LABORATORY INVESTIGATION ON LATERITIC AND BLACK COTTON SOILS STABILISED WITH GGBS AND ALKALI SOLUTIONS

Thesis

submitted in partial fulfilment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

by

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DECLARATION

I hereby declare that the Research Thesis entitled "**LABORATORY INVESTIGATION ON LATERITIC AND BLACK COTTON SOILS STABILISED WITH GGBS AND ALKALI SOLUTIONS"** which is being submitted to **National Institute of Technology Karnataka, Surathkal,** for the partial fulfilment of the requirement for the award of degree of **Doctor of Philosophy** in the **Department of Civil Engineering**, is a *bonafide report of the 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.

…………………………………………..

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CERTIFICATE

This is to certify that the Research Thesis entitled "**LABORATORY INVESTIGATION ON LATERITIC AND BLACK COTTON SOILS STABILISED WITH GGBS AND ALKALI SOLUTIONS"** submitted by Mrs. **AMULYA S. (Register Number: 165108CV16F02),** as the record of the research work carried out by her, is accepted as the Research Thesis submission in partial fulfilment of the requirements for the award of degree of Doctor of Philosophy.

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....

Chairman – DRPC

DEDICATED TO MY PARENTS,

FAMILY MEMBERS, FRIENDS

AND TEACHERS

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ABSTRACT

The natural aggregates are depleting in developing countries due to the excessive usage in road and building construction. The present work investigates the improved properties of lateritic and Black cotton (BC) soils stabilized with Ground Granulated Blast Furnace Slag (GGBS) and alkali solutions such as sodium hydroxide and sodium silicate. The lateritic and BC soils are stabilized with 15, 20, 25 and 30% of GGBS and the alkali solutions consisting of 4, 5 and 6% of Sodium Oxide (Na_2O) having Silica Modulus (Ms) of 0.5, 1.0 and 1.5 at a constant water binder ratio of 0.25. The Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) are obtained for both untreated and stabilized soils from standard and modified Proctor tests. The stabilized samples were air-cured for 0 (immediately after casting), 3, 7 and 28 days at ambient temperature. In case of stabilized lateritic soil, the maximum strength is achieved at 30% of GGBS and alkali solution consisting of 6% Na_2O and 1.0 Ms whereas, in case of stabilized BC soil, the maximum strength is achieved at 30% GGBS and alkali solution consisting of 6% Na_2O and 0.5 Ms at both standard and modified Proctor densities. The stabilizedlateritic soil with 25 and 30% of GGBS and alkali solution consisting of 5 and 6% of Na_2O having 0.5 and 1.0 Ms is found to be durable after 28 days curing at both densities. Whereas, the stabilized BC sample having 25 and 30% of GGBS and alkali solution consisting of 5 and 6% of Na_2O with Ms of 0.5 only at modified Proctor density have passed durability. The stabilized lateritic soil with 30% of GGBS and alkali solution consisting of 6% of Na_2O having Ms of 1.0 at both densities and the stabilized BC soil with 25% of GGBS and alkali solution consisting of 5% of Na_2O having Ms of 0.5 only at modified Proctor density achieved the highest flexural strength, fatigue life and the densified structure. The formation of calciumsilicate hydrate and calcium aluminosilicate hydrate structures resulted in a remarkable improvement of compressive strength, flexural and fatigue life of the stabilized soils due to the dissolved calcium ions from GGBS, silicate and aluminium ions from alkali solutions. The design of high and low volume roads is proposed by replacing the conventional granular layer with the durable stabilized soil and stress-strain analysis is carried out using pavement analysis software. The comparison of the cost of the conventional material with the proposed stabilized soils are carried out.

Keywords: Lateritic soil, Black cotton soil, Stabilization, Ground Granulated Blast Furnace Slag, Sodium hydroxide, Sodium silicate, Durability, Flexural strength, Fatigue, Microstructure analysis, Pavement analysis, Cost comparison.

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NOMENCLATURE

Symbol	Abbreviation
AIL	Aggregate interface layer
ASTM	American society for testing and materials
b	Average width of specimen
BC	Black cotton soil
CBR	California bearing ratio
CH	High plastic clay
CI	Intermediate plastic clay
CSH	Calcium silicate hydrates
CTB	Cement-treated base
CTSB	Cement-treated sub-base
d	Average depth of specimen
E	Young’s modulus
E_{CTB}	Resilient modulus of cement-treated base and sub-base
ESAL	Equivalent single axle wheel load
ε_t	Horizontal tensile strain at the bottom of top layer
ε_z	Vertical compressive strain at the top of subgrade
FT	Freezing-thawing
GGBS	Ground granulated blast furnace slag
GSB	Granular sub-base
IRC	Indian roads congress
IS	Indian standards
ITS	Indirect tensile strength
K_2O	Potassium oxide
KOH	Potassium hydroxide
L	Span length
M	Molar
MDD	Maximum dry density
M_R	Modulus of rupture
M_{RGSB}	Resilient modulus of GSB
M_{RS}	Resilient modulus of subgrade
Ms	Silica modulus
msa	Million standard axles

Na_2O	Sodium oxide
Na_2SiO_3	Sodium silicate
NaOH	Sodium hydroxide
NASH	Sodium alumino silicate hydrates
NH	National highway
OGPC	Open graded premix carpet
OMC	Optimum moisture content
P	Maximum applied load
R^2	Regression value
SH	State highway
SiO_2	Silica
T	Thickness of road in meters
UCS	Unconfined compressive strength
WBM	Water bound macadam
WD	Wetting- drying
WMM	Wet mix macadam
σ_{r1}	Horizontal stress at the bottom of layer 1
σ_{r2}	Horizontal stress at the bottom of layer 2
σ_{r3}	Horizontal stress at the top of layer 3
σ_{Z1}	Vertical stress at interface 1
σ_{Z2}	Vertical stress at interface 2

CONTENTS

CHAPTER 1

INTRODUCTION

1.1 General

Transportation plays a significant role in the socio-economic and cultural development of any country. In India, road transportation provides the most flexible service to 87% of passengers and 60% of goods traffic movement from origin to destination. Based on the location and function, the roads are mainly divided into Expressways, National Highways (NH), State Highways (SH) and other roads. India is having 54.83 lakh km of the road network and out of which 1.2 lakh km of NH, 1.5 lakh km of SH and 52.07 lakh km of other roads including Major district roads, Minor district roads and village roads (MoRTH Annual report 2017-18). Based on the traffic intensity in terms of cumulative standard axles, the roads are also classified as heavy and low volume roads. As per Indian Roads Congress (IRC), the roads which carry traffic load less than 1 million standard axles (msa) are named as low volume roads and more than 1 msa traffic load-carrying roads are called as heavy volume roads. The surface which enables the vehicles to move over it is called as pavement. The pavement should be firm and non-yielding with a good riding quality and less slippery nature. Also, pavement should be structurally strong such that it should sustain heavy wheel loads and their repeated application. Based on the structural behaviour, the pavements are classified as flexible and rigid pavements.

1.2 Flexible pavements

The flexible pavements have low or negligible flexural strength and flexible pavements are flexible in their structural action under the application of wheel load. The vertical compressive stress developed by wheel load is maximum on the pavement surface and it will be distributed to the lower layers in a truncated cone shape and thereby the layer system was developed. The flexible pavementconsists of a topmost surface course, followed by a binder course, base course, sub-base course and subgrade as a bottom-most layer. The top surface course is a thin bituminous layer that directly receives the wheel load imposed on it and it should prevent the penetration of surface water into the pavement layers. The base course layer is the most important layer as it enhances the load-carrying capacity and disperses the load to the larger area. The granular sub-base course serves as an effective drainage layer also should sustain the lower magnitude of compressive stresses than the base course. The lowermost layer is the soil subgrade which supports all the above-laid layers and traffic loads. The subgrade layer is usually made of natural soil or borrowed soil. The typical cross-section of the flexible pavement is depicted in Figure 1.1.

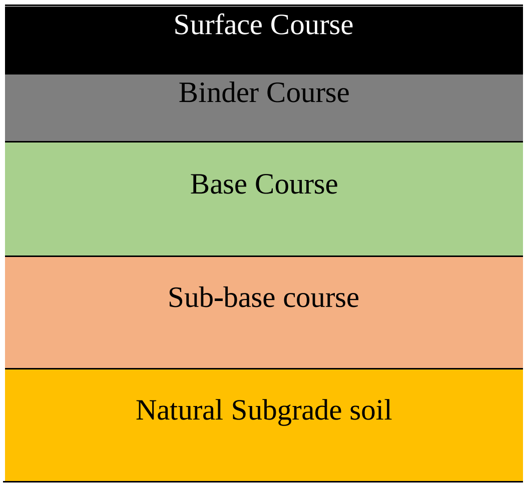


Figure 1.1 The typical cross-section of a conventional flexible pavement

1.2.1 Advantages and limitations of the flexible pavements

The flexible pavements are usually designed for 20 years and the initial cost of construction will be less. The magnitude and the repetitions of the wheel load will be taken into an account for the design of flexible pavement and the structural evaluation can be done by providing the overlay to strengthen the pavement. The surface can be opened for traffic within 24 hours of construction as the curing period required is less. But there are some of the limitations such as the surface layer will get deteriorated when the surface is exposed to stagnant of water and also it requires periodic maintenance. The flexible pavements will be designed for lesser design life compared to rigid pavements and night visibility is very poor. The flexible pavements demand a higher quantity of hard aggregates for a total thickness of the pavement.

The rate of growth of infrastructure industries in India is gradually increasing which boosts up the Indian economy. Due to urbanization, the infrastructure industries demand the huge requirement of aggregates which tends to the gradual depletion of natural resources. In order to reduce the use of depleting gravels, the naturally and abundantly available natural soil needs to be used.

Soil is one of the major resources and locally available material helps in the construction of low-cost roads. The Indian Council of Agricultural Research has classified the soils based on depth, pH, texture and process of formation into 8 Groups such as Alluvial Soils, Black Soils, Red and Yellow soils, Laterite soils, Mountains soils, Desert or Arid Soils, Saline Soils and Peaty or Organic soils. Many soil classification systems are practiced in highway engineering based on the grain size such as textural soil classification, Burmister descriptive classification, Casagrande soil classification, Unified soil classification, Highway research board classification, Federal aviation agency classification and Compaction classification. The soil to be used as a highway material should be stable, incompressible, drainable and easy to compact. As both lateritic and Black Cotton (BC) soils are abundantly available in Konkan belt of India where the average annual rainfall in more than 3500 mm. Hence these two soils are considered in the present work.

1.2.2 Lateritic Soil

The Lateritic soil is end products tropical weathering or laterization processes where the tropical weathering is an intensive and prolonged chemical weathering of underlying parent sedimentary, igneous and metamorphic rocks.The lateritic soil possesses high iron and aluminium oxides with poor lime, magnesia, potash and nitrogen covering 2.48 lakh sq. km of land which is around 8.2% of the total land area of India. The lateritic soil is mainly located in coastal regions, western and eastern Ghats with 1000 to 1500 m above mean sea level and Karnataka, Kerala, Tamil Nadu, Madhya Pradesh and the hilly areas of Odisha and Assam. It is having much economic value as a building material and durable due to the weathering process.

1.2.3 Black Cotton (BC) Soil

Black cotton soil is another major group of soil having medium to high compressible inorganic clay formed due to the solidification of volcanic lava. The name black cotton is due to its black colour and cotton is the main crop grown in that soil. It occurs in parts of Maharashtra, Gujarat,Madhya Pradesh, Andhra Pradesh and some parts of Karnataka and Tamil Nadu. It covers around 5.46 lakh sq. km of the land area of India. BC soil is characterized with high clay content, water holding capacity, expansiveness, plasticity index, swelling, shrinkage, fertility and due to which, it imposes low bearing capacity. The BC soil is found rich in calcium carbonate, potash,magnesium carbonate and lime with small traces of phosphoric content. Due to its peculiar characteristics, it is unsuitable for the construction purpose. Hence, the soil stabilization technique needs to be adopted to improve its engineering properties to use as a construction material.

1.3 Soil Stabilization

Soil stabilization is a technique to improve the engineering properties of the natural soil to use as a construction material. There are mechanical and chemical stabilization methods. The mechanical stabilization of soil is done by proportioning the materials and compacting the layers using rollers, compactors and tampers or by incorporating nailing and barriers. The stability of the soil is based on the degree of compaction which improves the bearing capacity, lowers the permeability and compressibility(Makusa 2012). The chemical stabilization is a technique of stabilizing the soil using additives such as cement, lime and bitumen. The degree of stabilization depends on the proportion of additives, nature of the soil, moisture content, curing period and dry density achieved from the compaction. The chemicals such as calcium chloride, sodium chloride and sodium silicate in the solution form which reacts with source materials and alter its chemical properties.

1.4 Marginal Materials

The materials used for the construction of the conventional base and sub-base course costs around 30 to 40% of the total cost of road construction. In order to minimize the cost of construction, the waste or marginal materials which are also called substandard materials can be used. Marginal materials wholly do not meet the specifications of normal road materials but can be used successfully under special conditions after subjected to a particular treatment. There are different types of natural materials such as hard rock, weak rock, natural gravels, duricrusts and manufactured materials which are man-made such as fly ash, slag and Construction and Demolition waste (Cook et al. 2002). Slag is an industrial by-product left after the required metal is extracted from its ore. There are many slags available from the metal industries such as dross, iron ore tailings, fly ash, ground granulated blast furnace slag and pozzolans.

1.4.1 Ground Granulated Blast Furnace Slag (GGBS)

The GGBS is a by-product material of iron and steel manufacturing industries which is obtained by quenching molten iron slag in water or steam in a blast furnace. After quenching, a granular product used to be dried and ground into a fine powder. The GGBS is mainly composed of Calcium oxide, Silica dioxide, aluminium oxide and magnesium oxide. The presence of the high amount of calcium oxide helps in developing the compressive strength of the mixture hence, the GGBS can be used as a supplementary cementitious material replacing the ordinary Portland cement for greener and sustainable construction (Akinwumi 2014; Amadi 2010; Kulkarni and Sharma 2016; Yadu and Tripathi 2013).

1.5 Concept of alkali activation

Alkali activation is an exothermic reaction between the aluminosilicate source materials and alkali solutions. The aluminosilicate materials are considered as latent hydraulics and are characterized as finely divided and non-crystalline or poorly crystalline similar to pozzolans but containing sufficient calcium to form cementitious compounds after interacting with water. GGBS is one of the examples of latent hydraulics and alkali solutions can be the hydroxides of sodium and potassium or the silicates of sodium and potassium or the combination of both hydroxides and silicates. The process of activation of slag initiates with the attack by the alkalis on the slag particles, thus breaking the outer layer and then a polycondensation of reaction products. (Wang et al. 1994) explained that the initial reaction products are formed due to the dissolution and precipitation and later, a solid-state mechanism is followed where the reaction takes place on the surface of the formed particles, dominated by slow diffusion of the ionic species into the unreacted core. The nature of the anion in the solution also plays a determining role in activation, particularly in early ages and especially about paste settings(Fernández-Jiménez et al. 2006; Palomo et al. 2014). The final product formed by alkali-activated slag is Calcium Silicate Hydrates (CSH) (Palomo et al. 2014).

1.6 Need of present investigation

Due to the modernization of India, the rate of growth of transport vehicles also increasing which demands the construction of durable and economical pavements. The construction of a conventional base and sub-base course requires a huge amount of gravels due to which the natural resources are depleting and the extensive use of cement affects the environment significantly with high emission of carbon dioxide. In order to reduce the use of depleting gravels, the naturally and abundantly available soil can be used also, the cement can be replaced with the waste and marginal material such as GGBS. The concept of alkali activation in the soil stabilization is adopted and its behaviour will be observed through laboratory investigations.

1.7 Objectives and scope of the present study

The present study aimed to stabilize the lateritic and black cotton soil with different dosages of GGBS stabilized with the combination of alkali solutions. The laboratory tests will be conducted on the stabilized soil and the behaviour will be observed. The main objectives of the present research work are as follows.

1. To study the improved geotechnical and engineering properties of the Lateritic and Black Cotton soils stabilized with the GGBS and alkali solutions.
2. To evaluate the durability of the stabilized Lateritic and Black Cotton soils under alternate wetting-drying and freezing-thawing cycles.
3. To evaluate the flexural strength and fatigue behaviour of the stabilized soils which are found durable.
4. To analyze the microstructure images of durable soils obtained from the Scanning Electron Microscope.
5. To design the low and high volume roads using the durable stabilized soils and the stress-strain analysis of the pavement is made using pavement analysis software IITPAVE suggested by IRC.
6. To carry out the cost analysis of the stabilized material.

The scope of the present study is to stabilize the Lateritic and Black Cotton soil with different dosages of GGBS treated with the combination of two alkali materials such as Sodium Hydroxide and Sodium Silicate. The standard and modified Proctor were conducted on both natural and stabilized soils to find the

Optimum Moisture Content (OMC) and Maximum Dry Density (MDD). The Unconfined Compressive Strength (UCS), California Bearing Ratio (CBR) and durability tests were conducted in the laboratory to evaluate the engineering properties of the stabilized soils. Also, the flexural strength and fatigue behaviour of the durable samples were tested.

1.8 Organization of the thesis

The present work has been divided into seven chapters and compiled for a better understanding of the research work.

Chapter 1 discussesthe importance of the discipline and principles of soil stabilization. Also, it includes the need, objectives, and scope of the present investigation.

Chapter 2 discusses the extensive literature work regarding the development of stabilization and stabilizing materials used in recent days. The previous works which provide the scope for the present investigation are discussed.

Chapter 3discusses the materials being used and the methodology adopted to find the engineering properties of stabilized soils in the present work. The procedure of laboratory tests such as UCS, durability, flexural strength and fatigue test as per relevant codes are discussed. The microstructure image obtained from the scanning electron microscope and its features are discussed.

Chapter 4 discusses the test results of lateritic soil stabilized with different contents of GGBS and alkali solution consisting of various dosages of Na_2O and silica modulus.

Chapter 5 presents the test results of BC soil stabilized with various dosages of GGBS and alkali solution consisting of various Na_2O content and silica modulus.

Chapter 6proposes the design of low and high volume pavements as per IRC: SP: 72-2015 and IRC: 37-2018 respectively replacing granular materials with stabilized lateritic and BC soils. The analysis of critical strains using IITPAVE software is performed on both conventional and cement-treated base and sub-bases. Cost comparison has been made as per the Schedule of Rates specified by the Mangalore Public Works Department, Karnataka.

Chapter 7 Summarizes the investigation and conclusions are drawn based on the experimental and analytical study.

CHAPTER 2

LITERATURE REVIEW

2.1 General

In this chapter, the review of soil stabilization by various methods and additives are discussed in detail. The mechanism involved in the concept of using marginal materials with alkali solution is explained and the research carried out on this concept is also reviewed in this chapter.

2.2 Stabilization of natural soil

The concept of soil stabilization has begun from the 1960's and '70s. Soil stabilization is the process of modification of soils with or without additives to increase the load-carrying capacity and service life of the constructed structure. Soil stabilization not only aims at increasing the compressive strength but other physical characteristics of the soil as well. The stabilized soil should be able to withstand against the environmental conditions, daily and seasonal temperature variations, moisture variation, microbial and chemical effects due to the natural or man-made causes. Mechanical stabilization is a proportionate mixture of coarse aggregates, fine aggregates, silt and clay to attain the strength and durability (Winterkorn and Pamukcu 1991). The strength of the soil-aggregate mixture is obtained due to the dry density achieved from the compaction and the grading curve suggested by Fuller and Thompson in 1907.

Chemical stabilization is the process of treating soil by adding one or more chemicals and rather than physicochemical, physical interaction takes place between soil particles and chemicals. The reactions such as flocculation, hydration, pozzolanic, ion-exchange, precipitation, oxidation, hydration, and carbonation depending on the type of soil, the chemical composition of additives and soil, degree of mixing, curing period and the density achieved from compaction (M Gidigasu 1976). The additives used for chemical stabilization are cement, lime, fly ash, bituminous materials may be cutback asphalt, tar and emulsions and industrial waste or marginal materials. Stabilization of soils mainly lateritic and BC or clayey soils was done using a chemical stabilization technique.

2.2.1 Lateritic soil stabilization

The term laterite was first coined by Buchanan in 1807. The work on stabilization of lateritic soil using cement showed the significant improvement in terms of CBR, UCS, Indirect Tensile Strength (ITS) and triaxial shear strength (James and Pandian 2016; Joel and Agbede 2011), Based on the microstructure analysis and Raman spectroscopy results, the stabilized lateritic soil can be used as a base course (Mengue et al. 2017) also the fatigue behaviour of the 3% cement-treated soil was analyzed and found the stresses and strain levels are much lower than untreated soil thus the failure does not occur (Portelinha et al. 2012). The stabilization of lateritic soil using lime improves the CBR, UCS, swelling potential, hydraulic conductivity, shear strength after 28 days curing and quick lime treated lateritic soil is found to be durable and superior to hydrated lime (Amadi and Okeiyi 2017; James and Pandian 2016; Jawad et al. 2014; Ta'negonbadi and Noorzad 2017) and the lime treated lateritic soil doesn't qualify for bases and thus suggested for sub-base course (Ola 1977; Olinic and Olinic 2016). Similarly, the stabilization of lateritic soil was done using ashes like oil palm fronds (Nnochiri and Aderinlewo 2016), Bagasse ash (Osinubi et al. 2009), rice husk ash (Akinyele et al. 2015), sawdust ash (Edeh et al. 2014) and sugar cane straw ash (Amu et al. 2011) showed improved geotechnical properties but ash stabilized soil doesn't stand alone as a pavement material hence recommended to add cement. Usage of fine powders such as tire powder, micro silica and nanostructured clay in lateritic soil stabilization helps to increase the strength to use as an improved subgrade or sub-base layer (Gordan and Adnan 2015; Onyelowe et al. 2019).

Many types of research on stabilization of lateritic soil were done with different types of fibres such as extensible fibres (Ola 1989), Polymer sack fibres (Menon and Ravikumar 2019), Arecanut coir (Lekha et al. 2015) and coconut coir (Marathe et al. 2015; Upadhyay and Singh 2017). Though the CBR, UCS, flexural and shear strength of the stabilized soil increases, it was suggested to use for the construction of low volume roads. Works on stabilization of lateritic soil using bio enzyme such as terrazyme showed the increased geotechnical properties but failed as a durable material (Muguda and Nagaraj 2019; Panchal et al. 2017; Sahoo et al. 2018; Shankar et al. 2009). Later the stabilization of lateritic soil using industrial waste or marginal materials such as slags mainly GGBS, fly ash, dross and pozzolans was done. The soft soil treated with 15 to 20% of basic oxygen steel slag fines cured for 28 days increased the UCS due to the formation of Calcium Silicate Hydrates (CSH) bond (Tsai et al. 2014). The addition of 16% of aluminium dross showed the correlation between CBR and UCS with a regression (R^2) value of 0.81 (Busari et al. 2018). When 10% of the soil was replaced with crushed slag met the engineering properties of the mix meets the requirement of sub-base (Sudla et al. 2018). Similarly, 15% of Pozzolans replacement increased the strength and modulus of elasticity due to the cation exchange (A. Allahverdi et al. 2008; Apampa and Jimoh 2016; Bahadori et al. 2019; Onyelowe et al. 2019). The calcium oxide rich fly ash and GGBS improved UCS(Akinwumi 2014; Amadi 2010; Kulkarni and Sharma 2016; Yadu and Tripathi 2013).

The alkali chemicals were also used either alone or mixed with waste materials for soil stabilization. The soil treated with reactive magnesia and lime cured for 7 days increased the strength due to the formation of the CSH bond (Gu et al. 2015). Similarly, the hydroxides of sodium and potassium helped in developing stability and hydraulic conductivity of the tropical soils (Nyamangara et al. 2007). Lateritic soil treated with 5% of phosphoric acid increased the compressive strength to 4 MPa after 28 days curing (Medina and Guida 1995) and addition of sodium silicate (Na_2SiO_3) with 5% of lime into the lateritic soil improved the geotechnical properties due to the formation of Van der Waals intermolecular forces between silica tetrahedrons (Youssef et al. 2010). Lateritic soil treated with sodium hydroxide (NaOH) also increased the strength to 4.2 kg/cm^{2}and Atterberg limits (Mishra and Singh 2018; Olaniyan et al. 2011). The behaviour of the combination of slag with alkali solution in concrete, mortar and paste was observed and found the significant improvement in strength and durability properties. The concept of using calcium oxide rich slag materials replacing cement with alkali solution into the soil stabilization is discussed in the further section.

2.2.2 The concept of geopolymer system and alkali activated system

After 20 years of Gluchovskij’s work, Davidovits. (1979) coined a new term “geopolymer” defined as a material originated by inorganic poly-condensation, which is so called as geo polymerisation. Geopolymerization involves the chemical reaction of alumino-silicate oxides with alkali polysilicates yielding polymeric Si-O-Al bonds (Davidovits., 1991) and Davidovits. (1994), proposed that the activation of materials rich in silicon and aluminium such as fly ash, rice husk ash, which are either from geological origin or by product with highly alkaline solutions. Malhotra et. al. (1996), explained that the materials which are rich in silica and aluminium and having low calcium (class F) fly ash and silica fume are called as pozzolanic materials which are finely divided form possessing cementitious properties, and in the presence of moisture, chemically react with calcium hydroxide at ambient temperatures to form cementitious compounds. The geopolymerisation process is an exothermic polycondensation reaction involving alkali activation by a cation in solution. The reaction leading to the formation of polysialates which are differed with the Si:Al ratio. Additional amounts of amorphous silica must be present to form either the polysialate-siloxo or polysialatedisiloxo structure of geopolymer. The amorphous to semi-crystaline three-dimensional silico-aluminate structures are of Poly-sialate type (-Si-O-Al-O-), Poly sialate-siloxo (-Si-O-Al-O-Si-O-) and Poly sialate-disiloxo (-Si-O-Al-O-Si-O-Si-O-). For geopolymers based silico-aluminates, poly sialate was suggested which consists of SiO_4 and AlO_4. Poly sialates are having the empirical formula: $M_n\{-(SiO_2)z\text{-}AlO_2\}_n$, wH_2O where M is a cation such as potassium, sodium or calcium, n is a degree of polycondensation, z is 1, 2, 3 and the reaction equation are given in Equation

$$(Si_2O_5.Al_2O_2)_n + H_2O + OH^- \longrightarrow Si(OH)_4 + Al(OH)^{4-} \tag{2.1}$$

$$Si(OH)_4 + Al(OH)^{4-} \longrightarrow (-\,Si - \underset{\displaystyle O}{\overset{|}{\underset{|}{O}}} - Al - \underset{\displaystyle O}{\overset{|}{\underset{|}{O}}}\, -)_n + 4H_2O \tag{2.2}$$

(Dovidovits., 1999). From the classification of alkali activated cementitious materials, geoplymers have similarity with the zeolites and consists of alkaline aluminosilicate systems (R-A-S-H, where R= Na or K) (Krivenko., 1994 and Pacheco-Torgal et al., 2008). The major chemical product for geopolymer is amorphous hydrated alkali-aluminosilicate (Duxson., 2007). Jeffery et al. 2012 have explained that the geopolymer cements will be developed by a series of distinct reaction such as dissolution, polymerization and growth of the structure. The dissolution of aluminosilicate species within a highly basic, alkaline environment, polymerization of the dissolved minerals into hydration products like natural zeolites and final hardening of the matrix by excess water exclusion and the growth of crystalline structure. The Figure 2.1 illustrates the polymerization process in alkali activated geopolymers.

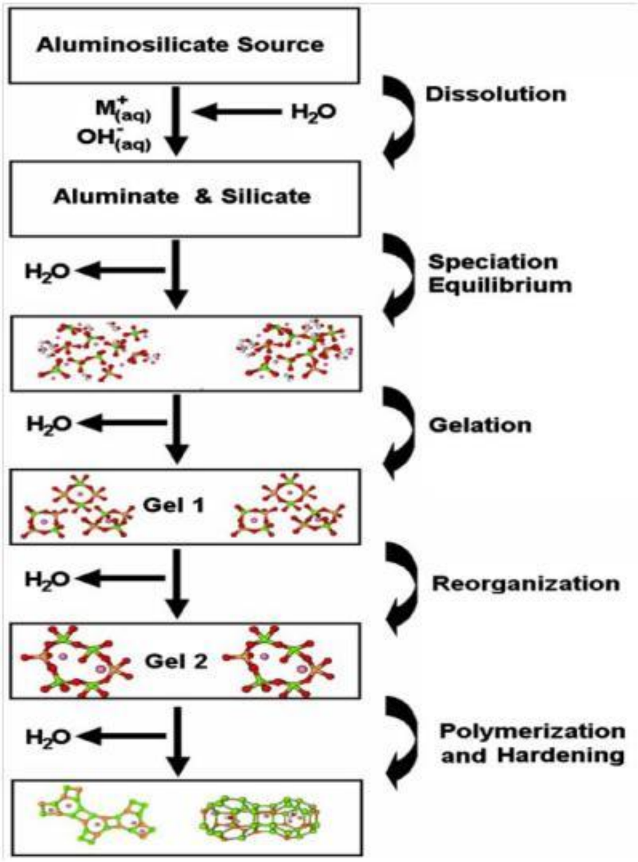


Figure 2.1 The polymerization process in alkali activated geopolymers (Vijaya, 2010)

Another concept called as Alkali Activated System, which also explains the reaction between alumina silicate source materials with alkaline solution. Though the concept and source materials are same, but differed from the final product and this difference is due to the composition of source materials. The alkali activation process initiates with the attack of the alkali solution on slag particles. The slag materials are called as latent hydraulic binders. These materials are also characterized as finely divide and non-crystalline or poorly crystalline similar to pozzolans, but containing sufficient calcium to form cementitious compounds after interacting with water. GGBS is one of the examples of latent hydraulics. The process of activation of slag initiates with the attack by the alkalis on the slag particles, thus breaking the outer layer and then a polycondensation of reaction products. Wang et al. (1994) suggested that though the initial reaction products form due to dissolution and precipitation, at later ages, a solid-state mechanism is followed where the reaction takes place on the surface of the formed particles, dominated by slow diffusion of the ionic species into the unreacted core. The nature of the anion in the solution also plays a determining role in activation, particularly in early ages and especially about paste setting (Fernández et al., 2001, Fernández et al., 2003). Final product formed by alkali activated slag is Calcium Silicate Hydrates (C-S-H) Gluchovsky (1959); the major difference being the rate and intensity of the reaction.

The laboratory tests on samples treated with slag and alkali solution such as fly ash with 14M NaOH having fly ash to NaOH ratio of 1.4 achieved 30 MPa compressive strength when cured at 80^0C (Bakkali et al. 2016). The comparative study between the fly ash treated with NaOH and potassium hydroxide (KOH) was done and found that the UCS of 7.7 MPa was observed in the case of NaOH than with KOH after 28 days curing (Singh et al. 2015). When fly ash and GGBS mixed with both NaOH and Na_2SiO_3, nearly 90% of the compressive strength was achieved in 7 days of ambient temperature curing and found that it was directly proportional to the molarity of the alkali solution and GGBS content(Kattimani et al. 2015). A study on the mixture of fly ash blended with NaOH and Na_2SiO_3 showed the improved CBR and UCS (Dungca and Codilla 2018) and the obtained strength was due to the formation of additional CSH bond coexisted with sodium aluminosilicate hydrates (NASH) which helped in developing shear bond strengths (Phoo-Ngernkham et al. 2015; Phummiphan et al. 2017).

The GGBS treated with different alkali solutions such as Na_2SiO_3, Sodium carbonate, and NaOH was tested for compressive, split tensile, flexural and shearstrength. From the test results, it was found that among all chemicals, NaOH was rated first, sodium carbonate was second and Na_2SiO_3 was third in terms of shear and flexural strength (Narender Reddy et al. 2013). The comparative study on the samples of GGBS treated with Na_2SiO_3 and KOH cured underwater and followed by heat curing found that the water cured samples followed by heat curing gave the better compressive strength than only heat cured samples (Qureshi and Ghosh 2014).

The compressive strength increases with increase in dosage of potassium oxide (K_2O) till 6% but further increase in dosage to 8% reduced the strength at silica modulus (Ms) which is defined as ratio of Silica (SiO_2) to Na_2O of 0.8 and the increased strength was due to the hydration products formed from CSH bond(Qureshi and Ghosh 2013). When NaOH and Na_2SiO_3 were added to the GGBS, the Sodium Oxide (Na_2O) content and the Ms played an important role in achieving the compressive strength. The Na_2O dosage of 2% was found most favourable as it achieved the good mechanical strength with slight efflorescence (Allahverdi et al. 2010) and the increased Ms to 2.4 showed the lower mechanical strength, low setting time and end products were formed of crystalline CSH gel (Bernal et al. 2011). Also, the mixture gave the compressive strength up to 167 MPa and dynamic elastic modulus of 28 GPa at the age of 56 days due to the polycondensation mechanism which was similar to that of the hydration process of portland cement (Yang et al. 2012b). The best combination of alkali solutions with GGBS was found to beNa_2SiO_3or potassium silicate and NaOH or KOH(Petermann et al. 2010).

When a kaolinite clay was treated with NaOH, the end product formed was highly influenced by the concentration of NaOH. 4 N NaOH forms high quantity compounds than 1 N NaOH due to the formation of sodium aluminium silicate hydrates (Sivapullaiah and Manju 2005).

The use of fly ash treated with NaOH and Na_2SiO_3 in the lateritic soil stabilization showed the 7 day UCS meeting the required strength for light and heavy traffic pavements (Phummiphan et al. 2016, 2018).

2.2.3 BC Soil Stabilization

BC soil is a clayey soil which poses serious problems to the performance of roads. The roads constructed on BC soil develop undulations due to which the upheave occurs in the upper layers. When water gets access into the pavements, it saturates the subgrade soil and thus the soil loses its bearing capacity. Hence, it is considered as a problematic soil and it requires proper treatment to use as a pavement material.

The stabilization of BC soil using cement reduced the plasticity index significantly and 28 days cured treated samples showed the improved UCS to 5.25 times and 50% reduced strains at failure (Amadi and Osu 2018; Oza and Gundaliya 2013). The cement stabilized BC soil decreased the swelling pressure which makes soil less expansive thus the soil becomes stable, durable and resistant to deformation (Abdelkrim and Mohamed 2013). The BC soil stabilized with 4 to 6% of lime showed improved CBR and UCS, reduced permeability and swelling pressure of the mixture(Sharma and Sivapullaiah 2016)(Kanddulna et al. 2016; Singh and Vasaikar 2015). The 4% lime stabilized BC soil showed increased UCS by 3 times the natural soil due to the hydration process of pozzolanic materials, durable in Wetting-Drying (WD) and Freezing-Thawing (FT) test with weight loss less than 14% after 12 cycles and sustaining fatigue life of 150,000 (Amulya et al. 2018). Reappraisal of lime stabilization on BC soil found that when the dosage of lime was more than optimum content, the UCS reduced due to the formation of silica gel observed from microstructure analysis (Yang et al. 2012b). The expansive soil stabilized with fibre reduced the swelling potential and pressure which depends on the content and length of the fibre(Latifi et al. 2018). The strength of the fibre stabilized soil increased with an increase in fibre content and the optimum dosage was found to be 0.5% of the dry mass and achieved maximum CBR (Subramani and Udayakumar 2016). Similar work was done by (Amulya et al. 2018) and found 1% as the optimum dosage of coir fibre which achieved the maximum UCS, 15% more CBR than the natural soil. Also, the expansive soil with recycled ashes and fibres improved the engineering properties(Punthutaecha et al. 2006).

