then levels off to a much slower rate. Fatigue or alligator cracking does not occur until considerable loading and then it increases further leading to pavement weakening. Climate variations develop transverse and longitudinal cracking in pavements, which usually break down and spall under traffic.

In recent years, highways have experienced an increase in severity and extent of permanent deformation (rutting) especially due to poor sub-grades. The increased rutting can be attributed to increased axle loads, traffic volumes and poor sub-grades. Rutting develops with an increasing number of load applications and is caused by a combination of densification and shear related deformation, which may occur in any layer of the pavement structure. There are two types of rutting. In one type, a weak sub-grade, sub-base, or base course below the surface layer allows permanent strain to develop and repeated loads depress the pavement into weak underlayment. This type of rutting is usually associated more with pavement structure design and underlying materials than with surface materials. The other type of rutting is due to wear of the top surface or instability of the paving mix leading to flow of material on to the two sides of the wheel path. The layer in which rutting occurs is determined by loading magnitude and the relative strength of the pavement layer.

1.7 OBJECTIVES AND SCOPE OF THE PRESENT STUDY

The main objective is to investigate the stabilization mechanism of some of the commercially available additives along with some marginal materials to better understand their potential usage in pavement construction. Experiments are performed in the laboratory to determine whether these products improve the properties of the soils.

1. To study the improvement of geotechnical and engineering properties of Lateritic soils and Black Cotton soil by stabilizing with different stabilizers at varying curing periods.
2. To qualitatively evaluate the durability of treated specimens with regard to wet-dry and freeze-thaw cycles.
3. To evaluate the Resilient modulus, Indirect Tensile strength and Fatigue behavior of stabilized soils.
4. To perform the stress strain analysis using KENPAVE and IITPAVE software for low volume and high volume pavements and preparation of design charts with stabilized soils.
5. To carry out the cost analysis for stabilized and unstabilized soil mixes.

Scope of this study includes the stabilization of two types of soils for low and high volume pavements. Over the years engineers have tried different methods, including thermal, electrical, mechanical or chemical means, to stabilize soils that are subjected to reduction in strength properties as a function of variation in moisture content. Many studies were carried out on the effect of different stabilizers on these improvement measures, but no significant effort has been made on the long term impact of these treatments on the flexible pavement performance. In the present investigation, an effort is made to study the effect of traditional and non-traditional stabilizers for improving the engineering properties of soils used for pavement sub surface layers. Soil samples, obtained from the sites, were kept for oven drying. The standard and modified proctor tests were performed to determine the Optimum Moisture Conditions (OMC), and the Maximum Dry Density (MDD). In order to determine the engineering properties such as Consistency limits, Unconfined Compressive Strength (UCS), California Bearing Ratio (CBR), Permeability, Durability, Fatigue, Indirect Tensile (IDT) strength and triaxial tests were conducted in the laboratory. LS and BC soil by stabilizing with different dosages of chemicals like Terrabind, Terrasil, Cement, Road Building International grade 81 (RBI 81), Fly Ash (FA), Arecanut coir and aggregates were used at varying curing periods.

1.8 ORGANIZATION OF THE THESIS

The present work has been divided into nine chapters and compiled in this thesis for the purpose of better understanding and clarity of the proposed problem.

Chapter 1 includes importance and principles of soil stabilization. It also covers the motivation for the present investigation with objectives and scope of the work.

Chapter 2 discusses the detailed review of the literature about the concept of soil stabilization, traditional and non-traditional stabilization mechanisms, behavior of the stabilized soil under repeated loading and research work carried out on this area.

Chapter 3 presents methodology adopted and detailed laboratory investigation of two types of soils selected. The geotechnical properties of the soils were determined adopting the procedures detailed in relevant Indian Standard Codes. The soils are treated with different dosages of stabilizers and change in properties such as consistency limit, Compaction, CBR, UCS, Durability, IDT strength and Fatigue behavior were determined for stabilized soils.

Chapter 4 discusses on results of the tests carried out on locally available Lateritic soils (LS1 and LS2) stabilization. LS1was stabilized with Terrasil, Terrabind with Fly ash and 3% cement with Arecanut coir. LS2 was stabilized with Cement, RBI 81 and Aggregates.

Chapter 5 presents results of the tests carried out on BC soil stabilized with Terrasil, Terrabind with Fly ash and RBI 81.F

Chapter 6 provides details on the durability studies on stabilized LS and BC soils, as per the ASTM D 559 and 560 for wet-dry and freeze-thaw tests.

Chapter 7 discusses the IDT strength and Fatigue studies on stabilized soil specimens.

Chapter 8 details KENPAVE and IITPAVE analysis and Cost analysis for treated and untreated soil. The analysis using KENLAYER is performed on the standard cases from the pavement design catalogues for IRC: SP: 72-2007 and IRC: 37-2012.

The analysis was performed to reduce the total pavement thickness, for which new cases have been developed and run in KENLAYER and IITPAVE. Also economic analysis was performed to determine the savings in the cost of construction for both low and high volume roads. Cost comparison has been made as per the Schedule of Rates specified by the Mangalore Public Works Department, Karnataka.

Chapter 9 Summarizes the investigation and conclusions are drawn based on the experimental and analytical study.

CHAPTER 2

LITERATURE REVIEW

This chapter provides a review of research findings on discussion of selected previous works on soil stabilization with the traditional and non-traditional stabilizers used in the study. Soil Stabilization has proved to be an effective and economical method to improve the strength of pavement layers. This section also discusses the different mechanisms of modification and stabilization, and the resulting improvement in soil properties.

2.1 LATERITIC SOIL STABILIZATION

Soil stabilization has a history which reaches at least 300 years into the past. The term laterite was first introduced by Buchanan (1807) to describe a vesicular, unstratified and porous material with yellow color due to high iron content occurring in Malabar, India. It undergoes all the effects of natural modification with an additional long term strengthening. Soil stabilization has a greater role in the soil conditions and mineralogical properties than modification. Lateritic soils are locally available materials that are cheaper and can be used for construction. It is a highly weathered natural material formed as a result of secondary physicochemical processes resulting in the concentration of hydrated oxides of iron or aluminum, either by residual accumulation or by solution, movement, and chemical precipitation (Gidigasu 1971). The accumulated hydrated oxides are sufficiently concentrated to affect the character of the deposit in which they occur. It has a high bearing capacity and low compressibility and does not present any difficult foundation engineering problem except in cases of high clay content. Soils which contain hydrated oxides of iron and aluminum may become less plastic on drying. This is partly because of the dehydration of sesquioxides which creates a stronger bond between the particles, resisting the penetration by water (Bwalya 2006).

2.2 NON-TRADITIONAL STABILIZATION

There are two types of stabilizers, categorized as traditional and non-traditional. Traditional stabilizers include cement, fly ash, lime, etc. and they were in use for centuries. On the other hand, the use of non-traditional stabilizers (like lignin derivatives, enzyme, acids, polymers etc.) is very limited. Even though a few researchers have performed laboratory and field evaluation with non-traditional stabilizers, their mechanism of stabilization was not thoroughly examined.

Presently, usage of liquid stabilizers for soil stabilization gets more attention than other stabilization methods and since the technique is practical, trustable, economic and convenient, it has a wide acceptance. A variety of natural polymers, such as lignosulfonate and synthetic polymers are marketed, but the constituents of these polymers are typically undisclosed by suppliers. Gow et al. (1961), Mollah et al. (1995), Chen (2004) and Lemes et al. (2005) demonstrated lignosulfonate as an effective stabilizer. The lignosulfonate was used to treat a soil-aggregate mixture, and then CBR tests were performed on compacted specimens. Soaked specimens showed an increase in strength after curing for a week, but the strength increase was markedly less than that was seen with unsoaked specimens. This phenomenon seems to be linked to the hydrophilic nature of the lignosulfonate, as it will tend to dissolve in water. New method of soil stabilization by adding different chloride compounds which included NaCl, $MgCl_2$ and $CaCl_2$ in varying amounts (2, 4 and 8%) to the soil gave positive effects on the compaction characteristics, consistency limits and compressive strength. The increase in the percentage of each of the chloride compounds improved consistency limits and MDD, but decreased OMC (Abood et al. 2007). Liquid stabilization is considered as an appropriate solution for fast operation and implementation of geotechnical projects, which can limit water absorption, soil erosion, losing water and soil settlement (Eisazadeh 2010, Ali 2012).

The geotechnical properties have been improved when soil is treated by mixing lime and sodium silicate with an initial consumption of 4 and 2 % respectively. The soil has improved as the sodium silicate content increased with significant reduction in PI and swelling potential. The maximum reduction has been found at the mix of 6% lime and 2% sodium silicate, whereas the highest CBR has been obtained for soil with of 6% lime and 2.5% sodium silicate. The reaction time is a significant parameter in the process, where strength improves with the increase in time (Maaitah 2012). The Sulphonated oil-treated materials had an increase in strength over the test period, and it was concluded that these stabilizers need a curing time of a few dry months to reach their maximum strength. Moreover, the material seemed better when applied for a laterite soil containing a reactive clay mineral. This chemical stabilizer can be recommended for using on low cost unpaved roads to reduce construction and maintenance costs, which ensures that the unpaved roads remain drivable in rainy weather and dust-free in dry conditions (Far et al. 2013). Commercial liquid stabilizers, TX-85 and SH-85, significantly improved the strength of the laterite soil, in a much faster and cost efficient way compared to traditional stabilizers such as lime and cement (Marto et al. 2014).

According to Scholen (1992) non-standard stabilizers are byproducts of industrial processes (e.g. fermentation), modified specifically for using as stabilizers and the author tried to describe the enzyme stabilization mechanism. It was reported that the enzymes act as catalysts to speed up a chemical reaction in which large organic molecules in the soil react to form an intermediary that exchanges ions with the clay structure. The clay lattice then breaks down preventing further absorption of water and loss of soil density. Wright-Fox et al. (1993) observed an increase of 15% in the UCS of silty clay, with enzyme stabilization, but no significant improvement was observed in the index properties. It was concluded that, enzymes may increase the shear strength for some soils, and the soil stabilization with enzymes should be considered for various applications, but only on a case-by-case basis. Santoni et al. (2002, 2005) performed tests on silty-sand material with the nontraditional stabilizers. The stabilizers tested were Type I Portland cement, hydrated lime, cationic asphalt emulsion, an acid, four enzymes, two lignosulfonates, a petroleum emulsion, three polymers and a tree resin. It was observed that among the traditional stabilizers (cement, lime, and asphalt emulsion) only the cement increased the UCS by more than 100% over the control for both wet and dry tests. The petroleum emulsion, tree resin and lignosulfonate showed good waterproofing potential, but no significant dry

UCS improvement. Overall it was determined that the nontraditional stabilizers gained strength quicker than the traditional stabilizers. Tolleson et al. (2003) studied the effect of the enzyme stabilizer PZ-22X with several soil types, ranging from poorly graded sand (SP) to cohesive clay (CL). The enzyme stabilizer improved CBR and soil stiffness geogauge values for cohesive clay. The effectiveness of this particular enzymatic stabilizer did not depend on the properties of the fines in the soil, but on the quantity of fines. Velasquez et al. (2005) studied the effect of two enzymes as soil stabilizers on two soil types to determine how and under what conditions they function. The enzymes produced a high concentration of proteins and observations suggest that the enzymes behave like a surfactant, which affects its stabilization performance. The specimens were subjected for varying curing periods to determine their performance. It was observed that an increase in the E value as the curing time increased but that an increase in enzyme application rate, as suggested by manufacturers, did not improve the effectiveness of the stabilization process. Shankar et al. (2009) observed that geotechnical properties of laterite soil have been much improved by stabilizing with enzyme dosage of 200 ml/ 2 m^3 of soil. It is suggested first examining the effect of bio-enzyme on soil stabilization in the laboratory before actual field trials.

2.3 TRADITIONAL STABILIZATION

Extensive research has been completed pertaining to the use of traditional stabilizers, like lime and cement. Cement is an effective stabilizer material that can be used to improve the properties of different soils, silts, clays and even granular materials. Cement was used as a stabilizer material as a trial basis in 1917 and after that gained acceptance as an alternative to improve weak soils, leading to several research and construction works (Noor 1994, Horpibulsuk et al. 2003, Basha et al. 2005, Horpibulsuk et al. 2008, Houssain 2010). The studies concentrated in the strength enhancement and durability properties and various factors including the type of stabilizer, genetic origin, mineralogical and chemical composition were found to be affecting the stabilized soil performance. Bugge and Bartelsmeyer (1961) observed that cement improves the strength and alters the volume change of soil by immediately reducing its plasticity index (PI). The reduction in soil plasticity is a result of calcium ions released during the initial hydration reactions.

According to Mitchell (1981) and Anon (1990), cement stabilization is one of the most suitable stabilization methods for pavements, which involves three processes viz. cement hydration, cation exchange reaction and pozzolonic reaction carbonation. Geiman (2005) also explained that the strength gain in soils using cement stabilization is due to pozzolanic reactions. Cement contains calcium, which helps in the pozzolanic reactions; and silica, which breaks down the clay minerals in the soil. The only requirement in this process is necessary amount of water for initiating the carbonation and hydration reactions. Water cement ratio is an important parameter affecting the strength of cement stabilized coarse-grained soils at OMC and on the wet side of the optimum, and higher strength can be obtained when this ratio is less (Horpibulsuk et al. 2006). Oyediran and Kalejaiye (2011) explained the mechanism with which cement reacts with water. With the addition of water, cementation occurs as a result of hydration of cement that creates a matrix between soil aggregates with strong bonding between them. The stabilized soil matrix obtains its strength from honeycomb like structure, which does not allow soil particles to slide over each other. The effect of cement reduces the affinity for water and thereby water holding capacity of the soil and the soil strength enhances with time due to pozzolanic reaction (Gomez and Anderson 2012).

Aydogmus et al. (2004) examined some of the mechanical properties when 6% cement is added to a typical cohesive soil with and without geogrid reinforcement. The addition of cement to clayey soil reduces noticeably the OMC and marginally the MDD for the same compaction effort. The strength of soil-cement tends to increase in a linear manner with increasing cement content. A local clay sandy soil was stabilized by Kenai et al. (2006) using different dosages of cement (6, 8, 10, 12, 15 and 20 % by weight of dry soil) for varying curing periods and strength parameters were analysed. Dynamic compaction was employed for soil samples and optimum dosage was obtained as 8%. Syed et al. (2007) conducted performance studies on soil samples collected from various borings with addition of 3, 4 and 5 % cement. Results indicated that the MDD for the cement stabilized subgrades varied from 1.68 to 2.02 g/cc with an average value of 1.83g/cc. The UCS values also increased with addition of cement with respect to curing days and it was concluded that stabilizing the in-situ subgrade soils with small amounts (4% by weight) of cement is a technically viable, cost effective and speedy way to prepare the subgrades for the reconstruction of the airfield pavements. Sadek et al. (2008) reported that the MDD increases and OMC decreases with the increase in sand and cement additives. The results showed that the additive admixtures altered the engineering properties of tropical peat soils and higher strength was obtained from samples that had been cured for 14 days compared with 7 days cured samples. Jaritngam et al. (2013) investigated on lateritic soil with different cement content by conducting tests on UCS and elastic modulus for different curing days and observed that 3% optimum cement content treated soil can be used as a base course material in highways.

Cement stabilization is more effective in granular soil because of the easiness in pulverization and mixing, and is economical since low dosage of cement is sufficient for stabilization (Das 1994, Hicks 2002, Chavva et al. 2005, Zhang and Tao 2008). Granular soils with less than 40% fines can act as an adequate raw material for cement stabilization. For cement-stabilized aggregate base layer samples, higher cement content produced higher UCS values and a linear correlation between these two was also observed (Peng and He 2009). Addition of 1 or 2 % of cement to granular base materials provided mixes performing better than recycled cement concrete materials (Haichert et al. 2012). It was also noted that, adding more cement may cause in increased stiffness leading to brittle failures.

Mohammad et al. (2000) evaluated Indirect Tensile Strength (ITS) and resilient modulus characteristics of soil-cement mixtures using cylindrical specimens with diameter 102mm and height 64mm. The increase in compactive effort and curing period resulted in significant improvement of tensile strength and also, the reduction of cement content caused decrease in strength and resilient modulus values. White and Gnanendran (2005) prepared treated specimens by compacting as per standard proctor and gyratory compaction methods, and observed that the influence of compaction method on ITS and resilient modulus is not significant. The cement stabilized base was produced in the field by stabilizing lateritic soil with cement to improve shear strength and resilient modulus.

A commercial cementitious material named Road Building International Grade 81 (RBI 81) was tried by researchers to improve the soil properties. It is an additive in powder form, acting on soil to reduce the inter-particular voids in soil and also minimize the absorbed moisture in the soil contributing to the maximum compaction. The effect of RBI 81 with different percentages shows significant improvement in geotechnical properties including soaked and unsoaked CBR values of lateritic soil, red soil and kaolinite and the optimum stabilizer dosage was obtained as 4%, 2% and 6% respectively (Anitha et al. 2009). Inclusion of RBI 81 along with moorum improved the geotechnical properties of subgrade soil. Various proportions of soil and stabilizers were tried and the optimum was obtained at 71% soil, 20% moorum and 4% RBI 81 which provided a six times increase in the soaked CBR value (Patil and Patil 2013). Limited research works were carried out by using RBI 81 in lateritic soil, compared to that in expansive soil.

2.4 COIR STABILIZATION

India is considered as the largest Arecanut producing country in the world. The total area under cultivation is more than 388,000 hectares and the annual production is approximately 482,000 tonnes, with the states Karnataka and Kerala accounting for nearly 72 per cent of the total production (Campco 2014). Among all the natural fiber-reinforcing materials, Arecanut fibre appears to be a promising material because it is inexpensive, abundantly available and the crop is very high potential perennial. The husk of the Arecanut is a hard fibrous portion covering the endosperm. It constitutes 30–45% of the total volume of the fruit. Areca husk fibers are predominantly composed of hemicelluloses.

The concept of soil reinforcement was first developed by Vidal (1996). It was established that the introduction of reinforcement elements in a soil mass increases the shear resistance of the soil matrix. Lekha and Sreedevi (2005) studied on coir fiber for stabilization of weak sub grade soils, which included treating the weak soil with coir fibre at different quantities and studying the changes in OMC, MDD and CBR values. The OMC was found to be increased with the increase in the percentage of coir fibre content and correspondingly, MDD decreased. Tang et al. (2007) investigated the effects of discrete short polypropylene fiber (PP-fiber) on the strength and mechanical behaviour of uncemented and cemented clayey soil. The test results indicated that the inclusion of fiber reinforcement within uncemented and cemented soil caused increase in the UCS, shear strength and axial strain at failure, decrease in the stiffness and the loss of post-peak strength, and change in the behaviour of cemented soils from brittle to more ductile.

Kumar and Singh (2008) tried different combinations of polypropylene fiber and fly ash on soil. It was observed that the addition of fiber to soil satisfy all the geotechnical properties to meet the requirements of sub base layer. Bijayananda et al. (2011) conducted a series of laboratory soaked and unsoaked CBR tests on randomly oriented fiber reinforced and unreinforced specimens of clayey soil, compacted at OMC and MDD. Coir fiber has been used as a reinforcing material to investigate its beneficial use in rural road sub grade soil. From CBR test results, the engineering performance of coir fiber inclusion was examined. The results indicated that the inclusion of coir fiber enhanced the CBR strength of the soil specimens significantly. Clayey soils mixed with fibers showed remarkable increase in the CBR strength in comparison with the same soils without fiber inclusions. That is, randomly oriented discrete fiber reinforcements in clayey sub grade offered higher resistance to penetration than unreinforced one, under similar loading conditions. Shankar et al. (2012) studied on lithomargic clay stabilized with different percentages of sand and coir, and improvement in almost all properties was observed. The CBR both in soaked and unsoaked conditions, increased as the percentage of sand increased from 0 to 40 and coir from 0 to 0.5. With the increase in the sand content, the UCS values of blended soil for both light and modified compaction densities increased up to a certain limit, whereas, the increase of coir content resulted in a continuous increase in UCS. Even though Arecanut coir is a biodegradable material, according to Ramaswamy and Aziz (1989), its strength and condition beyond a period of one year after placement should not be of any concern, as by that time the coir would have already played a very important role in providing a self-sustaining subgrade for most of the soil types. The loss of strength of the coir with time can be well compensated by the gain in strength of the subgrade within the same time frame.

Kar and Pradhan (2012) studied on soil stabilized with fly ash and fiber reinforced fly ash for low volume roads. Soaked CBR values for reinforced fly ash soil showed good improvement. A study by Sarbaz et al. (2014) on soil specimens reinforced with palm fibers and bitumen coated fibers showed that palm fibers significantly increases the CBR strength of the sand specimens. Maheshwari et al. (2012) conducted a series of laboratory tests on unreinforced and fiber reinforced black cotton soil with different amount of fibers and there was a significant increase in CBR value with the inclusion of fibers. As per the Indian Road Congress (IRC) standard IRC 37-2012, the flexible pavement sections resting on fiber reinforced soil for traffic volumes of 1 to 150 msa were designed and modeled using finite element software Plaxis 2D. Considerable reduction in deformation was obtained on the top of sub-grade due to reinforcing of sub-grade soil using fibers.

2.5 BLACK COTTON SOIL STABILIZATION

The origin of expansive soils is related to a complex combination of condition sand processes resulting in the formation of clay minerals with a particular chemical structure which expands during the reaction with water. All clay soils are not expansive and the degree of expansion varies with the type of clay mineral predominantly present in the soil mass. The presence of montmorillonite provides high swell-shrink potentials to these soils and the soil is very hard when it is dry, but loses strength completely when it is wet (Chen 1988). Nicholls and Davidson (1958), Sinha et al. (1957) and Davidson and Handy (1960) conducted studies on the use of sulfite lignin in various civil engineering applications. Adding lignin to clay soils increased soil stability by causing dispersion of the clay fraction and it was demonstrated that sulfite lignin is effective in soil stabilization and dust control for unpaved pavements and also confirmed that lignin admixtures indeed improve some engineering properties related to soil stability. It was also noted that the strength of lignin-treated soil increases rapidly with an increase in the duration of air curing. Gow et al. (1961) used a lignosulfonate to treat a soil-aggregate mixture and determined its effectiveness by CBR test. For unsoaked specimens, CBR value showed higher improvement after one week curing, whereas for soaked specimens the improvement was slightly less than that for unsoaked ones. This may be due to the water loving behavior of lignosulfonate. Investigations showed that lignosulfonate along with a small amount of sulphuric acid, was helpful in improving the shear strength and resilient modulus of cohesive soil (Puppala and Hanchanloet 1999, Tingle and Santoni 2003). Many types of polymers are generally considered as good soil stabilizers since the hydrocarbon chains in polymers become entwined within the soil particles enabling stabilization. Polymers also act as a binder to glue the soil particles together and reduce dust, and stabilize the entire soil matrix (Brown et al. 2004).

Extensive researches have been conducted using Fly Ash (FA) for stabilization of different types of soils including BC soil. Leelavathamma and Pandian (2005) studied on FA with BC soil in layered system and it was observed that the BC soil and the top FA layer improve the CBR strength due to the pozzolanic reaction of FA. The addition of FA to BC soil results in significant improvement in the CBR, due to the pozzolanic effect and hence it can be used as a base material for pavements. The presence of calcium in FA results in the pozzolanic reaction leading to increased strength upon soaking. In the case of FA with low calcium content, the soaked CBR will be less than the unsoaked CBR (Pandian and Krishna, 2001, 2003). Kim et al. (2012) studied the effect of lignin based coproduct and FA on sandy clay. UCS samples were tested on dry and wet conditions and observed that the biofuel coproducts had excellent resistance to moisture degradation for silty clay soil. The Sulphonated oil-treated specimens had an increase in strength over the test period, and it was also observed that the stabilizers need longer curing time to reach their maximum strength. Santoni et al. (2005) and Tingle and Santoni (2003) conducted experiments to evaluate the stabilization of clay soils and silty sand with nontraditional stabilizers, including an acid, enzymes, a lignosulfonate, a petroleum emulsion, polymers, and a tree resin.

Geiman (2005) carried out a study on traditional and non-traditional stabilizers against three Virginia soils. The selected stabilizers were: quicklime, hydrated lime, pelletized lime, cement, lignosulfonate, synthetic polymer, magnesium chloride, and RBI 81. The RBI 81 was observed to be more effective in increasing the strength of soils tested. Cementitious stabilizer may be useful in situations where workability of the soil rather than strength of the soil is a priority. The majority of strength gain for samples treated with lime, lignosulfonate, synthetic polymer, and RBI 81 occurs within 7 days of curing period. Resource center for Asphalt and Soil Training Academy (2008) conducted laboratory studies on properties of soils treated with RBI 81. The four different types of Sandy Clayey soils showed substantial increase in CBR value (20% with 2% stabilizer). The increase and rate of increase after 7 days in UCS value of stabilized soil samples with increasing stabilizer content was not substantial. It was observed that a low percentage (about 1-2%) of stabilizer is effective to improve the properties of such clayey soils. The CBR value of the Loamy soil increased with 2% and higher proportions of stabilizer and therefore it appears that material is effective. The soil treated with pond ash and RBI 81 improves the soaked CBR strength. For a mixture of soil, pond ash and RBI 81 (in the proportion of 76:20:4) Patil and Patil (2013) observed an increase in CBR from 3% to 13%. Usage of RBI 81 stabilizer improved the strength and swelling characteristics of two clayey soils (Gunturi et al. 2014) with 3 to 4 times increase in the soaked CBR values as that of the untreated soils. The swelling potential of clayey soil samples decreased with increase in RBI 81 dosage and curing period. Lekha and Shankar (2014) used RBI 81 with BC soil and observed that the engineering properties of soil has been increased with increase in dosages, since the cementitious hydration produces a glue that gives strength and structure in the treated soil.

2.6 DURABILITY STUDIES

Durability is one of the major requirements of stabilized soils indicates the retaining of performance even under critical weathering conditions. Bulman (1972) recommended the use of WD test as a check for the suitability of a soil for stabilization because it bears a closer relationship to the strength parameters of the material. Some recent studies focused on to understanding of the durability of natural or stabilized soils with respect to the influence of environmental factors such as WD, FT cycles and immersion on various engineering properties (Allam and Sridharan 1981, Shihata and Baghdadi 2001, Al-Obaydi et al. 2010). Khoury and Zaman (2002, 2007) determined the effect of FT and WD cycles on cylindrical specimens of stabilized aggregate soil. For soil stabilized with 10% Class C FA, resilient modulus and UCS values of 28-days cured specimens were observed to be increasing up to 12 FT cycles, and beyond which a reduction was experienced. The same soil specimens with 15% Cement Kiln Dust (CKD) and 10% class C FA (or fluidized bed ash) were cured for 28 days and then subjected to WD cycle prior to testing for resilient modulus and the values were observed to be decreasing after 30 cycles. Among different durability test procedures used by Zhang and Tao (2008) on cement stabilized samples, the tube suction test showed promising results as an alternative to the regular soil durability test. Ibrahaim et al. (2011) observed that the expansive soil stabilized with 4 and 6% lime become more durable against the cycles of wetting and drying. Arun et al. (2013) used lime with different purity levels for stabilization and observed that low purity lime can be used for silty soil but in the case of clayey soils, it did not show much structural strength improvement. Amadi (2014) conducted a series of durability tests on BC soil with CKD and quarry fines, and the results observed for higher dosage of these stabilizers satisfied the durability criteria. Effect of purity of lime on strength and durability of three selected soils viz. silty soil, BC soil and clayey soil were evaluated in the laboratory.

2.7 MECHANISM OF STABILIZATION

The actual mechanism of any stabilization process should be properly interpreted for its efficient application in field. The soil stabilization performance can be represented as coating and/or binding of soil particle to create stabilized soil with improved characteristics. Prusinski and Bhattacharja (1999) summarized the overall stabilization process in a cementitious stabilizer treated soils into four different processes, namely cation exchange, flocculation and agglomeration, cementitious hydration and Pozzolanic reaction. Cation exchange is a quick reaction of clay and stabilizer within a few minutes of mixing, leading to a soil with improved texture. The tetrahedral (T) and octahedral (O) combination of clay minerals in the ratio of 1:1 (1T and 1O) or 2:1 (2T and 1O) have charge deficiency and attracts the cations or water molecule (Dhakal 2012). Generally, single valency cations like sodium or potassium (Na^+ or K^+) are prevalent in clay minerals, but during stabilization, they are replaced by the higher valance cations like Al^{+3}, Ca^{+2}, Mg^{+2} etc. present in the stabilizer. This process of cation exchange provides a reduced thickness of diffused double layer (Geiman et al. 2005). The calcium, which is abundantly present in the stabilizer, is released in suspension of stabilizer-soil-water and will be available for the stabilization of soil. The general reaction of the cement with water that yields calcium is presented in equations 2.1 and 2.2.

$$2C_3S + 6H \rightarrow C_3S_2H_3 + 3Ca(OH)_2 \quad (2.1)$$

$$2C_2S + 4H \rightarrow C_3S_2H_3 + Ca(OH)_2 \quad (2.2)$$

Where, $H = H_2O$, $C = Ca$, $S = SiO_2$, C_3S = tri-calcium silicate, C_2S= di-calcium silicate and C-S-H = $C_3S_2H_3$, calcium-silicate-hydrate = $C_3S_2H_3$.

The cement contains calcium aluminate hydrate which further stabilizes the flocculated clay particles by yielding glue like structure with C-S-H. The strength provided by cementitious hydration in cement treated soil provides an extra strength and that makes cement stabilized soil stronger than any other stabilized soils. The rapid strength gain continues from mixing time to a month or more, and may even continue for a few years. Pozzolanic reaction is a long-term process which produces more stable hydrates and aluminates of calcium after few months of mixing. The pH environment present in the system enables further reaction of the silica and alumina with the clay particles and provides extra strength to the stabilized soils and a minimum pH of 12.4 is necessary for pozzolanic reaction (Harty 1970, Eades and Grim 1960).

2.8 RESILIENT MODULUS AND CBR CORRELATIONS

In the recent past, researchers and transportation agencies started considering Resilient Modulus (E) values of materials for pavement design analysis. Material characterization only using CBR value may not be realistic in most of the cases, since it is a static parameter and cannot account for the actual pavement responses under the dynamic loads of moving vehicles. CBR value corresponds to the peak resistance that is developed to a monotonic shear failure. On the other hand, E is determined based on the permanent strains from dynamic load tests, which is only a fraction of the total strain that is induced. It simulates pavement behaviour under repeated loading conditions, which replicate normal traffic loading. Also, it has been shown that the E depends on the applied stress levels. Black (1961) developed a relation between CBR and bearing capacity to estimate the in-situ CBR value from the cohesion, true angle of internal friction and suction of the soil. A method of correlation by considering the confining action of the mould used in laboratory tests was also proposed. Since the CBR was found to be a simple test which could be effectively used for the characterisation of subgrade strength, correlating the CBR with the E was considered necessary. The correlation was developed by Heukelom and Klomp (1962) as Equation 2.3.

$$E\ (\text{MPa}) = 10 \times \text{CBR} \tag{2.3}$$

This equation was derived from the results of wave propagation testing conducted at very low strain levels and dynamic deflection testing. The results were modified for suitable values of Poisson's Ratio and the modulus was correlated to a series of CBR values. The equation was originally developed for a modulus range of 2-200 MPa.

A study by Brown (1966) stated that the present design methods based on use of the CBR test were abandoned in California, and advocated the study of soil mechanics to understand the response of soils and granular materials to repeated loading. This study also highlighted the need for incorporating the non-linear stress-strain characteristics in design and evaluation. Most of the equations that followed were based on the results of the Heukelom and Klomp tests. The differences between the equations can be attributed to the degree to which the results were modified to account for the fact that the wave propagation was done at very low strain levels. A similar equation was developed by the US Army Corps (Green and Hall 1975).

$$E\ (\text{MPa}) = 37.3 \times \text{CBR}^{0.71} \tag{2.4}$$

The South African Council on Scientific and Industrial Research (CSIR) adopted modified equations of the form E = k × CBR, where k is the factor that accounts for local factors and additional laboratory testing (Paterson et al, 1978). Despite of many other equations that were formulated thereafter, the Heukelom and Klomp equation continued to be the preferred relationship. However, many authors, including Powell et al. (1984) have noted that when the wave propagation data is compared with repeated load tests at more realistic stress levels, the observed values of E were found to be significantly lower than ones predicted by the Heukelom and Klomp equation. Authors also presented another equation, which was adopted by the Transport and Road Research Laboratory (TRRL), Crowthorne, UK.

$$E\ (\text{MPa}) = 17.6 \times CBR^{0.64} \tag{2.5}$$

Angell (1988) and some other researchers were of the opinion that, the Heukelom and Klomp equation underestimates the modulus for CBR values less than 5, and overestimates the same for CBR values above 5. This ideology led to the formulation of correlations that consisted of different equations for different ranges of CBR values. The Main Roads Department, Queensland adopted the following relationship:

$$E\ (\text{MPa}) = 21.2 \times CBR^{0.64}\ (CBR < 15), \text{ and}$$
$$E\ (\text{MPa}) = 19 \times CBR^{0.68}\ (CBR > 15) \tag{2.6}$$

Due to difficulties associated with cyclic testing used to characterize the soil subgrade, the other approximate methods for estimating E values are often based on shear strength measures only, and do not consider the magnitude of cyclic deviator stress. A procedure to relate the soil-index properties and the moduli obtained from UCS test, to E has been described by Drumm et al. (1990). Researchers observed that while the resilient behavior of granular materials, defined by E and Poisson’s ratio, is affected by factors such as stress level, density, grading, fines content, maximum grain size, aggregate type, particle shape, moisture content, stress history and number of load applications, the resilient response is mostly influenced by the applied stresses and moisture content of the material. Frederick et al. (2000) characterized the stress-strain relationships by a stress-dependent E and a constant or stress-dependent

Poisson's ratio as well as by decomposing both stresses and strains into volumetric and shear components.

The Indian Roads Congress (IRC, 2001) adopted a relationship that was a direct combination of the Heukelom and Klomp equation and the TRL equation:

$$E\ (MPa) = 10 \times CBR\ (CBR < 5), \text{ and } E\ (MPa) = 17.6 \times CBR^{0.64}\ (CBR > 5) \qquad (2.7)$$

Sukumaran et al. (2002) studied the suitability of some of the aforementioned equations for soils with CBR ranging from 11 to 40, and also used a finite element analysis to determine a correlation between CBR and E. A suitable correlation could not be developed using the Heukelom and Klomp equation and a more accurate estimation of the E was done using correlations with the UCS. Rosyidi et al. (2006) conducted a study using the spectral analysis of surface wave and correlations were formulated between CBR and the dynamic shear moduli of the samples, which showed good agreements with field observations. Erlingsson (2007) analysed 20 samples with a CBR range of 40-140 and concluded that the CSIR relationship is the best in terms of prediction of subgrade modulus and was observed to be more conservative than the Heukelom and Klomp equation. Usluogullari et al. (2008) also carried out finite element studies to predict CBR values and found the model to give reasonable predictions when compared to experimental values of CBR. Austroads (2009) compared various E and CBR correlations developed and used in various countries and suggested that the main limiting factors of using these relationships are dependency of subgrade modulus on subgrade stress, variability of the subgrade modulus relationship with material type and dependency of the subgrade strain relationship on subgrade modulus. Ekwulo et al. (2009) studied the suitability of three known CBR-dependent methods for pavement design using a layered elastic analysis and concluded that flexible pavements designed using these methods are susceptible to rutting failure and hence recommended the use of mechanistic design procedures. Anochie-Boateng et al. (2010) examined the validity of the Heukelom and Klomp and the TRRL relationships for 14 subgrade soils and found that the E value was either overestimated by 40% or underestimated by 100% or more. The authors also tried to fit the E and CBR values into a variety of mathematical forms, including exponential, linear and logarithmic functions, but could not find any suitable relation (based on statistical analysis). Putri et al. (2010) compared E values computed from CBR and Unconfined Cyclic Triaxial (UCT) tests and found that values from the former method were much higher.

With a realisation of the misconceptions in the E and CBR correlations, attempts have been made to correlate the subgrade modulus with other soil parameters. Sukumaran et al. (2002) noted that both the sub grade modulus and CBR could be correlated with the undrained shear strength (s_u) using the following Equations.

$$CBR = 0.62s_u \text{ (psi)} \tag{2.8}$$

$$E \text{ (psi)} = 100 - 500s_u \text{ (PI>30), and } E \text{ (psi)} = 500 - 1500s_u \text{ (PI<30)} \tag{2.9}$$

It was also noted that on combining Equations (2.8) and (2.9), the correlation between E and CBR could range from E (psi) = 160 CBR to E (psi) = 2420 CBR, which is a very vast range for correlation. The conclusion was that the best correlation for subgrade modulus could be done with the UCS test (Q_u).

$$E \text{ (ksi)} = 0.307Q_u \text{ (psi)} + 0.86 \tag{2.10}$$

Al-Amoudi et al. (2002) conducted laboratory and in-situ tests to correlate the CBR value to the Clegg Impact Hammer values, and found suitable correlations. Rao et al. (2008) carried out a similar study to develop a model for the prediction of CBR from the modulus calculated from Portable Falling Weight Deflectometer tests. Patel et al. (2010) correlated CBR with soil index properties, including LL, PL, PI, SL, OMC and MDD. But these types of correlations defeat the very purpose of getting accurate estimates of the subgrade modulus, since the modulus will then have to be calculated from estimated values of CBR, which can only worsen the scenario.

2.9 PAVEMENT PERFORMANCE

Pavement design methods are generally categorized into two, Empirical methods and Mechanistic-Empirical methods. Empirical methods are established on experience gained in practice and from observation of the performance of existing or specially constructed pavements under different traffic conditions. Hveem and associates developed the first empirical methods using CBR method during 1930's. In 1972, the American Association of State Highway Officials (AASHO) developed an empirical pavement design guide based on an equation (prediction model) with coefficients that were statistically obtained from the AASHO test road. The main drawback of empirical methods were, they are restricted to a particular extent of pavement and traffic loads only, and they are insufficient to account a new material or different traffic loads outside the range considered (Lav et al. 2006). This leads to the development of mechanistic empirical methods for pavement design. In this method, the pavement structure and load configuration are assumed. Generally the pavement structure is simplified to three distinct layers (Dormon and Edwards 1968).

Yoder and Witczak (1975) define two types of pavement distresses, structural and functional failures. In structural failure a collapse of the entire structure or a breakdown of one or more pavement components makes the pavement incapable to sustain the loads imposed on the surface. Functional failure occurs when the pavement is unable to execute its purpose without causing discomfort to drivers or passengers or imposing high stresses on vehicles. These failures may be due to inadequate maintenance, excessive loads, climatic and environmental conditions, poor drainage leading to poor subgrade conditions, disintegration of the component materials, surface fatigue and excessive settlement, volume change of subgrade soils due to wetting-drying and freezing-thawing, etc.

Pavement analysis is generally conducted to determine the responses, including stresses strains and displacements, in a pavement structure during the application of a wheel load. The horizontal tensile strains developed at the bottom of the surface layer, which control the fatigue cracking, and the vertical compressive strains, developed at the top of the subgrade, which control the permanent deformation, are considered as the critical strains in any pavement structure. If the design life is less than the governing failure criterion in terms of number of standard axles, then the related pavement configuration is considered as satisfactory and acceptable as a valid design. Otherwise, layer thickness and/or material properties are adjusted to reach an acceptable configuration (Lav et al. 2006).

Many researchers used a software package named KENPAVE, which has a tool called KENLAYER exclusively for flexible pavement, working based on Burmister's layer elastic theory (Lav et al. 2006, Erlingsson and Ahmed 2013). The design lives obtained from CIRCLY and KENLAYER analyses were similar for a pavement structure with cement and flyash stabilized subbase materials. Gedafa (2006) used KENLAYER and Highway Development and Management (HDM 4) softwares to compare the performance of stabilized soil for flexible pavement. It was observed that the life of pavement predicted by HDM 4 is less than that predicted by KENLAYER. Ziari and Khabiri (2007) used the KENLAYER program to compute the stresses and strains in typical flexible pavements considering the pavement layers are either completely bonded or completely unbounded. The different CBR subgrade strength was stabilized with lime by Selvi (2015) and analysis was carried out using KENPAVE software for different traffic conditions. The tensile strain at the bottom of bituminous layer and compressive strain on lime stabilized subgrades values were within the permissible limits and also the design catalogues were developed for different subgrade strengths.

A computer program called IITPAVE was developed by IIT Kharagpur, India to perform stress strain analysis for flexible pavements. Das and Pandey (1999) used this program to design thicknesses of asphalt concrete surfacing with different grades of bituminous binders for various thicknesses of granular layers and different subgrade moduli values. The strains at the critical points were obtained and compared with the allowable strains calculated using the fatigue and rutting criteria, and thickness charts were prepared for 5 to 50 msa traffic. A few researchers have worked on IITPAVE for the computation of stresses and strains in flexible pavements (Garg 2014, Praveen et al. 2014, Ravinder and Sachdeva 2014, Bagui 2014). IRC also recommends this software for the design of flexible pavement sections (IRC: 37: 2012). Lakshmanan et al. (2014) obtained soaked CBR of 11% for the combination of 30% fly ash +10% Copper slag + 60% soil as subgrade layer. IITPAVE has been used for the computation of stresses and strains in flexible pavements with this material.

2.10 SUMMARY

From the above literature review it is observed that most of the soils have been stabilized either by mechanical or chemical methods. Depending on the strength requirement and the type of stabilizer, appropriate methods were selected. Even the locally available soil, industrial wastes etc. were also utilized. In India, no study has been reported on the usage of Arecanut coir fibre as a soil stabilizer, even though it is available abundantly in many parts of the country. Also only a few studies have been reported on the usage of nontraditional stabilizers and RBI 81 in soil. Durability also has a major role in soil stabilization for pavement construction, and researchers tried different methods to assess the same. Analysis and design of pavement sections by determining the responses on different layers were generally carried out using various software programs. The efforts are made to use different types of stabilizers with LS and BC soil, and to evaluate their engineering properties after stabilization.

CHAPTER 3

MATERIALS AND METHODS

3.1 MATERIALS USED

The main materials used in this study are Lateritic soil, Black Cotton soil and various stabilizers. The following sections provide brief information pertaining to the materials selected for soil stabilization.

3.1.1 Lateritic Soil (LS)

The term Laterite first appeared in scientific literature over about 200 years ago. The word Laterite was first introduced by Buchanan (1807) to denote a building material used in the mountainous regions of Malabar, India. Lateritic soils have been found in this region because of high rainfall, high temperature and high humidity with alternate wet and dry period, which is an ideal condition for laterization. Nearly 60-80 per cent of the soils are laterites in the country. The colour ranges from red to yellowish red and the depth varies from 100 to 500 cm for this soil. On exposure to air, it rapidly hardens and becomes highly resistant to weathering, and hence it is frequently used as a building material comparable to bricks. The laterites have been mostly originated from igneous rocks and are well drained, residual with the presence of excessive Iron (Fe) and Aluminium (Al). Two types of Lateritic Soils, LS1 and LS2, were collected from two locations (Puttur and NITK campus) in Dakshina Kannada (DK) district, Karnataka, India.

3.1.2 Black Cotton Soil (BC Soil)

In North Karnataka region the land under cultivation is mostly of BC soil. In general BC soils are one of the worst foundation soils, characterized by heaving and cracking of pavements, foundations, channel as well as reservoir linings. The effect of swelling and shrinkage of these soils are seen to be more critical when extreme environmental conditions prevail. The BC soils are also characterized by their poor strength in tension and shear in wet conditions with large settlement. This soil covers considerably large area nearly one third of Indian land. These areas being extensively developed in the fields of transportation, irrigation, industrialization, etc., any method to improve the construction techniques in the soils is considered to be of vital importance from the view of economy in developing India. In a broad sense the engineering problem in BC soils is to identify the presence of the expansive deposits and try to stabilize the soil using better materials or techniques. BC soil for the present study was collected from the Naragund Taluk of Gadag district, Karnataka, India.

