STUDIES ON RESILIENT AND PERMANENT DEFORMATION CHARACTERISTICS OF GRANULAR PAVEMENT LAYERS

A Thesis

Submitted in partial fulfilment of the requirements

for the award of the degree of

Doctor of Philosophy

Submitted by

Noolu Venkatesh

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Department of Civil Engineering

National Institute of Technology

Warangal-506004, India

July 2019

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Declaration

This is to certify that the work presented in the thesis entitled **"STUDIES ON RESILIENT AND PERMANENT DEFORMATION CHARACTERISTICS OF GRANULAR PAVEMENT LAYERS"** is a bonafide work done by me under the supervision of **Dr. M. Heeralal and Dr. Rakesh J Pillai** and was not submitted elsewhere for the award of any degree. I declare that this written submission represents my ideas in my own words and where others ideas or words have been included, I have adequately cited and referenced the original sources. I also declare that I have adhered to all principles of academic honesty and integrity and have not misrepresented or fabricated or falsified any idea / data / fact /source in my submission. I understand that any violation of the above will be a cause for disciplinary action by the Institute and can also evoke penal action from the sources which have thus not been properly cited or from whom proper permission has not been taken when needed.

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CERTIFICATE

This is to certify that the thesis entitled "**STUDIES ON RESILIENT AND PERMANENT DEFORMATION CHARACTERISTICS OF GRANULAR PAVEMENT LAYERS"** being submitted by **Mr. NooluVenkatesh** for the award of the degree of **DOCTOR OF PHILOSOPHY** to the Faculty of **Civil Engineering** of **NATIONAL INSTITUTE OF TECHNOLOGY, WARANGAL** is a record of bonafide research work carried out by him under my supervision and it has not been submitted elsewhere for award of any degree.

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This Thesis entitled "**STUDIES ON RESILIENT AND PERMANENT DEFORMATION CHARACTERISTICS OF GRANULAR PAVEMENT LAYERS"** by **Mr. Noolu Venkatesh** is approved for the degree of Doctor of Philosophy.

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Abstract

Civil engineering practice has gone a long way from using California bearing ratio (CBR) value alone for designing of flexible pavements. Empirical methods of design have given way to limiting shear failure methods, limiting deflection methods, regression method based on pavement performance or road test and finally landing up on Mechanistic Empirical Pavement Design methods. Mechanistic-empirical design practice for flexible pavement as suggested by AASHTO relies mainly on the resilient modulus of the granular materials. Though the resilient modulus is assumed to account for plastic strain accumulation, some subgrade soils with more fine content undergo excessive plastic deformation under repeated loads even though they have remarkable resilient properties. Hence, permanent deformation is also an important parameter, which should be considered in the design process. In the presentthesis investigations are carried out to study the resilient and permanent deformation characteristics of granular materials. Two clayey soils, with intermediate plasticity and high plasticity are selected for the experimental work. Stabilization of clayey subgrade will be essential for better long term performance of the flexible pavement. Studies regarding the resilient and permanent deformation characteristics of chemically stabilized subgrade materials are also very less reported in the literature. Calcium carbide residue(CCR) is a by-product from acetylene factories is used to improve subgrade soils in the present work. Utilization of recycled aggregates as a subbase material is also considered in the present study. The resilient and permanent deformation behaviour of both natural and recycled aggregates are compared.

As the first research objective, an attempt has been made to evaluate the potential of calcium carbide residue as an admixture to improve the engineering behaviour of the two clayey soils (both CH and CI). The role of different percentages of these binder contents in altering the atterberg limits, compaction characteristics, strength, mineralogical and morphological

behaviour has been examined. The plasticity properties and compaction characteristics of both clayey soils have improved considerably with the addition of calcium carbide residue. Significant improvement in the strength properties such as unconfined compressive strength and California bearing ratio has been observed with the addition of calcium carbide residue up to 8% for the black cotton soil, whereas for red soil the maximum strength was observed at 4% binder content. The increase in strength can be attributed to the formation of calcium-based minerals formed as a result of pozzolanic reactions in the soil-binder mixture which is confirmed from mineralogical and morphological studies. Second objective of the present study is to evaluate the influence of calcium carbide residue in resilient modulus of clayey subgrade soils. The improvement in resilient modulus of two clayey soils (with low plasticity and high plasticity) with the addition of calcium carbide residue is investigated by carrying out repeated load triaxial (RLT) tests. The influence of variation in moisture content, deviatoric stress and confining pressure on the repeated loading behaviour of virgin soils and CCR stabilized soils is examined. Two regression models (Universal and NCHRP model) reported in literature are found to exhibit very good fit with the experimental data. Third objective is to examine the permanent deformation behaviour of clayey subgrade soils by subjecting the samples to large number of loading cycles in cyclic triaxial apparatus. The influence of confining pressure, moisture content and deviator stress levels on the permanent deformation behaviour is examined. Effect of calcium carbide residue on permanent deformation behaviour of both the clays is analysed. The transition of the soil sample from stable to unstable state can be observed and critical stress levels established with the help of shake down theory. Based on the experimental investigations it is observed that in the case of black cotton soil specimens prepared at OMC, the elastic shakedown limit is at 50% stress level, whereas plastic creep stage is observed at 60% stress level. For a higher water content OMC+2% and OMC+4%, the elastic shake down range is found to be in between 40% and

50% stress levels and the plastic creep stage commenced at 40% stress level. For Red soil, the elastic shakedown limit for samples prepared at OMC is found to be at the 30% stress level, whereas plastic creep stage is observed at 40% stress level. For a higher moulding water contents of OMC+2% and OMC+4%, the elastic shake down range is observed below 20% stress level and the plastic creep stage commenced at 30% stress level itself. Therefore, it can be inferred that with increase in the sub-grade water content, the stress level corresponding to shakedown decreases, leading to rutting under repeated load. The stabilization with calcium carbide residue is found to be more effective in worst case scenario with higher water content and cyclic stress levels. Two regression models reported in the literature, Power law model and VTT model, are used to predict the plastic strains with a number of load cycles and stress levels as the variables. Fourth objective is divided into two phases. The first phase deals with the comparison of resilient and permanent deformation behaviour of natural and recycled aggregates. Effect of confining pressure and deviatoric stress levels on resilient and permanent deformation of natural and recycled aggregates were investigated. The permanent deformation behaviour is quantified using the critical cyclic stress levels based on the shake down theory. The experimental data for the permanent deformation of the aggregates is also found to fit well with VTT model and power law model. The second phase deals with pavement model tests carried out on test tank with untreated and treated clayey soil as subgrade material and the natural aggregate as the base material.

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List of Symbols and notation

A, B, a, b, c = regression parameters (A is also limit value for maximum permanent axial strain)

A_1 = material and stress-strain parameter given (function of stress ratio and resilient modulus)

A_2–A_4, D_2–D_4 = parameters that are functions of stress ratio q/p

C = apparent cohesion

C_u = Coefficient of Uniformity

fn = shape factor

G_p= shear modulus with respect to permanent deformation

h = repeated load hardening parameter (function of stress to strength ratio)

K_p= bulk modulus with respect to permanent deformation

k_1, k_2, k_3 = Material constants

L = stress path length

LL = Liquid Limit

m = slope of static failure line

M_R = Resilient Modulus

N = number of load applications

N_1, N_2, N_3 = Material Constants

p = mean normal stress

P_a = Atmospheric Pressure

PL = Plastic Limit

PI = Plasticity Index

p^0 = modified mean normal stress = $\sqrt{3}.p$

p_0 = reference stress

p^* = stress parameter defined by intersection of static failure line and p-axis in p-q space

q = deviator stress

q_0 = modified deviator stress = $\sqrt{(2/3)}.q$

R_f= ratio of measured strength to ultimate hyperbolic strength

S = static strength

w_c = Moisture content

γ_{dr} = Maximum Dry Density

γ_d= Maximum Density

ε_i= permanent strain for first load cycle

ε_N= permanent strain for load cycle N

ε_r= resilient strain

$e_{s,p}$= permanent shear strain for N > 100

$e_{v,p}$= permanent volumetric strain for N > 100

$\varepsilon_{1,p}$= accumulated permanent strain after N load repetitions

$\varepsilon^*_{1,p}$= additional permanent axial strain after first 100 cycles

$\varepsilon_{1,p}(N_{ref})$ = accumulated permanent axial strain after given number of cycles N_{ref}(N_{ref}> 100)

$\varepsilon_{0.95S}$= static strain at 95% of static strength

σ_3 = confining pressure

σ_d = Deviatoric Stress

τ_{oct} =Octahedral Shear Stress

Φ = angle of internal friction

Chapter 1

Introduction

1.0 General introduction

The economic and social development of nations all over the world relies to a large extent on the quality of its road transportation system.The flexible pavements constitute the major part of India's road network. Proper design of flexible pavements demands accurate prediction of plastic strains in the granular layers. Improper design results in pavement failures, which leads to spending billions of dollars for the rehabilitation of pavement (Muhanna et al., 1998; Puppala et al., 1999; Puppala et al., 2009) .The main three causes of failures in flexible pavement, are thermal cracking, rutting and fatigue cracking. Rutting occurs on the pavement surface due to the permanent deformation of pavement layers subjected to repeated wheel loads. It is assumed that the base, subbase and subgrade layers contribute to the permanent deformation of flexible pavement (Puppala et al., 1999; Puppala et al., 2009).

1.1 Empirical- Mechanistic pavement design

The most common parameter used to evaluate pavement layer strength is the California Bearing Ratio (CBR). Even though the CBR is not a fundamental soil property, its significance lies in the fact that it is the basis of the CBR Method of Pavement design which is still, by far, the most popular pavement design method used in developing countries.

The water content, dry density and as well as the texture of the soil influence the CBR value. Normally, the CBR test in the laboratory is conducted on test samples prepared at the dry density and water content is likely to be achieved in the field. The field dry density can be fairly well predicted, but the main difficulty is to determine the stable moisture content at which the test is to be conducted. The local practice, which is also used in many other countries, is to use the 4-day soaked CBR method.

Civil engineering practice has gone a long way from the times where the CBR value was heavily depended upon for flexible pavement design. Empiristic methods of design has since then given way to limiting shear failure methods, limiting deflection methods, regression method based on pavement performance or road test, finally landing up on Mechanistic Empiristic Pavement Design methods [MEPD]. The MEPD considers both fatigue and rutting failure criteria for flexible pavements (Puppala et al., 1999; Jegatheesan and Gnanendran 2015). The rutting occurs on the pavement surface due to the permanent deformation of pavement layers subjected to repeated wheel loads. It is assumed that base, subbase and Subgrade layers contribute for the permanent deformation of flexible pavement (Puppala et al., 1999). Proper design of flexible pavements demands accurate prediction of plastic strains in the granular layers. Improper design results in pavement failures leading to spending of billions of dollars for the rehabilitation of pavement (Muhanna et al., 1998; Puppala et al., 1999; Puppala et al., 2009).

Resilient modulus of subgrade soil is one of the most important material parameters used to characterize its nonlinear response under repeated loading in Mechanistic-empirical design of flexible pavements (NCHRP 2004; AASHTO 2008). Just as stiffness of any linear elastic material under gradual loading is quantified by Young’s modulus, the stiffness of granular material subjected to repeating load as experienced in pavement layers is characterized by resilient modulus. Resilient modulus is defined as the ratio of applied cyclic deviatoric stress to recoverable (resilient) strain developed in the material after a specified number of load cycles. The initial plastic response of the granular material is not considered as the long term performance of pavements is given primary importance. Resilient modulus of subgrade soil is generally obtained by conducting repeated triaxial test on soil samples in the laboratory.

Mechanistic-empirical pavement design has categorised resilient modulus into 3 levels based on accuracy. Level 1 consists of resilient modulus values, which are determined from repeated load triaxial tests. Level 2 comprises resilient modulus values obtained from empirical correlations with other soil parameters such as California Bearing Ratio (CBR) and unconfined compressive strength (UCS). In Level 3, the parameter is correlated to index properties of soil and has very low accuracy. Several researchers have studied the resilient behaviour of granular materials and the factors affecting resilient modulus (Puppala*et al*., 1999; Mallel*et al*., 2004; Nazzal and Mohammad, 2010; Rahman and Erlingsson, 2015). Some of the important factors, which influence resilient modulus of granular materials, are soil gradation, water content, microstructure, dry density, stress state and stress history (Rada and Witczak, 1981; Li *et al*., 1994; Zaman *et al*., 1994;Han and Vanapalli, 2016; Mamatha and Dinesh, 2017).Stress levels and moisture content are known to be two key variables, which influence the resilient modulus. Resilient modulus is found to increase with increase in confining pressure and reduce with increase in cyclic deviatoric stress levels. The resilient modulus is observed to decrease with increasing moisture content for unsaturated soils due to lubricant effect and pore pressure build up (Butalia*et al*., 2003; Heydinger, 2003; Han and Vanapalli, 2016). Several researchers have reported that the suction generated in partially saturated soils plays a crucial role in their resilient behaviour, which in turn depends on the moisture content variations (Edris, 1976; Fredlund *et al*., 1977; Han and Vanapalli, 2016; Mamatha and Dinesh, 2017). Resilient modulus of granular materials increases with increase in dry unit weight, but its effect is less prominent compared to moisture content and stress levels.

Over the past few decades, significant research has been reported in literature to consider the plastic deformation in pavement design (Smith et al., 1975; Lentz and Baladi, 1993; Korkiala and Dawson, 2007). In most of these studies, resilient modulus is heavily relied upon to accommodate the plastic response of soils. It is also reported in the literature that some of the subgrade soils with more fine content may possess good resilient characteristics, but will still exhibit excessive plastic deformations when subjected to repeated loads (Ullditz, 1993; Puppala, 1999; Cong Lin et al., 2011). This observation clearly indicates the need to evaluate the permanent deformation behaviour of subgrade soils under different stress levels for complete characterization of the material. It is reported in the literature that for granular materials, permanent strain accumulation is very slow under low cyclic stresses even at higher number of load cycles. However, at high stress levels the rate of accumulation of permanent deformation increases (Lekarp and Dawson 1998). Substantial amount of research have been reported in the past to evaluate the permanent response of soil based on stress level, moisture content and confining pressure (Puppala, 1999; Cong Lin et al., 2011). However, only few works were reported considering the permanent deformation of stabilized subgrade soils.

1.2 Need for study

The subgrade soils used in Indian road pavements are mostly fine-grained soils. Among these, the clayey soils are susceptible to moisture changes. In addition to this, some soils with high resilient modulus may still experience high permanent deformation with repetition of applied loads. Puppala *et al.* (2009) confirmed that some of these soils such as silts may have moderate to high resilient modulus values, but they undergo high permanent deformations. It is also important to take into consideration the variability of strength and resilient characteristics of clayey subgrade soils (Vorobieff and Murphy (2003). Li and Selig (1996) (Anupam *et al.,* 2016) suggested two approaches in order to control the permanent deformation on the top of the subgrade layer:

i) To improve the subgrade by stabilisation or modification,

ii) Deviatoric stress reduction by increasing the thicknesses of the upper layers.

The shortcoming with the second approach is that even by controlling the stress level, the subgrade layer could still be exposed to moisture changes and undergo high permanent deformations. It is also known that increase in moisture content significantly affects the subgrade soil deformation. In these circumstances, the improvement of the soil with a stabiliser may be an ideal solution for restricting permanent deformation resistance.The addition of chemical stabilizers is a widely used technique to improve the engineering properties of clayey soil. It includes traditional stabilizers like cement, lime, and fly ash. Effect of stabilization depends upon the calcium exchange and pozzolanic reactions between soil and stabilizers. Cement stabilization is one of the extensively used techniques to rectify the deficiencies in engineering properties of expansive soils, especially for pavement applications. An advantage of cement stabilization is that the required strength can be attained in a shorter period. The effect of moisture content, replacement ratio, compaction effort, curing period, and cement content on theengineering characteristics and microstructure of cement aided soils is widely being researched (Tatsuoka & Kobayashi, 1983;; Horpibulsuk & Miura, 2001; Horpibulsuk *et al.*, 2011b). In order to restrict the stabilization cost, replacement of cement by the industrial by-products like fly ash, pond ash, and granulated blast furnace slag has been widely adopted in practice. Many researchers studied the behaviour of fly ash stabilized soils (Sharma & Reddy, 2004; Edil *et al.*, 2006), and it was noticed that theaddition of fly ash leads to increasing reactive surface of soil-cement clusters for the pozzolanic reactions (Horpibulsuk *et al.*, 2009). The curing period also plays a vital role in enhancing thestrength of black cotton soil stabilized with lime and fly ash (Zha *et al.*, 2008).

One of the by-products from the acetylene gas industries is calcium carbide residue (CCR) and it is formed due to the reaction between calcium carbide and water. In developing countries like India, there are many acetylene gas production units and PVC chemical plants, which produce CCR in huge amounts (Sharma & Reddy, 2004; Phetchuay *et al.*, 2014). It is usually dumped in the landfills causing environmental problems to landfills due to its alkalinity. Jiang *et al.* (2015) performed multi-scale investigation on clayey soils stabilized with CCR and lime and reported the optimum dosages of lime and CCR as 6% and 8 %, respectively, for silty clay stabilization. However, CCR stabilization was found to be more effective than lime due to its superior mechanical performance. The existence of ettringite and non-crystalline phase calcium silicate hydrate (C-S-H) after 7 days of curing was identified when the effect of CCR and biomass ash binder on soft Bangkok clay was investigated (Vichan & Rachan, 2013).

The empirical pavement design procedure followed in India (IRC-37 Guide for Design of Flexible Pavement Structures 2012) uses empirical relationships for pavement thickness design. However, these relationships are usually derived from a specified material type, traffic load and environmental condition that are different from the conditions that are found in India (or other parts of the world). Therefore, Mechanistic -Empirical pavement design procedure is best preferred wherein all the mentioned conditions can be considered for different materials, different loading and environmental conditions.

1.3 Scope and objective

The main objectives of the present study are given below:

- To study the influence of calcium carbide residue stabilization on properties of subgrade soils.

- To determine the effect of CCR stabilization on resilient modulus of subgrade soils under repetitive loading.
- To investigate the effect of CCR stabilization on permanent deformation behavior of subgrade soils under repetitive loading.
- To perform permanent deformation analysis by laboratory Pavement Model with base layer prepared with natural and recycled aggregates.

In order to achieve the objectives, a detailed experimental study is planned. The scope of the study is limited to standard laboratory tests and repeated load triaxial tests carried out on two locally available clayey soils without and with the addition of calcium carbide residue. The two clayey soils selected have different plasticity characteristics. The laboratory pavement model study was carried out on a two layer model consisting of a subgrade layer and a base layer. For subgrade layer, untreated and treated clayey soils were considered and for base layer recycled aggregates and natural aggregates were considered for the study.

1.4 Structure of dissertation

This dissertation study is divided into seven chapters.

Chapter 2 deals with the effect of several factors on resilient modulus permanent deformation. Summary of Regression models are used to predict resilient and permanent deformation. Introduction about shake down theory and usage of shake down theory to characterize the permanent deformation of granular materials.

Chapter 3 presents the materials used in the experimental programme and index properties of clayey soils and chemical composition of soils and stabilizers and preparation of soil samples for cyclic triaxial test. It is also explained about different test procedures that were used in dissertation.

Chapter 4 consists of engineering properties of subgrade soil and effect of CCR stabilization on strength properties of clayey soils. Mineralogical and morphological studies on natural soil and CCR stabilised soils were investigated.

Chapter 5 deals with resilient response of CCR stabilized subgrade soils moulded at different moisture contents under different stress states. Regression analysis is carried out with the experimental data using universal model and NCHRP model and the corresponding coefficients are obtained.

Chapter 6 presents the behaviour of Black cotton soil, Red soil stabilized with CCR subjected to repeated loading under varying deviator stress levels and water contents. Repeated load triaxial tests were carried out on virgin soil samples as well as samples treated with CCR. The samples were prepared at three different moisture contents and the tests were carried out under different confining stress and deviatoric stress levels. The permanent strain data obtained from the experiments is fitted with two regression models, namely VTT modeland Power law model, which takes in to account the effect of stress levels and number of load cycles.

Chapter 7 deals with resilient and permanent deformation characteristics of natural and recycled aggregates. Effect of deviatoric stress levels and confining pressure on resilient and permanent deformation of aggregates were determined. Laboratory pavement model tests were conducted to find out the deformation of pavement structure with different subgrade soils. Effect of water content on deformation characteristics of pavement structures were also determined.

Chapter 2

Literature review

2.1 Introduction

A pavement is a structure, which undergoes repeated loading under wheel path of varying magnitude. The pavement is structurally classified as a flexible or rigid pavement. Both types of pavements typically possess stiff soil layer as the bottom layer. A flexible pavement consists of one or two top layers made of bituminous concrete overlying base layers made of granular materials. A rigid pavement consists of stiff cement concrete layer overlying a granular layer. The load carrying capacity and the performance of pavement depends mainly on the response of these materials and their interplay in transferring the stresses from one layer to the other. Hence the material parameters are the most critical in the pavement design process.

The pavement granular materials are characterized by various experiments developed in the course of time. The initial experiments developed were based on the soil classification system. Later development of empirical tests used not much theoretical basis. The empirical tests were predominant for a long time due to their simplicity and ease of analysis. In the meantime, the industry and the development in vehicles with multi axle wheel configurations and heavy load capacities led to the changes in the pavement structural design. This has initiated the need to develop mechanistic pavement design methods, which in turn has given a way to more robust and reliable material characterization techniques.

Laboratory experiments provide data for understanding the behavior of the pavement materials. Only a limited number of data under constrained condition can be obtained under laboratory conditions. If one needs to understand the complete spectrum of the mechanical property under realistic loading conditions, the laboratory experimental data to be conceived are considerable and such experimental data can become meaningless if not interpreted properly. A constitutive model can fill this gap and a properly calibrated and validated model can be used for predicting the material response under varying loading and environmental conditions. This chapter reviews the literature on experimental characterization and modelling of the response of granular material used in pavement engineering.

2.2 Material characterization for flexible pavement design

Till early 20th century, pavements were constructed using the rule of thumb procedure. As there wasn't a standard procedure of pavement construction back in those days, they adopted some methods from the knowledge gathered from long term observations. Later on, empirical methods were developed and were standardized based on performance observations, but were confined only to the local materials available and thus couldn't be adopted in the case of new or alternate materials. Materials were characterized based on simple index tests and these methods did not consider the procedure of construction and frost susceptibility of pavement materials. The main problem of traditional design methods is that they are restricted only to a few types of pavement materials and design procedures. Due to urbanization, the rate of availability of local virgin aggregates decreased and as a result engineers were forced to adopt by-products and recycle aggregates as pavement materials. On the other hand, due to increase in traffic, the deformation of roads increased resulting in a hike in maintenance cost. The empirical methods failed to adapt to the above-mentioned situations. To overcome these drawbacks, mechanistic design procedures were developed. These techniques are used to figure out the deformations of pavement materials and layers under different loading and environmental conditions. These methods provide resilient modulus factor for better understanding the characterization and permanent deformation for finding out rutting in the pavement layers.

2.3 Resilient Modulus of granular material

The stiffness of granular material subjected to repeating load as experienced in pavement layers is characterized by resilient modulus. Resilient modulus is defined as the ratio of applied cyclic deviatoric stress to recoverable (resilient) strain developed in the material after a specified number of load cycles. The initial plastic response of the granular material is not considered as the long-term performance of pavements is given primary importance. Resilient modulus of subgrade soil is one of the most important material parameters used to characterize the nonlinear response under repeated loading in Mechanistic-empirical design of flexible pavements (NCHRP 2004; AASHTO 2008). Resilient modulus of subgrade soil is generally obtained by conducting repeated triaxial test on soil samples in the laboratory. Mechanistic-empirical pavement design has categorised resilient modulus into 3 levels based on accuracy. Level 1 consists of resilient modulus values, which are determined from repeated load triaxial tests. Level 2 comprises resilient modulus values obtained from empirical correlations with other soil parameters such as California Bearing Ratio (CBR) and unconfined compressive strength (UCS). In Level 3 the parameter is correlated to index properties of soil and has very low accuracy. Permanent strain and resilient strain of granular materialsare illustrated in Figure 2.1.

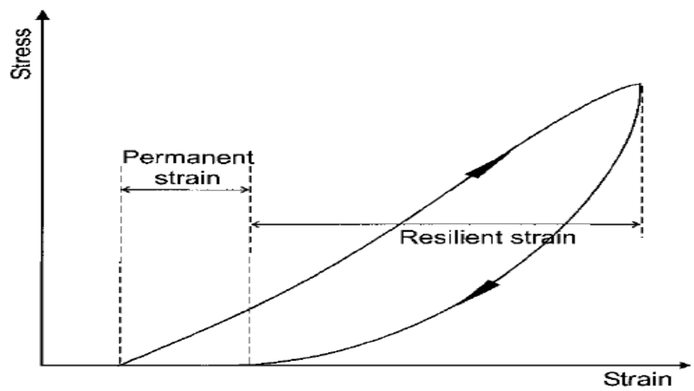

Figure 2.1 Strains in granular materials during one cycle of load application

2.3.1 Factors affecting resilient modulus

a) Effect of stress

Monismith et al. (1967) studied the effect of confining pressure on granular materials wherein the confining pressure increased from 20 kPa to 200 kPa, i.e, there is nearly 500% increment in resilient modulus is observed. Morgan (1966) conducted repeated triaxial load test on granular material and observed that increment in deviatoric stress results in increase in resilient modulus under constant confining pressure. Allen and Thompson (1974) reported that the constant confining pressure on specimen results in the enhancement of resilient modulus compared to variable confining pressure. According to Smith and Nair (1973), principal stress is less effective on resilient modulus, i.e., with change in total stress from 70 kPa to 140 kPa there is little increment observed in the resilient modulus. Morgan (1966) conducted repeated triaxial test on base materials and noted that with increase in deviatoric stress resilient modulus decreased. Kolisoja (1997) noted that Poisson's ratio is directly proportional to deviatoric stress and indirectly proportional to confining pressure.

Rout *et al*. (2012) investigated the effect of lime and cement stabilization on three subgrade soils and reported nearly 2.5 - 2.8 times increase in resilient modulus of the soils. Kang *et al*. (2014) studied the stiffness behaviour of soft clays stabilized with fly ash and lime kiln dust and reported a significant increment in resilient modulus and unconfined compressive strength (UCS). In addition, they proposed an empirical correlation between resilient modulus and UCS of stabilized subgrade soils.

b) Effect of Density

Trollpe et al. (1962) noted 50% increment in resilient modulus when loose soil is replaced with dense soil. Brown and Selig (1991) reported that the effect of density on resilient modulus is less for granular material but Hicks and Monismith (1971) argued that the effect of density is more on partially crushed aggregates compared to fully crushed aggregates and the reason behind it is that the fine content is more in case of crushed aggregates. Barksdale and Itani (1989) noted that the density is effective when lower stress is applied compared to higher stress level.

c) Effect of grading and fine content

Hicks and Monismith (1971) noted that the addition of fines to partially crushed aggregates results in the increment in resilient modulus. Hicks (1970) observed that the addition of fine content from 2 to 10% results is minimal increment in resilient modulus but Barksdale and Itani (1989) reported that nearly 60% increment in resilient modulus is observed on addition of 12% fines. Jorenby and Hicks (1986) noted that the addition of clay fines to crushed aggregates causes enhancement in resilient modulus due to the filling of the pore spaces. Thom and Brown (1988) reported that the uniformly graded crushed aggregates possess higher resilient modulus compared to well graded aggregates. Heydinger et al. (1996) observed that the addition of moisture leads to the increment in resilient modulus of uniformly graded aggregates up to the optimum moisture content and studied the effect of grading which showed minimal effect in case of gravel whereas for limestone with open gradation showed higher resilient modulus compared to other grades.

d) Effect of moisture content

Haynes and Yoder (1963) stated that the increment in moisture from 70% to 97% results in 50% decrement in resilient modulus. Hicks and Monismith (1971) observed that the addition of moisture to base materials leads to the constant decrement in resilient modulus. Thom and Brown (1987) observed decrement in resilient modulus with increasing moisture content due to the lubrication effect. A study was performed by Raad et al. (1992) on different parameters whereas moisture content is considered as the most effective in case of well graded compared to uniformly graded materials. Dawson et al. (1996) observed that the addition of moisture tends to increase the resilient modulus of base materials up to the optimum moisture content.

Edil *et al*. (2006) used fly ash as a stabilizing material for enhancing the resilient modulus of fine grained soils and concluded that fly ash stabilization is more effective for low plasticity clayey soils compared to organic soils. They also observed that stabilization is more effective at higher water contents. Ardah *et al.* (2017) investigated the effect of cement-fly ash stabilization on resilient behaviour of four different subgrade soils at different moisture conditions and found that the effect of stabilization is more in the case of soils moulded at higher water content.

e) Effect of stress history & Number of load cycles

Boyce et al. (1976) conducted cyclic triaxial test on crushed lime stone specimens and observed that the specimens subjected to preloading with few load cycles resulted in the reduction of stress history and after that it got nullified. Hicks (1970) observed that the effect of stress history is eliminated after 100 load cycles but Allen (1973) reported that the stress history is eliminated after 1000 load cycles.

Most of the standard testing protocols give much emphasis to the number of load repetitions. All protocols include a conditioning sequence of 500 to 1000 cycles to eliminate errors due to improper seating of end platens and consecutively the cycles are repeated to around 100 to 500 times to remove the influence of plastic strain. However experiments carried out by many researchers have shown that several thousand cycles of loading were required to remove the plastic strain (Seed *et al*. (1962), Tanimoto and Nishi (1970), Brown (1974) and Houston *et al.* (1993)). The number of load applications also contributed to the variation in the material response. Studies by Seed *et al*. (1962) have confirmed that for a certain range of stresses even after 10000 load repetitions, the permanent deformation increases. Similarly, experiments carried out by Tanimoto and Nishi (1970) suggested that the selection of an appropriate number of stress applications plays a prominent role in the determination of the actual resilience characteristics. They also suggested that the resilient strain couldn't reach a constant value within 10000 load applications. Moore et al. (1970) observed that the increase in number of load cycles caused minimal increment in resilient modulus, which is due to the loss of moisture content. Puppala et al. (2009) performed repeated load triaxial test on three different soils and observed that variation in resilient modulus is minimal for all the soils.

f) Loading frequency and duration

A wide range of frequencies of loading along with loading magnitudes has been used. A load cycle duration of 0.1 second with a 0.9 second rest period is normally adopted in most of the standards (LTPP, 1996; AASHTO: T307, 2003). Tanimoto and Nishi (1970) indicated through experiments that for large frequencies, the resilient strain was considerably low. This is not at all surprising as large frequency results in the small load duration and thus the response can be in the elastic regime.

g) Effect of aggregate shape

Heydinger et al. (1996) observed that the gravel showed higher resilient modulus compared to lime stone. Allen and Thompson (1974) noted that the crushed aggregate particles showed higher resilient modulus compared to uncrushed aggregates. Barksdale and Itani (1989) reported that the angular particles had higher resilient modulus properties compared to rounded particles.

h) Compaction methods

As the stiffness of the sample influences the Resilient modulus (M_R) and this in turn is based on the compact effort, major studies have been carried out to develop an understanding of the various compact efforts such as kneading compaction, vibratory compaction, and static compaction. Seed *et al*. (1962) studied the variation in M_R due to kneading and static compaction. Lee *et al.* (1995) conducted M_R test on the specimen prepared using impact and vibratory compaction methods. The specimens prepared by using vibratory compaction exhibited higher dry density at increased water content, lesser permanent deformation and 40% increment in M_R when compared to that of the specimen compacted using impact compaction. The variation is attributed to the non-uniform densification using impact compaction and due to the different fabric stress history of the compacted soil. Mamatha and Dinesh (2017) conducted repeated trixal test on Black cotton soil and lime stabilized Black cotton soil with different density and observed that density is more effective in case of virgin Black cotton soil compared to lime stabilized Black cotton soil.