The stabilization of BC soil using bio enzyme and Eko enzyme showed an improvement in UCS and CBR to 9 kPa and 390% respectively(Kushwaha et al. 2018; Sen and Singh 2015). Terrazyme treated BC soil cured for 14 days also showed an improvement in CBR and UCS of 84% and 200% more than the natural soil(Agarwal and Kaur 2014; Gunji and Kannali 2019). Stabilization of BC soil was done with wood-ash (Okagbue 2007), powdered glass (Edeh et al. 2014) and burnt ash (Rangaswamy 2016) showed improved geotechnical properties, bearing capacity, swelling characteristics and compressibility behaviour of the soil but the treated soil alone cannot be used for the construction purpose.

The BC soil stabilized with sodium chloride is not effective in terms of swelling characteristics (Gueddouda et al. 2011). An increased hydraulic conductivity with the increase in phosphate dosage and increased friction angle to 20% in the case of di-ammonium phosphate treated clayey soil(Eltarabily et al. 2015). The chloride compounds such as sodium chloride, magnesium chloride and calcium chloride treated BC soil improved the soil strength and shear strength (Tamadher et al. 2007). The BC soil stabilized with NaOH had a significant effect on Atterberg limits (Sahu Rajesh Jain 2016), which increased UCS to 42 MPa (Olaniyan et al. 2011). The BC soil treated with 3 mole per litre of Na_2SiO_3 increased the shear strength, CBR and compressive strength (Hossein Moayedi 2012; Maaitah 2012; Mane et al. 2017; Youssef et al. 2010). The clayey soil stabilized with steel slag up to 30% replacement showed improved swelling potential and UCS due to compaction and CBR due to the reduced free swell value (Shalabi et al. 2017; Zumrawi and Babikir 2017). The high plastic clay stabilized with GGBS improved index properties, compaction parameters, UCS and CBR at 20% of GGBS content than the low plastic clay stabilized with GGBS due to the formation of CSH bond(Pathak et al. 2014; Sivrikaya et al. 2014; T.R and Preethi 2014). The triaxial test on GGBS stabilized BC soil showed reduced cohesion and increased angle of friction thus making the soil more resistant (Pathak et al. 2014). Similarly, the BC soil was stabilized with the fly ash and found improved geotechnical properties (T.R and Preethi 2014; Yadu and Tripathi 2013).

The laboratory tests were conducted on the clayey soil replaced with 40% of fly ash activated with 12M KOH cured for 28 days gave strength up to 1900% more than that of the natural soil (Elkhebu et al. 2018). Similarly, 10M KOH showed the improved strength and curing time required to achieve strength depends on the source of binder and activator (Teing 2019). The clay soil treated with palm oil fuel ash mixed with NaOH and KOH increased compressive strength up to 1200 kPa after 28 days of curing and bearing capacity up to 192% of untreated soil due to the formation of Si-O-Si and Al-O-Si bond(Pourakbar et al. 2016). The Stabilization of sandy clay with fly ash treated with NaOH and Na_2SiO_3 showed the strong dependency of mechanical strength on the ratio of activator to the fly ash where 12.5 molal of activator, 90 days curing period showed the better strength but the increased water binder (w/b) ratio from 0.75 to 1.0% reduced the strength (Cristelo et al. 2013). The clay treated with high calcium fly ash and alkali solutions such as NaOH and Na_2SiO_3 gave the strength 1.2 times that of the clay treated with alkali-activated Portland cement(Phummiphan et al. 2016). Similarly, the BC soil is treated with volcanic ash activated with calcium hydroxide and KOH showed the increased Atterberg limits and density at 8% of calcium hydroxide and 7% of KOH. Also, the obtained UCS was 16.6 MPa after 90 days of air curing at room temperature due to the polycondensation process (Miao et al. 2017). The stabilization of BC or clay soil by the use of GGBS treated with NaOH and Na_2SiO_3 showed that the slag content, alkali to the slag ratio and curing period are the main factors affecting the strength properties of the mixture (A. Allahverdi et al. 2008; Singhi et al. 2017).

2.3 Reviews on Ms, Na_2O, w/b ratio, and Binder Content

The investigation on the potential usage of GGBS in stabilizing the soil with 3, 6, 9 and 12% of GGBS content showed that there was an increase in MDD, UCS, and CBRwith the increase in GGBS(Yadu and Tripathi 2013). The optimum amount of GGBS to stabilize the soft soil was found to be 9%, which may be due to the presence of water (moisture) which helps in the further formation of the cementitious compound between the soil's Calcium hydroxide and the pozzolanic GGBS. The effect of Na_2SiO_3 alone on kaoline clay slurry considering Ms in the range of 1.74 to 3.25 was investigated and found that increasing the amount of fluidizer caused the faster stabilization of dispersion structure and, the best fluidization properties were possessed by $Na_2SiO_3$with the silicate moduli in the range of 2 to 2.5 with a concentration of approximately 0.3 % by weight to the kaolin dry mass(Stempkowska et al. 2017). Higher Ms i.e., above 3 had weaker stabilization properties which can be associated with the reduction of pH of the slurry. The lower silica moduli smaller than 2 precipitated free silica in the suspension and increased the alkalinity of ceramic slurry. Considering the properties of GGBS and alkali solutions, Mohammad et al. 2016 investigated the effect of $Na_2SiO_3$with GGBS paste with respect to shear strength. A constant 2% of cement in clay with the different ratios of $Na_2SiO_3$of 1, 1.5, and 2% by weight of dry soil were mixed with the different percentage of slag of about 3%, 4% and 5% by weight of the dry soil. It was observed that the most effective amount which impact on cohesion value was at 1 to 1.5% Na_2SiO_3 (preferably 1%) which may be due to the reason that the sodium silicate gel fills the voids between soil particles and further increased dosage forms weak bonding in gel and consequently the cohesion will be decreased. It was also observed that the greater amount of GGBS increases the shear strength and the optimal amount of GGBS was around 5%. But the $Na_2SiO_3$is rarely used as an independent activating unit since it doesn't possess enough activation potential to initiate the pozzolanic reaction alone. Rather, it is commonly mixed with the NaOH or KOH as a fortifying agent to enhance the alkalinity and increase overall specimen strength. So, for the best combination of alkali solutions, the Na_2SiO_3 or potassium silicate and NaOH or KOHwere used (Petermann et al. 2010). The compressive strength of alkali-activated GGBS paste also increased with an increase in the added amount of Na_2O in the mixture where the highest compressive strength of 166.32 MPa was found (Yang et al. 2012a).

2.4 Reviews on silica modulus

The compressive strength increased at 4 to 8 % of Na_2O ata faster rate and further increasefrom 8 to 10%of Na_2O the improvement in strength was gradualand a further increase in Na_2O beyond 10%,the strength decreased due to excess of Na_2O content(A. Allahverdi et al. 2008). The clay soil stabilized with 1M NaOH increased the UCS to 1.15 times than ordinary Portland cement and increased molarity of NaOH from 4 to 12M resulted in increased mechanical strength(Ghadir and Ranjbar 2018; Thomas et al. 2018).The alkali-activated GGBS paste was prepared by varying Potassium Oxide(K_2O) from 4 to 10 % by keeping a constant percentage of SiO_2 as 8% and water binder ratio is maintained to 0.32. The maximum compressive strength of 51.44 MPa was achieved at 8 % of K_2O. This may be due to the reason that the increase in cations from KOHwhich provide charge balance and anions in Na_2SiO_3 reacts with Ca^{+2} dissolving from slag grains and forms the primary CSH and further increase in K_2O to 10% reduced the strength due to the reason that all particles might not be completely utilized to produce CSH gel. Early strength was obtained at the high percentage of K_2O content(Qureshi and Ghosh 2013).

2.5 Effect of water binder ratio

Thew/bratio of 0.25 to 0.35 was suitable since the flow of the geopolymer increases with an increase in the quantity of water thus the compressive strength of the geopolymer concrete was inversely proportional to the water to geopolymer binder ratio (Patankar et al. 2013).

The effect of aging on compressive strength of pumice based geopolymer composites was evaluated and found that the best mix design was obtained at 0.36 w/b ratio, 0.68 of Ms and 10% of Na_2O content(Yadollahi et al. 2015). Study onthe effect of dosage and Ms of the alkali-activated solution on the properties of slag mortars considering the NaOH and Na_2SiO_3as alkali solutions with GGBS, it is found that the different dosages of Na_2O to the weight of slag and different Ms values were used. From the investigation, it was found that compressive strength is mainly affected by Na_2O concentration and a minimum of 4% of Na_2O concentration should be used. To attribute the CSH gel formation, the optimal Ms of 0.8was recommended for the maximum compressive and flexural strength(Chi and Huang 2012).

Similar work was reported to know the effects of Ms and dosage of the alkali-activated solution on properties and microstructural characteristics of alkali-activated fly ash mortars(Chi 2015). The compressive strength of alkali-activated fly ash mortar increases with a dosage of Na_2O. At the same dosage of Na_2O and at the higher the Ms of alkaline solution, the compressive strength enhanced for alkali-activated fly ash mortars. This was due to NaOH leads to an immediate dissolution of aluminosilicate solid. This attributed to the higher capacity of NaOH to favour the release of monomers of silicate and aluminate. An increase in the amount of Na_2SiO_3 in the mix favoured the strength development of compositions.

2.6 Durability studies on alkali-activated soil

The durability study such as sulfate attack test was conducted on the alkali-activated GGBS stabilized clay soil and found that the stabilized samples immersed in sodium sulfate for 120 days showed crack free due to the hydration products (CSH) and reduction in UCS due to the increased larger voids (Jiang et al. 2018). The durability of samples replacing GGBS with a ternary cementitious system containing calcium aluminate cement, calcium sulphate and ordinary Portland cement subjected to Sodium chloride, sodium sulphate was conducted and combination of sodium chloride and sodium sulphate enhanced the durability with less porosity, UCS and chloride content(Li et al. 2018). Clayey soil stabilized with alkali-activated fly ash under sulphate attack showed that the specimens immersed in 5% of sodium sulphate solution for 3 weeks gave integrated structure (Singhi et al. 2017). When GGBS treated with magnesium dioxide tested for durability under 5% of concentrated sulphate attack, the integrity in the samples was achieved compared to Portland cement samples. Whereas in the case of the WD test, the samples found decreased strength after the 5th cycle (Jiang et al. 2018). The fly ash geopolymer showed susceptibility against sulphate attack due to the formation of the Na-Al-Si network structure (Sukmak et al. 2015). Similar work was done on the durability of alkali-activated lithomargic clay stabilized with fly ash or GGBS was found durable in WD and FT test (Amulya et al. 2018).

2.7 Microstructure Analysis of alkali-activated material

From the Scanning Electron Microscope (SEM) technique, microstructure images of the alkali-activated fly ash mortars(Chi 2015), biomass silica stabilized organic soils(Hassan et al. 2019), pumice based geopolymer composites(Yadollahi et al. 2015), alkali-activated olivine soil(Pourakbar et al. 2017) and alkali-activated lithomargic clay using fly ash and GGBS(Amulya et al. 2018)showed the crystal orientation of the surface of samples which is due to the formation of strong bonds.

2.8SUMMARY

From the above literature review, it is observed that the stabilization of different soils was done using different materials to improve their engineering properties to use for construction purposes. The additives such as cement, lime, ashes, enzymes, fibres and marginal materials such as fly ash, aluminium dross and GGBS used in lateritic and BC soil stabilization were reviewed. Also, the stabilization using chemicals such as NaOH, Na_2SiO_3, KOH, sodium and potassium carbonates etc. were reviewed. The studies on the use of marginal materials along with alkali solutions in improving the mechanical properties of the concrete, mortar and pastes were discussed. The literature on durability and microstructural studies of the mixes are reviewed. Based on the gap of the literature review, the use of GGBS and alkali solution in the stabilization of lateritic and BC was derived as a scope of the present work.

CHAPTER 3

MATERIALS AND METHODS

3.1 General

This chapter deals with the materials used in the present investigation. Lateritic soil, BC soil, and alkali solution are the important materials used in the stabilization.

3.2 Materials

3.2.1 Soil

3.2.1.1 Lateritic soil

The word Laterite is referred to as soil as well as rock type and is named by a Scottish physician, Francis Buchanan-Hamilton in southern India in 1807. The lateritic soil features vary according to climate depth of occurrence and location. As the lateritic soil is rich in iron oxide, it is red in colour. For the present work, the lateritic soil is procured from the nearby sites of the National Institute of Technology Karnataka, Karnataka, India.

3.2.1.2 BC Soil

In India, expansive soils are called BC soil and the name is derived from its colour. BC soil is rich in silica, lime, iron, magnesia and alumina and the clay mineralogy belongs to the smectite group of which montmorillonites are predominant (Y 2013). As the BC soil is highly problematic in road construction, an attempt is made to improve its engineering properties. For the present investigation, the BC soil is procured from Kadur, Chikmagalur district, Karnataka, India.

3.2.2 Stabilizers

In the present work, the GGBS and alkali solutions are used.

3.2.2.1 GGBS

The GGBS is the material to be used as an alternative binder. The GGBS is produced as a by-product in the steel processing iron industries. The molten slag in the blast furnace will be cooled down by jet streams of water and quenched at 800^0C. The partially cooled slag will be subjected to the air in a rotating drum and then grounded to fines. The chemical composition of GGBS depends on its ore and it mainly consists of Calcium Oxide (CaO), SiO_2, Aluminium Oxide (Al_2O_3) and Magnesium Oxide (MgO). The GGBS is considered to be the most sustainable, durable and strength attributing material in the construction works. The GGBS is procured from the Jindal Steel Works, Bellary, Karnataka, India. The physical and chemical properties of the GGBS are tabulated in Tables 3.1 and 3.2 respectively.

Table 3.1 Physical properties of GGBS

Properties	Results
Specific gravity	2.86
Size (Micron)	<75
Water content (%)	24.5
Loss of ignition	0.05

Table 3.2 Chemical properties of GGBS

Oxides	CaO	SiO_2	Al_2O_3	MgO	Fe_2O_3	MnO
Amount (%)	37.3	37.8	14.3	8.8	0.98	0.01

3.2.2.2 Alkali Solutions

The alkaline material can be solids or fluids which dissolve in water. The alkalinity or acidity of materials is measured using the pH scale. The alkali materials will be having pH more than 7 which helps the polymerization reaction. For the present work, the combination of two alkali solutions such as NaOH and Na_2SiO_3 are used.

1. **Sodium Hydroxide (NaOH)**

The NaOH is an inorganic alkaline material and it is generally called caustic soda. The NaOH consists of sodium cation and hydroxide anions. It is highly soluble in water and burns the skin as it absorbs the moisture and carbon dioxidefrom the atmosphere. NaOH is available in the form of flakes or pellets. For the present work, the flakes were used.

2. **Sodium Silicate (Na_2SiO_3)**

The $Na_2SiO_3$is a neutral or alkali solution which is commonly called as water glass. It is available in transparent colourless liquid or in white powder form and easily soluble in water. For the present work, the Na_2SiO_3in liquid form was used. The Na_2SiO_3 consists of 18.6% of Na_2O, 32.6% of SiO_2 and 48.8% of water with 1.75 of Ms.

3.2.3 Water

The potable water available in the laboratory was used.

3.3 Methods Adopted

The laboratory tests were conducted at room temperature of around 25 to 27^{0}C with the relative humidity of 54 to 56%. The engineering and geotechnical properties of the mixture were found as per Indian Standards (IS) but the durability studies were conducted as per American Standards of Testing Materials (ASTM) standards and even the IS: 4332: Part IV- 1968 gives the durability test procedure. The grain size sieve analysis of soil was carried out as per IS: 1498-1970 and classified the lateritic soil as high plastic clay (CH) and BC soil as intermediate plastic clay (CI). The specific gravity (IS 2720: Part 3: Sec 1: 1980) andAtterberg limits (IS 2720: Part 5: 1985 and Part 6: 1972) of the untreated soil was carried out. The standard Proctor (IS 2720: Part 7: 1980) andmodified Proctor tests (IS 2720: Part 8: 1983) were conducted in the laboratory to obtain the OMC and MDD. The CBR testwas carried out for both untreated and stabilized soil as per IS: 2720-16 1987.

3.3.1 Unconfined Compressive Strength (UCS) Test

The UCS test was conductedon cylindrical samplesof 38mm diameter and 76mm length forstabilized and untreated soil at both Proctor densities. The samples were air-cured at ambient temperature for different curing periods of 0 (immediately after casting samples), 3, 7 and 28 days. The cured samples were tested under the gradual application of the axial load at the rate of 1.25 mm/min. The test was conducted as per IS 2720: Part 10: 1991.

3.3.2 Durability Test

The durability of the road materials should maintain stability, integrity and bonding withthe soil under cyclic weathering change and adverse conditions over years of exposure (Dempsey and Thompson 1968). The durability test includes the WD and FT tests conducted on a cylindrical samples having dimension 38 × 76 mm same as the size of UCS moulds. The WD test was carried out as per ASTM 559D and during the WD test, the samples will be immersed in water for 5 hours and oven-dried at 71^0C for 42 hours. Each wetting and drying of a sample called one cycle. The weight loss of samples at every cycle will be noted down. The percentage weight loss after 12 cycles of WD should not exceed 14%. Similarly, the FT test was conducted as per ASTM 560D by freezing samples at -23^0C for 24 hours and thawing at 21^0C for 23 hours. Each freezing and thawingare called one cycle. The weight loss at every cycle will be noted down. The percentage weight loss after 12 cycles of FT should not exceed 14%.

3.3.3 Flexural strength test

The flexural strength of the durability passed stabilized soil was conducted as per IS 4332: Part 6: 1972 at both Proctor densities. The rectangular beam of 300×75×75 mm was cast for the durability passedstabilized soil samples at both Proctor densities and cured for 28 days. The gradual application of load at the rate of 1.25 mm/min was applied to the sample under two-point loading until it breaks. The breaking load will be noted down and the flexural strength of the sampleswas calculated using Equation (3.1) where the weight of the beam is neglected.

$$M_R = \frac{Pl}{bd^2} \qquad (3.1)$$

Where,

P- Maximum applied load

l- Span length

b- Average width of the specimen

d- Average depth of specimen

M_R- Modulus of rupture

3.3.4 Fatigue Test

The fatigue test was conducted on the stabilized soil samples to determine the behaviour of materials under repeated cyclic loading condition which follows the sinusoidal curve. The durability passed stabilized soil samples having a dimension of UCS sample were cast and cured for 28 days. The repetitive cyclic load was applied axially on the sample at 1 Hz frequency. The fatigue life of each sample was noted down.

3.3.5 Chemical composition

The chemical composition of the soil such as SiO_2, Al_2O_3, Fe_2O_3, CaO, and MgO are found in stabilized soil. The pH and electrical conductivity of the stabilized soil were noted down. The elements like SiO_2, Fe_2O_3 and Al_2O_3were found as per IS 2720: Part 25: 1982 whereas, CaO and MgO were found by titration. The chemical composition of the stabilized soil helps to know the utilization of oxides in forming strength attributing structure.

3.3.6 Microstructure analysis using SEM

The microstructure images of the stabilized soil samples were collected using the SEM technique. The SEM technique gives the image of stabilized samples at different resolutions. In the present work, the microstructure images of the samples were collected at a resolution of 2k and 10 micrometers. The obtained image shows the presence of voids or closely packed structures which helps in analyzing the strength attributing structure.

3.4 Work Plan

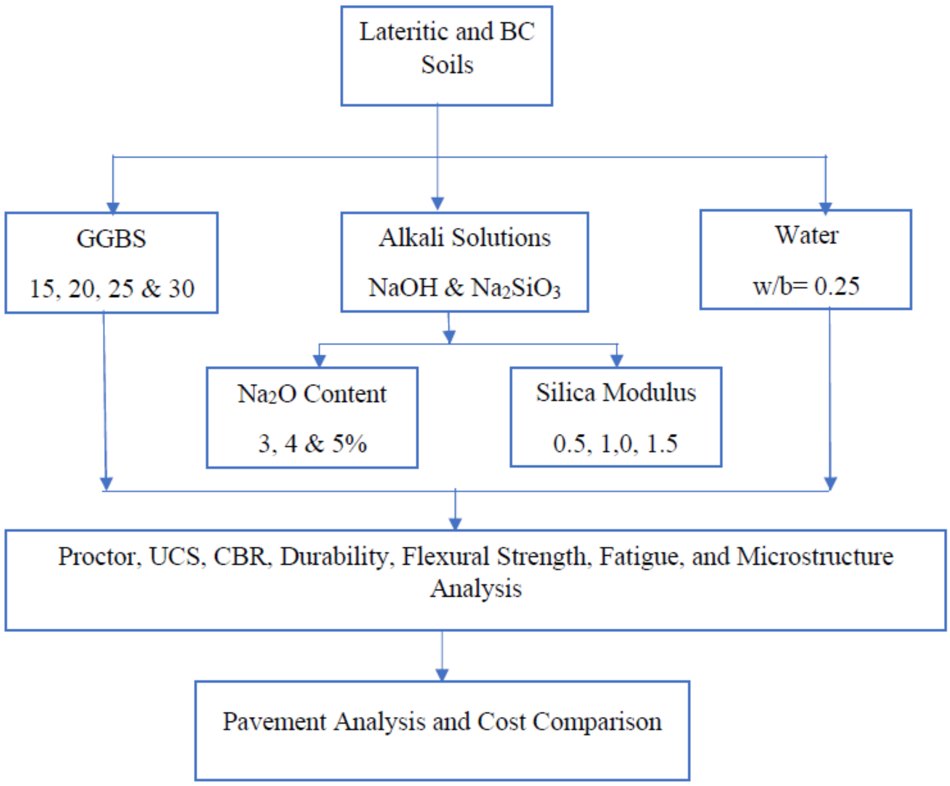


Figure 3.1 The work plan of the present study

3.5 Dosage Calculation

Illustration (1): GGBS- 15%, Soil- 85%, Na_2O- 4%, Ms- 1.0, w/b- 0.25

For a mix of 1kg or 1000g,

Amount of GGBS = (15/100) ×1000= 150g.

Amount of soil = 1000-150= 850g

Amount of Na_2O = 4% of GGBS= (4/100) × 150= 6g

To calculate SiO_2,

Silica Modulus= SiO_2/Na_2O = 1.0

= SiO_2 = 1.0×Na_2O

= SiO_2 = 6g

To calculate amount of Na_2SiO_3,

Na_2SiO_3 consists of 18.7% of Na_2O, 32.5% of SiO_2 and 48.8% of water.

Hence, 1 kg of Na_2SiO_3 consists of 187g of Na_2O, 325 g of SiO_2 and 488 g of water.

As 325g of SiO_2 is available from 1000g of Na_2SiO_3

6g of SiO_2 can be obtained from (6×1000)/325=18.5 g

Therefore, for the present dosage, 18.5g of Na_2SiO_3 is required.

<u>To calculate the amount of Na_2O available from 18.5g of Na_2SiO_3,</u>

As 1000g of Na_2SiO_3 consists of 187g of Na_2O

Therefore 18.5g of Na_2SiO_3 consists of (18.5×187)/1000 = 3.5g of Na_2O.

But we need 6g of Na_2O and 3.5g of Na_2O can be obtained from 18.5g of Na_2SiO_3.

Therefore, remaining (6-3.5) = 2.5g of Na_2O needs to get from NaOH.

<u>To calculate the amount of NaOH required to get 2.5g of Na_2O,</u>

$$2\,NaOH \longrightarrow Na_2O + H_2O$$

The atomic weight of sodium is 23, oxygen is 16, hydrogen is 1.

Therefore, 80g of NaOH consists of 62g of Na_2O.

As 62g of Na_2O is present in 80g of NaOH

2.5g of Na_2O can be obtained from (2.5×80)/62 = 3.2g of NaOH

Therefore, the required amount of NaOH for the given dosage is 3.2g.

<u>To calculate the amount of water</u>

Amount of water in Na_2SiO_3,

As 1000g of Na_2SiO_3 consists of 488g of water

Hence, 18.5g of Na_2SiO_3 consists of (18.5×488)/1000 = 9g of water in Na_2SiO_3.

Amount of water in NaOH,

80g of NaOH consists of 18g of water

Hence, 3.2g of NaOH consists of (3.2×18)/80 = 0.72g of water in NaOH.

Therefore, the total amount of water in alkali solution= (9+7.2) = 18.72g.

Amount of water from w/b ratio

(water/binder) = 0.25

Amount of water= (0.25×150) = 37.5g.

Therefore, the amount of water from alkali solution and from water binder ratio is (37.5+18.75) = 56.3g.

Optimum Moisture Content obtained from this dosage is 25%.

Therefore, the amount of water to be added to the mixture to achieve MDD is (25/100) × 850 = 212.5g.

But the amount of water to be added to the mixture of soil, GGBS and alkali solution to meet MDD excluding water content in alkali solution. Hence, the amount of water to be added is (212.5-56.3) = 156.2g of water.

3.6 Alkali Solution Preparation

Initially, the NaOH solution has to be prepared by adding 3.2 g of NaOH flakes to the 37.5 g of water obtained from the w/b ratio. Due to the exothermic reaction, some amount of water will be evaporated and hence, an extra amount of water should be added to maintain the mass to the calculated mass of NaOH and water. Once the solution is prepared, add 18.5 g of Na_2SiO_3 and stir it well. The prepared solution kept overnight for the reaction to takes place. The amount of soil and GGBS should be calculated as per the dry density of the mixture and are mixed evenly, then the prepared alkali solution and 156.2 g of water should be added to maintain OMC to achieve MDD and mixed thoroughly.

3.7 Summary

In this chapter, the materials used like lateritic and BC soil, stabilizers like GGBS, NaOH, and Na_2SiO_3 are discussed. The methodology adopted for various tests is elaborated. Tests like durability, which is useful for the stabilized material were discussed. The flexural strength, fatigue, chemical tests and microstructure analyses of the stabilized soil were discussed. The dosage calculation and preparation of alkali solution in the laboratory are discussed. The subsequent chapter discusses the stabilization of lateritic soil.

CHAPTER 4

STABILIZATION OF LATERITIC SOIL

4.1 General

This chapter deals with the stabilization of lateritic soil using GGBS along with alkali solutions. Various laboratory tests were conducted to evaluate the engineering properties of lateritic soil. The engineering properties of the untreated lateritic soil are tabulated in Table 4.1.

Table 4.1 Engineering properties of the lateritic soil

Sl no.	Property	Lateritic Soil
1	Specific Gravity	2.6
2	Grain size distribution (%) a) Gravel b) Sand c) Silt d) Clay	 21 20 20 39
3	IS Classification of Soil	CH
4	Consistency limits (%) a) Liquid Limit (LL) b) Plastic Limit (PL) c) Plasticity Index (PI)	 66 33 33
5	Proctor tests Standard Proctor a) OMC (%) b) MDD (g/cc) Modified Proctor a) OMC (%) b) MDD (g/cc)	 24.5 1.58 22.5 1.72
6	CBR Value (%) At Standard Proctor density a) Unsoaked condition b) Soaked condition At Modified Proctor density a) Unsoaked condition b) Soaked condition	 7 4 18 9
7	UCS (kPa) At Standard Proctor density At Modified Proctor density	 428 530

4.2 Atterberg limits

The consistency limits of the lateritic soil are tabulated in Table 4.1 and it is classified as high plastic clay (CH). The Atterberg limits of the stabilized soil couldn't be found as the stabilized soil becomes hard and stiff when GGBS and alkali solution were mixed together due to the exothermic reaction.

4.3 Proctor tests

The standard and modified Proctor tests were conducted on both stabilized and untreated soil samples. Initially, the oven-dried soil and GGBS were mixed thoroughly followed by adding the calculated amount of alkali solutions. The OMC and MDD of each mix obtained from standard and modified Proctor tests are tabulated in Tables 4.2 and 4.3 respectively.Hereafter, the samples of different combinationsare represented sequentially in x-y-z form. Where x is GGBS content (%), y is Na_2O dosage (%) and z is Ms at constant water binder ratio of 0.25. For example, the sample of 30-6-1.0 indicates 30% of GGBS, 6% of Na_2O and 1.0 Ms.

Table 4.2 The standard Proctor test results of stabilized soil

Na_2O dosage (%)	15% GGBS		20% GGBS		25% GGBS		30% GGBS	
	OMC (%)	MDD (g/cc)	OMC (%)	MDD (g/cc)	OMC (%)	MDD (g/cc)	OMC (%)	MDD (g/cc)
At 0.5 Ms								
4	23	1.69	23.2	1.72	20.5	1.69	20.5	1.7
5	23	1.67	23.6	1.71	21	1.7	21	1.72
6	24	1.65	24	1.7	21.5	1.72	20.5	1.72
At 1.0 Ms								
4	23	1.66	23	1.71	20	1.7	20	1.68
5	23	1.65	23.8	1.69	21.5	1.7	20.5	1.71
6	23	1.64	24	1.65	21	1.68	20	1.7
At 1.5 Ms								
4	26.5	1.57	24	1.7	20.5	1.6	21	1.65
5	24	1.6	24.5	1.65	21	1.65	21.5	1.69
6	24.5	1.58	25	1.6	23	1.66	21.8	1.65

Table 4.3 The modified Proctor test results of stabilized soil samples

Na_2O dosage (%)	15% GGBS		20% GGBS		25% GGBS		30% GGBS	
	OMC (%)	MDD (g/cc)	OMC (%)	MDD (g/cc)	OMC (%)	MDD (g/cc)	OMC (%)	MDD (g/cc)
At 0.5 Ms								
4	18.5	1.81	20	1.82	18	1.8	18.5	1.8
5	18	1.77	19.5	1.8	18.5	1.82	18	1.81
6	19.5	1.76	19	1.8	18.5	1.83	17	1.82
At 1.0 Ms								
4	20.5	1.77	21	1.83	19.5	1.8	19	1.8
5	18.5	1.78	19.5	1.8	19	1.81	18	1.83
6	18	1.75	19	1.8	17	1.84	17	1.82
At 1.5 Ms								
4	22.5	1.69	21.5	1.7	19.5	1.75	19.5	1.78
5	20.5	1.7	23.5	1.65	19	1.79	18.5	1.81
6	19	1.74	22	1.75	18	1.82	17.7	1.8

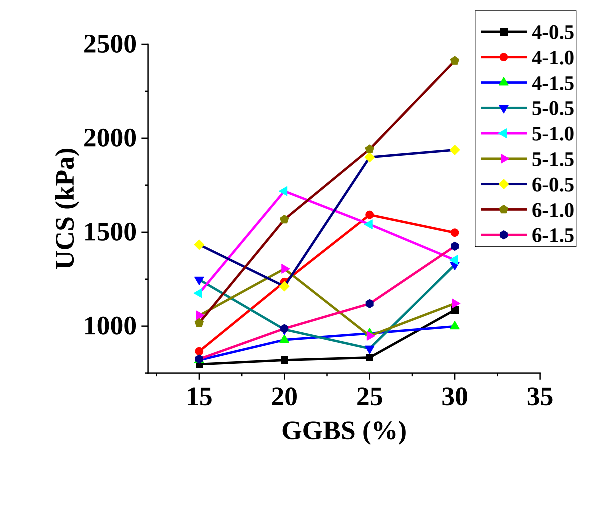
The MDD of 1.72g/cc is achieved for the stabilizedsoil sample of 30-6-0.5 from standard Proctor test and 1.84 g/cc for stabilized soil sample of 25-6-1.0 from the modified Proctor test. The MDD at higher GGBS content may be due to filling of fines into the voids during compaction and hence densifies the stabilized soil(Lekha et al. 2015). The obtained OMC and MDD of the stabilized soil were considered for the sample preparation for UCS, CBR, durability, flexuralstrength and fatigue tests.

4.4 Unconfined Compressive Strength (UCS) Test

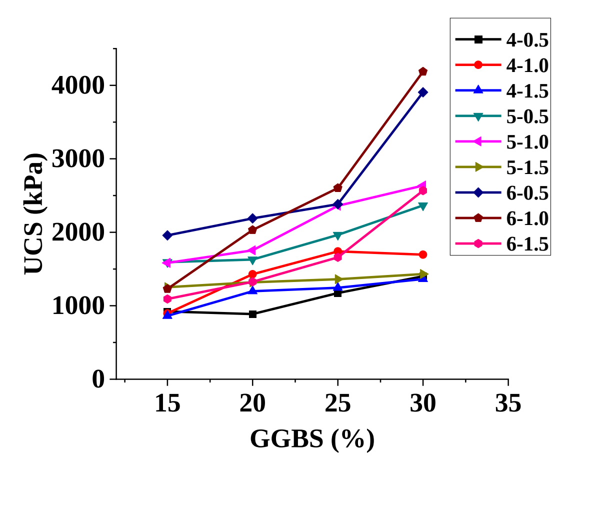
4.4.1 At standard Proctor density

The cylindrical samples of stabilized soil at both standard and modified Proctor densities were air-cured at ambient temperature for 0 (Immediately after casting), 3, 7 and 28 days and tested for UCS. At standard Proctor density, the highest UCS of 6341kPa is achieved for the sample of 30-6-1.0 after 28 days curing which is 14.8 times that of the natural soil. The variation of UCS values with a variation of GGBS, Na_2O dosage, Ms and curing period at standard Proctor density are depicted in Figures 4.1 (a-d), 4.2 (a-d), 4.3 (a-d) and 4.4 (a-c).