3.2 STABILIZERS

3.2.1 Terrasil

Terrasil is a nanotechnology based 100 per cent organo silane, water soluble, ultraviolet and heat stable, reactive soil modifier to waterproof soil subgrade. It reacts with water loving silanol groups of sand, silt, clay and aggregates to convert it to highly stable water repellent alkyl siloxane bonds and forms a breathable in-situ membrane. It resolves the critical sub-surface issues. The soil structures at untreated and treated conditions are presented in Figures 3.1 and 3.2. Physical properties of Terrasil are tabulated in Table 3.1.

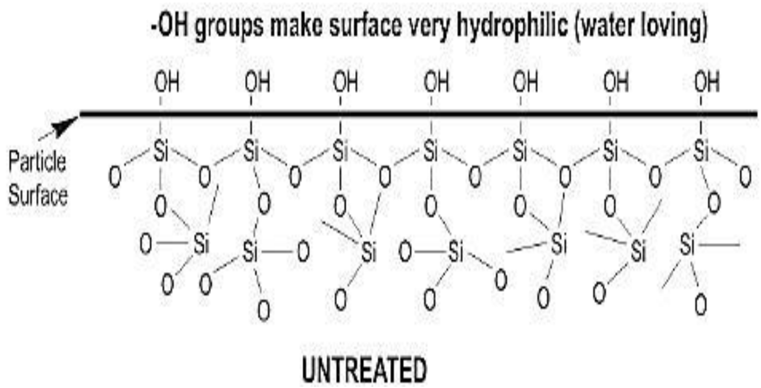


Fig. 3.1 Untreated soil surface silicate structure

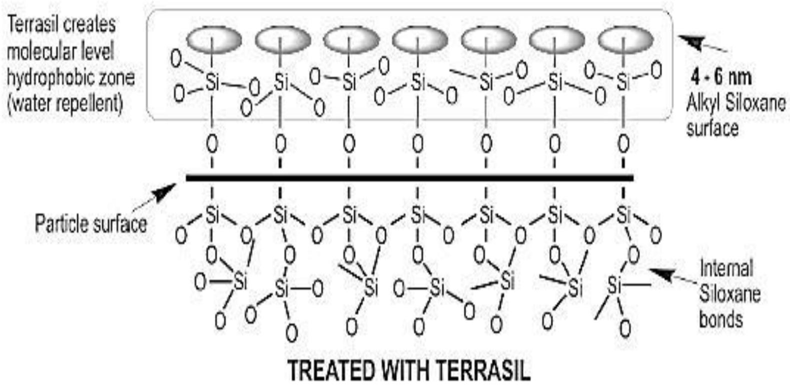


Fig. 3.2 Treated soil surface silicate structure

Table 3.1 Technical specifications of Terrasil

Property	Description
Appearance	Pale yellow liquid
Solid content	68±2%
Viscosity at 25ºC	20-100cps
Specific gravity	1.01
Solubility	Forms water clear solution
Flash Point	Flammable 12ºC
Terrasil : water	1 kg : 400 liters
Dosage	0.8, 1.2 and 1.6% by weight of dry Soil

3.2.2 Terrabind

Terrabind is a revolutionary advanced lignosulphonate liquid ionic organic compound for the purpose of soil stabilization. Terra Nova Technologies is the first company to manufacture this technology in India. Terrabind alters the properties of road base materials (soil/aggregate) at a molecular level thus rendering greater compaction, load bearing and cohesiveness. Organic lignins allow for polymeric binding between soil grains. Electrolyte emulsions attack the clay lattice of soil by altering the ionic charge in clay and breaking down the capillary action of clay soil particles and this makes the soil particles to attract and retain moisture. It is available in the liquid concentration and is to be diluted in water in specified proportion before mixing with the soil. Physical properties of Terrabind are tabulated in Table 3.2.

Table 3.2 Physical properties of Terrabind

Properties	Description
Form	Liquid
Odor	Sharp, sulphurous
Color	Dark Amber
Wetting Ability	Excellent
Boiling Point	182°C
Solubility in Water	Complete
Specific Gravity	1.7
pH	1
Weight per gallon	14.19 lb

3.2.3 Fly Ash (FA)

Fly ash is a marginal material, which is easily available at all thermal power stations, and can be utilized for road construction. It has an additional advantage of environmental friendliness by means of minimizing the disposal problems. FA used in this study was obtained from Thermal power station, Raichur, Karnataka, India. Physical properties of FA are tabulated in Table 3.3.

Table 3.3 Properties of Fly Ash

Properties	Test values
Type	Class F
Specific gravity	1.975
Water content (%)	0.16
Loss on Ignition (%)	0.43
Size	< 45 µ
pH	8.12

3.2.4 Cement

Cement is a binder, a substance that sets and hardens on drying and also reacts with carbon dioxide in the air dependently, and can bind other materials together. Basic raw materials used in the manufacture of cement are calcium carbonate found in lime Stone or chalk, and silica, alumina and iron oxide found in clay or shale. The cement used in the present investigation is OPC 43 grade and the basic properties are tabulated in Table 3.4.

Table 3.4 Physical properties of Cement

Sl. No	Test Conducted	Results Obtained	Requirements as per IS 8112-1989
1	Specific gravity	3.16	--
2	Normal consistency	32%	--
3	Setting time, (minutes)	Initial 60	Not > 30 min
		Final 225	Not > 600 min
4	Fineness (m^2 / kg)	330	Not < 300 m^2/kg
5	Soundness (mm) –Le Chatelier test	2.50 (Expansion)	Not > 10 mm

3.2.5 Road Building International grade 81 (RBI 81)

A Proprietary Cementitious Stabilizer named RBI 81 was used to enhance the strength of soil. It is a powder additive which acts on soil to reduce voids between soil particles and to minimize absorbed water in the soil to achieve maximum compaction. The properties of RBI 81 are tabulated in Table 3.5.

Table 3.5 Properties of RBI 81

Property	Description
Odour	Odourless
pH	12.5
Specific Gravity	2.5
Solubility	In water
Flammability	Nonflammable
Shelf Life	12 months (dry storage)
Bulk density	700/m^3

3.2.6 Aggregate

Aggregate is a collective term for the mineral materials such as sand, gravel, and crushed stone that are used with a binding medium (such as water, bitumen, Portland cement, lime, etc.) to form compound materials (such as bituminous concrete and Portland cement concrete). Aggregate is also used for base and sub-base courses for both flexible and rigid pavements. The crushed granite aggregates 12.5mm down size, with gradation as listed in Table 3.6, were used for this investigation.

Table 3.6 Sieve analysis of Aggregates

IS Sieve size (mm)	Percentage passing
12.5	100
10	91
4.75	21
2.36	5

3.2.7 Arecanut Coir

Arecanut coir was collected from Puttur, DK district, Karnataka state, India. The dry Arecanut shells, which are brown in colour, were collected for the present work and the coir from the shell was extracted manually in the laboratory. The physical and chemical compositions of Arecanut coir are tabulated in Tables 3.7 and 3.8. The aspect ratio and specific gravity of Arecanut coir considered for the study are 80 and 0.67 respectively. Figure 3.3 shows the physical appearance of Arecanut coir.

Table 3.7 Physical properties of Arecanut Coir

Diameter (mm)	Length (mm)	Density (g/cc)	Young Modulus (kN/mm^2)	Tensile Strength (kN/m^2)
0.35	28	1.09	27	2.2

Table 3.8 Chemical composition of Arecanut Coir

Cellulose (%)	Hemicellulose (%)	Lignin (%)	Ash (%)	Pectin (%)	Wax (%)
Nil	35-64.8	13-24.8	4.4	Nil	Nil

Fig. 3.3 Arecanut coir

3.3 METHODS USED

The basic tests for Grain Size distribution (IS:2720, Part-IV), Specific Gravity (IS:2720, Part-III), Atterbergs limits (IS:2720, Part-V), Compaction characteristics (IS:2720, Part-VII, VIII), UCS (IS:2720, Part-X), CBR (IS:2720 Part-XVI), Permeability Test (IS: 2720 (part XVII), Durability (ASTM D 559, 560), Tri axial test (AASHTO T 307-99), IDT strength (ASTM D 6931) Fatigue behaviour, etc. were performed.

3.3.1 Free Swell Index test (FSI)

A set up for FSI test is presented in Figure 3.4. According to IS: 2720, Part-XI, 10gram oven dry soil passing through 425µ IS Sieve is taken. Soil sample is poured in two glass graduated cylinders of 100 mL capacity. One cylinder is filled with kerosene and the other with distilled water up to the 100 mL mark. After removal of entrapped air by gentle shaking or stirring with a glass rod, the soil in both the cylinders is allowed to settle for sufficient time (not less than 24 h). The soil samples are allowed to attain an equilibrium state of volume without any further change in the volume of the soil. The final volume of soil in each of the cylinders shall be read out.

The level of the soil in the kerosene graduated cylinder is read as the original volume of the soil samples, since kerosene being a non-polar liquid, it does not cause swelling of the soil. The level of the soil in the distilled water cylinder shall be read as the free swell level and the free swell index of the soil is calculated using Equation 3.1

$$FSI\ (\%) = \left(\frac{Vd-Vk}{Vk}\right) \times 100 \qquad (3.1)$$

Where,

V_d= volume of soil specimen read from the graduated cylinder containing distilled water.

V_k= volume of soil specimen read from the graduated cylinder containing kerosene.

Fig. 3.4 Set up for FSI

3.3.2 Durability Test

Durability is defined as the ability of a material to retain stability and integrity over years of exposure to the destructive forces of weathering and hence it is one of the most important factors for any stabilized soil (Dempsey 1968). A good stabilizer should help, not only in gaining the strength, but also to retain its bonding with soil during the seasonal changes. Hence, checking durability is vital before recommending any stabilizer for practical applications. There are mainly two tests for durability – Wet Dry (WD) and Freeze Thaw (FT). For the present study, the procedures as per ASTM D559 and 560 were adopted. Soil specimens with 76mm height and 38mm diameter were prepared and then they were subjected to 7 days moist curing. The test contains 12 cycles of each WD and FT. In wet cycle, specimens were submerged in water at room temperature for 5 hours, then its dimensions and weight were taken, and in dry cycle, the specimens were dried at a temperature of 71°C for 42 hours. Then specimens were thoroughly brushed parallel and again dimensions and weight were taken. This procedure is repeated for 12 cycles. In Freeze cycle, samples were placed in water-saturated felt pads and stood on carriers in a freezer at a temperature not higher than -10°C for 22 hours. Thawing was done by keeping them in a moisture room for 22 hours and dimensions and weight were taken after brushing. The weight loss of specimen for WD and FT should not be more than 14% after 12 cycles.

3.3.3 Tri axial Test

Resilient Modulus (E) was determined as per AASHTO-T307 (2007) specification in an HS 28.610 cyclic triaxial testing system, depicted in Figure 3.5. The machine was fitted with a closed loop servo-controlled hydraulic loading system to apply dynamic loads through a loading piston to the specimen kept in a chamber. During the test, water was filled in the chamber and pressure was applied to the specimen. A digital controller was used to control the machine and a data acquisition system with computer to collect and store the data. The load applied to the specimen was measured by means of a load cell placed on the loading piston and the vertical deformations using two vertical linear variable differential transducers (LVDTs) kept over the specimen. Different deviator stresses and confining pressures were applied to determine the E values. The deviator stress was applied in half sinusoidal wave form with 0.1 s loading period and 0.9 s rest period in each pulse. To provide seating load, 500 cycles of repeated cyclic loading was applied and then 100 cycles for each deviator stress with different confining pressure values. The resilient strain and E values were determined using the recovered deformation under deviator stress for all cycles of each loading stage. The resilient modulus is determined from a specific type of cyclic triaxial test, and is determined from Equation (3.2).

$$E = \frac{\Delta\sigma a}{\Delta\varepsilon a} \tag{3.2}$$

Where, $\Delta\sigma a$ is the amplitude of the repeated axial stress and $\Delta\varepsilon a$ is the amplitude of the resultant recoverable axial strain.

Fig. 3.5 Tri axial set up

3.4 SELECTION OF STABILIZERS

Selecting the stabilizer type depends on number of factors including:

- Gradation of soil
- Plasticity index (PI) of soil
- Availability and cost of the stabilizer and appropriate construction equipment
- Its long term effect on strength etc.

3.5 DOSAGE CALCULATION

Dosage rates can be specified in many different ways, but the most common way to define the dosage rate is based on the dry weight of soil to be treated. Manufacturer’s recommendations for the stabilizers used in this research are given as a percentage of the dry weight of the untreated soil. Accordingly, the amount of stabilizer to be used was found from the following method.

Terrabind

Dosage: 1mL of concentrated Terrabind liquid for every 3kg soil

Soil taken: 30kg

Amount of Terrabind for 30 kg soil =30/3=10mL.

MDD of soil: 17.7g/cc, OMC of soil: 16.31%, Natural moisture content: 6%

Water to be added= [(16.31-6) +2%] x 30/100= 3.69 liters of water. (2% extra amount of water is added to compensate the evaporation loss)

Terrasil

Terrasil dosage of one liter per ton weight of soil is recommended by manufacturer. But in this study three dosages, 800mL, 1200mL and 1600mL for 1000kg of soil, are considered. Trials were conducted by treating the soil at 0.8% (dosage-1), 1.2% (dosage-2) and 1.6% (dosage-3) by weight of dry soil and variations in engineering properties were studied.

The stabilizer application to soil was done in two stages. First Terrasil was diluted at 150% of the OMC and added to soil, mixed properly and the mixture was kept for air drying, which made the mixture surface 90-95% water resistant. In the second stage, 1 to 2% of cement by weight of soil was added before compaction to achieve a desired proctor density of 98 to 100%.

Ordinary Portland Cement (OPC)

OPC 43 grade was selected and dosages are considered as 0, 3, 6, 9 and 12% to dry weight of soil.

RBI 81

As per the relevant literature and manufacture suggestions, different percentages as 2, 4, 6 and 8% to dry weight of soil are considered for this study.

Curing

The samples were tested at various curing periods like 0, 7, 14, 28, 60, 90 and 365 days. All the samples prepared were labeled according to the period of curing. The specimens are kept in desiccators maintaining 100% humidity. The cured specimens are tested immediately after designated curing period.

3.6 CHEMICAL ANALYSIS

Chemical analysis of the soils was performed including pH test, electrical conductivity, silica, alumina, etc. Table 3.9 presented the methods and reference in determining various parameters.

Table 3.9 Methodology and methods for chemical analysis

Parameters used	Methods	References
pH meter	Electrometric method (Standard method)	IS: 2720 (Part 26) -1997
Conductivity (mS/cm)	Potentiometric method	IS: 2720 (Part 26) -1997
Silica (SiO_2)	Gravimetric method	IS: 2720 (Part 25) -1982
Iron Oxide (Fe_2O_3)	Gravimetric / Colorimetric method	IS: 2720 (Part 24) -1976
Aluminium Oxide (Al_2O_3)	Gravimetric method	IS: 2720 (Part 24) -1976
Sulphates (SO_4)	Turbidimetric method	IS: 2720 (Part 27) -1977
Calcium Oxide (CaO)	E.D.T.A Titrimetric method	American Public Health association, APHA (2005)
Magnesium Oxide (MgO)	E.D.T.A Titrimetric method	APHA (2005)

3.7 KENPAVE AND IITPAVE ANALYSIS

In this present research work, the critical stress strain analysis has been carried out to predict the performance of flexible pavement. To examine the performance of a flexible pavement system laid over stabilized sub grade, multilayer KENLAYER analysis was carried out to compute stresses and strains. The process of computing the stresses and strains in a multilayer flexible pavement system is highly complex and time consuming even after assuming that all the layers are homogenous, isotropic and continuous. It is with this background that many researchers have developed multilayer analysis algorithms like DAMA, ILLI-PAVE, MICH-PAVE, VESYS, PDMAP, ELSYM-5, BISAR, CHEVE and KENLAYER etc., which are very effective in solving majority of multilayer problems. However, KENLAYER algorithm, developed by Yang (2004) is considered to be the best performed algorithm in many of the reported case studies and accurate enough to give satisfactory stress, strain values. It also offers much flexibility like number of classes of axle loads and number of seasons to be incorporated while inputting the data. Hence, in the present work, the KENLAYER program is used and analyzed. This KENLAYER analysis includes damage analysis and distress models to predict the life of the new pavement. The damage analysis is based on the horizontal tensile strain at the bottom of specified layers, usually the surface layer, and the vertical compressive strain at the top of the sub-grade layer. Instead of reading in the Z coordinates, simply specifying the total Number of Layers for Top Compression (NLTC), the Layer Number for Bottom Tension (LNBT), and the Layer Number for Top Compression (LNTC), the program will determine the Z coordinates of all necessary points and compute the required strains. If several radial coordinate points are specified under single wheel or several x and y coordinate points under multiple wheels, the program will compare the strains at these points and select the most critical ones for damage analysis.

A computer program called FPAVE was developed by IIT Kharagpur, India in 1997 for the computation of stresses in a pavement structure, which was later modified as IITPAVE (Das and Pandey 1999). IRC suggests this program for the design of flexible pavements (IRC 37 2012). Any combination of traffic and pavement layers can be tried using IITPAVE by providing inputs like number of layers, layer thickness, Poisson’s ratio, resilient modulus, tyre pressure and wheel load, similar to KENPAVE and critical strains are obtained as outputs.

3.8 WORK PLAN

The Flow chart for work plan for the investigation is provided in Figure 3.6 below.

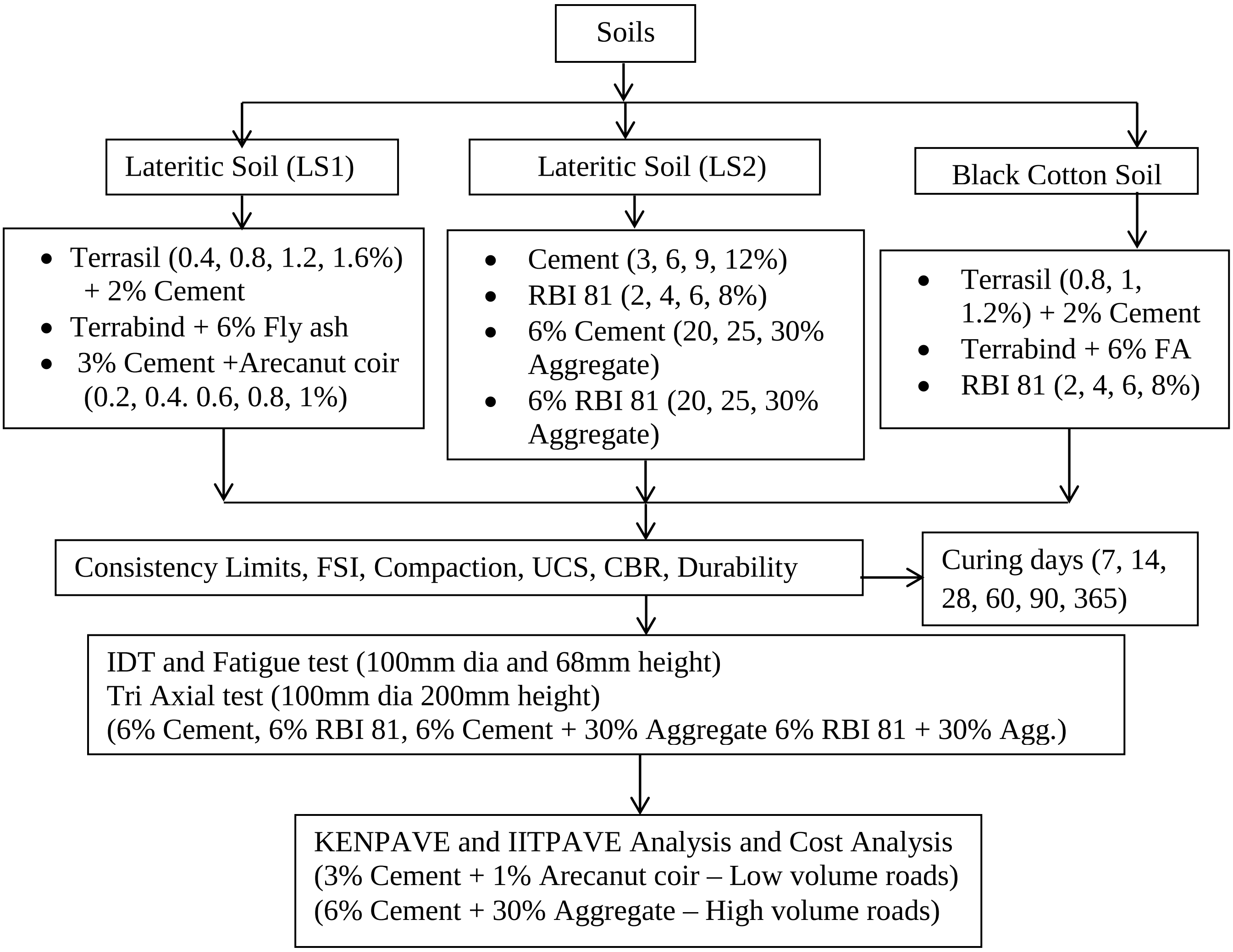


Fig. 3.6 Flow chart for test methodology

CHAPTER 4

LATERITIC SOIL STABILIZATION

4.1 GENERAL

The earliest investigations on soils by Voelcker dates back to 1893 and by Leather to 1898. They had categorized four major soils in India like Gangetic alluvium, Black cotton soil, Red soil and Laterite soil. In Karnataka, Lateritic soils are commonly found along the coastal region. The untreated lateritic soil have presented many problems in road construction and maintenance, but it has also been reported that they can be good materials for sub base and base construction for light and medium trafficked roads, if properly stabilized. This chapter deals with the Lateritic Soils (LS1 and LS2) stabilization with different stabilizers like Terrasil, Terrabind, Fly ash, Cement, RBI Grade 81, Arecanut coir and Aggregate with different dosages and curing days (7, 14, 28, 60, 90 and 365 days). The stabilizing mechanisms of non-traditional stabilizers are not fully inferred, and their proprietary chemical composition makes very difficult to measure the mechanisms and to predict their performance. Engineering properties of stabilized soil are described in the following section. The laboratory tests were conducted on two types of soils, LS1 and LS2, for the properties like specific gravity, grain size distribution, consistency limits, compaction characteristics, UCS, CBR and co-efficient of permeability, a summary of which is presented in Table 4.1. The soils are classified as Silty clayey (SC) and Silty Sand (SM) as per the Indian Standards (IS) procedure respectively.

Table 4.1 Basic properties of Lateritic Soils

Sl no.	Property	LS1	LS2
1	Specific gravity	2.45	2.65
2	Grain size distribution (%)		
	a) Gravel	9	28
	b) Sand	44	47
	c) Silt	15	20
	d) Clay	32	05
3	Consistency limits (%)		
	Liquid Limit (LL)	56	39
	Plastic Limit (PL)	29	27
	Plasticity Index (PI)	27	12
4	IS Soil Classification	SC	SM
5	Engineering Properties		
	IS Standard Compaction		
	a) MDD, γdmax (g/cc)	1.68	1.98
	b) OMC (%)	19.2	12.33
	IS modified Compaction		
	a) MDD, γd max (g/cc)	1.91	2.04
	b) OMC (%)	14.0	10.38
6	CBR Value (%)		
	IS Standard Compaction		
	a) Unsoaked condition	15	32
	b) Soaked condition	2	6
	IS Modified Compaction		
	a) Unsoaked condition	13	28
	b) Soaked condition	3	8
7	UCS (kPa)		
	Standard Compaction	138	198
	Modified Compaction	206	342
8	Co-efficient of permeability		
	Standard Compaction (cm/sec)	0.35×10^{-7}	1.10×10^{-7}
	Modified Compaction (cm/sec)	0.20×10^{-7}	0.93×10^{-7}

4.2 LS1 STABILIZATION WITH TERRASIL

4.2.1 Stabilizer

Terrasil, a water soluble compound, dissolves in water to form a water clear solution. It forms a permanent water repellent layer on all types of soils, aggregates and other inorganic road construction materials. The reaction leads to permanent siliconization of the surfaces by converting the water loving silanol groups to water repellent siloxane bonds. It helps in substantial reduction in soil water infiltration and erosion.

4.2.2 Chemical Dosage

To assess the suitability of Terrasil as a soil stabilizer, both natural soil and chemically stabilized soil were tested for engineering properties and strength parameters. The CBR and UCS were determined for different curing periods. The chemical is diluted in water at 1:100 concentrations and then mixed with soil in different dosages (dosages 1 to 4). Constant 2% cement was used for all the dosages, to enhance the bonding between soil particles and also to ensure additional strength. The calculations are tabulated in Tables 4.2 and 4.3 for both standard and modified proctor compaction. A detailed caluculation for dosages is provided in Appendix I.

Table 4.2 Dosage calculations for standard compaction

Dosages					
Quantity of Chemical	0	1 (2% of weight of water)	2 (4% of weight of water)	3 (6% of weight of water)	4 (8% of weight of water)
MDD	1.68g/cc	1.71g/cc	1.74g/cc	1.85g/cc	1.90g/cc
OMC	19.20%	17.50%	16.50%	15.00%	13.50%
Vol. of soil	85.05cm^3	85.05cm^3	85.05cm^3	85.05cm^3	85.05cm^3
Wt. of soil	142.88g	145.44g	147.99g	157.34g	161.60g
Vol. of water	27.43mL	25.45mL	24.42mL	23.60mL	21.82mL
Vol. of chemical	-	0.51mL	0.98mL	1.42mL	1.75mL

Table 4.3 Dosage calculations for modified compaction

Dosages					
Quantity of Chemical	0	1 (2% of weight of water)	2 (4% of weight of water)	3 (6% of weight of water)	4 (8% of weight of water)
MDD	1.91g/cc	1.98g/cc	1.99g/cc	2.01g/cc	2.02g/cc
OMC	14.00%	13.00%	12.50%	12.00%	11.00%
Vol. of soil	85.05cm^3	85.05cm^3	85.05cm^3	85.05cm^3	85.05cm^3
Wt. of soil	162.45g	168.40g	169.25g	170.95g	171.80g
Vol. of water	22.74mL	21.89mL	21.16mL	20.51mL	18.90mL
Vol. of chemical	-	0.44mL	0.85mL	1.23mL	1.51mL

4.2.3 Effect on Consistency Limits

As the percentage of chemical increases, there is an improvement in Consistency limits of soil as listed in Table 4.4. For pavement construction, soil with lesser LL and PI values are considered due to its good characteristics. For untreated soil, LL, PL and PI were 56, 29 and 27 % respectively and for further addition of chemical, all these values were found to be decreasing. Test was conducted immediately after mixing soil with chemical. This is due to the chemical reaction, causing substantial reduction in soil water infiltration and chemically treated soils do not allow absorption of water, resulting in reduced plasticity.

Table 4.4 Consistency limits of treated and untreated soil

Dosage	LL (%)	PL (%)	PI (%)
0	56	29	27
1	44	32	12
2	43	29	14
3	41	27	14
4	38	27	11

4.2.4 Effect on Compaction

The effects of chemical dosage on MDD and OMC of soil for standard and modified compaction immediately after mixing are presented in Table 4.5. As chemical dosage increases, the MDD increases and OMC decreases for both standard and modified compactions. While adding the chemical to the soil, it reacts with the soil particles and makes the surfaces water proof permanently and stiffens the soil.

Table 4.5 Compaction values for treated soil

Dosage	Standard Compaction		Modified Compaction	
	MDD (g/cc)	OMC (%)	MDD (g/cc)	OMC (%)
0	1.68	18.0	1.91	14.0
1	1.71	17.5	1.98	13.0
2	1.74	16.5	1.99	12.5
3	1.85	15.0	2.01	12.0
4	1.90	13.5	2.02	11.0

4.2.5 Effect on Permeability

Permeability tests were carried out on soil with different chemical dosages after seven days curing and the test results are tabulated in Table 4.6. The test results indicate that, as the dosage increases from zero to dosage 4, there is a considerable decrease in permeability. The chemical reaction leads to permanent siliconization of the surfaces by converting the water loving silanol groups to water repellent siloxane bonds. But the test results indicate that there is not much variation in the co-efficient of permeability beyond chemical dosage 2.

Table 4.6 Permeability values for seven days treated soil

Dosage	Co-efficient of Permeability (cm/sec)
0	2.31×10^{-04}
1	2.18×10^{-04}
2	2.06×10^{-04}
3	1.98×10^{-04}
4	1.95×10^{-04}

4.2.6 Chemical Composition of Treated Soil

Chemical composition was analyzed as per the standard procedures. From Table 4.7, it can be observed that silica oxide, aluminium oxide and calcium oxide percentages were increased after stabilizing with Terrasil. It indicates that, silica, alumina and Cao are the major components to increase the strength.

Table 4.7 Chemical composition of Terrasil treated soil

Oxides (%)	SiO_2	Fe_2O_3	Al_2O_3	S	CaO	MgO	pH	EC (µS/cm)
LS1	55.36	6.24	4.98	0.042	0.019	0.003	5.91	1.27
LS1+ Terrasil	58.26	5.06	7.21	0.19	2.45	2.94	8.10	4.28

LS1: Lateritic soil; SiO_2: Silica oxide; Fe_2O_3: Iron oxide; Al_2O_3: Aluminium oxide; CaO: Calcium oxide; MgO: Magnesium oxide; S: Sulphate; EC: Electrical conductivity

4.2.7 Effect on UCS

The samples are cured for different curing days for the chemically treated soil and the results are tabulated in Table 4.8. As the dosage increases, UCS increases up to certain level and beyond that UCS marginally decreases. The UCS values increases more rapidly within the early 28 days curing period and beyond that, only marginal improvement in strength was observed. It means that reactions happened till 28 days; therefore, no major improvement occurred in the UCS after 28 days curing period. The dosage 2 is found to be optimum for all the cases, and for further increase in dosages, the strength either decreases marginally or remains same. This reduction occurs due to the nature of water based stabilizer. Adding excess amount of it to the soil, increases the moisture content to a higher value than the optimum amount. The excess moisture content decreases the compressibility of the soil particles and increases the water filled pores inside the soil. These factors decrease the strength of the soil (Scholen 1995; Katz et al. 2001; Rauch et al. 2002; Tingle and Santoni 2003).

Table 4.8 UCS values of standard and modified compaction for treated soil

Curing days	UCS in kPa									
	Standard Compaction					Modified Compaction				
	LS1	Dosage				LS1	Dosage			
		1	2	3	4		1	2	3	4
0	138	154	245	163	159	206	232	284	264	240
7	142	258	398	314	302	215	304	446	365	358
14	146	474	498	424	415	212	414	606	518	490
28	146	548	636	610	602	209	709	788	762	733
60	145	547	634	608	599	207	702	789	763	735
90	148	548	630	610	600	208	704	790	764	736
365	150	555	628	610	600	208	705	790	767	736

4.2.8 Effect on CBR

CBR tests were conducted for both unsoaked and soaked condition with different curing periods. Since maximum UCS value was obtained for treated soil with dosage 2, the CBR test was carried out for the same dosage. The results are tabulated in Table 4.9. The unsoaked and soaked CBR values of untreated soils are 13% and 3% respectively. There is a considerable increase in the load bearing capacity of the soil with the increase in curing period. The soaked CBR value is found to be 16% after 28 days of curing.

Table 4.9 CBR test results for optimum dosage 2

Curing days	Unsoaked CBR (%)	Soaked CBR (%)
0	20	4
7	43	6
14	61	12
28	73	16
60	89	17

4.2.9 Major Findings

Based on the tests conducted the following findings have been drawn.

- The consistency tests conducted on chemically treated soil shows that the PI decreases as the chemical dosage increases.
- The standard and modified compaction tests conducted on treated soil indicate that as the dosage increases the MDD increases and OMC decreases, because the stabilizer neutralizes the clay particles and ignores the water molecules from clay particles.
- At optimum chemical percentage (dosage 2) the UCS value after 28 days curing period was found to be 636kPa and 788kPa for standard and modified compactions respectively, whereas for untreated soil, the values were 125kPa and 168kPa. The UCS value increases gradually up to 28 days of curing and beyond this there is a marginal increase.
- The soaked CBR values improved by 5 times with Terrasil treatment after 28 days curing.

4.3 LS1 STABILIZATION WITH TERRABIND AND FLY ASH

Terrabind is a new lignosulphonate liquid ionic organic compound suitable for soil stabilization. This chemical is available in liquid form and is to be diluted in water at specified proportion before mixing with soil. Since the LS1 has low strength in terms of CBR, it alone cannot be used in pavement sub surface layers. The Terrabind has a catalyst effect when combined in clay soil with 5-10% FA by weight of soil. A constant 6% of FA was used for further improvement of soil, which acts as a cementitious stabilizer for soil improvement and is highly recommended to use in combination with Terrabind for a higher strength sub grade. FA is a marginal material and is easily available; hence its use has an additional advantage of environmental friendliness by means of reducing its disposal in landfills.

4.3.1 Chemical Dosage

Dosage of 1mL of concentrated Terrabind liquid for 3kg of soil was considered as per the manufacture's suggestions. The amount of stabilizer to be used was calculated from the following method.

For Modified compaction, OMC = 14% and MDD = 1.91g/cc

Natural Moisture Content (NMC) = 5%

Remaining moisture content = 14% – 5% + 2% =11% (2% extra water was added to compensate evaporation loss).

Amount of water required = [(OMC – NMC) + 2] × 3kg = [(14 – 5) + 2] × (3/100)

= 0.33 liters of water

4.3.2 Effect on Consistency Limits

The treated and untreated soil samples were tested for consistency limits within 30min after mixing. From the test results tabulated in Table 4.10, it can be seen that, engineering properties are slightly enhanced by stabilizing Terrabind and FA alone, whereas better improvement has occurred when these stabilizers were used in combination. The PI values were decreased, due to the densification of soil. Terrabind attacks the clay lattice of the soil which alters the ionic charge in clay and creates a chemical bond between the clay particles.

Table 4.10 Consistency limits for treated and untreated soil

Dosage	LL (%)	PL (%)	PI (%)
LS1	56	29	27
LS1 + Terrabind	49	30	19
LS1 + 6% FA	50	28	22
LS1 + Terrabind + 6% FA	53	43	10

LS1: Lateritic Soil 1; FA: Fly Ash

4.3.3 Effect on Compaction

The compaction test was conducted within half an hour after the addition of Terrabind and 6% FA, and slight increase was observed in the MDD of treated soil. Results are tabulated in Table 4.11. During compaction, the finer portion of FA may be squeezed into the voids of soil particles, thus resulting in an increase in MDD. Marginal decrease was observed in OMC, which can be attributed to the progressive hydration process of the FA that consumed some amount of water inside the voids.

Table 4.11 Compaction values for treated and untreated soil

Dosage	Standard Compaction		Modified Compaction	
	MDD (g/cc)	OMC (%)	MDD (g/cc)	OMC (%)
LS1	1.68	18.00	1.91	14.00
LS1 + T	1.72	18.06	1.92	13.80
LS1 + 6% FA	1.75	17.90	1.89	12.34
LS1 + T + 6% FA	1.89	17.80	2.01	11.72

LS1: Lateritic Soil 1; T: Terrabind; FA: Fly Ash

4.3.4 Chemical Composition of Treated Soil

Variation in the chemical composition of treated and untreated soils can be observed from Table 4.12. For treated soil, the concentrations of oxides such as SiO_2, Al_2O_3, CaO, etc. increase whereas that of Fe_2O_3 decreases. The addition of stabilizer results in the formation of various chemicals which bind the soil particles together, creating a crystalline matrix. The presence of FA increases the pH of the soil, resulting in the release of alumina and silica from the pozzolans and form the clay structure.

Table 4.12 Chemical composition of treated soil

Oxides (%)	SiO_2	Fe_2O_3	Al_2O_3	S	CaO	MgO	pH	EC (µS/cm)
LS1	55.36	6.24	4.98	0.042	0.019	0.003	5.91	1.27
LS1+ Terrabind	56.06	5.22	11.48	0.172	0.98	1.89	7.12	5.53
LS1+ Terrabind + 6% FA	61.95	4.25	7.77	0.330	2.14	1.56	10.26	6.22

LS1: Lateritic Soil 1; SiO_2: Silica oxide; Fe_2O_3: Iron oxide; Al_2O_3: Aluminium oxide; CaO: Calcium oxide; MgO: Magnesium oxide; S: Sulphate; EC: Electrical conductivity

4.3.5 Effect on UCS

UCS test was conducted on untreated and treated soil samples for different curing periods up to 365 days, at OMC and the values are presented in Table 4.13. It can be observed that UCS increased for stabilized soil samples and showed an increasing trend with curing period.

Table 4.13 UCS values for treated and untreated soil

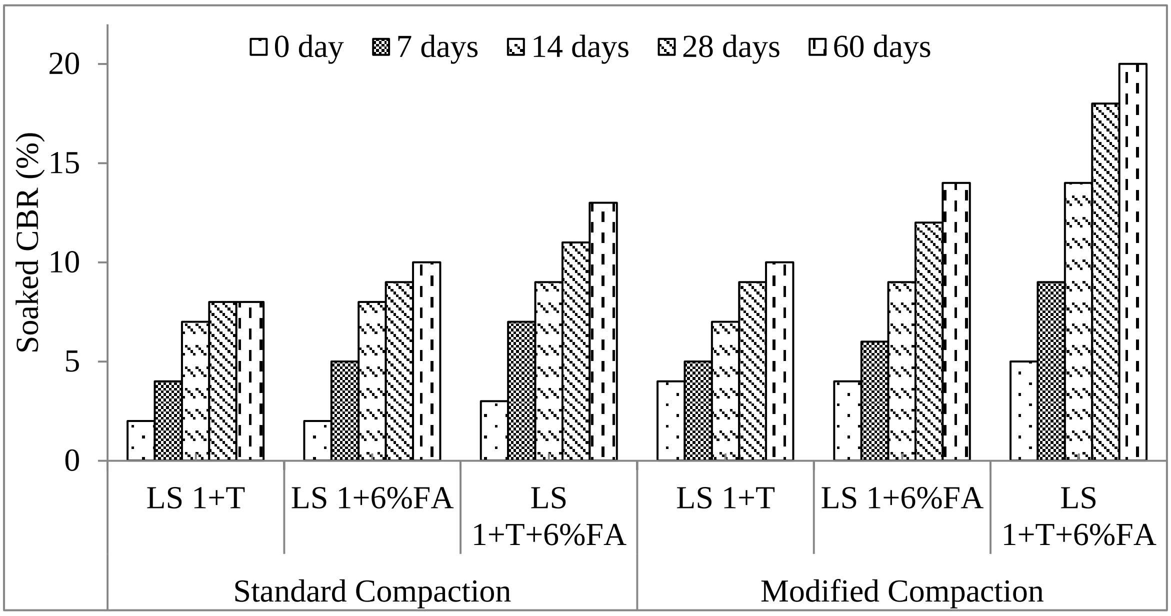
Curing days	UCS (kPa)							
	Standard Compaction				Modified Compaction			
	LS1	LS1+ T	LS1 + 6% FA	LS1+T +6% FA	LS1	LS1 + T	LS1 + 6% FA	LS1 + T + 6% FA
0	138	154	189	245	206	232	286	384
7	142	302	375	421	215	392	538	987
14	146	459	606	727	212	605	845	1124
28	146	532	853	925	209	938	1096	1334
60	145	598	904	1045	207	995	1123	1591
90	148	600	906	1051	208	998	1135	1601
365	150	601	910	1058	208	1000	1142	1619

LS1: Lateritic soil 1; T: Terrabind; FA: Fly Ash:

It can be observed that marginal improvement in strength after 60 days of curing for all the mixes. This tendency may be due to the effective cation exchange process in treated samples which generally takes longer period. The cation exchange and the increased internal friction of clay particles due to flocculation and agglomeration, result in a reduction in soil plasticity, an increase in shear strength and an improvement in texture (Prusinski and Bhattacharya, 1999).

4.3.6 Effect on CBR

CBR test was conducted on untreated and treated soil specimens at OMC in soaked condition for different curing periods. In soaked condition, the low CBR of LS1 alone is due to the dominance of the clay fraction. The higher CBR of Terrabind and FA treated soil is due to its better strength characteristics, primarily because of friction. The increase in soaked CBR is because of the pozzolanic reaction in the presence of water due to the free lime content in FA. Figure 4.1 depicts the variation of soaked CBR values for different combinations at OMC with curing periods. The soaked CBR of Terrabind and FA samples increased rapidly during the 28 days curing, which is due to the cementation caused by the pozzolanic reaction between the soil particles.



LS1: Lateritic soil 1; T: Terrabind; FA: Fly Ash

Fig. 4.1 Soaked CBR values for treated soil at different curing days

4.3.7 Major Findings

- The consistency tests conducted on treated soil shows that the PI decreases for the Terrabind and FA mix.
- The standard and modified proctor compaction tests conducted on treated soil indicate increase in MDD and marginal decrease in OMC.

- The UCS value after 28 days curing period was found to be 925kPa and 1334kPa for standard and modified compactions respectively. The UCS value increases gradually up to 28 days of curing and beyond this, the increase is marginal.
- For treated soil prepared by modified compaction and subjected to 28 days curing, the soaked CBR increased by 6 times compared to the untreated soil. The continuous improvement in the CBR with higher curing period is due to the cementitious hydration between the FA and soil particles.

4.4 LS1 STABILIZATION WITH ARECANUT COIR AND CEMENT

4.4.1 Arecanut Coir

The dry Arecanut shells, which are brown in colour, were collected for the present work and the coir from the shell was extracted manually in the laboratory. Different quantities of coir can cause different effect in the same soil sample. Insufficient quantity of coir may lead to less stabilization of the soil whereas excess quantity may result in ineffective stabilization and decrease the strength of the soil. Hence, to determine the optimum quantity of coir the CBR and UCS tests were conducted on each of the soil sample with varying percentages of coir by weight of soil. The different percentages of coir considered in the present study are listed in Table 4.14.

Table 4.14 Dosage of Arecanut Coir

Dosage	% by Weight of Soil	Weight per 1kg of Soil (gm)
1	0.2	2
2	0.4	4
3	0.6	6
4	0.8	8
5	1.0	10

4.4.2 Cement

In this investigation, 3% of ordinary Portland cement 43 grade, collected from the local market, was used based on earlier studies.

4.4.3 Stabilization using Arecanut Coir with 3% Cement

The soil mixed with coir does not require any curing as there is no chemical reaction takes place between soil and coir. In the present study the soil has been stabilized further by adding three per cent cement to enhance the bonding and strength. The addition of cement enhances the bonding and friction between soil and coir. The strength of the soil in terms of CBR and UCS has been evaluated for 7, 14, 28, 60, 90 and 365 days of curing.

4.4.4 Sample Preparation

The UCS and CBR specimens were cast in the laboratory as per specified standard procedure. CBR tests were carried out both in moist and soaked conditions. To prepare soil-coir mixtures, required quantity of Arecanut coir was added and thoroughly mixed with dry soil and then water was added in two stages to prepare more homogenous specimens. In the first stage, half of the water was added to the mixture, followed by 15 min continuous hand mixing and then the remaining water was added, followed by 5 min hand mixing. In the case of soil cement coir mix, the dry soil, cement and coir were added and mixed together and then required quantity of water was added. The OMC and MDD were maintained for all the dosages. Samples were cured for varying curing periods by maintaining the moisture content. After completion of curing period, specimens for soaked CBR test were placed in water for 4 days and then taken out and allowed to drain before being loaded.