2.3.2.Determination of Resilient Modulus

While the use of M_R to quantify the mechanical property of granular material is well developed on sound theoretical reasoning, difficulties were involved related to the testing protocol that can lead to the material exhibiting resilient elastic strain. Unlike the CBR test, the test method to find M_R is not unique. The AASHTO guide (AASHTO, 1986) prescribed a standard method for testing the resilient modulus of subgrade Soils (AASHTO: T274-82, 1986). Many protocols were developed to calculate resilient modulus and later AASHTO proposed two different procedures AASHTO: T292 (1991) and AASHTO: T294 (1992) for determining the resilient modulus of soils. The Strategic Highway Research Program (SHRP) under the Long term Pavement Performance (LTPP) program developed a testing protocol to determine the M_R named it as P46 (LTPP, 1996) and later modified and brought out in AASHTO: T307-99 (2003).

These MR testing methods are based on more sophisticated experimental and measurement system and requires skilled technical manpower. The test can be considerably complex when compared to conventional test methods such as CBR, Dynamic Cone Penetration (DCP), and Unconfined compression (UCC) test.

NCHRP report 1-37A (NCHRP, 2004) pointed out that the resilient modulus test can be measured directly from the laboratory or obtained through the use of correlation with other material strength parameters such as CBR. NCHRP report 1-37A (NCHRP, 2004) also suggested models to determine M_R from other experiments such as CBR, PI (Plasticity Index) and aggregate gradation, asphalt layer coefficient and DCP. Such development lead the researchers to calculate the MR from results obtained from such conventional tests. Different initiatives were undertaken by several investigations to develop statistical correlations relating MR with alternate test methods. Such attempts were drivenaway due to high cost and sophistication of M_R test. Kim (1992) tried to characterize M_R from Resonant Column (RC) and Torsional Shear (TS) test. Rahim and George (2004) carried out experiments to correlate M_R and soil index properties and came out with two different equations, one for fine grained soils (equation 2.2) and another for coarse soil (equation 2.3).

$$M_R = 16.75\left({}^{LL}/_{w_c} * \gamma_{dr}\right)^{2.06} + \left({}^{\#200}/_{100}\right)^{-0.59} \qquad 2.1$$

$$M_R = 307.4\left({}^{\gamma_d}/_{w_c}\right)^{0.86}\left({}^{\#200}/_{\log cu}\right)^{-0.46} \qquad 2.2$$

Here LL is liquid limit, γ_{dr} is dry density/maximum dry density, γd is dry density, *wc*is the moisture content, #200 is the percentage passing the 0.075mm sieve and *cu* is the uniformity coefficient.

Other prominent alternate test method include back calculation from non-destructive testing (NDT) such as the Falling Weight Deflectometer (FWD) test and estimations from the AASHTO Guide algorithm (AASHTO, 2002).

2.3.3. Regression models to predict Resilient Modulus

Many models have been put forth to define the resilient modulus with respect to factors that influence the behavior of the material. The usage of these relations to predict the resilient behavior of granular materials are prevalent for their simplicity and easy implementation rather than for their reliability in predicting the realistic behavior of the material.

Dunlap (1963) conducted several cyclic triaxial tests on base materials to find the relation between confining pressure and resilient modulus and proposed a relation equation, which is in log-log form and he also observed that cyclic deviatoric stress is less effective in finding the resilient modulus.

$$M_R = K\sigma_3^n \qquad 2.3$$

Thompson and Robnett (1979) conducted repeated triaxial test on base materials, observed that the deviatoric stress is the more predominant factor in determining the resilient modulus, and proposed a bilinear model between resilient modules and deviatoric stress.

$$M_R = K_1 + K_2\sigma_d \qquad 2.4$$

Moossazadeh and Witzcak (1981) proposed a semi log model between deviatoric stress and resilient modulus and stated that confining pressure is not effective in the determination of resilient modulus.

$$M_R = K\sigma_d^n \qquad 2.5$$

Garg and Thompson (1997) performed a large number of triaxial tests on granular materials and found out that both confining pressure and deviatoric stress are effective in finding the resilient modulus and proposed the below relation.

$$M_R = N_1 q^{N_2} {\sigma_3}^{N_3} \qquad 2.6$$

Hicks (1970) conducted several triaxial tests on granular materials and proposed bulk stress model, which considers both confining pressure and deviatoric stress. It is also called as K-φ model.

$$M_R = k_1 \emptyset^{k_2} \qquad 2.7$$

Uzan (1985) performed numerous cyclic triaxial tests on granular material and proposed a model with respect to both the confining pressure and deviatoric stress and it was widely used to characterize the resilient response, whichconsidered the influence of the sum of the principal stresses. The main drawback of the K-θ model was that it assumed a constant Poisons ratio but in reality, it can vary with the magnitude of the stresses.

$$M_R = k_1 P_a \left({}^{\theta}/_{P_a}\right)^{k_2} \left({}^{\sigma_d}/_{P_a}\right)^{k_3} \qquad 2.8$$

This model uses both bulk stress and octahedral stress for the determination of resilient modulus. According to this model, the resilient modulus is determined using the following equation

$$M_R = K_1 p_a \left({}^{\theta}/_{p_a}\right)^{K_2} \left({}^{\tau_{oct}}/_{p_a} + 1\right)^{K_3} \qquad 2.9$$

With the help of constitutive relationships, it is possible to predict the behavior of materials. The simplest model is behavior elastic and the popular Hooke's law belongs to this category, where the stresses and strains are related with the help of a material parameter, the Young's Modulus. However, for complex materials such as soils, the material behaviour cannot be approximated to a linearized elastic one. Further, a wide range of factors such as the state of stress, residual or initial stress, volume changes under shear and stress history influences the response of granular materials. In such cases, appropriate constitutive relationships were formulated using the concepts of plasticity, taking into consideration the factors influencing the behavior of soil which provide a realistic representation of the observed behaviour.

The evolution of constitutive modelling in soil mechanics with special relevance to pavement granular materials has come from the simple elastic models to the highly complex plasticity based models. The purpose of all these models is to achieve a better agreement between the predicted and observed soil behavior. A brief review of the popular constitutive models used for predicting the behavior of pavement granular materials is described here. Broadly the models can be classified into two categories i) Elastic models and ii) Elastic-plastic models.

Elastic models are the simplest of all and yet are still used widely in pavement engineering applications. The isotropic elastic model belongs to this category of linear elastic models.

In elastic – plastic models, the soil behaviour is characterized by the existence of recoverable and irrecoverable deformations called the elastic and plastic deformations respectively. It is observed that there exists a yield surfacefor soils, where the response of the soil changes from elastic to plastic. For stress changes inside a chosen yield surface, the response is elastic. As far as the magnitude of the stresses increase, and the yield criterion is satisfied, the response of the material is that of elastic – plastic material. There are four basic requirements for an elastic-plastic model to be fully characterized. These are:

- Elastic properties
- Yield surface
- Plastic potential
- Hardening rule

Elastic properties defines the way in which the elastic deformations of the soil can be described. Yield function defines the combination of stress state which distinguishes the elastic and the plastic behavior of granular material. The plastic potential specifies the relative magnitudes of the various components of plastic deformation. Hardening rule relates the increment in stress with increment in strain. The definition of yield surface explains the condition at which the material subjected to external loading changes its response from purely elastic (linear and nonlinear) to plastic.

The elastic – plastic models bring out the actual response that the material undergoes in the field conditions, yet there is complexity involved in modelling it. It would be apt to quote the following which reflects the usefulness of elastic – plastic models and the importance of yield surfaces in such models, “To date, the most popular approach to stress-strain relations in soils was to assume the elastic behavior or, more recently, to apply the concepts of plasticity theory. There are obvious advantages if the plasticity theory is proved relevant as it provides a considerable simplification of mathematical treatment, and in particular the limit theorems would be of great practical value. However, for these idealized concepts to apply,first it is necessary to establish experimentally the existence of yield surfaces.” (Barden and Khayatt, 1966)

Pavement granular materials, which are particulate in nature, exhibit nonlinear and pressure dependent response even at low traffic loads. The layer properties are not constants but are the functions of the confinement pressure. Either majority of the investigations, characterizing the mechanical response of granular layers, have appealed to elastic-plastic models or resilient modulus based models. Studies by White *et al*. (1997), Helwany *et al*. (1998), and Sukumaran (2004) considered the elastic-plastic behavior of granular layers. Saad *et al*. (2005) studied the effect of elastic-plasticity of base and subgrade on the dynamic response of pavement systems by considering the granular base as elastic-plastic (Drucker-Prager type, Drucker and Prager (1952)) considered the subgrade as an elastic-plastic strain hardening (Cam-Clay type) material. Chazallon *et al*. (2006) developed a simplified method for the finite element modeling of the evolution of rut depth with time, based on shakedown theory. Kettil *et al*. (2007) used a elastic material model to describe the elastic strain and Drucker-Prager model was usedto describe the plastic strain as well as the strain hardening of the unbound granular layers. Studies by Hadi and Bodhinayake (2003), Park and Lytton (2004), and Kuo and Huang (2006) used resilient modulus models to characterize the stress-strain response of granular layers. The Drucker-Prager model (Drucker and Prager, 1952) is assumed to characterize the response of the material to a large extent. It behaves linearly elastic up to the yield stress, above which it behaves plastically. The yield surface expands with subsequent load applications. The important characteristic of the granular material is that it behaves elastically at lower stress levels and plastic at higher stress levels.

2.4 Permanent Deformation of granular material

Empiristic Pavement Design methods considers both fatigue and rutting failure criteria for flexible pavements (Puppala et al., 1999; Jegatheesan and Gnanendran 2015). The rutting occurs on the pavement surface due to the permanent deformation of pavement layers subjected to repeated wheel loads. It is assumed that base, subbase and Subgrade layers contribute for the permanent deformation of flexible pavement (Puppala et al., 1999). Proper design of flexible pavements demands accurate prediction of plastic strains in the granular layers. Improper design results in pavement failures, which leads to spending of billions of dollars for the rehabilitation of the pavement (Muhanna et al., 1998; Puppala et al., 1999; Puppala et al., 2009).

Over the past few decades, significant research has been reported in literature, which tried to consider the plastic deformation in pavement design (Smith et al., 1975; Lentz and Baladi, 1993; Korikanta and Dawson, 2007). In most of these studies, resilient modulus is heavily relied upon to accommodate the plastic response of soils. It is also reported in the literature that some of the subgrade soils with more fine content may possess good resilient characteristics, but will still exhibit excessive plastic deformations when subjected to repeated loads (Ullditz, 1993; Puppala, 1999; Lin et al., 2011). This observation clearly indicates the need to evaluate the permanent deformation behaviour of subgrade soils under different stress levels for complete characterization of the material. It is reported in the literature that for granular materials permanent strain accumulation is very slow under low cyclic stresses even at higher number of load cycles. However, at high stress levels the rate of accumulation of permanent deformation increases (Lekarp and Dawson 1998).Hence, permanent deformation of the subgrade soil is needed to be evaluatedalong with resilient modulus for complete characterization of the soil.

Rutting generally is seen as a groove or despondency on the surface of flexible pavements. It also providessteering difficulties for drivers. To overcome the problems due to rutting, we must understand the elasto-plastic performance of granular material under different loading conditions. This part summarizes and discusses the impact of individual factors on permanent deformation of granular material and constitutive models.

2.4.1. Factors affecting permanent strain

a) Effect of stress

Stress plays a vital role in accumulation of permanent deformation of granular material.

Morgan (1966) conducted repeated load triaxial test on granular material and observed that with increase in deviatoric stress, accumulation of permanent strain decreases and with respect to confining pressure, it was observed vice versa. Lashine et al. (1971) studied the behaviour of crushed stone under repeated load in drained condition and observed that the permanent strain is interrelated to the ratio of deviatoric stress to the confining stress. Similar results observed by Paute et al. (1996) for unbound aggregates. Leite et al. (2011), Gabr and Cameron (2012)confirms that the permanent deformation is directly proportional to the deviatoric stress and inversely related to the confining pressure even when recycled aggregates are replaced with virgin aggregates. Chauhan et al. (2008) observed same results for reinforced and unreinforced soils. Brown and Hyde (1975) observed similar permanent deformation for crushed stone with constant confining pressure (CCP) and variable confining pressure (VCP). Abdelkrim et al. (2006) argued that the permanent accumulation strain is more in VCP stage when compared to CCP stage due to the stress history.

Raymond and Williams (1978), Pappin (1979), Thom(1988) characterized permanent deformation with respect to static shear test and stated thatthestatic failure line is considered as a boundary for permanent strain under repeated loading. Lekarp and Dawson (1998) argued that as the accumulation of permanent strain was not a sudden process,it is not necessary to consider static failure stage in finding the permanent deformation of the granular material.

Aiban (2005) studied the behavior of geo-textile reinforced granular material and found out that the geo-textile reinforcement does not show any significant effect in the decrement of permanent deformation above 200kPa stress level. Chahuan et al. (2007) initiated the study of permanent deformation behaviour of stabilized silty sand with fly ash and fiber. They reported that nearly 21% decrement in permanent strain with silty soil+30% flyash stabilized with coir fiber is observed where as 18% percentage decrement is observed in permanent strain when stabilized with synthetic fiber. They also reported that the accumulation of plastic strain of soil (both stabilized and un stabilized soil) is directly proportional to the deviator stress level and inversely proportional to the confining pressure.

b) Effect of number of load applications

The accumulation of permanent strain is not a sudden process. Each increment of stress in repeated load triaxial test (RLTT) contribute considerable amount of permanent deformation, the number of load cycles are to be taken into account in finding the permanent deformation.

Morgan (1966) observed permanent deformation behavior of granular material under repeated triaxial test and found that it was increased even after applying two million load cycles. Barksdale (1972) correlated the permanent deformation with log number of repetitions and his studies showed that there was a sudden increase in permanent deformationat higher number of load repetitions. According to Brown and Hyde (1975) after 1000 load cycles there was no increase in the permanent deformation of well graded crushed granite. Lekarp (1997) and Lekarp and Dawson (1998) argued that for only lower stress levels, the limiting value permanent deformation occurred at a constant number of load repetitions whereas for higher stress levels, accumulation of permanent strain keeps on increasing. Yang et al. (2007) studied the long term behavior of cohesive subgrade soils and observed that the accumulation of permanent deformation was same from 10,000 cycles to 1,00,000 cycles. Abu-Farsakh et al. (2012) observed similar behaviour for stabilized granular base with geogrid.

Chauhan et al. (2008) studied the behavior of reinforced silty sand with fiber and reported that at 100 repetitions fiber reinforcement decreases permanent strain by only 1.35% whereas for 10,000 cycles it was 21%.Kumar et al. (2016) conducted a series of triaxial tests to observe the accumulation of plastic strain of unreinforced and fly ash-rise husk ash stabilized low plasticity clayey soil. They observed about 64% reduction in plastic strain when the soil is stabilized with fly ash where as 67% reduction is observed when the soil is stabilized with rice husk ash. One more interesting observation noted is that up to 1000 load cycles the rate of accumulation of permanent strain is following the same trend for both un stabilized and stabilized low plasticity clayey soil.

c) Effect of moisture content

The stiffness of granular materials depends on the amount of water present in it. Up to ample amount of water, stiffness increases and then decrease. According to Dawson et al. (1996), pore pressure increases and even permanent strain also increases due to the availability of free moisture in the pavement. Saevarsand Erlingsson (2015), who studied the pavement materials using Asphalt pavement test (A.P.T) track, obtained similar results.

Haynes and Yoder (1963) observed 100% more permanent strain when the degree of saturation was increased from 60% to 80%. Lin et al. (2011) also observed similar behavior for silty subgrade soil and stated that 5-20% incrementin optimum moistureis observed resulting in an increment of 300% in the permanent strain. Barksdale (1972) studied the permanent strain behavior of base course material and came up with the result that about 68% of permanent strain wasobservedfor soaked condition when compared to partially statured condition. Aiban et al. (2006) who studied the permanent strain behavior of geo textile observed similar results, cement stabilized subgrade, and observed that geo textile stabilized subgrade showed higher loading capacity under soaked condition compared to unsoaked condition.

Thom and Brown (1987) considered the effect of pore pressure and stated that the permanent strain is not only due to the increase in the pore pressure but also due tothe lubricant effect.

Abu-Farsakh et al. (2012)consideredthe effect of reinforcement on resilient and permanent deformation of base materials and observed that the effect of Geogrid at optimum moisture stage is more when compared to dry of optimum and wet of optimum. Abu-Farsakh et al. (2014) observed thatat OMC+2.5% stage, untreated granular material showed less permanent deformation than geo grid reinforced granular base materials at 100 load cycles. The reason is thatthe sample compacted at wet of optimum needs more load cycles to interlock due to dispersed fabric structure. Murab et al. (2014) investigated the effect of cement stabilization on deformation behaviour of weak subgrade soils and observed that the stabilization is more effective when the soil is compacted at wet of optimum condition than optimum moisture condition. They concluded that the effect of moisture and deviatoric stress levels are the driven forces for the accumulation of plastic strain.

d) Effect of density

Effect of density in granular material is predominantly expressed in terms of compaction effort. In general, rate of compaction is inversely proportional to permanent strain.

Barksdale (1972) tested several base course material and 185% increment in permanent strainis observedbetween 95% and 100% compaction effort. Allen (1973) conducted several repeated load tri-axial tests for granular base material and crushed lime stone and observed 80% reduction in strain for granular base material and 22% reduction for crushed lime stone when the samples were compacted using modified Proctor density instead of standard proctor density. It was supported by the Holubec (1969) and explained that rounder particles are less pounced than angular particles because of initial higher relative density.

Vanniekerk et al. (2000) observed that base and sub-base materialsmainly consists of crushed rock having same grain size because of which permanent deformation resistance increased with increase in compaction effort. Aiban et al. (2006) observed that as the density increases the geo-textile effect on load carrying capacity decreases. Lin et al. (2011) observed15% decrement in permanent deformation for silt subgrade when density increases from 91% to 95%. Bilodeau et al. (2008)found 41% less permanent deformation for aggregates when the compaction effort changed from 92% to 100%. Wayne et al. (2011) studied the behavior of geo-grid stabilized unbound aggregates and stated that the geo-grid reinforcement was effective in the decrement of permanent strain even when the lower layers of geo grid are compacted with less density compared to the top layers of geo grid.

e) Effect of fines content

Barksdale (1972), Thom and Brown (1988), Ekblad and Isacsson (2006)observed the effect of fines content on the granular material and found that with increase in fines content there was decrease in permanent deformation resistance.

According to Mishra et al. (2010) if the fines having plasticity index more than 10 it shows drastic effect on permanent deformation of unbound granular material. This was supported by the Cerni et al. (2012), who investigated on unbound granular material with calcareous fines (non plastic) and silty clay fines under similar stress and moisture and found that calcareous fines have less permanent strain compared to silty clay fines.

Mishra et al. (2012) observed that 8% fines are limiting value for crushed aggregates whereas 4% fines are limiting for uncrushed gravel and stated that fines effect is more pronounced in permanent deformation compared to resilient modulus. Soliman and Shalaby(2015)observed that 9% fines are optimum content for gravel material where as 4.5% for lime stone.

f) Effect of grading and aggregate type:

Dunlap (1966) observed a decrement in permanent deformation when the relative density is increased by grading under similar compaction effort. Thom and Brown (1988) observed minute change in permanent strain for heavily compacted poorly graded specimens whereas un compacted uniformly graded samples exhibits less permanent strain. Biodeau et al. (2008) observed that gradation not only influences the permeability but also the frost susceptibility. Konrad (2007) stated that the frost heave has considerable effect in permanent deformation only if the frost susceptibility is more than 1%. Bilodeau et al. (2008) statedthat the granular materials having lesser fine particle and finer mineralogy gradation exhibit greater influence on frost susceptibility.

Allen (1973) observed higher plastic strain accumulation for rounded particles compared to angular particles (crushed stone) with similar density and surface characteristics due to better interlocking in angular particles. Barksdale (1972) observed that blade shape crushed aggregates are more pronounced to permanent strain accumulation compared to other shapes of crushed aggregates. Rao et al. (2002) believed that crushed particles exhibit higher resistance to accumulation of permanent strain compared to uncrushed particles due to the load distribution capacity. According to Mishra et al. (2012), for angular particles having fines content less than 8%, amount of voids plays a vital role in accumulation of permanent deformation

g) Effect of principal stress reorientation

Principal stress reorientation plays a predominant role in soil stiffness. The concept of reorientation principle is difficult to understand, as the lab behavior is not simulating the field conditions.

Youd (1972) observed that whenever a rotation in principal stress axes occur, there was an increase in density for sands in shear box. Ansell and Brown (1978) studied the behavior of crushed limestone and stated that the density is directly associated with the amount ofobserved cyclic shear. Chan (1990) reported that the limestone showed lesser resistance to permanent strain compared to no shear stress condition when shear stress is used in the cyclic tests. From the results, we can observe that the bi-directional shear stress pronounce lesser resistance to the accumulation of permanent strain compared to unidirectional shear stress.

h) Effect of stress history

For granular materials, stress history is directly associated with the accumulation of permanent strain. Brown and Hyde (1975) observed that the immediate application of higher stress pronounced more plastic strain compared to successive application of stresses due to which materials get stiffer for each load application up to limited number of load cycles.

CEN 2004 introduced a multi stage repeated load tri axial test (M.S.R.L.T) procedure to determine the accumulation of permanent strain in granular material. M.S.R.L.T test procedure is applied on a single specimen in several stress paths and the test results can be related to field condition due to the inclusion of stress history. Rahman and Erlingsson (2014) studied the behavior of accumulation of strain using M.S.R.L.T test procedure and revised Gidel (2001) model and VTT model (2005) according to M.S.R.L.T Procedure.

Abu-farakh et al. (2015) studied the plastic response of five different soils in Louisiana with cement and lime stabilization and concluded that cement stabilization was more effective for silty and sandy soils whereas lime stabilization was suitable for high plasticity soils. Patel and Sety (2016) used steel slag, fly ash and dolomite mix as base course material. They observed about 83% reduction in plastic strain when compared to standard wet mix macadam.

2.4.2. Regression Models to predict permanent strain

Lentz and Baladi (1981) conducted repetitive triaxial test on sand. Based on theobservations, heproposed a model to find out the permanent deformations with the number of load cycles and difference between the principalstresses to co relate with the static triaxial test results.

$$\varepsilon_{1,p} = \varepsilon_{0.95s} ln\left(1 - {}^{q}/_{s}\right)^{-0.15} + \ln(N)\left\{\frac{a\left({}^{q}/_{s}\right)}{\left[1-b\left({}^{q}/_{s}\right)\right]}\right\} \qquad 2.10$$

Jouve et al. (1987) used an innovative method to derive the permanent deformation model using volumetric strain and shear modulus. He not only considered stresses but also load cycles to propose this equation.

$$K_p(N) = \frac{P}{(N)e_{v,p}} G_p(N) = \frac{q}{3e_{s,p}(N)} \qquad 2.11$$

$$G_p = \frac{A_2\sqrt{N}}{\sqrt{N}+D_2} \frac{G_p}{K_p} = \frac{A_3\sqrt{N}}{\sqrt{N}+D_3} \qquad 2.12$$

Barksdale (1972)conducted several cyclic triaxial tests on base course material in order to find the relation between the number of load applications and permanent deformation and proposed a relation with respect to the log-number of repetitions.

$$\varepsilon_{1,p} = a + blog(N) \qquad 2.13$$

Sweere(1990) performed large number of cyclic triaxial tests on granular base materials using 10,00,000 load repetitions. He didn't agree regression equation which was proposed by Barksdale (1972)proposed another equation which is in log-log form as per equation 2.14.

$$\varepsilon_{1,p} = aN^b \qquad 2.14$$

Wolff and Visser (1994) studied the long-term deformation behavior of the material using heavy vehicle simulator (HVS) and differentiated the accumulation of plastic strain into two stages. In the first stage, quick development was seen in plastic deformation but rate of deformation constantly decreased up to 12, 00,000 repetitions. In the second stage, plastic strain development is very slow and there is no development in deformation rate. Log-Log model didn't satisfy the results and hence below model was proposed to calculate the accumulated strain.

$$\varepsilon_{1,p} = (cN + a).(1 - e^{-bN}) \qquad 2.15$$

Khedr(1985)observed permanent strain accumulation of crushed limestone by using cyclic triaxial test and proposed an equation using number of load cycles under variable confining pressure (VCP).

$$\frac{\varepsilon_{1,p}}{N} = A_1 N^{-b} \qquad 2.16$$

Paute et al. (1988) observed continuous increment in permanent accumulated strain towards asymptotic value. Paute proposed an equation without considering first 100 load cycles.

$$\varepsilon^*_{1,p} = \frac{A_4\sqrt{N}}{\sqrt{N}+D_4} \qquad 2.17$$

In the separate study Paute et al. (1996) expressed an equation regarding permanent strain with respect to first 100 load repetitions.

$$\varepsilon = A * \left(1 - \left(\frac{N}{100}\right)^{-b}\right) \qquad 2.18$$

According toLekarp and Dawson (1998), this model was considered not fit for higher stress conditions.

We are familiar with the point that permanent deformation is not a sudden process but it is gradual accumulation at the applied stress level. Stress plays a vital role in the deformation of granular material.

Barksdale (1972) proposed an equation related to the permanent deformation by using cyclic triaxial test based on the repetitive deviatoric stress at constant confining pressure stage (CCP).

$$\varepsilon_{1,p} = \frac{q / a\sigma_3^b}{1 - \left[\frac{(R_f q) / 2(c \text{ Cos } \theta + \sigma_3 \text{ Sin } \theta)}{1 - \text{Sin } \theta}\right]} \qquad 2.19$$

He proposed this equation using hyperbolic equation, which was proposed by Duncan and Chang (1970) for static triaxial test for static test. Sweere(1990) validated the Lentz and Baladi model for different granular material and observed that this model is not suitable for other granular material.

Lashine et al. (1971) carried cyclic triaxial test at 20,000 load cycles with different stress levels and proposed below model.

$$\varepsilon_{1,p} = 0.9 \frac{q}{\sigma_3} \qquad 2.20$$

Pappin (1979) observed plastic strain behavior of limestone in the cyclic triaxial test and proposed an equation for accumulated permanent strain with respect to the length of stress path.

$$\varepsilon_{s,p} = (fnN)L\left(\frac{q^0}{p^0}\right)_{max}^{2.8} \qquad 2.21$$

Pappin (1979) stated that higher permanent strain occurs at static failure stage only.

Paute et al. (1996) proposed a constant value for permanent strain and it changes with maximum shear stress ratio.

$$A = \frac{\frac{q}{(p+p^*)}}{b\left(m-\frac{q}{(p+p^*)}\right)} \qquad 2.22$$

Theyse (2002) proposed an equation for permanent deformation with respect to the number of load cycles using heavy vehicle simulator (HVS)

$$\varepsilon_{1,p}(\%) = (m.N) + (A_2)(1 - e^{-B_2N}) \qquad 2.23$$

Tseng and Lytton (1981) correlated the resilient and permanent deformation with respect to the number of load cycles. This model is used for base, subbase and subgrade also.

$$\varepsilon_p = \beta_1\left(\frac{\varepsilon_0}{\varepsilon_r}\right)\varepsilon^{-\left(\rho/N\right)^{B}}\varepsilon_v \qquad 2.24$$

VTT model is used to determine the accumulated permanent deformation in unbound granular material (UGM) and this model is derived by Korkiala-Tanttu (2007)based on the Mohr's -Columb theory by considering the number of load cycles and stresses.

$$\varepsilon_p(N) = BN^b\,\frac{R}{A-R} \qquad 2.25$$

$$R = \frac{q}{q_f} = \frac{\sigma_1-\sigma_3}{q_o+Mp} \qquad 2.26$$

$$M = 6.\mathrm{Sin}\,\phi/3 - \mathrm{Sin}\,\phi\; q_o = c.6.\mathrm{Cos}\,\phi/3 - \mathrm{Sin}\,\phi \qquad 2.27$$

Gidel (2001) conducted several cyclic triaxial tests on UGM by varying number of load cycles, stress rate, and proposed a model using maximum deviator stress and mean stress.

$$\varepsilon_p(N) = \left[1 - \left(\frac{N}{100}\right)^{-b}\right]\left(\frac{L_{max}}{p_a}\right)^{u}\left(m + \frac{s}{p_{max}} - \frac{q_{max}}{p_{max}}\right) \quad 2.28$$

$$L_{max} = \sqrt{p_{max}^2 + q_{max}^2} \quad 2.29$$

Veverka (1979) performed several cyclic triaxial tests on granular materials with respect to the number of repetitions in order to determinethe resilient deformation and plastic strain. He correlated resilient modulus and permanent deformation and proposed another model.

$$\varepsilon_{1,\mathrm{p}} = a\varepsilon_r N_b \quad 2.30$$

Yang et al. (2008) performed cyclic triaxial tests on cohesive soils and derived a model based on the influence of stresses and load cycles with different moisture conditions with respect to resilient modulus.

$$\varepsilon_p = A \times SL^b \times \left(\frac{M_r}{M_{r,i}}\right)^c \times N^D \quad 2.31$$

Mohammad et al. (2006) studied both resilient and plastic deformation on treated as well as non-treated base material and correlated with the below equation.

$$M_R = 225\varepsilon_p^{-0.25} \quad 2.32$$

Mohammad et al. (2004) worked on the multi stage repeated load triaxial test (MS RLT) to study the long-term performance of granular material whereas Shu-Rong et al (2007) modified the gidel model and VTT model according to MS RLT condition.

These models are validated using different density, moisture conditions, stress levels and grading.

Modified VTT model:

$$\varepsilon_p(N) = (N - N_{i-1} + N_i^{eq})^{-b}\,\frac{R}{A-R} \quad 2.33$$

Modified GIDEL model

$$\varepsilon_p(N) = \left[1 - \left(\frac{N - N_{i-1} + N_i^{eq}}{100}\right)^{-b}\right]\left(\frac{L_{max}}{p_a}\right)^n \left(m + \frac{s}{p_{max}} - \frac{q_{max}}{p_{max}}\right) \qquad 2.34$$

Modified Tseng and Lytton (1989):

$$\varepsilon_p = \beta_1 \left(\frac{\varepsilon_0}{\varepsilon_r}\right) \varepsilon^{-\left(\rho / N - N_{i-1} + N_i^{eq}\right)^{B}} \varepsilon_v S \qquad 2.35$$

2.5 Characterization of permanent deformation behavior of granular materials by shake down theory

For longdurations under repeated loading, materials do not behave in a unique way up to failure. They follow different stress paths even if all the other conditions are same but for the stress state. Even if a material is seemed to have a propensity to attain equilibrium (constant rate of plastic strain accumulation) under a definite load after a certain number of cycles altering the stress state might cause it to behave quite differently which means there is certain limit of stressfor every material above which, after a number of cycles, the material becomes unstable. This limit in literature, called the “shakedown limit” or the “critical stress level”. ‘Stress level’ indicates the ratio of the given deviator stress to the deviator stress at failure determined from static triaxial tests. Four such stress limits have been determined but only three of them are applicable to granular materials. Determination of these stress levels, then, becomes extremely important in rutting prediction models and can even question the existence of the present day design procedures.

Pauteet al. (1996) observed that the rate of permanent strain accumulation with increasing load cycles decreases constantly to such an extent to define a limit value for the permanent strain. On the other hand, Morgan (1966), Barksdale (1972) and Sweere(1990) reported continuous increment in permanent strain under repeated loading.Maree (1982)also found that the permanent strain increased at a constant rate for high stress states but between 5,000 and 10,000 load cycles an exponential increase in permanent strain occurred resulting in failure within a short duration.Work by Kolisoja (1998) involving a very large numbers of cycles, stated that the development of permanent deformation may not be expressible as a simple function. That is where Shakedown Theory comes in to play.

This demarcation of stress levels is done with the help of a theory developed by Melan (1936) called the Shakedown theory. While limit analysis is used to solve Instantaneous collapse, shakedown analysis could be used for analysing failure under repeated load applications. This theory was used for design and assessment of structures operating beyond elasticity. Sharp and Booker (1984) successfully implemented this theory for pavement applications to find out the deformation behaviour of granular materials under repeated loading. Werkmeister et al., (2004) took a different approach to classify the plastic strain behaviour of granular materials using ratio of plastic strain to plastic strain rate.