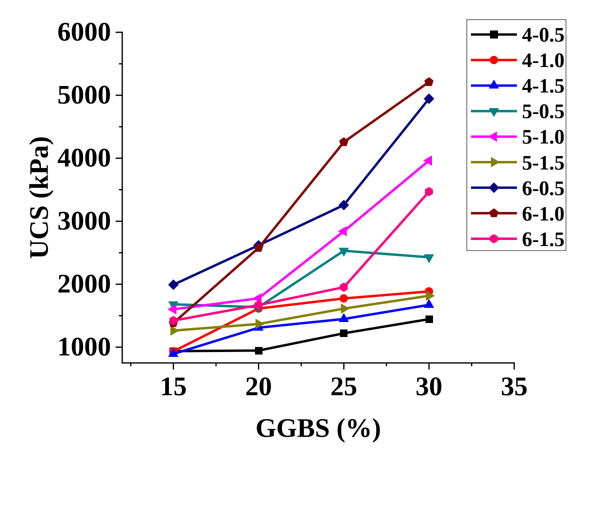
4.4.1.1 Effect of GGBS content



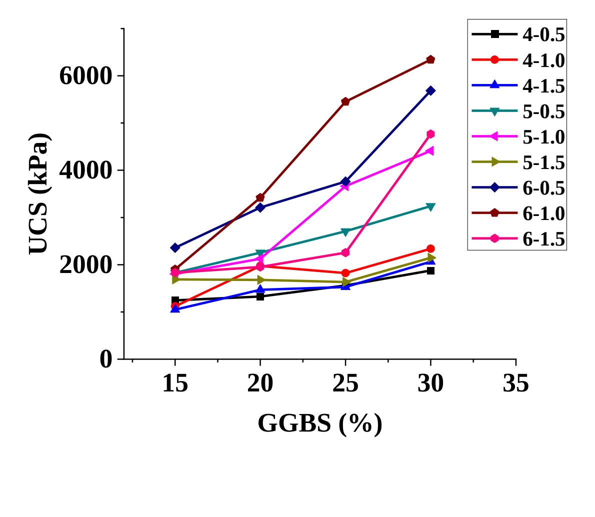
a) 0 days curing



b) 3 days curing



c) 7 days curing



d) 28 days curing

Figure 4.1 The variation of UCS for different GGBS content at standard Proctor density

From Figure 4.1 (a-d), It is observed that as GGBS in the lateritic soil mix increases from 15 to 30%, the UCS keeps on increases. The highest UCS is obtained for samples at 30%GGBS replacement. As the GGBS is rich in CaO, the dissolute Ca ions react with silicates and aluminates available from alkali solutions forms aluminosilicate hydrates. The highest mechanical strength was achieved when GGBS increased to 30% due to the polymerization (Phoo-Ngernkham et al. 2015).

4.4.1.2 Effect of Na_2O dosage

At standard Proctor density, when the Na_2O content in the stabilized soil increases from 4 to 6%, the UCS increases gradually for any curing period. The highest UCS is achieved for thesoil sample consisting of 6% of Na_2O dosage. The Na_2O dosage helps in the dissolution of solid particles and reacts with GGBS to form a strong CASH bond. The variation of UCS at different Na_2O content is depicted in Figure 4.2 (a-d).

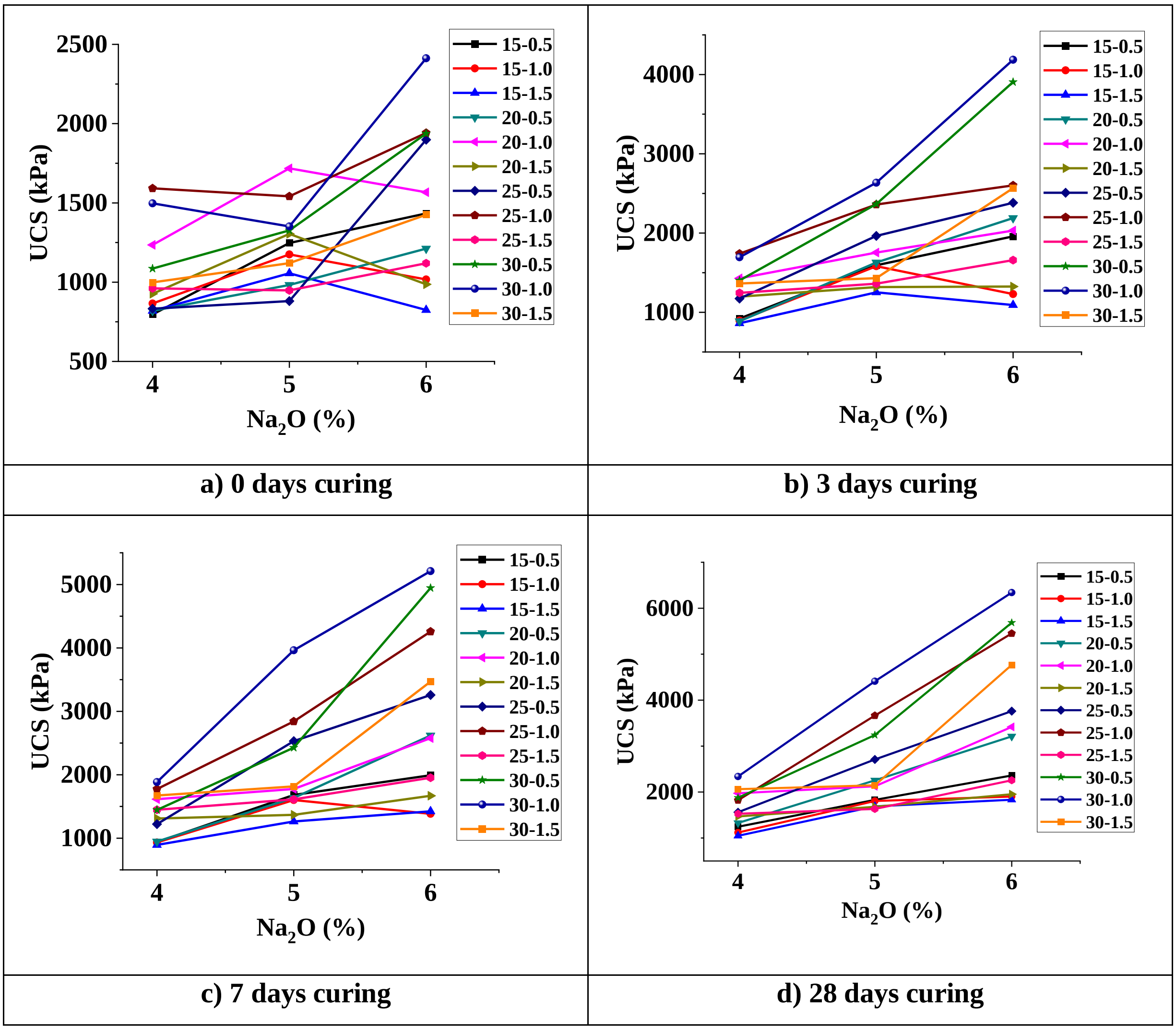


Figure 4.2 The variation of UCS for different Na_2O dosages at standard Proctor density

4.4.1.3 Effect of Ms

At standard Proctor density, as Ms of the alkali solution in the stabilized soil increases from 0.5 to 1.0, the UCS increases, whereasa further increase in Ms of alkali solution from 1.0 to 1.5, the UCSdecreases. It is observed that when Ms is 1.0, the concentration of Na_2O and SiO_2 will be equal and the increased pH level in the mix creates the alkaline environment and thus enhances the polymerization process. The increased Ms to 1.5 shows the detrimental effectlike efflorescence and brittleness (Firdous and Stephan 2019). The highest UCS is achieved for the samples prepared with alkali solution having 1.0 Ms and the variation of UCS for differentMs at all curing periods is depicted in Figure 4.3 (a-d).

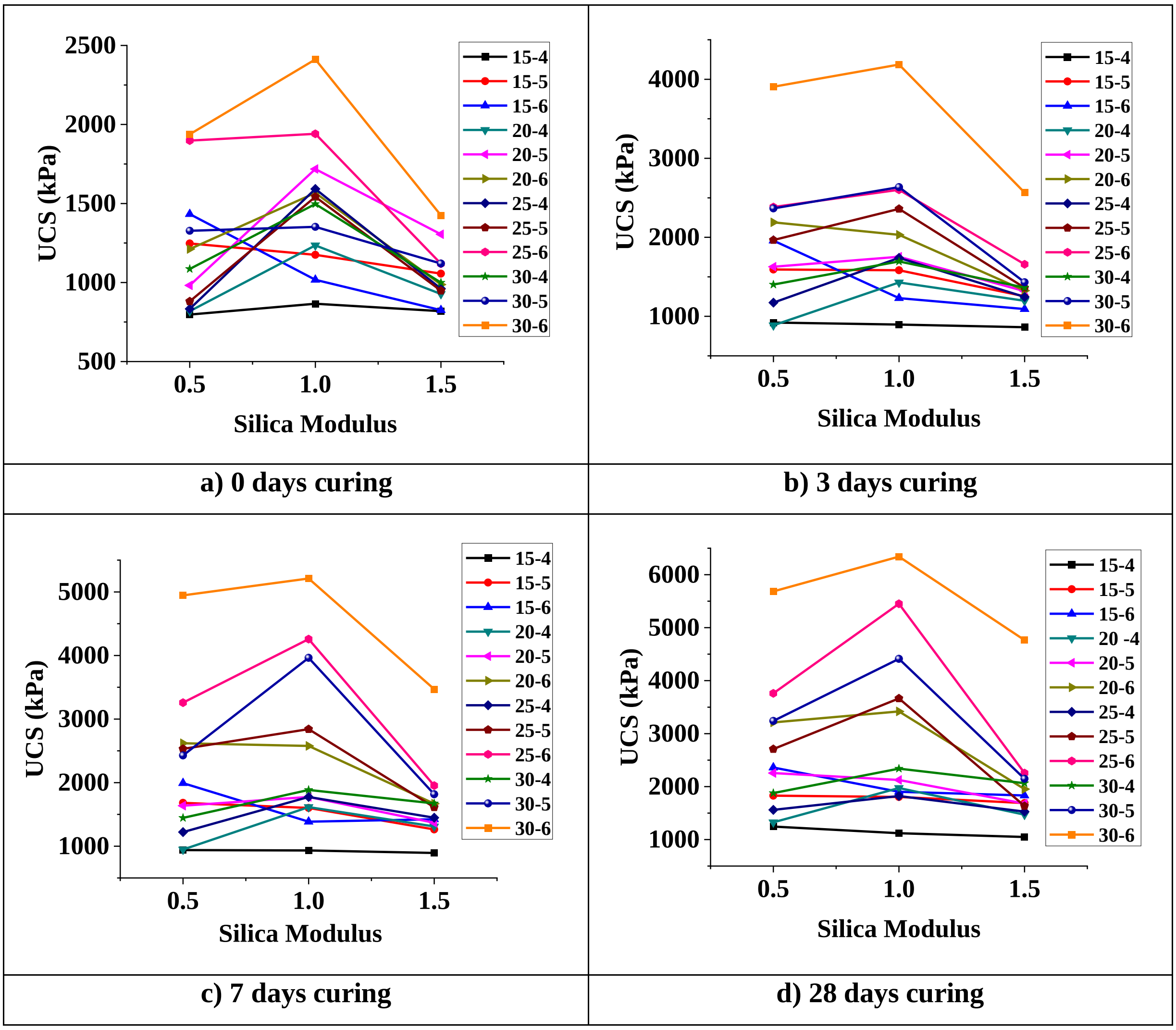
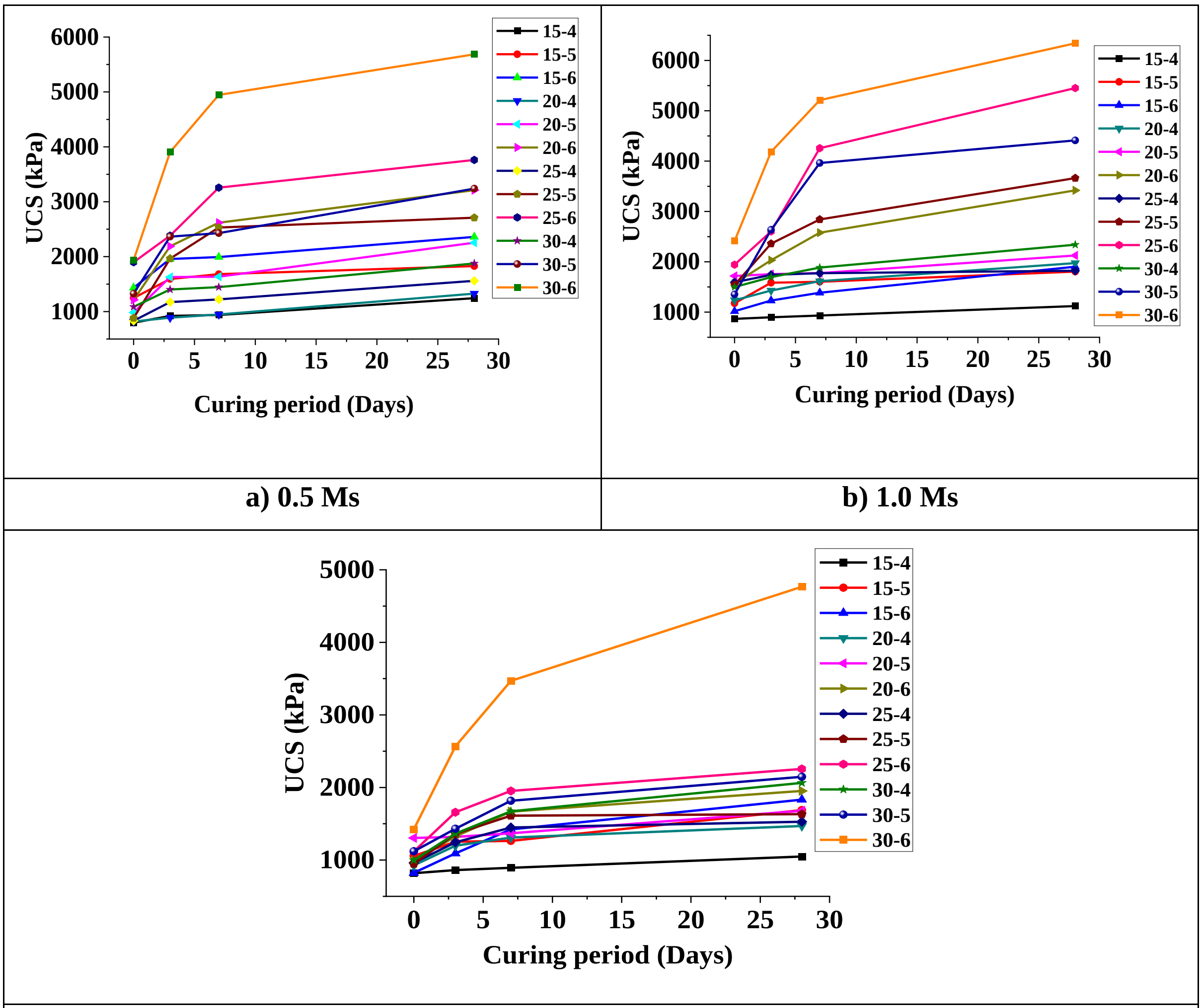


Figure 4.3 The variation of UCS for different Ms at standard Proctor density

4.4.1.4 Effect of the Curing period



a) 0.5 Ms

b) 1.0 Ms

c) 1.5 Ms

Figure 4.4 The variation of UCS for different curing period at standard Proctor density

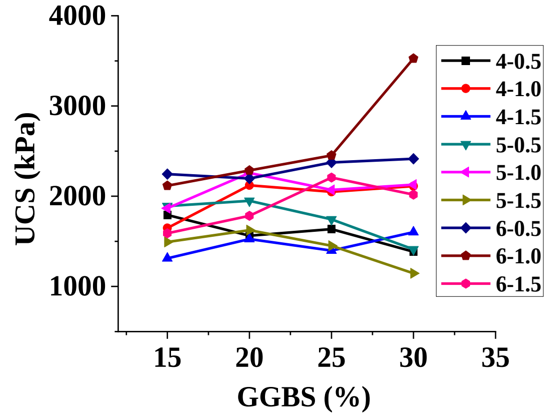
The stabilized soil samples were cured at air temperature for a period of 0, 3, 7 and 28 days. From Figure 4.4 (a-c), it is found that as the curing period of the stabilized soil increases, the UCS increases gradually. The generated heat rapidly increases the rate of polymerization initially and as the curing period further increases, at ambient temperature, the gradual polymerization helps to form calcium aluminosilicate hydrate and hence the UCS increases (Memon et al. 2013; Pourabbas Bilondi et al. 2018). When samples were cured beyond 28 days the improvement in strength was marginal. Therefore, a maximum of 28 days curing was considered in this study.

4.4.2 At modified Proctor density

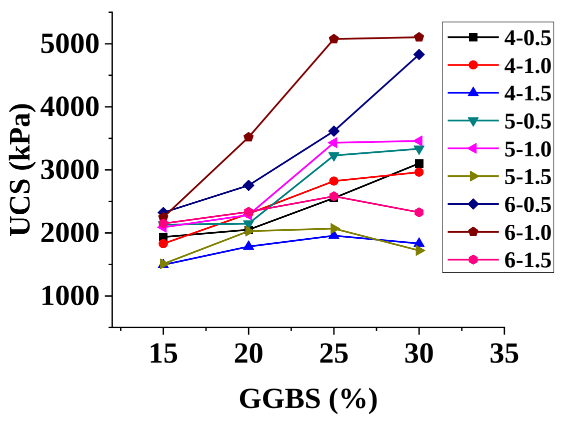
The stabilized soil was tested for UCS at modified Proctor density and from the test results, the highest UCS of 9901kPa was obtained for soil sample of 30-6-1.0 after 28 days curing which is 18.6 times that of the untreated soil.

4.4.2.1 Effect of GGBS content

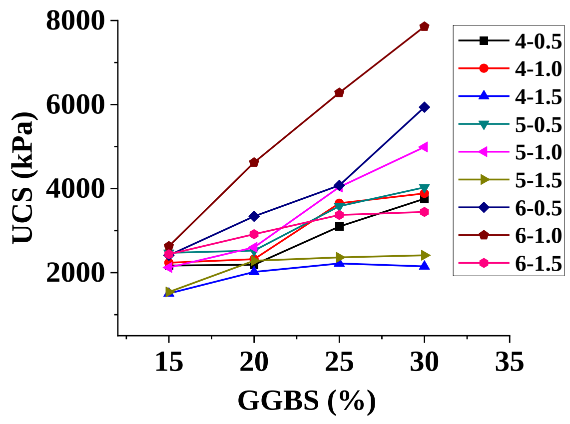
At modified Proctor density, when the stabilized soil replaced with 15 to 30% of GGBS content, the UCS increases gradually. The highest UCS was obtained for the stabilized soil replaced with 30% of GGBS at modified Proctor density. The highest UCS may be due to the heavy compaction effort the voids will be reduced. The variation of UCS for differentGGBS content is depicted in Figure 4.5 (a-d).



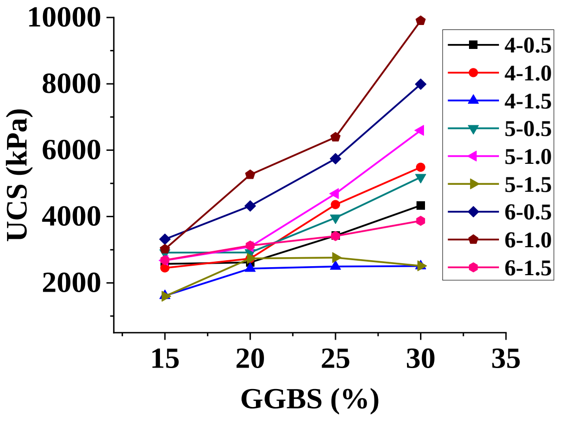
a) 0 days curing



b) 3 days curing



c) 7 days curing

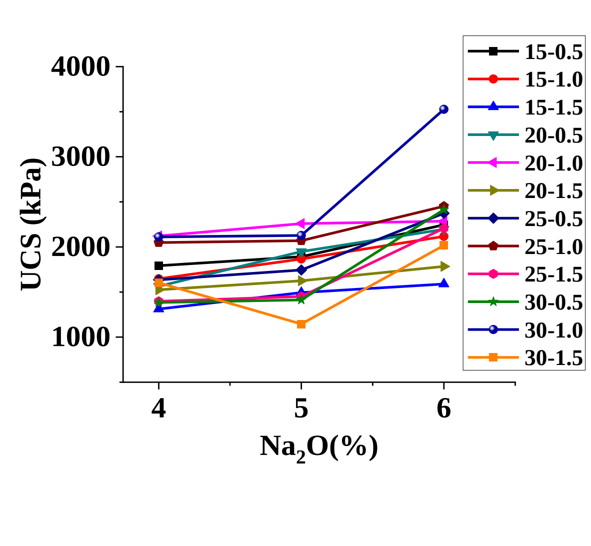


d) 28 days curing

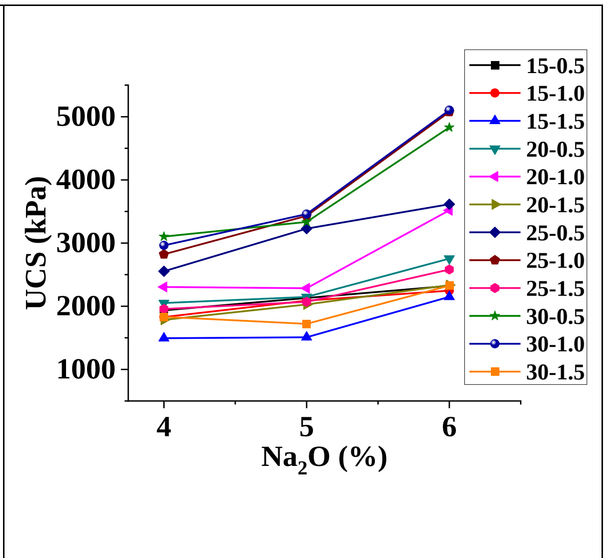
Figure 4.5 The variation of UCS for different GGBS at modified Proctor density

4.4.2.2 Effect of Na_2O

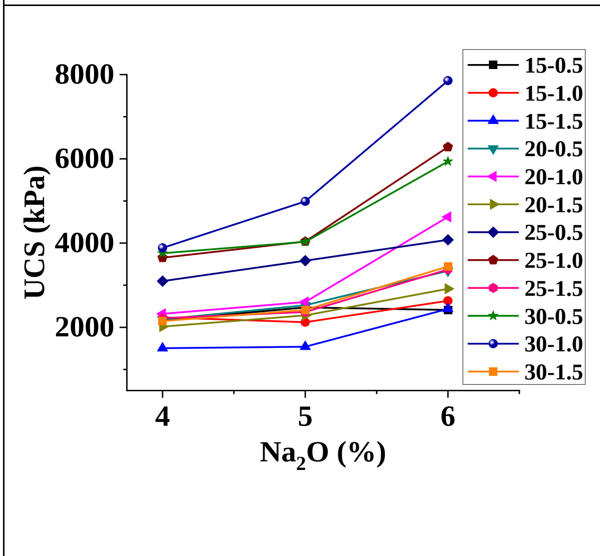
As Na_2O content in the stabilized soil increases from 4 to 5%, a slight increase in UCS at all curing periods was observed, whereas a further increase in Na_2O from 5 to 6%, increases the UCS rapidly. The variation of UCS for different Na_2O dosage are depicted in Figure 4.6 (a-d).



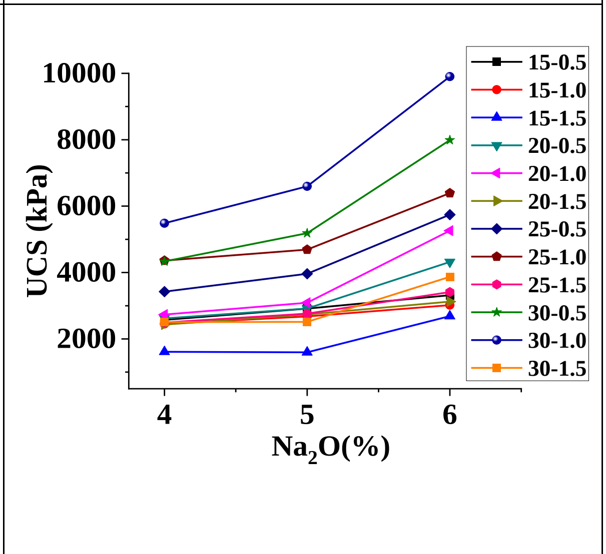
a) 0 days curing



b) 3 days curing



c) 7 days curing

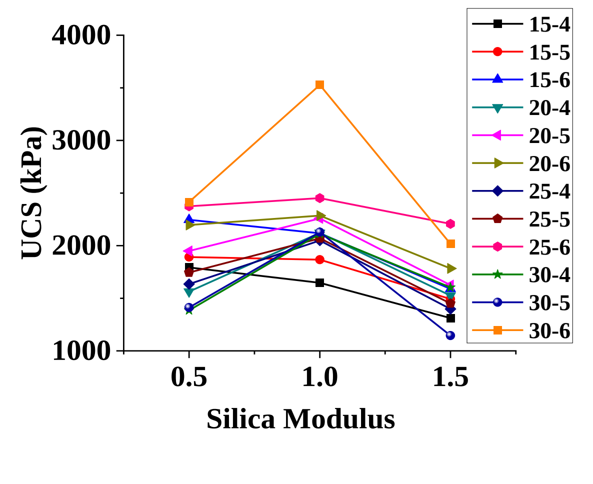


d) 28 days curing

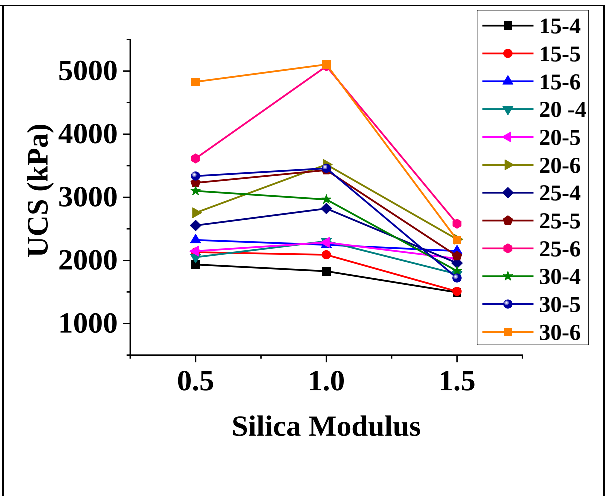
Figure 4.6 The variation of UCS for different Na_2O dosages at modified Proctor density

4.4.2.3 Effect of Ms

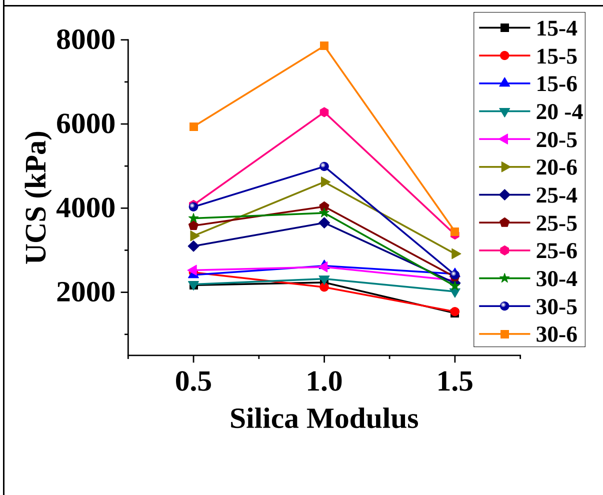
At modified Proctor density, when Ms of the alkali solution in the stabilized soil increases from 0.5 to 1.0, the UCS increases and further increase inMs to 1.5, the UCS decreases rapidly at all curing periods and the variation of UCS at differentMs are depicted in Figure 4.7 (a-d).



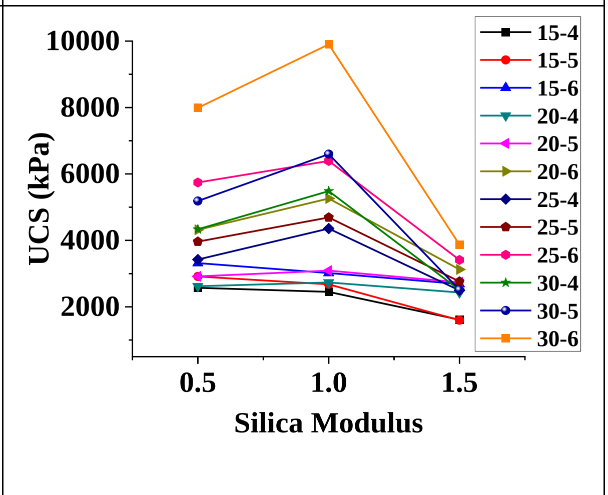
a) 0 days curing



b) 3 days curing



c) 7 days curing

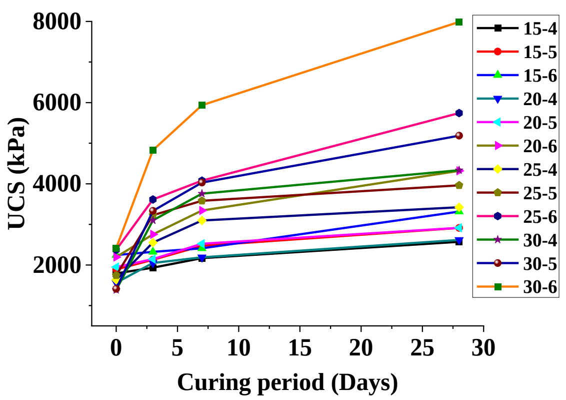


d) 28 days curing

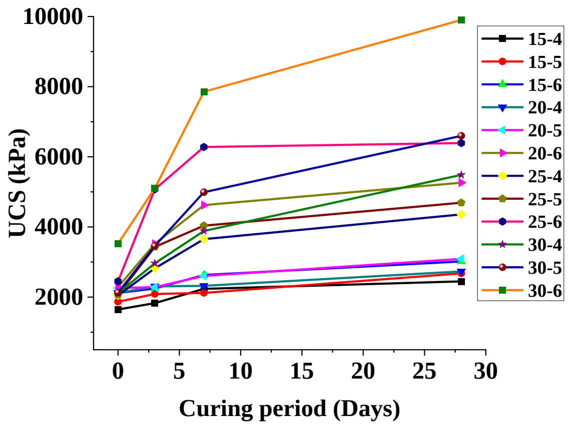
Figure 4.7 The variation of UCS for different Ms of stabilized soil at modified Proctor density

4.4.2.4 Effect of curing periods

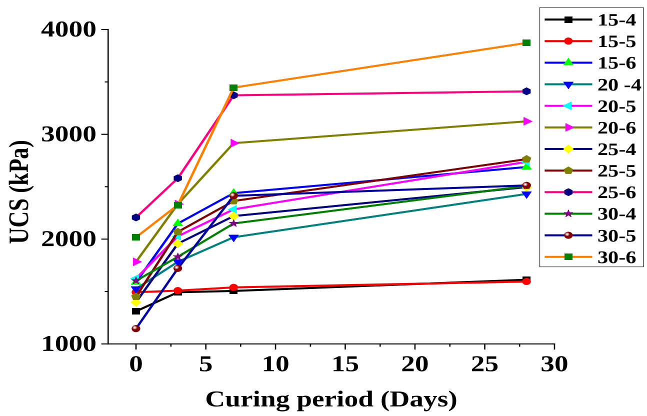
The UCS of the stabilized soil increases with the increase in the curing period. The 28 days cured stabilized lateritic soil samples have achieved the highest UCS at modified Proctor density and the variation is depicted in Figure 4.8 (a-c). When samples were cured beyond 28 days the improvement in strength was marginal. Therefore, a maximum of 28 days curing was considered in this study.



a) 0.5 Ms



b) 1.0 Ms

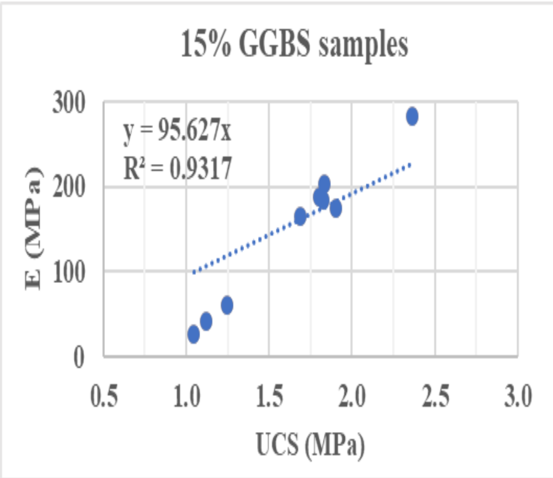


c) 1.5 Ms

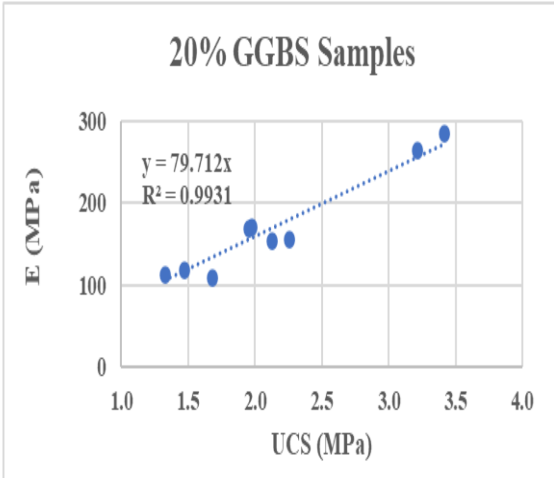
Figure 4.8 The variation of UCS for different curing periods at modified Proctor density

4.4.3Relationship between UCS and Young's Modulus

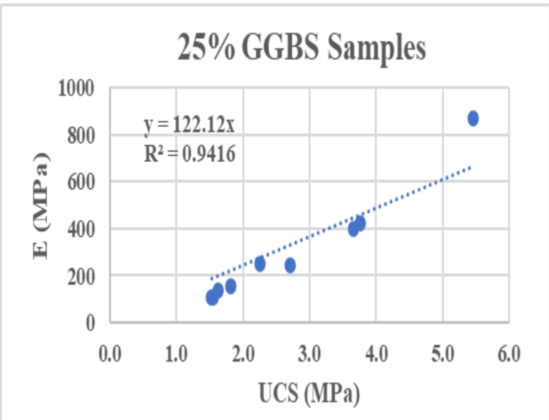
The relation between UCS and modulus of elasticity of stabilized lateritic soil cured for 28 days for standard Proctor and modified Proctor densities are established and depicted in Figure 4.9 to Figure 4.10 respectively. At lower GGBS replacement the correlation is not much significant, but for the higher GGBS replacement (30%) the correlation is quite good. The R^2 values with zero intercept are more than 0.95.