4.4.5 Effect of Coir Content on Compaction

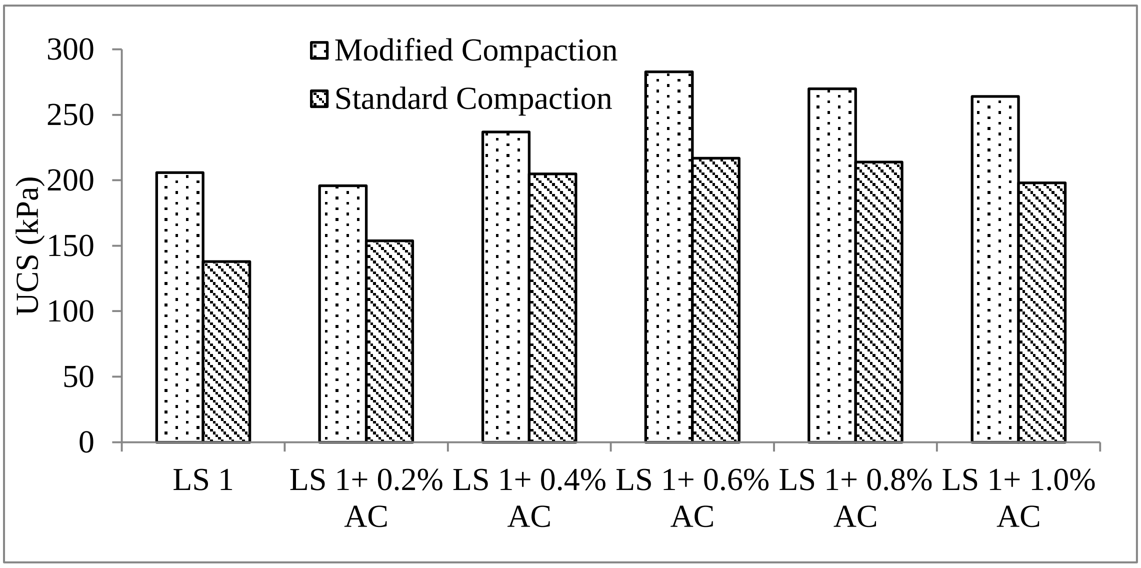
Compaction tests were conducted on lateritic soil reinforced with different percentages of Arecanut coir for both modified and standard proctor cases. The results are tabulated in Table 4.15. It shows that, as the coir percentage increases, the MDD decreases, due to lateritic soil being heavy in weight compared with the coir and was replaced by the light weight coir. But on the other hand the OMC increases with increase in percentage of coir, since the coir absorbs more water.

Table 4.15 Compaction test results

Sample	Modified Compaction		Standard Compaction	
	MDD (g/cc)	OMC (%)	MDD (g/cc)	OMC (%)
LS1	1.69	17.0	1.63	19.2
LS1+ 0.2 % Coir	1.68	19.6	1.63	19.9
LS1+ 0.4 % Coir	1.66	20.0	1.59	20.5
LS1+ 0.6 % Coir	1.64	20.6	1.55	21.2
LS1+ 0.8 % Coir	1.58	21.0	1.51	22.2
LS1+ 1.0 % Coir	1.47	23.0	1.43	23.8

4.4.6 Effect of Coir Content on UCS

The tests were conducted for both standard and modified compaction and the results are depicted in Figure 4.2. As the percentage of coir increased, the UCS value also increased up to a certain limit and beyond that it slightly decreased. The results in Table 4.16 indicate that, the optimum strength was obtained at 0.6% of coir and 3% of cement content, and further increase in coir leads to decrease in strength.



LS1: Lateritic Soil 1; AC: Arecanut Coir

Fig. 4.2 UCS values for Arecanut coir treated with seven days curing period

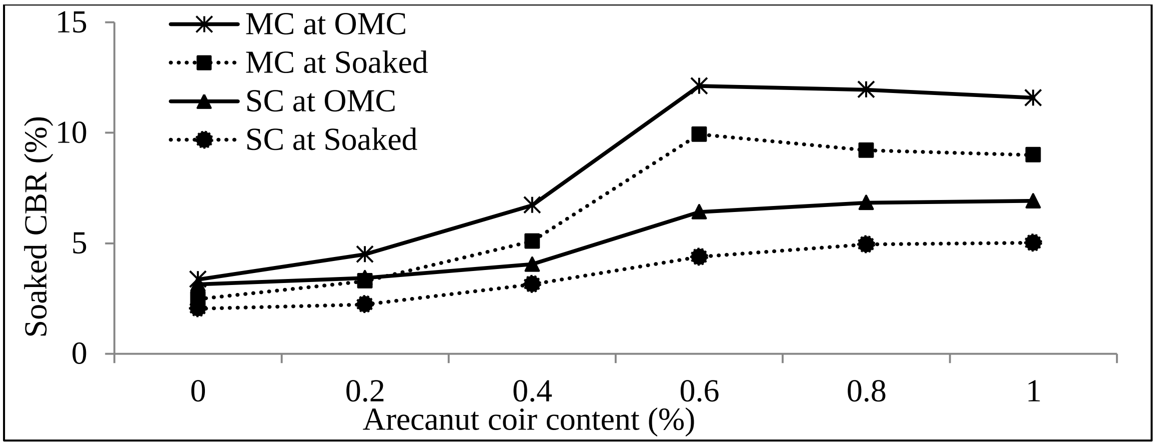
Table 4.16 Variation of UCS with curing period for cement coir treated soil

Dosage	UCS (kPa)											
	7 days		14 days		28 days		60 days		90 days		365 days	
	S	M	S	M	S	M	S	M	S	M	S	M
LS1	142	215	146	212	146	209	145	207	148	208	150	208
LS1+3% C+0% AC	275	345	304	421	332	456	357	495	362	501	365	505
LS1+3% C+0.2% AC	376	520	485	602	504	631	531	645	540	649	546	651
LS1+3% C+0.4% AC	495	615	522	687	564	728	583	738	594	712	599	714
LS1+3% C+0.6% AC	514	717	600	896	635	902	650	931	658	938	662	942
LS1+3% C+0.8% AC	417	608	576	704	598	874	606	882	614	895	618	898
LS1+3% C+1.0% AC	383	543	532	622	563	798	589	802	593	804	602	807

LS1: Lateritic Soil 1; C: Cement; AC: Arecanut Coir; M: Modified Compaction; S: Standard Compaction

4.4.7 Effect of Coir Content on CBR

There is an increase in the CBR value with the increase in the percentage of coir as shown in Figure 4.3. The addition of coir imparts some amount of shear resistance to the soil, leading to the CBR increase. The increase in the strength was less, due to lack of chemical reaction taking place between Arecanut coir and soil.



MC: Modified compaction; SC: Standard Compaction

Fig. 4.3 CBR Values for Arecanut coir treated soil with seven days curing period

Good improvement in CBR value is observed with constant dosage of 3 per cent cement from Table 4.17. As the curing period increased, the CBR values also increased and the maximum CBR value was obtained at 0.6% coir content, and then it decreased. Increase in CBR value is due to the better resistance to the penetration of the plunger caused by the presence of coir. This resistance may be made up of bond between soil mix. The increase in CBR value can also be attributed to the better packing of different fractions.

Table 4.17 Variation of unsoaked and soaked CBR with curing period

Dosage	CBR (%)							
	7 days		14 days		28 days		60 days	
	OMC	Soaked	OMC	Soaked	OMC	Soaked	OMC	Soaked
LS1+3% C+ 0% AC	40	8	50	16	62	19	78	19
LS1+3%C+ 0.2% AC	54	12	62	24	65	26	81	27
LS1+3%C+ 0.4% AC	63	16	68	26	76	30	86	30
LS1+3%C+ 0.6% AC	59	18	72	40	79	42	90	43
LS1+3%C+ 0.8% AC	53	17	70	19	80	20	88	21
LS1+3% C+ 1% AC	54	16	71	20	80	19	85	19

LS1: Lateritic Soil 1; C: Cement; AC: Arecanut coir; M: Modified Compaction; S: Standard Compaction

4.4.8 Major Findings

Based on the tests conducted in the laboratory the following findings have been observed.

- Addition of Arecanut coir to the LS1 resulted in medium improvement in the soil properties and the optimum content was found to be 0.6% by weight of soil.
- The addition of Arecanut coir along with 3% of cement by weight of soil resulted in significant increase in the UCS values.

- CBR values are increased as the dosage and curing days increase. Higher CBR values were found to be for 0.6% coir with 3% cement and beyond that there was a marginal decrease for higher dosages.
- This Arecanut coir soil stabilization is more economical since it is naturally available as an agricultural waste and also only a small amount of cement is sufficient to achieve the optimum stabilization. Hence, overall cost of the road construction can be reduced while comparing with the conventional methods.

4.5 LS2 STABILIZATION WITH CEMENT

Cement stabilization is well known for its ability to stabilize lateritic soil and also provides high strength gain in a shorter time. In this study, cement is used as 3, 6, 9 and 12% to the dry weight of soil and for further enhancement of strength 12mm down aggregates (20, 25 and 30%) are used with optimum cement content. The following section provides the test results on soil cement stabilization and soil cement aggregate stabilization. Different curing periods like 7, 14, 28 and 60 days are provided for all the treated specimens.

4.5.1 Effect of Cement

4.5.1.1 Consistency limits

Tests were conducted on cement treated soil to examine the effect of stabilizer content on the consistency limits of the soil. For this purpose, tests were conducted 30 minutes after the addition of cement. The consistency limits of cement treated soil specimens are shown in Figure 4.4, which indicates that soil cement mixture showed an increase in the LL value for all the cement content. The PI values decreased with the addition of cement.

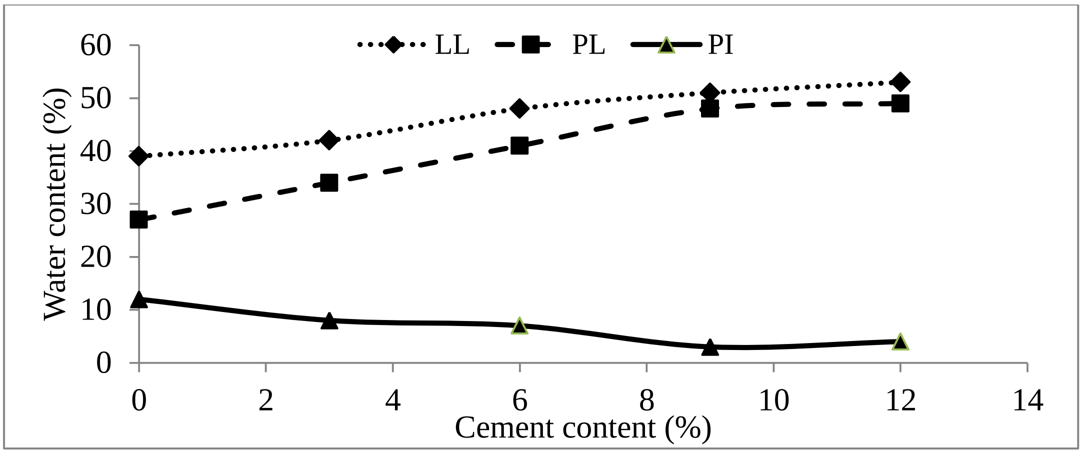


Fig. 4.4 Consistency limits of the cement soil mixture

4.5.1.2 Compaction characteristics

The test was conducted on soil and soil with cement for dosages of 3, 6, 9 and 12% immediately after mixing with cement. From Figures 4.5 and 4.6, it is observed that, OMC decreased for 3 and 6% of cement, but with further increment of cement it increased. At lower dosages, the network may be small and isolated whereas at higher dosages the reaction products may form large interconnected networks. The reaction process taking place in cement stabilization does not depend upon soil minerals, but on reaction of water and hence almost all types of soils can be stabilized by cement (Montgomery 1998).

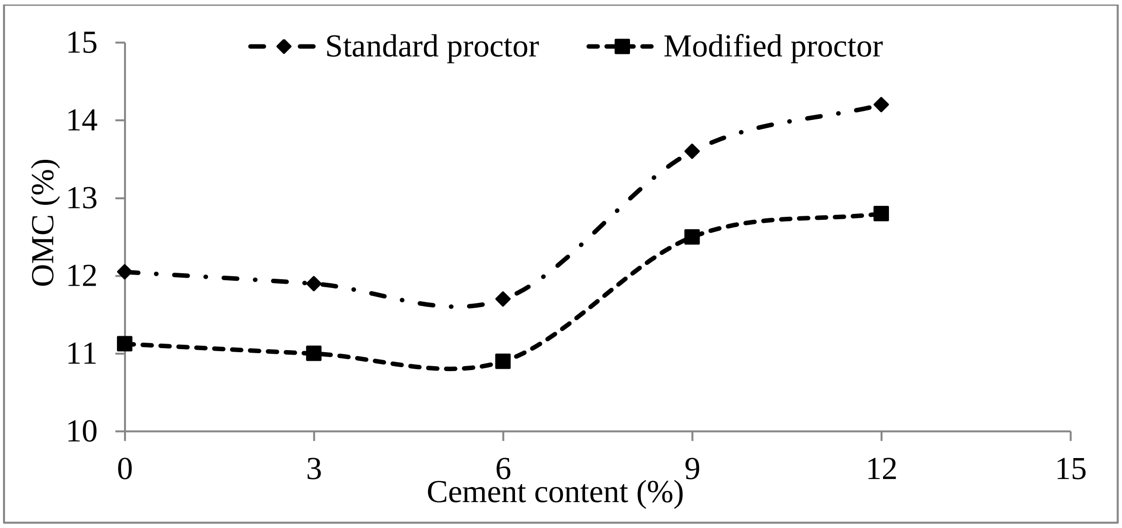


Fig. 4.5 Variation of OMC for standard and modified proctor for stabilized soil

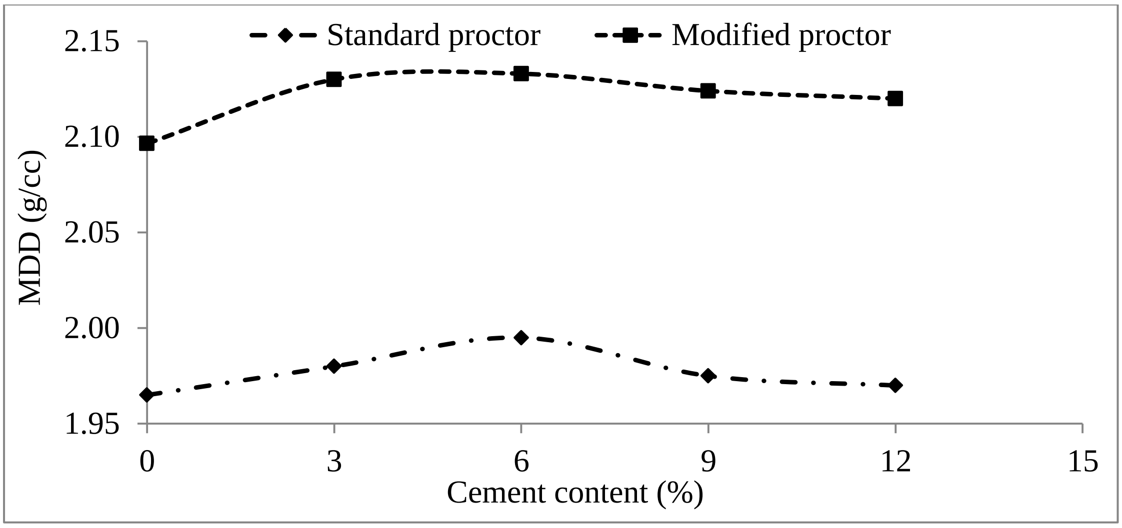


Fig. 4.6 Variation of MDD for standard and modified proctor for stabilized soil

4.5.1.3 Chemical composition of treated soil

The presence of cement increases the pH of the soil as listed in Table 4.18. The high pH releases alumina and silica from the pozzolans to form the clay structure. These free alumina and silica react irreversible with the calcium ions to form calcium aluminum silicates that are similar to the components of Portland cement. These calcium silicates have net negative charges, which attract ionized water (molecules that act as dipoles) to create a network of hydration bonds that cement the particles of the soil together (Scholen et al. 1992).

Table 4.18 Chemical composition of cement treated soil

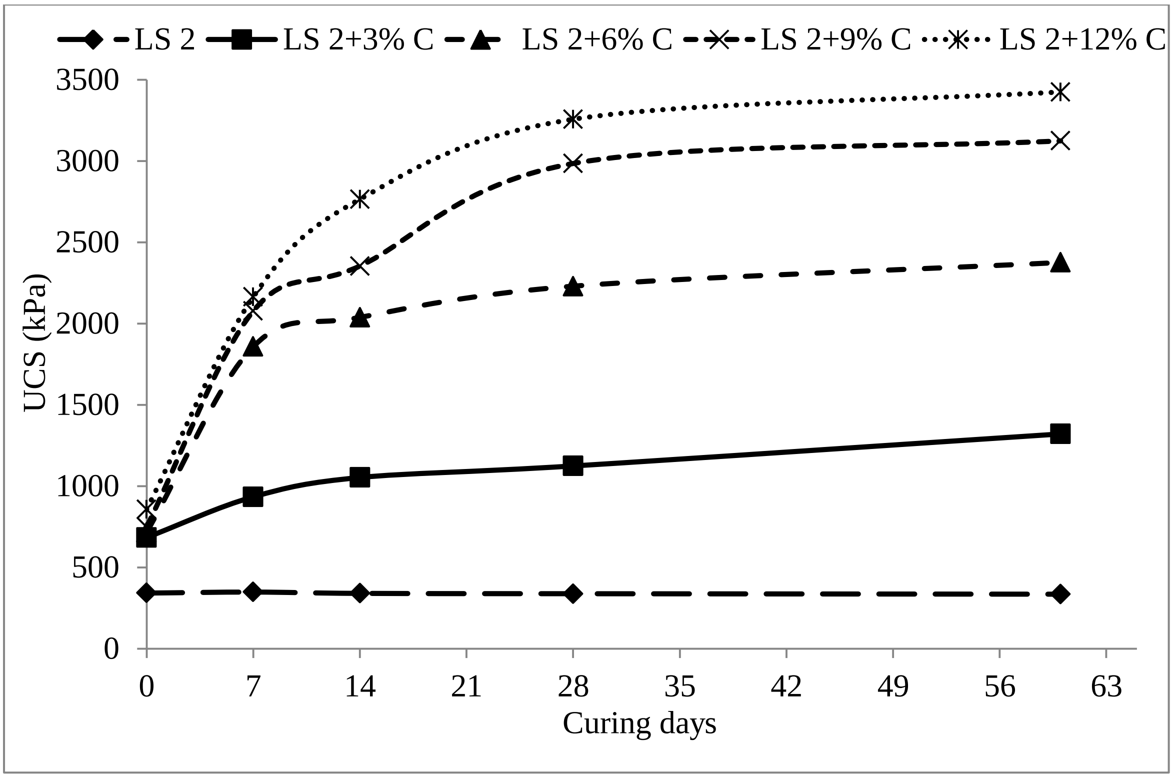
Oxides (%)	SiO_2	Fe_2O_3	Al_2O_3	S	CaO	MgO	pH	EC (µS/cm)
LS2	67.3	7.99	4.85	0.031	0.011	0.006	5.7	1.35
LS2+ Cement	69.00	3.85	14.21	0.32	8.22	3.65	14.23	1.95

LS2: Lateritic soil 2; SiO_2: Silica oxide; Fe_2O_3: Iron oxide; Al_2O_3: Aluminium oxide; CaO: Calcium oxide; MgO: Magnesium oxide; S: Sulphate; EC: Electrical conductivity

4.5.1.4 UCS

For both treated and untreated soil specimens, UCS was determined at modified proctor compaction densities. To check the effect of curing time, the tests were conducted on treated soil specimens for 0, 7, 14, 28 and 60 days of moist curing at OMC, 2% above OMC (wet side) and 2% below OMC (dry side). From Figure 4.7

it can be observed that, the compressive strength increases with cement content and curing period. As the time progresses, formation of dicalcium silicates takes place due to hydration of cement and it is responsible for the enhanced strength at the later stages. The compressive strength increased even after 28 days of curing due to the ion exchange between soil and cement, which leads to cementation. The upper limit UCS of 1716 kPa after seven days cured was introduced to prevent the excessive use of cement (Khanna et al. 2012). This criterion was achieved for cement content of 6% even with seven days curing and hence it can be considered as the optimum cement content for the present study.



LS2: Lateritic Soil 2; C: Cement

Fig. 4.7 Variation of UCS values at OMC for different curing days

The UCS values of the treated soil specimens with different water content (dry side, OMC and wet side) are presented in Table 4.19. It was observed that compressive strength is higher for the dry side compared to other cases. This research work intimates that the water cement ratio is an important factor in the mechanical behavior of treated soil. The observed results are in concert with the outcomes of

Miura et al. (2002), by obtaining decreasing strength with the increase in the water cement ratio.

Table 4.19 UCS values of cement treated soil specimens

Curing days	Cement content (%)	UCS (kPa)		
		Wet side	OMC	Dry side
0	3	582	684	765
	6	674	716	789
	9	632	749	859
	12	702	857	963
7	3	598	934	1256
	6	1174	1858	2032
	9	1487	2077	2786
	12	1624	2164	2876
14	3	672	1053	1324
	6	1256	2038	2715
	9	1501	2354	3105
	12	1796	2765	3256
28	3	721	1124	1632
	6	1396	2230	2879
	9	1974	2985	3457
	12	1875	3257	3682
60	3	763	1321	1803
	6	1874	2376	3458
	9	2087	3124	3721
	12	2471	3425	3969

In this study, an attempt is also made to develop an equation for the UCS value using the test results. The logarithmic trend lines drawn for the compressive strength versus curing days data, determined from the experimental study, provided a better fit as depicted in Figure 4.8. Based on the data, Equation (4.1) was obtained to predict the UCS values of soil treated with cement content 3, 6, 9 and 12 % for different curing periods.

$$UCS = X + Y \ln t \qquad (4.1)$$

Where t is curing days, X and Y are the coefficients obtained by least-square regression. The values of coefficients (X and Y) are tabulated in Table 4.20.

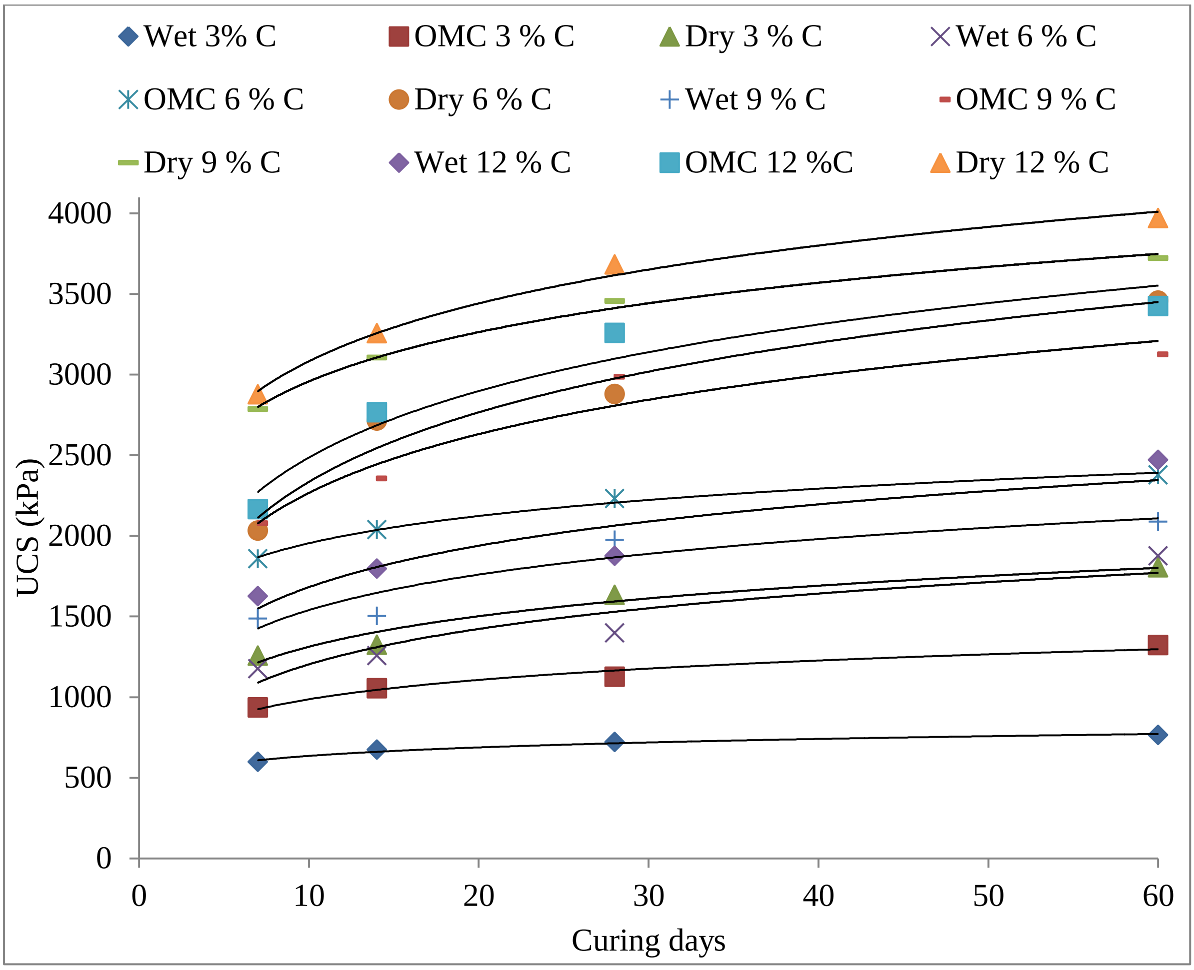


Fig. 4.8 Variation in UCS with curing time for various cement content

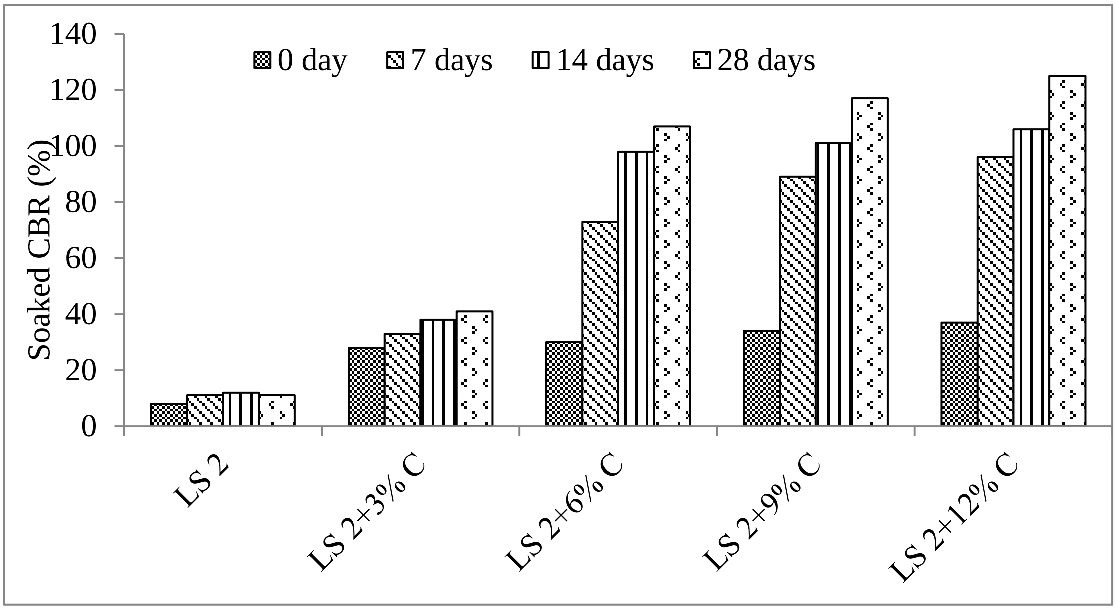
Table 4.20 Values of X and Y parameters

Cement content (%)	X			Y		
	Wet	OMC	Dry	Wet	OMC	Dry
3	76	173	273	460	588	684
6	317	244	623	474	1392	902
9	318	526	442	808	1055	1942
12	371	518	595	829	1891	1112

4.5.1.5 CBR

The CBR tests were conducted on treated and untreated soil specimens prepared at modified proctor compaction densities. The soil specimens were kept for

0, 7, 14, and 28 days of moist curing by covering the CBR mold in polyethene covers which was then covered by wet gunny bags.



LS2: Lateritic Soil 2; C: Cement

Fig. 4.9 Effect of curing on the CBR of treated soils for different curing days

From Figure 4.9 it can be observed that, the CBR for untreated soil has no significant changes with curing. The addition of cement has increased the CBR of the mixes continuously for all the curing periods. It also can be seen that, for 3% of cement content, the CBR increased with curing, since the higher cement content resulted in the formation of more hydration products. Further, the gain in CBR is generally rapid during the first seven days of curing, indicating that the formation of cementitious products due to hydration of cement was completed within the initial seven days. The rate of increase of CBR was rapid during the first seven days curing for higher cement contents and beyond this period, it was low.

4.5.2 Effect of Cement and Aggregate

High quality of aggregate materials is becoming increasingly scare and expensive, and therefore optimizing the use of locally available materials is becoming an economic necessity. When cement is not adequate to achieve the desired strength and requirements, cement in combination with aggregate may provide the needed improvement. In general target strength can be achieved through a good design process which identifies soil cement aggregate combination to obtain desired strength and resilient modulus properties. In this study an attempt is made to utilize different dosages of aggregate for optimum cement treated soil.

4.5.2.1 Compaction characteristics

The test was conducted on soil with optimum amount of cement (6%) and aggregates with different dosages (20, 25 and 30% by weight of soil). Table 4.21 presents the soil cement aggregate compaction results. It was observed that, the MDD for soil aggregate mix does not follow any specific trend. This may be due to the fact that, the addition of extra amount of coarse particulate matter has resulted in lesser close packing in the soil aggregate mix.

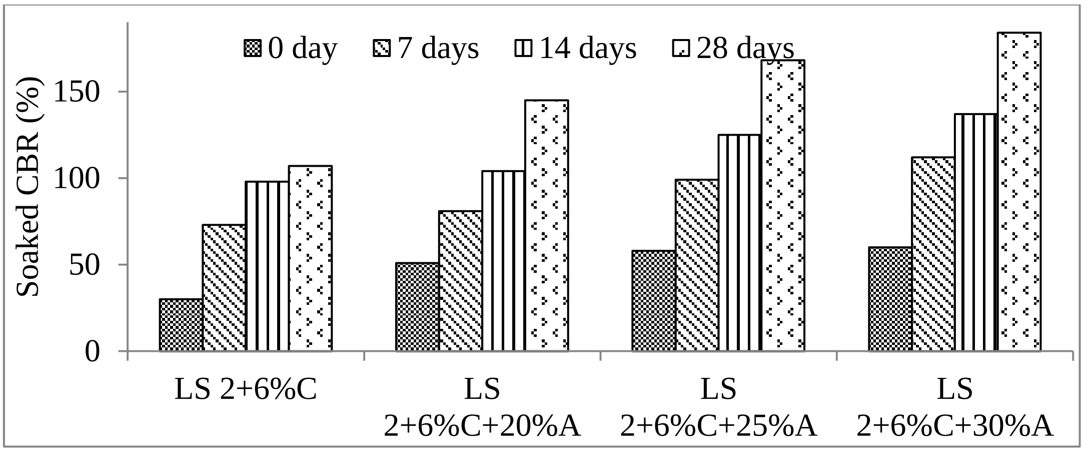
Table 4.21 Compaction values for soil cement aggregate samples

Soil Mix	Standard Proctor		Modified Proctor	
	OMC (%)	MDD (g/cc)	OMC (%)	MDD (g/cc)
LS2	12.05	1.97	10.04	2.09
LS2+ 6% Cement	11.70	1.99	10.90	2.13
LS2+ 6% Cement +20% Aggregates	12.10	1.97	10.90	2.21
LS2+ 6% Cement +25% Aggregates	11.50	1.98	9.50	2.26
LS2+ 6% Cement +30% Aggregates	11.10	1.99	9.20	2.33

LS2: Lateritic Soil 2

4.5.2.2 CBR

The test was conducted on soil with 6% cement and different percentages of aggregates. Figure 4.10 shows the effect of cement and aggregates on the CBR of lateritic soil mixes. The addition of aggregates to the treated soil has resulted in higher resistance to penetration as compared to treated soil without aggregate.



LS2: Lateritic Soil 2; C: Cement; A: Aggregate

Fig. 4.10 Soaked CBR values for soil cement aggregate mixtures

4.5.2.3 Resilient modulus

In this study, test was conducted for LS2, soil with 6% cement and soil with 6% cement and 30% aggregate mix cured for seven days. Tables 4.22 and 4.23 present the testing sequence values for treated and untreated soil specimens.

Table 4.22 Testing sequence for LS2

Sequences (200 Cycles each)	Axial Stress, σ_1		Confining pressure, σ_3	Cyclic Stress	Bulk Stress, θ	Recovered Deformation	Resilient strain	E (MPa)
	kPa	kgs	kPa	kPa	kPa	mm	mm/mm	
0 (500 to 1000cycles)	196.5	157	103.4	93.1	403.3	0.00	0.00000	00.00
1	39.3	31	20.7	18.6	80.7	0.22	0.00110	16.91
2	58.0	46	20.7	37.3	99.4	0.25	0.00125	29.84
3	76.6	61	20.7	55.9	118.0	0.29	0.00145	38.55
4	65.5	52	34.5	31.0	134.5	0.26	0.00130	23.85
5	96.5	77	34.5	62.0	165.5	0.31	0.00155	40.00
6	127.6	102	34.5	93.1	196.6	0.34	0.00170	54.76
7	130.9	105	68.9	62.0	268.7	0.35	0.00175	35.43
8	193.0	154	68.9	124.1	330.8	0.39	0.00195	63.64
9	255.0	204	68.9	186.1	392.8	0.44	0.00220	84.59
10	165.4	132	103.4	62.0	372.2	0.31	0.00155	40.00
11	196.5	157	103.4	93.1	403.3	0.32	0.00160	58.19
12	289.5	232	103.4	186.1	496.3	0.40	0.00200	93.05
13	231.0	185	137.9	93.1	506.8	0.32	0.00160	58.19
14	262.0	210	137.9	124.1	537.8	0.32	0.00160	77.56
15	386.1	309	137.9	248.2	661.9	0.44	0.00220	112.82

Table 4.23 Testing sequence for stabilized soil

Sequences (200 Cycles each)	LS2 + 6% Cement			LS2 + 6% Cement + 30% Aggregate		
	Recovered Deformation (mm)	Resilient strain (mm/mm)	M_R (MPa)	Recovered Deformation (mm)	Resilient strain (mm/mm)	E (MPa)
1	0.07	0.00035	53.14	0.07	0.00035	53.14
2	0.10	0.00050	74.60	0.08	0.00040	93.25
3	0.13	0.00065	86.00	0.10	0.00050	111.80
4	0.10	0.00050	59.18	0.10	0.00050	62.00
5	0.14	0.00070	83.04	0.12	0.00060	103.33
6	0.18	0.00090	100.72	0.15	0.00075	124.13
7	0.20	0.00100	62.00	0.15	0.00075	82.67
8	0.27	0.00135	78.62	0.20	0.00100	124.10
9	0.31	0.00155	120.06	0.25	0.00125	148.88
10	0.19	0.00095	60.81	0.16	0.00078	80.00
11	0.23	0.00115	80.96	0.19	0.00095	98.00
12	0.36	0.00180	103.39	0.29	0.00145	128.34
13	0.26	0.00130	66.84	0.22	0.00110	84.64
14	0.30	0.00150	82.73	0.22	0.00110	112.82
15	0.30	0.00150	160.30	0.26	0.00130	190.92

The resilient modulus test results for the untreated and treated soils specimens are presented in Figures 4.11, 4.12 and 4.13. It shows that combined cement treated and cement aggregate soils have yielded the highest E enhancements when compared with control soils at the same confining pressure and corresponding deviatoric stress. These results were expected, as combined cement aggregate treatment results in a stronger and stiffer material than the untreated soil specimens. It also shows that the E values of the treated specimens increased with increasing confining pressure. The increase in E values is attributed to the fact that applying higher confinement to the treated specimens tends to compress them and make them denser and stronger, resulting in better stiffness and higher E value. Due to time consumption and availability of instrument, only optimum dosage samples were tested.

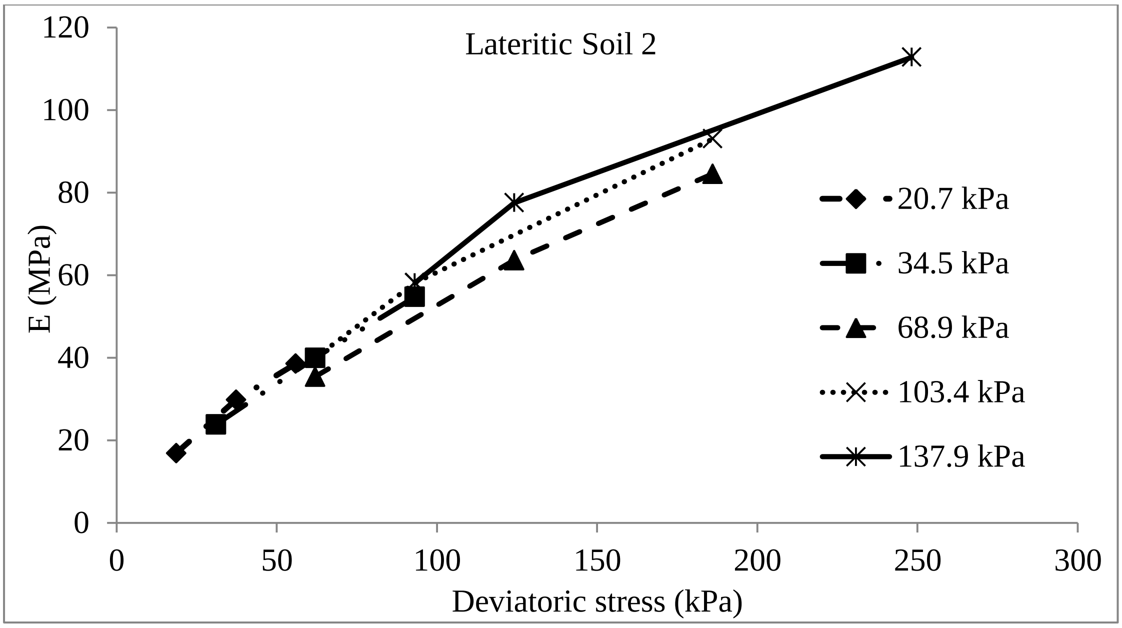


Fig. 4.11 Resilient modulus at different confining pressure for lateritic soil

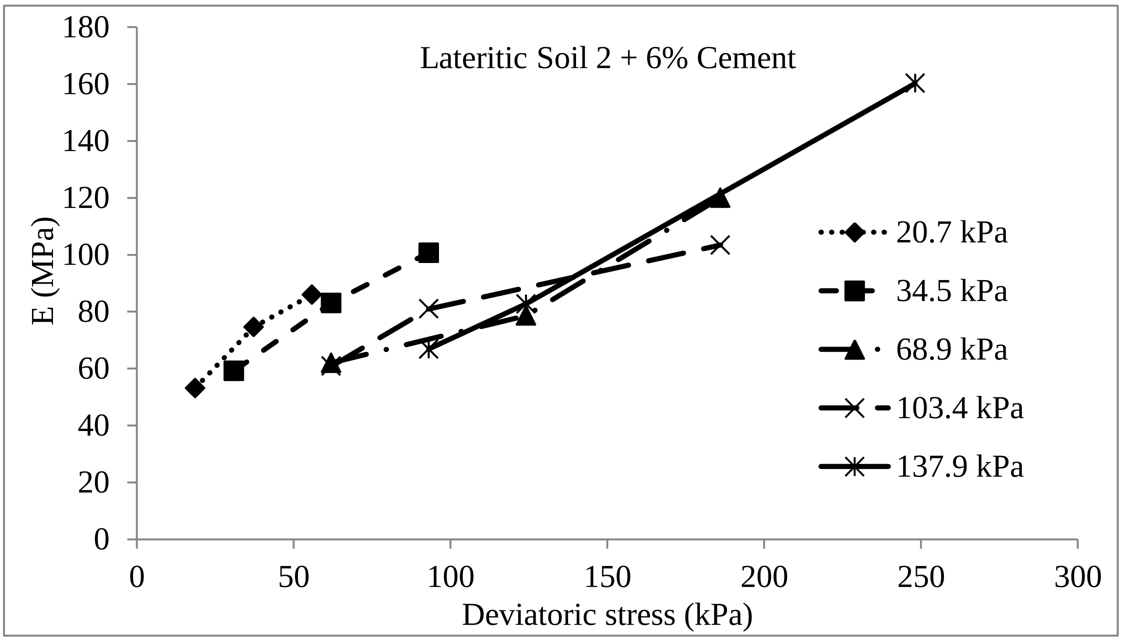


Fig. 4.12 Resilient modulus at different confining pressure for cement treated soil

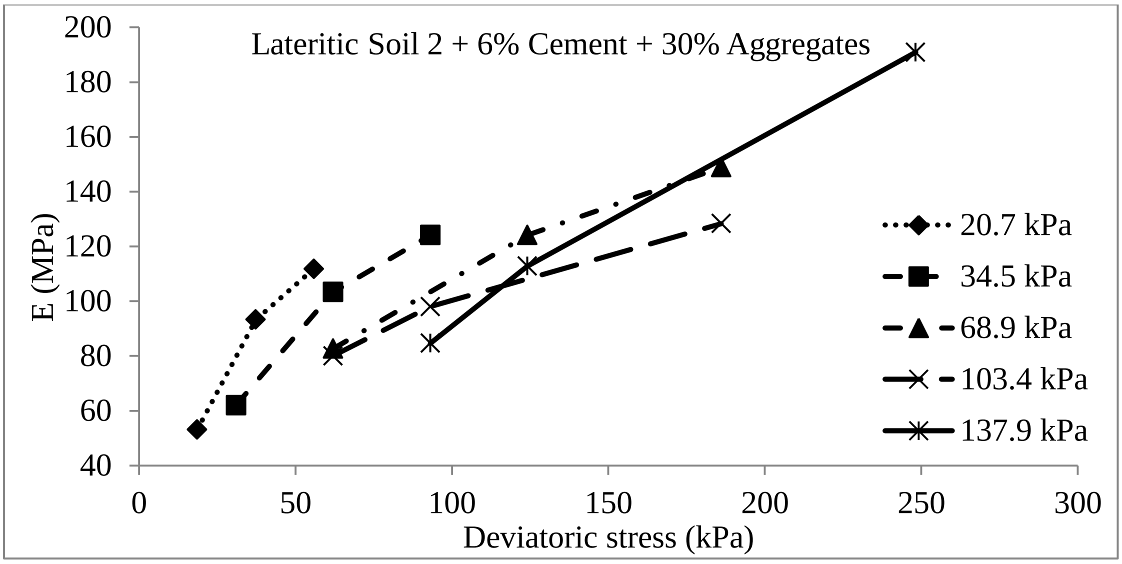


Fig. 4.13 Resilient modulus at different confining pressure for cement treated aggregate soil

4.5.3 Major Findings

The following findings were obtained from the current experimental investigation;

- The stabilizer helps significantly to improve the strength of LS2. Cement is observed to be an effective stabilizer and further addition of aggregates improves the soil properties.
- Aggregate treated soils show decreased OMC and increased MDD values.
- With the increase in the cement content, increment in the UCS and CBR strength of treated soil is observed to be higher. The increase in UCS and CBR strength is caused by the improvement in both the internal friction and cohesion of the stabilized soil.
- Addition of optimum cement content (6%) to soil results a UCS improvement by 5.3 – 6.6 times for 7 – 28 days curing.
- Cement content, aggregate and curing time play a significant role on the strength of treated soil. CBR value shows an improvement by 10.2 – 16.7 times with the addition of aggregates (30%) and cement (6%) for different curing periods.
- Logarithmic regression models were found to be effective in predicting the UCS of cement treated soil.