According to classical shakedown theory, there are four different material responses under cyclic/repeated loading. They are (a) purely elastic, (b) elastic shakedown, (c) plastic shakedown (or plastic creep) and (d) incremental collapse (or ratcheting). The purely elastic phase has no plastic strains at any point of time. The stress strain curve is linear with no energy dissipation. This phase can be ignored for granular material like soil as it not an elastic material and will retain some amount of strain in any case. In the second phase i.e. elastic shakedown phase, stress-strain curves initially exhibit hysteresis loops up to a certain number of cycles, but eventually getthinned down to straight elastic line with no more plastic strain accumulation. This behaviour will be observed under low cyclic stress levels where after some repetitions the response becomes completely resilient. The material is then said to be “shaken down” or as having attained shakedown. The stress level beyond which the material ceases to behave this way is called the shakedown limit or range A behaviour. At very high stress levels, the material response become unstable.

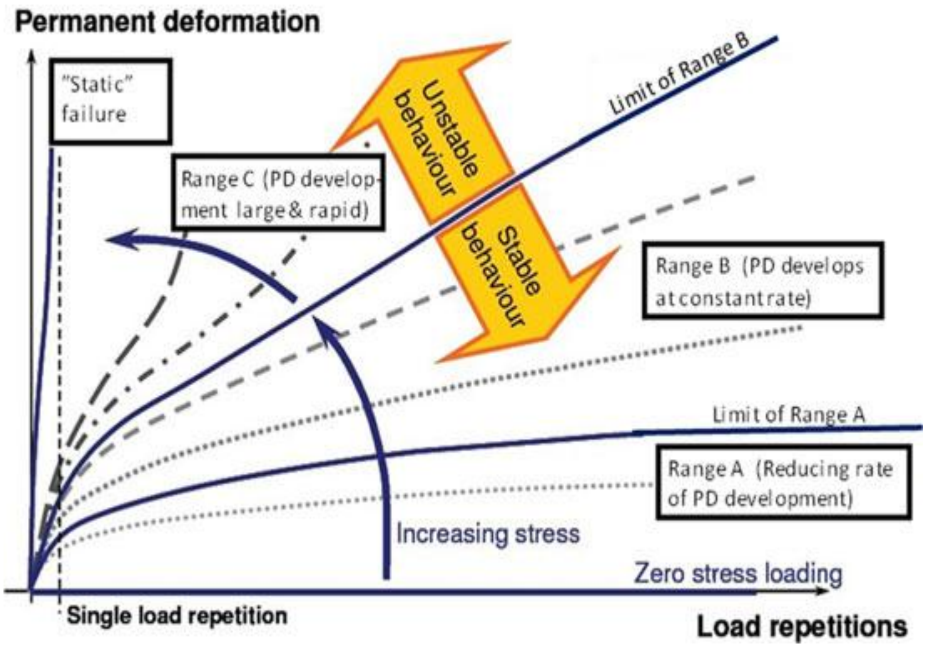

Figure 2.2. Shakedown behaviour by range (adapted from Theyse(2007); PD = Permanent deformation).

With load application, the material quickly reaches failure point with an exponential increase in plastic strain. At this stage, there is no cessation of strain accumulation. This is called incremental collapse or ratcheting, and is represented as Range C. These stress levels should never be experienced in a pavement subgradeas it rapidly leads to failure. At stress levels between the elastic shakedown and ratcheting phase, the plastic strain accumulates with the number of loading cycles. It is a transitional stage between the elastic shakedown and Incremental collapse stage. In this range, after a number of cycles, instead of the stress-strain loop getting thinned away into a line as in elastic shakedown, the loop gets completely closed with the net plastic strain in that cycle being zero. This behaviour is denoted as Range B. This range may be acceptable in low volume roads where maintenance will be executed at frequent intervals.

Dawson and Wellner (2004) have determined critical stress level using Shakedown approach by plotting plastic strain rate vs. plastic strain and it is represented in figure 2.3. They were able to identify two types of curves in those plots and demarcated between them to fix a critical stress level.

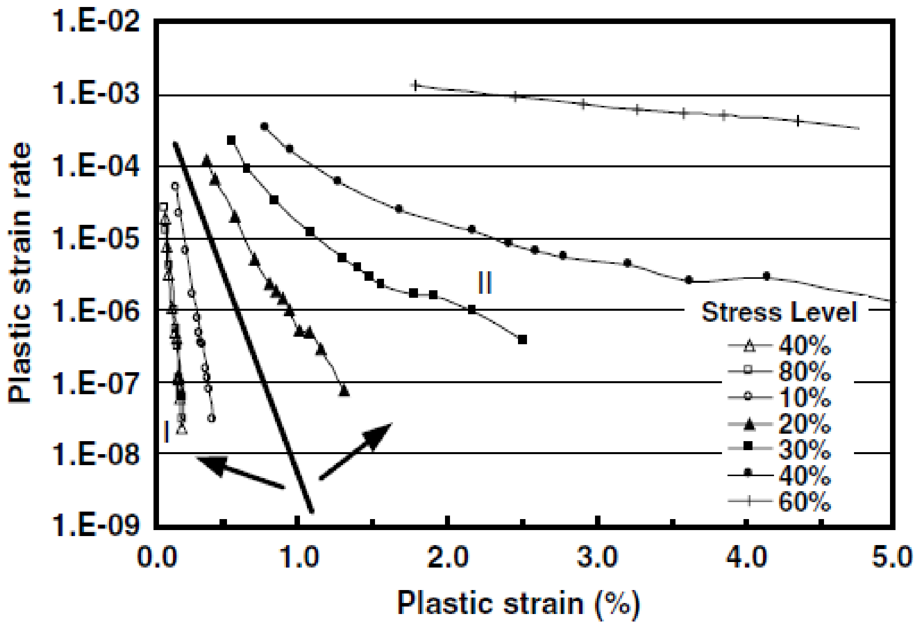

Figure 2.3 Relationship between plastic strain to plastic strain rate (%)

From the figure 2.3, Type 1 implies a steadily decreasing plastic strain rate. They will be stable under repeated loading. They will be “shaken down”. The curves close to the boundary line represents plastic creep stage. Type II region (where curves are concave upward) exhibits ratcheting behaviour where plastic deformation is accumulated rapidly. In figure, the critical stress level is in between 10% and 20%.

Werkmeister (2004) based on his research, has proposed a method to determine the shakedown range of unbound granular materials using RLT test data. This has also been incorporated in to the European standard (CEN, 2004).

$$\text{Range A:} \epsilon_{3000} - \epsilon_{5000} < 0.045 \times 10^{-3},$$

$$\text{Range B: } 0.045 \times 10^{-3} < \epsilon_{3000} - \epsilon_{5000} < 0.4 \times 10^{-3,}$$

$$\text{Range C: } \epsilon_{3000} - \epsilon_{5000} > 0.4 \times 10^{-3}$$

Where ϵ_{3000} and ϵ_{5000} are the plastic strains at 3000 and 5000 load cycles respectively in the cyclict riaxial test.

Dawson and Huvstig (2014) studied the shakedown approach and they have selected some prediction models, calibrated and validated them. The models includes the VTT model developed by Korkiala-Tanttu at the VTT Research Centre of Finland and the Gidel Model. The VTT model related permanent strain to stress ratio and number of loading cycles. This model includes a material parameter and a stress state parameter. The Gidel model, on the other hand, has three parameters, which have to be determined from the RLT test data and validated.

Rahman and Erlingsson (2013) conducted Multistage RLT tests instead of Single stage RLT tests to find out the critical stress levels. In the Single Stage RLT tests (SSRLT), a cyclic loading of constant amplitude is used. To study the influence of various stress levels on the material response, different specimens have to be used. On the other hand, a Multi Stage RLT test can apply cyclic loading of different magnitudes on the same specimen. This arrangement simulates field conditions better than SSRLT tests since repeated loadings of varying magnitudes are common in the pavements. Effect of stress history on the material behavior will also be considered in this approach. Research works were mainly done using SSRLT and therefore prediction models are based on the results from them. Rahman and Erlingsson (2015) have extended these prediction models for permanent deformation based on the results they obtained from MSRLT tests, by the application of the time hardening formulation.

2.6 Summary of literature review

Based on the literature review it can be concluded that very less studies related to the resilient and permanent deformation behaviour of chemicallystabilized soils have been carried out in the past. Consequence of calcium carbide stabilization on resilient and permanent deformation of clayey soils are not found in the literature. As stabilization of weak subgrade soil will be most important in all major transportation infrastructure projects throughout the world and calcium carbide residue is an industrial waste product which has the potential to be used as a stabilizing agent; more research has to be carried out focussing this topic. In the present research work it is proposed to investigate the resilient and permanent deformation behaviour of clayey soil treated with calcium carbide residue and to study the effects of different factors like moulding moisture contents, cyclic deviatoric stress levels and confining pressure on the behaviour by conducting repeated load triaxial tests.

Chapter 3

Experimental Program

3.1 Introduction

In order to accomplish this research, basic properties, engineering properties, resilient modulus and permanent deformation behaviour of clayey soils, stabilized soils, virgin aggregates and recycled aggregates are to be used for flexible pavement design. Effect of moisture content, confining pressure and deviator stress levels on the resilient and permanent deformation behaviour of a clayey subgrade soil was investigated within the framework of shake down theory. Basic properties of two different clayey soils, virgin and recycled aggregates were determined.

3.2 Experiments

3.2.1 Specific Gravity

Specific gravity test was performed by density bottle method by following the guidelines presented in IS 2720 (part I). Test was performed for five times and the average value was noted.

3.2.2 Particle size distribution:

The particle size distribution characteristics of the samples were assessed as per IS 2720 Part4(1985). Sieve analysis is performed to determine the distribution of the coarser particles and the hydrometer method is used to determine the distribution of the finer particles.

3.2.3Atterberg Limits

Experiments to determine theAtterberg limits were carried out on air-dried soil passing through 0.425 mm sieve. The soil sample was mixed with the required amount of water and wrapped in vinyl bags for curing at 25^0 C prior performing experiments to achieve equal spreading of water. Tests were performed as per IS 2720 Part 5 (1985).

3.2.4 Compaction test

The procured soils were dried and pulverized and experiment was carried out using soil fraction finer than 4.75mm. For finding the optimum moisture content and maximum dry density for the binder-amended soil, the standard Proctor test was conducted.These tests were conducted as per IS: 2720 Part 8 (1980) on various mixes which were prepared on the basis of dry weight. The mouldwith standard volume of 1000cc was used and the material was compacted by 25 blows in three layers. Standard hammer of 4.9kg weight falling from a height of 45 cm was used for compaction and the test was repeated with anincrease in water content. Dry density was calculated for all water contents to obtain the compaction curve. The water content at maximum dry density was considered as optimum moisture content.

3.2.5 Unconfined compressive test

Soil was mixed thoroughly with the required amount of CCR and wrapped in vinyl bags for curing at 25^0 C. UCS tests and CBR tests were carried out on these cured samples after 7 and 28 days. UCS tests were performed as per IS 2720 Part 10 (1991). Specimens used in this study were having a diameter of 38mm and height of 76mm and were prepared at maximum dry density and optimum moisture content.

3.2.6 California bearing ratio

CBR tests were carried out as per IS: 2720 Part 16 (1973)**.** A surcharge of weight 25N was used throughout the testing. The rate of loading 1.25mm/min was applied to the specimen up to the failure of thespecimen. Metalplunger of 50mm in diameter and 100 mm in height was used.The soils used in these tests were dried and pulverized and experiment was carried out using soil fraction finer than 4.75mm.

3.3 Materials

3.3.1 Black cotton soil:

Black cotton soils (BC) of India are well known for their expansive behavior. In India, Black cotton soil covers around 0.7 million square kilometres (i.e., 20-25 % land area approximately). Black cotton soil consists of montmorillonite mineral and it possess challenges to civil engineers worldwide due to the high swelling and shrinking behaviour exhibited by them and it is found mainly in Deccan plateau. The Black cotton (BC) soil was collected from a construction site in Warangal,Telangana,India. The soil sample was dried and stored in containers, and the required amount was pulverized and used for different experiments. Index properties of soil were presented in Table 3.1.

3.3.2 Red soil

Red soil is generatedfrom rich sedimentary rock. Red soil nearly occupies 6.2-lakh km in India and it is second largely available soil in India after Black cotton soil. Locally available subgrade soil commonly known as red earth, used in the present study was collected from Warangal, Telangana, India. Soil was air dried, crushed and sieved through 4.75 mm sieve. The soil is collected by open excavation, from a depth of 1 meter from natural ground level. Index properties of soil presented in Table 3.1.

Table 3.1 Index properties of BC soil and Red soil

S.NO	Properties	BC soil	Red soil	IS codes
1	Specific gravity	2.65	2.59	IS-2720 part 3 (1980)
2	Grain size analysis Gravel (%) Sand (%) Silt (%) Clay (%)	 4 28 39 29	 0 44 32 24	IS-2720 part 4 (1985)
3	Liquid Limit(LL)	59	42	IS-2720 part 5 (1985)
4	Plastic Limit(PL)	18	22	
5	Plasticity Index(PI)	41	20	
6	IS Soil classification	CH	CI	IS-1498 (1970)

3.3.3Natural and Recycled aggregate

Recycled aggregates is the demolished concrete waste from old buildings and both natural and recycled aggregates procured from NIT Warangal campus. Subbase material prepared using according MORTH specification. Properties of subbase materials presented in table 3.2.Table 3.3 represents the elemental chemical composition of BC soil, Red soil and CCR. The chemical composition of the clays was evaluated with the help of the X-ray fluorescence (XRF) spectroscopy technique. A Phillips PW 2404 X-ray fluorescence spectrometer was used for the XRF analysis. The chemical composition of BC soil and Red soil tentatively indicates the mineral present in those soils. the higher amount of silica to alumina ratio in case of BC Soil suggests the presence of 2:1 mineral present vice versa the 1:1 mineral presence in case of red earth. The red colour to the red earth is attained due to the presence of higher amount of iron oxide. The presence of 40% of Calcium in CCR encouraged to consider the usage it as a potential stabilizer to improve the soil engineering behavior.

Table3.2Properties of subbase materials

S.No	Properties	Value	
1	Liquid Limit(LL)	22	
2	Plastic Limit(PL)	16	
3	Plasticity Index(PI)	6	
4	Optimum moisture content (%)	8 (Natural aggregate)	9.8 (Recycled aggregate)
5	Maximum dry density (g/cc)	2.32	2.15

3.3.4 Calcium carbide residue

Calcium carbide residue (CCR) is a by-product of acetylene gas industry and was collected from acetylene factories from a local source in India.CCRis usually dumped in the landfills leading to environmental problems because of its alkalinity. This increasing large quantity of stockpiled CCR by-products has resulted in serious environmental pollution (Du *et al*. 2011). Re-use application for CCR in soil stabilization has been identified as a low-carbon and low energy intensive means to rapidly deplete the growing stockpiles and furthermore, eliminate negative environmental connotations associated with stockpiling this by-product. Elemental composition of BC soil, Red soil and CCR is presented in Table 3.2. Figure 3.1 represents the particle size distribution of soils and CCR.

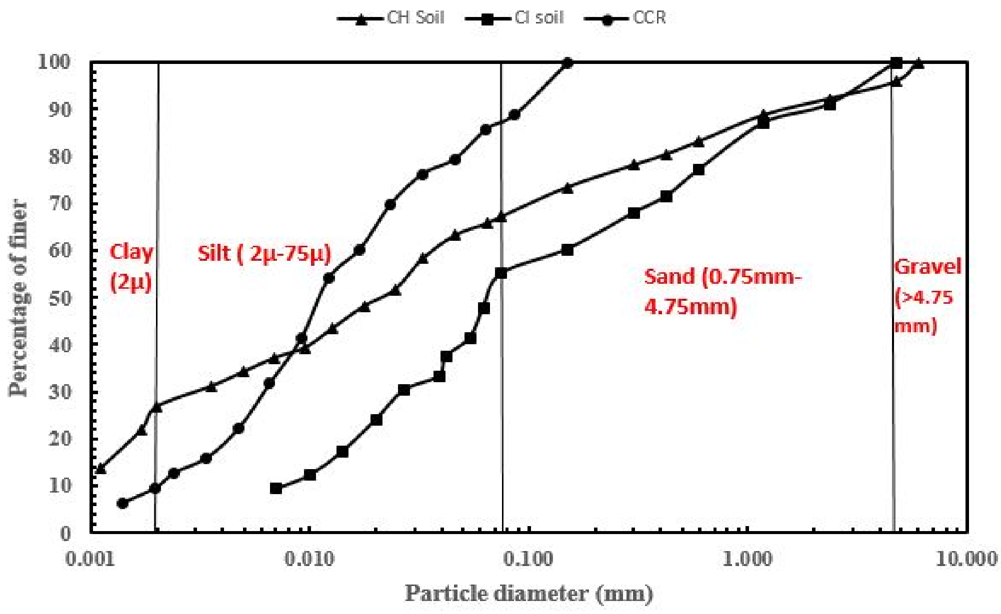

Figure 3.1 Particle size distribution CH soil, CI soil and CCR

Table 3.3 Elemental chemical composition of BC soil, Red soil and CCR

Constituent	BC	Red soil	CCR
Silica (SiO_2)	79.93%	60.24%	40.7%
Alumina (Al_2O_3)	10.59%	17.7%	10.17%
Ferrous (Fe_2O_3)	5.07%	18 %	8.04%
Calcium (CaO)	1.05%	2.1%	40%
Titanium (TiO_2)	0.54%	0.4%	0.65%
Magnesia(MgO) Sodium (Na)	2.11% 0.6%	1.4% -	0.44% -
Potassium (K)	1.11%	0.25%	-

3.4 Laboratory Testing Program

Repeated load triaxial (RLT) tests are the most preferred tests to determine the permanent deformation characteristics of unbound granular material (Lekarp 1999, Uthus 2007, Rahman and Erlingsson 2015). In the present study, an automated pneumatic cyclic triaxial apparatus with servo control and data acquisition system was used to determine the Resilient and permanent deformation characteristics of the soil. Figure 3.2 shows schematic figure of triaxial set up. Two-way cyclic loading replicates the occurrence of principal stress reversal caused by compression and extension as in the field, whereas one way cyclic loading deals only with compression (Andersen et al., 1980, Pillai et al., 2011, Puppala et al., 2009). In the present investigation, one-way cyclic loading is used to simulate the repeated loading on pavements.

Figure 3.2 Schematic diagram triaxial test

3.4.1 Specimen preparation

All the monotonic and repeated triaxial tests were carried out on soil samples with 75 mm diameter and 150 mm height. Prior to the preparation of specimens, dry soil was thoroughly mixed with the required amount of water and was kept in desiccators for 24 h to obtain uniform moisture content. The soil was then statically compacted in a cylindrical mould to obtain samples with the required density and moisture content. The inner surface of the mould was lubricated to reduce the side friction during compaction. After moulding, the specimens were immediately extruded from the split mould and then placed in plastic bags and stored in desiccators to avoid significant variations of moisture content before testing.

3.4.2Testing procedure for permanent deformation

A pneumatic actuator connected to a compressor through regulators and servo controlled valves was used to apply the required loads. Cyclic stresses were applied on the specimens in the form of haversine pulses with a frequency of 1 Hz. The specimens were tested at constant confining pressure of 20kPa and at seven different deviator stress levels (SL) (Yang and Huang 2007, Yang and Chang 2016), to assess the effect of confining pressure and deviator stress on resilient characteristics and permanent deformation. Stress level is defined as the ratio of cyclic deviator stress to ultimate stress at failure obtained from the static triaxial test. The moisture content of subgrade soils was found to increase by about 20 percent above the optimum moisture content over a period of two years after the completion of subgrade construction (Lin et al., 2011). In order to consider the moisture variation, in the present study the soil specimens were prepared at three different water contents (OMC, OMC+2% and OMC+4%) and the corresponding dry densities obtained from modified proctor compaction tests were adopted. Each samplewas subjected to 10,000 cycles of compression load under unconsolidated undrained conditions. The deformations were measured with the help of two high resolution linear variable differential transducers (LVDTs) with a precision of 0.01 mm. A submersible load cell with 5kN capacity was used to measure the load acting on specimen. A data acquisition system with a dedicated software was used to save the data to a computer.

3.4.3Resilient modulus test procedure

Resilient modulus test was conducted as per AASTHO T-307 (AASHTO-T 307. (2003)). for both clayey soils and stabilized soils and it is represented in Table 3.3. According to AASTHO T-307, 15 stress level combinations were applied on the specimens. The Resilient modulus test was performed with different deviatoric stress levels which consists of five deviatoric stress levels (13.8 kPa to 68.9 kPa) under three different confining pressures of 13.8, 27.6 and 41.4 kPa. For each stress level combination, 100 load cycles were applied and the average of last five cycles were taken for determining the resilient modulus.

Table 3. 4 Resilient modulus testing protocol based on AASTHO T-307

Confining pressure (kPa)	Deviator stress (kPa)	Number of load cycles
27.6	41.4	500
41.4	13.8	100
	27.6	100
	41.4	100
	55.2	100
	68.9	100
27.6	13.8	100
	27.6	100
	41.4	100
	55.2	100
	68.9	100
13.8	13.8	100
	27.6	100
	41.4	100
	55.2	100
	68.9	100

3.4.4 Laboratory pavement model test

The modifed repetitive loading testing facility is fabricated at NIT Warangal geotechnical engineering laboratory to evaluate the resilient modulus of the pavement material under simulated field conditions. The modified repetitive loadign testing facility is shown in figure 3.3.

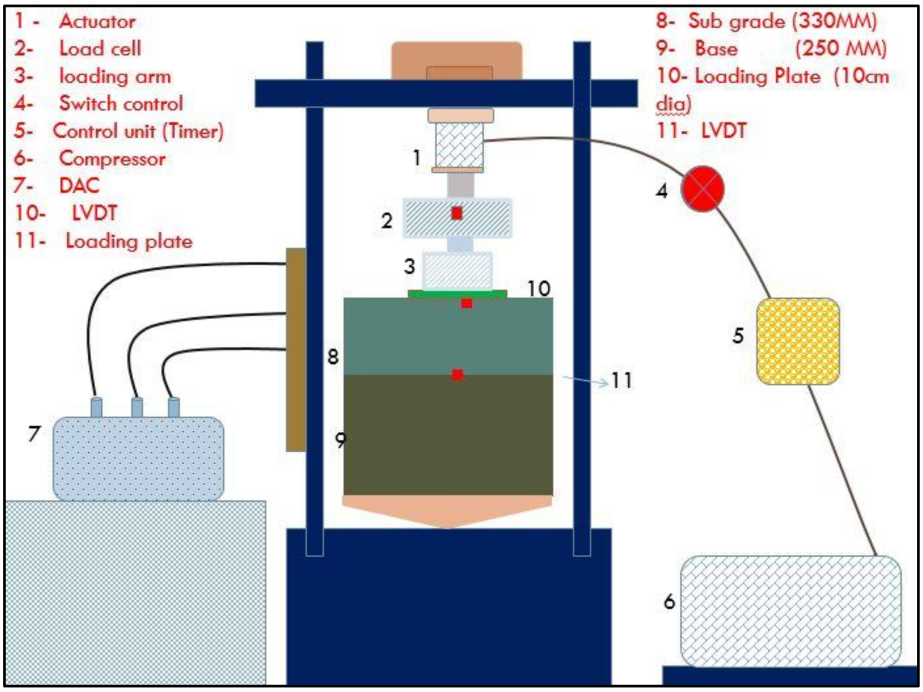

Figure 3.3 Schematic diagram laboratory pavement model test

The testing facility contains loading frame where the actuator is fitted to the instrument, which will apply the repetitive loading with the frequency of loading 5 sec for each loading repetition. The loading cell with the capacity of 50 kN is attached at the bottom of the actuator and it is connected to the DAS (Data actuation System) on the top of the loading arm. To regulate the flow in to the actuator the solenoid valve is provided and it is connected to the control unit where air flow is regulated through the solenoid by acting as on/off switch. To measure the deformation of laboratory simulated pavement under repetitive loading conditions, LVDT is arranged on the top of the testing tank and it is connected to the DAS.

3.4.5 The working mechanism

The modified repetitive loading equipment is working under air pressure under regulated flow conditions through the solenoid value by means of actuator. The air pressure is created in the actuator from air storage tank (compressor). The airline from the compressor is connected to the solenoid value and this solenoid value is connected to the switch control through which solenoid value is operated through the loading frequency timer. At the time of the operational condition, the air flows from compressor to the solenoid valve through the flexible pipe connection. From the solenoid valve, it is regulated and passed to the actuator. The actuator is applying the repetitive pressure of 550 kPa on the loading plate, which is having the diameter of 10cm. the repetitive load is applied on the loading plate, which is equal to the single axial load on pavement. Under applying load, the corresponding deformation is measured by using the LVDT which is connected to the DAC.

3.5 Methodology

The experimental program is divided in to five phases. The first phase deals with evaluating the engineering properties of clayey soils and CCR stabilized clayey soils. The second phase deals with studying the effect of water content, deviatoric stress and confining pressure on resilient modulus of clayey soils and CCR stabilized clayey soils. The third phase deals with understanding the effect of water content and deviatoric stress on permanent deformation of clayey soils and CCR stabilized clayey soils under constant confining pressure. The fourth phase deals with determining the resilient and permanent deformation characteristics of natural and recycled aggregates. The last phase i.e., deals with developing a permanent deformation Laboratory Pavement model with different subgrade layers and it is presented in flow chart 3.4

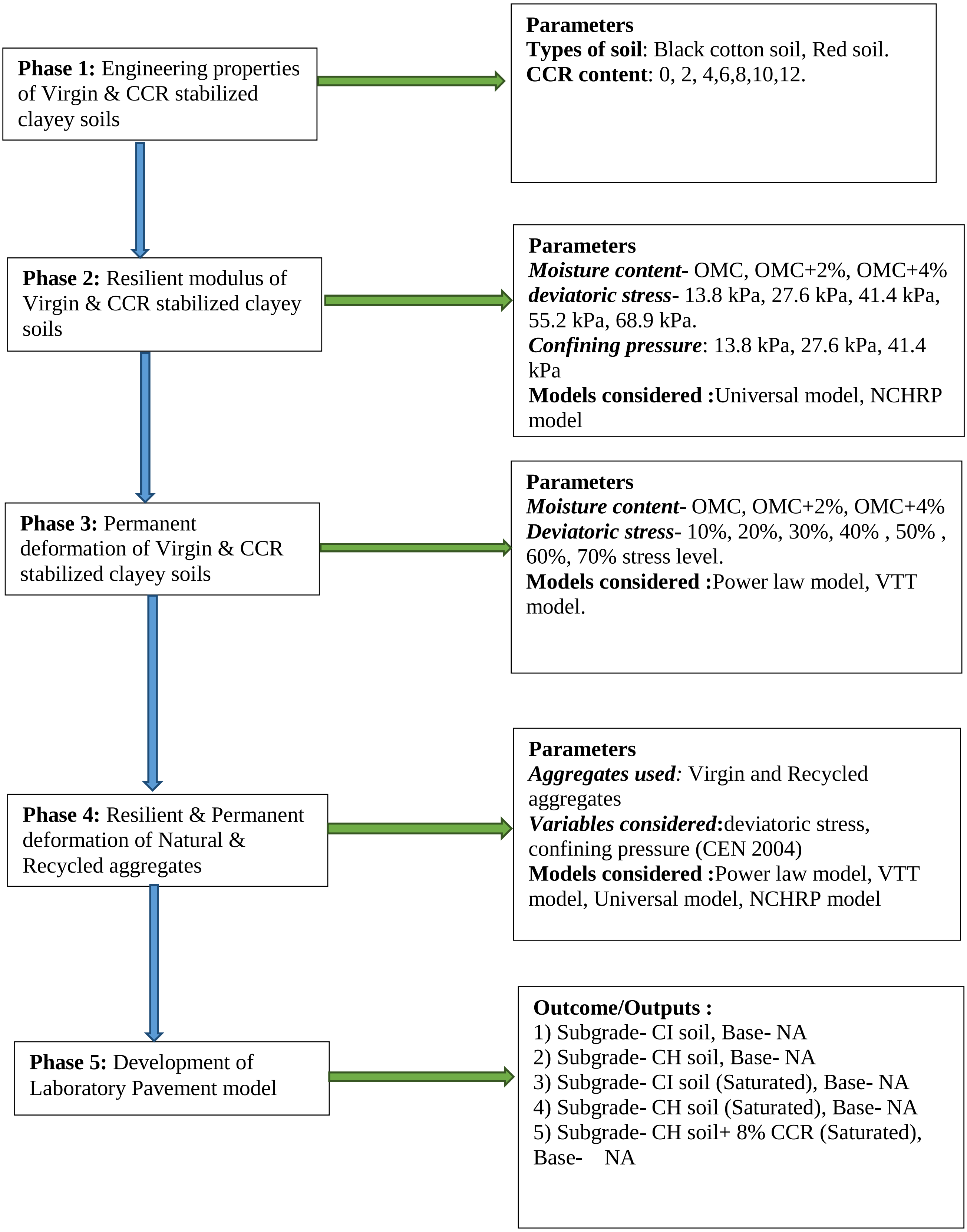

Figure 3.4: Flow Chart of methodology

3.6 Micro structural analysis

X-ray diffraction (XRD) analysis and scanning electron microscopy (SEM) of the mixture of BC soil, Red soil and CCR was carried out to study the mineralogical and micro-structural alterations occurring in the soil due to the stabilization with additives.Theresultswere compared with the PCPDF database. X-ray diffraction technique was carried out with the help of PANanalytical X-ray diffractometer with Cu-Kα radiation in steps of 0.02 at a rate of 1° (2θ) per minute, sweep from 6° to70° (2θ), according to the diffraction powder method. The XRD results were analysed with an X-Pert high score plus software based on database provided by Joint Committee of Powder Diffraction Data Service. XRD patterns of BC soil and Red soil are shown in Figure 3.5. The presence of quartz (Sio_2), montmorillonite ((Na, Ca)0.3$(Al,Mg)_2Si_4O_{16}(OH)_2 \cdot xH_2O$ and microline *(*$KAlSi_3O_8$)can be confirmed from the XRD pattern of Black cotton soil. Red soil exhibited peaks at pertaining to quartz(Sio_2), kaolinite ($Al_2Si_2O_5\,(OH)_4$)and hematite (Fe_2O_3)

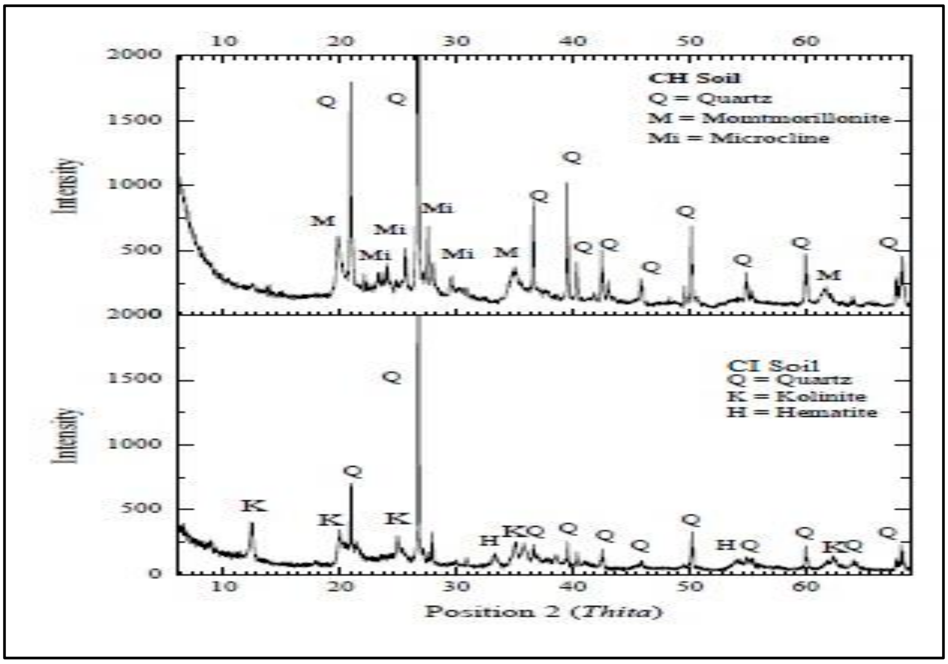

Figure 3.5 XRD images BC soil and Red soil

Morphological studies on soil samples werecarried out using TESCAN VEGA 3LMU microscope with conventional tungsten heatedcathode having live stereoscopic imaging using 3D beam technology. The samples werecoated with gold using a sputter coater prior to scanning. SEM images of BC soil and Red soils are shown in Figure 3.6. SEM micrograph of black cotton soil exhibited *undulating filmy particle* microstructure, whichconfirms the presence of montmorillonite (Mitchell and Soga 1993). Red soil micrograph reveals an unusual fibrous like microstructure of soil particles.