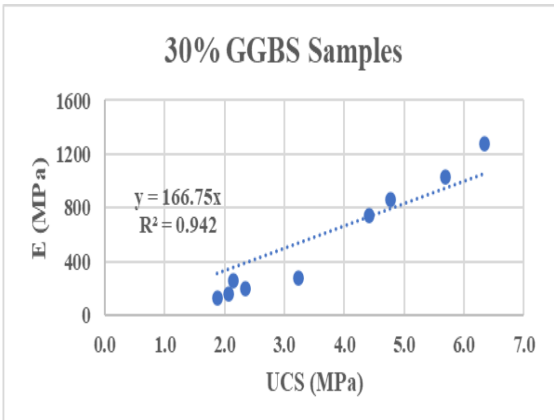
a) 15% GGBS Samples



b) 20% GGBS Samples

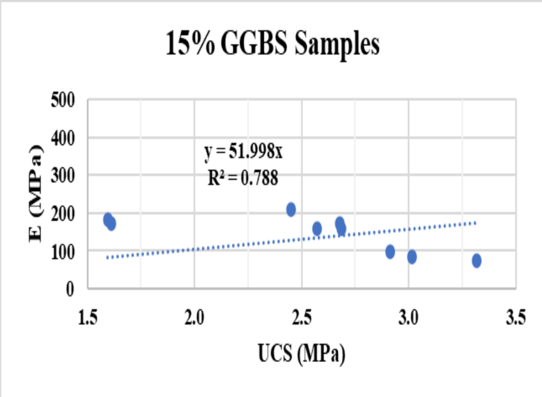


c) 25% GGBS Samples

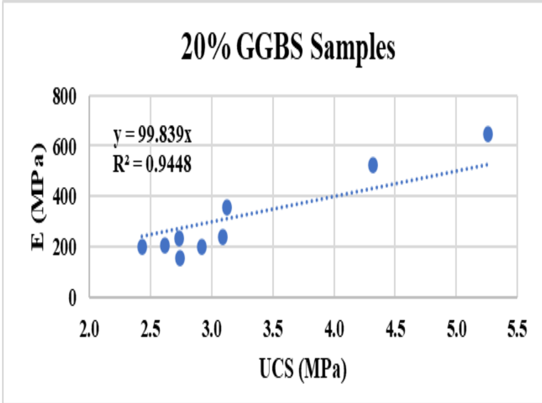


d) 30% GGBS Samples

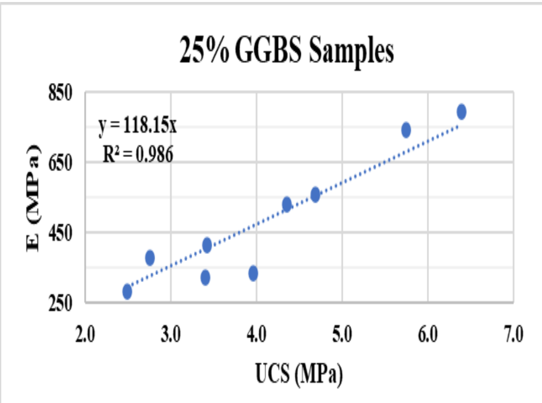
Figure 4.9The relationship between UCS and modulus of elasticity of stabilised lateritic soil at standard Proctor density



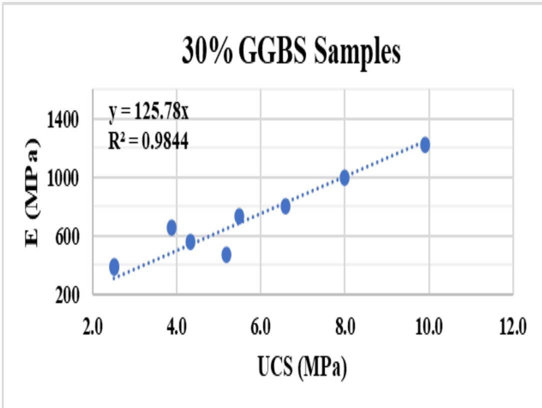
a) 15% GGBS Samples



b) 20% GGBS Samples



c) 25% GGBS Samples



d) 30% GGBS Samples

Figure 4.10 The relationship between UCS and modulus of elasticity of stabilized lateritic soil at modified Proctor density

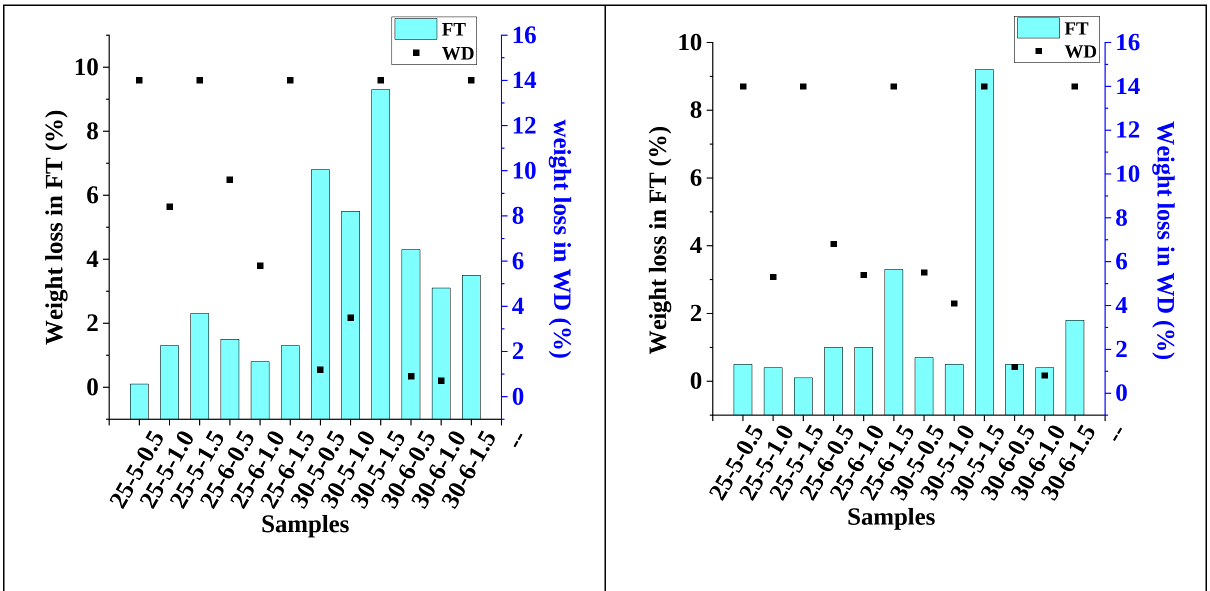
From the Figure 4.10 it is found that 15% GGBS samples are showing correlation having R^2 value 0.788 whereas higher GGBS dosage samples are showing correlation having R^2 more than 0.95.

4.5 California Bearing Ratio (CBR) Test

The stabilized soil samples were cured and tested for both soaked and unsoaked conditions. The cured samples kept for 4 days soaking found moisture absorption only 15%. The plunger could not penetrate into the soil as it was hard due to the high density, and resistance to the penetration was very high. The obtained CBR values were more than 100% and hence, the correlation between UCS and CBR couldn't be established. To ensure the strength of stabilized soil, a durability test need to be conducted.

4.6 Durability

The durability test consists of Wetting-Drying (WD) and Freezing-Thawing (FT) tests.The cylindrical samples of the UCS dimension were cast at both densities and air-cured for different curing periods. The percentage weight loss of samples after 12 alternate cycles of WD and FT tests are tabulated in Table 4.4 and the percentage weight loss of durability passed stabilized lateritic soil at both densities cured for 28 days are depicted in Figure 4.11 (a, b).



a) **Lateritic soil at standard Proctor density** b) **Lateritic soil at modified Proctor density**

Figure 4.11The percentage weight loss of stabilized lateritic soil at both Proctor densities

Table 4.4 The durability test results of lateritic soil at different curing periods

Samples	WD test				FT test			
	Curing periods (days)							
	0	3	7	28	0	3	7	28
	At Standard Proctor Density							
15-4-0.5	1st	1st	1st	1st	*17.8	*16.3	*15.7	*13.6
15-4-1.0	1st	1st	1st	1st	*16.5	*15.9	*15.1	*12.4
15-4-1.5	1st	1st	1st	1st	*15.8	*15.2	*14.9	*11.9
15-5-0.5	1st	1st	1st	1st	*16.1	*15.5	*14.9	*12.7
15-5-1.0	1st	1st	1st	1st	*15.2	*14.4	*14.2	*10.6
15-5-1.5	1st	1st	1st	1st	*16.4	*15.2	*14.3	*11.5
15-6-0.5	1st	1st	1st	1st	*13.9	*12.6	*10.5	*9.1
15-6-1.0	1st	1st	2nd	1st	*13.1	*11.2	*9.6	*8.3
15-6-1.5	1st	1st	1st	1st	*13.5	*13.1	*9.1	*8.9
20-4-0.5	1st	1st	1st	2nd	*18.8	*17.4	*15.9	*14.9
20-4-1.0	1st	1st	3rd	4th	*18.5	*16.3	*14.2	*14.5
20-4-1.5	1st	1st	1st	1st	*19.1	*17.1	*15.1	*14.7
20-5-0.5	1st	1st	1st	1st	*13.8	*12.3	*11.1	*8.5
20-5-1.0	1st	1st	1st	1st	*17.3	*16.2	*15.3	*14.8
20-5-1.5	1st	1st	1st	1st	*18.7	*17.2	*15.4	*15.2
20-6-0.5	1st	2nd	3rd	1st	*10.2	*9.8	*8.1	*7.3
20-6-1.0	1st	3rd	4th	1st	*8.3	*9.2	*7.3	*6.1
20-6-1.5	1st	1st	2nd	1st	*10.0	*9.5	*8.2	*7.1
25-4-0.5	1st	1st	1st	1st	*13.2	*8.2	*8	*2.6
25-4-1.0	1st	1st	1st	1st	*12.5	*8.1	*7.9	*2.1
25-4-1.5	1st	1st	1st	1st	*12.1	*8.7	*6.1	*1.8
25-5-0.5	1st	1st	6th	9th	*11.9	*6.8	*4.3	*0.5
25-5-1.0	**1st**	**3rd**	**9th**	**5.3**	***12.8**	***7.8**	***1.2**	***0.4**
25-5-1.5	1st	1st	1st	1st	*14	*7.2	*0.3	*0.1
25-6-0.5	**1st**	**8th**	***5.2**	***6.8**	***11.9**	***5.2**	***0.4**	***1.0**

Samples	WD test				FT test			
	Curing periods (days)							
	0	3	7	28	0	3	7	28
	At Standard Proctor Density							
25-6-1.0	**1st**	**4th**	***6.2**	***5.4**	***11.6**	***6.1**	***0.5**	***1.0**
25-6-1.5	1st	1st	4th	5th	*5.8	*5	*2.6	*3.3
30-4-0.5	1st	4th	6th	8th	*13.1	*10.3	*5.7	*3.6
30-4-1.0	1st	7th	8th	10th	*12.4	*10.2	*4.8	*3.1
30-4-1.5	1st	3rd	3rd	7th	*12.8	*10.4	*5.1	*3.8
30-5-0.5	**1st**	**3rd**	***9.2**	***5.5**	***10.2**	***5.5**	***1.1**	***0.7**
30-5-1.0	**1st**	**7th**	***7.3**	***4.1**	***9.7**	***5.1**	***0.8**	***0.5**
30-5-1.5	1st	1st	1st	1st	*13.1	*12.1	*10.3	*9.2
30-6-0.5	**1st**	***6.3**	***3.7**	***1.2**	***10.1**	***4.9**	***1.3**	***0.5**
30-6-1.0	**1st**	***6.6**	***3.3**	***0.8**	***9.9**	***4.5**	***0.5**	***0.4**
30-6-1.5	1st	8th	9th	11th	*7.3	*5.7	*1.5	*1.8
Samples	**WD test**				**FT test**			
	Curing periods (days)							
	0	**3**	**7**	**28**	**0**	**3**	**7**	**28**
	At Modified Proctor Density							
15-4-0.5	1st	1st	1st	2nd	*16.7	*15.1	*14.6	*10.3
15-4-1.0	1st	1st	1st	4th	*15.2	*14.9	*13.2	*9.4
15-4-1.5	1st	1st	1st	1st	*16.1	*15.4	*14.2	*9.7
15-5-0.5	1st	1st	1st	1st	*14.8	*13.5	*11.8	*9.3
15-5-1.0	1st	1st	1st	3rd	*14.5	*12.8	*10.2	*7.7
15-5-1.5	1st	1st	1st	3rd	*15.3	*14.9	*12.1	*11.2
15-6-0.5	1st	1st	2nd	3rd	*13.8	*12.8	*10.3	*8.7
15-6-1.0	1st	1st	3rd	1st	*12.9	*11.1	*9.3	*7.3
15-6-1.5	1st	1st	1st	1st	*13.2	*11.3	*9.7	*7.5
20-4-0.5	1st	1st	1st	2nd	*13.2	*12.8	*11.9	*10.7
20-4-1.0	1st	1st	1st	2nd	*12.7	*11.1	*11.3	*10.6
20-4-1.5	1st	1st	1st	1st	*12.5	*11.5	*11.5	*10.7

Samples	WD test				FT test			
	Curing periods (days)							
	0	**3**	**7**	**28**	**0**	**3**	**7**	**28**
	At Modified Proctor Density							
20-5-0.5	1st	1st	1st	3rd	*10.7	*10.5	*10.1	*8.3
20-5-1.0	1st	1st	1st	1st	*16.7	*15.3	*14.9	*3.2
20-5-1.5	1st	1st	1st	1st	*17.6	*16.1	*15.7	*5.6
20-6-0.5	1st	2nd	4th	5th	*12.4	*10.7	*9.9	*8.2
20-6-1.0	1st	3rd	6th	6th	*10.5	*10.3	*7.6	*6.3
20-6-1.5	1st	1st	2nd	2nd	*11.3	*10.5	*8.3	*7.1
25-4-0.5	1st	1st	1st	1st	*13.8	*8.6	*7.6	*3.8
25-4-1.0	1st	1st	1st	1st	*13.2	*8.0	*5.8	*1.8
25-4-1.5	1st	1st	1st	1st	*13.6	*9.3	*8.6	*3.5
25-5-0.5	1st	1st	5th	5th	*10.6	*3.4	*3.2	*0.1
25-5-1.0	**1st**	**6th**	**7th**	***8.4**	***13.1**	***6.3**	***3.1**	***1.3**
25-5-1.5	1st	1st	1st	8th	*12.9	*7.6	*1.7	*2.3
25-6-0.5	**1st**	**5th**	***7.9**	***9.6**	***9.6**	***4.8**	***2.4**	***1.5**
25-6-1.0	**1st**	**3rd**	***8.6**	***5.8**	***9.7**	***2.1**	***0.6**	***0.8**
25-6-1.5	1st	1st	*1st	4th	*1.6	*1.2	*2.7	*1.3
30-4-0.5	1st	1st	2nd	4th	*13.8	*12.3	*10.2	*8.4
30-4-1.0	1st	1st	4th	6th	*12.3	*10.4	*8.4	*7.1
30-4-1.5	1st	1st	1st	2nd	*11.6	*11.8	*9.7	*9.4
30-5-0.5	**1st**	***5.5**	***3.5**	***1.2**	***12.1**	***10.6**	***8.4**	***6.8**
30-5-1.0	**1st**	***10.4**	***6.1**	***3.5**	***11.6**	***9.7**	***7.6**	***5.5**
30-5-1.5	1st	1st	1st	2nd	*12.7	*11.1	*10.2	*9.3
30-6-0.5	**1st**	***6.2**	***5.6**	***1.2**	***10.3**	***8.6**	***6.7**	***4.3**
30-6-1.0	**1st**	***5.8**	***3.2**	***0.8**	***9.5**	***7.2**	***5.2**	***3.1**
30-6-1.5	1st	2nd	3rd	5th	*9.8	*8.1	*6.5	*3.5

Number of cycles at which samples collapsed (or) *Percentage weight loss after 12 alternate cycles

The untreated soil samples cured for different periods collapsed inthe first cycle of the WD test but passed all 12 cycles of FT test with weight loss of less than 14%. The stabilized soil samples of 25-5-1.0, 25-6-0.5, 25-6-1.0, 30-5-0.5, 30-5-1.0, 30-6-0.5 and 30-6-1.0cured for 28 days found durable under extreme weather conditions at both densities. But stabilized soil samples having Ms 1.5 failed in WD test. It is observed that the samples having GGBS of 25 and 30% and alkali solution consisting of 5 and 6% of Na_2O having Ms of 0.5 and 1.0 at both densities are found durable in both WD and FT tests which may be due to the achieved strength and density. Hereafter, the only durability passed samples which are cured for 28 days are tested for flexural, fatigue, chemical analysis and microstructure images analysis. The images of the stabilized lateritic soil under durability tests are depicted in the Figure 4.12 (a-c).

a) Lateritic soil kept in water

b) Lateritic soil kept for drying

c) Samples kept for freezing | **d) Samples kept for thawing**

Figure 4.12 Images of soil samples under durability tests

4.7 Flexural Strength Test

The failure load of the stabilized sample under two-point loading are noted down and the flexural strength is calculated using Equation (3.1). The highest flexural strength of 0.69 and 1.33MPa was achieved for a sample of 30-6-1.0 at standard and modified Proctor densities respectively. This may be due to the higher binder content (GGBS)which helpedtoattain the strength and at Ms 1.0, the equal concentration of Na_2O and SiO_2 helped in forming stable aluminosilicate hydrates. Due to the compaction effort, the samples compacted at modified Proctor density achieved more flexural strength than the samples compacted at standard Proctor density. The flexural strength of the stabilized lateritic soil at standard and modified Proctor density are depicted in Figures 4.13 and 4.14 respectively. The stabilized lateritic soil under flexural strength test is depicted in Figure 4.15.

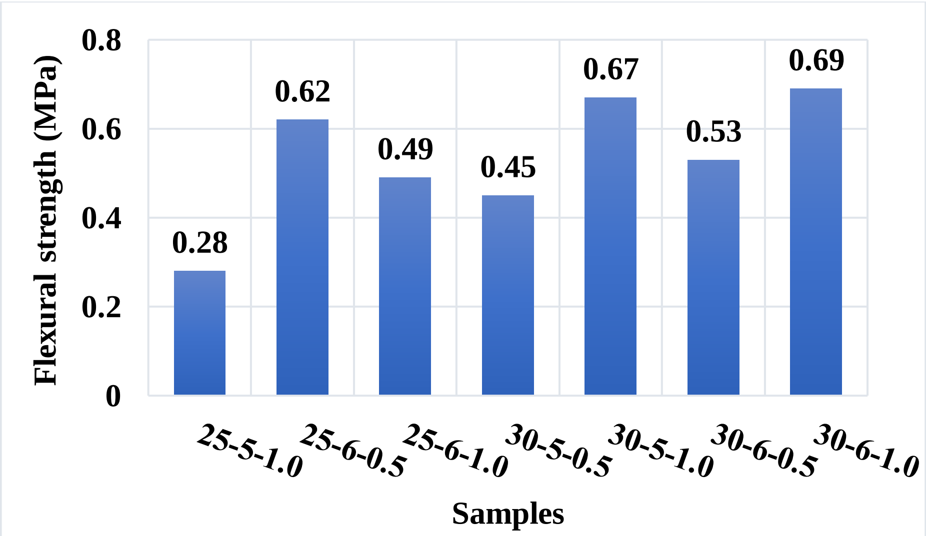


Figure 4.13 The variation of flexural strength of the stabilized lateritic soil at standard Proctor density

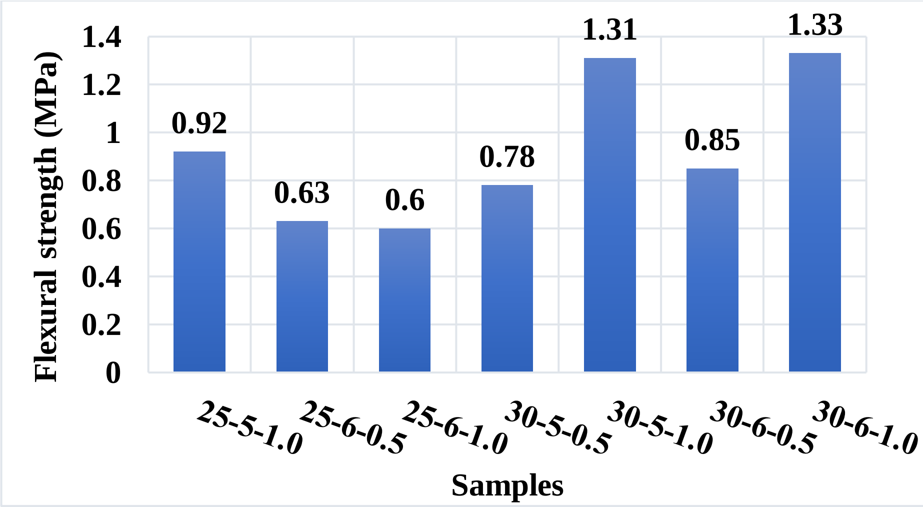


Figure 4.14 The variation of flexural strength of stabilized lateritic soil at modified Proctor density

Figure 4.15 The stabilized soil under flexural strength test

4.8 Fatigue Test

The durability passed stabilized soil samples having a dimension of UCS cured for 28 days at both densities were tested under repetitive loading conditions at 1Hz frequency. The minimum UCS of 1327 kPa at standard Proctor and 1412 kPa at modified Proctor densities were considered and 1/3rd, ½ and 2/3rd of the minimum UCS loads were applied on samples and number of repetitions were noted down. The fatigue life of stabilized lateritic soil samples at standard and modified Proctor density are depicted in Figures 4.16 and 4.17 respectively.

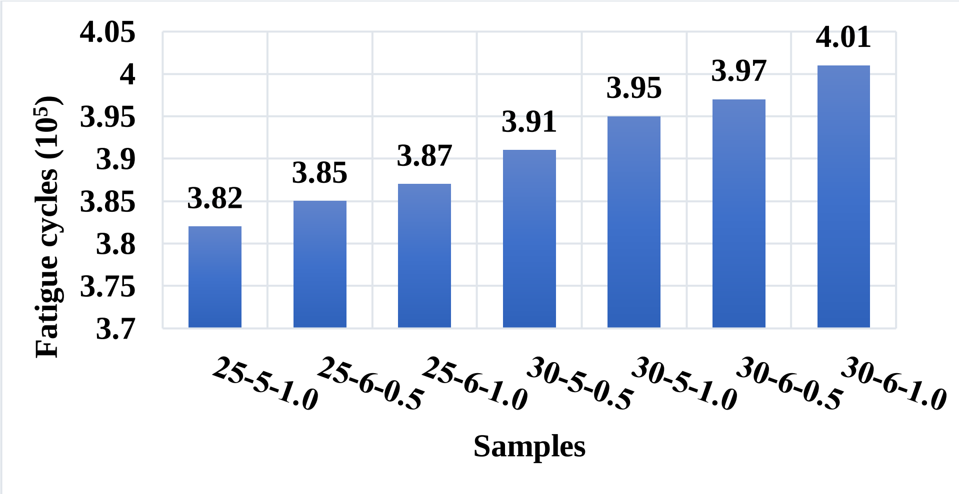


Figure 4.16 The fatigue life of stabilized lateritic soil at standard Proctor density

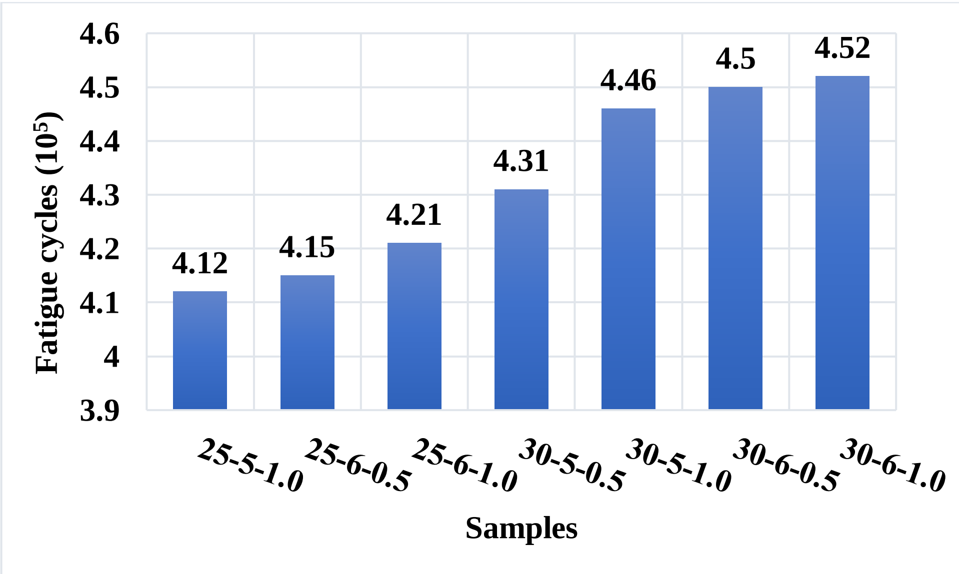


Figure 4.17 The fatigue life of stabilized lateritic soil at modified Proctor density

From the test results, it is observed that at standard and modified Proctor density the sample 30-6-1.0 is showing the fatigue life of 4.01×10^5 and 4.52×10^5 respectively at $1/3^{rd}$ of the UCS load application. It is also observed that the sample 30-6-1.0 at both densities showing better flexural strength and fatigue life is dueto high binder content (30%), optimum Ms (1.0) and high compaction effort which helps to achieve strong bonding between GGBS and alkali solution. The test set up of fatigue test is depicted in Figure 4.18.

Figure 4.18 Fatigue test set up

4.9 Chemical composition

The chemical composition of the untreatedand stabilized soil was found to know the utilization of oxides in the stabilized soil. The oxides such as SiO_2, Fe_2O_3, and Al_2O_3, CaO and MgO are found. The chemical composition of the stabilized soil is tabulated in Table 4.5. From the table, it is found that SiO_2 and Al_2O_3 are major components to help the reaction between GGBS and alkali solution which binds with soil to form an aluminosilicate structure. As the dosage of GGBS and Na_2O increases the utilization of the oxide increases to form bonding and it increases the pH level which providesthe alkaline environment for the polymerization reaction.

Table 4.5 The chemical composition of stabilized soil samples

Samples	**Proctor test**	**Oxides (%)**					**pH**	**Electrical conductivity (mili Siemens)**
		SiO_2	**Fe_2O_3**	**Al_2O_3**	**CaO**	**MgO**		
25-5-1.0	Standard Proctor density	52.4	3.5	11.7	0.2	0.04	10.39	2.88
25-6-0.5		55.3	3.5	10.4	0.2	0.03	10.4	3.22
25-6-1.0		62.8	3.4	7.8	0.1	0.01	10.52	3.64
30-5-0.5		66.8	3.6	8.3	0.18	0.03	10.45	3.65
30-5-1.0		45.3	3.8	5.6	0.12	0.02	10.57	4.01
30-6-0.5		46.8	3.7	5.3	0.13	0.03	10.52	4.27

30-6-1.0		46.9	3.6	5.1	0.12	0.02	10.59	4.5
25-5-1.0	Modified Proctor density	41.5	3.7	7.4	0.12	0.04	10.35	3.69
25-6-0.5		43.2	3.4	7.8	0.14	0.02	10.39	3.86
25-6-1.0		45.3	3.2	7.9	0.14	0.01	10.41	3.95
30-5-0.5		46.9	2.7	12.6	0.15	0.02	10.12	2.76
30-5-1.0		48.2	2.2	19.0	0.16	0.10	9.9	1.89
30-6-0.5		48.2	2.5	19.2	0.16	0.1	10.3	2.53
30-6-1.0		48.9	2.8	20.2	0.12	0.1	10.5	2.61

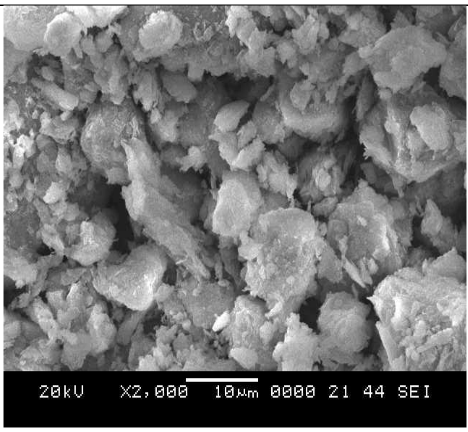
4.10 Microstructure analysis

The microstructure image of the sampleswas obtained from the electron microscopy using a Scanning Electron Microscope (SEM) technique. The samples were scanned using a high beam electron and the crystal orientation of the surface of the samples was obtained at a resolution of 2k and 10micrometers. The formation of an aluminosilicate structure is may be due to the polymerization reaction between the GGBS and alkali solution which binds soil with fewer voids. The images of the soil samples at standard and modified Proctor densities are depicted in Figures 4.19 (a-g) and 4.20 (a-g) respectively.

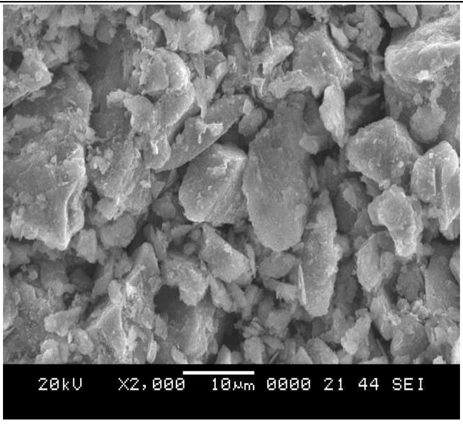
It is observed that the grey coloured, closely packed and flake-like structure represents the formation of an aluminosilicate structure. When GGBS mixed with the alkali solution, due to the polymerization reaction the aluminosilicate structure will be formed which helps to form a closely packed flake-like structure with fewer voids. As GGBS content and Na_2O dosage in alkali solution and Ms of 1.0 will help to form a compact structure helps to produce durable material. It is observed that the sample of 30-6-1.0 is giving the most compact structure compared to all other samples.

4.10.1 Standard Proctor density

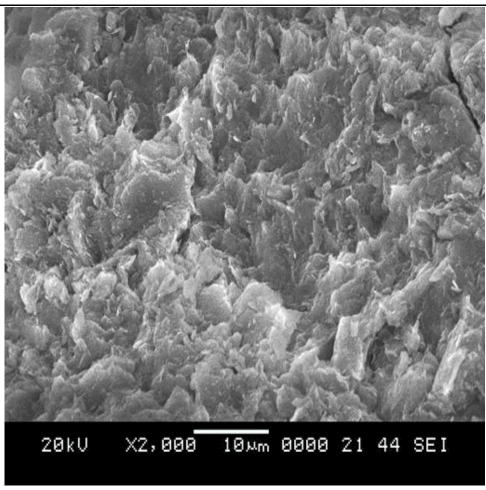
Among all durability passed stabilized samples compacted at standard Proctor density such as 25-5-1.0, 25-6-0.5, 25-6-1.0, 30-5-0.5, 30-5-1.0, 30-6-0.5 and 30-6-1.0, it can be inferred that the samples with high Na_2O content of 6% and Ms of 1.0 such as 25-6-1.0 and 30-6-1.0 are showing closely packed compact structure.



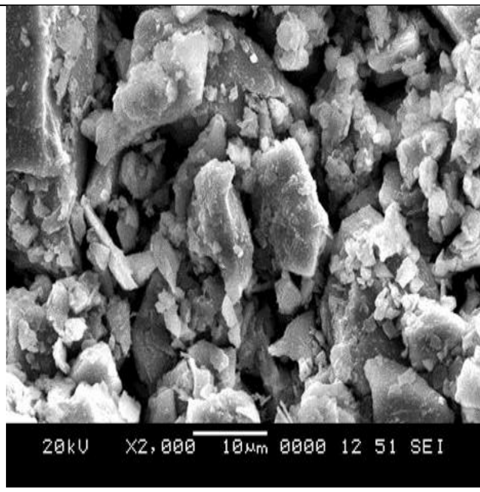
a) 25-5-1.0



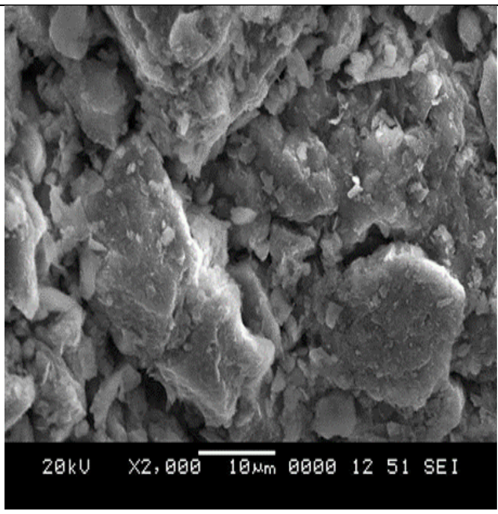
b) 25-6-0.5



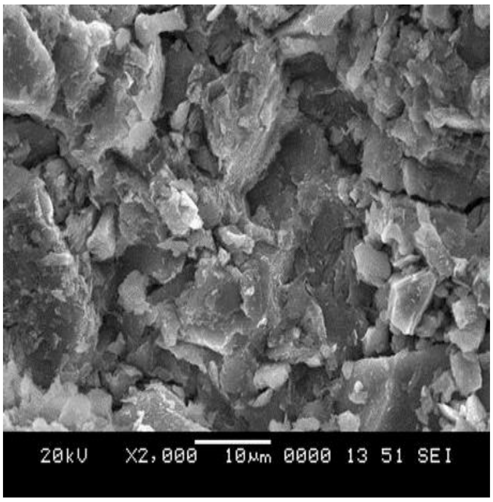
c) 25-6-1.0



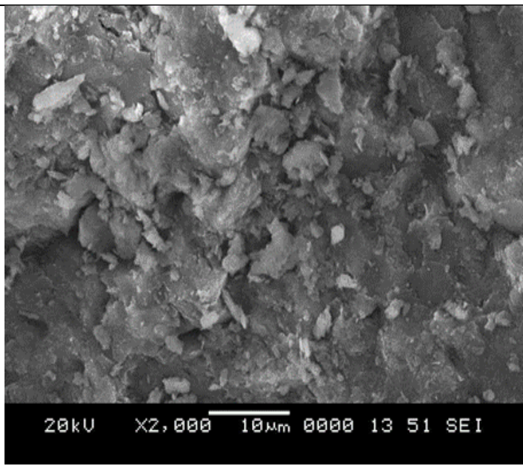
d) 30-5-0.5



e) 30-5-1.0



f) 30-6-0.5

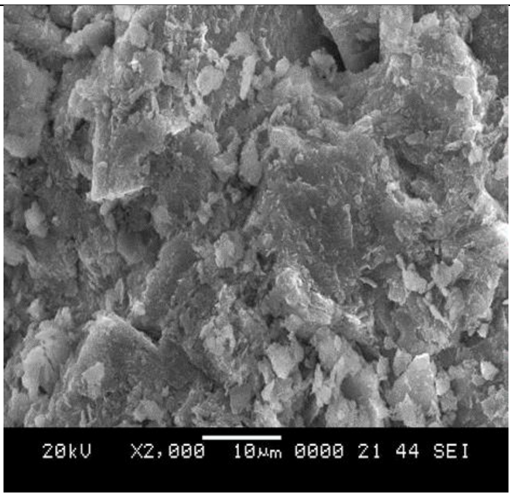


g) 30-6-1.0

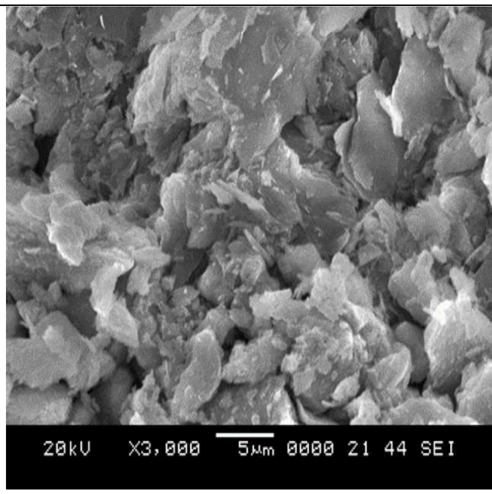
Figure 4.19 The microstructure images of the stabilized soil at standard Proctor density

4.10.2 Modified Proctor density

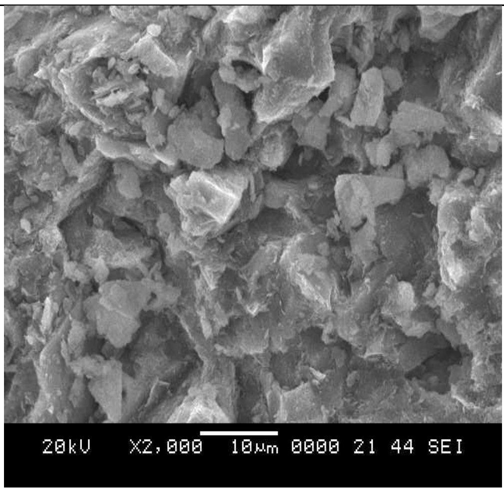
The microstructure images of the soil samples compacted at modified Proctor density are depicted in Figures 4.20 (a-g).It is observed that samples compacted at modified Proctor density are showing better strong behaviour than samples prepared at standard Proctor density due to the heavy compaction.