- Resilient modulus of 6% cement stabilized and 6% cement + 30% aggregate specimens increases with increase in confining pressure.
- Cement stabilized LS2 can be used for pavement construction since it meets the UCS, CBR and durability requirements. The usual strength criteria are a 7 days UCS of 1,720 kPa, a CBR of 100%, and an economic optimum cement content of 6%.

4.6 LS2 STABILIZATION WITH RBI 81

4.6.1 Effect of RBI 81 on Compaction Test

LS2 was stabilized with 2, 4, 6 and 8 % RBI 81 and also different percentages of aggregates (20, 25 and 30 %) were mixed with optimum RBI 81 content. Modified compaction was performed for all the soil specimens and the results are tabulated in Table 4.24. It is observed that, with the increase in percentage of stabilizer, the OMC and MDD of the soil increase gradually. The reaction between soil and the stabilizer has taken place and RBI 81 absorbs water, resulting in an increase of OMC. Whereas OMC of the soil RBI 81 aggregate mix is less than that of the 6% RBI 81 treated soil, and the density has been increased as the aggregate content increased. This may be due to the fact that addition of aggregate has resulted in a closure orientation of soil particle and the soil structure has obtained the MDD condition at lower moisture content.

Table 4.24 Modified proctor test results for soil with RBI 81 soil samples

Sample	OMC (%)	MDD (g/cc)
LS2	10.38	2.04
LS2 + 2% RBI 81	10.00	1.95
LS2 + 4% RBI 81	11.60	2.08
LS2 + 6% RBI 81	12.10	2.13
LS2 + 8% RBI 81	12.80	2.10
LS2 + 6% RBI 81 + 20% Aggregate	11.10	2.10
LS2 + 6% RBI 81 + 25% Aggregate	11.50	2.18
LS2 + 6% RBI 81 + 30% Aggregate	11.60	2.32

LS2: Lateritic Soil 2

4.6.2 Chemical Composition of Treated Soil

The presence of RBI 81 increases soil pH and it becomes more alkaline. The high pH, releases alumina and silica from the pozzolans and form the clay structure. These free alumina and silica react irreversible with the calcium ions to form calcium aluminum silicates that are similar to the components of RBI 81. The test results are tabulated in Table 4.25.

Table 4.25 Chemical composition of RBI 81 treated soil

Oxides (%)	SiO_2	Fe_2O_3	Al_2O_3	S	CaO	MgO	pH	EC (µS/cm)
LS2	67.3	7.99	4.85	0.031	0.011	0.006	5.7	1.35
LS2+ RBI 81	72.8	2.79	16.95	0.28	6.43	3.5	13.97	1.23

LS2: Lateritic Soil 2; SiO_2: Silica oxide; Fe_2O_3: Iron oxide; Al_2O_3: Aluminium oxide; CaO: Calcium oxide; MgO: Magnesium oxide; S: Sulphate; EC: Electrical conductivity

4.6.3 Effect of RBI 81 on UCS

After treating the soils with stabilizer, the UCS test was carried out as per the prescribed standards for curing periods 7, 14, 28 and 60 days. UCS values of RBI 81 treated soil at OMC for modified proctor compaction are presented in Table 4.26. In general, the UCS value increases with the increase in curing period and also with the increase of percentage of the stabilizer. Also, it can be seen that maximum strength increase is observed in 6% RBI 81, and is selected as the optimum percentage for addition of aggregate. This shows the extent of soil stabilizer reaction is the maximum for 6% dosage of RBI 81 which results in maximum strength gain.

Table 4.26 UCS results at OMC for different dosages of RBI 81

Dosage	UCS (kPa)			
	Curing days			
	7	14	28	60
LS2	348	340	338	335
LS2 + 2% RBI 81	1350	1660	2406	2538
LS2 + 4% RBI 81	2135	2555	3432	3549
LS2 + 6% RBI 81	2507	2920	3919	4414
LS2 + 8% RBI 81	2685	3178	4081	4524

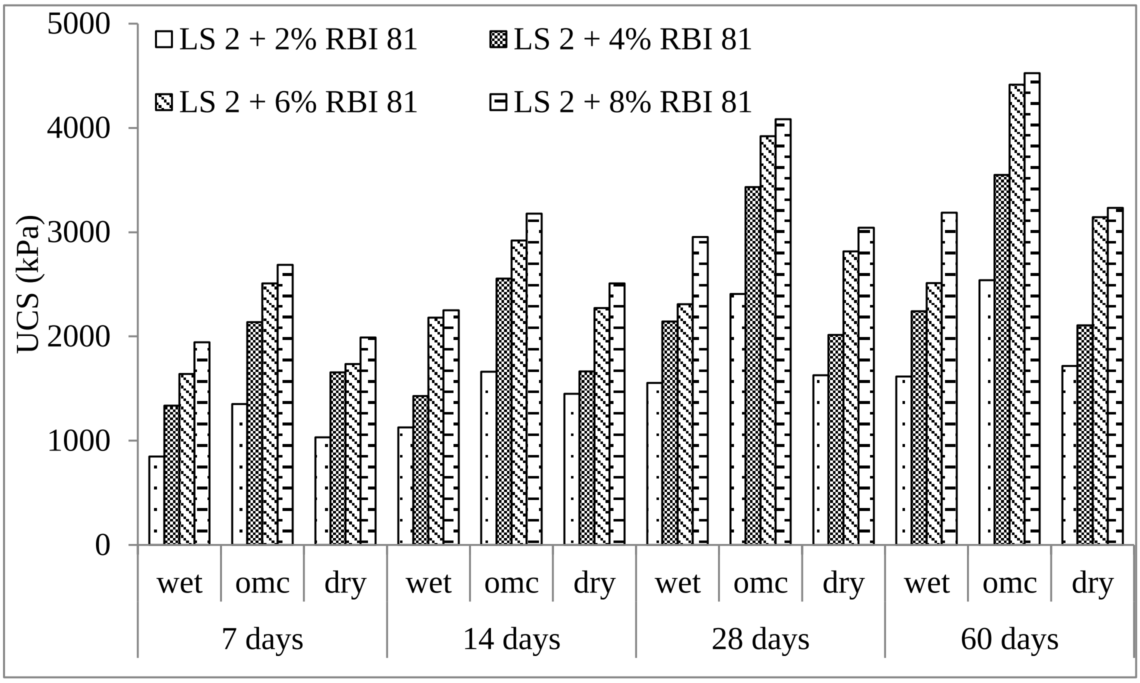
LS2: Lateritic Soil 2

UCS values for RBI 81 treated soil at dry side of OMC (- 2%) and wet side of OMC (+2%) for Modified proctor compaction values are given in Table 4.27. Figure 4.14 depicts different moisture conditions for treated soil. It is noted that wet side OMC values are lesser than the dry side of OMC, due to the presence of more water leading to the decrease in strength.

Table 4.27 UCS results at OMC ± 2% for different dosages of RBI 81

Dosage	UCS (kPa)							
	7 days		14 days		28 days		60 days	
	OMC+ 2%	OMC- 2%	OMC+ 2%	OMC- 2%	OMC+ 2%	OMC- 2%	OMC+ 2%	OMC- 2%
LS2 + 2% RBI 81	846	1033	1127	1450	1554	1627	1616	1717
LS2 + 4% RBI 81	1335	1653	1426	1664	2143	2014	2240	2105
LS2 + 6% RBI 81	1639	1735	2178	2272	2307	2814	2512	3142
LS2 + 8% RBI 81	1943	1989	2249	2509	2951	3040	3187	3232

LS2: Lateritic Soil 2

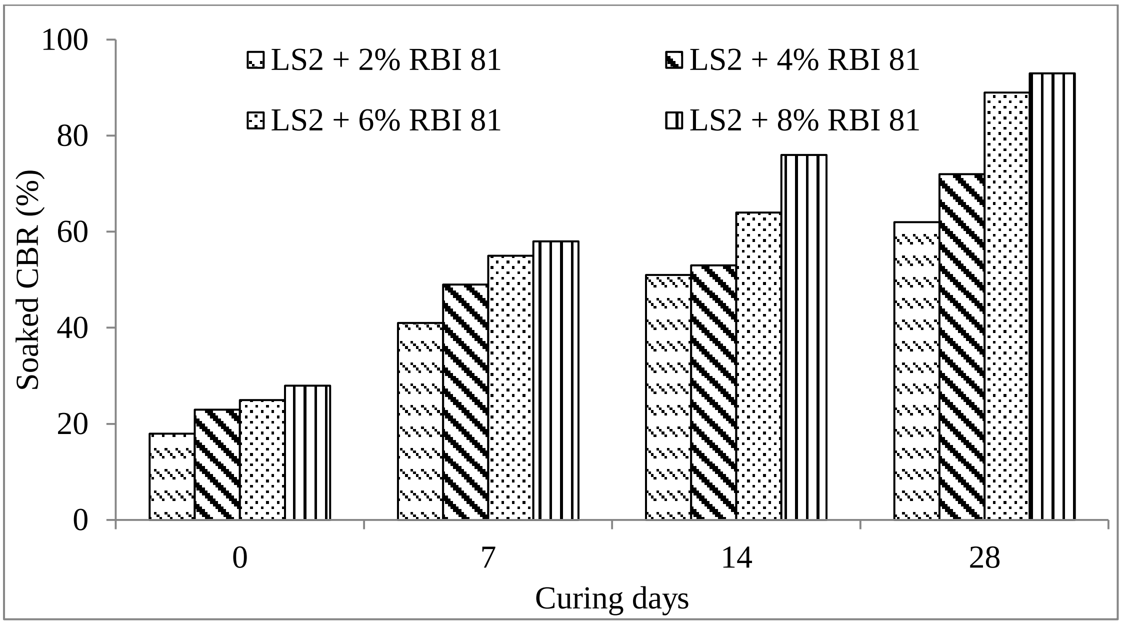


LS2: Lateritic Soil 2

Fig. 4.14 Variations of UCS values at different OMC conditions

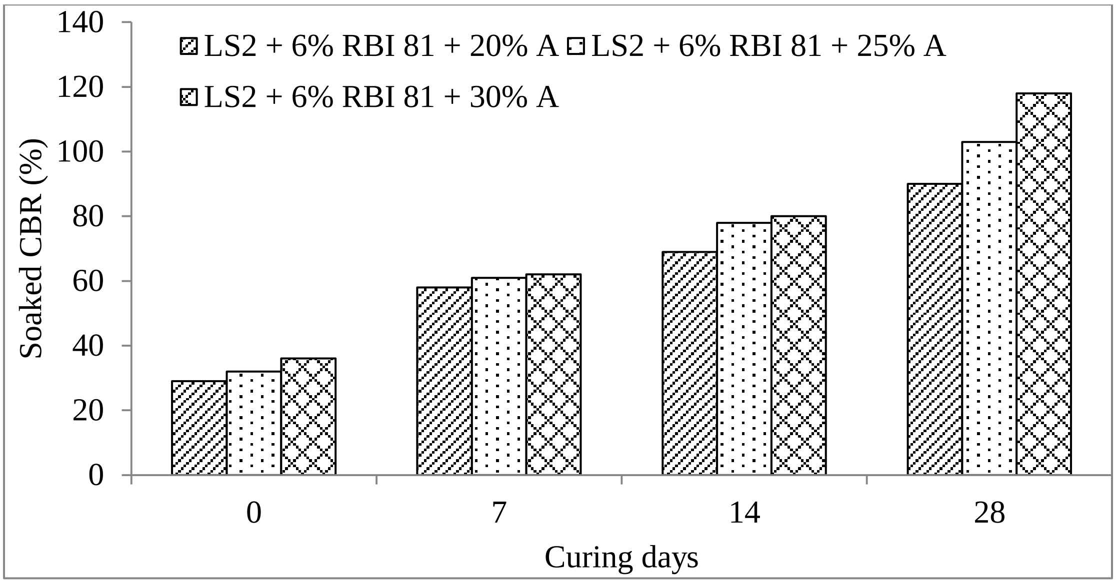
4.6.4 Effect of RBI 81 and Aggregate on CBR

The results obtained for CBR test for RBI 81 treated soil and soil with 6% RBI 81 and aggregates for different curing periods are graphically represented in Figures 4.15 and 4.16. It is seen that, the CBR values of 0, 2, 4, 6 and 8 % of RBI 81 treated soils keep gradually increasing with the increase in curing period. This indicates that the strength of the soil has improved considerably and it has more resistance to penetration after modifying with RBI 81.



LS2: Lateritic Soil 2

Fig. 4.15 Soaked CBR Values for RBI 81 treated specimens



LS2: Lateritc Soil 2

Fig. 4.16 Soaked CBR Values for optimum RBI 81 treated soil, soil + RBI 81 and aggregate specimens

4.6.5 Major Findings

- By adding 2, 4, 6 and 8% RBI 81 to the LS2, it is found that OMC first decreases for 2% RBI 81 and thereafter gradually increases. But there is no specific trend for MDD values of treated soil.
- Addition of aggregates to the 6% RBI 81 treated soil has resulted in decrease in OMC values. This may be due to the fact that addition of aggregates has resulted in a closure orientation of soil particles and the soil structure has obtained the MDD condition at lower moisture content.
- The UCS tests conducted for RBI 81 treated soils have shown significant increase in UCS. At 6% RBI 81 treated soil UCS improved by 7.2 – 13.2 times for 7- 60 days curing. This indicates that the stabilization reaction is optimum at 6%.
- The UCS tests conducted on wet and dry sides of OMC, to simulate the field conditions, indicate that dry side of OMC gives better strength than wet side of OMC.

- As curing period increases CBR value increases gradually for treated soil. This strength increases occur between 7 to 28 days and the smaller strength gain is due to the continued formation of cementitious material and hydration process.

CHAPTER 5

BLACK COTTON SOIL STABILIZATION

5.1 GENERAL

This chapter focuses on the improvement of BC soil, which presents critical geotechnical and structural engineering challenges in the world due to its volume change associated with variation in moisture contents. This soil has very low load bearing capacity when wet, and should be treated properly to use as sub grade. BC soil has low strength in terms of CBR and undergoes volume change with seasonal variation in moisture content, resulting in swelling and shrinkage. Hence, BC soil alone cannot be used in pavement construction. This study was carried out on BC soil procured from North Karnataka, India, where the soil is abundantly available. and stabilized with different stabilizers like Terrabind, Terrasil and RBI 81. The basic properties of BC soil are tabulated in Table 5.1.

Table 5.1 Geotechnical properties of BC soil

Sl No.	Property	BC soil
1	Specific gravity	2.5
2	Grain size distribution (%)	
	Gravel	04
	Sand	24
	Silt	51
	Clay	21
3	Consistency limits (%)	
	Liquid Limit	64
	Plastic Limit	31
	Plasticity Index	33
4	IS Soil Classification	CH
5	Standard Compaction	
	MDD (g/cc)	1.62
	OMC (%)	20.45
	Modified Compaction	
	MDD (g/cc)	1.77
	OMC (%)	16.32
6	CBR value (%)	
	Standard Compaction	

	Unsoaked condition	19
	Soaked condition	<1
	Modified Compaction	
	Unsoaked condition	28
	Soaked condition	1
7	UCS value (kPa)	
	Standard Compaction	152
	Modified Compaction	173

5.2 BC SOIL STABILIZATION WITH TERRABIND

Terrabind is a new advanced lignosulphonate liquid ionic organic compound suitable for soil stabilization. This chemical is available in the liquid form and is to be diluted in water at specified proportion before mixing with the soil. The Terrabind has a catalyst effect when combined with clayey soil and 5-10% of FA by weight of soil. A constant 6% of FA was used for further improvement of soil, which acts as a cementitious stabilizer for soil improvement and is highly recommended to use in combination with Terrabind for a higher strength sub grade.

5.2.1 Chemical Dosage

Dosage of 1mL of concentrated Terrabind liquid is considered for 3kg of soil. Since BC soil has high silt and clay content, higher quantity is required to break down the lattice and maximize the penetration of the Terrabind solution in soil. The amount of stabilizer to be used was calculated by the following method.

For Modified compaction, MDD of soil: 1.77g/cc, OMC of soil: 16.31%, Natural moisture content of soil: 6%

Water to be added: $[(16.31 - 6) + 2] \times (3/100) = 0.369$ liters of water (2% extra water was added to compensate the evaporation loss).

1mL of chemical is diluted in 369mL of water and then used with 3kg dry soil.

5.2.2 Sample Preparation

The required quantity of soil and FA was added and thoroughly mixed with dry state and 16% OMC (obtained from the modified proctor test). The Terrasil is diluted in OMC amount of water and then added in two stages to prepare more homogenous mixture. In the first stage, half of the water was added to the soil, followed by 15 minutes continuous hand mixing, and then the remaining water was added, followed by 5 minutes hand mixing. Samples were cured for varying curing periods by maintaining the moisture content. For every test conducted, minimum three specimens each were used and the average value was reported, ensuring the precision suggested by the standards.

5.2.3 Effect on Engineering Properties

The treated and untreated soil samples were tested for consistency limits and modified compaction. Each test was performed within 30 minutes after mixing. From the test results tabulated in Table 5.2, it can be seen that, engineering properties are slightly enhanced by stabilizing with Terrabind and FA alone, whereas better improvement has occurred when these stabilizers were used in combination. The PI values were decreasing as the curing days increased, due to the densification of soil. The addition of Terrabind and 6% of FA slightly increases the MDD of the treated soil. During compaction, the finer portion of FA may be squeezed into the voids of soil particles, resulting in an increase in MDD (McManis and Arman, 1989). Marginal decrease was observed in OMC, which can be attributed to the progressive hydration process of the FA that consumed some amount of water inside the voids.

Table 5.2 Geotechnical properties of stabilized soil

Mix	BC Soil	BC + Terrabind			BC + FA			BC + Terrabind + FA		
Curing Days		1	7	28	1	7	28	1	7	28
Consistency Limits (%)										
LL	64	60	58	57	58	55	53	55	51	50
PL	31	33	34	35	31	33	34	29	31	33
PI	33	27	24	22	27	22	19	26	20	17
Modified Proctor Compaction										
MDD (g/cc)	1.77	1.77	1.79	1.85	1.77	1.80	1.85	1.77	1.8	1.88
OMC (%)	16.31	16.31	16.29	16.25	16.33	16.31	16.26	16.31	16.21	16.11
Free Swell Index (%)										
FSI	50	19	8	8	11	9	6	9	5	2

According to the Indian Standards (IS) 2720, Part-XI, FSI was calculated. It can be observed from Table 5.2 that, the FSI values were reduced with increase in curing days. In this test soil passing through 425µ IS sieve was taken which mainly contains silt and clay particles. In Terrabind and FA stabilized samples, swelling was significantly reduced due to Terrabind attack on this clay lattice of the soil, which alters the ionic charge in clay and creates a chemical bond between the clay particles. It reduces shrink and swell by forming a chemical and physical bond between the clay particles that allows the moisture content of the soil to stabilize and reduces the movement of the soil.

5.2.4 Chemical Composition of Treated Soil

The results obtained for chemical analysis for oxides are presented in Table 5.3. Oxides such as SiO_2, Al_2O_3 and CaO showed an increase in concentration when treated, and also reduction is observed in Fe_2O_3. In comparison with untreated soil, treated soil produced higher pH and conductivity. The addition of stabilizer results in the formation of various chemicals which binds the soil particles together creating a crystalline matrix.

Table 5.3 Chemical composition Terrabind and FA treated soil

Oxides (%)	SiO_2	Fe_2O_3	Al_2O_3	S	CaO	MgO	pH	EC (µS/cm)
BC	57.12	6.08	8.05	0.091	0.005	0.013	8.22	1.17
FA	72.08	0.57	5.15	0.043	12.34	4.04	11.47	6.95
BC + T	58.29	2.71	9.75	0.025	0.012	0.017	8.51	1.16
BC + T+ 6% FA	62.89	2.18	10.04	0.052	0.02	0.015	12.43	1.22

T: Terrabind; FA: Fly Ash; SiO_2: Silica oxide; Fe_2O_3: Iron oxide; Al_2O_3: Aluminium oxide; CaO: Calcium oxide; MgO: Magnesium oxide; S: Sulphate; EC: Electrical conductivity

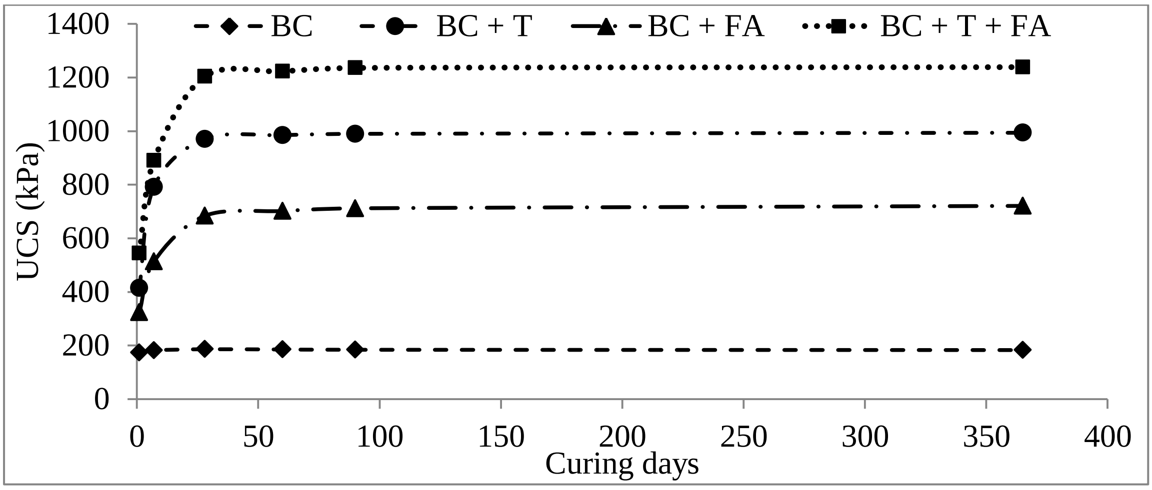
5.2.5 Effect on UCS

In order to simulate the field conditions for pavement construction, the stabilized specimens were prepared at three different moisture contents, at OMC and on the wet and dry sides of OMC (at OMC ± 2%), and the values are presented in

Table 5.4. As observed from the test results, wet and dry side of UCS values has no significant change on strength. From Figure 5.1, it can be observed that UCS increased for stabilized soil samples and showed an increasing trend with curing period. This tendency may be due to the effective cation exchange process in Terrabind samples which generally takes longer period. Increase in strength of these samples is due to the chemical reaction of Terrabind with clay lattice of the soil, which alters the ionic charge in clay and creates a chemical bond between the clay particles.

Table 5.4 UCS Values for untreated and treated specimens

Mix	BC Soil			BC soil + Terrabind			BC soil + 6% FA			BC + Terrabind + 6% FA		
Curing Days	1	7	28	1	7	28	1	7	28	1	7	28
UCS (kPa)												
OMC	173	182	186	415	792	971	324	513	684	544	890	1204
OMC - 2%	175	185	188	313	578	758	291	473	615	452	805	1063
OMC + 2%	168	172	179	289	634	838	264	426	593	410	746	946



BC: Black Cotton soil; T: Terrabind; FA: Fly Ash

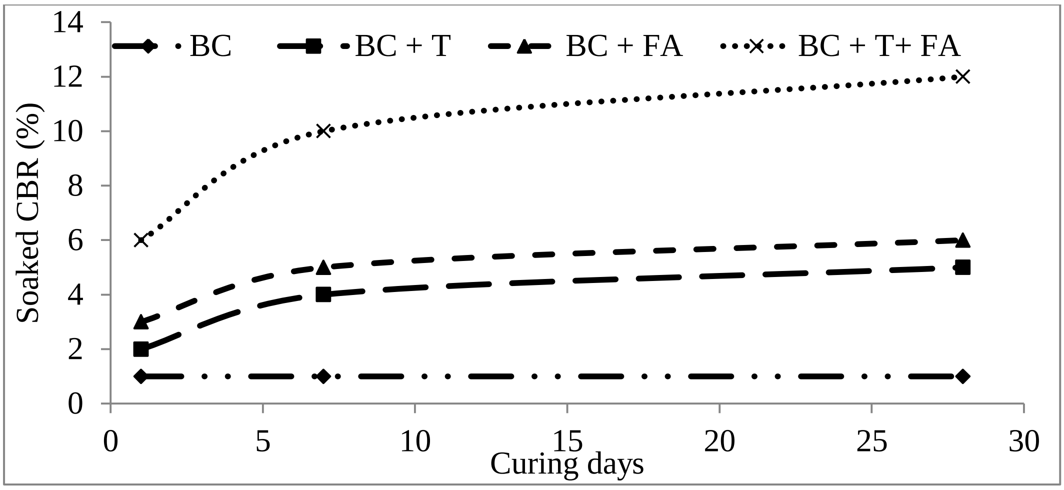
Fig. 5.1 Variation of UCS values at OMC

5.2.6 Effect on CBR

CBR test was conducted on untreated and treated soil specimens in soaked and unsoaked conditions for different curing periods at OMC and OMC ± 2%. From the test results (Table 5.5) it can be observed that, soaked CBR was increased for stabilized soil samples and showed an increasing trend with curing period, and also the dry side of OMC provided better strength than the wet side. For unsoaked condition, the variation in CBR can be attributed to the change in particle size distribution as well as the slight pozzolanic effect of the mix. In soaked condition, the low CBR of BC soil alone is due to the dominance of the clay fraction. The higher CBR of Terrabind and FA is due to its better strength characteristics, primarily because of friction. The increase in soaked CBR with the addition of Terrabind and FA to BC soil is mainly because of two factors: mobilization of frictional resistance and other pozzolanic reaction in the presence of water due to the free lime content in FA (Sivapullaiah et al. 1998). Figure 5.2 depicts the variation of soaked CBR values for different combinations at OMC with curing periods. The soaked CBR of Terrabind and FA samples increased rapidly during the 28 days curing, which is due to the cementation caused by the pozzolanic reaction between the soil particles. For Terrabind and FA mix, the soaked CBR value increased to 12, which is rated as a fair subgrade (IRC:SP:72-2007).

Table 5.5 CBR Values for untreated and treated samples

Curing days	OMC -2%		OMC		OMC +2%	
	Unsoaked	Soaked	Unsoaked	Soaked	Unsoaked	Soaked
BC Soil						
1	25	1	26	1	17	< 1
7	24	2	28	1	19	< 1
28	29	1	31	1	21	< 1
BC + Terrabind						
1	18	2	33	2	15	<1
7	23	3	40	4	14	1
28	26	4	49	5	11	1
BC + FA						
1	22	2	31	3	29	<1
7	32	4	38	5	31	2
28	39	6	42	6	33	3
BC + Terrabind + FA						
1	21	4	47	6	33	3
7	30	6	49	10	37	5
28	43	7	54	12	43	6

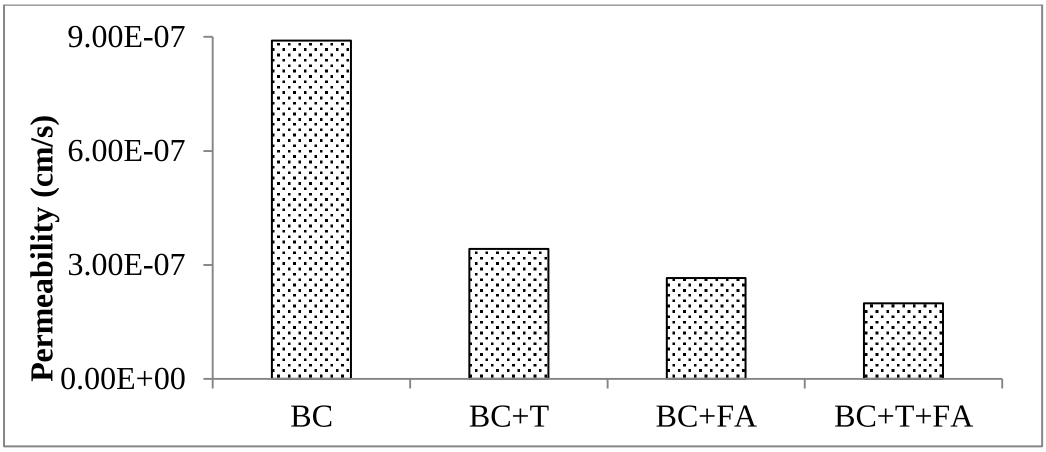


BC: Black Cotton soil; T: Terrabind; FA: Fly Ash

Fig. 5.2 Variation of soaked CBR values at OMC

5.2.7 Coefficient of Permeability

The permeability test was conducted using falling head method for untreated and treated soil mixes, prepared at OMC and MDD conditions. The treated samples were tested at seven days curing period. The treated soil samples were less permeable in nature and the values are depicted in Figure 5.3. Combination of Terrabind and FA shows a significant reduction in permeability values compared to BC soil. It might be the Terrabind liquid works on breaking down the capillary action of soil particles and thus it reduced the moisture retentive nature of BC soil.



BC: Black Cotton soil; T: Terrabind; FA: Fly Ash

Fig. 5.3 Coefficient of permeability values

5.2.8 Major Findings

The important conclusions obtained from the results are summarized as follows:

- Basic geotechnical properties like Atterberg limits, compaction characteristics and permeability improved when the soil was treated with Terrabind and FA.
- The UCS enhancement after 28 days curing, for BC+Terrabind, BC+FA and BC+Terrabind+FA was 4.2, 3.0 and 5.2 times that of the natural BC soil respectively.
- All stabilized soil mixes showed a significant increase in the soaked CBR values, with 2 to 12 times improvement compared to BC soil.
- The Terrabind and FA stabilization control the critical swelling problem of BC soil, by significantly reducing the FSI from 50% to 2%.
- Treatment of Terrabind and FA provided 6 to13 times higher fatigue life to the BC soil.

Considering these findings, it can be concluded that Terrabind and FA combination can be used to stabilize BC soils. Terrabind and FA alone may not provide adequate stabilization, and hence it is recommended to use both the stabilizers in combination for BC soil stabilization. The guidelines by IRC suggest pavement sections for different traffic volumes and subgrade conditions starting from 2% soaked CBR. It also recommends using soil with CBR > 10% as a modified subgrade layer in some cases. In this study, the stabilized soil successfully achieved this criterion and other tests confirmed its suitability as a pavement material.

5.3 BC SOIL STABILIZATION WITH TERRASIL

Terrasil is a water proof chemical, which arrest swelling in soil and makes water proof. In this study along with Terrasil, 2% cement was added for bonding soil particles and to enhance the engineering properties. Treated samples are kept for open air curing and tested for different curing days.

5.3.1 Dosage of Terrasil

General dosage of Terrasil is about one liter chemical per tonne weight of soil. Trials were conducted by treating the soil at 0.8% (dosage 1), 1.2% (dosage 2) and 1.6% (dosage 3) by weight of dry soil and variations in engineering properties were studied. The stabilizer application to soil was done in two stages. First Terrasil was diluted in water at 150% of the OMC and added to the soil, mixed properly and the mixture was kept for air drying, which made the mixture surface 90-95% water resistant. In the second stage, 2% of cement by weight of soil was added before compaction to achieve desired proctor density.

5.3.2 Effect of Terrasil on Consistency Limits

Since the treated samples become impervious at open air curing, the consistency limits were determined immediately after treating. From Table 5.6 it can be observed that, with the addition of chemical, consistency limits got improved, immediately after mixing.

Table 5.6 Variation of consistency limits with dosage

Dosage	LL (%)	PL (%)	PI (%)
0 (0%)	64	31	33
1 (0.8%)	68	53	15
2 (1.2%)	74	62	12
3 (1.6%)	65	52	13

5.3.3 Effect of Terrasil on FSI

It can be observed from Table 5.7 that, the FSI values were reduced with the increase in curing days. In this test, soil passing through 425µ IS sieve was taken which mainly contains silt and clay particles. In Terrasil with 2% cement stabilized specimens, water loving silonal groups of silt and clay converted to highly stable water repellent alkyl siloxane bonds, resulting to a significant reduction in swelling, from 50% to 4%, in 28 days of curing period.

Table 5.7 FSI values for treated soil

Dosage (%)	FSI (%)		
	7 days	14 days	28 days
0 (0%)	50	50	50
1 (0.8%)	15	11	09
2 (1.2%)	08	05	05
3 (1.6%)	06	04	04

5.3.4 Effect of Terrasil on Compaction Test

Modified compaction was conducted on treated soil samples and the results are presented in Table 5.8. Since open air curing is adopted for this study, compaction tests were conducted immediately after treating with chemical and hence only marginal difference was observed.

Table 5.8 Modified compaction test results for treated soil

Dosage	MDD (g/cc)	OMC (%)
1	1.70	17.50
2	1.73	17.20
3	1.74	17.00

5.3.5 Chemical Composition of Treated Soil

The oxides like SiO_2, Al_2O_3 and CaO percentage has been improved after stabilizing with Terrasil and the results are tabulated in Table 5.9.

Table 5.9 Chemical composition of Terrasil treated BC soil

Oxides (%)	SiO_2	Fe_2O_3	Al_2O_3	S	CaO	MgO	pH	EC (μS/cm)
BC soil	57.12	6.08	8.05	0.091	0.0045	0.013	8.22	1.17
BC + Terrasil	63.09	1.98	9.23	0.031	0.021	0.019	10.2	1.52

SiO_2: Silica oxide; Fe_2O_3: Iron oxide; Al_2O_3: Aluminium oxide; CaO: Calcium oxide; MgO: Magnesium oxide; S: Sulphate; EC: Electrical conductivity

5.3.6 Effect of Terrasil on UCS Test

The soil samples prepared at modified compaction MDD and OMC were used and open air curing was done for 7, 14, 28, 60, 90 and 365 days for the chemically treated samples. From Figure 5.4 it is evident that, as the dosage increases, the UCS increases up to 28 days curing and beyond that the change is marginal. The UCS value increase up to dosage 2 and further addition of stabilizer marginally affects the UCS values. The increase is due to the chemical reaction with the soil particles which water proofs the surfaces permanently and increases the load bearing capacity of soil.

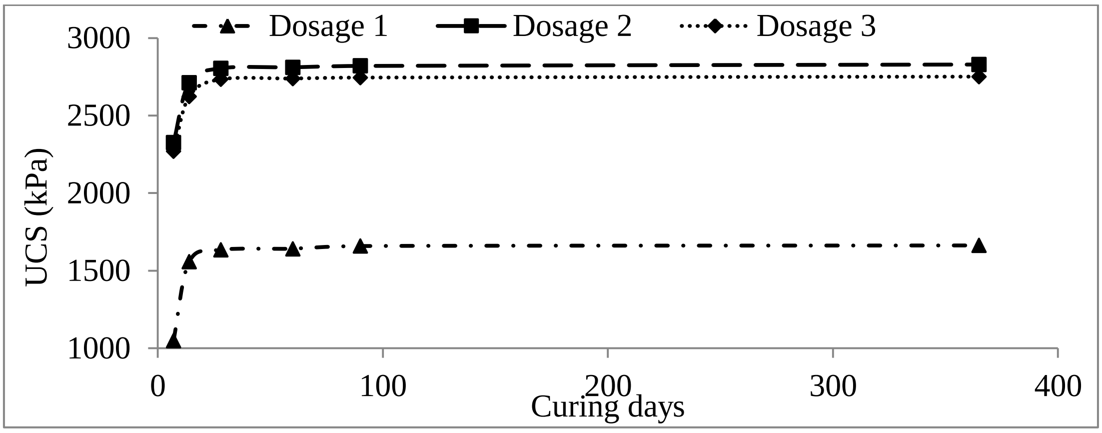


Fig. 5.4 UCS values for treated samples

5.3.7 Effect of Terrasil on CBR test

The CBR tests were performed on soaked and un-soaked specimens for modified proctor densities. Since the soaked values of CBR are generally considered for designs, the open air cured samples were soaked for four days before testing. The curing periods adopted were 7, 14 and 28 days. The samples after 7 days open air curing is shown in Figure 5.5. After the specified curing was over, the CBR specimens were taken out and tested and the results are presented in Figure 5.6.

Fig. 5.5 CBR treated samples after seven days open air curing

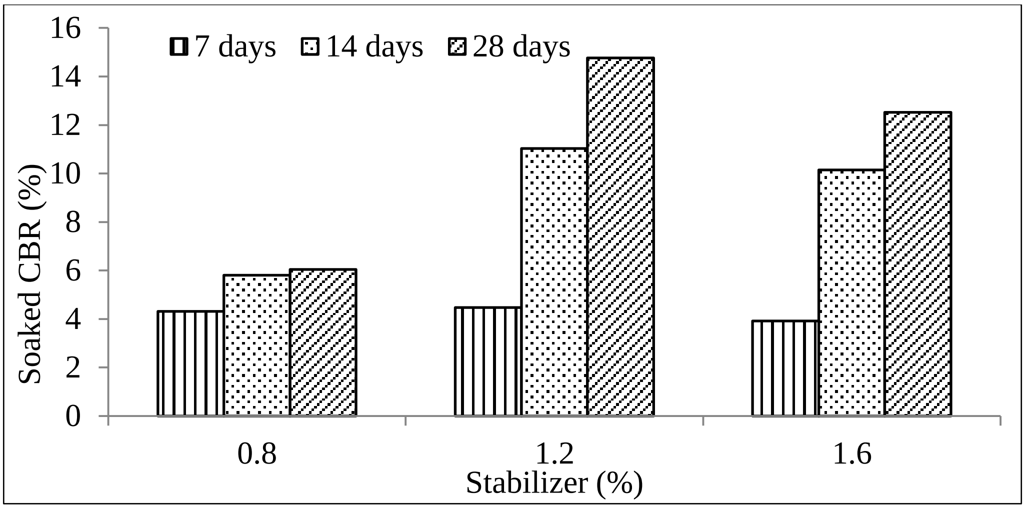


Fig. 5.6 Variation of CBR values for different percentage of stabilizer for soaked condition

5.3.8 Effect of Terrasil on Coefficient of Permeability

Permeability tests were carried out on BC soil with different chemical dosages and the test results are tabulated in Table 5.10. It indicates that, as the dosage increases, there is a drastic decrease in permeability and the soil becomes impermeable. The chemical reaction leads to permanent siliconization of the surfaces by converting the water loving silanol groups to water repellent siloxane bonds and this made the soil waterproof.

Table 5.10 Permeability test results for untreated and treated soils

Dosage	Coefficient of Permeability (cm/sec)
0	5.8×10^{-8}
1	Impermeable
2	Impermeable
3	Impermeable

5.3.9 Major Findings

The major findings from the study are:

- The increment in dosages resulted in the decrement of consistency limits. So it is clear that the chemical makes the soil stiff.
- It is observed that UCS strength increases with increase in dosage of stabilizer

and curing period. It is also observed that, dosage 2 shows a good increment, but further increase of dosage results in a marginal improvement of strength. Hence the dosage 2 is considered as the optimum chemical dosage.

- The CBR values increase with the increase in percentage of stabilizer.
- For dosage 2, 4 – 14 times CBR improvement is observed for different curing days.
- The soil was found to be impermeable with stabilization.

5.4 BC SOIL STABILIZATION WITH RBI 81

BC soil was mixed with different dosages (2, 4, 6 and 8 % by weight of soil) of RBI 81 to evaluate the engineering properties. The goal was to find the optimum amount of RBI 81 for pavement construction.

5.4.1 Effect of RBI 81 on Consistency Limits

Consistency tests are conducted immediately after mixing soil with the RBI 81. LL decreased and PL increased for treated soil as seen from Table 5.11. Increase in LL and a decrease in PL generally indicates an increase in strength. PI values were decreased as the increase in RBI 81 dosage, which ensures that the treated soil will exhibit improvement in strength and texture.

Table 5.11 Consistency limits for treated soil

Mix	LL (%)	PL (%)	PI (%)
BC soil	64	31	33
BC soil + 2% RBI 81	63	37	26
BC soil + 4% RBI 81	61	42	19
BC soil + 6% RBI 81	59	45	14
BC soil + 8% RBI 81	57	47	10

5.4.2 Effect of RBI 81 on Compaction test

The OMC and MDD of treated soil were determined immediately after mixing with RBI 81. As the dosage increases change in OMC and MDD was obtained as tabulated in Table 5.12. If the treated soil is kept open for a long time, it may become hard after a few minutes due to the cementitious action. The OMC shows slight increase for 2% stabilizer and then shows a gradual decrease with higher percentages of the stabilizer. The increase in the fine content resulting from the inclusion of RBI 81 led to more demand for water necessary to hydrate the particles in the soil, resulting in higher OMC values. On the other hand, the marginal increase in the MDD of samples is due to the immediate reactions between RBI 81 and soil, which is represented by the flocculation and agglomeration.

Table 5.12 Modified compaction test results for treated soil

Mix	MDD (g/cc)	OMC (%)
BC soil	1.77	16.32
BC soil + 2% RBI 81	1.78	18.11
BC soil + 4% RBI 81	1.78	21.04
BC soil + 6% RBI 81	1.80	19.41
BC soil + 8% RBI 81	1.82	17.10

5.4.3 Effect of RBI 81 on FSI

The FSI test was conducted immediately after mixing soil with RBI 81. The results tabulated in Table 5.13 indicate that, as the RBI 81 dosage increases, the swelling has been reduced to 9% from 50%. Reduction in swelling is due to the calcium ions which are present in RBI 81, leading to cation exchange when mixed in soil and water.

Table 5.13 FSI values for treated soil

Mix	FSI (%)
BC soil	50
BC soil + 2% RBI 81	39
BC soil + 4% RBI 81	23
BC soil + 6% RBI 81	11
BC soil + 8% RBI 81	09

5.4.4 Chemical Composition of Treated Soil

Variations of oxides in treated soil, as tabulated in Table 5.14, ensure that the RBI 81develops strong bond between the RBI 81 and clay particles, forming the products into high strength gain.

Table 5.14 Chemical composition of RBI 81 treated BC soil

Oxides (%)	SiO_2	Fe_2O_3	Al_2O_3	S	CaO	MgO	pH	EC (µS/cm)
BC soil	57.12	6.08	8.05	0.091	0.0045	0.013	8.22	1.17
RBI 81	48.20	1.65	6.23	0.48	53.8	0.9	10.98	1.29
BC soil+ RBI 81	58.31	5.49	16.71	0.59	38.06	1.89	17.98	1.37

SiO_2: Silica oxide; Fe_2O_3: Iron oxide; Al_2O_3: Aluminium oxide; CaO: Calcium oxide; MgO: Magnesium oxide; S: Sulphate; EC: Electrical conductivity

5.4.5 Effect of RBI 81 on UCS

Soil samples were prepared for modified compaction at OMC and MDD. UCS test results are presented in Table 5.15. It can be observed that as the dosage increases strength also increases. The most rapid strength increases occur between one month, and smaller strength gain after this period is due to continued hydration and formation of cementitious material which continues for several months. From an economic point of view and to avoid excess use of stabilizer, 6% RBI 81 has considered as the optimum dosage. Curing period also plays an important role in strength gain.

Table 5.15 UCS test results for treated soils

Curing days	UCS (kPa)			
	2% RBI 81	4% RBI 81	6% RBI 81	8% RBI 81
7	598	980	1250	1438
14	830	1350	1800	2123
28	949	1476	2045	2430
60	978	1557	2189	2536

5.4.6 Effect of RBI 81 on CBR

The improvement of CBR values of soil mixtures due to application of RBI 81 is tabulated in Table 5.16. The addition of RBI 81 has increased the CBR of the mixes continuously for all the curing periods. The hydration of RBI 81 it forms calcium silicate hydrate gel and with the addition of increasing amounts of RBI 81, more of this gel will be formed, ultimately resulting in increased strength.