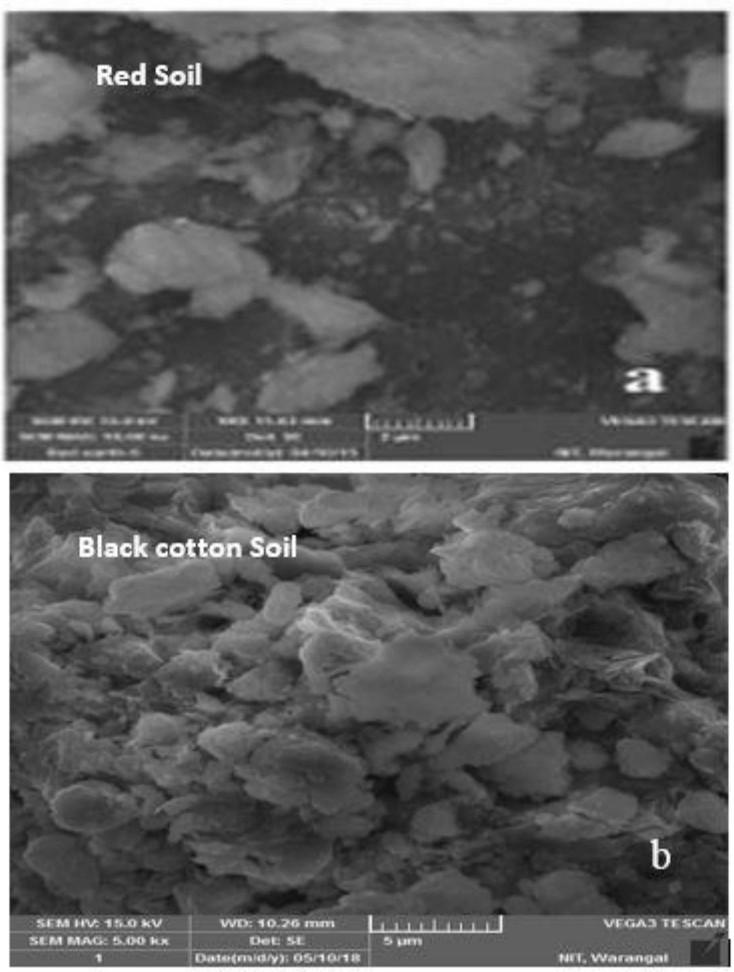

Figure 3.6 SEM images BC soil and Red soil

Chapter 4

Engineering properties of Virgin & CCR stabilized clayey soils

4.1 Introduction:

In the present study, multi scale laboratory analysis were carried out on Black cotton soil and Red soil stabilized with CCR. A series of tests were carried out to determine Atterberg limits, California Bearing Ratio (CBR) and unconfined compressive strength (UCS) of Black cotton soil and Red soil stabilized with CCR. Atterberg limit tests were used to find the optimum dosage of CCR for stabilization. Mineralogical and micro structural studies were conducted by using XRD and SEM analysis in order to understand the stabilization mechanism.

4.2 Material characterization

Two locally available subgrade soils were procured from Warangal, Telangana state, India. The soil samples brought from the sites were air dried, crushed and sieved through 4.75 mm sieve. Calcium carbide residue (CCR) is a by-product of acetylene gas industry and was collected from acetylene factories near Warangal. Atterberg limits tests were used to find the optimum dosage of CCR.

Figure4.1 presents the variation of Atterberg limits with CCR. It can be seen that liquid limit (LL) and plasticity index (PI) increase whereas the plastic limit (PL) decreased. Addition of CCR to BC soil resulted in an increase of coarser particle content and flocculation of clay particle due to the inclusion of Ca^{2+} from the cation exchange (Horpibulsuk *et al.*, 2011). This is the reason for reduction in LL and PI and increase in PL. It can be observed that addition of more than 8% CCR results in only nominal reduction in PI. It shows that at 8% CCR, BC soil can absorb maximum amount of Ca^{2+} ions. This point is known as CCR fixation point. Variation of Atterberg limits with addition of CCR to Red soil is shown in figure 4.2. It can be seen that liquid limit(LL) and plasticity index (PI) increase whereas plastic limit (PL) decreases. It can be noticed that with the addition of more than 4% of CCR to Red soil, there is only nominal reduction in plasticity index. From Figure 4.2, 4% CCR is the optimum dosage for the Red soil.

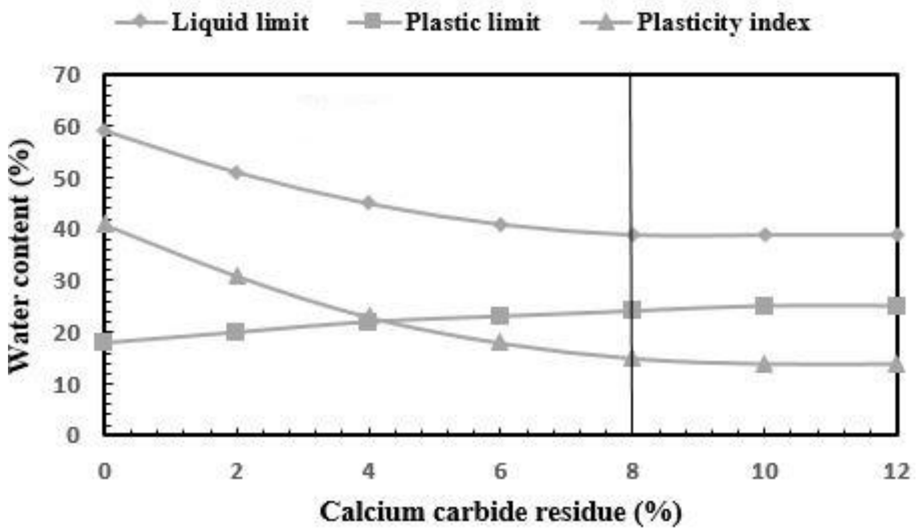

Figure 4.1 Variation of Atterbergs limits of BC soil with CCR

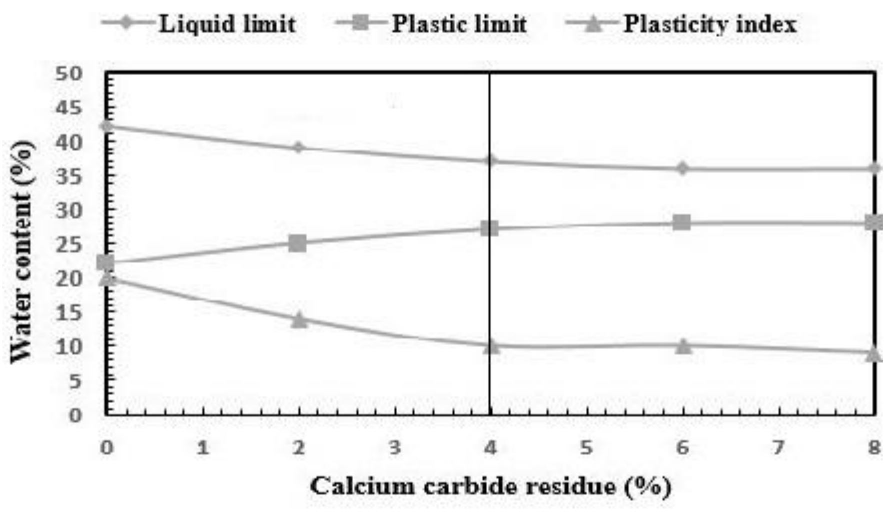

Figure 4.2 Variation of Atterbergs limits of Red soil with CCR

Results of modified Proctor compaction tests carried out on BC soil samples with and without modification with CCR are presented in Figure 4.3. Figure 4.4 represents OMC and MDD of Red soil with and without addition of CCR. The maximum dry density of both the soil samples is found to decrease whereas the optimum moisture content increased with the addition of CCR.The observed change in maximum dry density can be directly attributed to the lower specific gravity of the CCR, which lowers the overall weight of the mixture as the additive content increases. The change in optimum moisture content is associated to the particle agglomeration and flocculation caused due to the cation exchange resulting in higher water holding capacity during the compaction process. Kampala and Horpibulsuk (2012), who reported the reduction of maximum density with increasing CCR content for silty clay, obtain similar results. The reason of the phenomena is predominantly attributed to the flocculation and agglomeration, as a consequence of cation exchange, resulting in material bulking (Kinuthia et al. 1999).

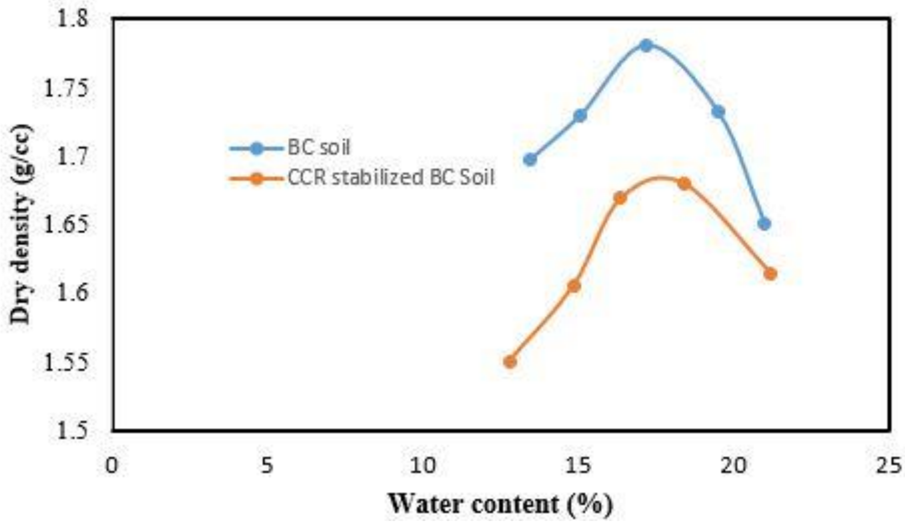

Figure 4.3 Compaction curves for BC and CCR stabilized BC soil

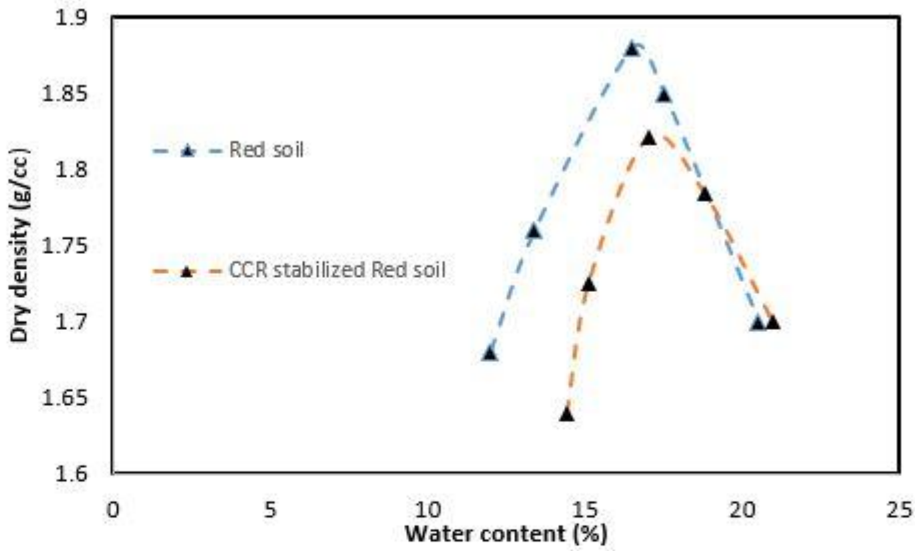

Figure 4.4 Compaction curves for Red and CCR stabilized Red soil

Samples for UCC tests were prepared at the maximum dry density and optimum moisture content obtained from the compaction tests. After moulding, the specimens were immediately extruded from the split mould and then placed in plastic bags and stored in desiccators to avoid significant variations of moisture content before testing.

The Variation of UCS values of BC soil for different CCR content corresponding to 7, 14, 28 days period of curing are obtained and are shown in Figure 4.5. It can be observed that with the addition of CCR to the BC soil, it is able to sustain higher stresses than the virgin BC soil. With an increase in curing time, an increase in UCS value was observed for the stabilized BC soil. The increase in UCS values of CCR stabilized soil is credited to the pozzolanic reaction between silica and alumina of BC soil and lime of CCR to form various types of cementing agents. The UCS of CCR stabilized soil is found to increase up to 8% CCR content and decrease with further increase in CCR content. The reason for this behavior is due to the complete consumption of $Ca(OH)_2$ by the natural pozzolanic material in the soil occurring at 8% CCR. More CCR content lead to the precipitation of free lime, which leads to a reduction in strength. It can be observed that the maximum UCS is coinciding with the CCR fixation point obtained from the Atterbergs limits.Similar results observed by Kampala and

Horpibulsuk (2012), who reported the reduction of maximum density with increasing CCR content for silty clay. The reason of the phenomena is predominantly attributed to the flocculation and agglomeration, because of cation exchange, resulting in material bulking (Kinuthia et al. 1999).

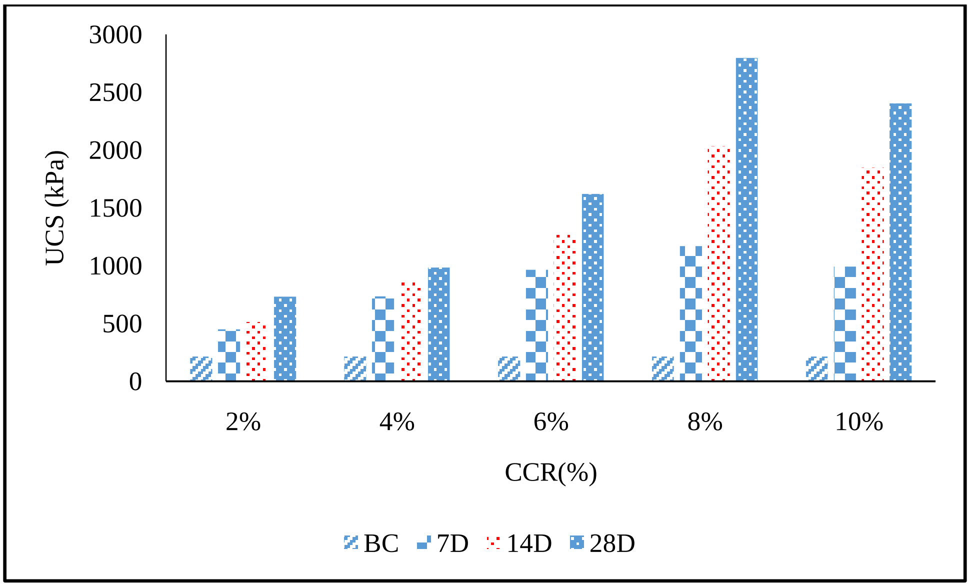

Figure 4.5 Variation in UCSof BC soil treated with CCR for curing periods of 7,14and 28 days

Figure 4.6 showed variation in UCS of Red soil with addition of CCR. The UCS value is increased up to 4% CCR addition, after that decrement is observed in UCS. It can be noticed that the addition of more than 4% of CCR to Red soil causes only nominal reduction in plasticity index. From UCS and Atterberg limits, 4% CCR is taken as optimum dosage for Red soil stabilization. The strength development for CCR and quicklime stabilized soft clayey soils in this study is consistent with those reported by Kampala and Horpibulsuk (2013). The early gain in strength can be attributed to the flocculation and agglomeration of the soil particles (Kinuthia et al. 1999) while the long-term strength development is the result of pozzolanic reactions(Wild et al., 1993).

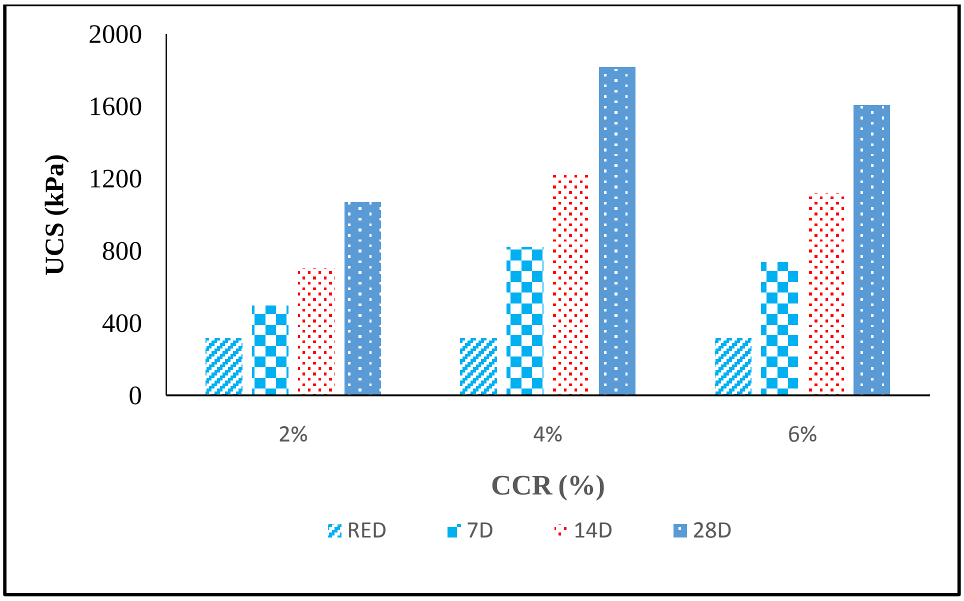

Figure 4.6Variation in UCS of Red soil treated with CCR for curing periods of 7,14and 28 days

CBR tests were performed for BC soil stabilized with varying percentages of CCR. All the samples were compacted at maximum dry density and with water content equal toOMC. CBR values obtained from different CCR contents are shown in Figure 4.7. From the figure, it can be noticed that the CBR value of soil-CCR mix increased significantly with an increase in CCR content (Ampadu et al 2007). The increase in UCS values of CCR stabilized soil is credited to the pozzolanic reaction between silica and alumina of BC soil and lime of CCR to form various types of cementing agents(Wild et al., 1993).

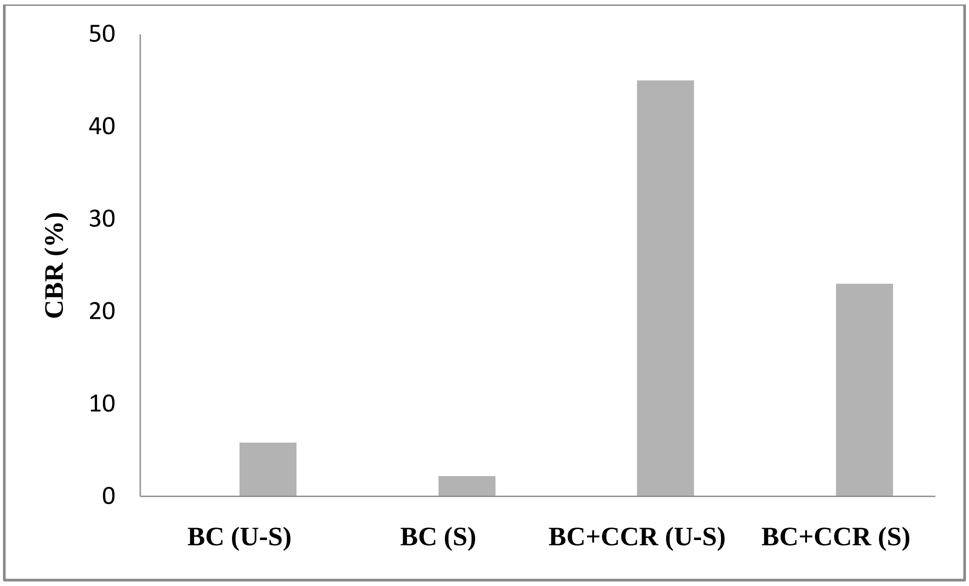

Figure 4.7 CBR of BC soil and CCR treated BC soil

Figure 4.8 represents CBR values of Red soil and CCR stabilized Red soil for both soaked (S)and unsoaked condition(U-S). There is nearly 42-55% decrement observed when the soil is in soaked condition compared to unsoaked condition. Drastic increment of CBR is noted when red soil is stabilized with CCR in both saturated and un-saturated condition.

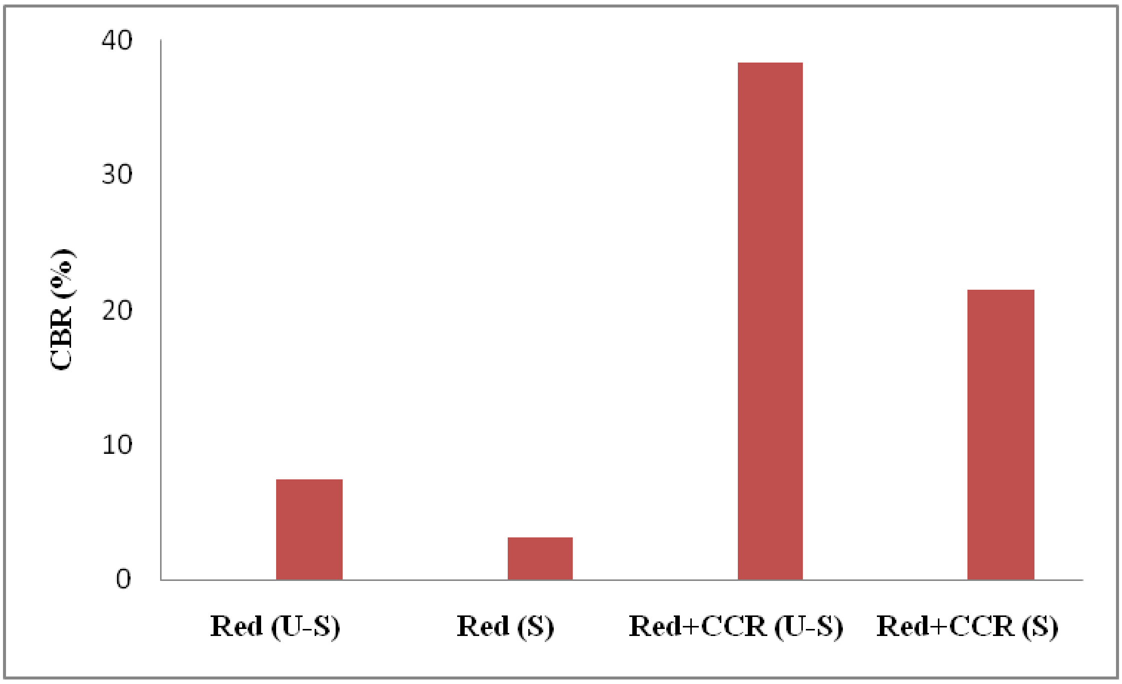

Figure 4.8 CBR of Red soil and CCR treated Red soil

PAN analytical X-pert powder diffractometer is used to study the Mineralogical behavior of soil samples. Specimens were scanned from 6º (2θ) and 70º (2θ) angle using copper K-alpha radiation at a scanning rate of 2º/min. X-pert high score plus software is used to find out the mineral present in the soil using the data obtained from XRD test. Figure 4.9 represents the XRD patterns of BC and CCR stabilized BC soil. XRD pattern of soil with respect to water shows that the natural BC soil contains Volkonskoite along with Quartz and microcline as their major mineralsAfter treating with 8% CCR, the hydration products Anorthite ($CaAl_2Si_2O_8$) and Gismodine ($CaAl_2Si_2O_84H_2O$) are mainly formed on account of the occurrence of the pozzolanic reaction. Figure 4.10 represents XRD patterns of Red and CCR stabilized Red soil.The XRD pattern of Red soil at natural moisture contents possess quartz and kaolinite minerals. Addition of 4% CCR to the Red soil results in the formation of Gismonidite and Etteringite minerals. The pozzolonic reactions between the silica and alumina of BC soil and the lime of CCR is clearly evident from the X-Ray diffraction results confirming the reason for increase in UCS and CBR values with the addition of CCR.

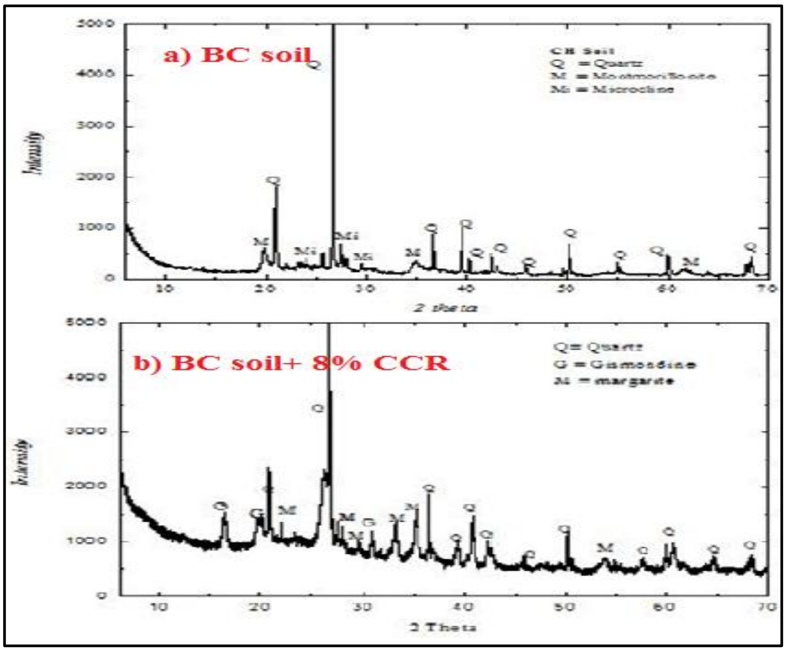

Figure 4.9 XRD patterns of BC soil and CCR stabilized BC soil

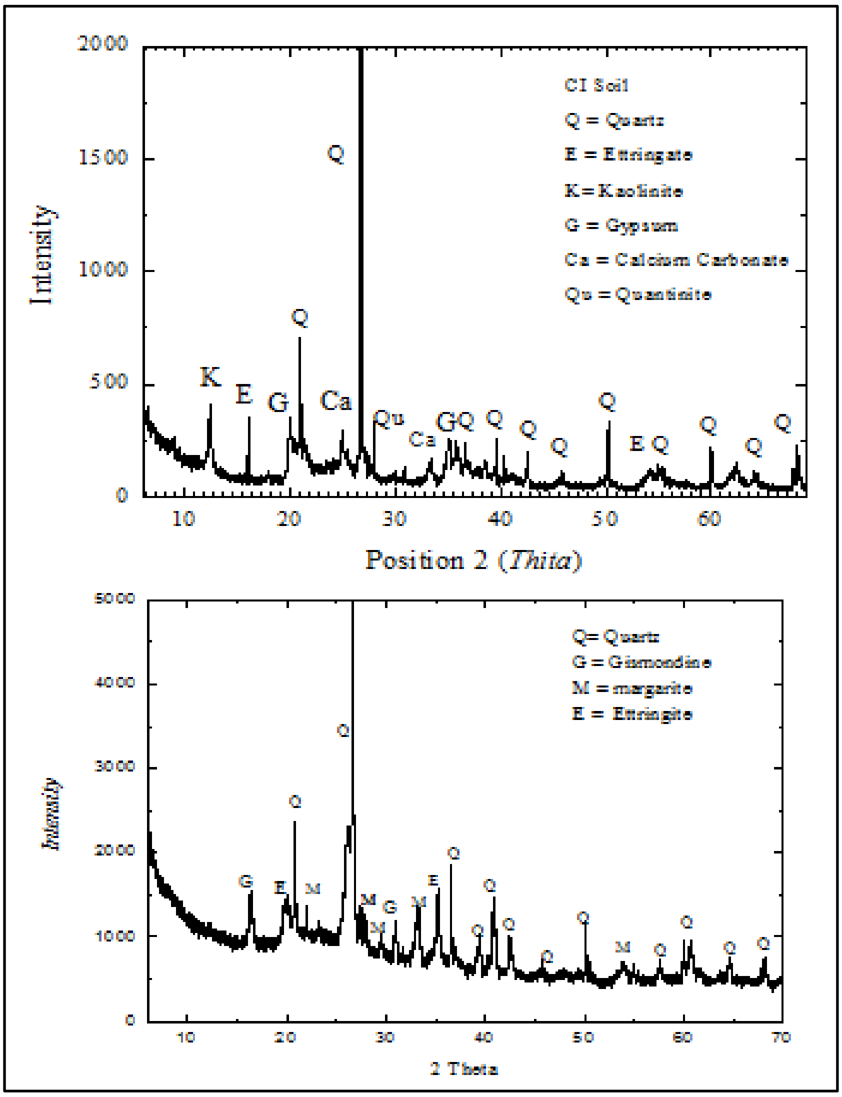

Figure 4.10 XRD patterns of Red soil and CCR stabilized Red soil

Scanning electron microscope images of the BC soil and soil treated with CCR are shown in Figure 4.11 and SEM images of Red soil and CCR stabilized Red soil are shown in Figure 4.12. The soil-CCR mix after curing for 28 days were scanned using electron microscope under different magnifications. The scanning electron micrographs shows the microstructure of BC soil and Red soil mixed with CCR.

Natural soil exhibits a film type of microstructure. It can be seen from the Figure 4.11 and 4.12 that the stabilized soil exhibits flocculated structure as compared to that of natural soil.

The precipitation of binders around the particles can also be clearly evidenced from the SEM micrographs of stabilized soils.