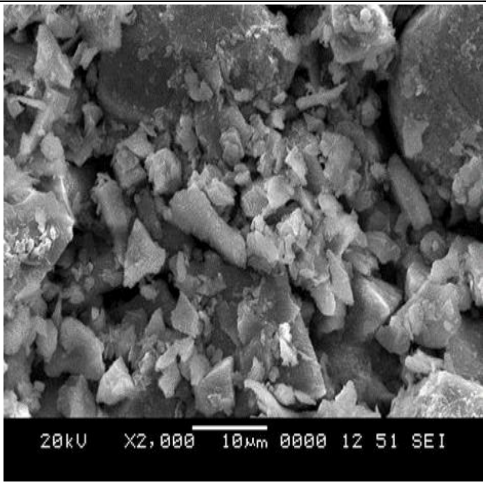
a) 25-5-1.0



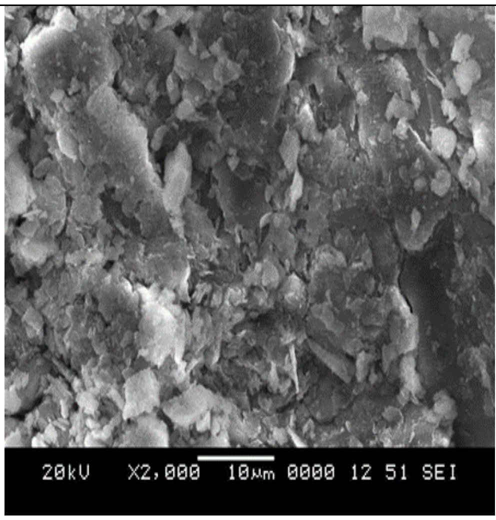
b) 25-6-0.5



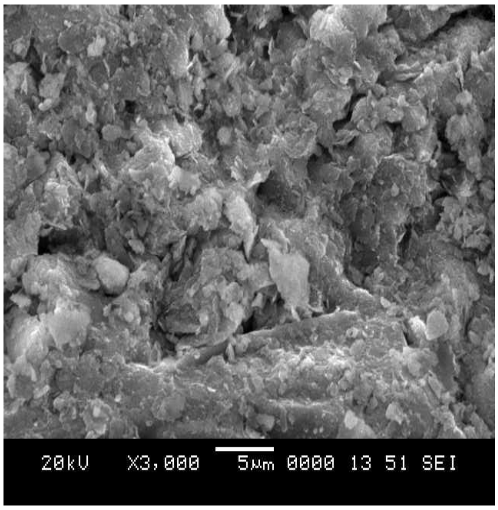
c) 25-6-1.0



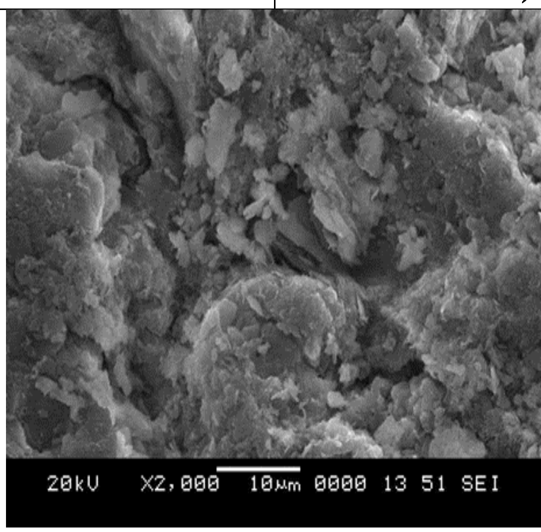
d) 30-5-0.5



e) 30-5-1.0



f) 30-6-0.5



g) 30-6-1.0

Figure 4.20 The microstructure images of the stabilized soil at modified Proctor density

The samples compacted at modified Proctor density also have shown the same trend as that of the samples compacted at standard Proctor density. The densified structure was observed from the samples having Na_2O of 6% and Ms of 1.0.

4.11 Major Findings

- The MDD of 1.72 g/cc is achieved for the stabilized soil sample of 30-6-0.5 from the standard Proctor test and 1.84 g/cc for the sample of 25-6-1.0 from the modified Proctor test.
- The highest UCS of 6341 and 9901 kPa is achieved for the sample of 30-6-1.0 after 28 days curing at standard and modified Proctor density respectively which is 14.8 and 18.6 times that of the untreated lateritic soil.
- The obtained CBR values were more than 100% hence, to ensure the strength of the stabilized soil, the durability test was conducted.
- The stabilized soil samples of 25-5-1.0, 25-6-0.5, 25-6-1.0, 30-5-0.5, 30-5-1.0, 30-6-0.5 and 30-6-1.0 cured for 28 days found durable under extreme weather conditions at both densities.
- The highest flexural strength of 0.69 and 1.33 MPa was achieved for a sample of 30-6-1.0 at standard and modified Proctor densities respectively.
- The stabilized sample of 30-6-1.0 is showing the fatigue life of 4.01×10^5 and 4.52×10^5 respectively at 1/3rd of the UCS load application at standard and modified Proctor density.
- It is observed that the sample of 30-6-1.0 is giving the most compact structure compared to all other samples.

CHAPTER 5

STABILIZATION OF BLACK COTTON SOIL

5.1 General

Most of the BC soilsare considered as the most expansive soil with high moisture susceptibility. The BC soil is featured with very low bearing capacity with high swelling and shrinkage characteristics. Due to its characteristics of high susceptibility, the soil forms the poor foundation for road construction. This chapter deals with the stabilization of BC soil to improve the engineering properties to use as a pavement material. The engineering properties of the untreated BC soil are tabulated in Table 5.1.

Table 5.1 The engineering properties of the BC soil

Sl no.	Property	BC Soil
1	Specific Gravity	2.5
2	Grain size distribution (%) a) Gravel b) Sand c) Silt d) Clay	 2 41 32 25
3	IS Soil Classification	CI
4	Consistency limits (%) a) Liquid Limit (LL) b) Plastic Limit (PL) c) Plasticity Index (PI) d) Shrinkage Limit (SL) e) Free Swelling Index (FSI)	 42 22 20 10 56
5	Proctor tests Standard Proctor a) OMC (%) b) MDD (g/cc) Modified Proctor a) OMC (%) b) MDD (g/cc)	 21.4 1.62 17.2 1.7

6	CBR Value (%) At Standard Proctor density a) Unsoaked condition b) Soaked condition At Modified Proctor density a) Unsoaked condition b) Soaked condition	 4 3 13 7
7	UCS (kPa) At Standard Proctor density At Modified Proctor density	 152 267

5.2 Atterberg limits

The consistency limits of the BCsoil are tabulated in Table 4.1 and it is classified as intermediate plastic clay (CI).The Atterberg limits of the stabilized soil couldn't be found as the stabilized soil becomes hard and stiffwhen GGBS and alkali solution were mixed together due to the exothermic reaction.

5.3 Proctor Results

The standard and modified Proctor tests were conducted on untreated and stabilized BC soil and the test results such as OMC and MDD are tabulated in Table 5.2. The highest MDD of 1.7 g/cc is obtained from the stabilized soil sample of 25-4-0.5 at standard Proctor density.

Table 5.2 The Standard Proctor test results of stabilized BC soil

Na_2O dosage (%)	15% GGBS		20% GGBS		25% GGBS		30% GGBS	
	OMC (%)	MDD (g/cc)	OMC (%)	MDD (g/cc)	OMC (%)	MDD (g/cc)	OMC (%)	MDD (g/cc)
At 0.5 Ms								
4	21.5	1.66	20.2	1.65	19.4	1.66	18.2	1.65
5	20.6	1.65	19.8	1.67	18.5	1.68	17.3	1.67
6	20.5	1.66	19.8	1.68	18.5	1.7	17.3	1.69
At 1.0 Ms								
4	20.3	1.64	19.4	1.63	18.3	1.63	17.3	1.64
5	20.2	1.65	19.4	1.65	18.4	1.66	17.2	1.66
6	20.0	1.65	17.8	1.66	17.1	1.68	17.0	1.67

At 1.5 Ms								
4	19.8	1.64	19.2	1.62	18.1	1.62	17.2	1.63
5	19.6	1.64	19.2	1.64	18.2	1.64	17.0	1.65
6	19.5	1.65	17.6	1.65	16.9	1.67	16.8	1.66

The modified Proctor test was conducted on the BC soil stabilized with GGBS and alkali solutions. The highest MDD of 1.81g/cc is achieved for the stabilized sample of 30-5-0.5 and the Proctor results are tabulated in Table 5.3.

Table 5.3 The Modified Proctor test results of stabilized BC soil

Na_2O dosage (%)	**15% GGBS**		**20% GGBS**		**25% GGBS**		**30% GGBS**	
	OMC (%)	**MDD (g/cc)**	**OMC (%)**	**MDD (g/cc)**	**OMC (%)**	**MDD (g/cc)**	**OMC (%)**	**MDD (g/cc)**
At 0.5 Ms								
4	17.2	1.74	16.0	1.76	15.2	1.78	14.0	1.79
5	16.8	1.76	15.6	1.78	14.3	1.8	13.1	1.81
6	16.5	1.67	15.6	1.68	14.3	1.7	13.1	1.72
Na_2O dosage (%)	**15% GGBS**		**20% GGBS**		**25% GGBS**		**30% GGBS**	
	OMC (%)	**MDD (g/cc)**	**OMC (%)**	**MDD (g/cc)**	**OMC (%)**	**MDD (g/cc)**	**OMC (%)**	**MDD (g/cc)**
At 1.0 Ms								
4	16.7	1.73	15.2	1.74	14.1	1.76	13.1	1.78
5	16.6	1.75	15.2	1.76	14.2	1.78	13.0	1.8
6	16.5	1.65	13.6	1.67	12.9	1.69	12.8	1.71
At 1.5 Ms								
4	16.7	1.71	15.0	1.73	13.9	1.75	13.0	1.77
5	16.8	1.76	15.0	1.75	14.0	1.77	12.8	1.76
6	16.5	1.62	13.4	1.66	12.7	1.68	12.6	1.70

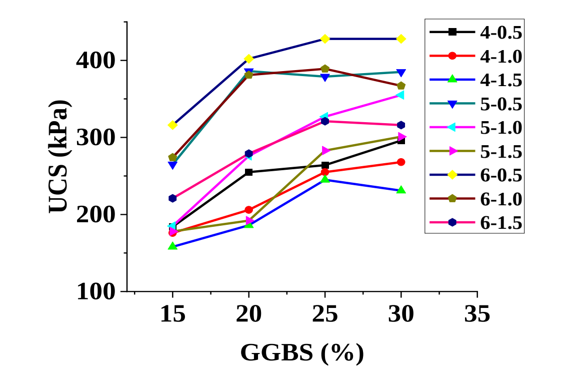
5.4 Unconfined Compressive Strength (UCS) Test

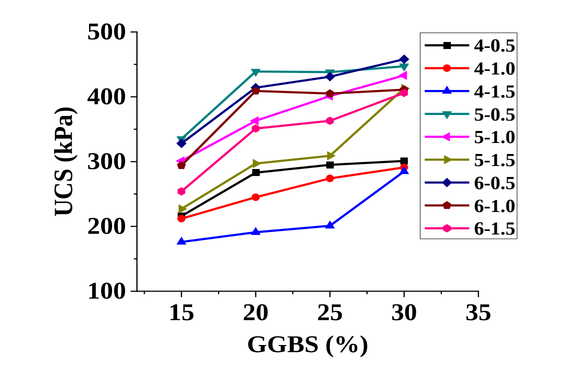
5.4.1 At standard Proctor density

The stabilized BC soil compacted at both standard and modified Proctor densities cured for different periods were tested for UCS. At standard Proctor density, the stabilized BC soil sampleof 30-5-0.5 cured for 28 days gives the highest UCS of 1407kPa which is 7 times that of the untreated BC soil.The variation of UCS with variation of GGBS, Na_2O, Ms and curing periods are depicted in Figures 5.1 (a-d), 5.2 (a-d), 5.3 (a-d) and 5.4 (a-c).

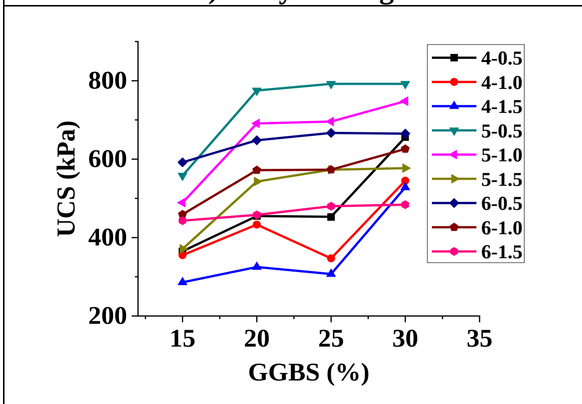
5.4.1.1 Effect of GGBS

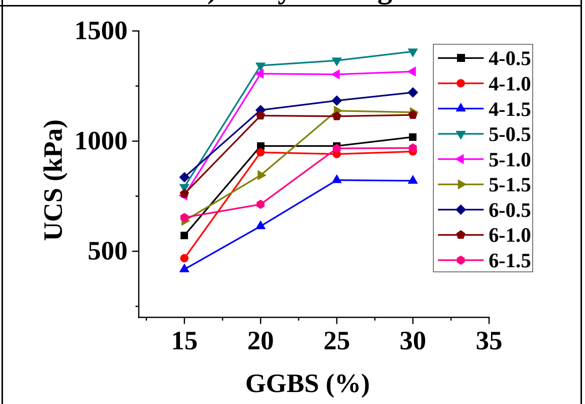
From Figure 5.1 (a-d), it is evident that, at all curing periods, the UCS increases rapidly with an increase in GGBS up to 20%. Further increase in GGBS to 30%, the UCS gradually increases. The highest UCS is observed for the sample replaced with 30% of GGBS is due to the presence of high fines in the mix.





a) 0 days curing	b) 3 days curing



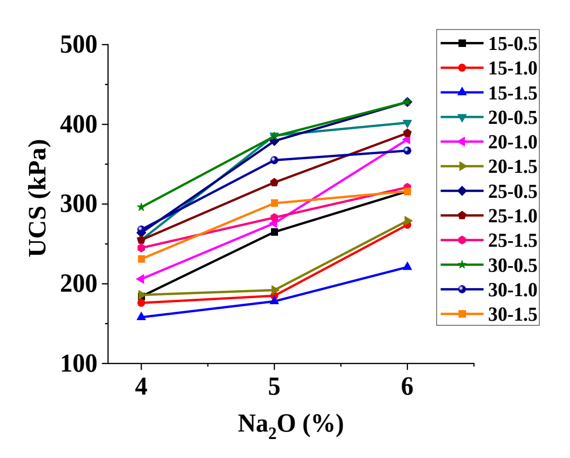


c) 7 days curing	d) 28 days curing

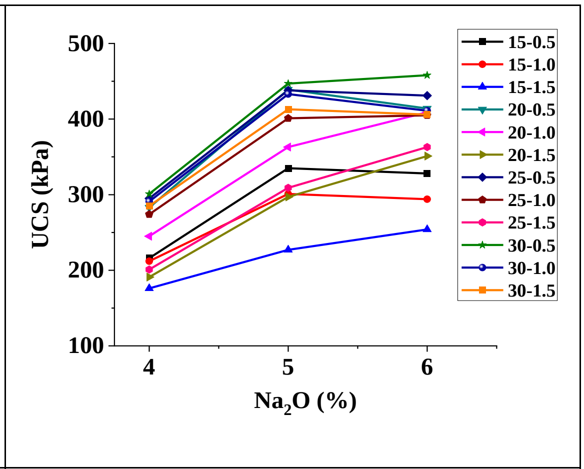
Figure 5.1 The variation of UCS for different GGBS content at standard Proctor density

5.4.1.2 Effect of Na_2O

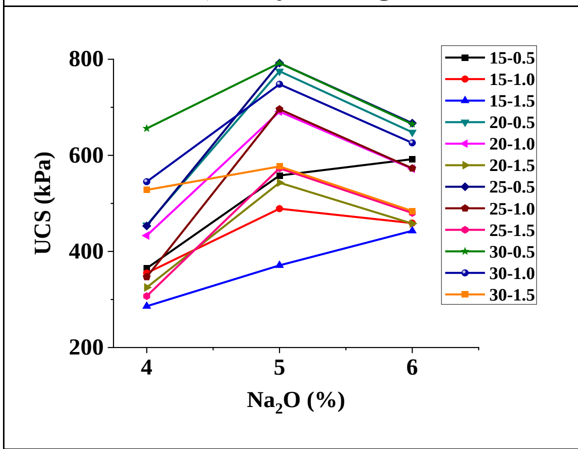
From Figure 5.2 (a-d), it is observed that, at standard Proctor density, as Na_2O dosage increases from 4 to 5%, the UCS of the stabilized soil gradually increases but further increase in Na_2O from 5 to 6% decreases the UCS. In the case of stabilized BC soil, the highest UCS is achieved at 5% of Na_2O dosage. As the highest UCS is achieved at 5% Na_2O dosage thus it can be considered as optimum dosage.



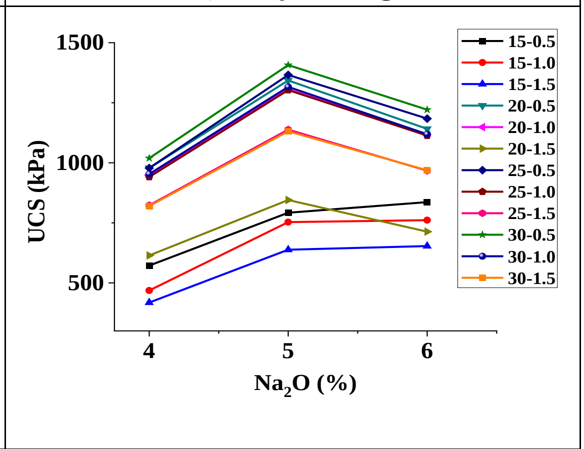
a) 0 days curing



b) 3 days curing



c) 7 days curing

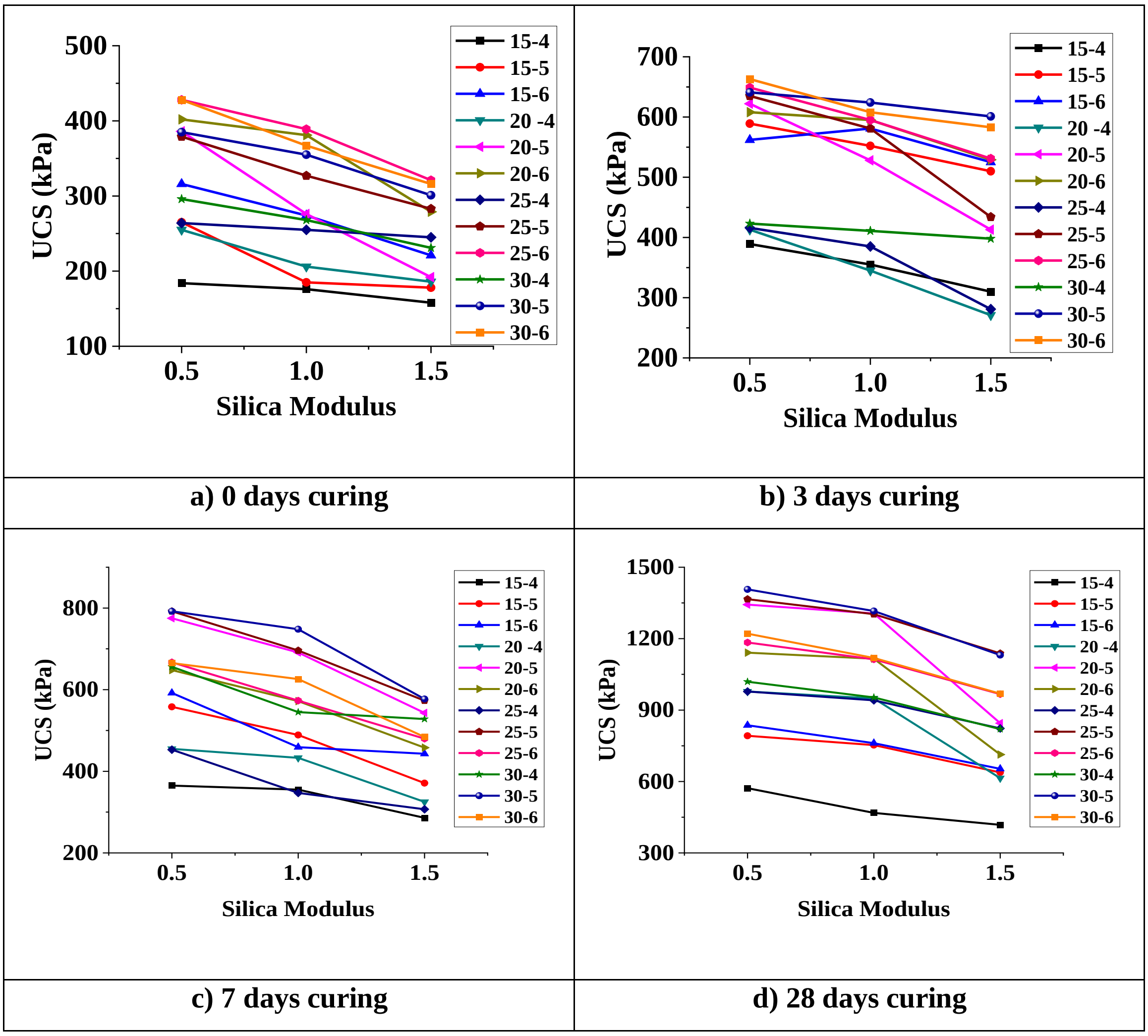


d) 28 days curing

Figure 5.2 The variation of UCS for different Na_2O dosage at standard Proctor density

5.4.1.3 Effect of Silica modulus

At standard Proctor density, when Ms of the stabilized soil increases from 0.5 to 1.5, the UCS decreases gradually and the variation is depicted in Figure 5.3 (a-d). This may be due to the increased fines in the BC soil contribute enough SiO_2 content to form an aluminosilicate structure. When Ms further increases to 1.5, the UCS decreases as the increased SiO_2 content precipitates causing detrimental effects like efflorescence and reduction in pH. Also, the high clay content demands less water to achieve strength and it can be obtained when Na_2SiO_3 amount is lesser than NaOH. Hence, the Ms of 0.5 is considered for treating BC soil with 30% of GGBS replacement and alkali solution containing 5% of Na_2O dosage.



a) 0 days curing

b) 3 days curing

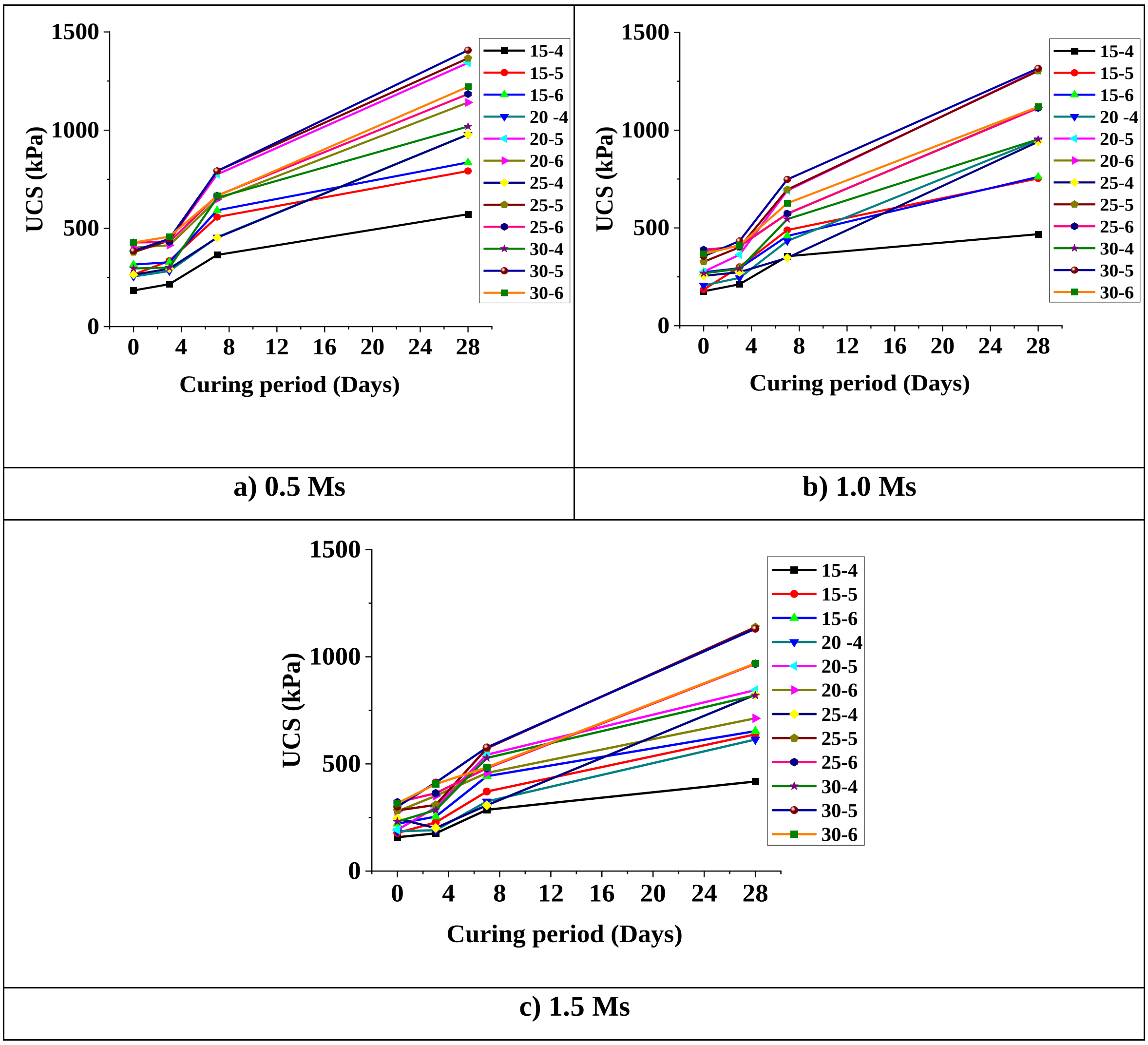
c) 7 days curing

d) 28 days curing

Figure 5.3 The variation of UCS for different Ms at standard Proctor density

5.4.1.4 Effect of curing period

From Figure 5.4 (a-c), when the curing of stabilized BC soil increases, the UCS increases gradually which is due to the polymerization process helps in developing CSH and aluminosilicate structure. Thus, the highest UCS is achieved for the stabilized BC soil cured for 28 days.



a) 0.5 Ms

b) 1.0 Ms

c) 1.5 Ms

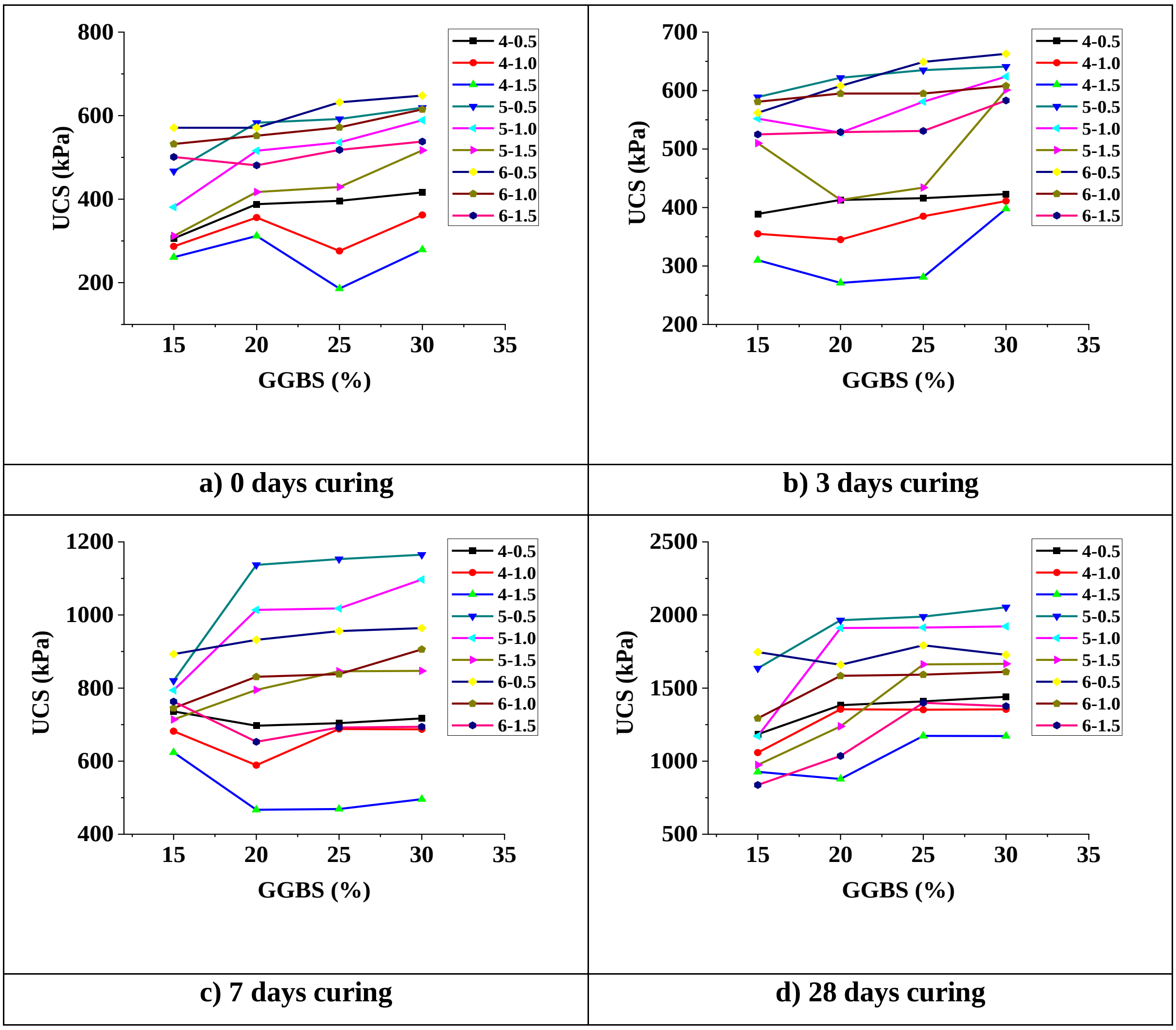
Figure 5.4 The variation of UCS for different curing period at standard Proctor density

5.4.2 At modified Proctor density

The stabilized BC soil compacted at modified Proctor density and tested for UCS after air curing for different periods. The highest UCS of 2053 kPa is observed for the sample of 30-5-0.5 cured for 28 days which is 6.5 times that of the untreated soil. The variation of UCS with the variation of GGBS, Na_2O, Ms and curing periods are depicted in Figures 5.5 (a-d), 5.6 (a-d), 5.7 (a-d) and 5.8 (a-c).

5.4.2.1 Effect of GGBS

From Figure 5.5 (a-d), it is observed that, at modified Proctor density, a gradual increase in UCS is observed when GGBS content increases to 30% in the stabilized soil. This may be due to the increased fines in the mix.



a) 0 days curing

b) 3 days curing

c) 7 days curing

d) 28 days curing

Figure 5.5 The variation of UCS for different GGBS content at modified Proctor density

5.4.2.2 Effect of Na_2O

From Figure 5.6 (a-d), it is observed that, when Na_2O dosage in the stabilized sample increase from 4 to 5%, the UCS increases, but further increase in Na_2O content in alkali solution from 5 to 6%, the UCS decreases. Utilization of 5% of Na_2O in the polymerization process increased the strength and further increase in Na_2O to 6% decreased the UCS due to the excess of Na_2O.