Table 5.16 CBR test results for treated soils under modified compaction

Curing days	Soaked CBR (%)			
	2% RBI 81	4% RBI 81	6% RBI 81	8% RBI 81
7	4	5	9	12
14	7	10	14	17
28	9	12	17	19

5.4.7 Major Findings

The major observations from the study are listed below:

- With the increase in the stabilizer content, both PI and FSI values are reduced.
- The UCS showed an increase with RBI 81 content and curing period, but major improvement was limited to 28 days.
- BC soil treated with 6% RBI 81 offers good improvement in CBR strength, with 1 – 17 times improvement for optimum dosage.
- Evidence for stabilization can be seen from change in chemical composition of soils when treated with the stabilizer. Percentages of Calcium Oxide, Alumina and sulphates, which are important byproducts, are increased on stabilization.

CHAPTER 6

DURABILITY STUDIES

Highway engineers have recognized that variation in climatic conditions is a major factor affecting the pavement performance and these variations are generally resulted from WD and FT actions, or a combination of these actions. The influence of climatic changes on a pavement structure indicates their effects on the materials in the structure and this led to studies to assess the performance of pavement materials under various weathering actions. Researchers have been giving importance to correctly interpret the behavior of stabilized soils under the influence of FT and WD mechanisms.

Durability is defined as the ability of a material to retain stability and integrity over years of exposure to the destructive forces of weathering (Dempsy and Thompson 1968). A good soil stabilizer should not only provide initial strength gain, but it should maintain the strength characteristics by retaining the bonding with soil under seasonal cyclic changes and adverse conditions. Hence, checking the durability is vital before recommending any stabilizer for practical applications. There are mainly two tests for durability, the WD and FT test, as per ASTM D 559 and 560 for cement stabilized soil. Since there are no standard procedures to evaluate the durability of non traditional stabilizers, the ASTM suggested methods were adopted in the current study for all stabilized soils. This chapter deals with durability characteristics of Lateritic and Black cotton soil stabilized with different stabilizers. The UCS test specimens stabilized with different stabilizer dosages and moist cured for seven days were subjected to durability tests.

6.1 IMPORTANCE OF DURABILITY STUDIES

Each stabilizer has improved the quality of soil which enhanced with time progresses. The ability of a stabilizer to maintain desired properties over the life of a pavement is also one of the important requirements in a stabilization mechanism. The durability of stabilized materials is a major concern in cold regions and heavy rainfall areas, due to both frost heave and freeze-thaw cycling.

6.2 SAMPLE PREPARATION

UCS specimens, with 38mm diameter and 78mm height, were selected for durability test. Three replicate specimens were prepared for each specific combination and were moist cured for 7 days at room temperature. The durability of stabilized specimens is determined using a sequence of wet-dry and freeze-thaw cycles. It consumes 45 to 50 days to complete the test with the specified 12 durability cycles and the weight loss after 12 cycles should be less than 14%. Figure 6.1 depicts the specimen during wetting, drying and thawing time.

Durability has been checked for all the stabilizers and the detailed results are as follows.

- LS1 with Terrasil
- LS1 with Terrabind
- LS1 with cement and Arecanut coir
- LS2 with cement
- LS2 with RBI 81
- BC soil with Terrabind
- BC soil with Terrasil
- BC soil with RBI 81

Fig. 6.1 Durability specimens during wet-dry and thawing cycles

6.3 WET DRY CYCLES

The wet dry test results for stabilized lateritic soil (LS1 and LS2) are presented in Table 6.1. LS specimens without stabilization failed in the first wet cycle itself, but most of the treated specimens with 7 days curing performed better. The study reveals that, Terrasil and Terrabind chemicals could not sustain for more than five WD cycles, whereas in Cement with 1% Arecanut coir, Cement (for 6, 9 and 12 %) and RBI 81 (for 6 and 8 %) stabilized specimens could pass all the 12 cycles. The soil-cement-coir mixtures passed the WD criteria only for mixture with 1.0% coir. The pozzolanic reaction of cement and RBI 81 in the presence of water resulted in positive effects, which help to the specimens to achieve the criterion of 12 WD cycles. It is also noted that, only cement stabilized soil showed weight loss within 14% after 12 cycles. The negative values for weight loss corresponding to some wet cycles, as shown in Table, actually indicates increase in weight due to the absorption of water during wetting (A sample calculation is presented in the Appendix II).

Table 6.2 presents the WD test results for BC soil, and shows that Terrabind and Terrasil chemicals could complete only a maximum of three WD cycles without failure. But higher dosages of RBI 81 treatment help soil to withstand till 11 cycles in WD test.

As the dosage of cement and RBI 81 increases durability of soil increases and cementation in the soil matrix improves. At lower dosages the bond between chemical and soil is low and it could not sustain durability test. As the dosage increases the chemical bond between soil properties and chemical increases and can sustain durability test.

6.4 FREEZE THAW CYCLES

The FT test results for stabilized specimens for both LS and BC soils are tabulated in Tables 6.3 and 6.4. Specimens could withstand the 12 cycles within 14% weight loss for all mixtures. The negative weight loss values for some FT cycles indicates the increase in weight. In case of LS, specimens with coir and RBI 81 showed comparatively higher weight loss due to the scaling caused by fibers during thawing. Among all stabilizers, cement produced the minimum weight loss for all dosages and cycles.

Table 6.1 WD cycles percentage weight loss for stabilized lateritic soil

Specimens	cycles	1	2	3	4	5	6	7	8	9	10	11	12
LS1+ 0.4% Terrasil + 2% C	W	10.67	Collapsed										
	D	14.48											
LS1+ 0.8% Terrasil + 2% C	W	11.23	Collapsed										
	D	16.74											
LS1+ 1.2% Terrasil + 2% C	W	-3.17	1.07	3.84	5.21	7.59	9.73	Collapsed					
	D	6.98	7.15	8.41	9.72	10.36	12.36						
LS1+ 1.6% Terrasil + 2% C	W	-9.91	-7.49	-3.89	5.37	5.96	8.38	14.87	37.94	37.29	46.08	47.26	49.09
	D	2.07	3.66	9.09	10.97	13.33	27.85	31.56	43.13	44.07	52.86	55.58	57.35
LS1 + Terrabind + 6% FA	W	-8.94	-9.13	-4.56	6.18	6.96	7.48	15.01	22.91	27.14	34.86	42.81	50.12
	D	1.97	2.84	8.12	9.85	12.18	26.74	29.65	35.32	42.12	49.12	56.01	60.48
LS1+3%C+0.2% AC	W	-2.04	Collapsed										
	D	8.98											
LS1+3%C+0.4% AC	W	-1.78	0.67	Collapsed									
	D	9.93	12.94										
LS1+3%C+0.6% AC	W	-1.21	0.58	0.94	Collapsed								
	D	10.23	11.56	14.32									
LS1+3%C+0.8% AC	W	-1.86	-2.12	1.28	5.64	7.61	9.85	Collapsed					
	D	7.69	8.95	10.24	12.38	14.56	16.58						
LS1+3%C+ 1% AC	W	-2.36	-3.26	-2.52	0.50	1.13	1.92	2.74	3.57	9.67	10.25	11.18	12.66
	D	8.91	9.16	10.72	13.33	14.68	15.24	17.21	19.97	20.12	21.45	21.98	22.23
LS2+3% C	W	3.36	3.13	3.17	2.66	2.52	2.60	Collapsed					
	D	8.29	8.74	8.44	8.74	9.14	9.93						

LS2+6% C	W	2.11	0.97	2.36	1.98	1.90	1.95	0.28	0.01	0.14	0.28	0.30	0.38
	D	8.76	8.97	8.54	8.95	9.25	8.98	8.78	8.58	8.48	9.07	9.21	9.30
LS2+9% C	W	2.45	2.59	2.70	2.33	2.35	2.38	2.23	2.31	2.43	2.15	2.10	2.10
	D	8.98	9.22	8.78	9.06	9.41	9.16	8.98	8.75	8.64	8.69	8.80	8.89
LS2+12% C	W	2.95	3.01	3.24	2.91	2.89	3.00	2.82	2.91	3.02	2.80	2.74	2.75
	D	9.23	9.39	8.80	9.16	9.60	9.29	9.10	8.76	8.74	9.12	9.24	9.34
LS2+2% RBI 81	W	-3.27	Collapsed										
	D	8.22											
LS2+4% RBI 81	W	-3.29	-2.56	Collapsed									
	D	6.22	11.09										
LS2+6% RBI 81	W	-3.72	-2.42	-1.92	-2.24	-0.77	0.07	0.39	1.92	3.99	5.96	6.87	7.34
	D	8.52	8.79	8.52	9.24	10.14	11.86	12.28	13.25	13.65	14.21	15.67	15.98
LS2+8% RBI 81	W	-2.19	-2.81	-1.14	-2.39	-0.24	-2	-1.31	-0.14	0.83	1.62	3.52	5.62
	D	7.35	9.57	10.86	11.66	12.43	10.17	11.16	12.32	13.48	15.15	17.59	19.4

LS2: Lateritic soil 2; W: Wet; D: Dry; C: Cement; AC: Arecanut Coir; RBI: Road Building International grade 81

Table 6.2 WD cycles percentage weight loss for stabilized BC soil specimens

Specimens	cycles	1	2	3	4	5	6	7	8	9	10	11	12
BC Soil	W	Collapsed											
	D												
BC + Terrabind	W	14.28	19.67	Collapsed									
	D	16.34	22.34										
BC + Terrabind+ 6% FA	W	10.32	13.81	15.02	17.34	Collapsed							
	D	11.76	14.29	16.43	19.52								
BC + 0.8% Terrasil+ 2% C	W	14.21	18.32	Collapsed									
	D	15.82	20.63										
BC + 1% Terrasil+ 2% C	W	12.00	15.82	18.98	Collapsed								
	D	14.32	17.40	20.21									
BC + 1.2% Terrasil+ 2% C	W	11.23	13.14	16.67	Collapsed								
	D	12.67	15.20	19.30									
BC +2% RBI 81	W	Collapsed											
	D												
BC + 4% RBI 81	W	12.87	15.89	Collapsed									
	D	14.57	18.31										
BC + 6% RBI 81	W	9.43	11.61	12.94	14.51	14.97	15.43	16.04	17.23	Collapsed			
	D	10.32	13.02	14.32	15.43	16.21	16.99	17.42	18.76				
BC + 8% RBI 81	W	6.12	7.45	9.31	10.74	11.23	12.01	12.78	13.69	14.72	15.03	Collapsed	
	D	7.89	8.32	10.42	11.92	12.89	13.29	13.97	14.85	15.10	16.21		

BC: Black Cotton soil; W: Wet; D: Dry; C: Cement; FA: Fly Ash; AC: Arecanut Coir; RBI: Road Building International grade 81

Table 6.3 FT cycles percentage weight loss for stabilized lateritic specimens

Specimens	cycles	1	2	3	4	5	6	7	8	9	10	11	12
LS1+ 0.4% Terrasil + 2% C	F	0.30	0.42	0.48	0.71	0.88	1.63	1.81	2.04	2.33	2.56	2.73	2.73
	T	0.35	0.30	0.36	0.48	0.77	0.94	1.58	1.87	2.21	2.45	2.62	2.56
LS1+ 0.8% Terrasil + 2% C	F	-0.13	-0.07	-0.02	0.08	0.37	0.49	0.61	0.78	1.02	1.13	1.31	1.66
	T	-0.23	-0.10	-0.09	0.02	0.25	0.37	0.55	0.61	0.90	1.02	1.08	1.43
LS1+ 1.2% Terrasil + 2% C	F	0.13	0.20	0.20	0.20	0.38	0.62	0.80	0.50	0.80	1.09	1.33	1.57
	T	-0.05	0.05	0.09	0.13	0.14	0.32	0.56	0.62	0.68	0.97	1.21	1.39
LS1+ 1.6 % Terrasil + 2% C	F	-0.20	-0.02	0.04	0.07	0.36	0.60	0.77	1.07	1.19	1.37	1.60	2.02
	T	-0.06	-0.06	-0.01	0.01	0.24	0.48	0.60	0.77	1.07	1.19	1.37	1.84
LS1 + Terrabind + 6% FA	F	0.08	1.91	1.81	2.07	2.41	2.94	3.02	3.48	3.94	4.23	4.37	4.63
	T	1.99	2.04	3.44	3.51	3.62	3.69	3.94	4.12	4.26	4.34	4.57	4.98
LS1+3%C+0.2% AC	F	3.44	3.64	4.18	4.47	4.94	5.14	5.87	6.26	6.72	6.93	7.19	7.76
	T	3.54	3.95	4.32	4.62	4.98	5.38	5.98	6.45	6.96	7.24	7.42	7.88
LS1+3%C+0.4% AC	F	3.21	2.99	3.03	3.22	2.87	3.46	3.52	3.97	4.34	4.24	5.44	5.47
	T	2.62	3.47	3.07	3.33	3.11	3.47	3.85	4.17	4.81	4.94	5.45	5.67
LS1+3%C+0.6% AC	F	3.53	3.03	3.81	3.80	4.08	4.27	4.51	4.81	5.33	5.26	5.78	5.82
	T	3.06	3.43	3.77	4.00	4.34	4.63	5.01	5.28	5.34	5.39	5.85	5.88
LS1+3%C+0.8% AC	F	2.01	2.42	2.74	2.68	2.93	3.01	3.23	3.35	3.60	3.66	3.82	3.90
	T	2.45	2.60	2.83	2.89	2.99	3.23	3.37	3.50	3.71	3.81	3.83	4.35
LS1+3%C+ 1% AC	F	0.20	0.62	0.94	0.88	1.11	1.19	1.41	1.54	1.81	1.87	2.03	2.12
	T	0.25	0.39	0.63	0.69	0.79	1.04	1.18	1.58	1.53	1.62	1.64	2.28
LS2+3% C	F	-0.48	-0.47	-0.22	-0.10	0.07	-0.06	0.46	1.04	1.40	0.90	0.28	0.01
	T	-0.03	-0.21	-0.19	0.37	0.45	0.62	1.49	2.39	1.88	1.40	1.37	1.18
LS2+6% C	F	-0.50	-0.33	0.26	0.35	0.65	0.59	1.67	1.64	1.64	1.17	1.45	1.18
	T	0.11	0.07	0.55	1.03	1.09	1.59	1.99	2.24	1.97	1.70	1.57	1.20

LS2+9% C	F	-0.46	-0.28	0.47	0.37	0.49	0.28	1.15	1.14	1.44	1.20	1.08	1.06
	T	0.03	0.15	0.67	0.71	0.72	1.15	1.51	1.76	1.64	1.57	1.48	1.29
LS2+12% C	F	-0.32	-0.17	0.28	0.14	0.34	0.31	0.85	1.28	1.93	1.78	1.67	1.62
	T	0.18	0.19	0.47	0.59	0.68	0.93	1.68	2.19	2.20	1.78	1.74	1.73
LS2+2% RBI 81	F	0.31	0.52	1.08	1.38	1.86	2.07	2.82	3.22	3.70	3.91	4.19	4.77
	T	0.41	0.84	1.22	1.53	1.90	2.32	2.93	3.42	3.94	4.24	4.42	4.89
LS2+4% RBI 81	F	-0.06	-0.28	-0.24	-0.05	-0.40	0.20	0.27	0.74	1.12	1.02	2.25	2.28
	T	-0.66	0.22	-0.20	0.07	-0.16	0.22	0.61	0.94	1.60	1.74	2.27	2.49
LS2+6% RBI 81	F	0.32	-0.20	0.61	0.59	0.88	1.08	1.33	1.63	2.17	2.11	2.64	2.68
	T	-0.17	0.21	0.56	0.80	1.15	1.45	1.84	2.12	2.18	2.24	2.71	2.74
LS2+8% RBI 81	F	0.06	0.48	0.81	0.74	1.00	1.08	1.30	1.43	1.68	1.74	1.91	1.99
	T	0.51	0.66	0.89	0.96	1.06	1.31	1.45	1.58	1.80	1.90	1.92	1.99

LS2: Lateritic soil 2; F: Freeze; T: Thaw; C: Cement; AC: Arecanut Coir; RBI: Road Building International grade 81

Table 6.4 FT cycles percentage weight loss for stabilized BC soil specimens

Specimens	Cycles	1	2	3	4	5	6	7	8	9	10	11	12
BC Soil	F	0.31	4.90	5.89	7.13	7.51	7.82	7.38	8.00	7.38	8.00	9.68	8.56
	T	5.65	6.95	7.75	8.19	8.13	8.00	8.62	7.94	8.81	9.00	9.24	8.81
BC + Terrabind	F	0.06	1.81	1.81	2.57	2.91	3.01	3.26	3.58	3.95	4.12	4.26	4.74
	T	2.06	2.12	3.44	3.62	3.81	3.98	3.99	4.25	4.68	4.93	5.12	5.36
BC Soil + Terrabind+6% FA	F	0.19	2.35	2.80	3.25	3.56	3.75	3.98	4.21	4.39	4.51	4.59	4.70
	T	1.98	3.43	3.81	3.88	3.94	4.01	4.26	4.67	4.81	4.96	5.04	5.16
BC + 0.8% Terrasil+ 2%C	F	4.52	4.73	5.26	5.55	6.01	6.21	6.93	7.31	7.77	7.97	8.24	8.79
	T	4.62	5.03	5.40	5.69	6.05	6.45	7.03	7.51	8.00	8.29	8.47	8.91
BC + 1 % Terrasil+ 2%C	F	4.11	3.89	3.93	4.12	3.78	4.35	4.42	4.87	5.23	5.13	6.32	6.35
	T	3.53	4.37	3.97	4.23	4.01	4.37	4.75	5.06	5.69	5.82	6.33	6.55
BC + 1.2% Terrasil+ 2%C	F	1.40	0.89	1.69	1.68	1.96	2.16	2.40	2.71	3.24	3.17	3.70	3.74
	T	0.92	1.30	1.65	1.88	2.23	2.53	2.91	3.19	3.25	3.30	3.77	3.80
BC+2% RBI 81	F	4.90	5.11	5.64	5.92	6.38	6.58	7.30	7.68	8.13	8.34	8.60	9.16
	T	5.00	5.41	5.77	6.07	6.42	6.82	7.40	7.87	8.37	8.65	8.83	9.27
BC+4% RBI 81	F	3.99	3.77	3.81	4.00	3.66	4.23	4.30	4.75	5.11	5.02	6.20	6.23
	T	3.40	4.25	3.85	4.11	3.89	4.25	4.63	4.94	5.57	5.70	6.21	6.43
BC+6% RBI 81	F	2.63	2.13	2.91	2.90	3.18	3.38	3.62	3.92	4.44	4.38	4.90	4.94
	T	2.15	2.53	2.87	3.10	3.44	3.74	4.12	4.39	4.45	4.51	4.97	4.99
BC+8% RBI 81	F	1.54	1.96	2.28	2.21	2.47	2.55	2.77	2.89	3.14	3.20	3.36	3.44
	T	1.99	2.13	2.36	2.43	2.53	2.77	2.91	3.04	3.25	3.35	3.37	3.89

BC: Black Cotton soil; F: Freeze; T: Thaw; C: Cement; FA: Fly Ash; AC: Arecanut Coir; RBI: Road Building International grade 81

6.5 UCS STRENGTH ON WD SPECIMENS

The specimens completed 12 cycles of WD, were tested for UCS. It was observed that UCS values were decreased after WD cycles compared to the normal soil specimens after seven days curing, as listed in Table 6.5. For some deteriorated specimens the UCS test could not be performed, and the strengths were considered as negligible. The untreated specimens failed during the initial soaking period required before the commencement of the first WD cycle. The strength loss was observed to be less in the case of cement treated specimens due to the more effective pozzolanic reaction compared to specimens with RBI 81, and with the increase in stabilizer dosage the reduction in UCS was decreased.

Table 6.5 UCS values after WD cycles

Specimens	UCS (kPa)		% loss in strength due to WD cycles
	Normal	After 12 WD Cycles	
LS2 + 6% C	1858	1498	19.38
LS2 + 9% C	2077	1832	11.80
LS2 + 12% C	2164	2014	6.93
LS2 + 6% RBI 81	2507	1954	22.06
LS2 + 8% RBI 81	2685	2176	18.96

LS2: Lateritic soil 2; W: Wet; D: Dry; C: Cement; RBI: Road Building International grade 81

6.6 MAJOR FINDINGS

Major observations made from the durability studies of the stabilized soil are as follows:

- Natural soil exhibits no strength resistance against environmental factors and failed rapidly during soaking.
- The variations in weight are less in the wetting cycles and but it increases during drying.
- All stabilized soil specimens withstood 12 freeze and thaw cycles and weight loss was within 14%, satisfying the requirements.

- Terrasil and Terrabind chemicals could not complete the 12 cycles of WD, since they only make soil surface water proof, without providing much gain in strength.
- Cement and RBI 81 treated specimens could withstand 12 cycles of WD for higher percentage of stabilizers (6% above), due to their pozzolanic action.

CHAPTER 7

IDT STRENGTH AND FATIGUE STUDIES

7.1 INDIRECT TENSILE (IDT) STRENGTH TEST

In order to observe the behaviour of lateritic soil, soil-cement and soil-cement-aggregate mixtures on the tensile force, IDT strength test, specified by ASTM D 6931 was performed. This test is undertaken to determine the tensile strength of cylindrical specimens by applying a compressive load using a loading strip along its diametrical plane, in which the specimen is placed with its axis horizontal, and the loading is continued till failure. Cylindrical specimens were prepared at modified OMC and MDD conditions with a size of 68 mm height and 100 mm diameter and the loading was applied at the rate of 1 mm/min. The IDT strength setup and failed sample are shown in Figure 7.1.

Fig. 7.1 IDT strength set up and failed specimen

The IDT strength value (kPa) was determined by using an Equation:

$$IDT = \frac{2000\mathrm{P}}{\pi \mathrm{dt}} \tag{7.1}$$

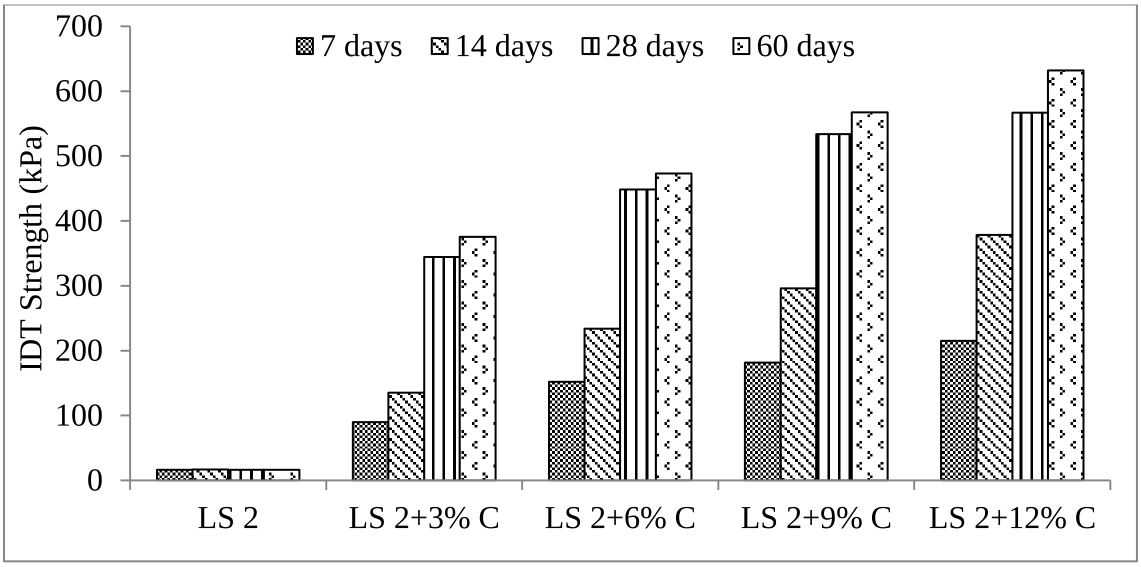
Where,

P = Ultimate load at which failure of sample occurred in N,

t = Thickness of specimen = 68 mm,

d = Diameter of specimen = 100 mm.

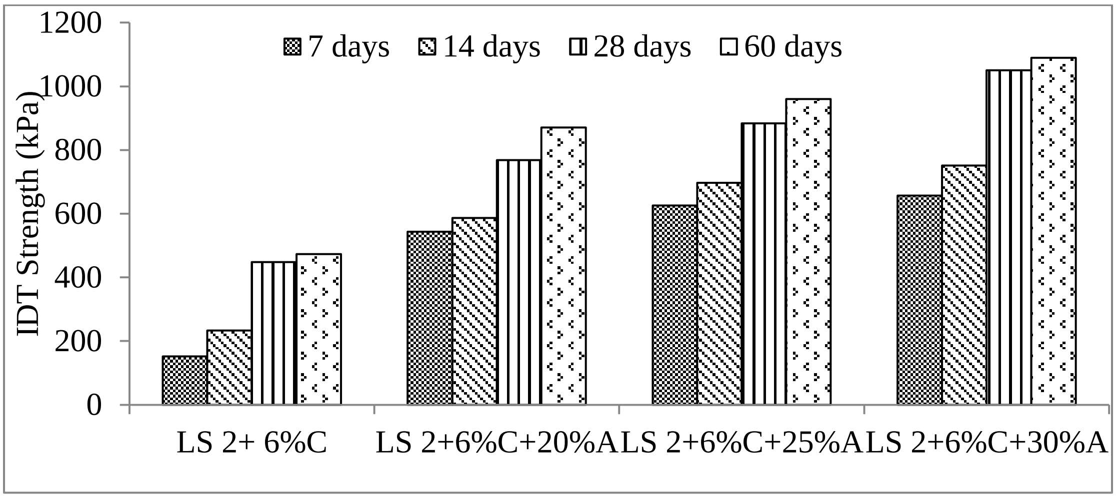
IDT strength was determined for treated and untreated soil specimens and values are depicted in Figure 7.2. It is observed that the strength tremendously increased with the addition 3% cement and it further increased with cement content. Curing period is found to be an important parameter for strength increase for cement stabilized specimens. The natural soil had no changes with curing days whereas the cement treated soil exhibited good strength from 90 to 600 kPa for different cement contents.



LS2: Lateritic Soil 2; C: Cement

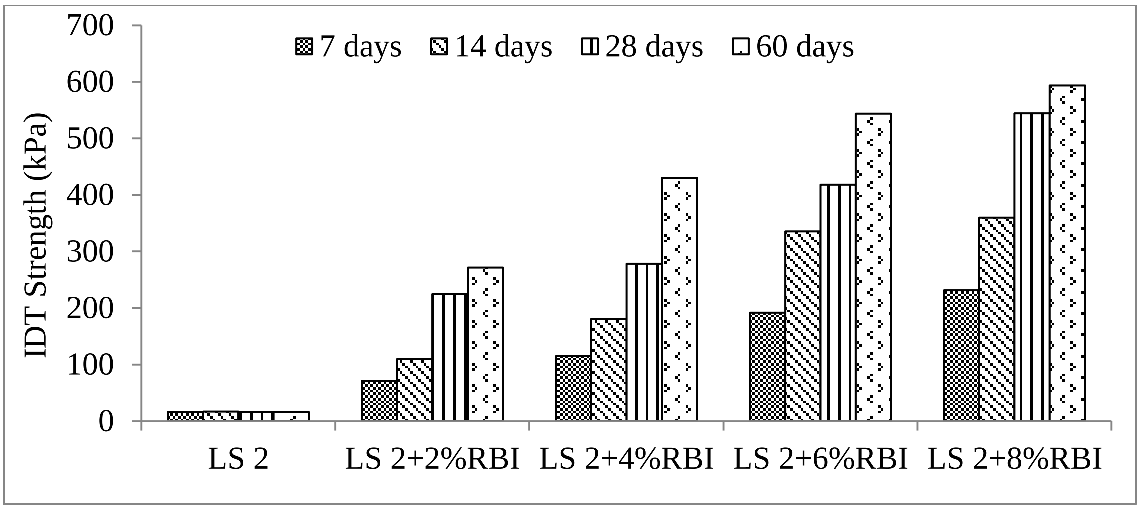
Fig. 7.2 Variation of IDT strength for cement specimens

Cementitious stabilized soil materials are usually characterized with good tensile strength and stiffness properties which are beneficial for pavement design. Figure 7.3 depicts the variation in IDT strength values for cement and aggregate stabilized specimens. For 6 % cement content different percentage of aggregates were added and tested for different curing days. IDT strength for soil cement aggregate mix significantly increased from 160 to 1090 kPa and hence it can be used as a pavement material.



LS2: Lateritic Soil 2; C: Cement; A: Aggregate

Fig. 7.3 Variation of IDT strength for cement aggregate specimens

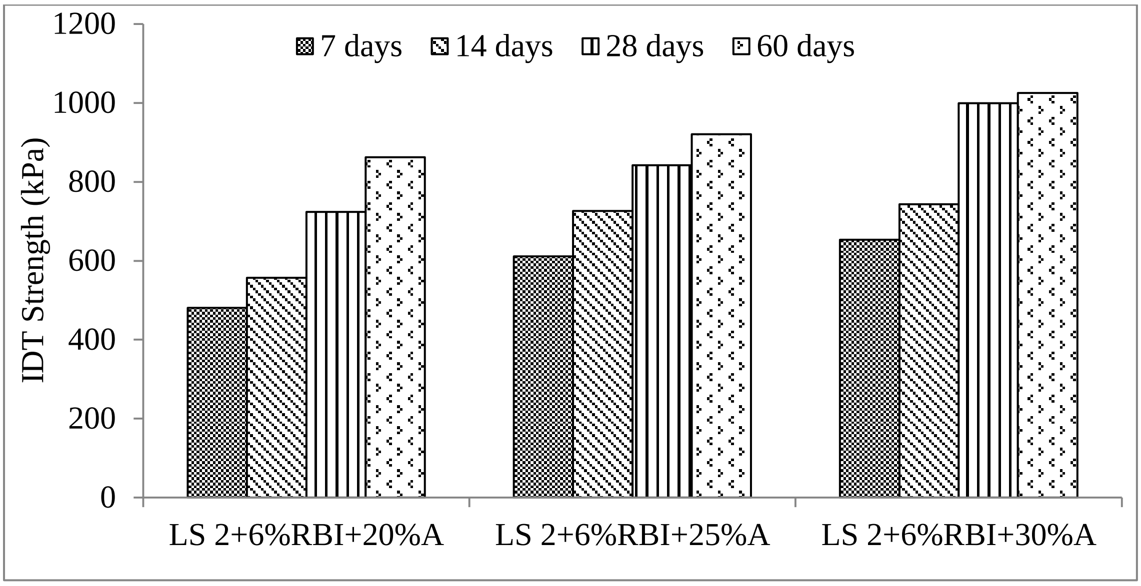


LS2: Lateritic Soil 2; RBI: Road Building International grade 81

Fig. 7.4 Variation of IDT strength for RBI 81 specimens

The IDT values of the RBI 81 stabilized soil ranged widely from about 70 to 590 kPa for varying dosages and curing periods, as observed in Figure 7.4. The increase in stabilizer content caused a general increase in IDT strength, but the margin of increase has decreased beyond 6% RBI 81. Specimens with lower dosage showed the minimum stiffness and strength value due to the lack of sufficient pozzolanic materials. Addition of different proportions of aggregates to soil with optimum RBI

81 content (6%) causes 1.6 – 3.4 times increase in IDT strength for curing periods, as presented in Figure 7.5.



LS2: Lateritic Soil 2; RBI: Road Building International grade 81; A: Aggregate

Fig. 7.5 Variation of IDT strength for RBI 81 aggregate specimens

7.2 FATIGUE TEST

Fatigue failure is one of the main distress mechanisms causing degradation of pavements and is an important parameter related to structural failure of pavements. Fatigue is resulted by repetitive action of traffic loads, leading to initiation of minor hairline cracks, followed by the development of these micro cracks into wider macroscopic cracks, and they propagate through the thickness of the pavement and causing catastrophic failure resulting in the loss of structural integrity of the pavement (Gupta and Veeraragavan 2009).

The Fatigue tests were conducted on Repeated Load testing machine shown in Figure 7.6. All experiments were conducted on specimens cured for predetermined period. The loading level in the present study was taken as a fraction of the respective UCS value of each specimen at the same dosages. The untreated and treated soil specimens with seven days curing period were tested for repeated loading with 1/3rd, ½ and 2/3rd of their UCS values.

Test Procedure

- The cylindrical specimen was mounted on the loading frame and the LVDTs were set to read the deformation of the specimen. The load cell was brought in contact with the specimen surface.
- In the control unit, through the dedicated software, the selected loading stress level, frequency of loading and the type of wave form were fed in to the loading device.
- The loading system and the data acquisition system were switched on simultaneously and the process of fatigue load application on the test specimen was initiated.
- The repeated loading, at the designated excitation level (i.e. at the selected stress level and frequency) was continued till the failure of the test specimen.
- The data acquisition system continuously recorded the vertical deformation of the test specimen with cycles of loading until the failure and the output was saved in a result file.
- The failure pattern of the test specimen was visually observed.

Fig. 7.6 Fatigue testing machine and sample arrangement

7.2.1 Effect of Fatigue Life on LS1

For LS1 repeated loading test was conducted to determine the fatigue behaviour on untreated and treated UCS specimens with 38mm diameter and 75mm height. The specimens, subjected to seven days moist curing, were tested at a frequency of 1 Hz and rest period of 0.1 sec, and the results are tabulated in Table 7.1. Fatigue Life is considered as the number of load cycles that the material can withstand at a given stress level. The untreated soil samples were found to be so weak that they could not withstand for more cycles, while the treated soil samples show a good improvement in fatigue life. For treated soil samples, fatigue life is found to be increasing with increase in curing period. Addition of cement and coir to soil increased its fatigue strength significantly. It is observed that the fatigue life of the soil samples tested was influenced by the dosage of coir used. At lower stress levels the specimens exhibited a higher fatigue life, and with further increase in stress level, the fatigue life of stabilized specimen reduced considerably.

7.2.2 Effect on Fatigue Life on BC soil

To bring out the effect of stabilizer on the performance of stabilized soils when subjected to repeated loads, experiments are conducted for seven days of curing with a frequency of 1 Hz. Repeated loads corresponding to $1/3^{rd}$, 1/2 and $2/3^{rd}$ of UCS strength for seven days curing were applied to seven days moist cured specimens. The results of the test are shown in Table 7.2. A comparative study between unstabilized and stabilized soils indicates that the stabilization is effective in improving the fatigue or endurance life of soil samples. The fatigue life of the stabilized soil rapidly increases for $1/3^{rd}$ and half load as the stabilizer content increases.

Table 7.1 Fatigue test results of untreated and treated LS1

UCS (kg)	Stress Level	Applied Load (kg)	Fatigue life (No of cycles)
LS1			
23	0.33	8	3980
	0.50	12	2969
	0.66	15	2632
LS1 + Terrasil			
50	0.33	17	80767
	0.50	25	71260
	0.66	34	51675
LS1 + Terrabind			
44	0.33	15	13567
	0.50	22	11764
	0.66	29	9991
LS1 + Terrabind+ 6% Fly Ash			
112	0.33	37	35183
	0.50	56	29435
	0.66	75	21376
LS1 + 3% Cement + 0.2% Arecanut Coir			
60	0.33	20	36453
	0.50	30	35985
	0.66	40	35324
LS1 + 3% Cement + 0.4% Arecanut Coir			
72	0.33	24	56672
	0.50	36	56124
	0.66	48	55824
LS1 + 3% Cement + 0.6% Arecanut Coir			
84	0.33	28	67947
	0.50	42	65452
	0.66	56	64325
LS1 + 3% Cement + 0.8% Arecanut Coir			
73	0.33	24	68087
	0.50	37	67941
	0.66	49	66547
LS1 + 3% Cement + 1% Arecanut Coir			
65	0.33	22	77104
	0.50	33	76854
	0.66	44	76158

Table 7.2 Fatigue test results of untreated and treated BC soil

UCS (kg)	Stress Level	Applied load (kg)	Fatigue life (No of cycles)
BC soil			
32	0.33	11	1052
	0.5	16	827
	0.66	21	475
BC soil + 0.8% Terrasil			
127	0.33	42	178964
	0.50	64	142478
	0.66	85	119874
BC soil +Terrabind			
94	0.33	32	15589
	0.50	47	12567
	0.66	63	10415
BC soil +Terrabind+6% Fly Ash			
106	0.33	35	20119
	0.5	53	18715
	0.66	71	16318
BC soil + 2% RBI 81			
67	0.33	22	178110
	0.50	34	142622
	0.66	45	127841
BC soil + 4% RBI 81			
112	0.33	37	159317
	0.50	56	142750
	0.66	75	121861
BC soil + 6% RBI 81			
143	0.33	47	167824
	0.50	72	159102
	0.66	96	143678
BC soil + 8% RBI 81			
165	0.33	54	169912
	0.50	83	147891
	0.66	110	124712

7.2.3 Effect on Fatigue Life on LS2

The IDT strength test specimens with 100mm diameter and 68mm height were for the fatigue test of LS2. Repeated load test was conducted on cement and RBI 81 treated, along with aggregates soil specimens by applying the 2/3rd load of the 7 days cured IDT strength. The test was conducted at a frequency of 1 Hz, rest period of 0.1 seconds and at the room temperature (31 - 38^{o}C). A failed LS2 specimen is showed in Figure 7.7.

Fig. 7.7 Failed specimen due to fatigue

The fatigue behavior of soil treated with cement and RBI 81 for different curing periods of 7, 14 and 28 days are shown in Table 7.3. The load considered as 7 days IDT strength value for all the curing days. It can be noted that treated soil with aggregate exhibits lesser cycles than the optimum cement, RBI 81 treated specimens. The increase in the number of failure cycles of cement treated sample is mainly due to the gradual strength the cement acquires in course of time due to hydration reaction. The samples are kept under moist curing conditions, which also favour the hydration reaction process which in-turn results in the increased strength development.

Table 7.3 Fatigue behavior of Cement and RBI 81 treated soil samples

Specimen	Curing days	Applied Load (Kg)	No. of Failure Cycles
LS2+6% Cement	7	127	4543
	14		13547
	28		16132
LS2 + 6% Cement+ 30% Aggregates	7	563	2143
	14		7654
	28		9875
LS2+6% RBI 81	7	137	4326
	14		13214
	28		16342
LS2+6% RBI 81+30% Aggregates	7	563	2243
	14		7834
	28		9812

7.3 MAJOR FINDINGS

- IDT strength values increased with stabilizer content for both cement and RBI 81 treated soils and tremendous improvement was observed with the addition of aggregates (with optimum stabilizer content).
- Curing period played an important role for all stabilized soils tested for IDT strength.
- Fatigue life increased for stabilized soil and the enhancement was improving with higher dosages for both LS and BC soil.
- For treated IDT strength specimens with LS2, fatigue life increased with increase in curing period.

CHAPTER 8

PAVEMENT ANALYSIS AND COST EVALUATION

8.1 GENERAL

Pavement design is one of the earliest branches in civil engineering. Dated to early 1920's, the thickness of pavement was purely based on experience and same thickness was used for a section of highway, though widely different soils were encountered. When years passed, various methods were developed by different agencies for determining the thickness of pavement required. Researchers observed that thickness was not alone the basis for design of pavement, but other parameters including stresses, strains, deflections, shear under the application of present and expected future loadings (external stresses) were also considered in the design analysis and possible developments were made in different stages to determine those parameters effectively. Widely known is that, the pavement design is being performed using the following methods:-

- Empirical method
- Analytical method
- Numerical method
- FEM analysis

8.2 FLEXIBLE PAVEMENT

The pavement structure in a flexible pavement generally deflects or bends to accommodate the traffic loads coming over it. The structure consists of different layers including the top wearing layer and the bottom subgrade which acts as the foundation. The load distribution in flexible pavements is accomplished through these layers. Each layer transfers the load to the layer below by spreading into a wider area thereby the stress is reduced from top to bottom. Thus the surface layer, which generally consists of a bituminous mixture, carries the maximum load, whereas the subgrade takes the minimum.

8.3 STRESSES IN FLEXIBLE PAVEMENTS

In order to characterize the behavior of a flexible pavement under the action of wheel loads, Yang (2004) considered it as a homogeneous half-space. A half-space has an infinitely large area and an infinite depth with a top plane on which the loads are applied. The original Boussinesq theory was based on a concentrated load applied on an elastic half space. The responses in a pavement structure (stresses, strains and deflections) due to a concentrated load can be integrated to obtain them due to loading over circular area. Several theories are available for the analysis of these responses in a flexible pavement based on the behavior of pavement materials used. But because of the large number and the complexity of factors involved, no single theory is likely to account for all aspects in the design and analysis of flexible pavements. The stresses in flexible pavements are mainly calculated using three layer concepts, with an assumption that a uniformly distributed load is applied over a circular contact area, though it is not completely true in the case of wheel loads applied through pneumatic tyres. Considering layer system generally the analytical solution to the state of stress or strain has several assumptions as listed below:

- The material properties of each layer are homogeneous
- Each layer has a finite thickness except for the lower layer, and all are infinite in the lateral directions
- Each layer is isotropic, i.e. the property at a specific point is the same in every direction
- Full friction is developed between layers at each interface
- Surface shear forces are not present at the surface
- The stress solutions are characterized by the material properties for each layer, Poisson's ratio and elastic modulus.

The critical stresses that can be calculated using three layer systems include,

σ_{z1}: Vertical stress at interface 1; σ_{z2}: Vertical stress at interface 2

σ_{r1}: Horizontal stress at the bottom layer 1; σ_{r2}: Horizontal stress at the bottom layer 2

Figure 8.1 shows a three layered pavement system, having surface, base and sub-grade as the three layers. h_1, E_1, μ_1 are the depth, modulus of elasticity and Poisson's ratio of surface course. h_2, E_2, μ_2 and h_3, E_3, μ_3 are the corresponding values of base course and sub-grade respectively. The sub-grade is considered to be of infinite thickness. P is the load applied, while p is the tyre pressure. Figure 8.2 illustrates the critical failure points in the flexible pavements.

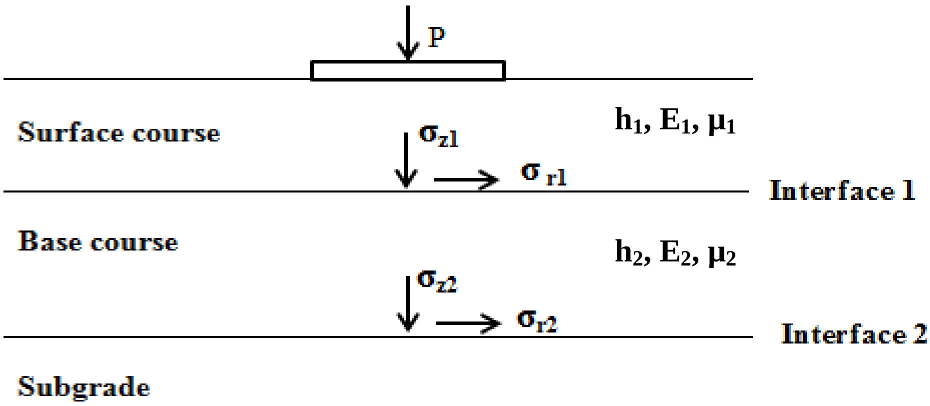


Fig. 8.1 Three layer system

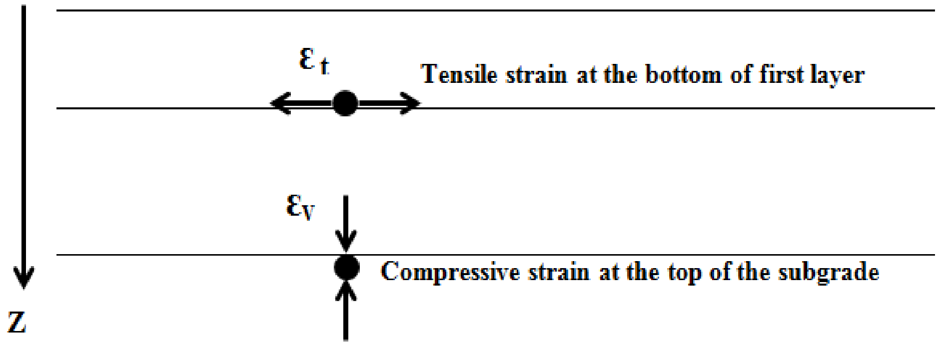


Fig. 8.2 Failure modes and Critical strains for flexible pavement

The vertical stress on the top of subgrade is an important factor in pavement design. The function of a pavement is to reduce the vertical stress on the subgrade so that detrimental pavement deformations will not occur. The allowable vertical stress on a given subgrade depends on the strength or modulus of the subgrade. To combine the effect of stress and strength, the vertical compressive strain has been used most frequently as a design criterion. The stresses in a two layer system depend on the modulus ratio E_1/E_2 and the thickness radius ratio h_1/a. Vertical surface deflection and vertical interface deflection are other two criteria used in the pavement design. In this investigation, KENPAVE Software is used to analyze low volume and high volume flexible pavements for both standard and modified cases as per IRC: SP: 72-2007 and IRC: 37-2012.