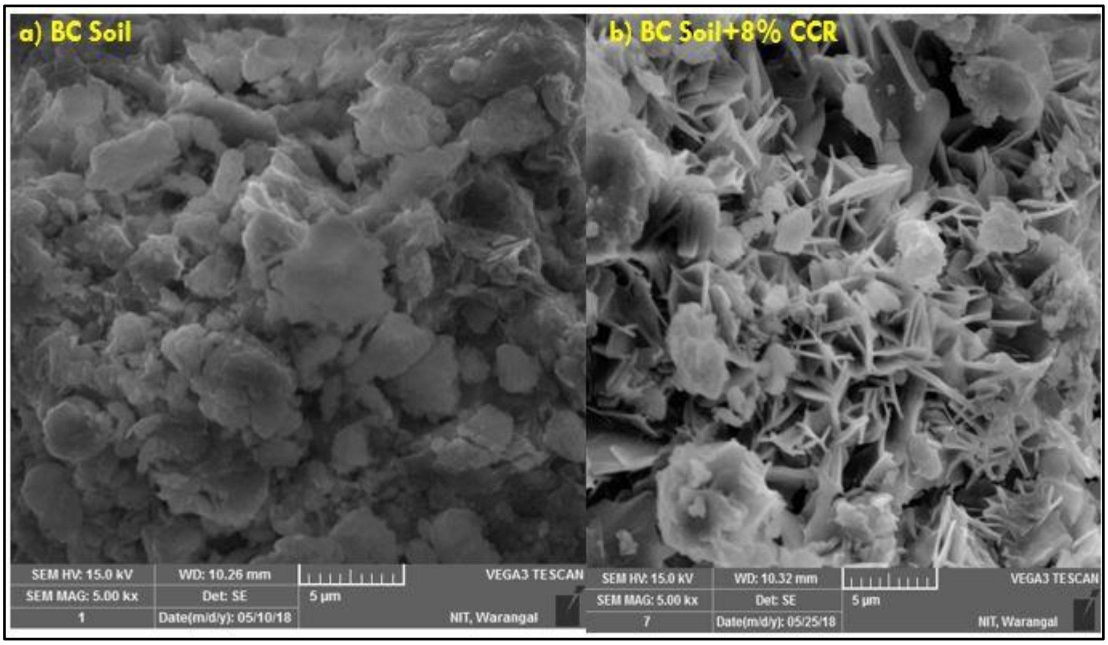

Figure 4.11 SEM images BC soil and CCR stabilized BC soil

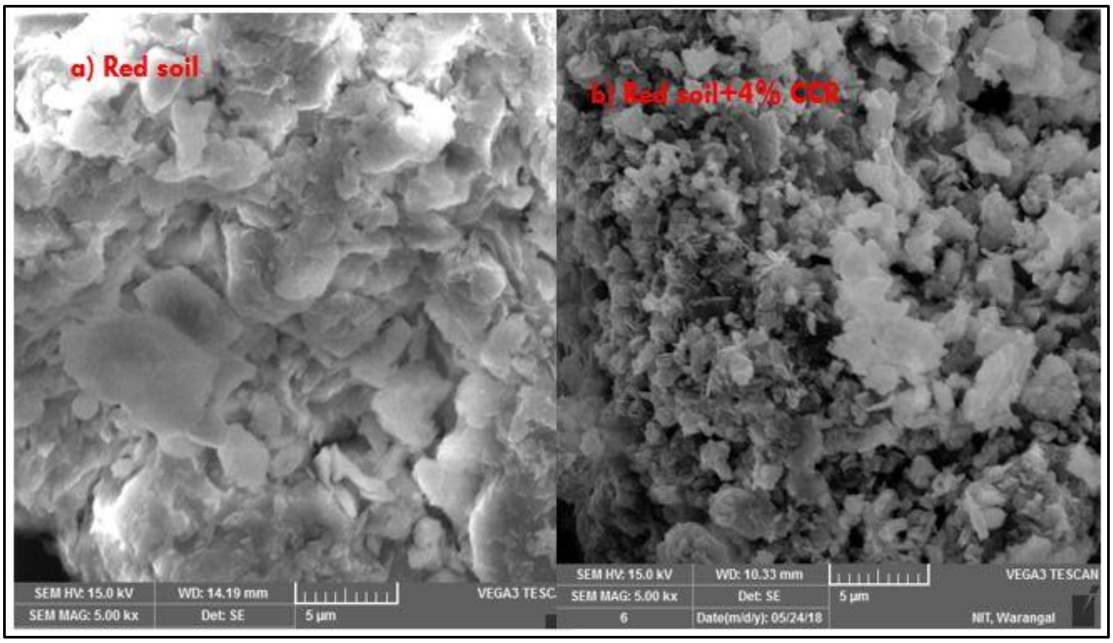

Figure 4.12 SEM images Red soil and CCR stabilized Red soil

4.3 Monotonic Triaxial Test

Static and cyclic triaxial tests were conducted on cylindrical specimens with 75mm diameter and 150mm height. Natural soils and soil mixed with optimum amount of CCR were statically compacted to maximum density in a cylindrical mould. The compacted samples were taken out and stored in a desiccator for 28 days for curing before testing.

Monotonic triaxial tests were carried out on the soil specimens under unconsolidated undrained condition for three different moisture contents (OMC, OMC+2% and OMC+4%). Effect of water content on deviator stress at failure for Red soil is shown in Figure 4.13 and for Black cotton soil is represented in Figure 4.14. The stress levels to be applied in the repeated load tests were determined using the deviator stress at failure.

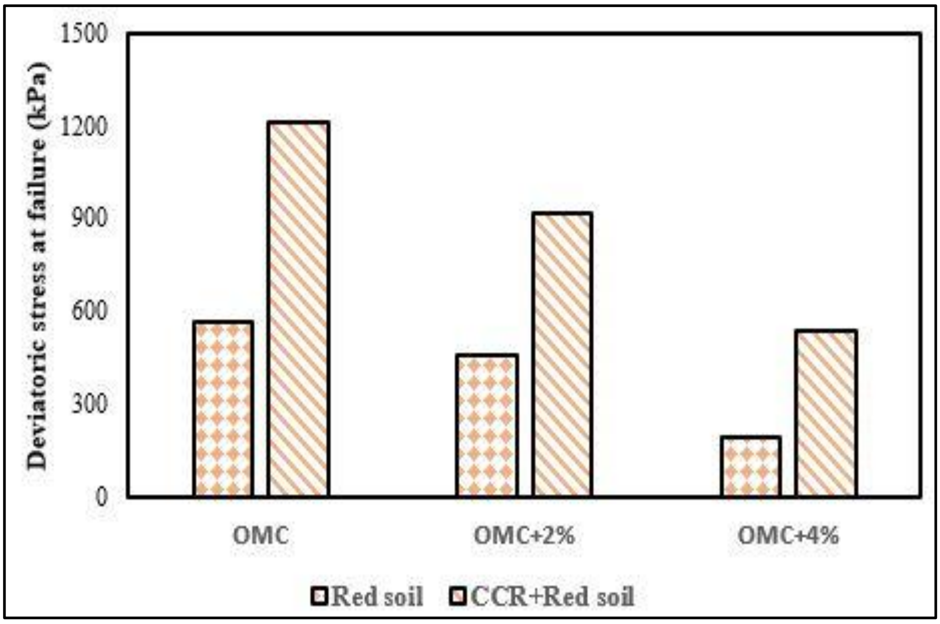

Figure 4.13 Deviatoric stress at failure for Red soil and CCR stabilized Red soil

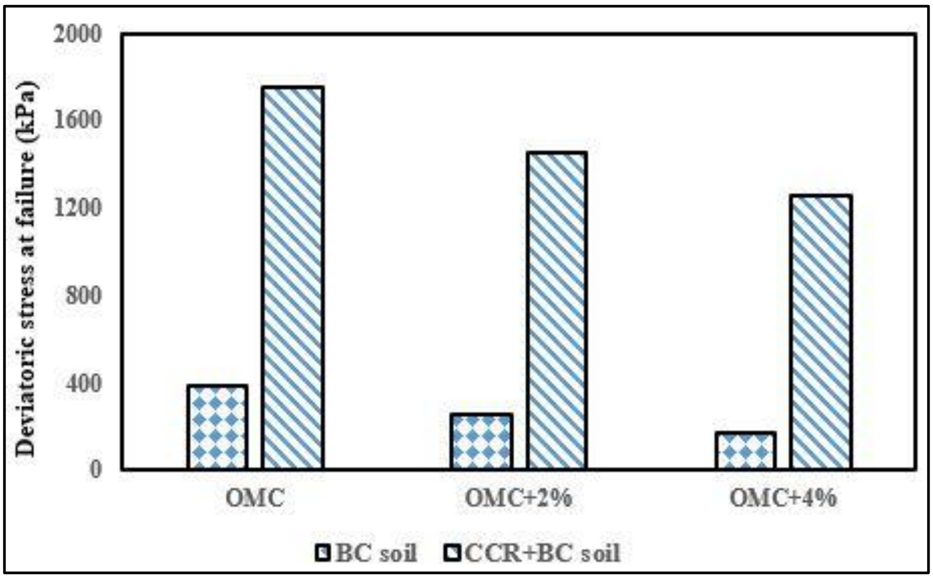

Figure 4.14 Deviatoric stress at failure for BC soil and CCR stabilized BC soil

4.4 Summary

The results indicated that 4% CCR was found to be optimal for soil for CI soil whereas 8% CCR was required in the case of CH soil. Stabilization with calcium carbide residue enhanced the unconfined compressive strength and California bearing ratio by 10-14.5 times when compared to the natural soil for CH soil whereas 7.5-11.8 enhancement observed for CI soil. The BC soil stabilized with both calcium carbide residue obtained 13.5-18 folds strength enhancement in CBR tests compare to natural soil.The formation of cementing agents such as calcium silicate hydrates and calcium aluminium silicates hydroxides liable for improved strength was evidenced from mineralogical studies using XRD. The formation of pozzolonic compounds and flocculation is evident from the SEM images also.

Chapter 5

Resilient Modulus

5.1 Introduction

Mechanistic-empirical design of flexible pavement AASHTO practice requires resilient characteristics of subgradesoil. Resilient modulus of subgrade soil is generally obtained by conducting repeated triaxial test on soil samples in the laboratory. Repeated triaxial tests were performed on soil specimens at three confining pressures and seven different deviator stress levels as recommended by AASHTO T-307 (AASHTO-T 307. (2003)).In the present chapter, effect of calcium carbide residue (CCR) on resilient modulus of the subgrade soils , effect of moisture content, confining pressure and deviator stress on both natural and CCR stabilized soils were investigated. Regression analysis was carried out with the experimental data using universal model and NCHRP model and the corresponding coefficients were obtained.

5.2 Resilient Modulus Characterization

Some of the important factors, which influence the resilient modulus of granular materials, are soil gradation, water content, microstructure, dry density, stress state and stress history (Rada and witczak, 1981; Li *et al*., 1994; Zaman *et al*., 1994). Stress levels and moisture content are known to be two key variables considered for the determination of resilient modulus.

5.2.1 Effect of confining pressure and deviatoric stress

The response of granular material is non-linear under repeated loads. The variation of resilient modulus with different deviatoric stress levels (13.8 kPa to 68.9 kPa) under three different confining pressures of 13.8, 27.6 and 41.4 kPa for natural soil and CCR stabilized soils were determined. In order to consider the variation in moisture content, the soil specimens were prepared at three different water contents (OMC, OMC+2 and OMC+4).

Variation of resilient modulus with stress levels for natural BC soil samples prepared with optimum moisture content is shown in Figure 5.1. It can be observed that the resilient modulus decreased nearly 23% when deviatoric stress increased from 13.8 kPa to 68.9 kPa under 41.4 kPa constant confining pressure. It can also be observed that resilient modulus increased nearly 37-48% when confining pressure increased from 13.8 kPa to 41.4 kPa.

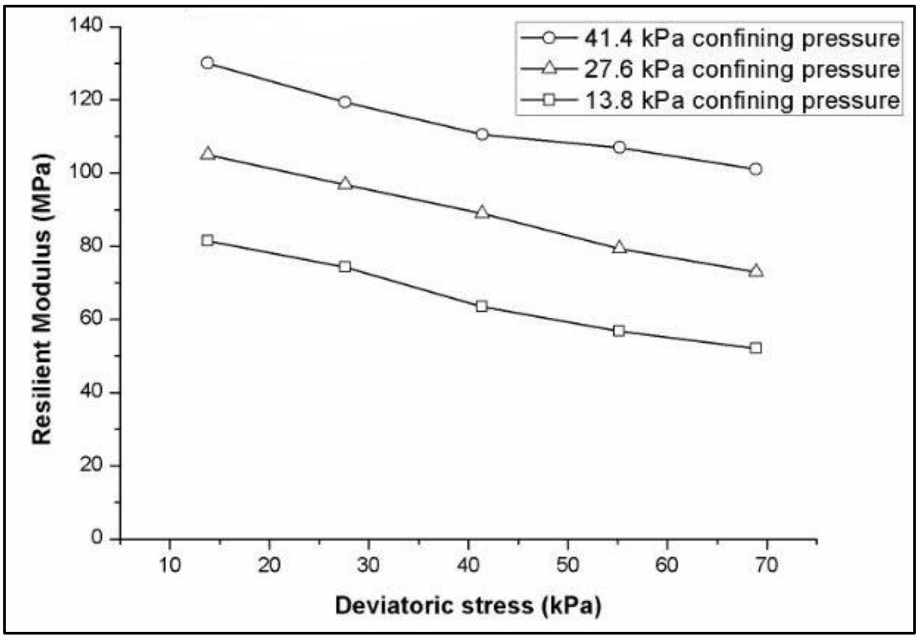

Figure 5.1 Variation of resilient modulus with stress levels of BC soil (OMC)

Figure 5.2 depicts the variation ofresilient modulus with stress levels for CCR stabilized BC soil samples prepared with optimum moisture content. It can be noted from Figure 5.2 that resilient modulus decreased nearly 19% when deviatoric stress increased from 13.8 kPa to 68.9 kPa under 41.4 kPa constant confining pressure. From Figures, 5.1 and 5.2 we can observe that deviatoric stress is more effective in case of natural soil compared to CCR stabilized BC soil. It can be also observed from Figure 5.1 and 5.2 that there is nearly 2.5 to 2.8 times increment in resilient modulus when the BC soil is stabilized with CCR.

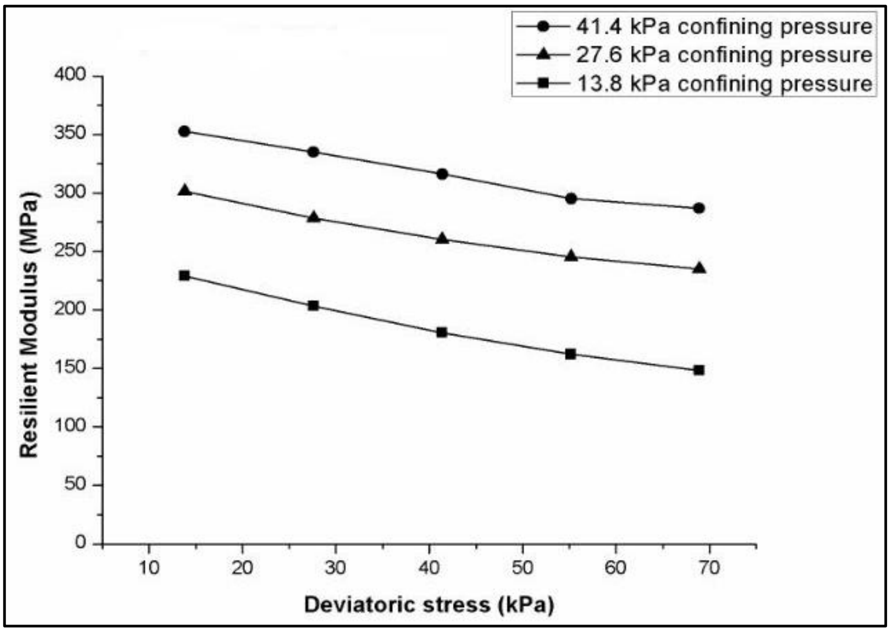

Figure 5.2 Variation of resilient modulus with stress levels of CCR stabilized BC soil (OMC)

Figure 5.3 exhibits thevariation of resilient modulus with stress levels for natural Red soil samples prepared with optimum moisture content. From Figure 5.3, it can be observed that resilient modulus decreased nearly 18%, when deviatoric stress increased from 13.8 kPa to 68.9 kPa under 41.4 kPa constant confining pressure. It can also be noted that resilient modulus enhanced nearly 36 - 44% when confining pressure increased from 13.8 kPa to 41.4 kPa.

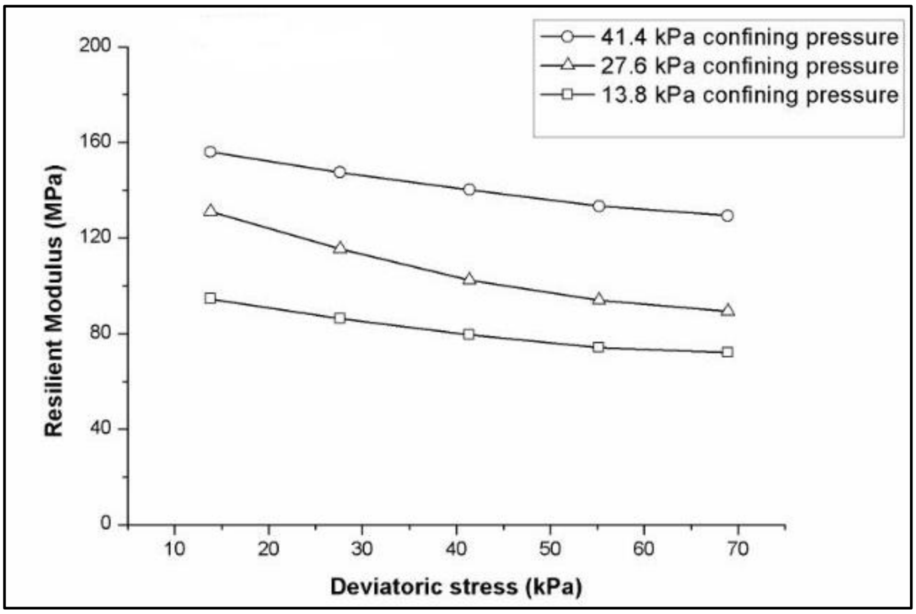

Figure 5.3 Variation of resilient modulus with stress levels of Red soil (OMC)

Variation of resilient modulus with stress levels for CCR stabilized Red soil samples prepared with optimum moisture content is shown in Figure 5.4. It can be noted from Figure 5.4 that the resilient modulus decreased nearly 17% when deviatoric stress increased from 13.8 kPa to 68.9 kPa under 41.4 kPa constant confining pressure. From Figures 5.3 and 5.4, we can observe that there is nearly 1.7 to 1.85 times increment in resilient modulus when the Red soil is stabilized with CCR.

Figure 5.5 depicts the variation ofresilient modulus with stress levels for natural BC soil samples prepared with OMC+2%. From Figure 5.5, it can be observed that resilient modulus decreased nearly 27% when deviatoric stress increased from 13.8 kPa to 68.9 kPa under 41.4 kPa constant confining pressure. From Figures 5.1 and 5.5, it can also be observed that deviatoric stress is effective at higher water content. It is also observed that resilient modulus increased nearly 42-53% when confining pressure increased from 13.8 kPa to 41.4 kPa.

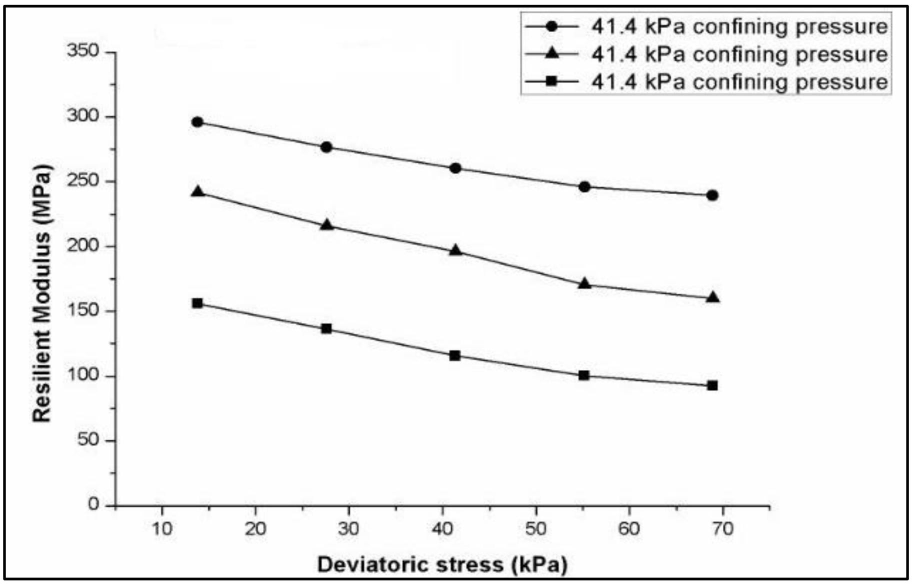

Figure 5.4 Variation of resilient modulus with stress levels of CCR stabilized Red soil (OMC)

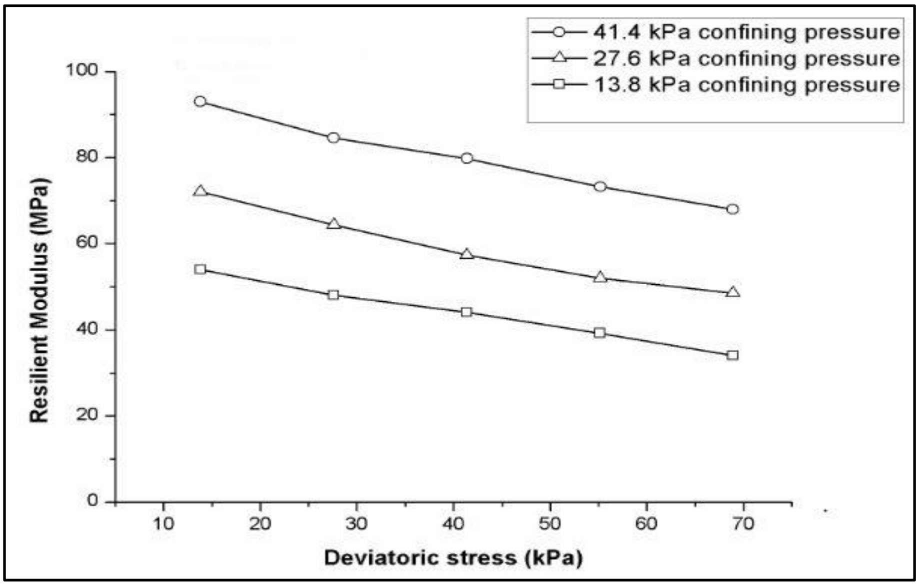

Figure 5.5 Variation of resilient modulus with stress levels of BC soil (OMC+2%)

Figure 5.6 represents the variation ofresilient modulus with stress levels for CCR stabilized BC soil samples prepared with OMC+2%. From Figure 5.6, it can be noted that resilient modulus decreased nearly 23% when deviatoric stress increased from 13.8 kPa to 68.9 kPa under 41.4 kPa constant confining pressure. It can be also observed from Figure 5.5 and 5.6

that there is nearly 2.8 to 3.0 times increment in resilient modulus when the BC soil is stabilized with CCR.

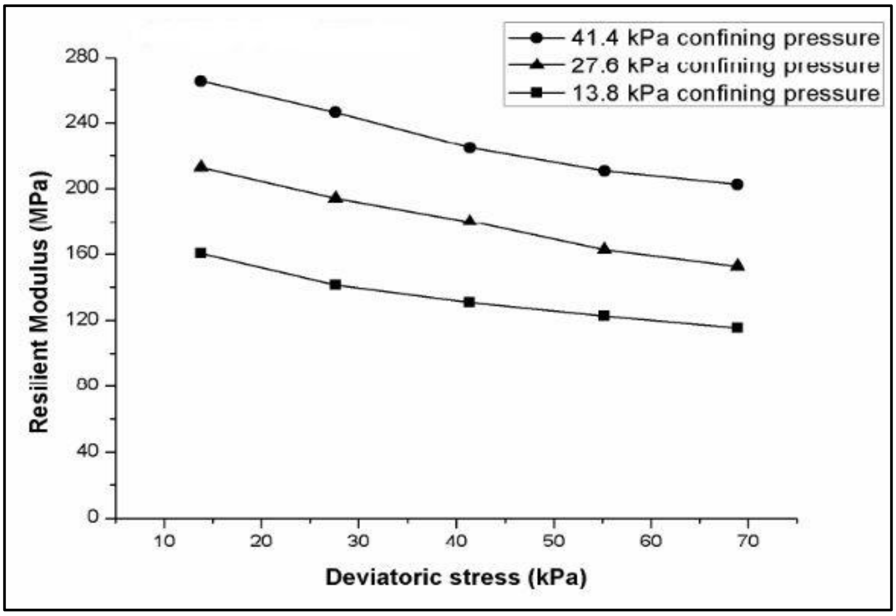

Figure 5.6 Variation of resilient modulus with stress levels of CCR stabilized BC soil (OMC+2%)

Figure 5.7 exhibit the variation of resilient modulus with stress levels for natural Red soil samples prepared with OMC+2%. From figure 5.7, it can be observed that resilient modulus decreased nearly 29%, when deviatoric stress increased from 13.8 kPa to 68.9 kPa under 41.4 kPa constant confining pressure. It can also be noted that the resilient modulus enhanced nearly 50-54% when confining pressure increased from 13.8 kPa to 41.4 kPa. From Figures 5.3 and 5.7, it can be noted that the effect of confining pressure is more effective at higher water content and it is due to the reduction in interaction between particles with increase in moisture content. Thus, increment in confining pressure results in the reduction in the gaps between the particles, thereby increasing the load carrying capacity. Therefore, at higher water contents confining pressure is more effective in enhancing the resilient modulus.

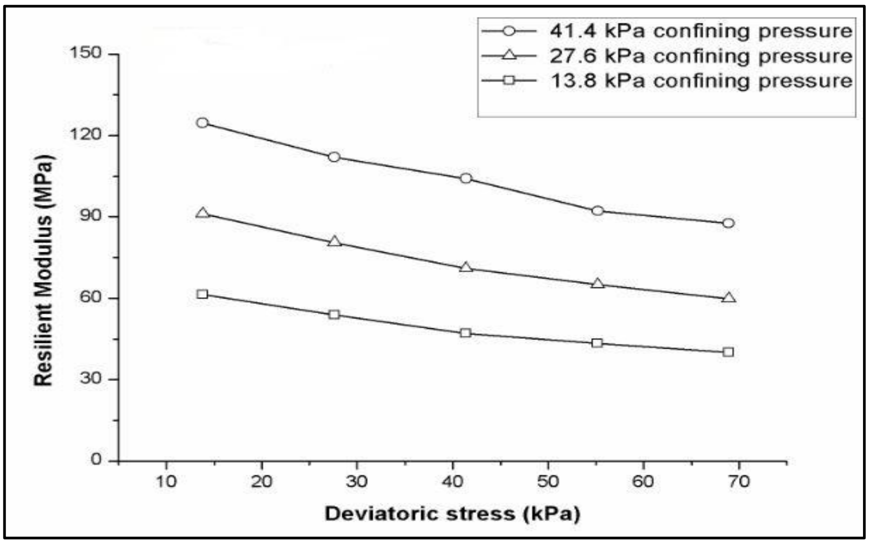

Figure 5.7 Variation of resilient modulus with stress levels of Red soil (OMC+2%)

Variation of resilient modulus with stress levels for CCR stabilized Red soil samples prepared with OMC+2% is shown in Figure 5.8. It can be observed that from Figure 5.8 that the resilient modulus decreased nearly 17% when deviatoric stress increased from 13.8 kPa to 68.9 kPa under 41.4 kPa constant confining pressure. From Figures 5.7 and 5.8 we can observe that there is nearly 1.8 to 2 times increment in resilient modulus when the Red soil is stabilized with CCR.

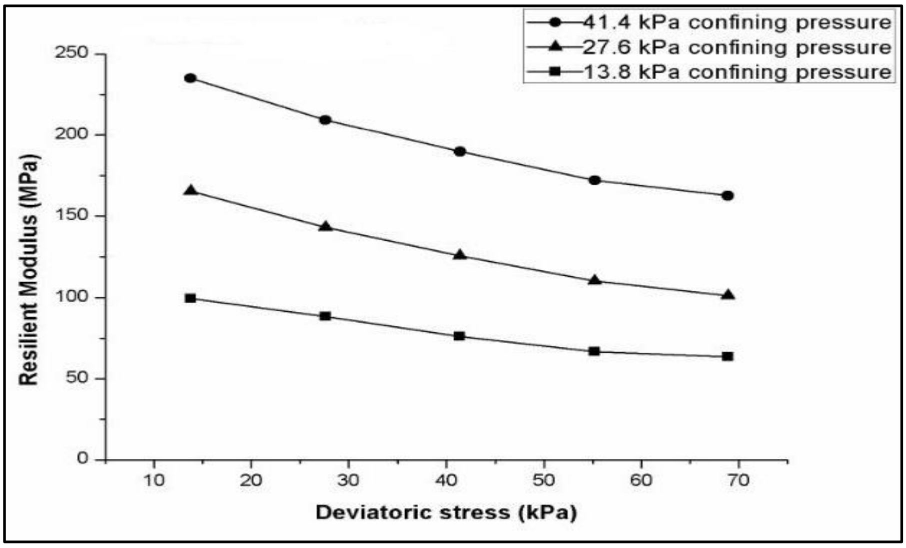

Figure 5.8 Variation of resilient modulus with stress levels of CCR stabilized Red soil (OMC+2%)

Figure 5.9 depicts the variation of resilient modulus with stress levels for natural BC soil samples prepared with OMC+4%. From Figure 5.9, it can be observed that resilient modulus decreased nearly 55% when deviatoric stress increased from 13.8 kPa to 68.9 kPa under 41.4 kPa constant confining pressure. From Figures 5.5 and 5.9, it can also be noted that further increase in moisture content by 2% caused reduction in resilient modulus by 28%, which is due to lubricant effect.

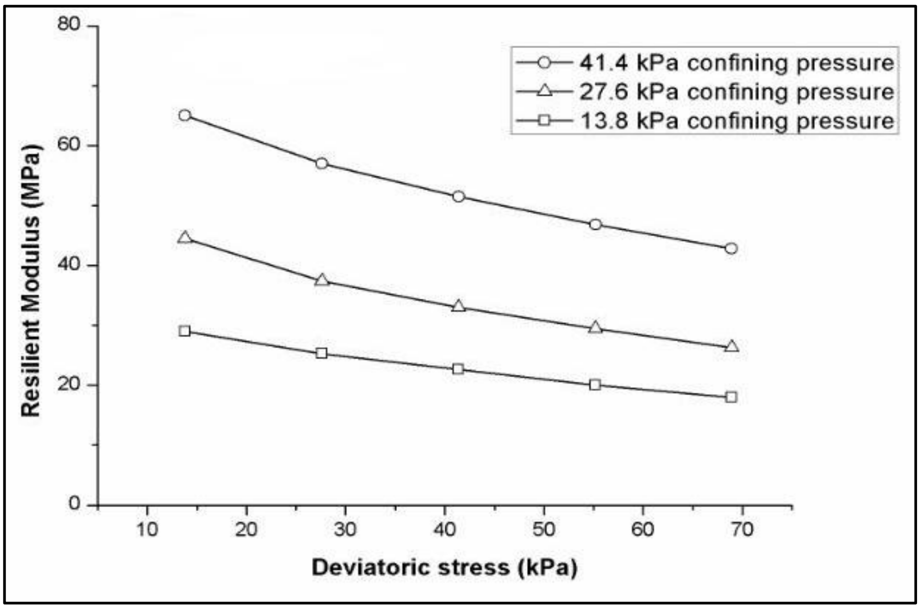

Figure 5.9 Variation of resilient modulus with stress levels of BC soil (OMC+4%)

Figure 5.10 shows the variation of resilient modulus with stress levels for CCR stabilized BC soil samples prepared with OMC+4%. From Figure 5.10, it can be noted that 20% reduction in resilient modulus is observed when deviatoric stress increased from 13.8 kPa to 68.9 kPa under 41.4 kPa constant confining pressure. In this case, for CCR stabilized BC soil at higher water content, the resilient modulus values were found to be 2.8 to 3.5 times more that of the natural BC soil.

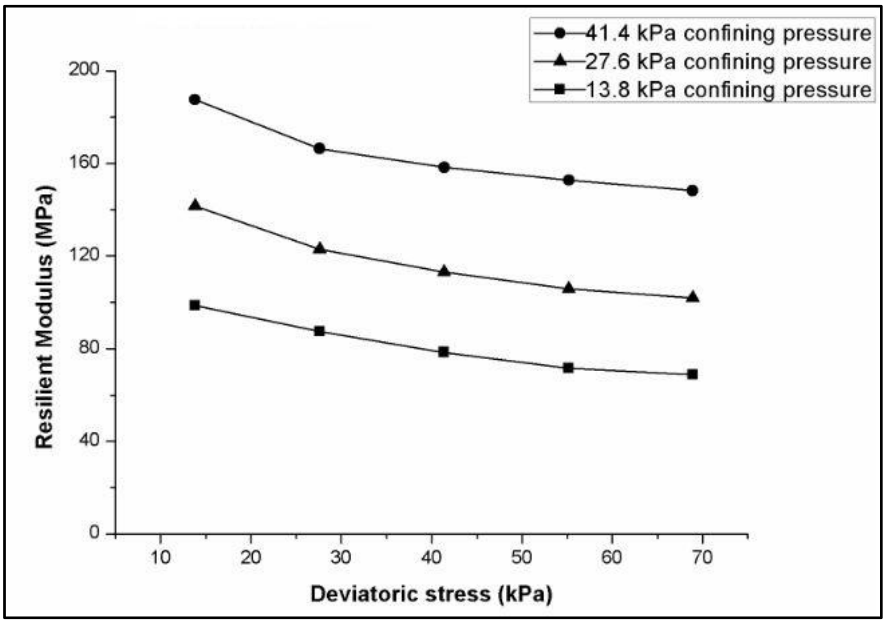

Figure 5.10 Variation of resilient modulus with stress levels of CCR stabilized BC soil (OMC+4%)

Figure 5.11 represents the variation ofresilient modulus with stress levels for natural Red soil samples prepared with OMC+4%. From Figure 5.11, it can be observed that resilient modulus decreased nearly 32%, when deviatoric stress increased from 13.8 kPa to 68.9 kPa under 41.4 kPa constant confining pressure.