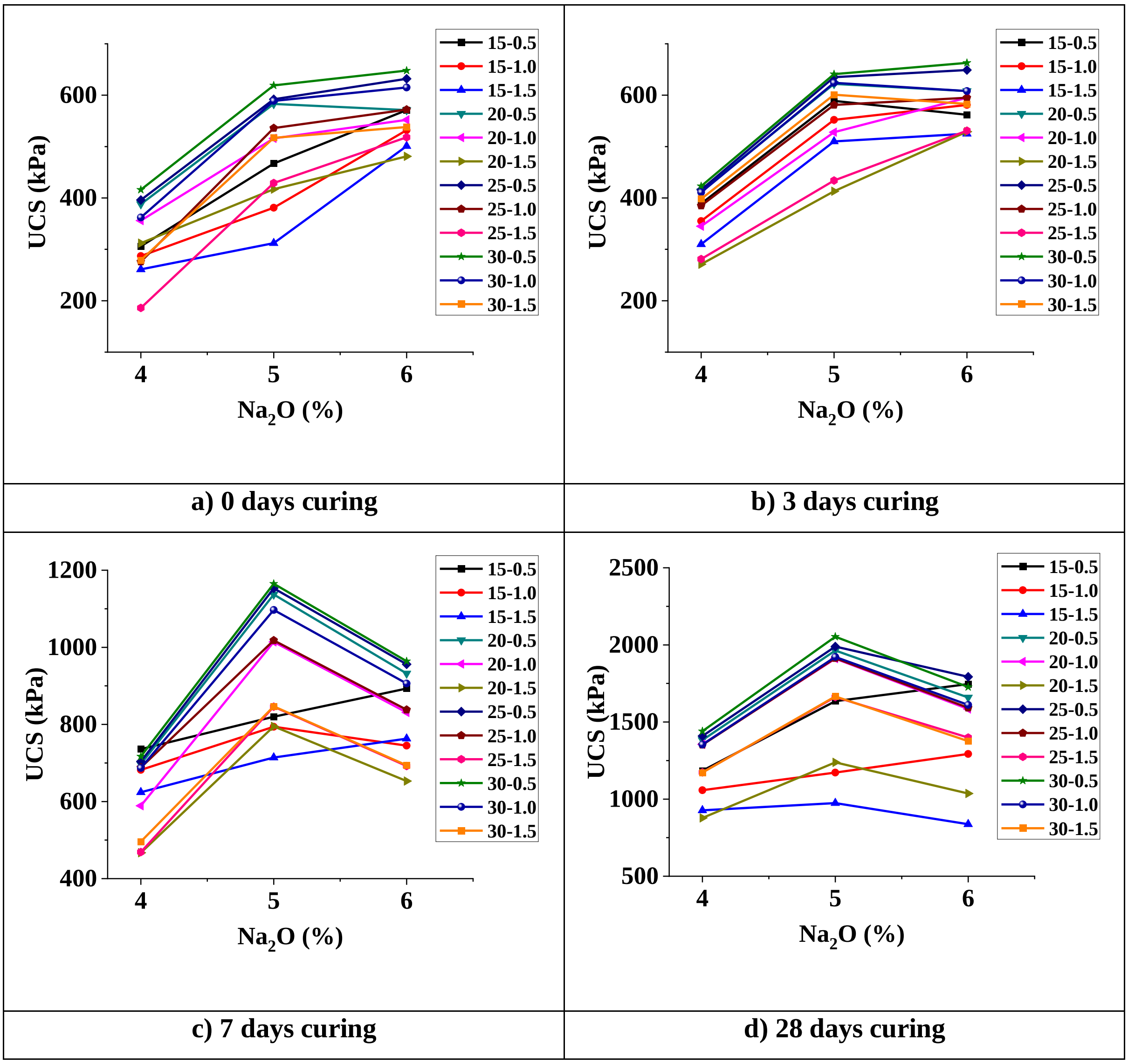
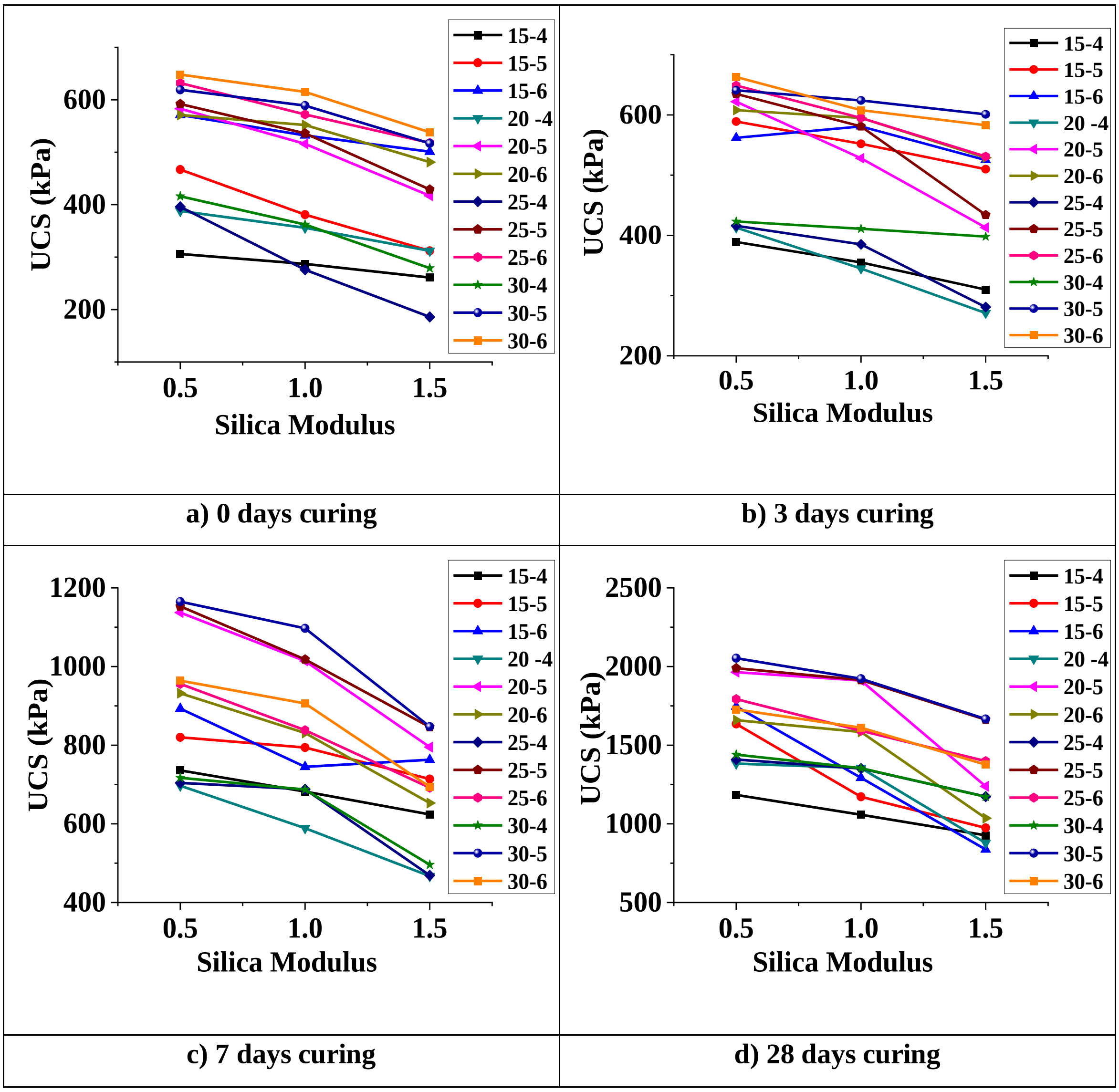


Figure 5.6 The variation of UCS for different Na_2O dosage at modified Proctor density

5.4.2.3 Effect of silica modulus

The highest UCS is observed for the stabilized BC soil having alkali solution consisting of 0.5 Ms at modified Proctor density and a further increase in Msto 1.0 and 1.5 gradually decreases the UCS and the variation is depicted in Figure 5.7 (a-d).



a) 0 days curing

b) 3 days curing

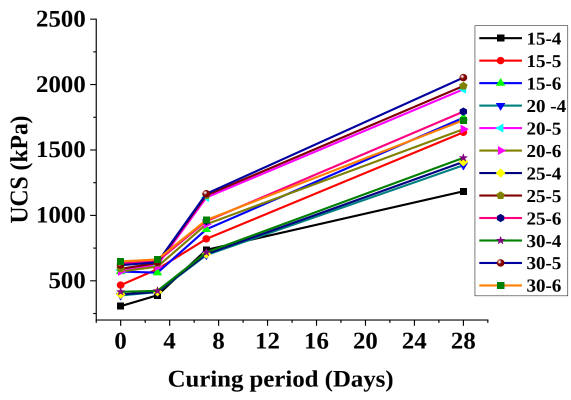
c) 7 days curing

d) 28 days curing

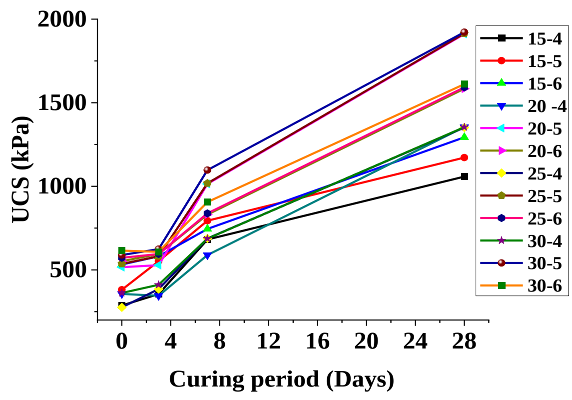
Figure 5.7 The variation of UCS for different silica modulus at modified Proctor density

5.4.2.4 Effect of curing periods

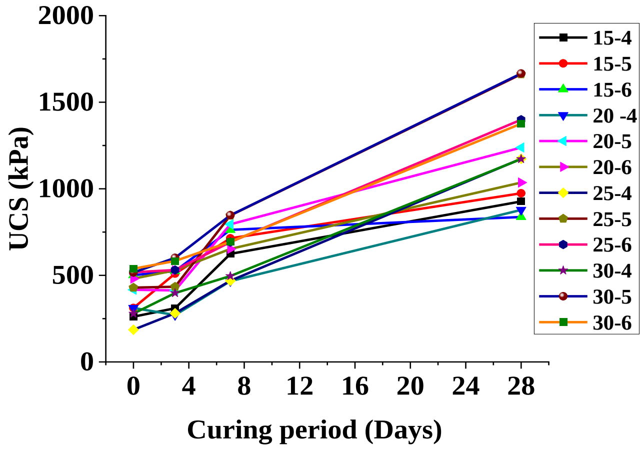
From Figure 5.8 (a-c), it is noticed that samples cured for 28 days are giving the highest UCS.The UCS increases with an increase in curing periods.



a) 0.5 Ms



b) 1.0 Ms

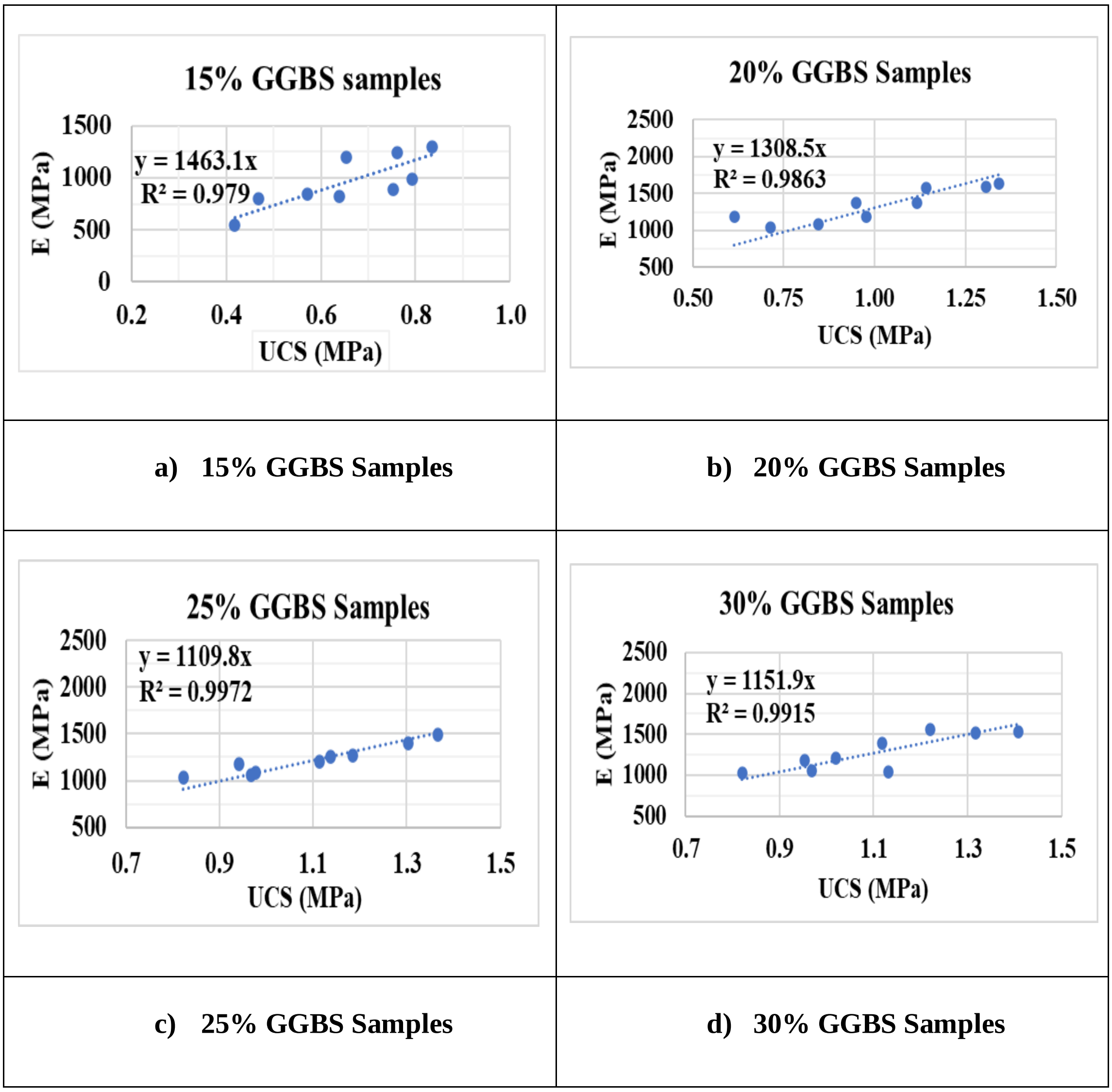


c) 1.5 Ms

Figure 5.8 The variation of UCS for different curing period at modified Proctor density

5.4.3 Relationship between UCS and Young's modulus

The relation between UCS and modulus of elasticity of stabilized BC soil cured for 28 days for standard Proctor and modified Proctor density are established and depicted in Figure 5.9 and Figure 5.10 respectively. At lower GGBS replacement the correlation is not much significant, but for the higher GGBS replacement (30%) the correlation is quite good. The R^2 values with zero intercept are more than 0.95.



a) 15% GGBS Samples

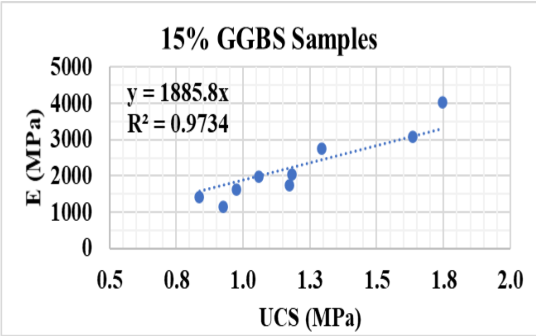
b) 20% GGBS Samples

c) 25% GGBS Samples

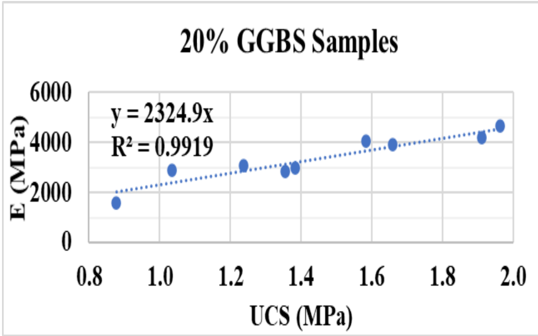
d) 30% GGBS Samples

Figure 5.9The relationship between UCS and modulus of elasticity of stabilized lateritic soil at standard Proctor density

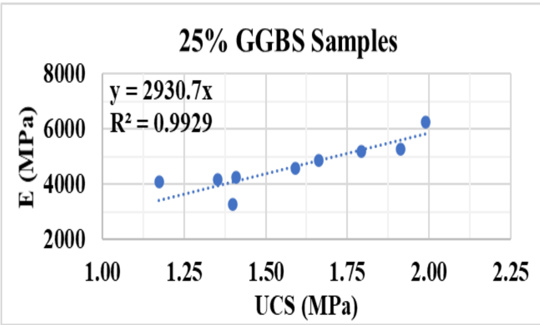
From the Figure 5.9 it is observed that there exists a correlation between laboratory UCS and Young's modulus values and the R^2 values for all graphs are found more than 0.98.



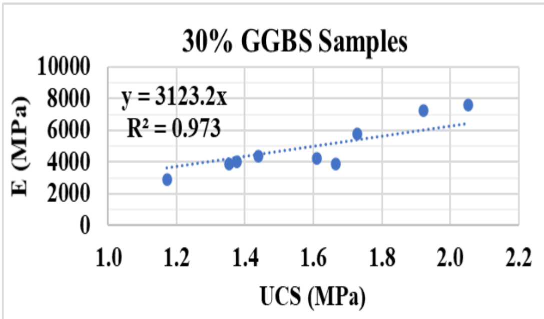
a) 15% GGBS Samples



b) 20% GGBS Samples



c) 25% GGBS Samples



d) 30% GGBS Samples

Figure 5.10 The relationship between UCS and modulus of elasticity of stabilized lateritic soil at modified Proctor density

From the Figure 5.10 it is observed that there exists a correlation between laboratory UCS and Young's modulus values and the R^2 values for all graphs are found more than 0.97.

5.5 California Bearing Ratio (CBR) Test

The CBR samples of the stabilized soil cured for 0, 3, 7 and 28 days. The CBR for both soaked and unsoaked conditions were tested. When the load was applied, the plunger could not penetrate into the soil as the soil became hard. The CBR values obtained are more than 100%. To ascertain the strength of the stabilized soil further, the durability test was conducted.

5.6 Durability

The WD and FT tests were conducted on all stabilized samples compacted at both densities and the percentage weight loss after 12 alternate cycles of WD and FT tests were noted down and the percentage weight loss after 12 cycles should not be more than 14%. The durability test results of stabilized BC soil at both densities are tabulated in Table 5.4.

The stabilized BC soil samples compacted at standard Proctor density failed in the WD test at all curing periods but passed 12 alternate cycles of FT test with percentage weight loss less than 14%. Whereas at modified Proctor density, BC soil sample 25-5-0.5, 25-6-0.5, 30-5-0.5 and 30-6-0.5 at modified Proctor density only have passed 12 cycles of WD and FT tests. All stabilized samples of BC soil could sustain the FT test. It is noticed that the stabilized BC soil having alkali solution atMs of 0.5 is more durable due to the achieved strength and density. The durable samples are represented in a bold row and the weight loss of durability passed samples are depicted in Figure 5.11. The stabilized BC soil under durability tests are depicted in Figure 5.12.

Table 5.4 The durability test results of BC soil at different curing periods

Samples	WD test				FT test			
	Curing periods (days)							
	0	3	7	28	0	3	7	28
	At Standard Proctor density							
15-4-0.5	1^{st}	1^{st}	1^{st}	1^{st}	*12.6	*10.3	*8.9	*6.2
15-4-1.0	1^{st}	1^{st}	1^{st}	1^{st}	*11.7	*9.7	*7.4	*5.8
15-4-1.5	1^{st}	1^{st}	1^{st}	1^{st}	*12.1	*9.1	*8.5	*7.8
15-5-0.5	1^{st}	1^{st}	1^{st}	1^{st}	*10.8	*8.8	*7.6	*5.4
15-5-1.0	1^{st}	1^{st}	1^{st}	1^{st}	*10.4	*8.4	*7.3	*5.1
15-5-1.5	1^{st}	1^{st}	1^{st}	1^{st}	*10.2	*7.9	*7.1	*5.9
15-6-0.5	1^{st}	1^{st}	1^{st}	1^{st}	*8.3	*7.2	*4.7	*2.4
15-6-1.0	1^{st}	1^{st}	1^{st}	1^{st}	*7.6	*5.7	*3.9	*1.7
15-6-1.5	1^{st}	1^{st}	1^{st}	1^{st}	*7.9	*6.1	*4.1	*1.1

Samples	WD test				FT test			
	Curing periods (days)							
	0	**3**	**7**	**28**	**0**	**3**	**7**	**28**
	At Standard Proctor density							
20-4-0.5	1st	1st	1st	1st	*10.4	*8.3	*7.6	*5.7
20-4-1.0	1st	1st	1st	1st	*9.1	*7.6	*6.3	*4.5
20-4-1.5	1st	1st	1st	1st	*9.9	*8.1	*7.3	*5.2
20-5-0.5	1st	2nd	1st	3rd	*9.6	*7.4	*5.7	*4.3
20-5-1.0	1st	1st	1st	3rd	*9.3	*7.5	*6.2	*4.6
20-5-1.5	1st	1st	1st	1st	*8.9	*7.6	*6.4	*4.7
20-6-0.5	1st	1st	1st	3rd	*10.3	*8.4	*7.9	*6.3
20-6-1.0	1st	1st	1st	1st	*9.2	*8.6	*8.0	*6.6
20-6-1.5	1st	1st	1st	1st	*9.7	*8.8	*8.3	*6.8
25-4-0.5	1st	1st	1st	1st	*9.3	*6.4	*6.1	*5.8
25-4-1.0	1st	1st	1st	1st	*8.2	*5.3	*5.6	*4.9
25-4-1.5	1st	1st	1st	1st	*9.1	*5.7	*5.9	*4.7
25-5-0.5	1st	4th	4th	9th	*8.8	*5.7	*5.7	*4.1
25-5-1.0	1st	3rd	3rd	7th	*8.5	*6.0	*5.8	*4.7
25-5-1.5	1st	1st	3rd	3rd	*8.2	*6.1	*5.9	*4.7
25-6-0.5	1st	4th	5th	6th	*8.4	*7.8	*7.1	*6.4
25-6-1.0	1st	2nd	2nd	5th	*8.1	*7.9	*7.9	*6.4
25-6-1.5	1st	1st	1st	2nd	*8.2	*7.9	*8.0	*6.9
30-4-0.5	1st	1st	2nd	3rd	*8.2	*6.1	*5.9	*4.6
30-4-1.0	1st	1st	3rd	5th	*7.7	*5.9	*5.7	*4.4
30-4-1.5	1st	1st	1st	2nd	*7.9	*6.0	*5.6	*4.8
30-5-0.5	1st	7th	7th	11th	*6.3	*5.6	*5.0	*3.9
30-5-1.0	1st	3rd	5th	9th	*6.1	*5.9	*5.7	*4.5
30-5-1.5	1st	1st	2nd	4th	*5.9	*6.2	*5.7	*4.7
30-6-0.5	1st	1st	7th	8th	*7.2	*7.6	*7.0	*6.2
30-6-1.0	1st	1st	5th	4th	*6.9	*7.7	*7.1	*6.3
30-6-1.5	1st	1st	1st	1st	*7.0	*7.8	*7.5	*6.7

Samples	WD test				FT test			
	Curing periods (days)							
	0	**3**	**7**	**28**	**0**	**3**	**7**	**28**
	At Modified Proctor density							
15-4-0.5	1st	1st	1st	1st	*13.7	*12.6	*10.4	*9.5
15-4-1.0	1st	1st	1st	1st	*12.1	*12.1	*9.3	*8.3
15-4-1.5	1st	1st	1st	1st	*13.1	*12.6	*10.2	*9.1
15-5-0.5	1st	1st	1st	1st	*10.5	*9.3	*8.6	*7.6
15-5-1.0	1st	1st	1st	1st	*10.2	*9.1	*8.4	*7.3
15-5-1.5	1st	1st	1st	1st	*9.5	*8.5	*8.1	*6.8
15-6-0.5	1st	1st	1st	1st	8.6	*7.9	*6.4	*5.8
15-6-1.0	1st	1st	1st	1st	7.8	*7.7	*5.5	*4.3
15-6-1.5	1st	1st	1st	1st	8.1	*7.8	*5.1	*4.1
20-4-0.5	1st	1st	1st	1st	10.3	*9.8	*5.8	*4.9
20-4-1.0	1st	1st	1st	1st	9.8	*9.2	*5.4	*4.5
20-4-1.5	1st	1st	1st	1st	10.1	*9.5	*5.7	*4.7
20-5-0.5	1st	3rd	1st	5th	*8.5	*7.4	*4.4	*4.1
20-5-1.0	1st	1st	1st	3rd	*8.2	*7.5	*4.7	*4.7
20-5-1.5	1st	1st	1st	2nd	*7.9	*7.6	*5.0	*4.7
20-6-0.5	1st	1st	2nd	3rd	*8.7	*8.6	*6.7	*5.0
20-6-1.0	1st	1st	4th	5th	*8.5	*8.8	*6.9	*5.1
20-6-1.5	1st	1st	1st	1st	*8.6	*8.9	*7.3	*5.3
25-4-0.5	1st	1st	1st	2nd	*7.6	*6.9	*5.9	*4.8
25-4-1.0	1st	1st	1st	4th	*6.9	*6.1	*5.2	*4.1
25-4-1.5	1st	1st	1st	1st	*7.1	*6.5	*5.7	*4.4
25-5-0.5	**1st**	**9th**	**11th**	**6.2**	***5.8**	***4.6**	***4.3**	***2.8**
25-5-1.0	1st	9th	9th	9th	*5.1	*5	*4.4	*3.3
25-5-1.5	1st	3rd	3rd	7th	*4.9	*5.1	*4.8	*3.5
25-6-0.5	**1st**	**5th**	**7th**	**5.8**	***6.3**	***7.0**	***6.7**	***5.1**

Samples	WD test				FT test			
	Curing periods (days)							
	0	**3**	**7**	**28**	**0**	**3**	**7**	**28**
	At Modified Proctor density							
25-6-1.0	1st	7th	8th	10th	*5.4	*7.01	*6.8	*5.2
25-6-1.5	1st	4th	5th	3rd	*6.1	*7.1	*7.1	*5.7
30-4-0.5	1st	1st	3rd	5th	*6.8	*5.4	*5.8	*4.7
30-4-1.0	1st	1st	5th	7th	*5.6	*4.2	*5.1	*4.2
30-4-1.5	1st	1st	2nd	2nd	*6.1	*5.1	*5.5	*4.5
30-5-0.5	**1st**	**8th**	**10th**	**5.3**	***5.6**	***3.6**	***4.2**	***3.0**
30-5-1.0	1st	4th	6th	11th	*4.9	*3.8	*4.1	*3.0
30-5-1.5	1st	1st	2nd	5th	*4.5	*4.0	*4.1	*3.5
30-6-0.5	**1st**	**3rd**	**6th**	***7.2**	***5.8**	***6.6**	***5.9**	***5.0**
30-6-1.0	1st	2nd	2nd	8th	*4.4	*6.9	*6.0	*5.3
30-6-1.5	1st	1st	1st	1st	*5.1	*6.9	*6.9	*5.5

Number of cycles at which samples collapsed (or) *Percentage weight loss after 12 alternate cycles

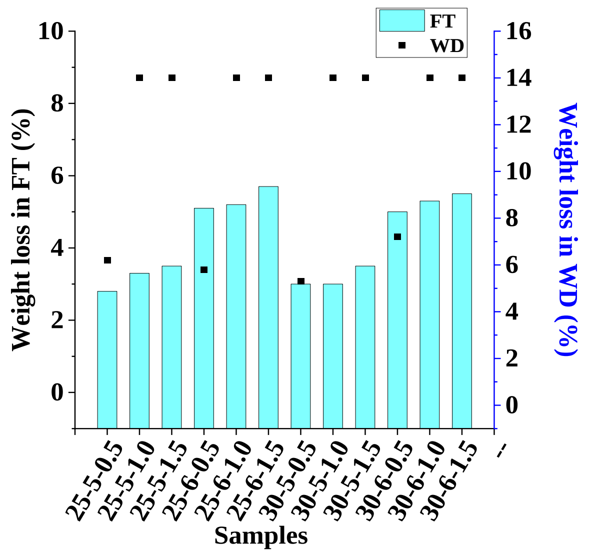


Figure 5.11 The percentage weight loss of stabilized BC soil during durability test

a) BC soil kept in water	b) BC soil kept for drying

c) Samples kept for thawing

Figure 5.12 Images of BC soil samples under durability

5.7Flexural Strength Test

The 28 days cured stabilizedBC soil samples were tested under two-point loading and the load at failure was noted down. The flexural strength of the stabilized soil is calculated using Equation (3.1). The variation of flexural strength of the stabilized BC soil is depicted in Figure 5.13.From the results, it is found that the sample of 30-5-0.5

is giving the highest flexural strength of 0.98MPa and it is also noticed that in the case of BC soil the Na_2O dosage of 5% plays a significant role in achieving maximum flexural strength.

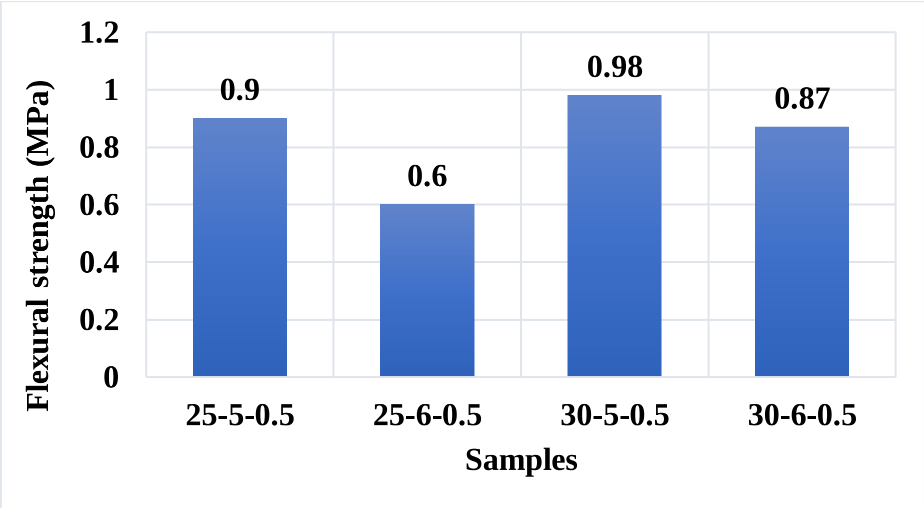


Figure 5.13 The flexural strength of the stabilized BC soil

5.8 Fatigue test

The cylindrical samples of stabilized soil were cured for 28 days and tested under repetitive loading at a frequency of 1Hz. The minimum UCS of durable stabilized samples at modified Proctor density is 1727 kPa. The 1/3rd, ½, and 2/3rd of the minimum UCS are applied to the samples. The fatigue life of durability passed stabilized BC soil samples is depicted in Figure 5.14. From the results, it is noticed that the sample of 30-5-0.5 is sustaining 1.37×10^5 repetitions at modified Proctor density at 1/3rd of the minimum UCS.

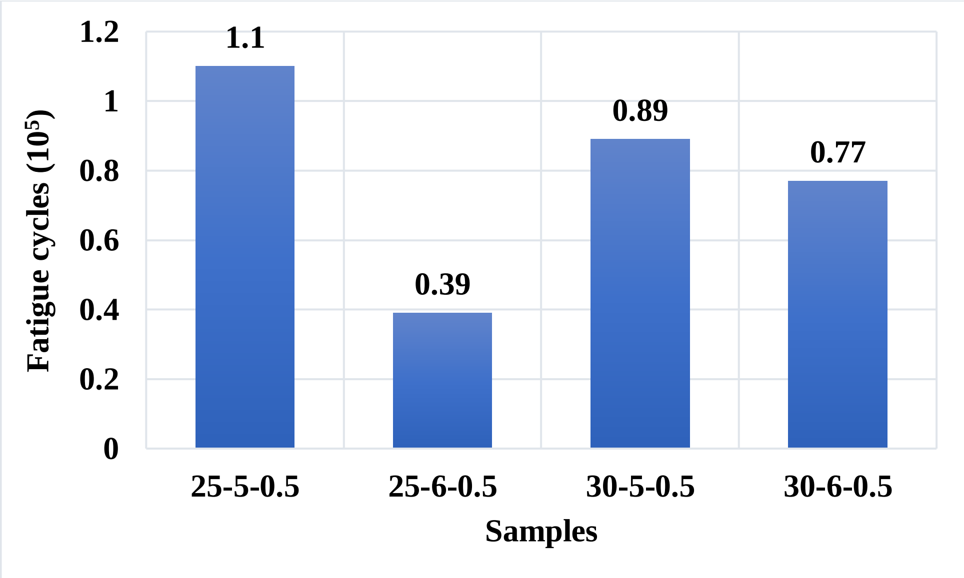


Figure 5.14 The fatigue life of the stabilized BC soil

5.9Chemical Analysis

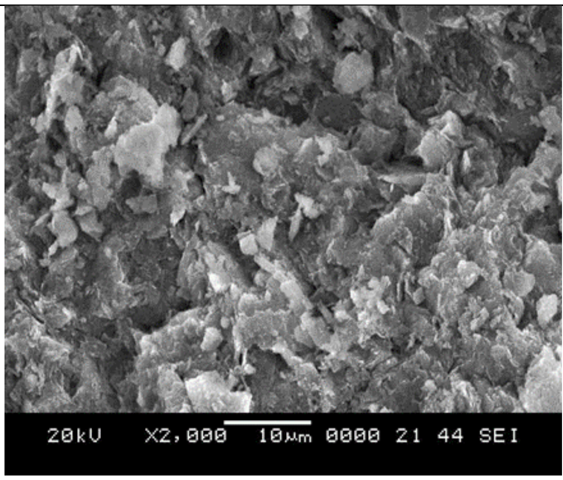
The chemical composition of the stabilized BC soil consists of SiO_2, Fe_2O_3, Al_2O_3, CaO, and MgO and their values are tabulated in Table 5.5.From the results, it is found that the utilization of SiO_2 and Al_2O_3 is more as these oxides help to form aluminosilicate structures which help in gaining strength. It is also observed that as GGBS and Na_2O dosage in the stabilized soil increases, the pH level increases which provides the alkaline environment for the polymerization reaction to happen.

Table 5.5 The chemical composition of stabilized BC soil at modified Proctor density

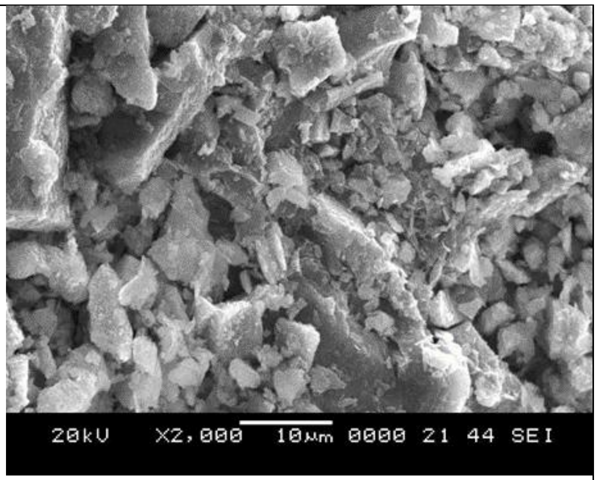
Samples	Oxides (%)					pH	Electrical conductivity (Mili Siemens)
	SiO_2	Fe_2O_3	Al_2O_3	CaO	MgO		
25-5-0.5	40.3	3.3	7.8	0.15	0.02	10.51	2.85
25-6-0.5	41.2	3.4	9.1	0.15	0.02	10.67	2.87
30-5-0.5	42.5	2.9	11.6	0.16	0.01	10.18	2.5
30-6-0.5	43.2	3.4	12.3	0.18	0.02	10.35	2.76

5.10Microstructure analysis

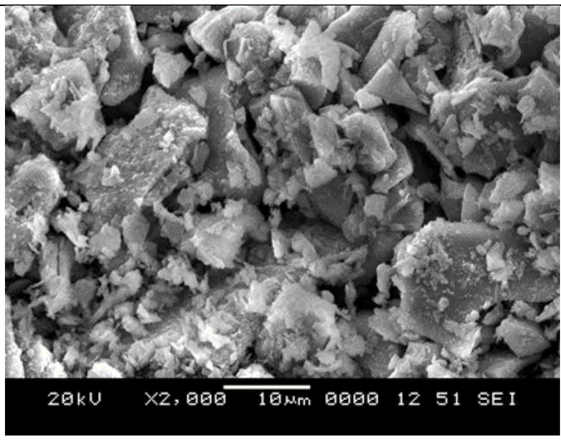
The microstructure images of the stabilized BC soil are obtained from the SEM technique at a resolution of 2K and 10micrometres wavelength. Comparing the images of the stabilized BC soil samples such as 25-5-0.5, 25-6-0.5, 30-5-0.5 and 30-6-0.5, it is observed that the samples 25-5-0.5 and 30-5-0.5 are showing closely packed compact structure compared to the stabilized soil with 6% Na_2O dosage in alkali solution. The microstructure images of the stabilized BC soil are depicted in Figure 5.15 (a-d).