8.4 RESILIENT MODULUS

Resilient modulus (E) of sub grade and granular layers, and fatigue and rutting values for a pavement structure were calculated using Equations (8.1) to (8.5) recommended by the IRC 37, 2012.

$$E\ (\text{MPa}) = 10 \times \text{CBR for CBR} \leq 5 \tag{8.1}$$

$$E\ (\text{MPa}) = 17.6 \times (\text{CBR})^{0.64} \text{ for CBR} > 5 \tag{8.2}$$

$$E_{gb}\ (\text{MPa}) = E_{sg} \times 0.2 \times h^{0.45} \tag{8.3}$$

E_{gb} – Modulus of granular base

Es_g – Modulus of sub grade (MPa)

CBR – California Bearing Ratio of sub grade (%)

h – Thickness of granular base (mm)

Fatigue Criteria: The relationship between fatigue failure of surface layer and tensile strain at the bottom of asphalt layer is represented by the number of repetitions as represented in the equation:

$$N_F = 2.021 \times 10^{-4}\ [1/\epsilon_t]\ [1/E_{bs}]^{0.854} \tag{8.4}$$

N_F = Number of cumulative standard axles to produce 20 % cracked surface area

ϵ_t = Tensile strain at the bottom of bituminous surfacing (micro strain)

E_b= Resilient modulus of bituminous surfacing (MPa)

Rutting Criteria: The relationship between rutting failure and compressive strain at the top of subgrade is represented by the number of load applications as represented in the equation:

$$N_R = 4.1656 \times 10^{-8} [1/\epsilon_z]^{4.5337} \quad (8.5)$$

N_R= Number of cumulative standard axles to produce rutting of 20mm

ϵ_z= Vertical sub grade strain (micro strain)

8.5 KENPAVE SOFTWARE

The analysis was done using the software called KENPAVE for pavement analysis, developed by Yang (2004) at the University of Kentucky. The KENLAYER computer program applies only to flexible pavements with no joints or rigid layers. The backbone of KENLAYER is the solution for an elastic multilayer system under a circular loaded area. The software does linear elastic multi-layer analysis to obtain the results including stresses, strains and deflections. It can be applied to layered systems under single, dual, dual-tandem and dual-tridem wheel configurations with different layer behaviors like linear elastic, nonlinear elastic and visco-elastic. Damage analysis can be made by dividing each year into a maximum of 12 periods, each with a different set of material properties. Each period can have different loading conditions, with single or multiple wheeled. The damage caused by fatigue cracking and permanent deformation in each period over all load groups can be summed up to evaluate the design life. There are several input parameters for analysis of pavement in KENPAVE and some of them adopted for the current study are listed below.

All layers are assumed to be linearly elastic with a constant elastic modulus.

- The number of periods in a year is 1.
- The number of load group is 0, 1, 2 or 3 depending on the wheel configuration.
- The number of layers varies among 2, 3 and 4.
- The number of Z coordinates is calculated depending upon the number of interfaces and the intermediate points for analysis.
- The number of responses is 5, which are displacement, vertical stress, vertical strain, Major Principle stress, minor principle stress and intermediate stress in the output.
- All layer interfaces are assumed to be bonded.

- SI units are used for calculations.

Loading Inputs

- Types of loading are Single Axle Single Wheel (SASW), Single Axle Dual Wheel (SADW), Tandem and Tridem Axles with dual wheel at the end of each axle.

  SASW / SADW = 10 kN

  Tandem axle = 20 kN

  Tridem axle = 30 kN

- The contact radius of circular loaded area is 15.08 cm for SASW.
- The contact radius of circular loaded area is 10.66 cm for SADW.
- The contact radius of circular loaded area is 10.66 cm for Tandem axle.
- The contact radius of circular loaded area is 10.66 cm for Tridem axle.
- The contact pressure on circular loaded area is 700 kPa.
- Centre to center distance between 2 dual wheels along Y-axis is 32.5 cm.
- Centre to center distance between 2 axle x along X-axis is 142 cm.

The thickness of each layer is measured in cm. For standard cases, the Pavement Design Catalogues are followed and for modified cases the required thicknesses are considered by trial and error method.

The analysis involves extensive use of KENPAVE software package for pavements. IRC: SP: 72-2007 and IRC: 37-2012, prepared by Indian Roads Congress, are dealing with flexible pavement for low and high volume traffic (maximum 150 million standard axles), respectively. In the first stage, analysis using KENLAYER is performed on the standard cases from the Pavement Design Catalogues. The second stage analysis is performed by maintaining the same stresses and strains, and the thickness of new layer with modified soil has been tried. The thickness of new material can be found out by analyzing the pavement for standard conditions.

Different combinations of new materials are analyzed to find the thickness of that material in the pavement section.

Material Property Inputs: The values of CBR, Resilient Modulus (E) and Poisson's Ratio used for different types of materials for design of low volume roads are tabulated in Table 8.1.

Table 8.1 Material property inputs

Material	Elastic Modulus (kPa)	Poisson's Ratio
WBM	E_{SG} x 0.2 x $h^{0.45}$	0.35
Granular Base (GB)	E_{SG} x 0.2 x $h^{0.45}$	0.35
Granular Sub-Base (GSB)	E_{SG} x 0.2 x $h^{0.45}$	0.35
Subgrade (SG)	10 CBR, (CBR < 5)	0.4
	17.6 x$(CBR)^{0.64}$, (CBR > 5)	0.4

E_{SG} = modulus of sub grade; h = Thickness of granular layers.

Traffic parameter: A vehicale may have different number of axles, and the load is distributed to these axles and transferred to the pavement surface through the wheels. A standard truck has two axles, front axle with two wheels and rear axle with four wheels. But to carry high loads multiple axles are provided. Since the design of flexible pavements is by layered theory, only the wheels on one side needed to be considered. For pavement design, the subgrade has been classified into five categories and traffic into seven, as listed in Table 8.2.

Table 8.2 Subgrade and Traffic categories

Subgrade Category	CBR (%)	Traffic Category	Cumulative ESAL Applications
S1	2	T1	10,000-30,000
S2	3 to 4	T2	30,000-60,000
S3	5 to 8	T3	60,000-100,000
S4	7 to 9	T4	100,000-200,000
S5	10 to15	T5	200,000-300,000
		T6	300,000-600,000
		T7	600,000-1,000,000

Stress Strain analysis for low volume roads: In India, low volume roads are generally constructed with 3.75m carriageway width and hard shoulders of width about 1.5m. An attempt has been tried to improve the design life for modified pavement layers.

Subgrade class: Subgrade is defined as a compacted layer, generally of naturally occurring local soil, assumed to be 300 to 500 mm in thickness depending on traffic volume, just beneath the pavement crust, providing a suitable foundation for the pavement. In the presumptive design CBR values for typical subgrade soils, Highly Plastic Clays and Silts, Silty Clays and Sandy Clays, and Clayey Sands and Silty Sands are provided as 2-3, 4-5, and 6-10 % respectively. Pavement Design Catalogue as per IRC: SP: 72-2007 are shown in Table 8.3.

Table 8.3 Total pavement thickness as per IRC: SP: 72-2007

Sub grade strength (CBR %)	Cumulative ESAL Applications						
	T1 10,000-30,000	T2 30,000-60,000	T3 60,000-1,00,000	T4 1,00,000-2,00,000	T5 2,00,000-3,00,000	T6 3,00,000-6,00,000	T7 6,00,000-1,000,000
S1 (2)	300mm	325mm	375mm	425mm	475mm	550mm	650mm
S2 (3 to 4)	200mm	275mm	325mm	375mm	425mm	475mm	525mm
S3 (5 to 8)	175mm	250mm	275mm	300mm	325mm	375mm	425mm
S4 (7 to 9)	150mm	175mm	225mm	275mm	300mm	325mm	375mm
S5 (10 to15)	125mm	150mm	175mm	225mm	275mm	300mm	350mm

8.6 IITPAVE SOFTWARE

IRC suggests IITPAVE program for the design of flexible pavements (IRC 37 2012). A satisfactory pavement analysis can be achieved through iterative process by varying layer thicknesses or, if necessary, by changing the pavement layer materials. Any combination of traffic and pavement layers can be tried using this software by providing inputs like number of layers, layer thickness, Poisson’s ratio, resilient modulus, tyre pressure and wheel load, similar to KENPAVE and critical strains are obtained as outputs. In the current study, this software was used to analyze both the conventional and modified high volume pavement sections. Analysis was carried out for dual wheel loading at the vertical plane of load application (radial distance is zero).

8.7 PAVEMENT ANALYSIS OF LOW VOLUME ROADS WITH STABILIZED SOIL

The pavement structure adopted from IRC: SP: 72-2007, for a CBR of 3 - 4% (case S2) and four traffic load conditions for conventional and modified cases, are tabulated in Table 8.4. The structure consists of a thin bituminous treated WBM layer, a granular base and a sub-base layer above the sub grade. For modified design, mixture of soil, 3% cement and 1% Arecanut coir with CBR value of 19% is considered as modified sub grade layer. The thickness of pavement with new material was arrived using trial and error method. The thickness of pavement sections have been considered based on the critical strains developed in the pavement layer, i.e., horizontal tensile strain at the bottom of the bituminous layer and vertical compressive strain at the top of sub grade.

Table 8.4 Pavement design catalogue as per IRC: SP: 72-2007 and modified case

Sub grade	Cumulative ESAL Applications 1,00,000 to 2,00,000 (T4)	2,00,000 to 3,00,000 (T5)	3,00,000 to 6,00,000 (T6)	6,00,000 to 1,000,000 (T7)
Poor (CBR 3-4%) (S2)	75 100 100 100	75 100 100 150	75 150 100 150	75 150 150 150
Modified (CBR 20%)	75 100 75 100	75 100 100 150	75 125 100 150	75 150 150 125

Modified Sub grade   WBM
Granular Sub-base   Bituminous treated WBM
All thickness are in mm

8.7.1 KENPAVE Analysis Results for Low Volume Roads

In KENPAVE software, thickness of different layers, material properties and loading conditions are provided as general input parameters and coefficients of rutting, fatigue etc. can also be provided for detailed analysis. Damage ratio, the ratio of actual load repetitions to the allowed repetitions, is a crucial parameter in pavement design. In no case, the actual load repetitions shall be more than the allowed repetitions, which indicates the pavement failure (Deepthi et al. 2012). Displacement values were generated at all the layer interfaces as presented in Table 8.5. All the stresses were reduced when the conventional layers in the standard cases were replaced with the stabilized soil layers. The results showed that the stress values are decreased for new material as compared to the conventional one, even after reducing the layer thickness. In the case of S2T4, even though the thickness is reduced from 375 to 350 mm for modified pavement structure, the stresses over the subgrade is less than that for the conventional pavement and this ensures that the adopted thickness is sufficient enough to bear the corresponding traffic loading presented in Table 8.6. The stress values were getting reduced for all the axle loads, whereas the displacement was higher for tandem and tridem axles in pavement sections with stabilized soil.

Table 8.5 Displacement values for conventional and modified layers for all axle loads

Displacement (mm)										
Traffic	H (mm)	Conventional				H (mm)	Modified			
		SASW	SADW	Tandem	Tridem		SASW	SADW	Tandem	Tridem
S2T4	0	2.07	1.81	2.13	2.25	0	1.72	1.51	1.76	1.87
	75	1.85	1.54	1.77	1.89	75	1.58	1.33	1.57	1.69
	175	1.60	1.33	1.57	1.69	175	1.41	1.20	1.44	1.55
	275	1.38	1.18	1.42	1.54	250	1.29	1.11	1.35	1.47
	375	**1.19**	**1.05**	**1.29**	**1.41**	**350**	**1.15**	**1.02**	**1.26**	**1.38**
S2T5	0	2.03	1.77	2.09	2.26	0	1.65	1.45	1.77	1.94
	75	1.81	1.50	1.74	1.85	75	1.50	1.27	1.52	1.64
	175	1.55	1.29	1.53	1.65	175	1.33	1.14	1.38	1.50
	275	1.34	1.14	1.39	1.50	250	1.21	1.05	1.30	1.42
	425	**1.08**	**0.96**	**1.21**	**1.32**	**400**	**1.03**	**0.92**	**1.17**	**1.29**
S2T6	0	1.83	1.61	1.93	2.10	0	1.47	1.31	1.63	1.79
	75	1.63	1.36	1.61	1.72	75	1.34	1.15	1.40	1.51
	225	1.31	1.12	1.36	1.48	200	1.16	1.01	1.25	1.37
	325	1.15	1.01	1.26	1.37	300	1.04	0.93	1.17	1.29
	475	**0.95**	**0.87**	**1.11**	**1.23**	**450**	**0.9**	**0.82**	**1.07**	**1.19**
S2T7	0	1.68	1.48	1.80	1.97	0	1.34	1.21	1.52	1.68
	75	1.48	1.25	1.50	1.61	75	1.22	1.05	1.30	1.42
	225	1.19	1.03	1.27	1.39	200	1.05	0.92	1.17	1.29
	375	1.01	0.90	1.15	1.27	350	0.91	0.83	1.08	1.20
	525	**0.85**	**0.78**	**1.03**	**1.15**	**500**	**0.80**	**0.74**	**0.99**	**1.11**

Table 8.6 Vertical stress values for conventional and modified layers for all axle loads

Vertical Stress (kPa)										
Traffic	H (mm)	Conventional				H (mm)	Modified			
		SASW	SADW	Tandem	Tridem		SASW	SADW	Tandem	Tridem
S2T4	0	700	700	700	700	0	700	700	700	700
	75	609.60	543.42	543.40	543.54	75	601.96	537.45	537.43	537.57
	175	324.19	227.94	227.98	227.98	175	304.84	215.08	215.17	215.17
	275	156.99	108.38	108.52	108.52	250	167.55	115.56	115.76	115.75
	375	**90.01**	**66.92**	**67.22**	**67.22**	**350**	**85.14**	**63.52**	**63.89**	**63.88**
S2T5	0	700	700	700	700	0	700	700	700	700
	75	610.98	544.13	544.11	544.25	75	605.07	539.17	539.18	539.31
	175	329.15	230.70	230.78	230.78	175	315.32	220.85	221.01	221.00
	275	164.09	112.22	112.43	112.42	250	181.06	122.89	123.20	123.19
	425	**73.97**	**56.59**	**57.08**	**57.08**	**400**	**69.02**	**52.89**	**53.52**	**53.51**
S2T6	0	700	700	700	700	0	700	700	700	700
	75	618.68	550.94	550.93	551.07	75	613.82	546.96	546.98	547.12
	225	233.35	158.02	158.22	158.22	200	271.01	185.27	185.57	185.56
	325	120.33	84.53	84.93	84.92	300	129.65	89.42	89.98	89.97
	475	**58.12**	**46.00**	**46.74**	**46.74**	**450**	**53.44**	**42.35**	**43.29**	**43.28**
S2T7	0	700	700	700	700	0	700	700	700	700
	75	623.09	554.16	554.17	554.32	75	620.62	551.48	551.52	551.66
	225	245.95	165.12	165.42	165.41	200	289.51	196.59	196.97	196.96
	375	91.07	65.92	66.58	66.57	350	96.61	68.33	69.19	69.18
	525	46.67	37.95	38.97	38.97	500	42.69	34.77	36.01	36.01

Excess vertical surface deflections in flexible pavements have always been a major concern and used as a criterion for pavement design. It is desirable to reduce the deflections as much as possible. This may be achieved with or without the improvement of soil subgrade before construction. For any pavement structure the sub grade should be located 500mm above the high flood level in any season. Figures 8.3 and 8.4 indicate both compressive and tensile strain values are reduced in the modified soil for all traffic conditions. Figures 8.5 and 8.6 depict that fatigue life is much higher than rutting life, which indicates that the critical failure of the pavement is due to fatigue. From Figure 8.7, it is observed that the damage ratio is more for conventional method, and its reduction is remarkable in the case of stabilized soil. Enhanced life span of modified pavement structure was proved from the damage analysis, with fatigue and rutting lives improvement by 4 – 5 times and 1.4 – 1.6 times respectively. Figure 8.8 (a-d) presents the view of LGRAPH in KENPAVE.

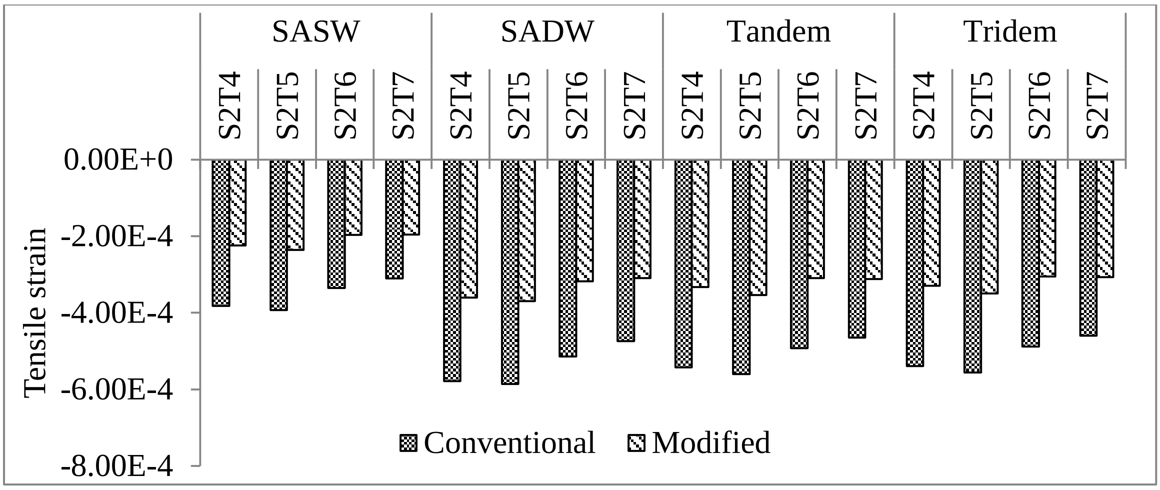


Fig. 8.3 Tensile strain values for different traffic conditions for different axle loads

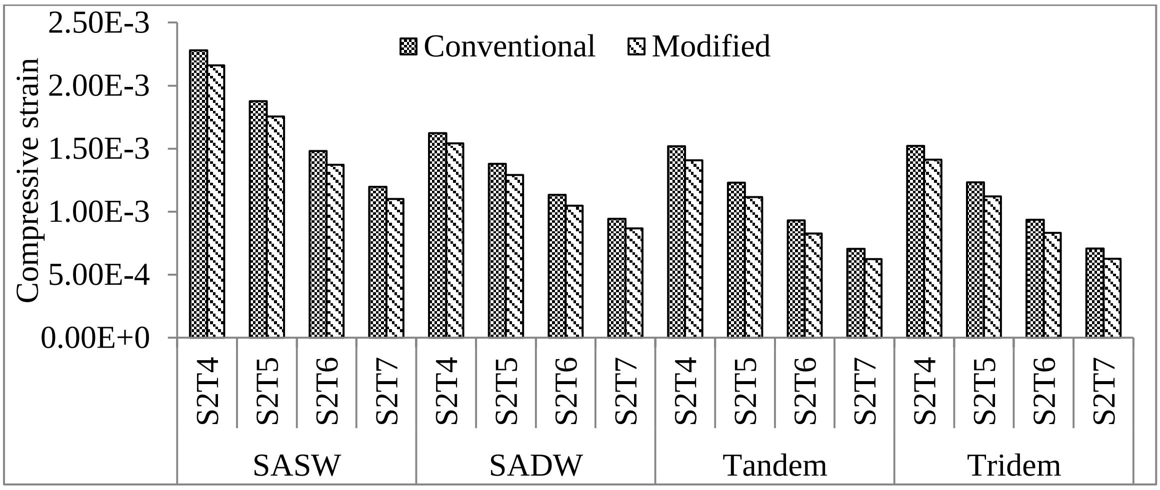


Fig. 8.4 Compressive strain values for different traffic conditions and axle loads

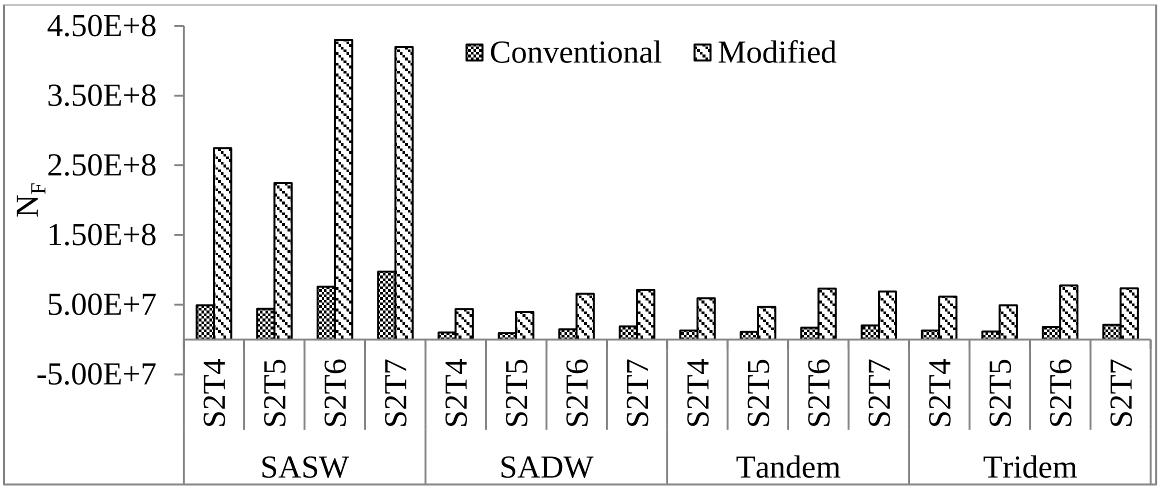


Fig. 8.5 Fatigue life of conventional and modified for different traffic and axle loads

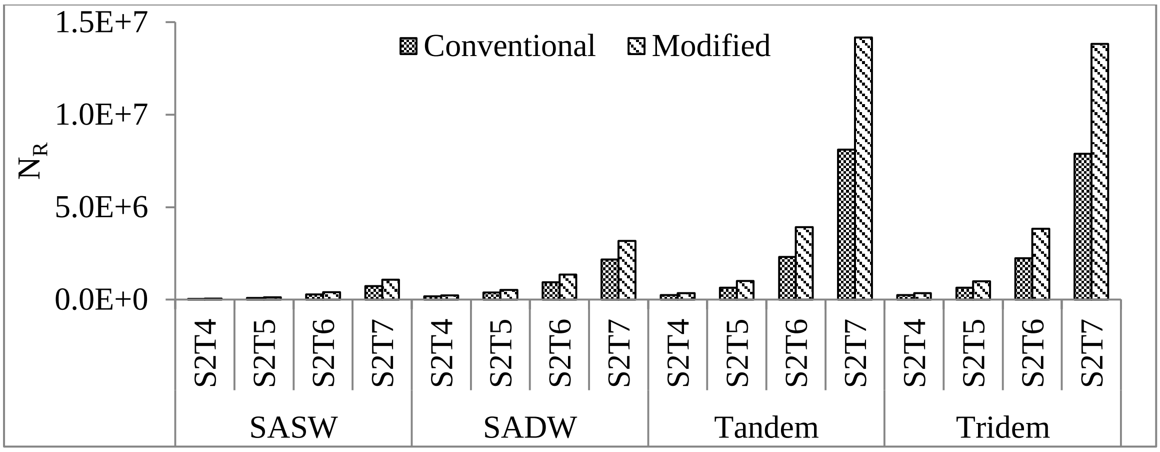


Fig. 8.6 Fatigue life of conventional and modified for different traffic and axle loads

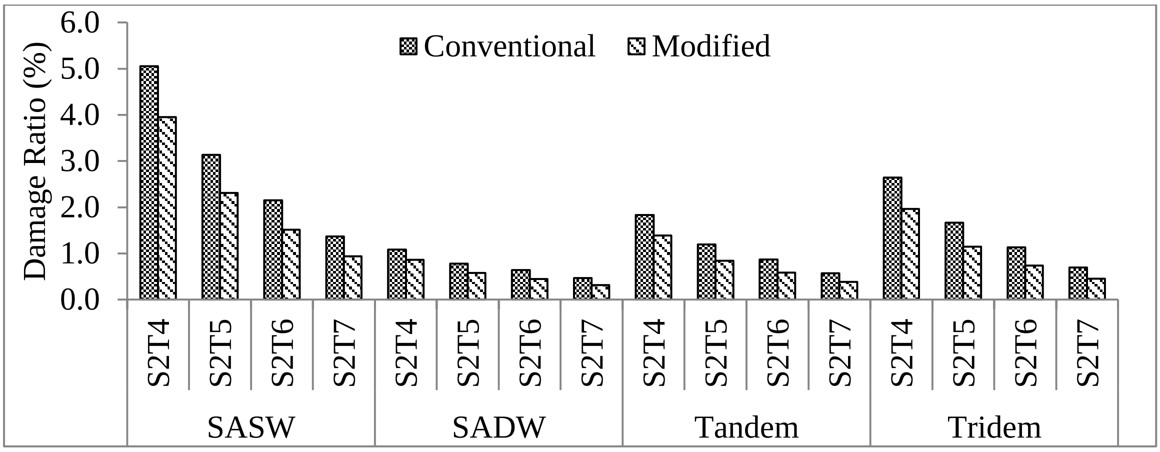
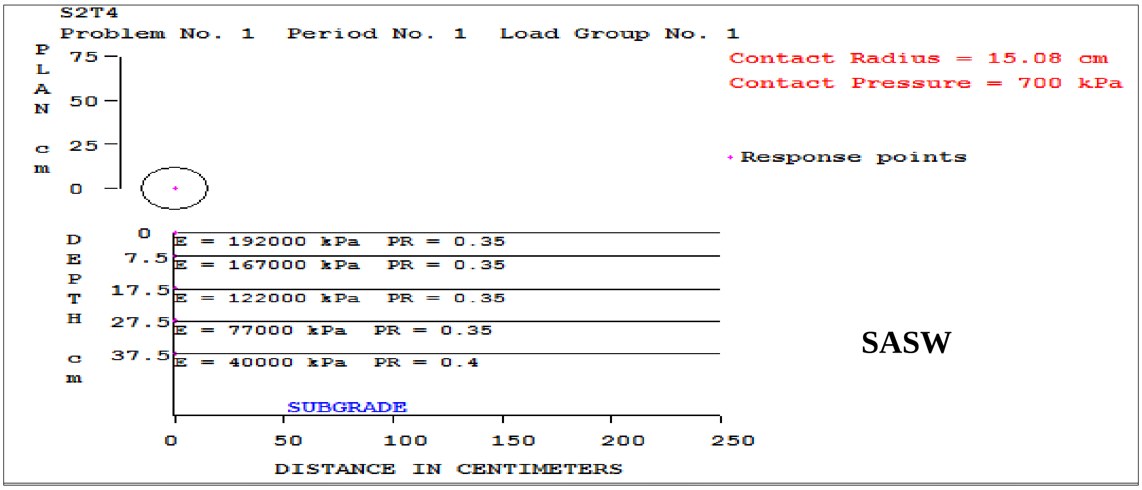
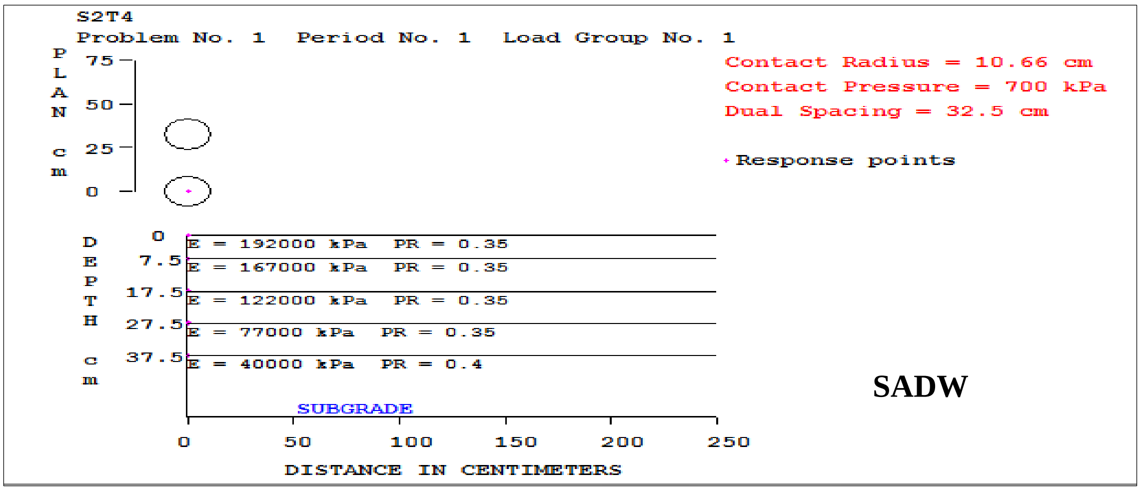


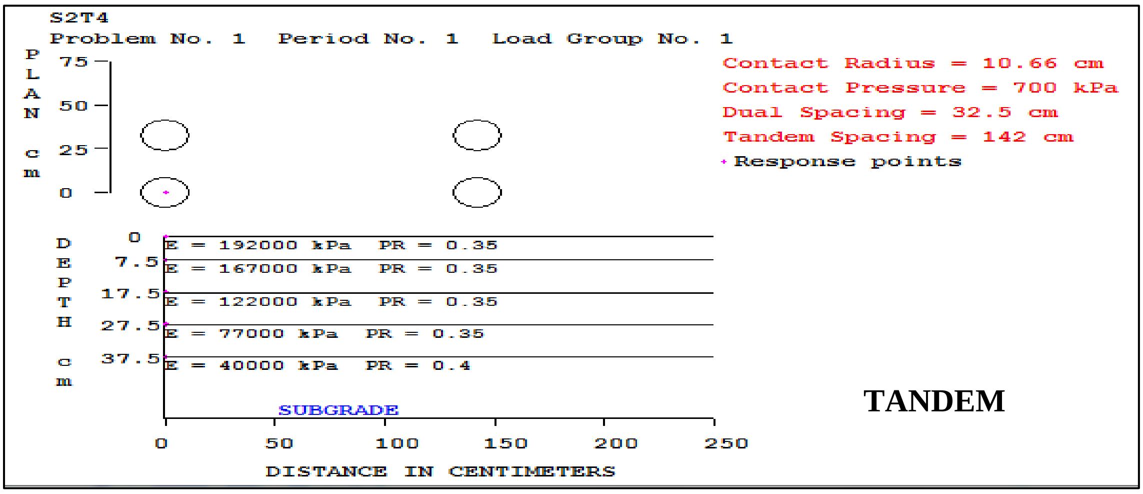
Fig. 8.7 Variations of damage ratio for conventional and modified cases for different traffic and axle loads



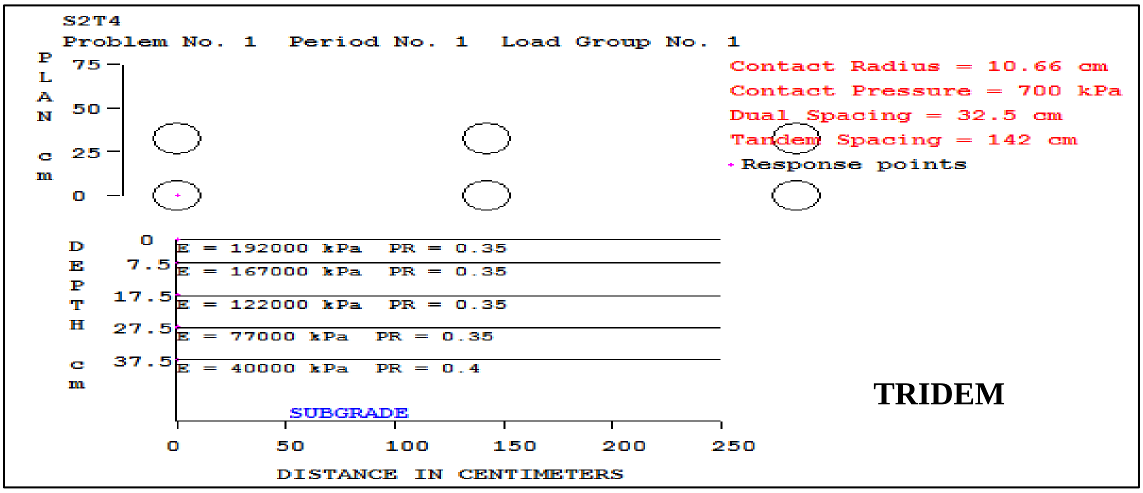
(a)



(b)



(c)



(d)

Fig. 8.8 (a-d) L graph snapshots for different axles

8.8 PAVEMENT ANALYSIS OF HIGH VOLUME ROADS WITH STABILIZED SOIL

The pavement design catalogues from IRC: 37-2012 are considered as the standard cases. The LS2 has CBR strength of 8%, and the pavement sections for this CBR from the catalogue are considered. The categorization of traffic and sub-grade strength has eight standard cases, with traffic loading conditions vary from 2msa to 150msa. Material properties used in this study are presented in Table 8.7.

Table 8.7 Material inputs for modified condition

Material	Elastic Modulus (MPa)	Poisson's Ratio
BC and DBM for VG30 bitumen @ Temperature 35ºC	1700	0.35
Aggregate Interlayer (AI)	750	0.35
Cement treated soil	600	0.25
Sub-grade (SG)	10 CBR (CBR < 5)	0.35
	$17.6 \times (CBR)^{0.64}$ (CBR > 5)	0.40

8.8.1 Thickness Reduction for Base and Sub Base Courses

Stabilized base and sub base course materials must meet certain requirements of gradation, strength and durability to qualify for reduced layer thickness design. UCS and durability requirements for bases and sub base treated with cement are presented in Tables 8.8 and 8.9.

Table 8.8 Minimum UCS values for cement stabilized soils

Minimum Unconfined Compressive strength, kPa		
Stabilized Soil Layer	Flexible pavement	Rigid pavement
Base course	5170	3447
Sub base/ select material/ subgrade	1723	1379

Table 8.9 Durability requirements

Type of Soil Stabilized	Maximum Allowable Weight Loss After 12 WD or FT Cycles (Per cent of Initial Specimen Weight)
Granular, PI < 10	11
Granular, PI > 10	8
Silt	8
Clays	6

Crack relief layer: A Stress Absorbing Membrane Interlayer (SAMI) using modified bitumen provided over the cementitious layer delays the cracks propagating into the bituminous layer. A crack relief layer of wet mix macadam of thickness 100mm sandwiched between the bituminous layer and treated layer is much more effective in arresting the propagation of cracks from the cementitious base to the bituminous layer. The aggregate layer becomes stiffer under heavier loads because of high confining pressure.

8.8.2 Stress Analysis for High Volume Roads

The stresses and vertical displacement at the interfaces and on the sub grade layer is calculated using multiple layer concepts using KENPAVE software for different traffic intensities and for subgrade strength of 8% CBR. Then analysis is performed with the objective of maintaining the same stresses and strains with conventional material. By trial and error method the thickness of the stabilized layer was varied and the pavement thickness was achieved by limiting the stresses and strains in each layer according to the conventional design as a bench mark. The conventional layers were replaced by stabilized soil with optimum thickness for 8% of subgrade strength and cumulative ESAL applications.

8.8.3 Damage Analysis for High Volume Roads

The damage analysis was carried out using the KENPAVE software to determine the tensile strain, compressive strain, number of allowable repetitions for fatigue and rutting, damage ratio and design life in years for different traffic intensities. A minimum thickness of the pavement is established for which the strains and damage ratio are reduced and the allowable number of repetitions and the design life are increased.

8.8.4 KENPAVE Analysis Results for High Volume Roads

Figures 8.9 and 8.10 show the conventional and modified pavement sections for CBR 8%. By keeping the top bituminous layers in the conventional sections as it is, the granular base layer was replaced by soil cement aggregate stabilized layer with an aggregate interlayer above this to prevent cracking. The thickness of the stabilized soil layer was determined based on trial and error and analyzed.

LS2 stabilized with 6% cement and 30% aggregate was used as the replacing material for the base layer. The granular base layer is replaced by the cement treated layer, and an aggregate interlayer with minimum 75mm thickness is provided over this layer which will act as the crack relief layer. When the cement stabilized soil is used in the modified case, the number of layers will change from four to five due to the addition of the aggregate interlayer. Stress at the layer interfaces after the replacements are calculated and then compared with the standard cases to check if the values are within the permissible limits. Analysis was carried out by changing the number of axles also. The Tables 8.10 and 8.11 shows the analysis carried out in the standard and modified cases. The stresses and displacements for modified sections are lesser than that for conventional sections at corresponding layer interfaces. Also strains and damage ratio are decreased (Figures 8.11 (a-d), 8.12 (a-d) and Table 8.13) and fatigue and rutting lives are increased for modified cases (Table 8.12). This indicates that the modified sections are safe and perform better than the conventional ones.