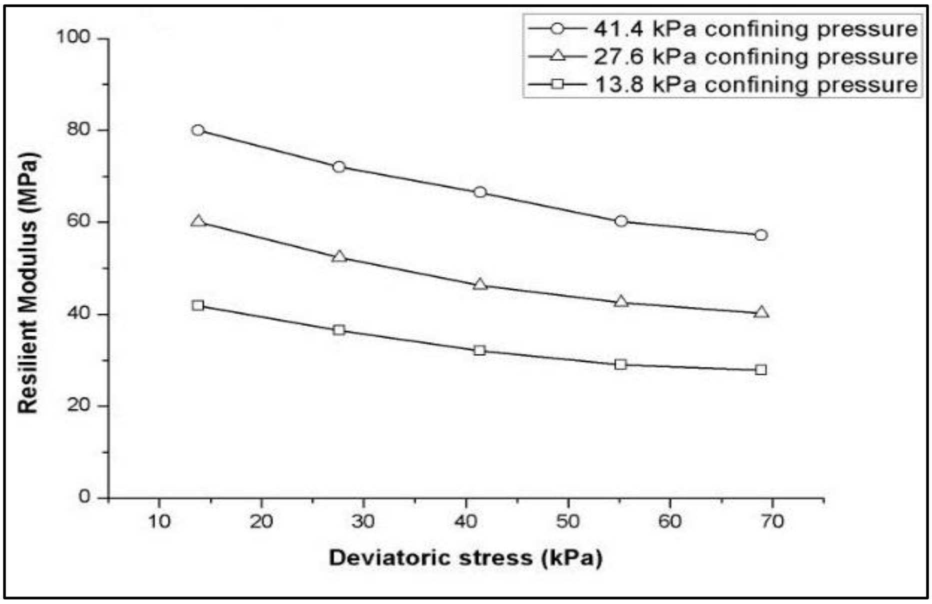

Figure 5.11 Variation of resilient modulus with stress levels of Red soil (OMC+4%)

Variation of resilient modulus with stress levels for CCR stabilized Red soil samples prepared with OMC+4% is shown in Figure 5.12. It can be observed from Figure 5.12 that the resilient modulus decreased nearly 22% when deviatoric stress increased from 13.8 kPa to 68.9 kPa under 41.4 kPa constant confining pressure. From Figures 5.11 and 5.12, we can observe that there is nearly 2.2 times increment in resilient modulus when the Red soil is stabilized with CCR.

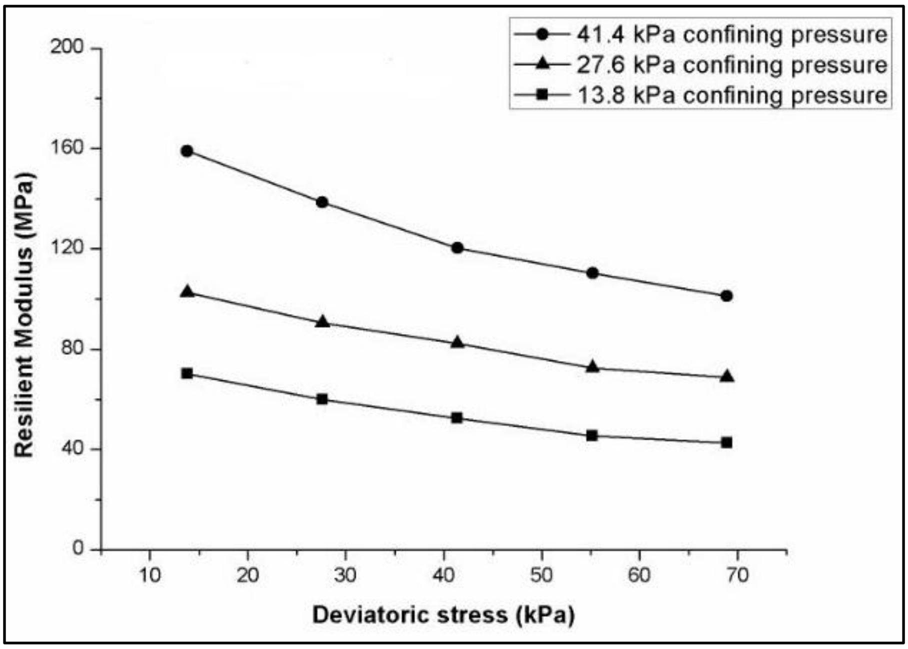

Figure 5.12 Variation of resilient modulus with stress levels of CCR stabilized Red soil (OMC+4%)

Chahuan et al. (2008) observed that the permanent deformation decreased by 21% when a silty sand material was stabilized with fly ash and fibre while 18% reduction was obtained with the addition of synthetic fibre. Anupam et al. (2016) reported that the accumulation of plastic strain reduced by 64% and 67%, when a clayey soil with intermediate plasticity was treated with fly ash and rice husk ash respectively. Patel and Shahu (2016) used fly ash, steel slag and dolomite mix as base course material and observed 83% reduction in plastic strain when compared to the standard wet mix macadam.

5.2.2 Effect of Moisture content

The moisture content of subgrade soils was found to increase by about 20 percent above the optimum moisture content over a period of two years after the completion of subgrade construction (Lin et al., 2011). In order to consider the moisture variation, in the present study the soil specimens were prepared at three different water contents (OMC, OMC+2% and OMC+4%) percentages of optimum moisture contents) and corresponding dry densities obtained from modified proctor compaction tests were adopted.

Figure 5.13 represents the variation of resilient modulus of BC soil, Red soil, CCR stabilized BC, Red soils under 13.8kPa confining pressure and 68.9 kPa deviatoric stress.

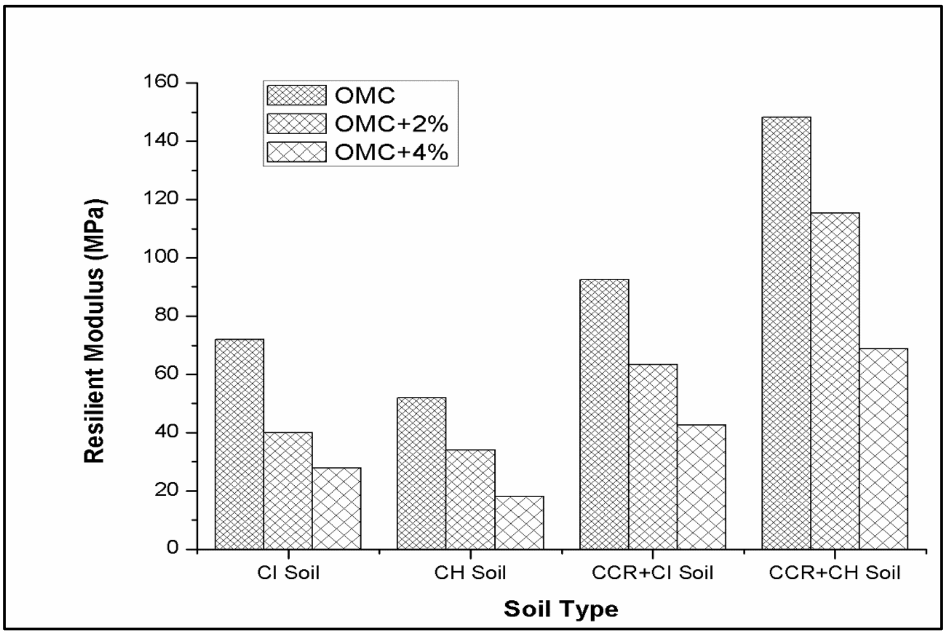

Figure 5.13 Resilient modulus of soils at different water contents

It can be inferred from Fig5.13 that when the BC soil water content was increased from OMC to OMC+4%, 52% reduction in resilient modulus was observed, whereas in case of Red soil, reduction observed in resilient modulus was 46%. The above cases infer that the stabilization is more effective at higher water content. The reduction in resilient modulus is noticed with increase in water content for both stabilized and virgin soils. As observed by the previous researchers, the resilient modulus values are found to increase with increase in confining pressure and reduce with increase in cyclic deviatoric stress for the stabilized soil samples also (Rout *et al*., 2012; Kang *et al*., 2014).

5.2.3 Relation between resilient modulus and CBR

Many equations are established in the literature to predict M_R, but the commonly used one is M_R-CBR models. The South African Council for Scientific and Industrial Research (CSIR), adopted equation of the form M_R = k*CBR, by modifying the k factor which depend on the nature of the material and laboratory tests (Paterson and Maree1978).

$$M_R(MPa) = 20.684 * CBR^{0.65}$$

Powell et al. (1984) suggested a relation based on in-situ CBR test and wavepropagation, this equation is valid only for CBR values between 1 -12.

$$M_R(MPa) = 17.616 * CBR^{0.64}$$

Uzan (1985) proposed a correlation based on statistical analysis and significance tests, the result allowed the establishment of the following relationship with a coefficient of correlation (R^2) of 0.69 for the M_R predicted from the Uzan model.

$$M_R(MPa) = 91.226 + 0.017 * CBR^2$$

Hopkins (1994a) reported CBR and MR correlation for cohesive subgrade soil and proposed an equation as follow

$$M_R(MPa) = 17.914 * CBR^{0.874}$$

IRC 37-2012 gave a relation between resilient modulus and CBR an equation as follow

$$M_R(MPa) = 10 * CBR \qquad \text{If CBR} < 5\%$$

$$M_R(MPa) = 176 * CBR^{0.64} \quad \text{If CBR} > 5$$

5.2.4 Regression models

Several regression models have been reported in the literature for determining resilient modulus values with stress invariants and moisture content as variables. One of the models was developed by Uzan (1985).which is known as the universal model. This model considersthe effect of both confining stress and deviator stress due to wheel load to predict the resilient modulus of unbound granular material using the expression given in Equation

$$M_R = K_1(\theta/p_a)^{k_2}(\sigma_d/p_a)^{k_3} \qquad 5.1$$

Here, θ is the bulk stress ($\theta=\sigma_1+2\sigma_3$) and $\sigma_d=\sigma_1-\sigma_3$ is the deviator stress whereas $\sigma_1$is the total vertical stress, σ_3 is the confining pressure and pa is the atmospheric pressure (101.4 kPa). k_1, k_2 and k_3 are the constants or parameters which can be obtained from multiple regression analysis of the repeated load triaxial test data. Table 5.1 and 5.2 show the values of constants k_1, k_2, k_3 and the coefficient of multiple determination (R^2) values obtained from the test data of the present study. R^2 values are found to be greater than 0.9 which provides evidence of the model showing a good correlation fit.

Table 5.1 Regression coefficients for universal model Uzan, (1985) (CH soil and CCR stabilized CH soil)

Water content	CH Soil		CCR stabilized CH soil	
	Parameters	R^2	Parameters	R^2
OMC	K_1=62.34	0.944	K_1=144.6	0.882
	K_2=4.25		K_2=6.21	
	K_3=-4.44		K_3=-7.2	
OMC+2%	K_1=45.89	0.902	K_1=98.25	0.912

	K_2=4.02		K_2=4.58	
	K_3=-4.87		K_3=-5.21	
OMC+4%	K_1=24.14	0.97	K_1=33.85	0.92
	K_2=2.28		K_2=3.88	
	K_3=-3.41		K_3=-4.01	

Table 5.2 Regression coefficients for universal model Uzan (1985) (CI soil and CCR stabilized CI soil)

Water content	CI Soil		CCR stabilized CI soil	
	Parameters	R^2	Parameters	R^2
OMC	K_1=69.451	0.945	K_1=129.47	0.901
	K_2=4.656		K_2=5.098	
	K_3=-4.01		K_3=-5.02	
OMC+2%	K_1=47.25	0.922	K_1=81.8	0.98
	K_2=4.234		K_2=4.28	
	K_3=-3.92		K_3= -4.26	
OMC+4%	K_1=18.25	0.895	K_1=32.64	0.944
	K_2=3.75		K_2=4.01	
	K_3= -3.6		K_3=-3.8	

Another model was developed by NCHRP (2004) in order to envisage the resilient modulus. This model uses both bulk stress and octahedral stress for the determination of resilient modulus. According to this model, the resilient modulus is determined using the following Equation 5.2.

$$M_r = K_1 p_a \left(\theta/p_a\right)^{K_2} \left(\tau_{oct}/p_a + 1\right)^{K_3} \qquad 5.2$$

In Equation (5.2), $\tau_{oct} = \frac{\sqrt{2}}{3}(\sigma_1 - \sigma_3)$, θ is the bulk stress, p_a is the atmospheric pressure and k_1, k_2, and k_3 are the parameters. Table 5.3 and 5.4 represent the values of constants k_1, k_2, k_3 and R^2values obtained from the present test data for NCHRP model.

Table 5.3: Regression coefficients for NCHRP model (NCHRP (2004)) (CH soil and CCR stabilized CH soil)

Water content	CH Soil		CCR stabilized CH soil	
	Parameters	R^2	Parameters	R^2
OMC	K_1= 4889.85	0.954	K_1=9245.65	0.975
	K_2=7.87		K_2=11.16	
	K_3=-9.26		K_3=-14.01	
OMC+2%	K_1=2875.36	0.884	K_1=6647.54	0.914
	K_2=6.59		K_2=9.25	
	K_3=-8.12		K_3=-11.92	
OMC+4%	K_1=1445.51	0.98	K_1=3412.47	0.95
	K_2=4.8		K_2=6.12	
	K_3=-7.02		K_3=-9.25	

Table 5.4: Regression coefficients for NCHRP model (NCHRP (2004)(CI soil and CCR stabilized CI soil)

Water content	CI Soil		CCR stabilized CI soil	
	Parameters	R^2	Parameters	R^2
OMC	K_1=5750.44	0.921	K_1=7374.01	0.933
	K_2=8.15		K_2=9.35	
	K_3=-9.48		K_3=-12.82	
OMC+2%	K_1=3385.21	0.977	K_1=4420.6	0.91

	K_2=6.046		K_2=8.68	
	K_3=-8.45		K_3=-11.85	
OMC+4%	K_1=1852.35	0.982	K_1=2258.45	0.925
	K_2=4.89		K_2=5.612	
	K_3=-7.75		K_3=-9.14	

5.3 Summary

The current work examined the effectiveness of calcium carbide residue on the enhancement of resilient modulus of two clayey soils. The results indicated that 4% CCR was found to be optimal for soil with low plasticity whereas 8% CCR was required in the case of high plasticity clay. The resilient modulus values of samples exhibited 1.9 to 2.7 times increment with the addition of CCR. The presence of calcium in calcium carbide residue lead to the flocculation of clayey particles thereby improving the subgrade characteristics of clayey soils. The effect of confining pressure, deviatoric stress levels and water content on the resilient modulus of virgin and stabilised clay samples was examined. The resilient modulus values were found to increase with increase in confining pressure and reduce with increase in deviatoric stress levels for both the virgin samples and samples treated with CCR. The stabilization with CCR was observed to be more effective at higher water contents as the resilient modulus values of virgin samples remarkably reduced in the presence of high moisture content. The universal model (Uzan, (1985)) and NCHRP model(NCHRP (2004)) were found to fit the experimental data for stabilized soil samples very well, using multiple regression analysis with high coefficient of determination.

Chapter 6

Permanent deformation of virgin & CCR stabilized clayey soils

6.1 Introduction

Mechanistic Empiristic Pavement Design methods [MEPD] considers both fatigue and rutting failure criteria for flexible pavements (Puppala et al., 1999; Jegatheesan and Gnanendran 2015). The rutting occurs on the pavement surface due to the permanent deformation of pavement layers subjected to repeated wheel loads.Proper design of flexible pavements demands accurate prediction of plastic strains in the granular layers. In most of these studies, resilient modulus is heavily relied upon to accommodate the plastic response of soils. It is also reported in the literature that some of the subgrade soils with more fine content may possess good resilient characteristics, but will still exhibit excessive plastic deformations when subjected to repeated loads (Ullditz, 1993; Puppala, 1999; Lin et al., 2011;Jegatheesan and Gnanendran 2015). This observation clearly indicates the need to evaluate the permanent deformation behaviour of subgrade soils under different stress levels for complete characterization of the material.In the present work, behaviour of Black cotton soils and Red soil stabilized with CCR subjected to repeated loading under varying deviator stress levels and water content is considered. The permanent strain data obtained from the experimentswas fitted with two regression models, namely VTT model (Korkiala-Tanttu (2007)) and power law model (Monismith et al. (1975), which takes in to account the effect of stress levels and number of load cycles.

6.2 Repeated Triaxial test

6.2.1 Effect of stress level (Red soil)

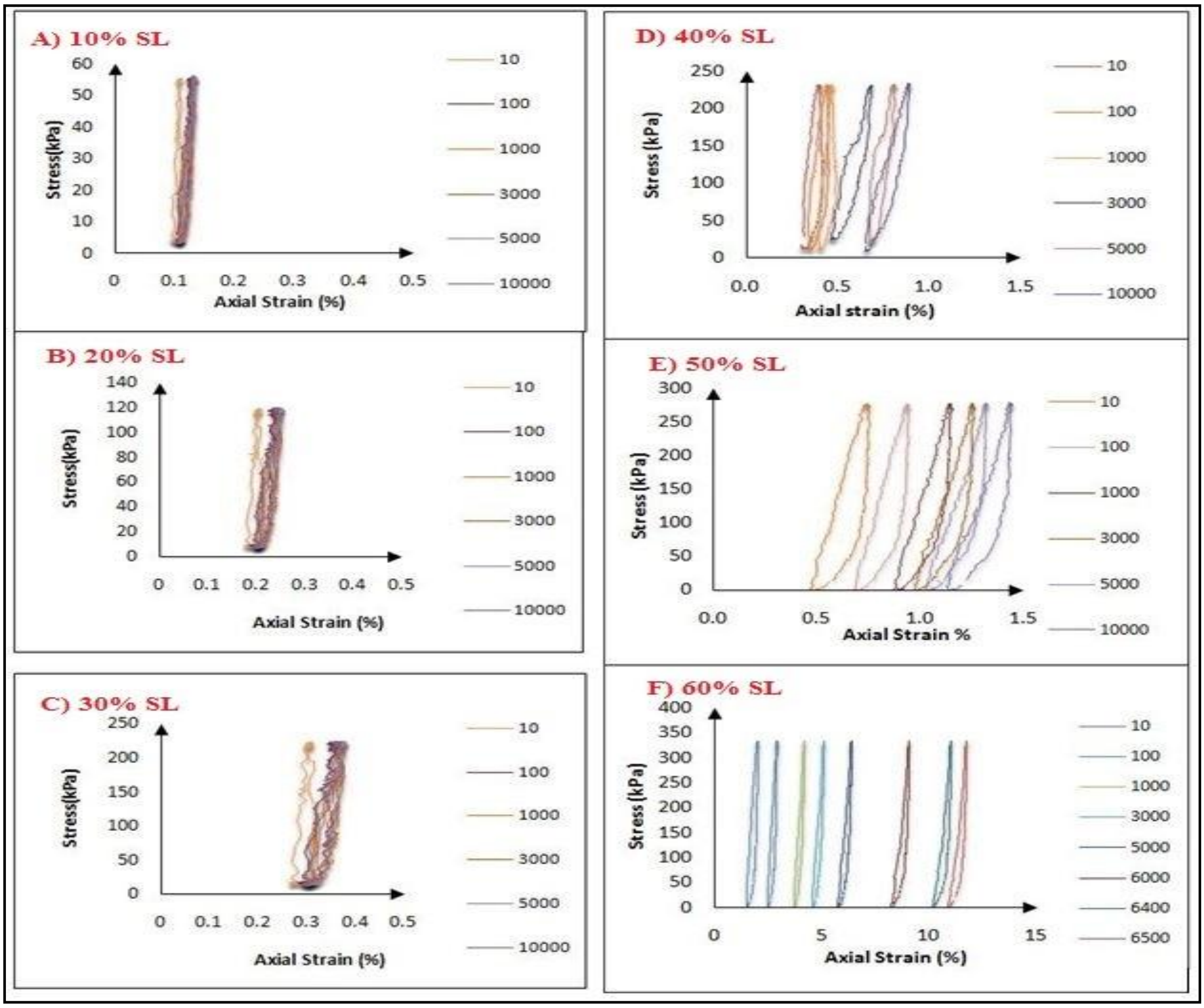

Figure 6.1 Stress–strain loop of Red soil at various stress levels

The cyclic stress-strain behaviour of the Red soil specimens prepared at OMC are shown in Figure 6.1. The results are shown for different confining pressures and stress levels ranging from 10% to 60%. From Figure 6.1, it can be observed that the specimen subjected to 10% stress level at a confining pressure of 20 kPa shows very narrow hysteresis loops which indicates less energy dissipation. The loops are very narrow and almost closed indicating an elastic shakedown. In the case of 20% stress level also elastic shakedown is reached after a few cycles. Though the loops for initial cycles are a little wider for 20% stress level they come to elastic equilibrium after a few cycles. Even 30% stress level shows the same trend, with wider hysteresis loops initially and attaining elastic equilibrium after about 50 cycles. On the other hand 40% stress level showed remarkable difference in the behaviour, where not only the hysteresis loops are wider but also considerable plastic deformation is observed. There is accumulation of plastic deformation up to 4000-5000 cycles and then it almost ceases. This behaviour can be explained using the plastic creep stage in the shakedown theory, where the net plastic deformation continues until it reaches a specific limit and then the stress-strain loops are closed (Yang and Huang 2007).

At 50%, stress level also, the plastic creep behavior can be observed with a rapid accumulation of plastic strain in the initial cycles and a constant rate later. However, when 60% stress level is applied, the material response can be found to be different. The hysteresis loops were very wide and the plastic strain rate increased rapidly and lead to the failureof the sample. The commencement of the failure process can be recognized by an increasing rate of permanent strain development, as explained by the incremental collapse in the shake down theory. This kind of behavior would result in the failure of the pavement by strain accumulation in the subgrade layer, experienced as rutting at the surface (Werkmeister et al., 2004, Werkmeister 2006).

6.2.2 Permanent Deformation Behaviour (Red soil)

Variation of permanent strain with number of load cycles of the Red soil specimens prepared at OMC under constant confining pressure is shown in Figure 6.2.From Figure 6.2, it can be clearly observed that the samples attained elastic limit within first few cycles for the stress levels up to 30%. At 40% and 50% stress levels, accumulation of permanent strain is rapid at initial load cycles and later the rate of accumulation become constant. This stage represents the plastic creep stage, according to shakedown theory. In the case of 60% stress level, rate of plastic strain accumulation increases rapidly leading to failure of the sample which corresponds to the incremental collapse stage.

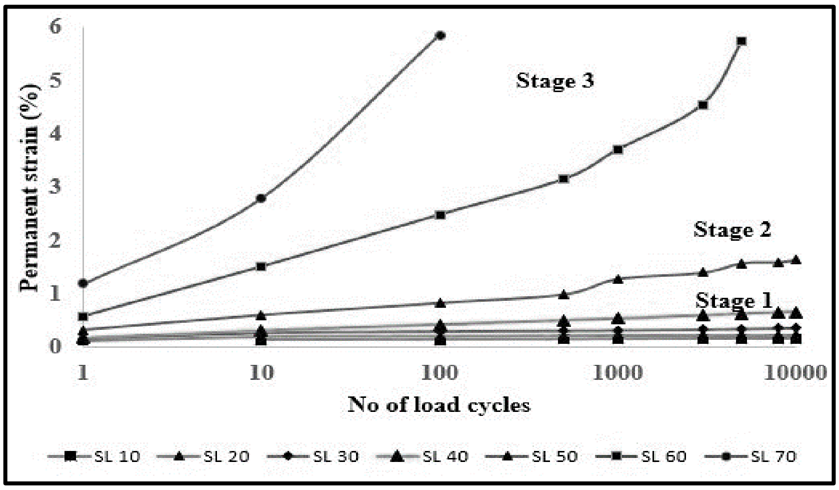

Figure 6.2 Variation of permanent deformation with stress levels of Red soil (OMC)

Figure 6.3 represents the variation of plastic strain with number of load cycles of the CCR stabilized red earth samples prepared at OMC. From Figure 6.3, it can be noted that plastic creep stage shifted from 50% stress level to 60% stress level and the effect of CCR stabilization will reduce the accumulation of plastic deformation. The plastic strain developed is much low for CCR stabilized Red soil when compared to same stress levels for the Red soil.

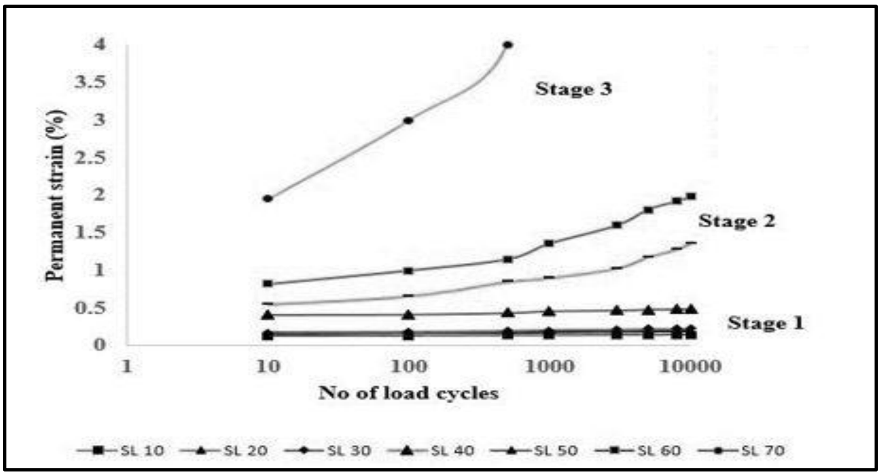

Figure 6.3 Variation of permanent deformation with stress levels of CCR stabilized Red soil (OMC)

Figure 6.4 depicts the variation of permanent strain with number of load cycles for the specimens prepared at OMC+2%. From Figure 6.4, it can be noted that for 10% and 20% stress levels, the sample shows negligible accumulation of plastic strain, whereas for 30% stress level, there is an increment in the plastic strain accumulation with increase in the number of cycles. The stable zone lies between 20 to 30 percent stress level. Stress levels of 30% and 40% exhibits plastic creep behavior and the only difference with the sample prepared at OMC is the magnitude of initial deformation. On the other hand, 50 and 60 percent stress levels display incremental collapse behavior leading to the failure of the sample.

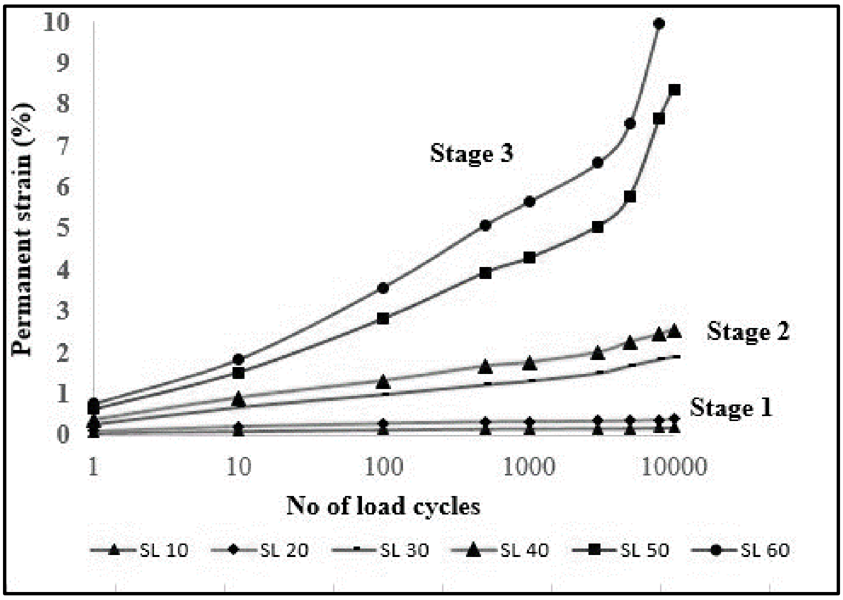

Figure 6.4 Variation of permanent deformation with stress levels of Red soil (OMC+2%)

Variation of permanent strain with number of load cycles of the CCR stabilized Red soil samples prepared at OMC+2% shown in Figure 6.5. From the figure, it can be noticed that the effect of confining pressure is increasing from the stress level 30% and above. The upper limit of the elastic shakedown state has been increased to 30%. The upper limit for the plastic creep stage is found to be at stress level 50%. These results indicate that the elastic shakedown range lies in between 20% and 30%.

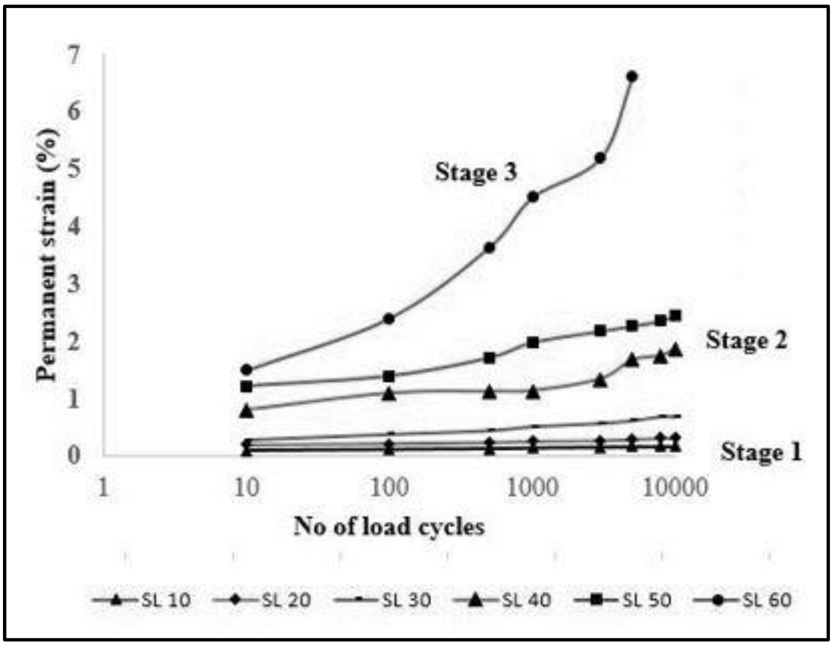

Figure 6.5 Variation of permanent deformation with stress levels of CCR stabilized Red soil (OMC+2%)

Similar graphs showing the plastic strain accumulation with number of load cycles for the samples prepared with initial moisture content of OMC+4%are presented in Figure 6.6. From Figure 6.6, it can be observed that the upper limit of the elastic shakedown state is 20% and 30% stress levels exhibits plastic creep behavior. The rest of stress levels (40%, 50%, 60%) come under the incremental collapse phase. Sudden transition from the stable state to the unstable state can be observed in these cases, which emphasize the effect of moisture content on permanent deformation behavior.