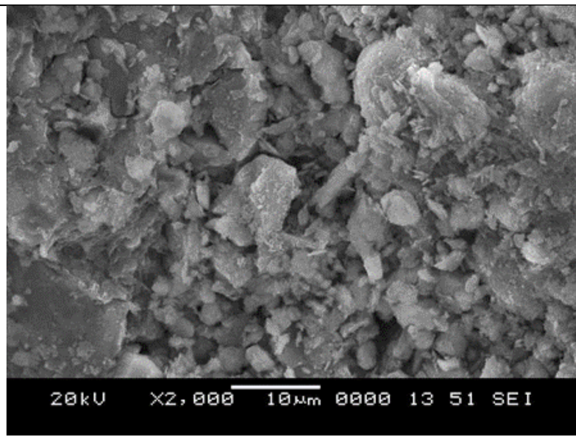
(a) 25-5-0.5



(b) 25-6-0.5



(c) 30-5-0.5



(d) 30-6-0.5

Figure 5.15 The microstructure images of the durability passed stabilized BC soil

5.11 Major Findings

- The highest MDD of 1.7 g/cc from the stabilized soil sample of 25-4-0.5 at standard Proctor density and 1.81g/cc from the stabilized soil sample of 25-4-0.5 at modified Proctor density.
- The maximum UCS of 1407 and 2053 kPa is achieved for the sample of 30-5-0.5 cured for 28 days at standard and modified Proctor densities respectively which is 7 and 6.5 times that of the untreated soil.
- The CBR of the stabilized soil is found to be more than 100%.
- The stabilized soil samples of 25-5-0.5, 25-6-0.5, 30-5-0.5 and 30-6-0.5 at only modified Proctor density passed the durability test with weight loss less than 14% after 12 alternate cycles.
- The highest flexural strength of 0.98 MPa was achieved from the sample of 30-5-0.5 at modified Proctor density.
- The sample of 30-5-0.5 is sustaining $1.37\text{x}10^5$ repetitions at modified Proctor densityat $1/3^{rd}$ of the minimum UCS.
- The samples of 25-5-0.5 and 30-5-0.5 are showing a closely packed compact structurecompared to the stabilized soil with a 6% Na_2O dosage in the alkali solution.

CHAPTER 6

PAVEMENT ANALYSIS AND DESIGN

6.1 General

Pavement design is one of the major components in road construction. The existence of pavement design is there from 1920 where the design was made purely based on the experience gained by engineers using the knowledge of soil mechanics and vast basic data. The same thickness was used for years for all types of roads. Later as years passed, many methods were derived by different agencies and various parameters mainly, traffic and loading, environment, materials and failure criteria were considered. The designed pavement is analyzed for displacement, tensile strain, compressive strain, fatigue, rutting parameters.

6.2 Flexible Pavement Design

The flexible pavement design suggested by providing the sub-base and base course above the subgrade to reduce its stress and a thin asphalt layer is provided above the base course as a wearing course. However, in current days, the design of flexible pavement and its construction is made considering the heavy wheel loads, higher traffic levels and mode of distress. The distresses or deformations such as rutting, cracking or shoving cause discomfort to the users. There are four different categories to design the flexible pavement such as,

- Empirical method
- Limiting shear failure method
- Limiting deflection method
- Mechanistic-empirical method.

6.3 Pavement Analysis

Earlier in 1885, Boussinesq analyzed the whole pavement as a homogeneous mass where the concentrated wheel load will be applied on an indefinite area and depth. The concentrated wheel load develops stresses, strains and deflections which will be integrated at the circular contact area of the tire. In 1943, Burmister proposed the two-layer system where the modulus ratio between pavement and subgrade is close to unity and later in 1945, the two-layer system theory extended to three-layer system theory. The analysis of pavement with many layers can be done with the advent of computer programming. The three-layer system and the stresses at the interfaces are depicted in Figure 6.1.

The following are the assumptions to be satisfied while considering the multi-layer system concept.

1. Each layer of the pavement is considered as a homogeneous, isotropic and linearly elastic.

2. Each layer has finite thickness with infinite areal extension and the lowest layer has infinite thickness.

3. The materials in each layer are considered weightless and a uniform pressure is applied on the surface through circular area.

4. The continuity conditions should be followed at all layer interfaces.

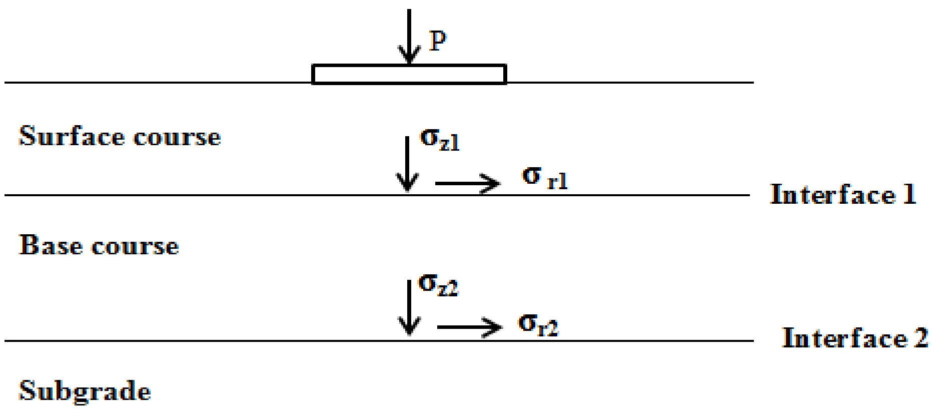


Figure 6.1 The three-layer system of flexible pavements and stresses

Where, σ_{z1}- Vertical stress at interface 1

σ_{z2}- Vertical stress at interface 2

σ_{r1}- Horizontal stress at the bottom of layer 1

σ_{r2}- Horizontal stress at the bottom of layer 2

σ_{r3}- Horizontal stress at the top of layer 3

The main function of pavement is to reduce the vertical stresses due to the wheel load applied on the pavement. The vertical stress beyond the allowable stress causes pavement deformation hence, the combined effect of stress and strength of the subgrade soil need to be analyzed using the vertical compressive strain criterion. The tensile strain at the bottom of the top layer and the compressive strain at the top of the subgrade are considered as the critical failure points and the strains are represented as ε_t and ε_z respectively. Figure 6.2 depicts the critical failure points and strains.

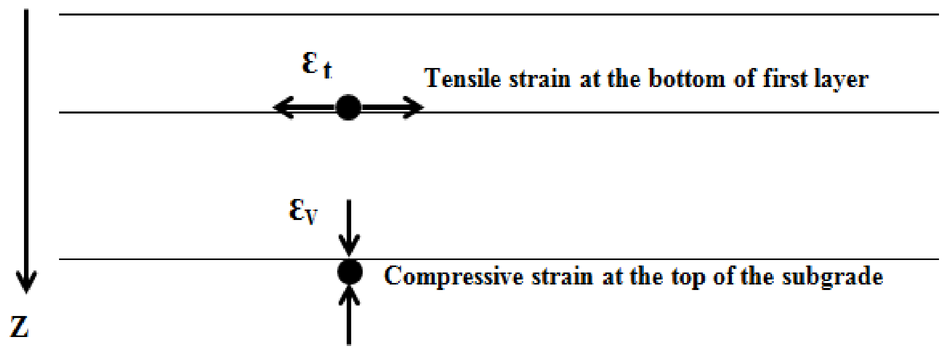


Figure 6.2 The Failure modes and critical strains for flexible pavement

The stresses and strains developed in low and high volume roads are analyzed using the IITPAVE software. The flexible pavement design considers the traffic in terms of a cumulative number of standard axles to be carried by the pavement during the design life. The road which carries less than 1 msa is referred to as low volume pavement and is designed as per IRC: SP: 72-2015 whereas the road carries above 1 msa are referred to as high volume pavements and are designed as per IRC: 37-2018.

6.4 Design Criteria

For the present study, the low volume pavement design is recommended as per IRC: SP:72-2015. The traffic in terms of Cumulative Equivalent Single Axle Load (ESAL) is divided into 9 categories (T1 to T9) and the traffic categories with ESAL are tabulated in Table 6.1. Earlier, low volume roads were considered when the cumulative number of standard axles were less than 1 msa. Recently, some of the low volume roads are carrying more than 1 msa and less than 2 msa as a feeder road to SH or NH. In special cases, these two values (1 and 2 msa) are added in the low volume code IRC: Sp:72-2015.

Table 6.1The traffic categories considered for the low volume pavement design

Traffic Category	ESAL Applications
T1	10,000 to 30,000
T2	30,000 to 60,000
T3	60,000 to 100,000
T4	100,000 to 200,000
T5	200,000 to 300,000
T6	300,000 to 600,000
T7	600,000 to 1,000,000
T8	1,000,000 to 1,500,000
T9	1,500,000 to 2,000,000

6.4.1 The Subgrade

The soaked CBR value of the soil at standard Proctor density is considered for the construction of subgrade of low volume pavements. The CBR of the subgrade soil is classified into five classes (S1 to S5) based on the quality of the soil and is tabulated in Table 6.2. The lateritic and BC soils considered in the present study comes under the S2 class of subgrade.

Table 6.2 The classification of the subgrade soil based on the quality

Quality of subgrade soil	Class of subgrade	Range of CBR (%)
Very poor	S1	2
Poor	S2	3-4
Fair	S3	5-6
Good	S4	7-9
Very good	S5	10-15

6.4.2 Sub-base Course

For the construction of granular sub-base (GSB), materials such as moorum, gravel, crushed stone, slag, brick and natural sand are used. To replace the stabilized soil by cementitious material (cement-treated soil) as a sub-base layer, the 7-day UCS of the sub-base should not be less than 1.7 MPa and the thickness of the sub-base course should not be less than 100 mm.

6.4.3 Base Course

For the CBR class S2 (CBR= 3 to 4), the gravel base course is recommended for the traffic up to 100,000 ESAL repetitions whereas, the conventional Water Bound Macadam (WBM), Wet Mix Macadam (WMM) or Crusher Run Macadam is recommended if the traffic is more than 100,000 ESAL. Also, the soil-cement or crushed stone material is recommended along with the black-topped road surface. To use the cement-treated soil as a base course, the mix should attain the minimum laboratory 7-day UCS of 3 MPa and the thickness should not be less than 100 mm.

6.4.4 Bituminous surfacing

The bituminous surfacing is recommended for the design traffic more than 60,000 ESAL in case of pavement with granular base and sub-bases. In the case of Cement Treated Base (CTB) and Cement Treated Sub-Bases (CTSB), the surface dressing is recommended for the traffic category of T1 to T5 and the surface dressing can be replaced with the 20 mm of premix carpet in case of higher traffic from T5 to T9.

6.5 Analysis of pavements using IITPAVE

IITPAVE software is developed by IIT Kharagpur, India. For the analysis of pavement, the materials are considered as elastic and isotropic. The single vertical wheel load is distributing over circular contact area on the surface of the pavement. The stresses, strains and deflections obtained due to the applied wheel load at different locations on the pavement will be computed using the software. The number of layers, the elastic modulus of the pavement layers in MPa, Poisson's ratio of each layer and the thickness in mm are the inputs for IITPAVE. The elastic modulus of the subgrade (M_{RS}) is calculated from laboratory CBR using Equations(6.1) and (6.2).

$$M_{RS} = 10 \times CBR \qquad \text{if CBR is 5\%} \quad (6.1)$$

$$M_{RS} = 17.6 \times (CBR)^{0.64} \qquad \text{if CBR} > 5\% \quad (6.2)$$

The elastic modulus of the granular base and sub-bases (M_{RGSB}) are calculated using the Equation (6.3)

$$M_{RGSB} = 0.2 \times (h)^{0.45} \times M_{RS} \qquad (6.3)$$

Whereas, the “h” is the thickness of granular layers in mm and the elastic modulus of the CTB and CTSB (E_{CTB}) are calculated from the laboratory UCS using the Equation (6.4).

$$E_{CTB} = 1000 \times UCS \qquad (6.4)$$

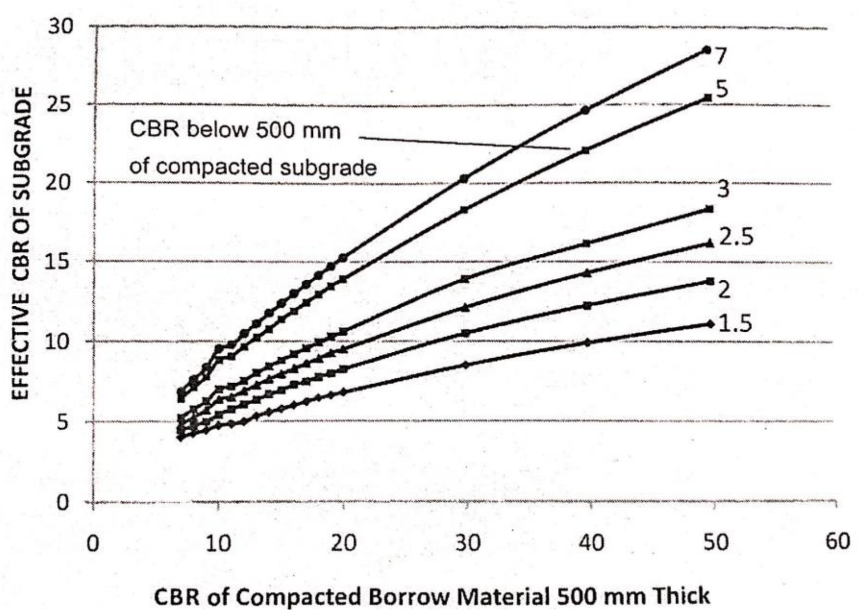
For the CTSB, the modulus value usually varies from 2000 to 6000 MPa. Since the low strength cemented sub-base would crack under heavy construction traffic, the design value of 600 MPa is recommended by IRC. The Poisson’s ratio of the granular materials is considered as 0.35 and that of the CTB and CTSB as 0.25.

In the present work, a single axle dual wheel load configuration is considered with the single axle load of 80 kN and hence the wheel load will be 20 kN withthe contact pressure of 0.8 MPa. The number of points for analysis at which the stresses, strains and deflections to be computed is chosen. The analysis was carried out along the depth of the pavement. Two critical strains like a horizontal tensile strain at the bottom of the top layer (ε_t) (bituminous layer) and the vertical compressive strain at the top of the subgrade (ε_z) along the application of the dual wheel load are computed.

6.6 Pavement design using lateritic soil

6.6.1 Conventional low volume pavement design

For the design of low volume pavements, the subgrade soil of CBR 4% is considered. Hence, the additional layer of modified soil having CBR more than or equal to 10% should be laid. The modified soil of CBR 10% is considered and hence the effective CBR of the subgrade is found to be 7% from Figure 6.3. The elastic modulus of the subgrade soil is calculated using Equation (6.2) and is found to be 61 MPa.



Source: IRC: 37-2012

Figure 6.3 The effective thickness of the subgrade CBR

As per IRC: SP:72-2015, for the design of low volume pavements, the catalogue of pavement design for granular base and sub-bases of low volume pavements is shown in Figure 6.4 and the total thickness of the pavement section for different subgrade class and traffic levels are tabulated in Table 6.3.

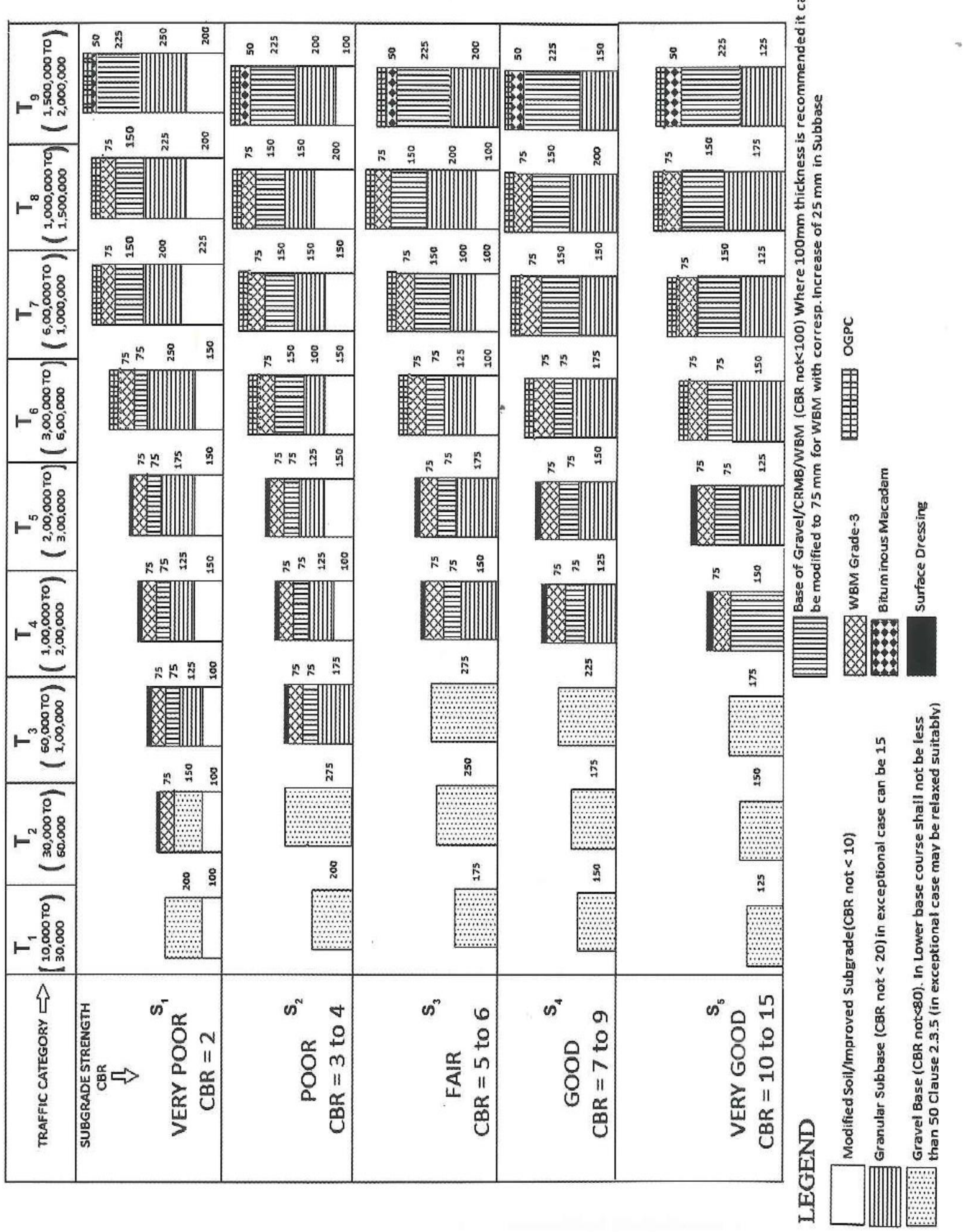


Figure 6.4 The design catalogue of low volume pavement consisting of granular base and sub-bases (Source: IRC SP: 72-2015)

Table 6.3 The total thickness of low volume pavements for different traffic and subgrade class

Subgrade soil classes	Traffic Categories								
	T1	**T2**	**T3**	**T4**	**T5**	**T6**	**T7**	**T8**	**T9**
	Total thickness (mm)								
S1 CBR= 2%	300	325	375	425	475	550	650	650	725
S2 CBR= 3 to 4%	**200**	**275**	**325**	**375**	**425**	**475**	**525**	**575**	**575**
S3 CBR=5 to 6%	175	250	275	300	325	375	425	525	475
S4 CBR= 7 to 9%	150	175	225	275	300	325	375	425	425
S5 CBR= 10 to 15%	125	150	175	225	275	300	350	400	400

The lateritic soil with CBR 4% at standard Proctor density is considered for the pavement design. The pavement structure for subgrade CBR of class S2 for all traffic categories is tabulated in Table 6.4.

Table 6.4 The layer thickness of low volume constructed on subgrade soil class S2 at all traffic categories

Traffic Categories	Pavement layer thickness (mm)						
	Surface dressing	**OGPC**	**BM**	**WBM (Grade 3)**	**Gravel Base**	**GSB**	**Modified Soil**
T1	-	-	-	-	200	-	-
T2	-	-	-	-	275	-	-
T3	✓	-	-	75	75	175	-
T4	✓	-	-	75	75	125	100
T5	✓	-	-	75	75	125	150
T6	-	✓	-	75	150	100	150
T7	-	✓	-	75	150	150	150
T8	-	✓	-	75	150	150	200
T9	-	✓	50	-	225	200	100

OGPC- Open Graded Premix Carpet, BM- Bituminous Macadam,

The modified soil of CBR 10% is followed by GSB, gravel base or base of gravel and WBM. The thickness of the granular layers is considered together and the elastic modulus of the granular layers is calculated using Equation (6.3). The $\varepsilon_z$was computed using the IITPAVE software and is tabulated in Table 6.5 and the $\varepsilon_t$cannot be found as low volume roads will not be havingbituminous concrete layer. To prevent the ingress of rainwater, OGPC/ surface dressing will be provided. The cross-section of the low volume pavements using granular materials is depicted in Figure 6.5.

Table 6.5 The analysis of IITPAVE for granular sub-base and base for low volume roads

Traffic Categories	The thickness of the granular layer (mm)	M_{RGSB} (MPa)	Horizontal tensile strain (ε_t) (10^{-3})	Vertical compressive strain (ε_z) (10^{-3})
T1	200	132	-	2.647
T2	275	153	-	1.597
T3	325	165	-	1.225
T4	375	176	-	0.9723
T5	425	186	-	0.7905
T6	475	195	-	0.6561
T7	525	204	-	0.5511
T8	575	213	-	0.4673
T9	525	204	-	0.5511

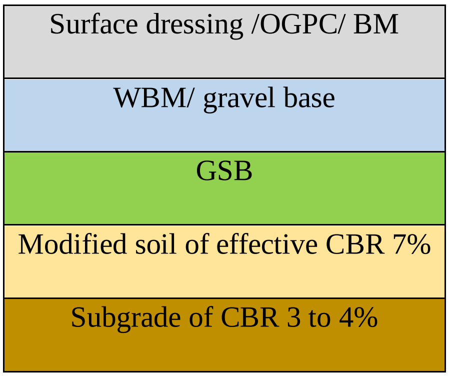


Figure 6.5 The cross-section of the low volume pavement consisting of granular layers

6.6.2 Proposed low volume pavement design using stabilized lateritic soil

The design catalogue of the low volume pavements as per IRC: SP:72-2015, consisting of CTB and CTSB is depicted in Figure 6.6. and the total thickness values of the low volume pavement are tabulated in Table 6.6.

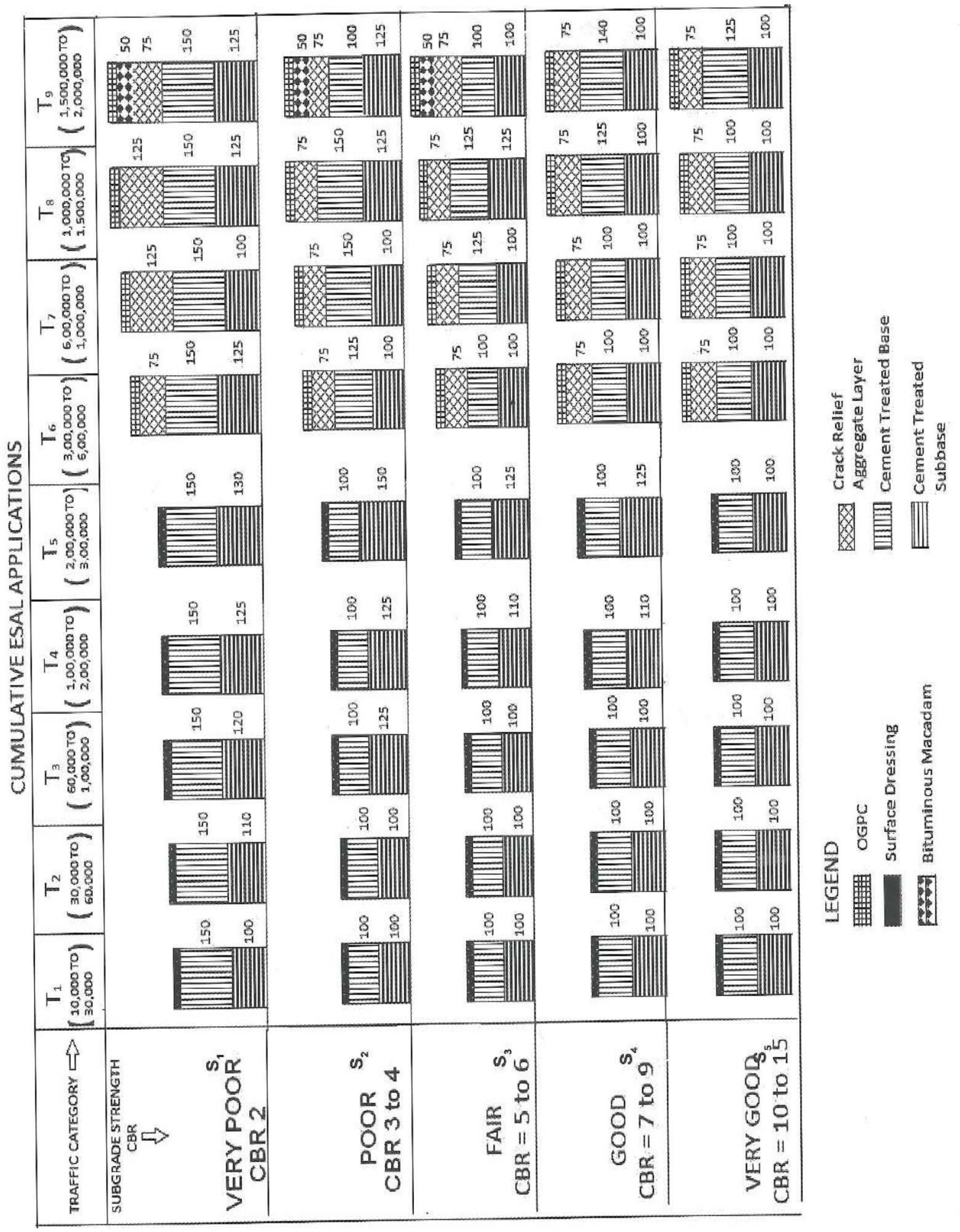


Figure 6.6 The design catalogue of cement-treated bases and sub-bases for low volume pavement (Source: IRC SP: 72-2015)

Table 6.6 The total thickness of the low volume pavement for cement-treatedbase and sub-bases

Traffic Categories	**Traffic categories**								
	T1	**T2**	**T3**	**T4**	**T5**	**T6**	**T7**	**T8**	**T9**
	Total thickness (mm)								
S1 CBR= 2%	250	260	270	275	280	350	375	400	400
S2 CBR= 3 to 4%	**200**	**200**	**225**	**225**	**250**	**300**	**325**	**350**	**350**
S3 CBR=5 to 6%	200	200	200	210	225	275	300	325	325
S4 CBR= 7 to 9%	200	200	200	210	225	275	275	300	315
S5 CBR= 10 to 15%	200	200	200	200	200	275	275	275	300

The thickness of each layer for the subgrade of class S2 having CBR 3 to 4% is tabulated in Table 6.7.

Table 6.7 The thickness of low volume pavement of CTB and CTSB of subgrade class S2

Traffic Categories	**Pavement layer thickness (mm)**					
	Surface dressing	**OGPC**	**BM**	**Crack relief aggregate layer**	**Cement treated base**	**Cement treated sub-base**
T1	✓	-	-	-	100	100
T2	✓	-	-	-	100	100
T3	✓	-	-	-	100	125
T4	✓	-	-	-	100	125
T5	✓	-	-	-	100	150
T6	-	✓	-	75	125	100
T7	-	✓	-	75	150	100
T8	-	✓	-	75	150	125
T9	-	✓	50	75	100	125

The additional layer of modified soil should be laid above the natural subgrade and the conventional granular layers should be replaced with CTSB and CTB. As per IRC, any material to be used as a CTSB, the minimum 7-day laboratory UCS should be more than 1.7 Mpa at standard Proctor density. The stabilized soil sample of 25-5-1.0 having 7-day UCS of 2841 kPa which is more than 1.7 MPa and hence it can be used as CTSB. In the case of CTB, the minimum 7-day laboratory strength should be more than 3 MPa. Therefore, the sample of 25-6-0.5 having UCS of 3257 kPa can be used as CTB. The elastic modulus of the CTSB and CTB is found to be 2841 and 3257 MPa respectively and are calculated using the Equation (6.4). The Poisson's ratio of CTSB and CTB is assumed as 0.25. The elastic modulus of the CTSB can vary from2000 to 6000 MPa, but 600 MPa should be considered for analysis. A 75 mm thick aggregate layer is provided above the CTB to avoid the propagation of cracks and the elastic modulus is considered to be 450 MPa with the Poisson's ratio of 0.35. The strains obtained from IITPAVE replacing the granular base and sub-bases with the CTB and CTSB are tabulated in Table 6.8. The cross-section of the proposed low volume pavements for different traffic conditions is depicted in Figure 6.7.

Table 6.8 The IITPAVE result of the low volume pavements consisting of CTSB and CTB

Traffic Categories	**Horizontal tensile strain (ε_t)**	**Vertical compressive strain (ε_z) (10^{-3})**
T1	-	0.9756
T2	-	0.9756
T3	-	0.8499
T4	-	0.8499
T5	-	0.7459
T6	-	0.6087
T7	-	0.5108
T8	-	0.4685
T9	-	0.4685

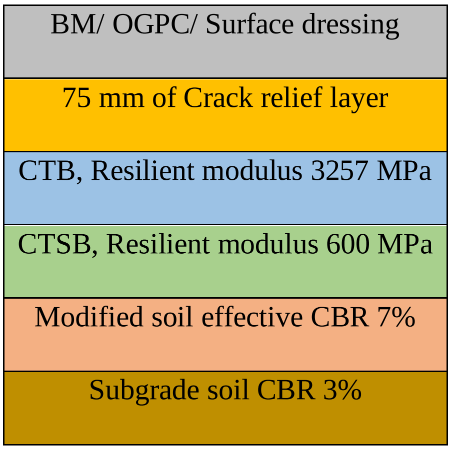


Figure 6.7 The cross-section of low volume pavement consisting of CTB and CTSB

The critical strains obtained from the analysis of conventional low volume pavement are compared with that of the low volume pavement consisting of CTSB and CTB. It is found that the strains obtained from the pavement consisting of CTB and CTSB as a base and sub-base course are much lesser than the granular layers due to the attained strength. Hence the stabilized soil can be recommended as a base course.

6.6.3 High volume pavement design using lateritic soil

The design of high volume pavement is done as per IRC:37-2018 where the traffic ranges from 5 to 50 msa. For the design of low volume pavements, the subgrade soil of CBR 4% is considered. Hence, the top 500 mm of natural subgrade soil needs to be replaced with modified soil having CBR more than or equal to 10%. The modified soil of CBR 10% is considered and hence the effective CBR of the subgrade is found to be 7% from Figure 6.3. The elastic modulus of the subgrade soil is calculated using Equation (6.2) and is found to be 61 MPa.The modified subgrade soil will be followed by GSB, Wet Mix Macadam (WMM), binder course and surface course.The elastic modulus of the conventional granular layers is calculated to be 191 Mpa using Equation (6.3) and that of the bituminous layers such as surface course and binder course together is 3000 MPa of VG 40 at 25^{0}C. The Poisson's ratio of all layers is considered as 0.35. The thickness of the pavement layers are tabulated in Table 6.9 and the critical strains developed in the pavement such as ε_t and ε_z were analyzed using IITPAVE software and the results are tabulated in Table 6.10. The cross-section of the conventional high volume pavement is depicted in Figure 6.8.

Table 6.9 The thickness catalogue of high volume pavement

Traffic (msa)	Pavement layer thickness in mm			
	GSB	WMM	Base/Binder course	Surface course
5	200	250	50	30
10	200	250	55	30
20	200	250	75	40
30	200	250	90	40
40	200	250	100	40
50	200	250	110	40

Table 6.10 The IITPAVE result of high volume pavement using granular base and sub-base

Traffic (msa)	Horizontal tensile strain (ε_t) (10^{-3})	Vertical compressive strain (ε_z) (10^{-3})
5	0.3730	0.4430
10	0.3564	0.4315
20	0.2726	0.3679
30	0.2404	0.3396
40	0.2214	0.3221
50	0.2046	0.3058

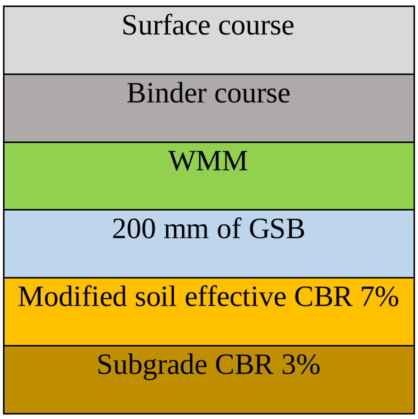


Figure 6.8 The cross-section of the high volume pavement using granular materials

6.6.4 Proposed high volume pavement using stabilized lateritic soil

As per IRC:37-2018, the thickness of the highvolume pavement using GSB and CTB are tabulated in Table 6.11.

Table 6.11 The thickness of the high volume pavements consisting of GSB and CTB

Traffic (msa)		Pavement layer thickness in mm			
	GSB	CTB	AIL	Base/Binder course	Surface course
5	200	145	100	50	30
10	200	155	100	50	30
20	200	170	100	55	30
30	200	155	100	60	40
40	200	160	100	60	40
50	200	165	100	60	40

The high volume pavements are proposed replacing WMM with the CTB. The top 500 mm of modified subgrade soil of 10% CBR is followed by 200 mm of GSB layer for different traffic and the elastic modulus of the GSB is found to be 132 MPa using Equation (6.3). As per IRC, the CTB material should attain the 7/28 days laboratory UCS of 4.5 to 7 MPa. Among the durability satisfied stabilized soil samples, the sample of 25-5-1.0 is giving UCS of 4690 kPa which is between 4.5 to 7.0 MPa. The elastic modulus of the CTB is calculated using Equation (6.4) and is found to be 4690 MPa. The 100 mm of the Aggregate Interface Layer (AIL) will be provided in between CTB and surface course to arrest the cracks propagating on to the surface course and the elastic modulus of AIL is considered as 450 MPa having Poisson's ratio of 0.35 for the analysis. The surface course and binder course will be considered together consisting of VG 40 grade of bitumen at 25^0C thus the elastic modulus is 3000 MPa is considered. The critical strains such as ε_t and ε_z are tabulated in Table 6.12. The cross-section of the high volume pavement consisting of GSB and CTB is depicted in Figure 6.9.