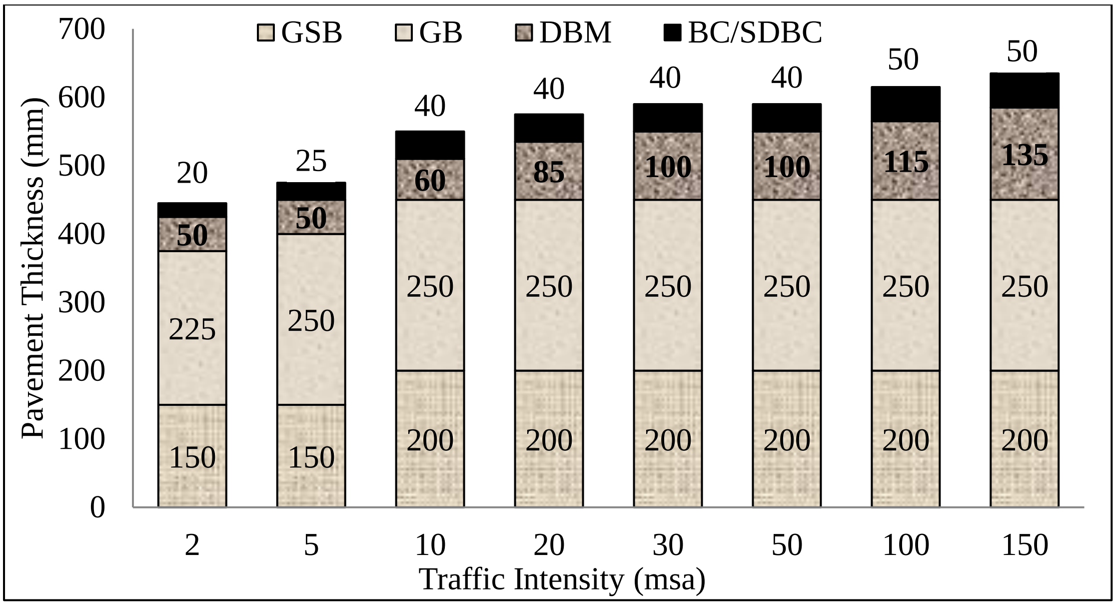


Fig. 8.9 Conventional pavement thickness as per IRC: 37-2012 for CBR 8%

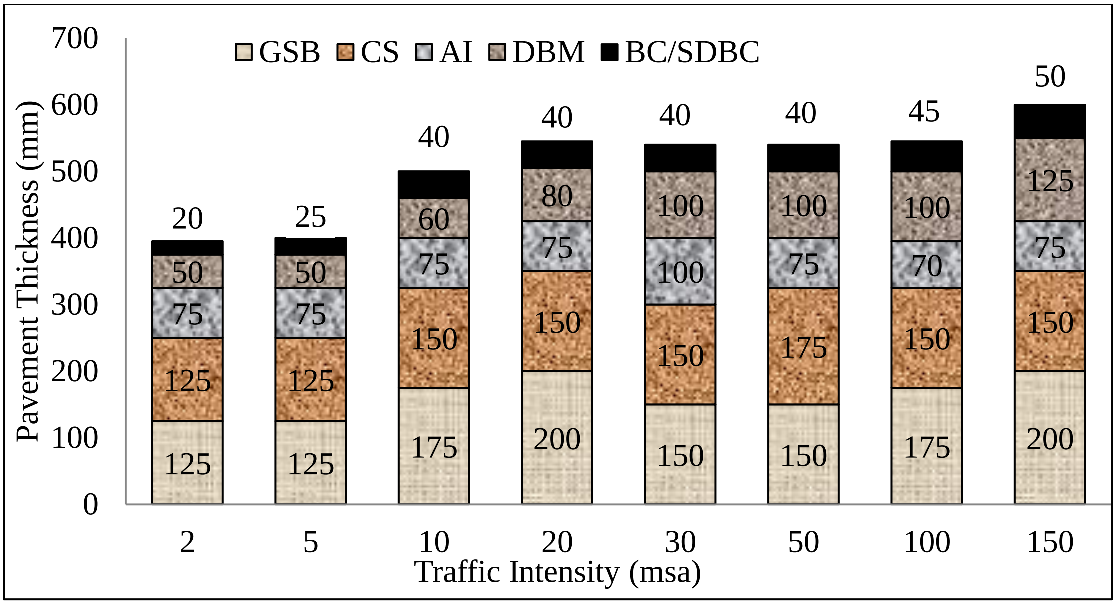


Fig. 8.10 Modified thickness with cement stabilized and AI for CBR 8%

Note:

GSB – Granular Sub-Base	CS - Cementitious Soil
DBM – Dense Bound Macadam	AI - Aggregate Interlayer
SDBC- Semi Dense Bituminous Course	BC - Bituminous Course

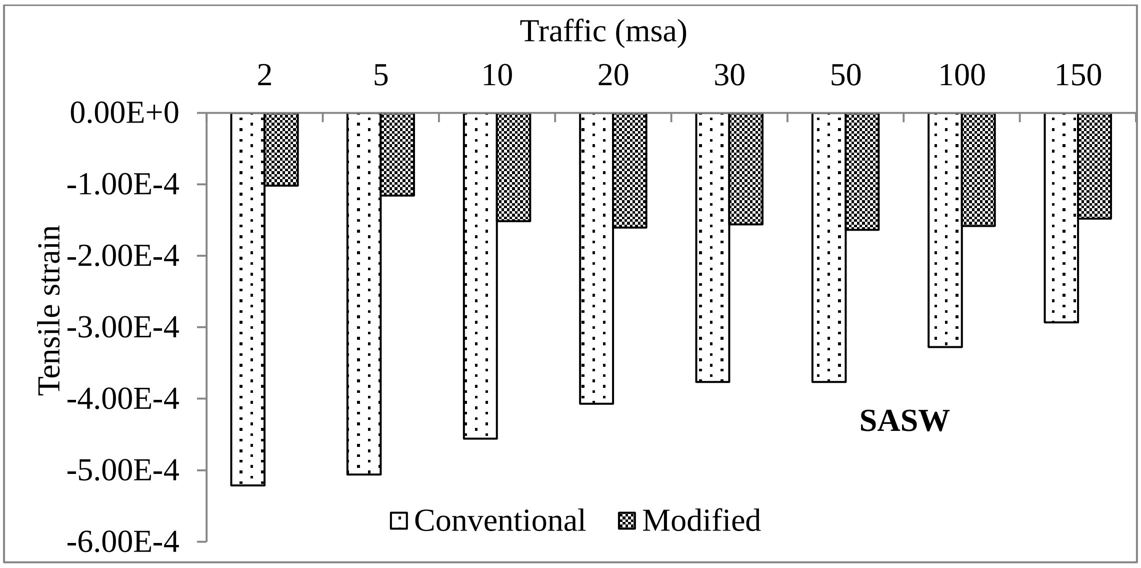
Table 8.10 Displacement values for conventional and modified for high volume roads

Displacement (mm)										
Traffic	H (mm)	Conventional				H (mm)	Modified			
		SASW	SADW	Tandem	Tridem		SASW	SADW	Tandem	Tridem
2msa	0	1.03	0.86	1.05	1.15	0	0.71	0.64	0.83	0.93
	70	1.01	0.82	0.97	1.04	70	0.71	0.62	0.77	0.84
	295	0.69	0.60	0.75	0.82	145	0.67	0.59	0.74	0.81
	445	**0.57**	**0.51**	**0.66**	**0.73**	270	0.61	0.55	0.70	0.77
						395	**0.52**	**0.48**	**0.63**	**0.70**
5msa	0	0.94	0.81	1.00	1.10	0	0.70	0.63	0.82	0.92
	75	0.95	0.78	0.93	1.00	75	0.70	0.61	0.76	0.83
	325	0.64	0.57	0.71	0.78	150	0.66	0.58	0.73	0.80
	475	**0.53**	**0.48**	**0.63**	**0.70**	275	0.60	0.54	0.69	0.76
						400	**0.52**	**0.48**	**0.62**	**0.69**
10msa	0	0.80	0.68	0.87	0.97	0	0.58	0.54	0.72	0.82
	100	0.80	0.67	0.81	0.88	100	0.59	0.52	0.67	0.74
	350	0.55	0.50	0.65	0.72	175	0.55	0.49	0.64	0.72
	550	**0.45**	**0.42**	**0.57**	**0.64**	325	0.50	0.46	0.61	0.68
						500	**0.42**	**0.40**	**0.55**	**0.62**
20msa	0	0.70	0.62	0.80	0.90	0	0.54	0.50	0.68	0.77
	125	0.71	0.60	0.75	0.82	120	0.54	0.48	0.63	0.70
	375	0.51	0.47	0.61	0.69	195	0.51	0.46	0.61	0.68
	575	**0.42**	**0.40**	**0.54**	**0.62**	345	0.46	0.43	0.58	0.65
						545	**0.39**	**0.37**	**0.52**	**0.59**
30msa	0	0.66	0.59	0.77	0.87	0	0.51	0.47	0.65	0.75
	140	0.66	0.57	0.72	0.79	140	0.50	0.45	0.60	0.67
	390	0.49	0.45	0.60	0.67	240	0.46	0.43	0.58	0.65
	590	**0.41**	**0.39**	**0.53**	**0.60**	390	0.43	0.40	0.55	0.62
						540	**0.38**	**0.36**	**0.51**	**0.58**
50msa	0	0.66	0.59	0.77	0.87	0	0.51	0.47	0.65	0.75
	140	0.66	0.57	0.72	0.79	140	0.50	0.45	0.60	0.68
	390	0.49	0.45	0.60	0.67	215	0.47	0.43	0.58	0.65
	590	**0.41**	**0.39**	**0.53**	**0.60**	390	0.43	0.40	0.55	0.62
						540	**0.38**	**0.36**	**0.51**	**0.58**
100msa	0	0.61	0.55	0.73	0.83	0	0.51	0.47	0.65	0.75
	165	0.60	0.53	0.67	0.74	150	0.50	0.45	0.60	0.67
	415	0.45	0.42	0.57	0.64	225	0.47	0.43	0.58	0.65
	615	**0.38**	**0.37**	**0.51**	**0.58**	375	0.43	0.40	0.56	0.63
						550	**0.37**	**0.35**	**0.51**	**0.58**
150msa	0	0.57	0.52	0.70	0.80	0	0.47	0.44	0.62	0.71
	185	0.56	0.49	0.64	0.71	175	0.46	0.42	0.57	0.64
	435	0.43	0.40	0.55	0.62	250	0.43	0.40	0.55	0.62
	635	**0.37**	**0.35**	**0.50**	**0.57**	400	0.40	0.38	0.53	0.60
						600	**0.34**	**0.33**	**0.48**	**0.55**

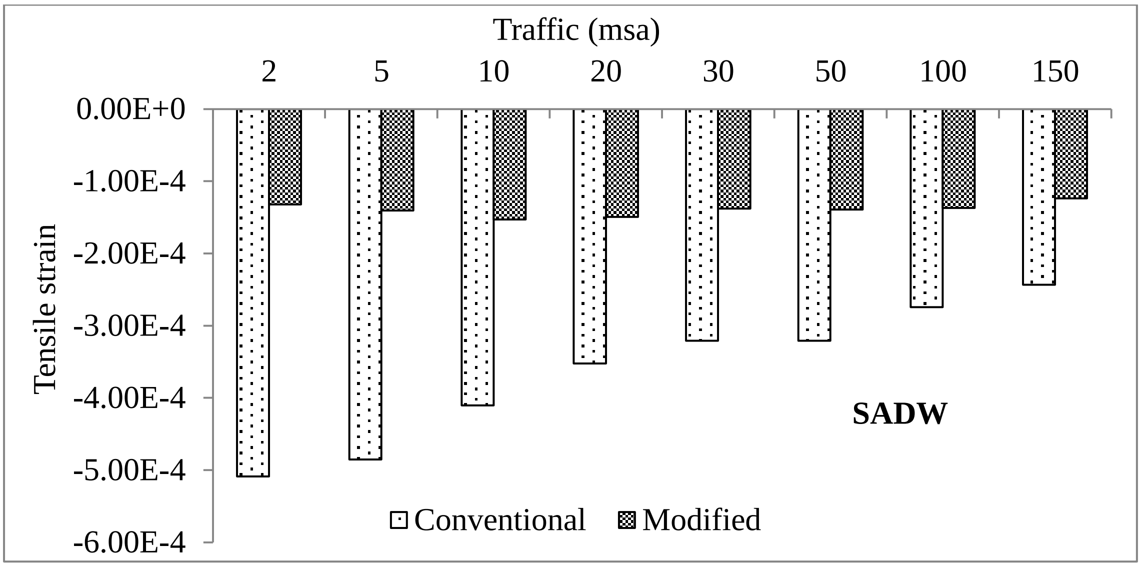
Table 8.11 Vertical stress for conventional and modified cases for flexible pavements

Vertical Stress (kPa)										
Traffic	H (mm)	Conventional				H (mm)	Modified			
		SASW	SADW	Tandem	Tridem		SASW	SADW	Tandem	Tridem
2msa	0	700	700	700	700	0	700	700	700	700
	70	489.04	367.46	367.51	367.55	70	574.57	496.92	496.97	497.09
	295	125.40	86.94	87.45	87.45	145	328.99	241.22	241.50	241.49
	445	**60.30**	**47.12**	**47.95**	**47.97**	270	92.27	66.96	67.58	67.56
						395	**50.67**	**40.52**	**41.43**	**41.41**
5msa	0	700	700	700	700	0	700	700	700	700
	75	467.69	348.10	348.16	348.20	75	556.48	473.84	473.92	474.01
	325	105.85	74.96	75.60	75.61	150	315.10	228.84	229.14	229.13
	475	**52.90**	**42.11**	**43.08**	**43.10**	275	88.92	64.72	65.38	65.36
						400	**49.16**	**39.45**	**40.41**	**40.39**
10 msa	0	700	700	700	700	0	700	700	700	700
	100	364.48	263.66	263.78	263.80	100	478.01	378.27	378.49	378.60
	350	90.58	65.76	66.58	66.60	175	273.10	189.23	189.76	189.91
	550	**38.48**	**32.05**	**33.33**	**33.38**	325	71.05	52.32	53.41	53.63
						500	**32.97**	**27.74**	**29.25**	**29.32**
20 msa	0	700	700	700	700	0	700	700	700	700
	125	284.85	203.45	203.63	203.64	120	415.19	314.36	314.66	314.67
	375	74.62	55.95	56.86	56.88	195	234.21	159.86	160.50	160.49
	575	**33.70**	**28.61**	**29.99**	**30.05**	345	64.84	48.20	49.42	49.42
						545	**27.94**	**23.96**	**25.66**	**25.69**
30 msa	0	700	700	700	700	0	700	700	700	700
	140	248.02	176.48	176.69	176.69	140	364.97	265.66	266.08	266.09
	390	66.89	51.02	51.99	52.02	240	168.28	113.06	113.99	113.99
	590	**31.19**	**26.75**	**28.19**	**28.26**	390	47.29	36.60	38.10	38.11
						540	**26.48**	**22.75**	**24.55**	**24.59**
50 msa	0	700	700	700	700	0	700	700	700	700
	140	248.02	176.48	176.69	176.69	140	362.41	263.97	264.39	264.40
	390	66.89	51.02	51.99	52.02	215	205.21	138.23	139.03	139.02
	590	**31.19**	**26.75**	**28.19**	**28.26**	390	47.40	36.69	38.19	38.20
						540	**26.57**	**22.83**	**24.62**	**24.66**
100 msa	0	700	700	700	700	0	700	700	700	700
	165	199.66	141.66	141.93	141.93	150	332.57	239.27	239.73	239.73
	415	56.37	44.08	45.15	45.19	225	183.98	124.45	125.27	125.27
	615	**27.54**	**23.98**	**25.52**	**25.60**	375	52.06	39.80	41.22	41.22
						550	**26.00**	**22.45**	**24.24**	**24.28**

150 msa	0	700	700	700	700	0	700	700	700	700
	185	169.75	120.44	120.78	120.78	175	278.46	194.32	194.89	194.90
	435	49.61	39.46	40.63	40.67	250	154.81	104.68	105.62	105.63
	635	**25.04**	**22.02**	**23.66**	**23.74**	400	46.87	36.25	37.79	37.82
						600	**21.93**	**19.27**	**21.22**	**21.29**



(a)



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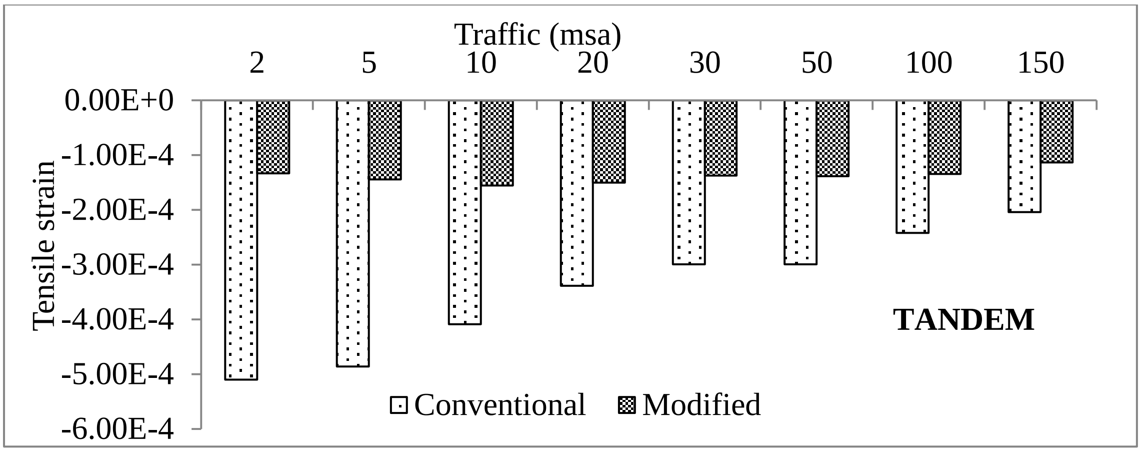

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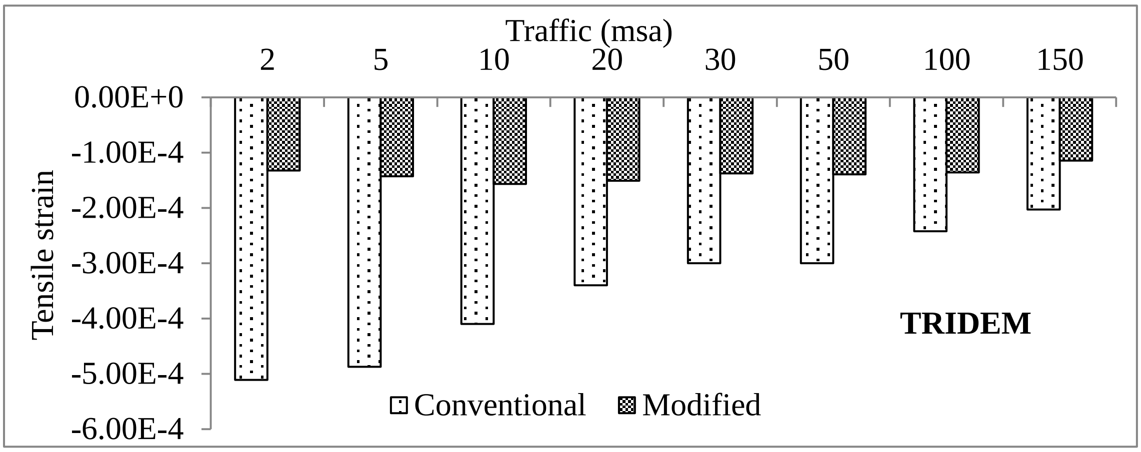
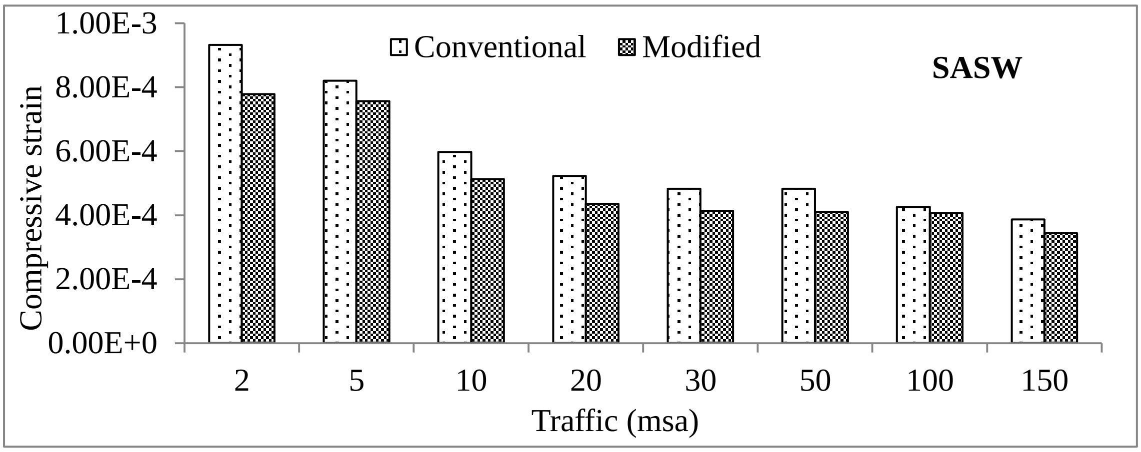
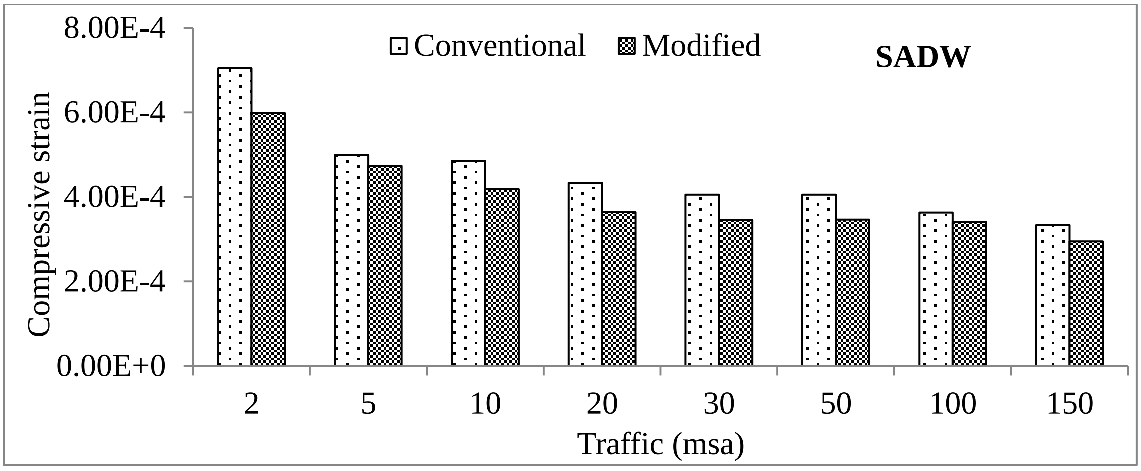


(d)

Fig. 8.11 (a-d) Variations of tensile strain for conventional and modified for flexible pavement



(a)



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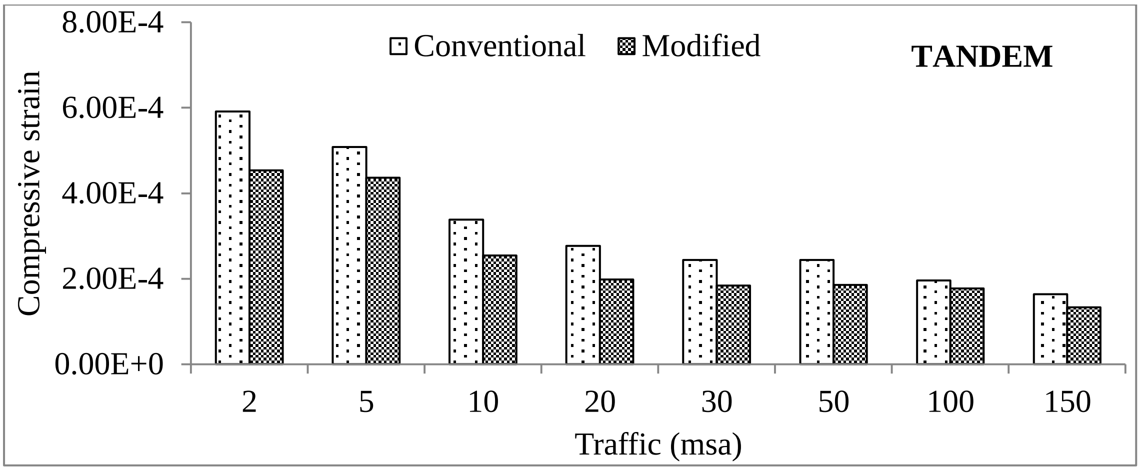

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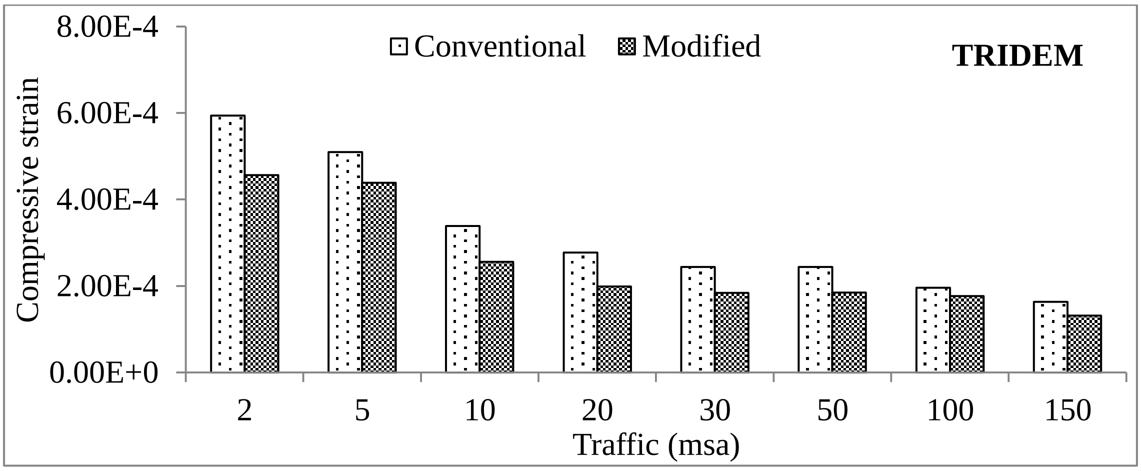

(d)

Fig.8.12 (a-d) Variations of compressive strain for conventional and modified for high volume road

Table 8.12 Fatigue and Rutting values for conventional and modified cases with different axle loads

Traffic (msa)	SASW				SADW			
	N_F		N_R		N_F		N_R	
	C	M	C	M	C	M	C	M
2	2.27E+06	1.30E+09	2.29E+06	5.18E+06	2.50E+06	4.69E+08	8.13E+06	1.70E+07
5	2.55E+06	7.85E+08	4.08E+06	5.91E+06	3.00E+06	3.71E+08	1.32E+07	1.91E+07
10	3.83E+06	2.76E+08	1.72E+07	3.44E+07	5.75E+06	2.68E+08	4.43E+07	8.64E+07
20	5.95E+06	2.20E+08	3.16E+07	7.19E+07	1.04E+07	2.93E+08	7.38E+07	1.63E+08
30	8.06E+06	2.46E+08	4.51E+07	9.10E+07	1.50E+07	4.01E+08	9.99E+07	2.06E+08
50	8.06E+06	2.06E+08	4.51E+07	9.47E+07	1.50E+07	3.87E+08	9.99E+07	2.04E+08
100	1.38E+07	2.33E+08	7.97E+07	9.87E+07	2.74E+07	4.10E+08	1.64E+08	2.18E+08
150	2.13E+07	3.05E+08	1.23E+08	2.11E+08	4.39E+07	6.11E+08	2.41E+08	4.23E+08
Traffic (msa)	Tandem				Tridem			
	N_F		N_R		N_F		N_R	
	C	M	C	M	C	M	C	M
2	2.47E+06	4.55E+08	1.80E+07	5.99E+07	2.45E+06	4.72E+08	1.77E+07	5.83E+07
5	2.98E+06	3.37E+08	3.58E+07	7.14E+07	2.95E+06	3.48E+08	3.52E+07	6.94E+07
10	5.83E+06	2.52E+08	2.28E+08	8.25E+08	5.78E+06	2.44E+08	2.24E+08	8.05E+08
20	1.22E+07	2.87E+08	5.64E+08	2.55E+09	1.20E+07	2.81E+08	5.58E+08	2.53E+09
30	1.97E+07	4.06E+08	1.00E+09	3.57E+09	1.94E+07	4.00E+08	9.94E+08	3.57E+09
50	1.97E+07	3.92E+08	1.00E+09	3.47E+09	1.94E+07	3.86E+08	9.94E+08	3.47E+09
100	4.49E+07	4.44E+08	2.71E+09	4.27E+09	4.47E+07	4.26E+08	2.70E+09	4.27E+09
150	8.76E+07	8.52E+08	6.10E+09	1.56E+10	8.86E+07	8.32E+08	6.15E+09	1.61E+10

Table 8.13 Damage ratio for conventional and modified with different axle loads

Traffic (msa)	Damage Ratio (%)							
	SASW		SADW		Tandem		Tridem	
	C	M	C	M	C	M	C	M
2	0.09	0.04	0.08	0.01	0.16	0.01	0.25	0.02
5	0.20	0.08	0.17	0.03	0.34	0.03	0.51	0.04
10	0.26	0.03	0.17	0.01	0.34	0.01	0.52	0.01
20	0.34	0.03	0.19	0.01	0.35	0.01	0.52	0.02
30	0.37	0.03	0.20	0.01	0.34	0.02	0.50	0.02
50	0.62	0.05	0.33	0.02	0.57	0.03	0.83	0.04
100	0.73	0.10	0.36	0.05	0.55	0.05	0.78	0.07
150	0.70	0.07	0.34	0.04	0.47	0.04	0.64	0.06

8.8.5 IITPAVE Analysis Results for High Volume Roads

IITPAVE results for conventional and modified sections for CBR 8% (shown in Figures 8.9 and 8.10) are presented in Table 8.14. The vertical stress (SigmaZ), displacement (DispZ), vertical strain (epZ) and tangential strain (epT) above the subgrade are reduced for modified sections, indicating that they are better than the conventional sections. The IRC restricts strains to check the safety of pavement composition. The allowable horizontal tensile strain in the bituminous layer and the allowable vertical compressive strain on the subgrade should be less than 178E-06 and 291E-06 respectively (for sections with VG 30 bituminous layers). From the results it can be seen that, all sections with stabilized soil (except the one for 2msa traffic) satisfy these criteria ensuring that the modified sections are safe.

Table 8.14 Pavement responses from IITPAVE for conventional and modified cases

Traffic	H (mm)	SigmaZ (kPa)	DispZ (mm)	epZ	epT	H (mm)	SigmaZ (kPa)	DispZ (mm)	epZ	epT
	Conventional					Modified				
2msa	0	691.10	0.107	76.9E-6	1.72E-4	0	691.10	0.082	125.2E-6	1.23E-4
	70	76.66	0.942	398.3E-6	1.47E-4	70	141.90	0.069	190.5E-6	6.85E-5
	295	9.50	0.620	82.2E-6	3.11E-4	145	37.01	0.061	68.9E-6	2.91E-5
	445	**6.13**	**0.531**	**85.0E-6**	**9.35E-2**	270	7.42	0.056	71.8E-6	4.08E-5
						395	**4.24**	**0.049**	**63.3E-6**	**4.19E-4**
5msa	0	691.10	0.102	88.5E-6	1.61E-4	0	691.10	0.081	128.6E-6	1.20E-4
	75	69.45	0.088	350.8E-6	1.31E-4	75	126.60	0.067	170.9E-6	6.23E-5
	325	8.14	0.058	71.3E-6	1.30E-4	150	34.44	0.060	64.5E-6	2.78E-5
	475	**5.36**	**0.050**	**72.6E-6**	**1.55E-1**	275	7.14	0.055	69.4E-6	3.07E-5
						400	**4.12**	**0.048**	**61.8E-6**	**8.78E-4**
10msa	0	691.10	0.102	88.5E-6	1.61E-4	0	691.10	0.072	141.6E-6	1.07E-4
	75	69.45	0.088	350.8E-6	1.31E-4	100	76.49	0.055	104.6E-6	4.07E-5
	325	8.14	0.058	71.3E-6	1.30E-4	175	25.66	0.051	47.7E-6	2.08E-5
	475	**5.36**	**0.050**	**72.6E-6**	**1.55E-1**	325	5.67	0.046	48.2E-6	2.35E-5
						500	**2.30**	**0.039**	**35.8E-6**	**5.61E-4**
20msa	0	691.10	0.081	133.5E-6	1.18E-4	0	691.10	0.068	146.7E-6	1.02E-4
	125	29.50	0.064	143.2E-6	5.73E-5	120	54.88	0.050	76.1E-6	3.03E-5
	375	5.92	0.047	45.6E-6	6.53E-4	195	20.57	0.047	38.7E-6	1.78E-5
	575	**3.08**	**0.040**	**45.6E-6**	**1.41E+0**	345	5.19	0.043	41.8E-6	2.16E+3
						545	**2.54**	**0.037**	**18.4E-6**	**7.10E-3**

30msa	0	691.10	0.078	138.5E-6	1.12E-4	0	691.10	0.065	150.2E-6	9.86E-5
	140	24.17	0.060	117.7E-6	4.82E-5	140	41.48	0.047	57.8E-6	2.38E-5
	390	5.47	0.046	42.2E-6	3.73E-3	240	13.46	0.043	26.6E-6	1.30E-5
	590	**7.27**	**0.042**	**62.2E-6**	**1.73E+0**	390	4.06	0.040	37.2E-6	4.52E-5
						540	**1.74**	**0.035**	**21.7E-6**	**1.18E+0**
50msa	0	691.10	0.078	138.5E-6	1.12E-4	0	691.10	0.065	150.0E-6	9.88E-5
	140	24.17	0.060	117.7E-6	4.82E-5	140	41.24	0.047	57.7E-6	2.40E-5
	390	5.47	0.046	42.2E-6	3.73E-3	215	16.99	0.044	32.2E-6	1.52E-5
	590	**7.27**	**0.042**	**62.2E-6**	**1.73E+0**	390	3.72	0.040	35.6E-6	1.13E-3
						540	**3.46**	**0.037**	**46.4E-6**	**1.16E+0**
100msa	0	691.10	0.073	144.1E-6	1.06E-4	0	691.10	0.065	150.6E-6	9.82E-5
	165	18.08	0.055	88.5E-6	3.73E-5	150	35.82	0.047	51.0E-6	2.20E-5
	415	4.67	0.043	35.4E-6	4.25E-3	225	15.02	0.044	29.4E-6	1.51E-5
	615	**7.81**	**0.041**	**83.9E-6**	**5.73E+0**	375	4.20	0.041	38.5E-6	4.83E-5
						550	**2.09**	**0.036**	**32.6E-6**	**8.28E-3**
150msa	0	691.10	0.070	147.2E-6	1.03E-4	0	691.10	0.062	153.0E-6	9.56E-5
	185	14.67	0.051	72.3E-6	3.13E-5	175	26.64	0.043	38.7E-6	1.76E-5
	435	6.60	0.040	49.9E-6	3.34E-2	250	12.16	0.041	24.3E-6	1.33E-5
	635	**429.10**	**0.266**	**4.6E-3**	**4.64E+1**	400	3.49	0.038	32.1E-6	4.05E-5
						600	**8.57**	**0.036**	**217.9E-6**	**6.52E-2**

8.9 MAJOR FINDINGS

An attempt has been made to study the effect of stresses and strains in flexible pavement subjected to SASW, SADW, tandem and tridem axle loading using KENPAVE software for low and high volume roads. IITPAVE software was also utilized for the analysis of high volume roads considering dual wheel loading. Based on the analysis the following observations are made:

- It is observed from the KENPAVE analysis that, when the number of wheels and axles increases the stresses and strains in the pavement layers decreases, i.e. the stress is more for SASW and less for SADW, Tandem and Tridem axle loading.
- The modified subgrade layer can be replaced by cement Arecanut coir soil without affecting the strength characteristics and the thickness chart is provided for low volume roads.

- Fatigue and Rutting lives are improved for all stabilized cases by 3.4 to 5.7 times and 1.3 to 1.8 times respectively for low volume roads.
- The cement aggregate stabilized soil can be used in place of base layer for high volume roads and a design chart is prepared for high volume roads by using a soil cement aggregate mix and an aggregate interlayer.
- Fatigue and Rutting lives are significantly improved for all high volume pavement sections by 9.4 to 572.7 times and 1.2 to 4.5 times respectively.
- Compressive and tensile strains and damage ratios were observed to be reduced in all modified cases compared to conventional cases.
- IITPAVE analysis showed that, all modified sections for high volume pavements in this study, satisfy the tensile and compressive strain criteria suggested by IRC.

8.10 COST ANALYSIS

8.10.1 Basic Material Cost

Alternate design has been established for all cases as per the design catalogue in the IRC SP: 72-2007 and IRC 37-2012 for each type of soil. Cost analysis has been carried out based on Schedule of Rates (SOR) 2014, Mangalore Public Works Department, and Karnataka. Cost Analysis included material cost, cost of construction, labour cost, transportation cost etc. Table 8.15 tabulated the basic material cost.

Table 8.15 Basic material cost

Sl. No.	Description	Amount (Rs)	Total Amount (Rs)/CUM
1	**Sand**		
	Basic Cost /CUM	1200.00	
	Add bulkage/Wastage/Royalty 20%	240.00	
	Total Cost /CUM	1440.00	1500.00
2	**Gravel Soil**		
	Basic Cost /CUM	240.00	
	Add bulkage/Wastage/Royalty 20%	50.00	
	Total Cost /CUM	290.00	300.00
3	**Stone Aggregate**		
	Basic Cost /CUM	1200.00	
	Add bulkage/Wastage/Royalty 20%	240.00	
	Total Cost /CUM	1440.00	1500.00
4	**Available soil**		
	Basic Cost /CUM	100.00	
	Add transportation cost 20%	30.00	
	Total Cost /CUM	130.00	150.00
5	**Providing and Laying of close graded, Grading II GSB**		
	Cost/CUM	2347.00	
	Cost of placing/labour charges etc. (25%)	586.00	
	Total Cost /CUM	2933.00	3000.00
6	**Providing & Laying Close graded WMM**		
	Cost/CUM	2100.00	

	Cost of placing/labour charges etc. (30%)	630.00	
	Total Cost /CUM	2730.00	3000.00
7	**WBM-I Metal ,Type A (13.2mm)**		
	Cost/CUM	2626.00	
	Cost of placing/labour charges etc. (30%)	788.00	
	Total Cost /CUM	3414.00	3500.00
8	**WBM-II Metal, Type B (11.2mm)**		
	Cost/CUM	2705.00	
	Cost of placing/labour charges etc. (30%)	811.00	
	Total Cost /CUM	3516.00	3600.00
9	**Bituminous Concrete**		
	Cost/CUM	11097.00	
	Cost of mixing/placing/labour charges, Power, Water, Tools and Tackles etc. (10%)	1110.00	
	Total Cost /CUM	12207.00	12300.00
10	**Dense Bituminous Macadam**	9531.00	
	Cost of mixing/placing/labour charges etc. (10%)	953.00	
	Cost/m^2	10484.00	10500.00
11	**Liquid Seal Coat**		
	Providing and laying seal coat using grit consisting of 0.06 cm of fine aggregate/10cm including cleaning the surface and pre mixed with 80/100 grade bitumen at 6.8kgs/10 m^2 including spreading rolling etc.	100.00/ m^2	
	Total Cost /m^2	100.00/ m^2	
	Cost / m^2		100.00/ m^2

8.10.2 Cost Analysis and Comparison

Cost effectiveness of the modified pavement sections by replacing the conventional material with stabilized soil has been evaluated and represented in the Tables 8.16 and 8.17. Cost was calculated by maintaining one meter length and one meter breadth for all sections and the corresponding thickness was considered for each layer. The detailed cost analysis is given in Appendix III.

Table 8.16 Cost comparison of standard and modified cases for low volume roads

Cases	For Standard Case (IRC: SP: 72-2007)		For Modified Case (For optimum dosage)		Saving in cost (%)
	Total thickness of section (mm)	Total cost of section (Rs.)	Total thickness of section (mm)	Total cost of section (Rs.)	
S2T4	**375** Bitumen treated WBM 75mm + WBM 100 mm + GSB 100 mm + Modified SG 100 mm	953.00	**325** Bitumen treated WBM 75mm + WBM 100 mm + GSB 75 mm + Modified SG 100 mm	858.00	10%
S2T5	**425** Bitumen treated WBM 75mm + WBM 100mm + GSB 100mm + Modified SG 150mm	968.00	**400** Bitumen treated WBM 75mm + WBM 100mm + GSB 100mm + Modified SG 150mm	938.00	3%
S2T6	**475** Bitumen treated WBM 75mm + GB 150mm + GSB 100mm + Modified SG 150mm	1148.00	**450** Bitumen treated WBM 75mm + WBM 125mm + GSB 100mm + Modified SG 150mm	1028.00	10%
S2T7	**555** Bitumen treated WBM 75mm + WBM 150mm+ GSB 150mm+ Modified SG 150mm	1298.00	**480** Bitumen treated WBM 75mm + WBM 100mm+ GSB 100mm+ Modified SG 125mm	1265.00	3%

Table 8.17 Cost comparison of standard and modified cases for high volume roads

Cases	Standard case (IRC:37-2012)		Modified case (For optimum dosage)		Saving in cost (%)
	Total thickness of section (mm)	Total cost of section (Rs.)	Total thickness section (mm)	Total cost of section (Rs.)	
2msa	**445** BC20mm+ DBM 50mm + GB 225mm + GSB 150mm	1896.00	**400** BC20mm+ DBM 50mm + AI 75mm+ CT 125mm + GSB 125mm	1413.00	25 %
5msa	**475** BC25mm+DBM 50mm + GB 250mm + GSB150mm	2033.00	**395** BC20mm+DB M 50mm + AI 75mm+CT 125mm + GSB 125mm	1474.00	27%
10msa	**550** BC40mm+ DBM 60mm + GB 250mm + GSB 200mm	2472.00	**500** BC40mm+ DBM 60mm + AI 75mm+ CT 150mm + GSB 175mm	1944.00	21%
20msa	**575** BC40mm+ DBM 85mm + GB 250mm + GSB 200mm	2735.00	**545** BC40mm+ DBM 80mm + AI 75mm+ CT 150mm + GSB 200mm	2229.00	18%
30msa	**590** BC40mm+ DBM 100mm + GB 250 mm + GSB 200 mm	2892.00	**540** BC40mm+ DBM 100mm + AI 100mm+ CT 150 mm + GSB 150 mm	2320.00	20%
50msa	**590** BC40mm+ DBM 100mm + GB 250 mm + GSB 200 mm	2892.00	**540** BC40mm+DB M100mm+AI75 mm+CT175m+ GSB 150 mm	2320.00	20%
100msa	**615** BC50mm+ DBM 115mm + GB 250 mm + GSB 200 mm	3173.00	**550** BC45mm+ DBM105mm+ AI7mm+CT 150mm+GSB 175 mm	2471.00	22%

150msa	**635** BC50mm+ DBM 135mm + GB 250 mm + GSB 200 mm	3383.00	**600** BC50mm+ DBM 125mm + AI 75mm+ CT 150 mm + GSB 200 mm	2825.00	16%

The savings in cost of construction in both low and high volume roads mainly depends upon sub grade strength and traffic intensity. It also depends upon the thickness of the standard and stabilized layers. It has been observed that for the proposed sections, the savings in low volume roads vary between 3 to 10% and in high volume roads between 16 to 27%.

CHAPTER 9

CONCLUSIONS

The major conclusions drawn from the studies are listed below:

9.1 LATERITIC SOIL STABILIZATION

Improvement of LS1 and LS2 through stabilization with Terrasil, Terrabind, Aggregate, Arecanut coir, FA, Cement, and RBI 81 allows the soils to be used as a subgrade or base material for pavements. Also soil stabilization generally improves the pavement performance by maintaining the resources in a cost effective way.

- Compared to the untreated soil, the soaked CBR increased by 2, 3 and 5 times for LS1+Terrasil, LS1+6%FA and LS1+Terrabind+6%FA respectively, after 28 days curing. Therefore these combinations can be used as modified subgrade layer for low volume roads.
- The addition of Arecanut coir along with 3% of cement by weight of soil resulted in significant increase in the UCS (1-3 times) and CBR values (5-13 times) for 7 and 28 curing days.
- Cement was observed to be an effective stabilizer and further addition of aggregates improves the soil properties.
- Resilient modulus for LS2, LS2+6% C and LS2+6% C+30% aggregate treated samples exhibit higher E values for cement aggregate combination at higher confining pressure.
- Cement stabilized LS2 satisfies the usual strength requirements (UCS and CBR) and durability criterion (maximum allowable loss in strength of 20%) at economic cement content of 6%.
- 6% RBI 81 treated specimens showed 5.9 to 14.7 times increase in the UCS compared to untreated soil and soaked CBR strength was above 90% with the addition of aggregates.

9.2 BC SOIL STABILIZATION

- The UCS enhancement for BC+Terrabind, BC+FA and BC+Terrabind+FA was 4.2, 3.0 and 5.2 times that of the natural BC soil respectively, after 28 days curing. The CBR improved by 2 to 12 times for different curing periods.
- The Terrabind and FA stabilization controlled the critical swelling problem of soil, by significantly reducing the FSI from 50% to 2%.
- The Terrasil treatment made the soil stiff and impermeable. The optimum dosage was obtained as 1.2% based on the UCS and CBR values.
- Swelling has been effectively reduced for RBI 81 treated soil, and FSI values decreased with dosage.
- RBI 81 treatment increased CBR values from 1% to 18% for 28 days curing.

Considering these findings, it can be concluded that all combinations can be used to stabilize BC soils. The guidelines by IRC suggest pavement sections for different traffic volumes and subgrade conditions starting from 2% soaked CBR. It also recommends using soil with CBR > 10% as modified subgrade layer in some cases. In this study, the stabilized soils successfully achieved the soaked CBR criteria and other tests confirmed its suitability as a pavement material.

9.3 DURABILITY STUDIES

- Both WD and FT cycles caused variations in volume, but it was more significant during drying wetting cycles.
- Liquid stabilizers are failed in wetting cycles due to less bonding and strength for both lateritic and BC soils.
- Soaking has significant effect on stabilized samples. The strength initially increased, and then decreased with increasing soaking periods. Higher amounts of (>6%) cement and RBI 81 addition only succeeded in improving the strength of soil against WD test.

- The stabilized BC soil specimens could not withstand six WD cycles for Terrasil and Terrabind mixtures, whereas, all samples passed the FT cycles criterion within 14% weight loss.

9.4 IDT STRENGTH AND FATIGUE STUDIES

- A high IDT strength was observed for cement, RBI 81 along with aggregate treated specimens.
- Fatigue life increased for treated lateritic soil and the enhancement was improving with coir dosage.
- Treatment of Terrabind and FA provided 6 to13 times higher fatigue life to the BC soil.

9.5 KENPAVE, IITPAVE AND COST ANALYSIS

- KENPAVE analysis showed the reduction in stresses, strains and displacement values for pavement sections with stabilized soil.
- Enhanced life span of modified pavement structure was proved from damage analysis, with significant fatigue and rutting life improvement for both low and high volume roads.
- Damage ratios were reduced for all the stabilized cases.
- Analysis showed that, modified subgrade layer can be replaced by cement Arecanut coir soil for low volume roads and cement aggregate stabilized soil can be used in place of base layer for high volume roads, without affecting the strength characteristics.
- It was observed from IITPAVE analysis that, all modified sections for high volume pavements in this study, satisfy the tensile and compressive strain criteria suggested by IRC.
- Cost analysis indicates saving of 3 to 10% in low volume roads and 16 to 27% in high volume roads.

9.6 SCOPE FOR FURTHER RESEARCH

- The work can be extended to field track and evaluated for a period of years.
- Studies using different new chemical stabilizers, fibers with different aggregate specifications can be taken up.