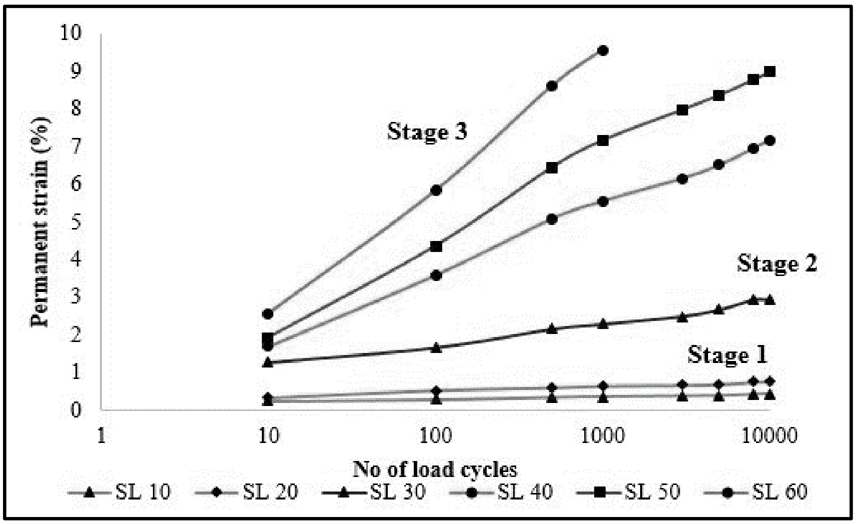

Figure 6.6 Variation of permanent deformation with stress levels of Red soil (OMC+4%)

Figure 6.7 exhibits the variation of permanent strain for CCR stabilized Red soil for samples prepared with OMC+4%. It is clearly observed that the effect of stabilization is higher when the specimen is close to saturation. Unlike in the case for natural Red soil, a definite plastic creep stage can be observed at 40% stress level before the incremental collapse.

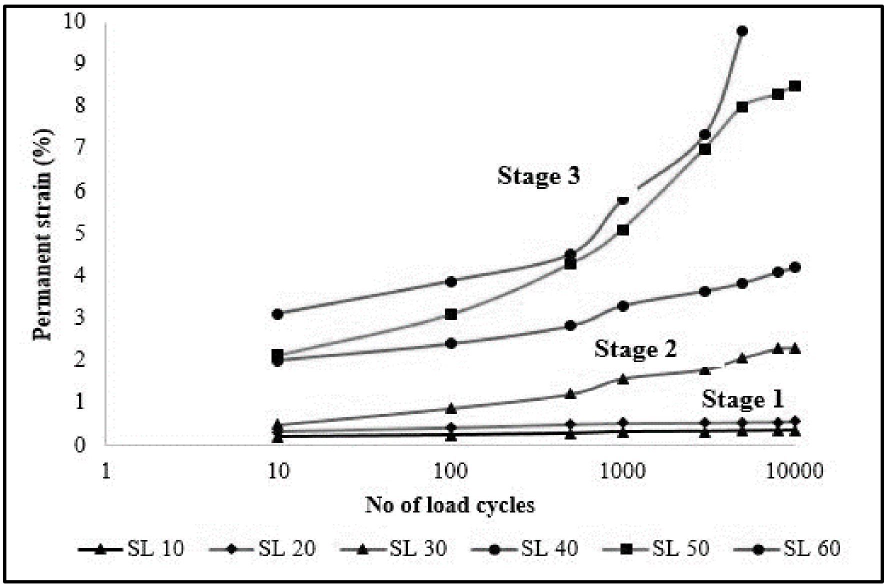

Figure 6.7 Variation of permanent deformation with stress levels of CCR stabilized Red soil (OMC+4%)

6.2.3 Effect of water content (Red soil)

Figures 6.8 and 6.9 presents the variation of permanent strain with deviator stress levels for the specimens prepared at OMC, OMC+2% and OMC+4%. Figures 6.8 and 6.11 illustrates the permanent strain for natural Red soil and CCR stabilized Red soil, respectively. From Figure 6.8, it can be noted that samples were in elastic shakedown stage for 10% and 20% stress levels. Similarly, stress levels corresponding to 30% - 50% fall in the plastic shakedown stage and stress levels corresponding to 60% fall into incremental collapse stage. The effect of water content on the accumulation of permanent strain is less at both elastic shakedown stage and incremental collapse stage, whereas the effect of water content is more pronounced in the plastic shakedown stage. From Figure 6.9, it can be observed that samples reached plastic shake down stage for 40% and 50% stress levels.

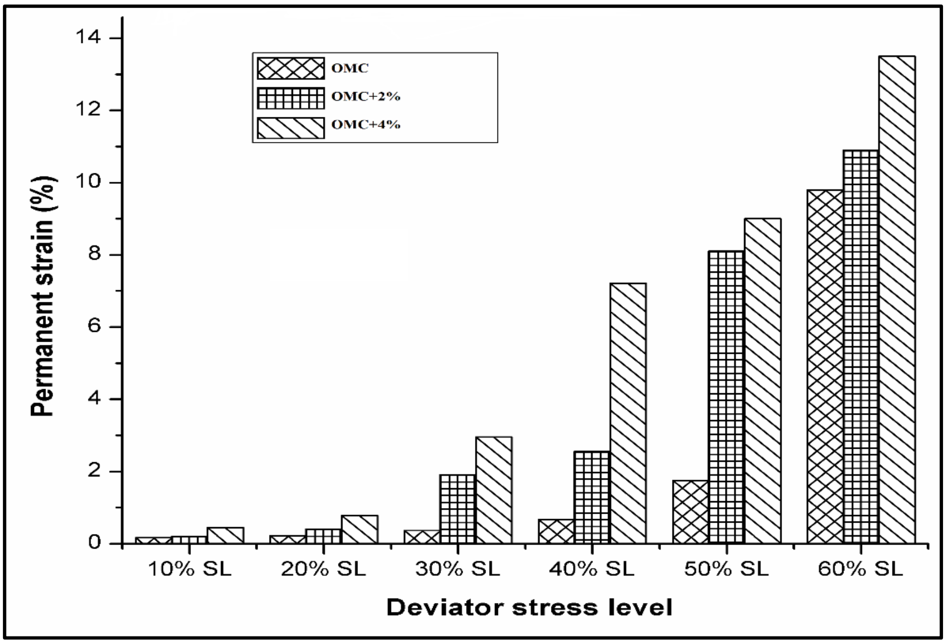

Figure 6.8 Variation of permanent strain with deviator stress level at OMC, OMC+2% and OMC+4% (Red soil)

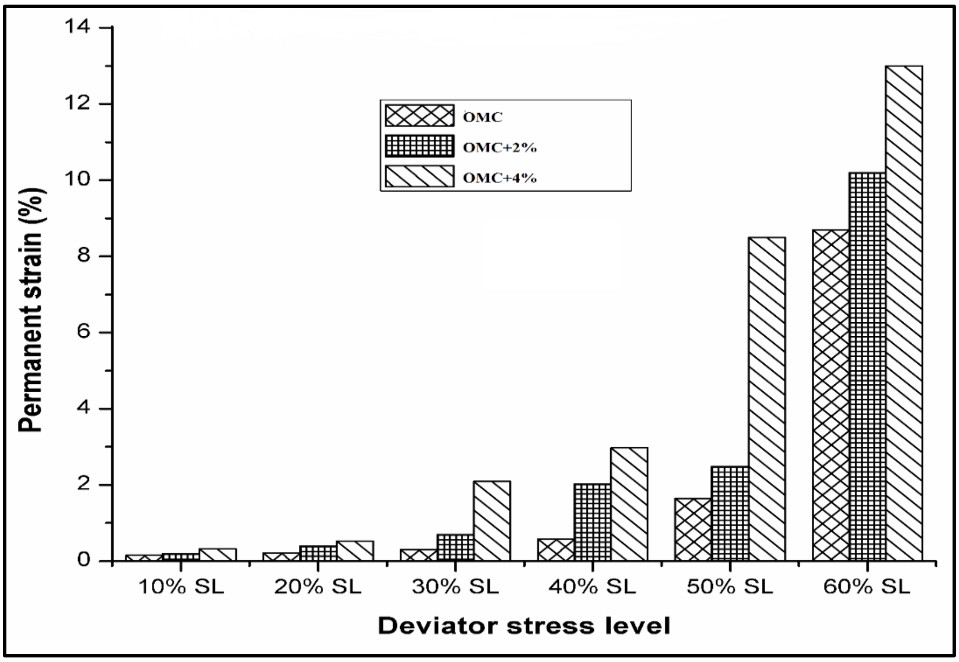

Figure 6.9 Variation of permanent strain with deviator stress level at OMC, OMC+2% and OMC+4% (CCR stabilized Red soil)

6.2.4 Permanent Deformation Behaviour (BC soil)

Figure 6.10exhibits the accumulation of permanent strain for the Natural BC soil samples prepared at optimum moisture content. It can be clearly observed from Figure 6.10 that for untreated Black cotton soil the rate of increase in plastic strain comes down after the initial hundred cycles forstress levels less than 40%. This phase corresponds to elastic shake down according to the shakedown theory. For 50% and 60% stress levels, the plastic creep stage can be observed in which the strain increases at a constant rate after the rapid development observed in the initial cycles. Incremental collapse stage with rapid development of plastic strain leading to collapse can be noticed for 70% stress level.

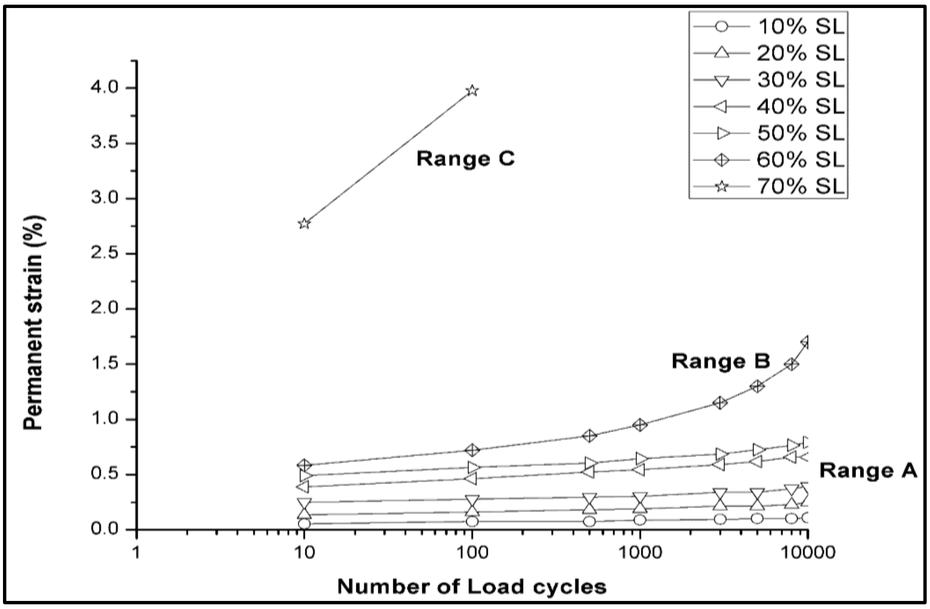

Figure 6.10 Variation of permanent deformation with stress levels of BC soil (OMC)

Variation of permanent strain with number of load cycles for the CCR stabilized BC soil specimens prepared at OMC is shown in Figure 6.11. It is observed that the stabilization is less effective for samples prepared at OMC, whereas no change is observed in the shake down ranges.

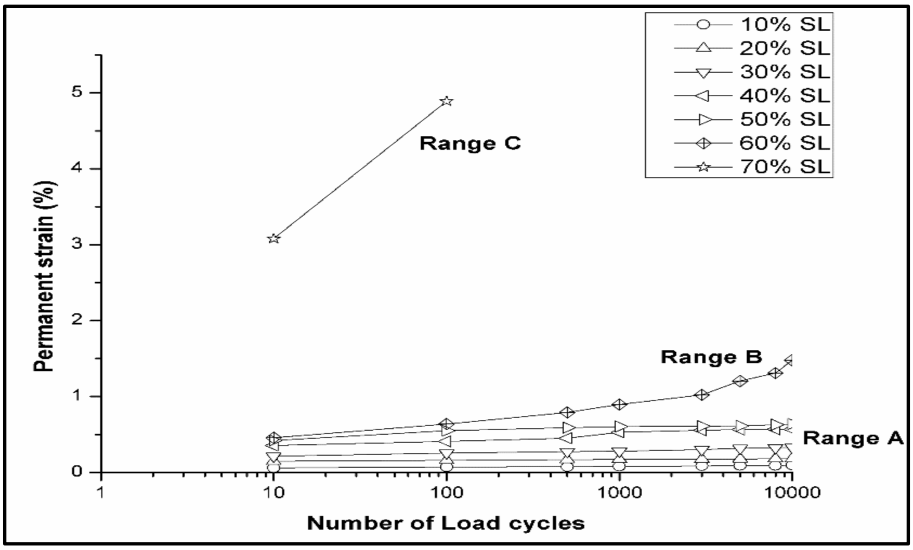

Figure 6.11 Variation of permanent deformation with stress levels of CCR stabilized BC soil (OMC)

The permanent deformation behaviour for untreated BC soil specimens prepared with OMC + 2% moisture is given in Figure 6.12. From Figure 6.12, elastic shake down phase for virgin Black cotton soil is observed for stress level less than 30%. The behaviour at 40% and 50% stress levels indicates plastic creep stage with constant rate of strain increment with number of cycles. In the case of 60% stress level, rapid development of strains can be noticed which represents the incremental collapse phase.

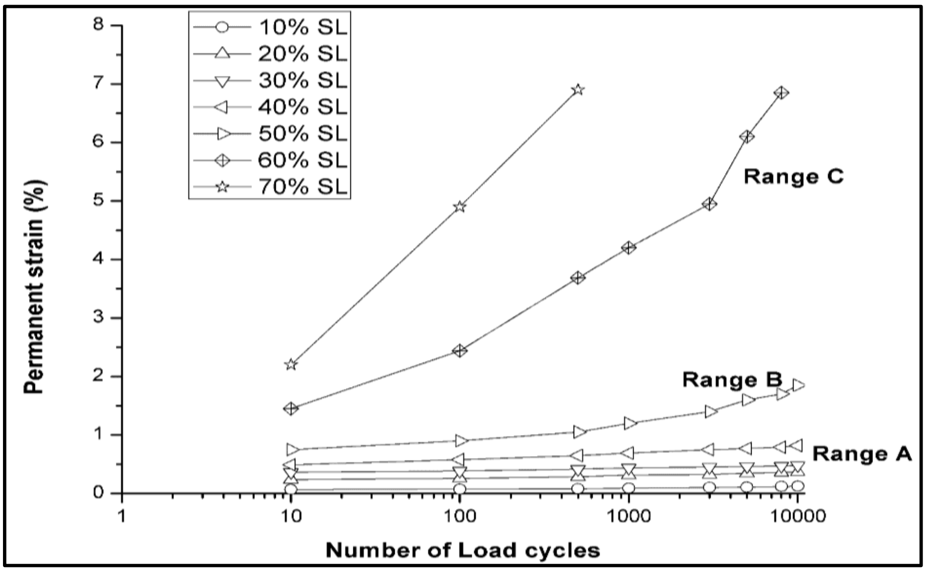

Figure 6.12 Variation of permanent deformation with stress levels of BC soil (OMC+2%)

Similar results for the CCR addition of soil samples prepared with OMC+2% are shown in Figure 6.13. From Figure 6.13, elastic shake down phase is seen up to a stress level of 40%. For 50% and 60% stress levels plastic creep stage is observed whereas incremental collapse stage with rapid plastic strain increments is noticed in the case of 70%.

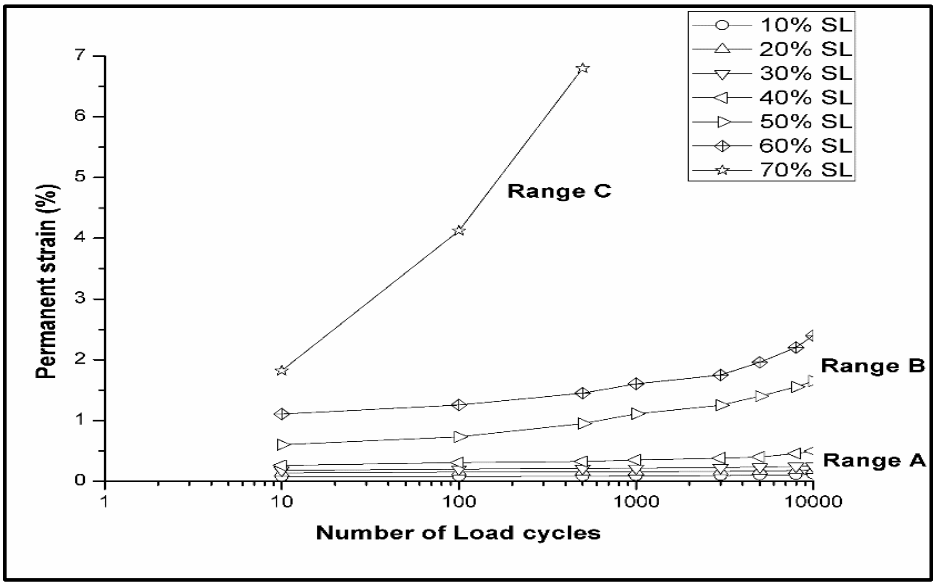

Figure 6.13 Variation of permanent deformation with stress levels of CCR stabilized BC soil (OMC+2%)

Figure 6.14represents the accumulation of permanent strain for the untreated and treated soil samples prepared with OMC+4%.The effect of higher moisture content is evident from figure 6.14 as the stress levels for elastic shake down stage, plastic creep stage and incremental collapse stage have reduced to 20%, 40% and 50% respectively.

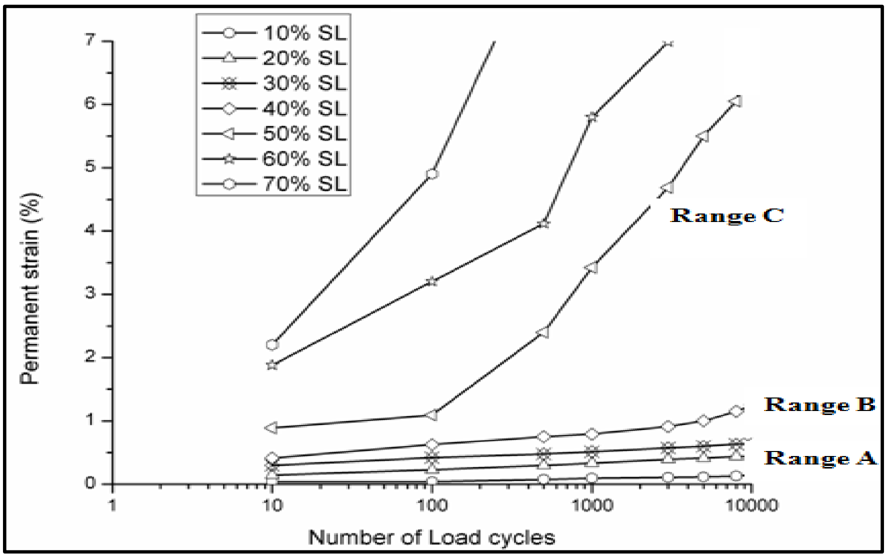

Figure 6.14 Variation of permanent deformation with stress levels of BC soil (OMC+4%)

Variation of accumulation of plastic strain for CCR stabilized BC soil prepared with OMC+4% shown in figure 6.15. Stabilization with CCR is effective at higher water content which can be observed from figure 6.15 where the elastic shake down phase is seen up to a stress level of 30% and that the incremental collapse has occurred at a stress level of 60% only.

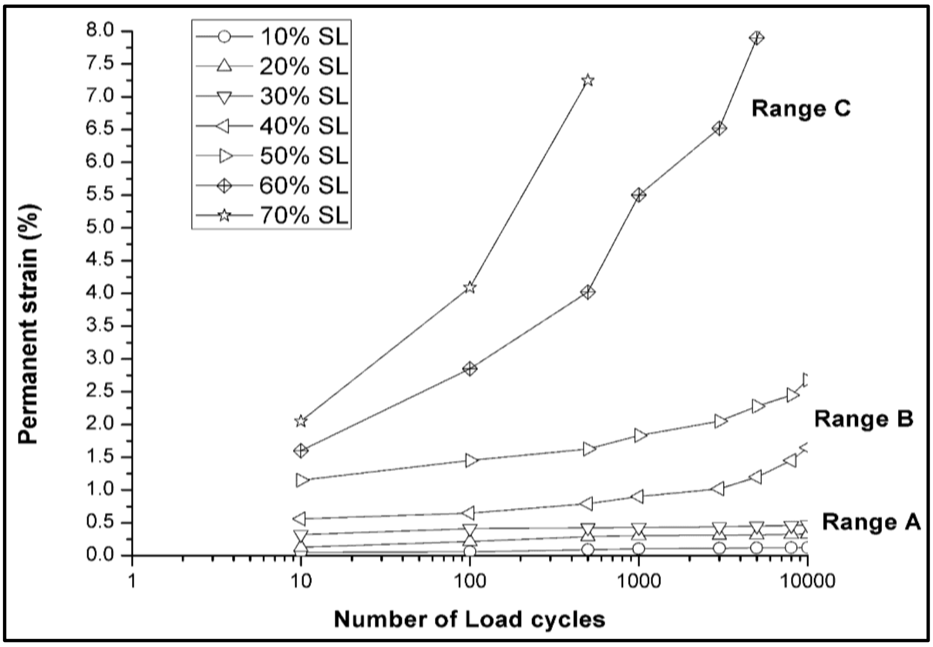

Figure 6.15 Variation of permanent deformation with stress levels of CCR stabilized BC soil (OMC+4%)

6.2.5 Effect of water content (BC Soil)

Figures 6.16 and 6.17 presents the variation of permanent strain with deviator stress levels for the specimens prepared at OMC, OMC+2% and OMC+4%. Figure 6.16 and 6.19 explains the permanent strain for natural BC soil and CCR stabilized BC soil respectively.

From figure, it can be noted that samples were in elastic shakedown stage for 10%, 20% and 30% stress levels.For 40% and 50% stress levels plastic creep stage is noted whereas incremental collapse stage with rapid plastic strain increments is noticed in the case of 60%. The effect of water content on the accumulation of plastic strain is less at both elastic shakedown stage and incremental collapse stage whereas the effect of water content is more pronounced in the plastic shakedown stage. For CCR stabilized soils (Figure 6.17), it can be observed that samples reached plastic shake down stage for 40% and 50% stress levels.

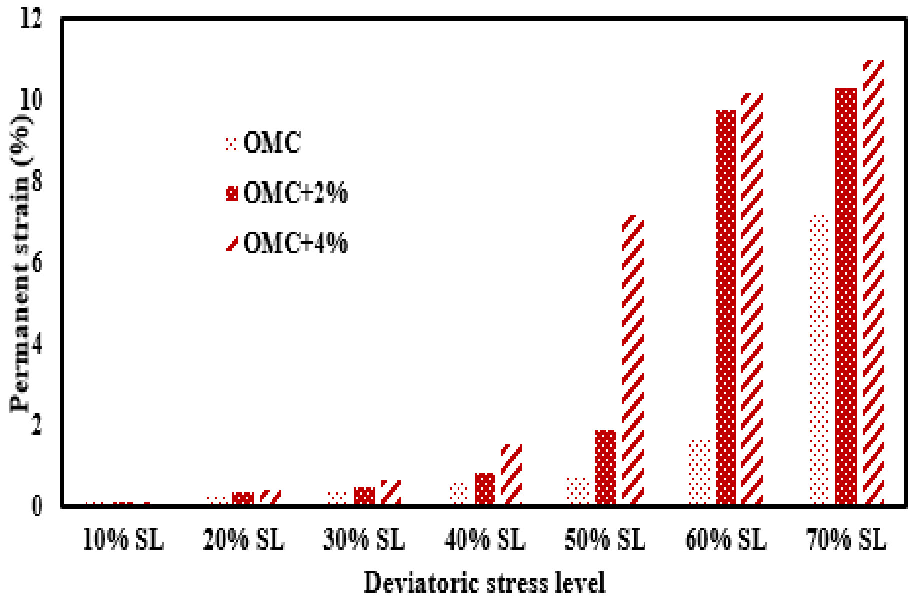

Figure 6.16 Variation of permanent strain with deviator stress level at OMC, OMC+2% and OMC+4% (BC soil)

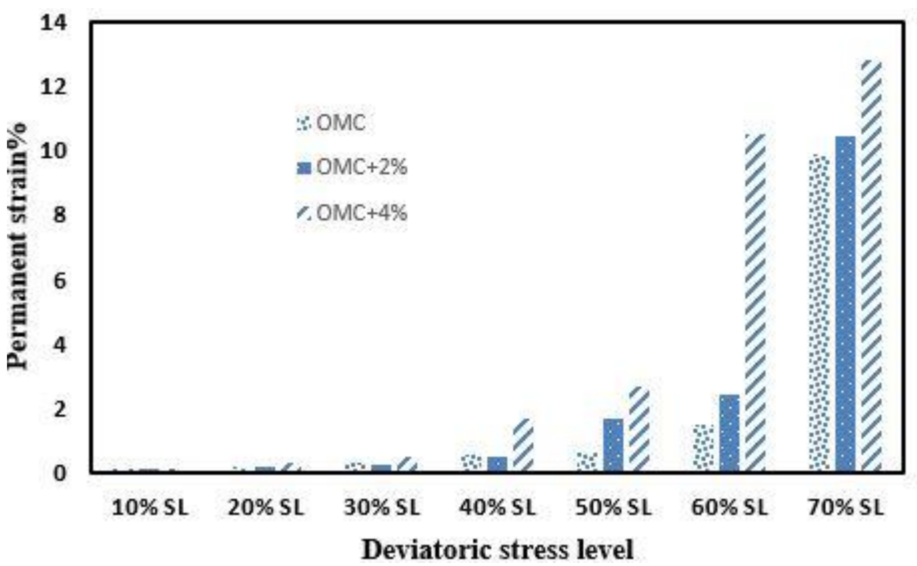

Figure 6.17 Variation of permanent strain with deviator stress level at OMC, OMC+2% and OMC+4% (CCR stabilized BC soil)

6.3 Shakedown Concept and Plastic Strain Rate

Yang and Huang (2007) used plastic strain rate to demarcate the permanent deformation behaviour based on shake down theory. Plastic strain rate is defined as the ratio of plastic strain rate to plastic strain (%). Figure 6.18 represents the variation of plastic strain rate with plastic strain for untreated and treated BC soil samples prepared at different moisture contents. Figure 6.18a shows the variation of plastic strain rate to plastic strain (%) for samples prepared at OMC. From Figure 6.18a, it can be clearly noted that the slope of the curve is almost negligible between plastic strains to plastic strain (%) up to 40% stress levels. This stage represents the elastic shake down. At 50% and 60% stress level, slope of the curve is nearly constant. In case of 70% stress level, slope of the curve keeps on increasing up to failure, which represents incremental collapse stage according to shake down theory. From Figure 6.18b, it can be noted that accumulation of permanent strain decreases with the addition of CCR. Here, elastic shake down stage is extended up to 50% stress level and incremental collapse stage started from 70% stress level. For samples prepared at OMC+2% and OMC+4% also same pattern is followed as explained in above cases.

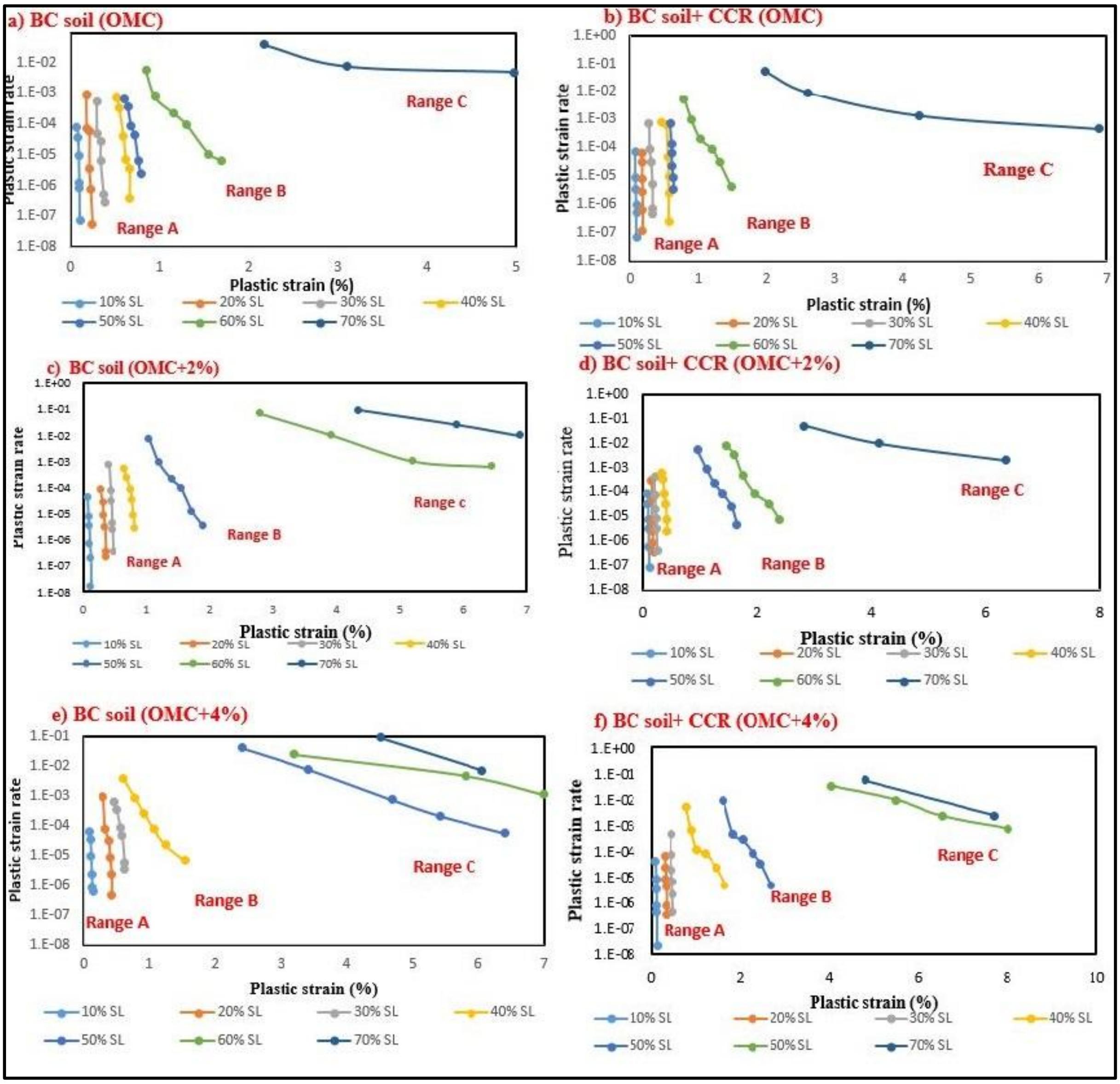

Figure 6.18 Relation of plastic strain rate and number of load cycles for BC soil (OMC)

6.4 Regression models

Permanent strain behaviour also can be modelled with the help of regression models. The assessment of various key factors controlling the permanent strain are required to develop the models. Some of the models reported in the literature were developed by considering the effect of stress levels or the effect of number of load cycles. A few models were developed by combining the influence of stress levels and number of load cycles. In the present study, power law model(Monismith et al. (1975)) and VTT model were used to fit the experimental data. Monismith et al. (1975) proposed power law model. In this model, the permanent strain developed is correlated to the number of loading cycles by a simple power law equation and it is also known as log-log model.

$$\epsilon_p = \epsilon_{app} \ N^K \qquad 6.1$$

ε_p = permanent strain, N =number of load cycles; ε_{app}, k are material parameters

Tables 6.1 and 6.2 shows the values of material parameters (ε_{app}, k) and R^2values for different stress levels, water content and confining pressures for Red soil, whereas, Tables 6.3 and 6.4 represents material parameters for BC soil. From the tables it can be seen that the determination coefficient (R^2) values are more than 0.92.