Table 6.12 The IITPAVE results of high volume pavement using GSB and CTB

Traffic (msa)	Horizontal tensile strain (ε_t) (10^{-3})	Vertical compressive strain (ε_z) (10^{-3})
5	0.3884	0.3212
10	0.3861	0.3009
20	0.3625	0.2697
30	0.3121	0.2799
40	0.3110	0.2718
50	0.3100	0.2640

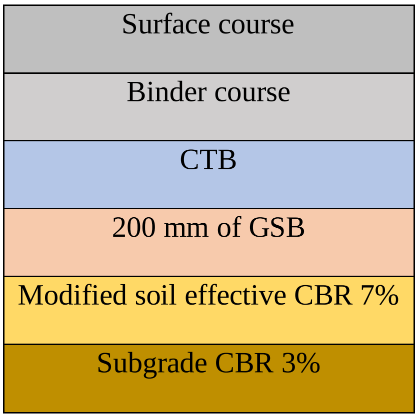


Figure 6.9 The cross-section of the proposed high volume using GSB and CTB

The critical strains obtained from the pavements with granular base and sub-bases are compared with that of the pavements with CTB and found that the difference in strains ismarginal. Hence the high volume pavements replacing the GSB with CTB are recommended.

6.7 Design of pavements using BC soil

The maximum UCS of 2053 kPa is achieved for the stabilized BC soil sample of 30-5-0.5 after 28 days curing at modified Proctor density which does not meet the requirement of CTB suggested by IRC: SP: 72 -2018. Therefore, the stabilized BC soil cannot be used as a CTB for low and high volume pavements.The maximum UCS of 2053 kPa achieved for the stabilized BC soil sample of 30-5-0.5 after 28 days curing at modified Proctor density CTB suggested by IRC: SP: 72 -2018. Therefore, the stabilized BC soil cannot be used as a CTB for low and high volume pavements.

6.8 Cost Comparison

For low volume pavements, the granular layers such as GSB and WBM were suggested. As per the Ministry of Road Transport & Highways (MoRTH)- 2013 recommended by IRC, the GSB of grading V and WBM of grading II are chosen. The cost of materials used for GSB including loading and transportation is found to be Rs. 940/m^3, WBM is Rs. 1133/$m^3$and WMM is Rs. 957/m^3 as per Schedule of Rates (SOR) 2018, Mangalore Public Works Department, Karnataka, India. To propose the low volume pavement, the conventional granular layers such as GSB and WBM are replaced with CTSB and CTB respectively. The stabilized lateritic soil sample of 25-5-1.0 and 25-6-0.5 are recommended for CTSB and CTB respectively. The cost of durability passed stabilized lateritic soil sample at standard and modified Proctor per m^3 are tabulated in Tables 6.13 and 6.14.

Table 6.13 The cost of stabilized lateritic soil samples at standard Proctor density

Sl. No.	Sample combination	Materials	Quantity of material required (kg/m^3)	Unit Cost of material (Rs/kg)	cost (Rs/m^3)	Total Cost (Rs/m^3)	Total Cost (10^7Rs/lane/km / "t" thickness of road)
1	25-5-1.0	GGBS	425	3	1275	4340	1.19
		NaOH	11.6	250	2900		
		Na_2SiO_3	65.4	2.5	163.5		
2	25-6-0.5	GGBS	430	3	1290	7315	2.01
		NaOH	23.7	250	5925		
		Na_2SiO_3	39.7	2.5	100		
3	25-6-1.0	GGBS	420	3	1260	4904	1.34
		NaOH	13.8	250	3450		
		Na_2SiO_3	77.5	2.5	194		
4	30-5-0.5	GGBS	516	3	1548	7572	2.08
		NaOH	14.1	250	5925		
		Na_2SiO_3	78.9	2.5	100		
5	30-5-1.0	GGBS	513	3	1539	5261	1.44
		NaOH	14.1	250	3525		
		Na_2SiO_3	78.9	2.5	197		
6	30-6-0.5	GGBS	516	3	1548	8792	2.41
		NaOH	28.5	250	7125		
		Na_2SiO_3	47.6	2.5	119		
7	30-6-1.0	GGBS	510	3	1530	5966	1.64
		NaOH	16.8	250	4200		
		Na_2SiO_3	94.2	2.5	236		

Table 6.14 The cost of stabilized lateritic soil samples at modified Proctor density

Sl. No.	Sample combination	Materials	Quantity of material required (kg/m^3)	Unit Cost of material (Rs/kg)	cost (Rs/m^3)	Total Cost (Rs/m^3)	Total Cost (10^7 Rs/lane/km/ "t" thickness of road)
1	25-5-1.0	GGBS	453	3	1357	4632	1.27
		NaOH	12.4	250	3100		
		Na_2SiO_3	70	2.5	174		
2	25-6-0.5	GGBS	458	3	1373	7778	2.13
		NaOH	25.2	250	6300		
		Na_2SiO_3	42.2	2.5	106		
3	25-6-1.0	GGBS	460	3	1380	5367	1.47
		NaOH	15.1	250	3775		
		Na_2SiO_3	85	2.5	212		
4	30-5-0.5	GGBS	543	3	1629	7984	2.19
		NaOH	25	250	6250		
		Na_2SiO_3	41.8	2.5	105		
5	30-5-1.0	GGBS	549	3	1647	5608	1.54
		NaOH	15	250	3750		
		Na_2SiO_3	84.5	2.5	211		
6	30-6-0.5	GGBS	546	3	1638	9289	2.55
		NaOH	30.1	250	7525		
		Na_2SiO_3	50.4	2.5	126		
7	30-6-1.0	GGBS	546	3	1638	9289	2.55
		NaOH	30.1	250	7525		
		Na_2SiO_3	50.4	2.5	126		

The cost of materials (including loading and transportation) of conventional GSB, WBM and WMM of low and high volume pavements are compared with that of the CTB and CTSB layers. For illustration, the traffic of T9 (150000 to 200000 msa) of low volume pavement consists of 200 mm of GSB and 225 mm of WBM. Therefore 200 mm of GSB costs about Rs. 185 which will be replaced with 125 mm of CTSB made of stabilized lateritic soil sample of 25-5-1.0 which costs Rs. 545 and 225 mm of WBM costs Rs. 255 which will be replaced with 100 mm of CTB made of stabilized lateritic soil sample of 25-6-0.5 costs Rs. 732. Similarly, in case of high volume pavements having 50 msa traffic consisting of 250 mm of WMM which costs Rs. 240 which will be replaced with 165 mm of CTB made of stabilized lateritic soil sample of 25-5-1.0 at modified Proctor density and the cost is found to be Rs. 764. The cost comparison of low and high volume pavements using both conventional and stabilized soil is tabulated in Table 6.15.

Table 6.15 The cost comparison of low and high volume pavements

Sl. No.	Pavements/ Traffic	Layers	Cost (Rupees)
1	Low volume pavements/ T9	Conventional i) 200 mm of GSB ii) 225 mm of WBM	 185 255
		Stabilized soil i) 125 mm of CTSB ii)100 mm of CTB	 545 732
2	High volume/ 50 msa	Conventional 250 mm of WMM	 240
		Stabilized soil 165 mm of CTB	 764

6.9 Summary

- The design of low and high volume pavements is proposed replacing the granular materials with the stabilized soil which meets the requirements.
- The conventional and proposed low volume and high volume pavements are suggested as per IRC: SP:72-2015 and IRC: 37-2018 respectively.
- For low volume pavements, the stabilized lateritic soil sample of 25-5-1.0 and 25-6-0.5 at standard Proctor density are suggested as CTSB and CTB respectively replacing GSB and WBM.
- For high volume pavements, the stabilized lateritic soil sample of 25-5-1.0 at modified Proctor density is suggested as CTB replacing WMM in conventional pavements.
- The critical strain analysis was carried out using IITPAVE software and found the strains of proposed low and high volume pavements are within the limits.

- The stabilized BC soil is not recommended for low and high volume pavement design as the stabilized soil does not meet the requirement of cement-treated base and sub-bases.
- From the cost analysis, it is noticed thatthe cost of stabilized lateritic soil sample of 25-5-1.0 and 25-6-0.5 at standard Proctor density costs 3 and 2.8 times that of the GSB and WBM respectively.
- In case of high volume pavements, the sample of 25-5-1.0 at modified Proctor density costs 3.2 times that of the WMM.

6.10 Limitations/ Drawbacks of the work

The limitations of the present research work are listed below.

- The proposed roads using stabilized lateritic soil is found expensive and uneconomical than the conventional pavement.
- The skilled labours with prior training is required to lay the alkali stabilized soil.

6.11 Future scope of the research work.

- The alkali stabilized soil can be tested for flexural strength adding natural fibres like arecanut coir, coconut coir etc.
- The alkali stabilization can be done on different soils and can be compared.

CHAPTER 7

CONCLUSIONS

The stabilized lateritic and BC soils using the GGBS treated with alkali solutions such as sodium hydroxide and sodium silicate showed the improved engineering properties and the granular layers in the pavement construction can be replaced with stabilized soil. The major conclusions are drawn from the present investigation are listed in this chapter.

7.1 Lateritic soil stabilization

- The stabilized lateritic soil sample of 30-6-1.0 cured for 28 days has achieved the UCS of 6341 and 9901 kPa which are 14.8 and 18.6 times that of the untreated soil at standard and modified densities respectively.
- The CBR of the stabilized soil is found more than 100% as the sample had become hard and the moisture content after 4 days of soaking was only 15%.
- All stabilized lateritic soil samples are found durable in the freezing-thawing test but soil stabilized with 25 and 30% of GGBS and alkali solution consisting of 5 and 6% of Na_2O at 1.0 Ms passed durability test with weight loss less than 14% after 12 alternate cycles.
- The stabilized lateritic soil sample of 30-6-1.0 has achieved the highest flexural strength of 0.69 and 1.33 MPa at standard and modified Proctor densities respectively.
- The stabilized lateritic soil sample of 30-6-1.0 sustained 4.01×10^5 and 4.52×10^5 repetitions at standard and modified Proctor densities respectively.
- The microstructure images of the stabilized samples showed a closely packed and densified structure due to the achieved strength.
- The stabilized soil sample of 25-5-1.0 as cement-treated sub-base and 25-6-0.5 as the cement-treated base is suggested in case of low volume pavements. Whereas in the case of high volume pavement design, the stabilized lateritic soil sample of 25-5-1.0 at modified Proctor density is suggested to use as cement-treated base.
- The strains of the proposed pavements are within the limits.

- From the cost analysis, the stabilized lateritic soil costs about 3 times that of the conventional granular layers.

7.2 BC soil stabilization

- The maximum UCS of 1407 and 2053 kPa is achieved for the stabilized soil sample of 30-5-0.5 cured for 28 days at standard and modified Proctor densities respectively which is 7 and 6.5 times that of the untreated soil.
- The CBR of the stabilized soil is found to be more than 100%.
- The stabilized soil samples of 25-5-0.5, 25-6-0.5, 30-5-0.5 and 30-6-0.5 passed the durability test at only modified Proctor density.
- The highest flexural strength of 0.98 MPa was achieved from the sample of 30-5-0.5 at modified Proctor density.
- The sample of 30-5-0.5 sustained 1.37×10^5 repetitions at modified Proctor densityat 1/3rd of the minimum UCS.
- The samples of 25-5-0.5 and 30-5-0.5 showed the closely packed compact structure.
- Based on test results, the stabilized BC soil cannot be recommended for base and sub-bases.

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APPENDIX I

Table I.1 The UCS and Young's modulus values of stabilized lateritic soil at both densities

Sample	Standard Proctor test		Modified Proctor test	
	UCS (MPa)	E (MPa)	UCS (MPa)	E (MPa)
15-4-0.5	1.2	61.3	2.6	157.5
15-4-1.0	1.1	42.3	2.4	209.0
15-4-1.5	1.0	26.9	1.6	172.0
15-5-0.5	1.8	183.5	2.9	99.2
15-5-1.0	1.8	187.5	2.7	171.9
15-5-1.5	1.7	164.0	1.6	182.0
15-6-0.5	2.4	283.2	3.3	72.6
15-6-1.0	1.9	174.8	3.0	84.7
15-6-1.5	1.8	203.5	2.7	159.3
20-4-0.5	1.3	112.3	2.6	204.2
20-4-1.0	2.0	171.1	2.7	232.6
20-4-1.5	1.5	117.0	2.4	197.9
20-5-0.5	2.3	155.6	2.9	202.1
20-5-1.0	2.1	153.9	3.1	238.0
20-5-1.5	1.7	109.1	2.7	153.6
20-6-0.5	3.2	264.4	4.3	521.3
20-6-1.0	3.4	284.7	5.3	648.0
20-6-1.5	2.0	168.0	3.1	356.8
25-4-0.5	1.6	104.6	3.4	412.6
25-4-1.0	1.8	154.7	4.4	527.9
25-4-1.5	1.5	106.6	2.5	282.3
25-5-0.5	2.7	242.0	4.0	333.2
25-5-1.0	3.7	398.2	4.7	556.7
25-5-1.5	1.6	138.6	2.8	375.9
25-6-0.5	3.8	425.1	5.7	743.2
25-6-1.0	5.5	866.1	6.4	792.7
25-6-1.5	2.3	247.4	3.4	322.1
30-4-0.5	1.9	129.9	4.3	558.0
30-4-1.0	2.3	197.1	5.5	738.5
30-4-1.5	2.1	156.7	2.5	381.5
30-5-0.5	3.2	280.1	5.2	473.2
30-5-1.0	4.4	745.5	6.6	802.1
30-5-1.5	2.1	256.0	2.5	399.4
30-6-0.5	5.7	1030.4	8.0	994.9
30-6-1.0	6.3	1280.4	9.9	1227.4
30-6-1.5	4.8	861.2	3.9	661.1

Table I.2 The UCS and Young's modulus values of stabilized BC soil at both densities

Sample	Standard Proctor test		Modified Proctor test	
	UCS (MPa)	E (MPa)	UCS (MPa)	E (MPa)
15-4-0.5	0.572	846.0	1.184	2046.0
15-4-1.0	0.468	795.0	1.058	1974.0
15-4-1.5	0.418	543.0	0.927	1137.0
15-5-0.5	0.792	994.0	1.635	3072.0
15-5-1.0	0.753	893.0	1.172	1746.0
15-5-1.5	0.638	824.0	0.974	1628.0
15-6-0.5	0.836	1302.0	1.746	4038.0
15-6-1.0	0.761	1247.0	1.293	2748.0
15-6-1.5	0.653	1203.0	0.837	1428.0
20-4-0.5	0.978	1185.0	1.383	2972.0
20-4-1.0	0.949	1368.0	1.355	2836.0
20-4-1.5	0.614	1183.0	0.878	1589.0
20-5-0.5	1.343	1628.0	1.964	4683.0
20-5-1.0	1.306	1593.0	1.911	4182.0
20-5-1.5	0.845	1086.0	1.238	3067.0
20-6-0.5	1.141	1582.0	1.659	3895.0
20-6-1.0	1.116	1375.0	1.584	4078.0
20-6-1.5	0.713	1038.0	1.036	2896.0
25-4-0.5	0.978	1085.0	1.409	4238.0
25-4-1.0	0.941	1173.2	1.352	4186.0
25-4-1.5	0.823	1033.6	1.173	4076.0
25-5-0.5	1.366	1486.0	1.989	6248.0
25-5-1.0	1.303	1405.0	1.914	5286.0
25-5-1.5	1.138	1254.0	1.662	4862.0
25-6-0.5	1.184	1264.0	1.793	5183.0
25-6-1.0	1.113	1202.0	1.592	4582.0
25-6-1.5	0.967	1054.0	1.399	3284.0
30-4-0.5	1.019	1204.0	1.440	4386.0
30-4-1.0	0.953	1176.0	1.354	3847.0
30-4-1.5	0.820	1026.0	1.172	2917.0
30-5-0.5	1.407	1528.0	2.053	7582.0
30-5-1.0	1.316	1514.0	1.922	7249.0
30-5-1.5	1.131	1045.0	1.666	3847.0
30-6-0.5	1.221	1564.0	1.727	5786.0
30-6-1.0	1.119	1395.0	1.611	4238.0
30-6-1.5	0.969	1056.0	1.376	4015.0

APPENDIX II

Table II.1 The sample durability test format of sample of 25-5-1.0 at standard Proctor density under WD test

WD for sample of 25-5-1.0 at standard Proctor density								
Number of cycles	**0 days**							
	Sample I				**Sample II**			
	Wetting	**Wt. loss (%)**	**Drying**	**Wt. loss (%)**	**Wetting**	**Wt. loss (%)**	**Drying**	**Wt. loss (%)**
1	Collapsed		Collapsed		Collapsed		Collapsed	
2								
3								
4								
5								
6								
7								
8								
9								
10								
11								
12								

WD for sample of 25-5-1.0 at standard Proctor density								
Number of cycles	**3 days**							
	Sample I				**Sample II**			
	Wetting	**Wt. loss (%)**	**Drying**	**Wt. loss (%)**	**Wetting**	**Wt. loss (%)**	**Drying**	**Wt. loss (%)**
	166.1				160.26			
1	181.3	-9.15	148.77	10.43	184.72	-15.26	148.5	7.34
2	178.65	-7.56	148.15	10.81	173.51	-8.27	148.17	7.54
3	177.62	-6.94	147.41	11.25	162.42	-1.35	139.99	12.65
4	176.76	-6.42	146.89	11.57	Collapsed		Collapsed	
5	176.21	-6.09	146	12.10				
6	175.53	-5.68	145.14	12.62				
7	174.79	-5.23	144.65	12.91				
8	174.2	-4.88	143.78	13.44				
9	173.46	-4.43	142.95	13.94				
10	Collapsed		Collapsed					
11								
12								

WD for sample of 25-5-1.0 at standard Proctor density								
Number of cycles	7 days							
	Sample I				Sample II			
	Wetting	Wt. loss (%)	Drying	Wt. loss (%)	Wetting	Wt. loss (%)	Drying	Wt. loss (%)
	153.13				153.26			
1	181.56	-18.57	149.04	2.67	179.5	-17.12	148.37	3.19
2	176.69	-15.39	144.34	5.74	175.12	-14.26	145.06	5.35
3	171.15	-11.77	140.21	8.44	172.86	-12.79	142.67	6.91
4	167.43	-9.34	136.54	10.83	170.95	-11.54	140.56	8.29
5	163.91	-7.04	133.25	12.98	169.08	-10.32	138.69	9.51
6	Collapsed		Collapsed		163.71	-6.82	134.27	12.39
7					160.17	-4.51	131.89	13.94
8					Collapsed		Collapsed	
9								
10								
11								
12								

WD for sample of 25-5-1.0 at standard Proctor density								
Number of cycles	28 days							
	Sample I				Sample II			
	Wetting	Wt. loss (%)	Drying	Wt. loss (%)	Wetting	Wt. loss (%)	Drying	Wt. loss (%)
	151.31				151.42			
1	177.9	-17.6	149.1	1.5	178.3	-17.7	148.1	2.2
2	177.7	-17.5	148.6	1.8	178.1	-17.6	148.4	2.0
3	177.4	-17.3	153.9	-1.7	177.9	-17.5	154.2	-1.8
4	176.5	-16.7	147.2	2.7	177.1	-16.9	146.9	3.0
5	176.2	-16.5	148.0	2.2	176.6	-16.7	147.8	2.4
6	175.6	-16.0	145.8	3.7	176.3	-16.4	145.6	3.8
7	175.3	-15.9	146.3	3.3	175.9	-16.2	145.8	3.7
8	175.0	-15.7	145.7	3.7	175.4	-15.8	145.1	4.2
9	174.9	-15.6	147.1	2.8	175.1	-15.7	146.4	3.3
10	174.5	-15.3	144.9	4.2	173.9	-14.8	143.3	5.3
11	174.4	-15.3	144.3	4.7	173.5	-14.5	142.5	5.9
12	173.9	-14.9	144.4	4.5	172.3	-13.8	142.1	6.1

Table II.2 The sample durability test format of sample of 25-5-1.0 at standard Proctor density under FT test

FT for sample of 25-5-1.0 at standard Proctor density								
Number of cycles	0 days							
	Sample I				Sample II			
	Freeze	Wt. loss (%)	Thaw	Wt. loss (%)	Freeze	Wt. loss (%)	Thaw	Wt. loss (%)
	172.27				173.13			
1	169.82	1.42	165.28	4.06	170.22	1.68	165.33	4.51
2	164.25	4.66	155.11	9.96	164.49	4.99	155.99	9.90
3	151.16	12.25	161.49	6.26	151.11	12.72	160.12	7.51
4	151.25	12.20	154.2	10.49	151.59	12.44	155.15	10.39
5	151.15	12.26	151.63	11.98	151.88	12.27	152.24	12.07
6	151.2	12.23	151.13	12.27	151.68	12.39	151.6	12.44
7	150.88	12.42	150.82	12.45	150.82	12.89	150.69	12.96
8	150.85	12.43	150.76	12.49	150.69	12.96	150.68	12.97
9	150.78	12.47	150.77	12.48	150.7	12.96	150.76	12.92
10	150.73	12.50	150.68	12.53	150.69	12.96	150.62	13.00
11	150.82	12.45	150.78	12.47	150.64	12.99	150.65	12.98
12	150.92	12.39	150.72	12.51	150.77	12.92	150.61	13.01

FT for sample of 25-5-1.0 at standard Proctor density								
Number of cycles	3 days							
	Sample I				Sample II			
	Freeze	Wt. loss (%)	Thaw	Wt. loss (%)	Freeze	Wt. loss (%)	Thaw	Wt. loss (%)
	162.22				164.54			
1	151.26	6.76	152.53	5.97	151.53	7.91	152.88	7.09
2	151.25	6.76	152.25	6.15	151.25	8.08	152.93	7.06
3	151.22	6.78	151.6	6.55	151.32	8.03	152.36	7.40
4	151.21	6.79	151.32	6.72	151.22	8.10	151.59	7.87
5	150.88	6.99	150.8	7.04	151.25	8.08	151.23	8.09
6	150.78	7.05	150.7	7.10	151.5	7.93	151.9	7.68
7	150.67	7.12	150.55	7.19	150.91	8.28	150.83	8.33
8	150.52	7.21	150.57	7.18	150.8	8.35	150.83	8.33
9	150.6	7.16	150.6	7.16	150.77	8.37	150.86	8.31
10	150.69	7.11	150.54	7.20	150.88	8.30	150.83	8.33
11	150.81	7.03	150.65	7.13	151.9	7.68	150.85	8.32
12	150.73	7.08	150.51	7.22	150.82	8.34	150.8	8.35

FT for sample of 25-5-1.0 at standard Proctor density								
Number of cycles	**7 days**							
	Sample I				**Sample II**			
	Freeze	**Wt. loss (%)**	**Thaw**	**Wt. loss (%)**	**Freeze**	**Wt. loss (%)**	**Thaw**	**Wt. loss (%)**
	153.17				151.58			
1	151.56	1.05	153.2	-0.02	152.1	-0.34	152.23	-0.43
2	151.55	1.06	151.71	0.95	152	-0.28	152.27	-0.46
3	151.45	1.12	150.11	2.00	151.98	-0.26	152.3	-0.47
4	151.21	1.28	151.29	1.23	151.68	-0.07	151.84	-0.17
5	151.2	1.29	150.91	1.48	151.45	0.09	151.48	0.07
6	150.76	1.57	150.78	1.56	151.22	0.24	151.2	0.25
7	150.6	1.68	150.63	1.66	150.95	0.42	151.5	0.05
8	150.46	1.77	150.47	1.76	150.78	0.53	150.86	0.47
9	150.5	1.74	150.4	1.81	150.86	0.47	150.83	0.49
10	150.69	1.62	150.56	1.70	150.99	0.39	150.96	0.41
11	150.85	1.51	150.49	1.75	150.85	0.48	150.92	0.44
12	150.55	1.71	150.53	1.72	150.55	0.68	150.93	0.43

FT for sample of 25-5-1.0 at standard Proctor density								
Number of cycles	**28 days**							
	Sample I				**Sample II**			
	Freeze	**Wt. loss (%)**	**Thaw**	**Wt. loss (%)**	**Freeze**	**Wt. loss (%)**	**Thaw**	**Wt. loss (%)**
	150.77				150.88			
1	150.47	0.20	150.51	0.17	150.59	0.19	150.63	0.17
2	150.59	0.12	150.46	0.21	150.68	0.13	150.64	0.16
3	150.68	0.06	150.65	0.08	150.7	0.12	150.69	0.13
4	150.73	0.03	150.57	0.13	150.71	0.11	150.68	0.13
5	150.58	0.13	150.57	0.13	150.61	0.18	150.7	0.12
6	150.55	0.15	150.56	0.14	150.65	0.15	150.5	0.25
7	150.45	0.21	150.45	0.21	150.64	0.16	150.5	0.25
8	150.22	0.36	150.42	0.23	150.63	0.17	150.4	0.32
9	150.25	0.34	150.44	0.22	160.55	-6.41	150.43	0.30
10	150.11	0.44	150.36	0.27	159.89	-5.97	150.36	0.34
11	150.09	0.45	150.35	0.28	159.88	-5.97	150.32	0.37
12	150.05	0.48	150.36	0.27	159.82	-5.93	150.23	0.43

APPENDIX III

Table III.1 The fatigue test results of durability satisfied lateritic and BC soil samples

Soil	Samples	Compaction	UCS (kPa)	Stress level	Applied Load (kPa)	Fatigue life (no of cycles x 10^5)
Lateritic soil	25-5-1.0	Standard Proctor Density	1327	0.33	438	3.82
				0.5	664	2.76
				0.66	876	2.44
	25-6-0.5			0.33	438	3.85
				0.5	664	2.82
				0.66	876	2.46
	25-6-1.0			0.33	438	3.87
				0.5	664	2.85
				0.66	876	2.57
	30-5-0.5			0.33	438	3.91
				0.5	664	3.71
				0.66	876	3.27
	30-5-1.0			0.33	438	3.95
				0.5	664	3.82
				0.66	876	3.72
	30-6-0.5			0.33	438	3.97
				0.5	664	3.86
				0.66	876	3.77
	30-6-1.0			0.33	438	**4.01**
				0.5	664	3.92
				0.66	876	3.81

Soil	Samples	Compaction	UCS (kPa)	Stress level	Applied Load (kPa)	Fatigue life (no of cycles x 10^5)
	25-5-1.0	Modified Proctor Density	1412	0.33	466	4.12
				0.5	706	4.03
				0.66	932	3.77
	25-6-0.5			0.33	466	4.15
				0.5	706	4.08
				0.66	932	3.81
	25-6-1.0			0.33	466	4.21
				0.5	706	4.13
				0.66	932	3.91
	30-5-0.5			0.33	466	4.31
				0.5	706	4.4
				0.66	932	3.92
	30-5-1.0			0.33	466	4.46
				0.5	706	4.34
				0.66	932	4.18
	30-6-0.5			0.33	466	4.50
				0.5	706	4.37
				0.66	932	4.36
	30-6-1.0			0.33	466	**4.52**
				0.5	706	4.43
				0.66	932	4.41

Soil	Samples	Compaction	UCS (kPa)	Stress level	Applied Load (kPa)	Fatigue life (no of cycles x 10^5)
BC soil	25-5-0.5	Modified Proctor Density	1727	0.33	570	1.1
				0.5	864	0.5
				0.66	1140	0.14
	25-6-0.5			0.33	570	0.09
				0.5	864	0.32
				0.66	1140	0.89
	30-5-0.5			0.33	570	**1.37**
				0.5	864	0.37
				0.66	1140	0.1
	30-6-0.5			0.33	570	0.77
				0.5	864	0.23
				0.66	1140	0.07

APPENDIX IV

Cost calculation

<u>Illustration 1: To find the cost of materials per m^3</u>

For stabilized lateritic soil sample of 25-5-1.0 at standard Proctor density

The dry density of sample 25-5-1.0 is 1.7 g/cc $= (1.7\times (10^{-3}))/ (10^{-2})^3$

$= 1700$ kg/m^3

<u>To calculate quantity of materials</u>

The sample of 25-5-1.0 consists of 25% of GGBS, 5% of Na_2O and 1.0 Ms

Therefore, Amount of GGBS = (25/100) ×1700 = 425 kg

The amount of soil = (1700- 425) = 1275 kg

From the dosage calculation as explain in the Chapter 3, the amount of NaOH= 11.6 kg and Na_2SiO_3= 65.4 kg

<u>To calculate cost of materials</u>

From the SOR, the cost of GGBS= Rs. 3/kg, NaOH= Rs. 250 /kg and Na_2SiO_3= Rs. 4 / litre

The density of Na_2SiO_3 is found to be 1.563 kg/litre

It indicates that 1 litre of Na_2SiO_3 = 1.563 kg

It means 1.563 kg costs Rs.4

Therefore, 1 kg of Na_2SiO_3 costs = (1×4)/1.563 = Rs. 2.5 / kg

Hence, the 11.6 kg of NaOH costs = (11.6 × 250) = Rs. 2900/ m^3

And 65.4 kg of Na_2SiO_3 costs= (65.4 × 2.5) = Rs. 163.5/ m^3

Also, 425 kg of GGBS costs = (425 × 3) = Rs. 1275/ m^3

Therefore, to lay the sample of 25-5-1.0 for m^3 of low volume road at standard Proctor density, the total cost includes cost of (GGBS+ NaOH+ Na_2SiO_3) = (1275+ 2900+ 163.5) = Rs. 4338.5/ m^3.

Similarly, the cost of all durability passed stabilized lateritic soil are calculated and tabulated in Table below.

To calculate the cost of one lane per km per thickness "t" of the road= (Total cost of materials per m^3 × single lane width in m × 1km length of road × thickness of road in m (t)

Therefore the material cost in Rs per km = 4338.5 × 2.75 × 1000 × t

= 1.2 × "t" crore Rs / m^3

Table IV.1The cost of stabilized soil sample per m^3

Sample	Proctor	Dry density (kg/m^3)	Total mass of mixture (kg)	Quantity of soil (kg)	Quantity of GGBS (kg)	Quantity of NaOH (kg)	Quantity of Na_2SiO_3 (kg)	Cost of GGBS (Rs/m^3)	Cost of NaOH (Rs/m^3)	Cost of Na_2SiO_3 (Rs/m^3)	Total cost (Rs/m^3)
25-5-1.0	Standard Proctor density	1700	1700	1275	425	11.6	65.4	1275	2900	163.5	4338.5
25-6-0.5		1720	1720	1290	430	23.7	39.7	1290	5925	99.25	7314.25
25-6-1.0		1680	1680	1260	420	13.8	77.5	1260	3450	193.75	4903.75
30-5-0.5		1720	1720	1204	516	23.7	39.7	1548	5925	99.25	7572.25
30-5-1.0		1710	1710	1197	513	14.1	78.9	1539	3525	197.25	5261.25
30-6-0.5		1720	1720	1204	516	28.5	47.6	1548	7125	119	8792
30-6-1.0		1700	1700	1190	510	16.8	94.2	1530	4200	235.5	5965.5
25-5-1.0	Modified Proctor density	1810	1810	1357.5	452.5	12.4	69.6	1357.5	3100	174	4631.5
25-6-0.5		1830	1830	1372.5	457.5	25.2	42.2	1372.5	6300	105.5	7778
25-6-1.0		1840	1840	1380	460	15.1	84.9	1380	3775	212.25	5367.25
30-5-0.5		1810	1810	1267	543	25	41.8	1629	6250	104.5	7983.5
30-5-1.0		1830	1830	1281	549	15	84.5	1647	3750	211.25	5608.25
30-6-0.5		1820	1820	1274	546	30.1	50.4	1638	7525	126	9289
30-6-1.0		1820	1820	1274	546	30.1	50.4	1638	7525	126	9289

Illustration 2: Cost estimation of pavements as per thickness suggested by IRC

Low volume pavements

For T9 traffic, the low volume pavements consisting of 200 mm of GSB and 225 mm of WBM,

As per SOR-2018, the cost of materials including the loading and transportation,

the GSB of grading V costs Rs. 940/ m^3,

Therefore, the providing 200 mm of GSB, the volume will be 0.2 m^3.

Hence, the cost of GSB for 0.2 m^3 is = (0.2×940) = Rs. 188/ m^3

Similarly, 225 mm of WBM of grading II costs Rs. 1133/ m^3

Therefore, the providing 225 mm of WBM, the volume will be 0.225 m^3.

Hence, the cost of WBM for 0.225 m^3 is = (0.225×1133) = Rs. 255/ m^3

The 200 mm of GSB will be replaced with 125 mm of CTSB and 225 mm of WBM will be replaced with 100 mm of CTB.

Therefore, the volume of 125 mm of CTSB will become 0.125 m^3 and 100 mm of CTB will become 0.1 m^3

Also, the CTSB of sample of 25-5-1.0 and CTB of sample of 25-6-0.5 at standard Proctor density is considered.

Hence, the cost of 0.125 m^3 of sample of 25-5-1.0 is = (0.125×4340) = Rs. 545/ m^3

Similarly, the cost of 0.1 m^3 of sample of 25-6-0.5 is= (0.1×7315) = Rs. 732/ m^3

Therefore the 0.1 m^3 of sample of 25-6-0.5 per km / lane costs = $(732 \times 2.75 \times 1000)$ = 20.13 lakhs Rs /m^3 / km / lane.

High volume pavements

For traffic 50 msa, the pavements consist of 250 mm of WMM which has to be replaced with 165 mm of CTB of sample of 25-5-1.0 at modified Proctor density

The volume of 250 mm of WMM is 0.25 m^3 and the cost of WBM per m^3 is Rs. 957/ m^3

Therefore, the cost of 0.25 m^3 of WMM is = (0.250×957) = Rs. 240/ m^3

The volume of 165 mm of CTB is 0.165 m^3 and the cost of CTB of sample 25-5-1.0 per m^3 is Rs. 4632/ m^3

Therefore, the cost of 0.165 m^3 of sample of 25-5-1.0 is = (0.165×4632) = Rs. 764/ m^3

Therefore the 0.165 m^3 of sample of 25-5-1.0 per km / lane costs = $(764 \times 2.75 \times 1000)$ = 21.01 lakhs Rs /m^3 / km / lane.

LIST OF PUBLICATIONS

INTERNATIONAL/ NATIONAL JOURNALS

Amulya, S. and Ravi Shankar, A. U. (2020) "Use of stabilized lateritic and black cotton soils as a base course replacing conventional granular layer in flexible pavement." *International Journal of Geosynthetics and Ground Engineering*., 6(5), **doi:** 10.1007/s40891-020-0184-8.

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Amulya, S., Ravi Shankar, A. U. and Praveen, M. (2018) "Stabilization of lithomargic clay using alkali activated fly ash and ground granulated blast furnace slag." *International Journal of Pavement Engineering*. **doi:** 10.1080/10298436.2018.1521520.

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B E	Civil Engineering	UBDT College of Engineering, Davanagere	2008 – 2012	77.6 %
Higher Secondary	Science	Sri Taralabalu Jagadguru Comp Pre-University College, Davangere	2006 – 2008	74.3 %
Class X		Siddaganga Composite High School & PU Science College, Davangere	2005 – 2006	86.1%

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INTERNATIONAL/ NATIONAL JOURNALS

Amulya, S. and Ravi Shankar, A. U. (2020) "Use of stabilized lateritic and black cotton soils as a base course replacing conventional granular layer in flexible pavement." *International Journal of Geosynthetics and Ground Engineering*., 6(5), **doi:** 10.1007/s40891-020-0184-8.

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