APPENDIX I

Dosage 1 calculation for Terrasil treated LS1

OMC = 17.50%, MDD = 1.71g/cc

Volume of the mold = $(\pi d^2/4) \times h$

d = Diameter of the mold, 38mm

h = Height of the mold, 75mm

Weight of soil = MDD × Volume of the mold

= 1.71 × 85.05

= 145.44g

Volume of water = Weight of soil × OMC

= 145.44 × (17.50/100)

=25.45mL

APPENDIX II

Weight loss calculations in Durability test

$$\% \text{ weight loss} = \frac{(\text{Intitial weight} - \text{Final weight})}{\text{Initial weight}} \times 100$$

For LS1+ 1.2% Terrasil + 2% C Specimen,

Initial weight	= 194.84g
Weight after wet cycle	= 201.02g
Weight loss	= 100 × (194.84 – 201.02) / 194.84
	= – 3.17%
Weight after dry cycle	= 181.24
Weight loss	= 100 × (194.84 – 181.24) / 194.84
	= 6.98%

APPENDIX III

Cost analysis

Lateritic soil 2 treated with 6% cement

Weight of one m^3 of treated soil = MDD in g/cc × $1m^3$

= 2.133 ×1000 Kg = 2133 Kg

Weight of soil in the sample = 2133/1.0 6 = 2012 Kg

Weight of cement in the sample = 2012 × 6/100 = 121 Kg

Cost of 2012 Kg of lateritic soil = 2012 × 150/2040 = Rs.148

Cost of 121 Kg of cement = 121 × 400/50 = Rs.968

Cost of one m^3 of treated soil = 148 + 968 = Rs.1116

Laterite soil 2 treated with 6% Cement+20% aggregate

Weight of one m^3 of treated soil = MDD x $1m^3$

= 2.21 × 1000 kg = 2210 kg

Weight of soil sample = 2210/(1+0.06+0.2) = 1754 kg

Weight of 20% Aggregates in the sample = 1754 × 0.2 = 351 kg

Weight of Cement = 1754 × 0.06= 105kg

Cost of 1754 kg of soil = 1754 × 150/2040 = Rs. 129

Cost of 105 kg of Cement = 105 × 400/50 = Rs. 840

Cost of 351 kg of aggregates = 351 × (1500/2720) = Rs. 194

Total cost of 1 m^3 of treated soil with 6% cement & 20% of Aggregates = 129+840+194 = Rs. 1163

Laterite soil 2 treated with 6% Cement+25% aggregate

Weight of one m^3 of treated soil = MDD x $1m^3$

= 2.26 × 1000 kg = 2260 kg

Weight of soil sample = 2260/(1+0.06+0.25) = 1724 kg

Weight of 25% Aggregates in the sample = 1754 × 0.25 = 431 kg

Weight of Cement = 1724 × 0.06= 104kg

Cost of 1754 kg of soil = 1724 × 150/2040 = Rs. 127

Cost of 104 kg of Cement = 105 × 400/50 = Rs. 832

Cost of 431 kg of aggregates = 431 × (1500/2720) = Rs. 238

Total cost of 1 m^3 of treated soil with 6% cement and 25% of Aggregates =127+832+238 = Rs. 1197

<u>Lateritic soil 2 treated with 6% cement and 30% aggregate</u>

Weight of one m^3 of treated soil = MDD × $1m^3$ = 2.33 × 1000 kg = 2330 kg

Weight of soil sample = 2330/(1+0.06+0.3) = 1713 kg

Weight of 20% Aggregates in the sample = 1713 × 0.3 = 514 kg

Weight of Cement = 1713 × 0.06 = 103kg

Cost of 1713 kg of soil = 1713 × 150/2040 = Rs. 126

Cost of 103 kg of Cement = 103 × 400/50 = Rs. 824

Cost of 514 kg of aggregates = 514 × (1500/2720) = Rs. 283

Total cost of 1 m^3 of treated soil with 6% cement and 20% of Aggregates

= 126+824+283 = Rs. 1233

Table Cost calculation for Conventional 2msa 8% CBR section

Item	Rate (Rs./m^3)	L × B (m × m)	H (m)	Quantity (m^3)	Cost (Rs)
1	*2*	*3*	*4*	*5 = 3 × 4*	*6 = 2 × 5*
BC/SDBC	12300	1 × 1	0.020	0.020	246.00
DBM	10500	1 × 1	0.050	0.050	525.00
GB	3000	1 × 1	0.225	0.225	675.00
GSB	3000	1 × 1	0.150	0.150	450.00
TOTAL COST FOR SECTION					1896.00

Table Cost calculation for Modified 2msa 8% CBR section

Item	Rate (Rs./m^3)	L × B (m × m)	H (m)	Quantity (m^3)	Cost (Rs)
1	*2*	*3*	*4*	*5 = 3 × 4*	*6 = 2 × 5*
BC/SDBC	12300	1 × 1	0.02	0.02	246.00
DBM	10500	1 × 1	0.05	0.05	525.00
AI	1500	1 × 1	0.075	0.075	112.50
CT	1687	1 × 1	0.125	0.125	210.88
GSB	3000	1 × 1	0.125	0.125	375.00
TOTAL COST FOR SECTION					1223.38

REFERENCES

Abood, T.T., Anuar, B.K. and Zamri B.C. (2007). “Stabilisation of silty clay soil using chloride compounds.” *J. of Eng. Sci. and Tech.,* 2(1), 102-110.

Al-Amoudi, O.S.B., Asi, I.B., Wahhab, H.I.A. and Khan, Z.A. (2002). “Clegg Hammer—California-Bearing Ratio Correlations.” *J. of Materials in Civil Eng.,* 14(6), 512-523.

Ali, F. (2012). “Stabilization of residual soils using liquid chemical.” *Electr. J. of Geotech. Eng.*, 17, 115-126.

Allam, M.M. and Sridharan, A. (1981). “Effect of wetting and drying on shear strength.” *J. of Geotch. Eng.*, 107, 421-438.

Al-Obaydi, M.A., Al-Kiki, I.M. and Al-Zubaydi, A.H. (2010). “Strength and Durability of Gypseous Soil Treated with Waste lime and Cement.” *J. of Al-Rafidain Eng.*, 18(1), 28-42.

Amadi, A.A. (2014). “Enhancing durability of quarry fines modified black cotton soil subgrade with cement kiln dust stabilization.” *Transp. Geotech.*, 1(1), 55-61.

American Association of State Highways and Transportation Officials (1994) “Standard method of test for determining the resilient modulus of soils and aggregate materials.” AASHTO Designation T 307 – 99. Washington, DC.

Angell, D.J. (1988). “Technical basis for the pavement design manual.” Main Roads Department, Brisbane, Queensland.

Anitha, K.R., Ashalatha, R. and Johnson, A.S. (2009). “Effects of RBI grade 81 on different types of subgrade soil.” *Proc. of 10th National Conference on Technological Trends (NCTT09),* Trivandrum, 165-170.

Anochie-Boateng, J., Tutumluer, E., Apeagyei, A. and Ochieng, G. (2010). “Resilient behavior characterization of geomaterials for pavement design.” *11th International Conference on Asphalt Pavements,* Nagoya, Japan.

Anon. (1990). “State-of art report on soil-cement.” *ACI Materials J.,* 87(4), 395-417.

Arun, U., Kanaujia, V.K., Ranjan, A. and Swami, R.K. (2013). “Effect of purity of lime on strength and durability of soil mixes.” *Ind. High.,* 41(8), 47-54.

ASTM D 559-2003. “Standard test methods for wetting and drying compacted soil cement mixtures.” American Society for Testing and Materials. Philadelphia.

ASTM D 560-2003. “Standard test methods for freezing and thawing compacted soil cement mixtures.” American Society for Testing and Materials. Philadelphia.

Austroads (2009). “Review of relationship to predict subgrade modulus from CBR.” Technical Report. AP–T130/09. ISBN 978-1-921551-23-9.

Aydogmus, T., Alexiew, D. and Klapperich, H. (2004). “Investigation of Interaction Behaviour of Cement-stabilized Cohesive Soil and PVA Geogrids. Geotechnical engineering with geosynthetics.” *Proc. of the third European Geosynthetics Conference*, Munich, Germany.

Bagui, S.K. (2014). “Determination of axle load limits and suggested vehicle damage factor for Indian condition based on a case study evaluation.” *J. of the Indian Roads Congress*, 75(2), 221-225.

Basha, E., Hashim, R., Mahmud, H. and Muntohar, A. (2005). “Stabilization of residual soil with rice husk ash and cement.” *Construct. Build. Mater.*, 19, 448–53.

Bijayananda, M., Mahipal, S.C. and Satyendra, M. (2011). “California bearing ratio of randomly oriented fiber reinforced clayey subgrade for rural roads.” *Proc., of Ind. Geotech. Conf.*, Kochi, India, 611-614.

Black, W.P.M. (1961) “The calculation of laboratory and in-situ values of California bearing capacity data.” *Geotechnique,* 11, 14-21.

Brown, J.J., Brandon, T.L., Daniels, W.L., DeFazio, T.L., Filz, G.M. and Mitchell. J.K. (2004). “Rapid stabilization/polymerization of wet clay soils: Phase I literature review.” Air Force Research Laboratory, Tyndall AFB Florida.

Brown, S.F. (1996). “Soil mechanics in pavement engineering.” *Geotechnique,* 46, 383-426.

Bugge, W. and Bartelsmeyer, R. (1961). “Soil stabilization with Portland cement.” *High. Res. Board*, 292, 1–15.

Bulman, J.N. (1972). “Soil stabilization in Africa.” *Transport and Road Research Laboratory Rep*., LR 476.

Bwalya, M. (2006). “Utilization and improvement of lateritic gravels in road bases.” Int. Institute for Aerospace Survey and Earth Sciences (ITC), Section Engineering Geology, Delft, The Netherlands.

Campco. "Arecanut" http://www.campco.org/index.php/arecanut (Sep. 26, 2015).

Chavva, P.K., Vanapalli, S.K., Puppala, A.J. and Hoyos, L. (2005). "Evaluation of strength, resilient moduli, swell, and shrinkage characteristics of four chemically treated sulfate soils from North Texas." *Geotechnical Special Publication 136: Proc., Geo-Frontiers 2005,* ASCE, Austin, Texas.

Chen, B. (2004). "Polymer-clay nanocomposites: an overview with emphasis on interaction mechanisms." *British Ceramic Transactions,* 103(6), 241-249.

Chen, F.H. (1988). "Foundations on expansive soils." Chen and Associates, 96 South Zuni, Denver, CO80223, USA.

Das, A. and Pandey, B.B. (1999). "Mechanistic-Empirical design of bituminous roads: an Indian perspective." *J. Transp. Eng.*, 125, 463-471.

Das, B.M. (1994). "Principles of foundation engineering." Montegomery, California: Brooks Cole Engineering Division.

Davidson, D.T. and Handy, R.L. (1960). "Soil stabilization." Highway engineering handbook, McGraw–Hill, New York, Sect. 21.

Deepthi, M.D., Praveen, R. and Babu, G.L.S. (2013). "Life-cycle cost analysis of long lasting pavements." *Ind. High.*, 41(4), 5-14.

Dempsey, B.J. and Thompson, M.R. (1968). "Durability Properties of Lime – soil Mixtures." *High. Res. Rec,* 235, 61-75.

Dhakal, S.K. (2012). "Stabilization of very weak subgrade soil with cementitious stabilizer." MS Thesis, Tribhuvan University. Louisiana state university and Agricultural and Mechanical College.

Dorman, G.M. and Edwards, J.M. (1968). "Developments in the application in practice of a fundamental procedure for the design of flexible pavements." *Proc. Second Int. Conf. on Structural Design of Asphalt Pavements*, University of Michigan, USA.

Drumm, E., Boateng, P.Y. and Pierce, T.J. (1990). "Estimation of subgrade resilient modulus from standard tests." *J. of Geotech. Eng,* 116 (5), 774-789.

Eades, J.L, and Grim. R.E. (1960). "Reaction of hydrated lime with pure clay minerals in soil stabilization." *Bulletin 262*, High. Res. Board, Washington, DC.

Eisazadeh, A. (2010). “Physicochemical behavior of lime and phosphoric acid stabilized clayey soil.” Doctoral dissertation, Universiti Teknologi Malaysia.

Ekwulo, E.O. and Eme, D.B. (2009). “Fatigue and rutting strain analysis of flexible pavements designed using CBR methods.” *African J. of Envir. Sci. and Tech.,* 3(12), 412-421.

Erlingsson, S. and Ahmed A.W. (2013). “Fast layered elastic response program for the analysis of flexible pavement structures.” *Road Materials and Pavement Design*. 14(1), 196-210.

Far, S.Y.Z., Kassim, K.A., Eisazadeh, A. and Khari, M. (2013). “An evaluation of the tropical soils subjected physicochemical stabilization for remote rural roads.” *Procedia Eng.,* 54, 817-826.

Frederick, L., Ulf, I. and Andrew, D. (2000). “State of the art. I: Resilient response of unbound aggregates.” *J. Transp. Eng,* 126(1), 66-75.

Garg, S (2014). “Flexible pavement design in India: past, present and future.” *Ind. High.,* 63 – 79.

Gedafa, D.S. (2006). “Comparison of flexible pavement performance using KENLAYER and HDM-4.” *Proc. of Fall Student Conference,* Ames, Iowa: Midwest Transportation Consortium, 1-14.

Geiman, C.M. (2005). “Stabilization of soft clay subgrades in Virginia phase I laboratory study.” Master of Science thesis, Virginia Polytechnic Institute and State University, Blacksburg.

Gidigasu, M.D. (1971). “A contribution to the study of the physico-chemical implications of tropical weathering and laerisation.” *J. Southeast Asian soc. Soil Eng.,* Bangkok, 2(2), 131-149.

Gomez, J.L. and Anderson, D.M. (2012). “Soil cement stabilization – mix design, control and results during construction.” *International Symposium on Ground Improvement.* Brussels.

Gow, A.J., Davidson, D.T. and Sheeler, J.B. (1961). “Relative effects of chlorides, lignosulfonates and molasses on properties of a soil-aggregate mix.” *High. Res. Board,* 282, 66-83.

Green, J.L. and Hall, J.W. (1975). “Non-destructive vibratory testing of airport pavements: experimental tests results and development of evaluation methodology and procedure." FAA-RD-73-205, Federal Aviation Administration, Washington, DC.

Gunturi, M., Ravichandran, P.T., Annadurai, R. and Divya, K.K. (2014). "Effect of RBI 81on CBR and swell behaviour of expansive soil." *Int. J. of Eng. Res.,* 3(5), 336-339.

Gupta, S. and Veeraragavan, A. (2009). "Fatigue behaviour of polymer modified bituminous concrete mixtures." *J. of the Ind. Roads Cong.,* 548, 55-64.

Haichert, R., Kelln, R., Wandzura, C., Berthelo,t C. and Guenther, D. (2012). "Cement stabilization of conventional granular base and recycled crushed Portland cement concrete." *Transp. Res. Rec.: J. Transp. Res. Board*, 2310, 121-126.

Harty, J.R. (1970). "Factors influencing the lime reactivity of tropically and subtropically weathered soils." Ph.D. Dissertation, Department of Civil Engineering, University of Illinois, Urbana.

Heukelom, W. and Klomp, A.J.G. (1962). "Dynamic testing as a means of controlling pavements during and after construction." *Proc. of the 1st Int. conf. on the design of asphalt pavements,* Ann Arbor, University of Michigan, United States, 667-79.

Hicks, R.G. (2002). "Alaska Soil Stabilization Design Guide." FHWA-AKRD- 01-6B. FHWA, U.S. Department of Transportation.

Horpibulsuk, S., Katkan, W. and Apichatvullop, A. (2008). "An approach for assessment of compaction curves of fine-grained soils at various energies using a one point test." *Soils Found*, 48(1), 115-125.

Horpibulsuk, S., Katkan, W., Sirilerdwattana, W. and Rachan, R. (2006). "Strength development in cement stabilized low plasticity and coarse grained soils: laboratory and field study." *Soils Found.*, 46(3), 351-366.

Horpibulsuk, S., Miura, N. and Nagaraj, T.S. (2003). "Assessment of strength development in cement-admixed high water content clays with Abrams' law as a basis." *Geotechnique,* 53(4), 439-444.

Houssain, K.M.A. (2010). "Development of stabilized soils for construction applications." *Ground Improv. J.,* 163(3), 173-185.

Ibrahaim, M.A., Moafaq, A.A. and Abdulrahman, H.A. (2011). "Long term strength and durability of clayey soil stabilized with lime." *Eng. and Tech. J.,* 29(4), 725-735.

IRC: 37-2012. "Tentative guidelines for the design of flexible pavements." The Indian

Roads Congress, New Delhi.

IRC: SP: 72-2007. “Guidelines for the design of flexible pavements for low volume rural roads.” The Indian Roads Congress, New Delhi.

IS: 2720-10 (1991) (Reaffirmed 2006): Methods of test for soils, Part 10: “Determination of unconfined compressive strength.” Indian Standards Institution, New Delhi, India.

IS: 2720-16 (1987) (Reaffirmed 2002): Methods of test for soils, Part 16: “Laboratory determination of CBR.” Indian Standards Institution, New Delhi, India.

IS: 2720-17 (1986) (Reaffirmed 2002): Methods of test for soils, Part 17: “Laboratory determination of permeability.” Indian Standards Institution, New Delhi, India.

IS: 2720-26 (1987) (Reaffirmed 2002): Method of test for soils, Part 26: “Determination of pH value.” Indian Standards Institution, New Delhi, India.

IS: 2720-3-2 (1980) (Reaffirmed 2002): Methods of test for soils, Part 3: “Determination of specific gravity, Section 2: Fine, medium and coarse grained soils.” Indian Standards Institution, New Delhi, India.

IS: 2720-4 (1985) (Reaffirmed 2006): Methods of test for soils, Part 4: “Grain size analysis.” Indian Standards Institution, New Delhi, India.

IS: 2720-5 (1985) (Reaffirmed 2006): Methods of test for soils, Part 5: “Determination of liquid and plastic limit.” Indian Standards Institution, New Delhi, India.

IS: 2720-7 (1980) (Reaffirmed 2011): Methods of test for soils, Part 7: “Determination of water content-dry density relation using light compaction.” Indian Standards Institution, New Delhi, India.

IS: 2720-8 (1983) (Reaffirmed 2006): Methods of test for soils, Part 8: “Determination of water content-dry density relation using heavy compaction.” Indian Standards Institution, New Delhi, India.

Jaritngam, S., Yandell, W.O. and Taneerananon, P. (2013). “Development of strength model of lateritic soil-cement.” *Eng. J.*, 17(1), 69-77.

Kar, R.K. and Pradhan, P.K. (2012). “Laboratory tests of reinforced fly ash mix for use as sub-base in low volume rural roads.” *Ind. High.*, 40(1), 29-35.

Katz, L.E., Rauch, A.F., Liljestrand, H.M., Harmon, J.S., Shaw, K.S. and Albers, H. (2001). “Mechanisms of soil stabilization with liquid ionic stabilizer.” *Transp. Res. Rec.: J. Transp. Res. Board*, 1757(1), 50-57.

Kenai, S. Bahar, R. and Benazzoug, M. (2006). “Experimental analysis of the effect of some compaction methods on mechanical properties and durability of cement stabilized soil.” *J. Mater. Sci.*, 41, 6956-6964.

Khanna, S.K., Justo, C.E.G. and Veeraragavan, A. (2014). “Highway engineering.” Soil stabilized roads, Nem Chand and Bros. Roorkee, India, 601-602.

Khoury, N. and Zaman, M.M. (2007). “Durability of stabilized base courses subjected to wet–dry cycles.” *Int. J. of Pave. Eng.*, 8(4), 265-276.

Khoury, N. and Zaman, M.M. (2002). “Effect of wet-dry cycles on resilient modulus of class c coal fly ash-stabilized aggregate base.” *Transp. Res. Rec.: J. Transp. Res. Board*, 1787, 13-21.

Kim, S., Gopalakrishnan, K. and Ceylan, H. (2012). “Moisture susceptibility of subgrade soils stabilized by lignin-based renewable energy coproduct.” *J. Transp. Eng.*, 138, 1283-1290.

Kumar, P. and Singh, S.P. (2008). “Fiber-reinforced fly ash subbases in rural roads.” *J. Transp. Eng.*, 134, 171-180.

Lakshmanan, M., Kamaraj, C., Lakshmi, S. and Gangopadhyay, S. (2014). “Utilization of fly ash and copper slag mix for reducing the pavement thickness on clayey subgrade.” *Int. conf. on Transp. Plan. and Implementation Methodologies For Developing Countries,* IIT Bombay, India.

Lav, H.M., Lav, M.A. and Goktepe, A.B. (2006). “Analysis and design of a stabilized fly ash as pavement base material.” *Fuel*, 85, 2359-2370.

Leelavathamma, B. and Pandian. N.S. (2005). “Effect of class C fly ash on the California bearing ratio behavior of soil-fly ash mixes and layered systems.” *J. of Testing and Evaluation*, 33(2), 1-6.

Lekha, B.M. and Shankar, A.U.R. (2014). “Laboratory performance of RBI 81 stabilized soil for pavements.” *Int. J. of Civil Eng. Res.*, 5(2), 105-110.

Lekha, K.R. and Sreedevi, B.G. (2005). “Coir fiber for the stabilization of weak subgrade soils.” Highway Engineering Lab, NATPAC, Kerala, India.

Lemes, A.P., Soto-Oviedo, M.A., Innocentini Mei, L.H. and Duran, N. (2005). “Processing, chemical structure and morphology of poly (hydroxybutyrate-co-valerate)/lignosulfonate composites.” *Proc., of the 2nd Mercosur Congress on Chemical Engineering,* Enpromer, Rio de Janeiro, Brazil, 1-6.

Maaitah, O.N. (2012). “Soil stabilization by chemical agent.” *Geotech. Geo. Eng*. 30, 1345-1356.

Maheshwari, K., Desai, A.K. and Solanki, C.H. (2012). “Analytical modeling of flexile pavement resting on fiber reinforced clayey soil.” *Proc. of Ind. Geotech. Conf.* Delhi, India, 1173-1176.

Marto, A., Latifi, N. and Eisazadeh, A. (2014). “Effect of non-traditional additives on engineering and microstructural characteristics of laterite soil.” *Arab J. Sci. Eng*., 39, 6949-6958.

McConnell, T. (2009). “Cement stabilisation applied to highway subgrade.” *T N Cocrete Pavement Conf.*, Portland Cement Association (PCA), Nashville.

McManis, K.L. and Arman, A. (1989). “Class C fly ash as full or partial replacement for portland cement or lime.” *Transp. Res. Rec.: J. Transp. Res. Board*, 1219, 68-81.

Mitchell, J.K. (1981). “Soil improvement state-of-the-art.” *Tenth Int. Conf. on Soil Mechanics and Foundation Engineering,* Sweden, l5-19.

Miura, N., Horpibulsuk, S. and Nagaraj, T.S. (2002). “Engineering behavior of cement stabilized clay at high water content.” *Soils Found.*, 41(5), 33-45.

Mohammad, L.N., Raghavandra, A. and Huang, B. (2000). “Laboratory Performance of Cement-stabilized Soil Base Mixtures.” Transp. Res. Rec., *J. of the Transp. Res. Board.*, 1721, 19-28.

Mollah, M.Y.A., Vempati, R.K., Lin, T.C. and Cocke, D.L. (1995). “The interfacial chemistry of solidification/stabilization of materials in cement and pozzolanic material systems.” *Waste Management*, 15(2), 137-148.

Montgomery, D.E. (1998). “Stabilised Soil Research Progress Report.” University of Warwick, Development Technology Unit School of Engineering, Coventry.

MORT&H. (2014). “Annual Report 2013-2014.” Ministry of Road Transport and Highways, New Delhi, India.

Nicholls, R.L. and Davidson, D.T. (1958). “Polyacids and lignin used with large organic cations for soil stabilization." *High. Res. Board.*, 37, 517-537.

Noor, M.J.M.L. (1994). "Durability and strength characteristics of cement stabilized modified Melaka series." *J. of Islamic Academy of Sciences*, 7(2), 137-141.

Oyediran, I.A. and Kalejaiye, M. (2011). "Effect of increasing cement content on strength and compaction parameters of some lateritic soils from Southwestern Nigeria." *Electr. J. of Geotech. Eng.*, 16, 1501-1514.

Pandian, N.S. and Krishna, K.C. (2001). "CBR behaviour of fly ash murrum mixes." *Ground Improvement*, 5(4), 177-181.

Pandian, N.S. and Krishna, K.C. (2003). "The pozzolanic effect of fly ash on the california bearing ratio behavior of black cotton soil." *J. of Testing and Evaluation*, 31(6), 479-485.

Patel, M.A. and Patel, H.S. (2012). "A review on effects of stabilising agents for stabilisation of weak soil." *Civil and Environmental Research*, 2(6), 1-7.

Patel, R.S. and Desai, M.D. (2010). "CBR predicted by index properties for alluvial soils of South Gujarat." *Proc. of the Indian Geotechnical Conf.*, Mumbai, India.

Paterson, W.D.L. and Maree, J.H. (1978). "An interim mechanistic procedure for the structural design of asphalt pavements." National Institute for Transport and Road Research, Pretoria, South Africa.

Patil, B.M. and Patil, K.A. (2013). "Improvement in properties of subgrade soil by using moorum and RBI Grade 81." *Int. J. of Sci. and Eng. Re*s., 4(5), 289-292.

Peng, Y. and He, Y. (2009). "Structural characteristics of cement stabilized soil bases with 3D finite element method." *Front. Arch. Civ. Eng. China.* 3(4), 428-434.

Portelinha, F.H.M., Lima, D.C., Fontes, M.P.F. and Carvalho. (2012). "Modification of a lateritic soil with lime and cement: An economical alternative for flexible pavement layers." *Soils and Rocks*, 35 (1), 51-63.

Powell, W.D., Potter, J.F., Mayhew, H.C. and Nunn, M.E. (1984). "The structural design of bituminous roads." Laboratory Report 1132, Transport and Road Research Laboratory, Crowthorne, UK.

Praveen, K., Ransichung, G.D., Mehta, M and Saboo. N. (2014). " Performance testing of bituminous mixes using falling weight deflectometer." *Int. Conf. on Transp. Plan. and Implementation Methodologies for Developing Countries*, IIT Bombay, India.

Prusinski, J.R. and Bhattacharja, S. (1999). “Effectiveness of Portland cement and lime in stabilizing clay soils.” *Transp. Res. Rec.: J. Transp. Res. Board,* 1652, 215-227.

Puppala, A.J. and Hanchanloet, S. (1999). “Evaluation of a new chemical (SA-44/LS-40) treatment method on strength and resilient properties of a cohesive soil.” *Proc., of the Transp. Res. Board Annual Meeting,* Washington, DC, USA, 1-11.

Putri, E.E., Rao, N.S.V.K. and Mannan, M.A. (2010), “Evaluation of the modulus of elasticity and resilient modulus for Highway Subgrades.” *Electr. J. of Geotech. Eng,* 15, 1285-1293.

RAASTA Center for Road Technology, Bangalore (2008). “Laboratory studies on properties of soils treated with RBI 81 stabilizer.” Report submitted to Legend Surface Developers Pvt. Ltd, New Delhi.

Ramaswamy, S. and Aziz, M. (1989). “Jute geotextile for roads.” *Proc. of Int. Workshops on Geotextile,* India, 137-143.

Ramulu, G., Shankar, S., Chowdary, V. and Prasad, C.S.R.K. (2012). “Influence of unbound material properties on rutting potential of low volume roads.” *Cement and Concrete Composites,* 42, 6377-6382.

Rao, C.N., George, V. and Shivashankar, R. (2008). “PFWD, CBR and DCP evaluation of lateritic subgrades of Dakshina Kannada, India.” *The 12th Int. Conf. of the International Association for Computer Methods and Advances in Geomechanics*, Goa, India.

Rauch, A.F., Harmon, J.S., Katz, L.E. and Liljestrand, H.M. (2002). “Measured effects of liquid soil stabilizers on engineering properties of clay.” *Transp. Res. Rec.: J. Transp. Res. Board,* 1787(1), 33-41.

Ravinder, K and Sachdeva, S.N. (2014). “Traffic analysis and design of flexible pavement with cemented base and subbase.” *Int. J. of Eng. Res. and Tech.,* 3(7), 1-5.

Rosyidi, S.A., Nayan, K.A.M., Taha, M.R. and Ismail, A. (2006). “Estimating G-max and field CBRof soil subgrade using a seismic method.” *The Online Journal of Nondestructive* Testing, 11 (6).

Sadek, D., Roslan, H and Abubakar, A. (2008). “Engineering properties of stabilized tropical peat soils.” *Electr. J. of Geotech. Eng,* 13, 1-9.

Santoni, R.L., Tingle, J.S and Nieves, M. (2005). "Accelerated strength improvement of silty sand with nontraditional additives." *Transp. Res. Rec.: J. Transp. Res. Board,* 1936, 34-42.

Santoni, R.L., Tingle, J.S. and Webster, S.L. (2002). "Stabilization of silty-sand with nontraditional additives." *Transp. Res. Rec.: J. Transp. Res. Board*, 1787, 33-41.

Sarbaz, H. Ghiassian, H. and Heshmati. A.A. (2014). "CBR strength of reinforced soil with natural fibres and considering environmental conditions." *Int. J. of Pave. Eng.,* 15(7), 577-583.

Scholen, D.E. (1992). "Nonstandard Stabilizers." Report, FHWA-FLP-92-011, US Department of Transportation.

Scholen, D.E. (1995). "Stabilizer mechanisms in nonstandard stabilizers." *Sixth Int. Conf. on Low-Volume Roads*, 6, 252-260.

Selvi, P. (2015). "Fatigue and rutting strain analysis on lime stabilized subgrades to develop a pavement design chart." *Tranp. Geotech.,* 2, 86-98.

Shankar, A.U.R., Chandrashekhar, A. and Prakash, B.H (2012). "Experimental investigation on lithomargic clay stabilized with sand and coir." *Ind. High.*, 40(2), 21-31.

Shankar, A.U.R., Rai, H.K. and Ramesha, M.I. (2009). "Bio-Enzyme stabilized lateritic soil as a highway material." *J. of Ind. Roads Cong.*, 70(2), 143-151.

Shihata, S.A. and Baghdadi, Z.A. (2001). "Simplified Method to Assess Freeze-Thaw Durability of Soil Cement." *J. of Materials in Civil Eng.*, 13(4), 243-247.

Sinha, S.P., Davidson, D.T. and Hoover, J.M. (1957). "Lignins as stabilizing agents for northeastern Iowa loess." *Iowa Academy of Science Proc.*, 64, 314-347.

Sivapullaiah, P.V, Prashanth, J.P, Sridharan, A. and Narayana, B.V. (1998). "Reactive silica and strength of fly ashes." *Geotech. and Geol. Eng.,* 16, 239-250.

Sukumaran, B., Kyatham, V., Shah, A. and Sheth, D. (2002). "Suitability of using California bearing ratio test to predict resilient modulus." Federal Aviation Administration airport technology transfer conference, Airport Technology Research and Development Branch, Federal Aviation Administration, Washington, DC.

Syed, I.M., Fuselier, G.K. and Hewitt, M. (2007). "Innovation in cement stabilization of airfield subgrades." *FAA Worldwide Airport Technology Conf.,* New Jersey, USA.

Tang, C., Shi, B., Gao, W., Chen, F. and Cai, Y. (2007). “Strength and mechanical behaviour of short polypropylene fiber reinforced and cement stabilized clayey soil.” *Geotextiles and Geomembranes*, 25, 194-202.

Terranova. “Terrabind – Material safety data sheet.” www.terranovatechnologies.com (Jun 22, 2014)

Tingle, J.S. and Santoni, R.L. (2003). “Stabilization of clay soils with nontraditional additives.” *Transp. Res. Rec.: J. Transp. Res. Board*, 1819, 72-84.

Tingle, J.S. and Santoni, R.L. (2003). “Stabilization of clay soils with nontraditional additives.” *Transp. Res. Rec.: J. Transp. Res. Board*, 1819(1), 72-84.

Tolleson, A.R, Shatnawi, F.M., Harman, N.E., and Mahdavian, E. (2003). “An evlauation of strength change on subgrade soils stabilized with an enzyme catalyst solution using CBR and SSG comparison.” Report R-03-UTC-ALTERPAVE-GEO-01, University Transportation Center, Orangeburg, S.C., US.

Usluogullari, O.F. and Vipulanandan, C. (2008). “FEM analysis of CBR test with cemented sand.” *Proceedings of the Center for Innovative Grouting Materials and Technology (CIGMAT) Conference*, Houston.

Velasquez, R., Marasteanu, M.O., Hozalski, R. and Clyne, T. (2005). “Preliminary laboratory investigation of enzyme solutions as a soil stabilizer.” Technical Report, Minnesota Department of Transportation Research Services Section.

Vidal, H. (1996). “The principle of reinforced earth.” *High. Res. Rec.*, Washington, DC, 282, 1-16.

White, G.W. and Gnanendran, C.T. (2005). “The influence of compaction method and density on the strength and modulus of cementitiously stabilised pavement materials.” *Int. J. of Pave. Eng.*, 6(2), 97-110.

Wright-Fox, R., Macfarlane, J.G., and Bibbens, R.F. (1993). “Alternate chemical soil stabilizers.” Minor Research Report, USA.

Yang, H.H. (2004). “Pavement analysis and design.” (2nd Edition). Pearson Prentice Hall, 677-681.

Yoder, E. J., and Witczak, M. W. (1975). “Principles of pavement design.” Wiley International Publication, John Wiley & Sons, Inc., New York, London.

Zhang, Z. and Tao, M. (2008). “Durability of cement stabilized low plasticity soils.”

J. of Geotech. and Geoenv. Eng., 134(2), 203-213.

Ziari, H. and Khabiri, M.M. (2007). "Interface condition influence on prediction of flexible pavement life." *J. of Civil Eng. and Manage.*, 13(1), 71-76.

Zydex. "Soil stabilization, water proofing and dust mitigation – Terasil." http://www.zydexindustries.com/roads.aspx (Sep. 10, 2015).

LIST OF PUBLICATIONS

INTERNATIONAL/ NATIONAL JOURNALS

1. **Lekha, B.M.,** Shankar, A.U.R. and Goutham, S. (2012). "Fatigue and engineering properties of chemically stabilized soil for pavements." *Indian Geotechnical Journal*. 43(1), 96-104.
2. **Lekha B.M.** and Shankar, A.U.R. (2014). "Laboratory performance of RBI 81 stabilized soil for pavements." *International Journal of Civil Engineering Research*. 5(2), 105-110.
3. **Lekha, B.M.,** Goutham, S. and Shankar, A.U.R. (2015). "Evaluation of lateritic soil stabilized with arecanut coir for low volume pavements." *Transportation Geotechnics*. 2, 20-29.
4. **Lekha, B. M.,** Goutham, S. and Shankar, A.U.R. (2015). "Effect of electrolyte lignin and fly ash in stabilizing black cotton soil." *Transportation Infrastructructure Geotech*nology. 2, 87-101.
5. **Lekha B.M.,** Chaitali, N., Goutham, S. and Shankar, A.U.R. (2014). "Laboratory investigation on black cotton soil stabilized with non traditional stabilizer." *IOSR Journal of Mechanical and Civil Engineering.* (IOSR-JMCE), 2, 07-13.
6. **Lekha, B.M.,** Goutham, S. and Shankar, A.U.R. (2015). "Strength and durability properties of RBI 81 stabilized black cotton soil as a pavement material." *Indian road congress* (Under Review).
7. **Lekha, B.M.,** Goutham, S. and Shankar, A.U.R. (2015). "Experimental investigation on lateritic soil stabilized with cement and aggregates." *Road materials and pavement design* (Under Review).

INTERNATIONAL / NATIONAL CONFERENCES

1. Shankar, A.U.R., Mithanthaya, I.R. and **Lekha, B.M.** (2012). "Laboratory studies on stabilized lateritic soil as a highway material." *Workshop on Non-Conventional Materials/ Technologies*. Central Road Research Institute, New Delhi.
2. **Lekha, B.M.,** Goutham, S. and Shankar, A.U.R. (2013). "Laboratory investigation of soil stabilized using nanotechnology chemicals." *Indian Geotechnical Conference 2013,* Roorkee, Uttarakhand, India.

3. Shankar A.U.R., **Lekha, B.M**. and Goutham, S. (2014). "Properties and performance of blended lateritic soil for gravel roads." *Proceedings of Indian Geotechnical Conference,* Kakinada, India

4. **Lekha, B.M.,** Goutham, S. and Shankar, A.U.R. (2014). "Laboratory performance of black cotton soil stabilized with terrasil as a pavement material." *Colloquium on Transportation Systems Engineering and Management*, CTR, CED, NIT Calicut, India.

5. **Lekha, B.M.,** Goutham, S, Chaitali, N. and Shankar, A.U.R. (2014). "Laboratory investigation on black cotton soil stabilized with non-traditional stabilizer." *International Conference on Innovations in Civil Engineering,* SCMS School of Engineering and Technology, Eranakulam, Kerala.

6. Priyanka, B.A, **Lekha, B.M**, Goutham, S. and Shankar, A.U.R. (2015). "KENPAVE Analysis for Low Volume Roads with Reduced Resilient Modulus Values." *2nd Conference on Transportation Systems Engineering and Management*. NIT Tiruchirappalli, Tamil Nadu.

7. **Lekha, B.M.,** Goutham, S. and Shankar, A.U.R. (2015). "Effect of RBI 81 on laterite soil as a pavement material." 53rd I*ndian Geotechnical Conference* – Conducted by Indian Geotechnical Society, Pune. (Under review).

8. **Lekha, B.M.,** Goutham, S. and Shankar, A.U.R. (2016). "Laboratory performance of lateritic soil and soil-aggregate mixture with RBI grade 81." *8th International Conference on Maintenance and Rehabilitation of Pavements (MAIREPAV8),* Singapore. (Under review).

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Degree	Major	Institute	Duration	Percentage/ CGPA
Ph.D	Performance studies on pavements using chemically stabilized soils	National Institute of Technology Karnataka,Surathkal	2012 – 2015	9.27/10
M Tech	Transpotation Systems Engineering	National Institute of Technology Karnataka,Surathkal	2009 – 2011	8.58/10
B E	Civil Engineering	K.V.G College of Engineering, Sullia , D.K., Karnataka	2004 – 2008	75.49 %
Higher Secondary	Science	Government Science College, Hassan	2002 – 2004	54.01 %
Class X		Sri C K S Girls High School, Hassan.	2001 – 2002	71.04 %

PROJECTS:

B.E: “Stress Analysis For Flexible Pavement Using KENPAVE Software.”

M.Tech: “Fatigue Behavior of Chemically Treated Bituminous Concrete Mixes.”

Undergone subjects in M.Tech

1. Pavement Design
2. Traffic engineering and Management
3. Urban Transport Planning
4. Statistical Methods
5. Soil Mechanics
6. Pavement Materials and construction
7. Traffic flow Theory
8. Operational Research
9. Traffic Design and Studio lab
10. Transportation Engineering lab

SOFTWARE SKILLS:

Programming	: Basics in C
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PULICATIONS:

Journals:

1) Ravi Shankar A U, Lekha B M and Goutham Sarang. (2013). Fatigue and Engineering Properties of Chemically Stabilized Soil for Pavements, Indian Geotechnical Journal, 43(1), 96–104.
2) Goutham Sarang, Lekha B M and Ravi Shankar A U. (2014). Aggregate and Bitumen Modified with Chemicals for Stone Matrix Asphalt Mixtures, IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE), 2, 14–20
3) Lekha B M, Goutham Sarang, Chaitali N and Ravi Shankar A U. (2014). Laboratory Investigation on Black Cotton Soil Stabilized with Non Traditional Stabilizer, IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE), 2, 07–13.

4) Lekha B M, Goutham Sarang and Ravi Shankar A U. (2015). Evaluation of lateritic soil stabilized with Arecanut coir for low volume pavements, Transportation Geotechnics, 2, 20–29.
5) Goutham Sarang, Lekha B M, Geethu J S and Ravi Shankar A U. (2015). Laboratory performance of stone matrix asphalt mixtures with two aggregate gradations, Journal of Modern Transportation, 23(2), 130–136.
6) Lekha B M, Goutham Sarang and Ravi Shankar A U. (2015). Effect of Electrolyte Lignin and Fly Ash in Stabilizing Black Cotton Soil, Transportation Infrastructure Geotechnology, 2(2), 87–101.
7) Ravi Shankar A U, Lekha B M, Goutham Sarang and Abhishek P. (2014). Performance and Fatigue behavior of semi dense bituminous concrete using waste plastics as modifier, Indian Highways, 5(2), 233–240.
8) Lekha B M, Goutham Sarang and Shankar A.U.R. Experimental investigation on lateritic soil stabilized with cement and aggregates. Road materials and pavement design (Communicated).
9) Goutham Sarang, Lekha B M, Krishna G and Ravi Shankar A U. Comparison of Stone Matrix Asphalt Mixtures with Polymer Modified Bitumen and Plastic Coated Aggregates, Road Materials and Pavement Design (Communicated).
10) Goutham Sarang, Lekha B M, Ravi Shankar A U and Someswara Rao B. Stone matrix asphalt mixtures with cellulose fiber and waste plastics, Indian Roads Congress Journals (Communicated).
11) Lekha B M, Goutham Sarang and Ravi Shankar A U. Strength and Durability Properties of RBI 81 Stabilized Black Cotton Soil as a Pavement Material, Indian Roads Congress Journals (Communicated).

Conference:

1) Goutham Sarang, Lekha B M and Ravi Shankar A U. (2014). Stone Matrix Asphalt using aggregates modified with waste plastics, GeoShanghai – 2014, International Conference on Geotechnical Engineering, Conducted by ASCE, Shanghai, China. Published in Pavement Materials, Structures, and Performance, Geotechnical Special Publication 239, 9–18.

2) Goutham Sarang, Lekha B M, Monisha M and Ravi Shankar A U. (2014). SMA mixtures with modified asphalt and treated aggregates, 2nd Transportation and Development Institute Congress. Conducted by ASCE, Orlando, Florida, USA. Published in 2nd T&DI Congress 2014 Proceedings, 290–299.
3) Lekha B M, Goutham Sarang and Ravi Shankar A U. (2013) Stabilization of Lithomargic Clay Using RBI 81 For Pavement Construction, Recent Advances in Civil Engineering (RACE – 2013).Conducted by Saintgits College of Engineering, Kottayam, Kerala.
4) Lekha B M, Goutham Sarang and Ravi Shankar A U. (2013). Laboratory investigation of soil stabilized with nano chemical, Indian Geotechnical Conference – 2013, Conducted by Indian Geotechnical Society, Roorkee.
5) Lekha B M, Goutham Sarang, Chaitali N and Ravi Shankar A U. (2014). Laboratory Investigation on Black Cotton Soil Stabilized with Non Traditional Stabilizer, International Conference on Innovations in Civil Engineering 2014, Conducted by SCMS School of Engineering and Technology, Eranakulam, Kerala.
6) Goutham Sarang, Lekha B M and Ravi Shankar A U. (2014). Comparison of Bituminous Mixtures Prepared in Marshall Compaction and Gyratory Compactor, Colloquium on Transportation Systems Engineering and Management. Conducted by NIT Calicut, Kerala.
7) Lekha B M, Goutham Sarang and Ravi Shankar A U. (2014). Laboratory Performance of Black Cotton Soil Stabilized with Terrasil as a Pavement Material, Colloquium on Transportation Systems Engineering and Management, Conducted by NIT Calicut, Kerala.
8) Ravi Shankar A U, Lekha B M and Goutham Sarang. (2014). Properties and Performance of Blended Lateritic Soil for Gravel Roads, Indian Geotechnical Conference – 2014, Conducted by Indian Geotechnical Society, Kakinada.
9) Goutham Sarang, Lekha B M, Pavan Patil and Ravi Shankar A U. (2015). Experimental Evaluation of Bituminous Concrete Using Chemically Treated Aggregates, 2nd Conference on Transportation Systems Engineering and Management, Conducted by NIT Tiruchirappalli, Tamil Nadu.

10) Priyanka B A, Lekha B M, Goutham Sarang and Ravi Shankar A U (2015). KENPAVE Analysis for Low Volume Roads with Reduced Resilient Modulus Values, 2nd Conference on Transportation Systems Engineering and Management, Conducted by NIT Tiruchirappalli, Tamil Nadu.

11) Goutham Sarang, Lekha B M, Ramesh Tejavath and Ravi Shankar A U. Laboratory Performance Comparison of Stone Matrix Asphalt Mixtures with Polymer Modified Bitumen and Cellulose Fiber Stabilizer, International Conference on Transportation and Development 2016, Conducted by ASCE, Texas, USA (Abstract Accepted).

12) Lekha B M, Goutham Sarang and Ravi Shankar A U. Laboratory performance of lateritic soil and soil-aggregate mixture with RBI grade 81. 8th International Conference on Maintenance and Rehabilitation of Pavements (MAIREPAV8) 2016, Singapore (Abstract Accepted).

13) Lekha B M, Goutham Sarang and Ravi Shankar A U. Effect OF RBI 81 on laterite soil as a pavement material, 53rd Indian Geotechnical Conference, 2015, Conducted by Indian Geotechnical Society, Pune (Abstract Accepted).