Table 6.1 Power law model parameters of Red soil

Water Content	Stress Level	K	ε_{app}	R^2
OMC	10 %	0.030	0.125	0.944
	20%	0.019	0.184	0.920
	30%	0.043	0.241	0.979
	40%	0.103	0.263	0.833
	50%	0.117	0.387	0.992
	60%	0.222	0.864	0.923
OMC+2%	10 %	0.055	0.120	0.954
	20%	0.075	0.241	0.96
	30%	0.139	0.517	0.979
	40%	0.165	0.689	0.982
	50%	0.213	0.918	0.928
	60%	0.229	1.307	0.921
OMC+4%	10 %	0.082	0.204	0.927
	20%	0.074	0.389	0.964
	30%	0.118	1.001	0.977
	40%	0.137	2.048	0.983

	50%	0.142	2.507	0.971
	60%	0.151	3.174	0.947

Table 6.2 Power law model parameters of CCR stabilized Red soil

Water Content	Stress Level	k	ε_{app}	R^2
OMC	10 %	0.039	0.107	0.953
	20%	0.030	0.158	0.955
	30%	0.026	0.236	0.937
	40%	0.063	0.318	0.996
	50%	0.108	0.546	0.996
	60%	0.383	0.424	0.945
OMC+2%	10 %	0.028	0.153	0.965
	20%	0.041	0.274	0.939
	30%	0.051	0.45	0.926
	40%	0.0989	0.736	0.901
	50%	0.109	0.99	0.990
	60%	0.219	1.21	0.992
OMC+4%	10 %	0.032	0.24	0.978
	20%	0.038	0.37	0.955
	30%	0.18	0.241	0.935
	40%	0.214	0.56	0.985
	50%	0.239	1.23	0.954
	60%	0.192	1.883	0.96

Table 6.3. Power law model parameters of BC soil

Water Content	Stress Level	Deviator Stress (kPa)	K	ε_{app}	R^2
OMC	10	42.12	0.0898	0.0457	0.9722
	20	84.25	0.0811	0.1101	0.9751
	30	126.37	0.0627	0.2065	0.9251
	40	168.49	0.0764	0.3251	0.9961
	50	210.62	0.0676	0.4098	0.9772
	60	252.74	0.1557	0.359	0.9319
	70	294.86	0.1694	1.8767	0.991
OMC+2%	10	19.76	0.0941	0.0497	0.9238
	20	39.52	0.0667	0.1954	0.9765
	30	59.28	0.0421	0.324	0.9855
	40	79.05	0.0736	0.4174	0.9992
	50	98.81	0.1335	0.5017	0.9618
	60	118.57	0.2524	0.7554	0.9596
	70	138.33	0.3483	0.9909	0.99
OMC+4%	10	11.47	0.2014	0.0214	0.9286
	20	22.93	0.1609	0.1042	0.9886
	30	34.39	0.1077	0.2403	0.9906
	40	45.86	0.1492	0.2928	0.9755
	50	57.33	0.3252	0.3388	0.9685
	60	68.79	0.2637	0.9402	0.9707
	70	80.26	0.2904	1.5884	0.98

Water Content	Stress Level	Deviator Stress (kPa)	k	ε_{app}	R^2
OMC	10	170.56	0.00593	0.053	0.983
	20	341.12	0.0111	0.1637	0.9223
	30	511.68	0.0652	0.1849	0.9833
	40	682.24	0.0721	0.304	0.9765
	50	852.53	0.1492	0.3337	0.9565
	60	1023.04	0.0605	0.3755	0.9626
	70	1193.54	0.3857	0.5642	0.9379
OMC+2%	10	133.65	0.0673	0.0608	0.91
	20	267.3	0.0283	0.1748	0.9791
	30	400.95	0.0887	0.2125	0.9003
	40	534.6	0.0477	0.44	0.9471
	50	668.25	0.1506	0.3906	0.98
	60	801.9	0.1115	0.7743	0.9201
	70	935.55	0.3167	0.9007	0.9923
OMC+4%	10	127.04	0.0238	0.1423	0.9342
	20	254.1	0.0443	0.2241	0.9478
	30	381.13	0.0626	0.2871	0.9373
	40	508.17	0.1576	0.3338	0.9288
	50	635.22	0.1182	0.8312	0.9922
	60	762.26	0.2675	0.8275	0.9922
	70	889.31	0.3213	0.9643	0.9977

Table 6.4 Power law model parameters of CCR stabilized BC soil

The equation for power law model does not take the effect of stress level into account. The VTT model developed by Korkiala-Tanttu (2005) incorporated another term R called the failure ratio into the equation and thereby made a prediction model that takes in to consideration the effect of stress levels also. According to VTT model, the permanent strain can be obtained based on equation 6.2.

$$e_p = C * N^b * {}^{R}/_{1-R} \qquad 6.2$$

In equation (6.2), ε_p is the permanent axial strain (%), N is the number of loading cycles and R is the failure ratio. C and b are the material parameters obtained by multiple regression using the test data from repeated triaxial tests. The failure ratio R is introduced in the model to consider the effect of stress level relative to the shear strength of the material in the development of plastic deformation. Table 6.5 and 6.6 shows the values of the material parameters and R^2 values are also greater than 0.9 for this model proving a good correspondence.

Table 6.5 Regression parameters of VTT model (Red soil)

	Red soil			**CCR stabilized Red soil**		
Water content	**C**	**b**	**R^2**	**C**	**b**	**R^2**
OMC	0.426	0.105	0.935	0.585	0.089	0.868
OMC+2%	1.092	0.14	0.904	1.154	0.092	0.934
OMC+4%	2.231	0.111	0.819	0.754	0.201	0.926

Table 6.6 Regression parameters of VTT model (BC soil)

	BC soil			CCR stabilized BC soil		
Water content	**C**	**b**	**R^2**	**C**	**b**	**R^2**
OMC	0.455	0.072	0.991	0.481	0.052	0.990
OMC+2%	0.74	0.052	0.938	0.572	0.066	0.975
OMC+4%	0.52	0.12	0.966	0.754	0.055	0.920

6.5 Summary

In this chapter, a series of repeated load triaxial tests were carried out on clayey subgrade soil in order to study its permanent deformation behaviour. The effect of CCR stabilization, moulding water content and stress levels on the permanent deformation behaviour of the subgrade soil was investigated and deformation behaviour was analysed through shakedown theory. For Black cotton soil specimens prepared at its OMC, the elastic shakedown limit is found to be at 50% stress level, whereas plastic creep stage was observed at 60% stress level. For a higher water content OMC+2% and OMC+4%, the elastic shake down range was found in between 40% and 50% stress levels and the plastic creep stage commenced at 40% stress level itself. For Red soil, the elastic shakedown limit prepared at its OMC was found to be at the 30% stress level, whereas its plastic creep stage was observed at 40% stress level. For a higher moulding water contents of OMC+2% and OMC+4%, the elastic shake down range was found to be below 20% stress level and the plastic creep stage commenced at 30% stress level itself. Therefore, it can be concluded that if the sub-grade water content increases, the stress level corresponding to shakedown becomes low leading to rutting under repeated load.

The stabilization is more effective in plastic creep and incremental stage compared to elastic shakedown stage. Conversely, the relationship between accumulated plastic strain and plastic strain rate canbe used for the purpose. VTT model and power law model were found to be useful to fit the experimental data for CCR modified soils and natural soil except for incremental collapses stress levels.

Chapter 7

Load deformation behavior of Laboratory pavement model

7.1 Introduction

The use of waste from construction and demolition (C&D) is a subject that has acquired great importance in recent years. Thousands of tons of waste is being produced every year in India and there is a need to make use of it, in order to make the construction industry in India more sustainable. In fact, vast majority of the natural resources extracted in India are used in construction, representing thata large amount of natural energy was being consumed in various construction activities. In the same way, the residues resulting from construction indicates that a large part of the waste was produced. These residues, which are mixtures of hazardous and non-hazardous waste unreasonably and unduly, occupy the soil. Therefore, construction and demolition waste management (RCD) has become a prior economic and environmental issue on the global scale.

In recent years, many researchers confirmed that construction and demolition waste can be used in base and subbase layers of flexible pavement (O'mahony1997; Bennert et al., 2000; Poon and Chan 2006). It not only decreases the dumping area but also used as an alternative material for virgin aggregates. It is well known that un bound granular layer plays a major structural contribution in the flexible pavement because the load that comes from base and subbase is a major factor for occurrence of rutting on top of the subgrade (Cerni et al. 2012).

The experimental program can be divided into two phases. The first phase deals with determining the resilient and permanent deformation of natural and recycled aggregates. The second phase deals with determining the permanent deformation of pavement model with different subgrade and base layers.

7.2 Deformation characteristics of base materials

Bennert et al., (2000) used recycled aggregates in base and sub-base applications with different proportions of natural aggregates and concluded that 75% of natural aggregates and 25% of recycled aggregates possess similar resilient response as natural aggregates. Motta (2005) observed the resilient modulus of recycled concrete aggregates over a period of time and found that the resilient modulus increases with increasing curing period on account of the existence of un-interacted cement particles in recycled concrete aggregates. Cerni et al. (2012) observed the permanent deformation behaviour of recycled aggregates under different stress conditions and found that at higher stress levels recycled concrete aggregates break easily compared to natural aggregates whereas at lower stress levels variation of accumulation of permanent strain is negligible. Vahid et al. (2014) used RAP and RAC as sub-base material and found that with increase in RAP and RAC content CBR and UCS values were decreased.

In the first phase, permanent deformation tests were conducted according to CEN-2004 (European standards). Resilient modulus of the soil samples was determined from repeated load triaxial test results. Elastic deformation was separated from permanent deformation at 500th loading cycle and this elastic deformation was used to compute the resilient modulus of the subbase material.

In the present phase, behaviour of natural aggregates and virgin aggregates subjected to repeated loading under five deviator stress levels and three confining pressures is noticed. The permanent strain data obtained from the experiments are fitted with two regression models, namely VTT model and power law model, which takes in to account the effect of stress levels and number of load cycles. For resilient modulus values, regression analysis is carried out with the experimental data using universal model and NCHRP model and the corresponding coefficients are obtained.

7.3 Permanent Deformation Behaviour (Natural aggregates)

7.3.1 Static triaxil test

The results of static triaxial tests conducted on natural and recycled aggregate samples prepared at optimum moisture contents under five confining pressure are presented in Figure 7.1.

All the monotonic and repeated triaxial tests were carried out on soil samples with 75 mm diameter and 150 mm height. Prior to the preparation of specimens, granular material was thoroughly mixed with the required amount of water and kept in a desiccator for 24 h to obtain uniform moisture content. The granular material was then statically compacted in a cylindrical mould to obtain samples with the required density and moisture content. The inner surface of the mould was lubricated to reduce the side friction during compaction. After moulding the specimens were immediately extruded from the split mould and then placed in plastic bags and stored in desiccators to avoid significant variations of moisture content before testing.

Figure 7.1presentsa comparison between the undrained shear strength of the natural and recycled aggregate samples prepared at optimum moisture content under five different confining pressures. Increase in deviatoric stress causes decrease in static strength for both natural and recycled aggregates, whereas increase in confining pressure results in increase in the shear strength.

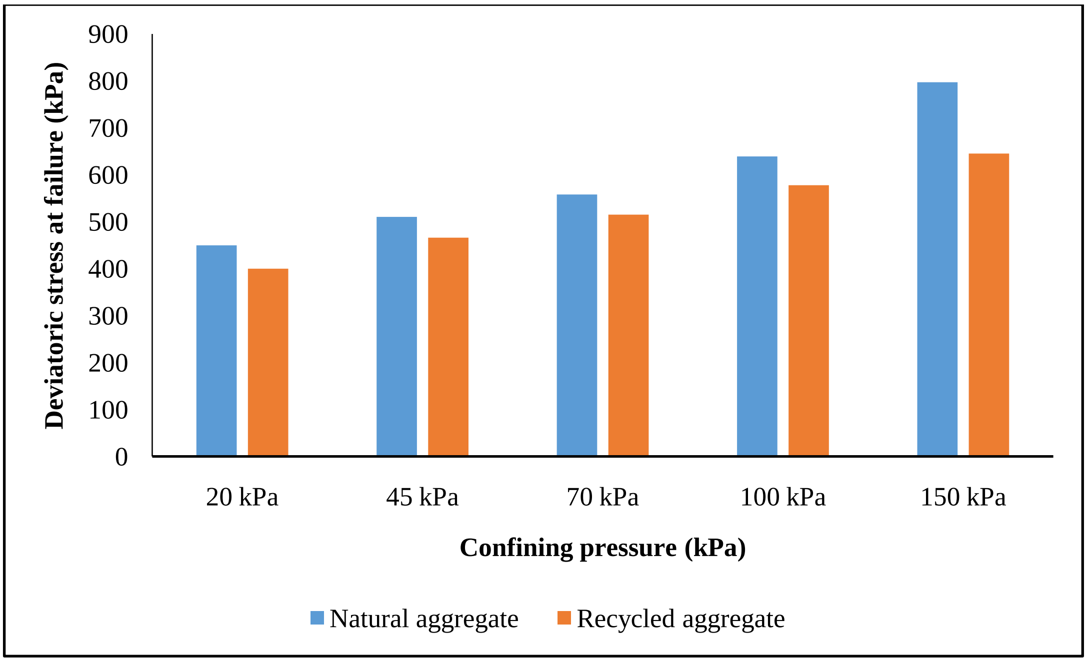

Figure 7.1 Static strength of natural aggregates and recycled aggregates at different confining pressure

The repeated load triaxial test results displaying the permanent deformation behaviour of natural aggregate samples prepared at OMC under 20kPa confining pressure is given in Figure 7.2.

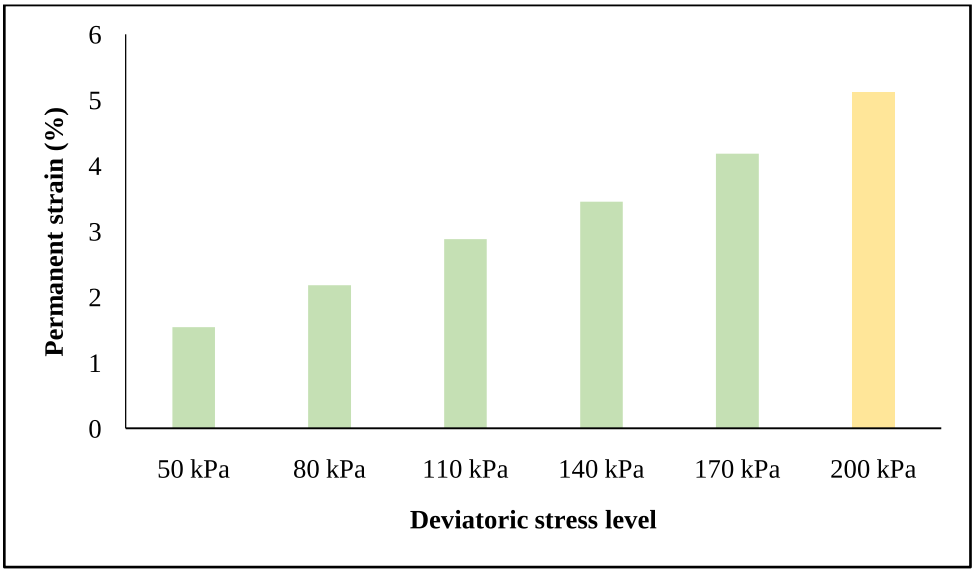

Figure 7.2 Variation of permanent strain of natural aggregate samples for different stress levels under 20 kPa confining pressure

From Figure 7.2, it can be observed that for natural aggregate samples, rate of increase in plastic strain comes down after initial hundred cyclesfor less than 170 kPa deviatoric stress level samples tested under 20 kPa confining pressure. This phase corresponds tothe elastic shake down stageaccording to the shakedown theory. At 200 kPa deviatoric stress, samples possess nearly 3.84mm deformation after 10,000 cycles and this stage represents the plastic shakedown stage. It can be observed from the figure that increase in deviator stress from 50 kPa to 200 kPa results in 7.8 times increment in permanent strain under 20 kPa confining pressure.

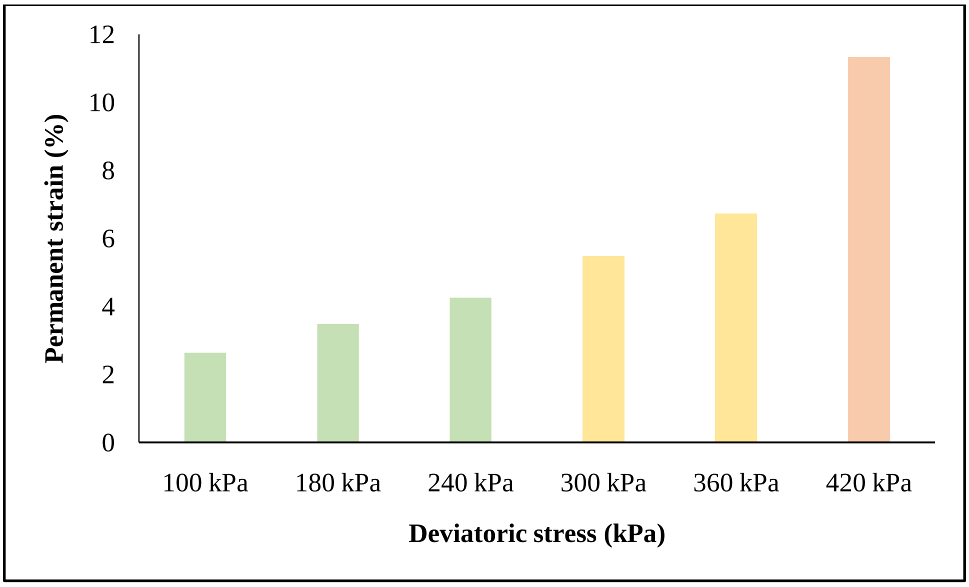

Figure 7.3 Variation of permanent strain of natural aggregate samples for different stress levels under 45 kPa confining pressure

Figure 7.3 represents the permanent deformation of natural aggregate samples prepared under confining pressure of 45 kPa. It can be observed from the graph that fornatural aggregate samples, the behaviour is within the elastic shake down phase for 240 kPa deviatoric stress level under 45 kPa constant confining pressure as accumulation of permanent strain is not observed even after 5000 cycles. The behaviour under 300kPa deviatoric stress level and 360kPadeviatoricstress level,sample showed plastic shake down behaviour. It can be noted from the figure that increase in deviatoric stress from 100 kPa to 420 kPa results in 2.2 times increment in permanent strain under 45 kPa confining pressure. Incremental collapse stage with rapid accumulation of plastic deformations leading to collapse can be noticedfor samples subjected to 420kPa and from graph it can be observed that incremental collapse stage values are near to static failure strength.

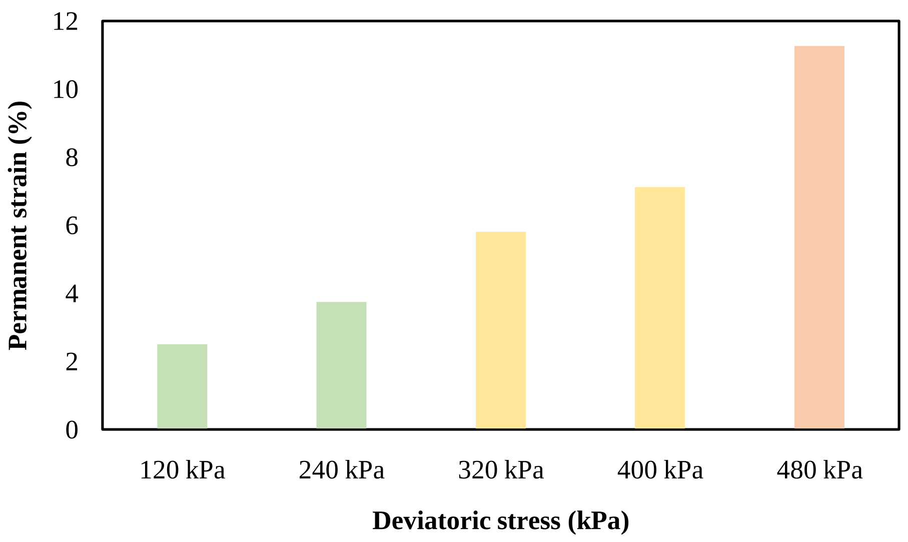

Figure 7.4 Variation of permanent strain of natural aggregate samples for different stress levels under 70 kPa confining pressure

The repeated load triaxial test results of natural aggregate samples prepared at optimum moisture content under 70 kPa constant confining pressure is given in Figure 7.4. From Figure, it can be observed that the sample showed elastic behaviour up to 240 kPa. At 480 kPa and 560 kPa deviator stress levels, sample showed incremental collapse stage and accumulation of permanent strain is nearly equal to failure static strength.

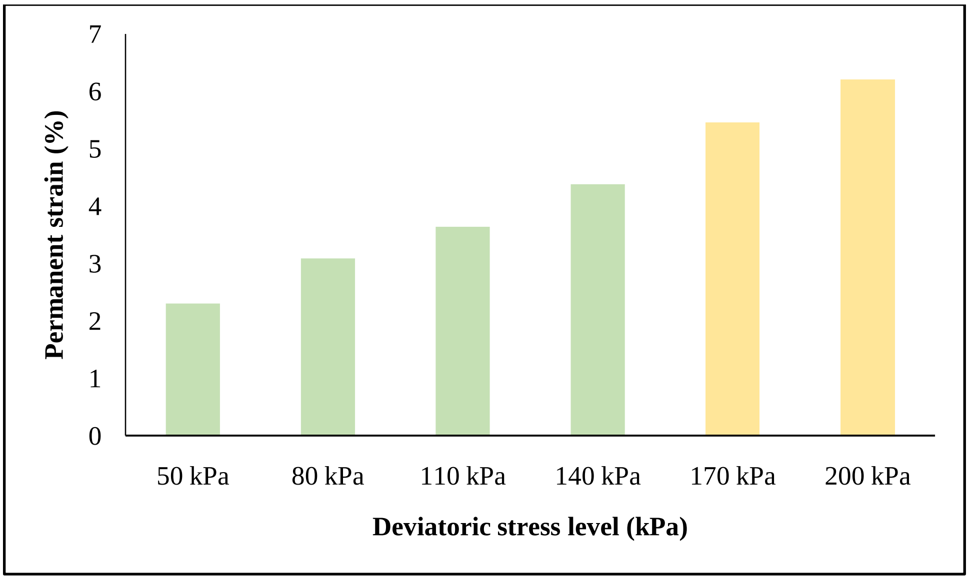

Figure 7.5 Variation of permanent strain of recycled aggregate samples for different stress levels under 20 kPa confining pressure

The repeated load triaxial test results of recycled aggregate samples prepared at optimum moisture content under 20 kPa confining pressure is represented in Figure 7.5. From Figure 7.5, it can be clearly observed that the samples attained elastic limit within first few cycles up to 110kPa deviatoric stress level. At 140 kPa and 170 kPa deviatoric stress levels, accumulation of permanent strain is rapid at initial load cycles and later the rate of accumulation becomes constant. This stage represents the plastic creep stage, according to the shakedown theory. Rapid development of permanent strains can be noticed for samples subjected to 200 kPa deviatoric stress level which represents the incremental collapse phase.

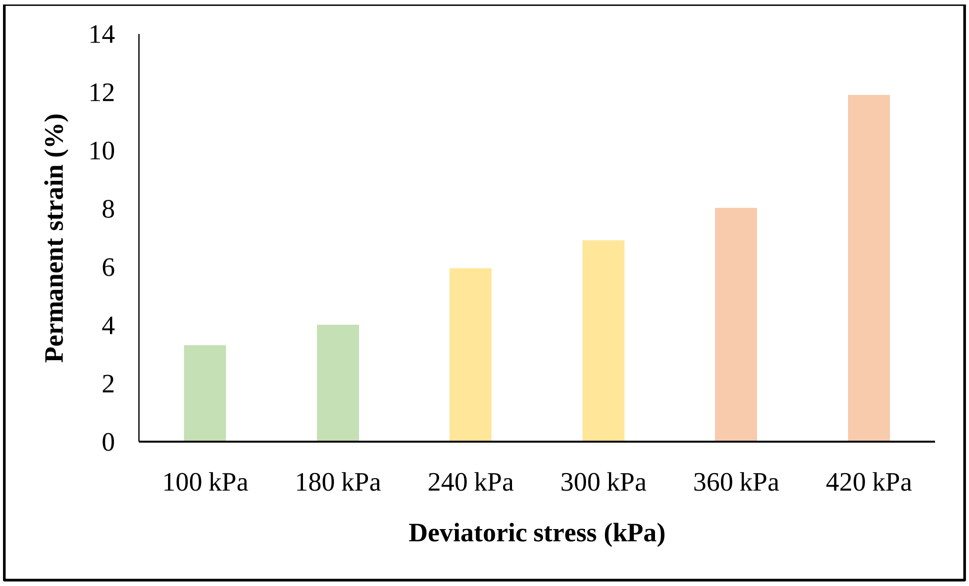

Figure 7.6 Variation of permanent strain of recycled aggregate samples for different stress levels under 45 kPa confining pressure

Figure 7.6 exhibits the permanent strain for recycled aggregate samples prepared at optimum moisture content under 45 kPa confining pressure. From Figure 7.6, it can be observed that for 100 kPa and 180 kPa deviator stress levels, the sample shows negligible accumulation of plastic strain. At 240kPa and 300kPa deviator stress levels, the samples shows plastic deformation. Incremental collapse stage with rapid accumulation of plastic deformations leading to collapse can be noticed for samples subjected to 360 kPa and 420kPa and from graph, it can be observed that the incremental collapse stage values are near to the static failure strength. Incremental collapse stage has started at a deviatoric stress of only 420kPa for natural aggregate sample subjected to 45 kPa confining pressure.

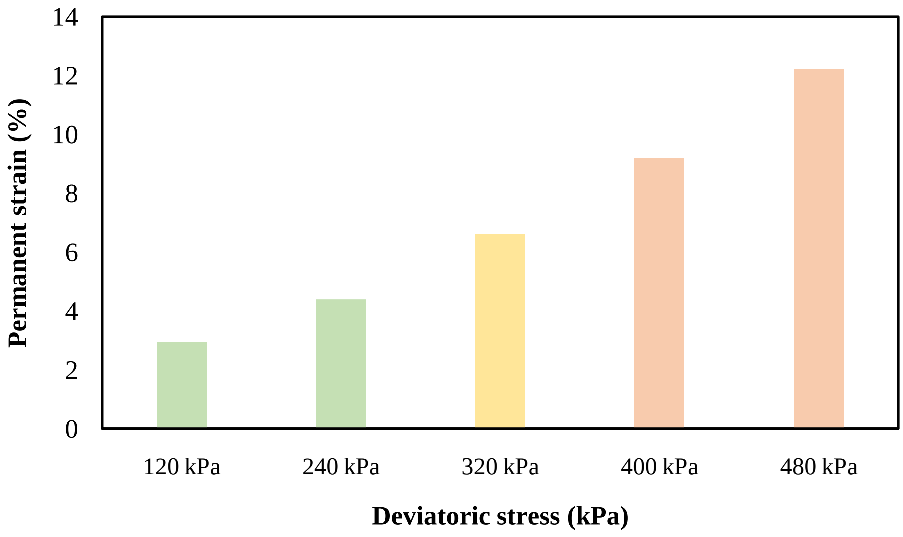

Figure 7.7 Variation of permanent strain of recycled aggregate samples for different stress levels under 70 kPa confining pressure

Similar accumulation of permanent strain for the recycled aggregate samples under 70kPa confining pressure is shown in Figure 7.7. It can be noticed from the graph that for the recycled aggregates samples, the behaviour is within the elastic shake down phase for 240 kPa deviatoric stress level under 70 kPa constant confining pressure as accumulation of permanent strain is not observed even after 5000 cycles. From Figure 7.7, it can be noticed that the plastic creep stage has shifted from 320kPadeviatoric stress level to 400kPadeviatoric stress level when the natural aggregates are replaced with recycled aggregates. Rapid development of permanent strains can be noticed for samples subjected to 480 kPa deviatoric stress level representing the incremental collapse phase.

7.3.2 Regression models

Regression analysis is carried out using the experimental results in order to develop models for predicting the permanent deformation behaviour based on the number of load cycles and stress levels. Power law model and VTT model are two regression models, which can be used to fit the permanent deformation behaviour of subgrade geo materials, obtained from repeated load triaxial tests.Monismith et al. (1975) proposed power law model to correlate the permanent strain with number of load cycles using the following equation.

$$\epsilon_p = \epsilon_{app} \; N^K \qquad 7.1$$

ε_p = permanent strain, N =number of load cycles; ε_{app}, k are material parameters

Table 7.1 Regression parameters of power law model

Confining pressure	**Deviatoric stress (kPa)**	**Natural aggregates**			**Recycled aggregates**		
		A	**b**	**R^2**	**A**	**b**	**R^2**
20 kPa	50	0.394	0.153	0.959	1.447	0.054	0.952
	80	1.042	0.089	0.988	1.296	0.092	0.900
	140	1.008	0.159	0.899	1.264	0.147	0.957
	200	3.483	0.040	0.987	3.849	0.045	0.927
45 kPa	180	2.727	0.036	0.993	2.657	0.041	0.978
	300	2.963	0.084	0.954	2.513	0.108	0.986
	360	4.172	0.053	0.922	6.408	0.010	0.976
70 kPa	180	3.607	0.016	0.986	2.905	0.045	0.923
	300	2.598	0.099	0.946	2.120	0.127	0.957
	360	5.237	0.044	0.978	4.440	0.073	0.949

Table 7.2 shows the values of material parameters (ε_{app}, k) and R^2values for different stress levels, water content and confining pressures for natural aggregate samples whereas Table 7.3 represents the material parameters for recycled aggregate samples. From the tables, it can be seen that the determination coefficient (R^2) values are more than 0.9.

The Power law model does not consider the effect of stress level whereas the VTT model proposed by Korikala- Tantu (2007) includes the effect of stress level using a term called failure ratio (R) as given below.

$$e_p = C\, N^b\, {}^{R}\!/_{1-R} \qquad 7.2$$

Here e_p is the permanent strain, C and b aremodel parameters, N denotes the number of load cycles and R represents the failure ratio. Model parameters (C and b) and the R^2values determined for the natural and recycle aggregate samples are presented inTable 7.4. Both the models are able to fit the experimental data very well for natural aggregate samples as well as the recycled aggregate samples, as indicated by the high R^2values. The models are not able to represent the experimental results corresponding to incremental collapse stage and hence that data is not considered in the regression analysis.

Table 7.2 Regression parameters of VTT model

Confining pressure (kPa)	Natural aggregates			Recycled aggregates		
	C	**b**	**R^2**	**C**	**b**	**R^2**
20	2.804	0.144	0.905	2.6	0.125	0.902
45	2.956	0.151	0.899	2.1	0.148	0.924
70	3.22	0.153	0.919	3.8	0.11	0.88

7.4 Resilient modulus

Empirical procedures used California Bearing Ratio (CBR) as a stiffness parameter of subgrade soil for flexible pavement design. These CBR values failed to reflect the dynamic nature of traffic loads and therefore, the mechanistic design methods preferred to use resilient modulus. Mechanistic-empirical design of flexible pavement AASHTO practice requires resilient characteristics of the samples. Resilient modulus (M_R) of subgrade soil was introduced as a stiffness parameter by AASTHO (1993) and is defined as the ratio of deviatoric stress to recoverable strain for samples subjected to cyclic triaxial tests.

Resilient modulus is sensitive to moisture content, deviator stress and confining pressure (Dai & Zollars, 2002; Malla & Joshi, 2008; Ren & Vanapalli, 2017). In the present study, resilient modulus of the natural and recycled aggregate samples was determined from repeated load triaxial test results. Elastic deformation was separated from permanent deformation at 500th loading cycle and this elastic deformation was used to compute the resilient modulus of the aggregate sample.

Figures 7.8 and 7.9 shows the resilient modulus values of natural aggregate and recycled aggregate samples under different confining pressures and deviatoric stress levels. It can be observed from both the figures that the resilient modulus increases with increasing confining pressure and decreases with increasing deviatoric stress level. It can be observed from Figures 7.8 and 7.9that by replacing natural aggregate with recycled aggregate, nearly 20-50% decrease in resilient modulus is observed.

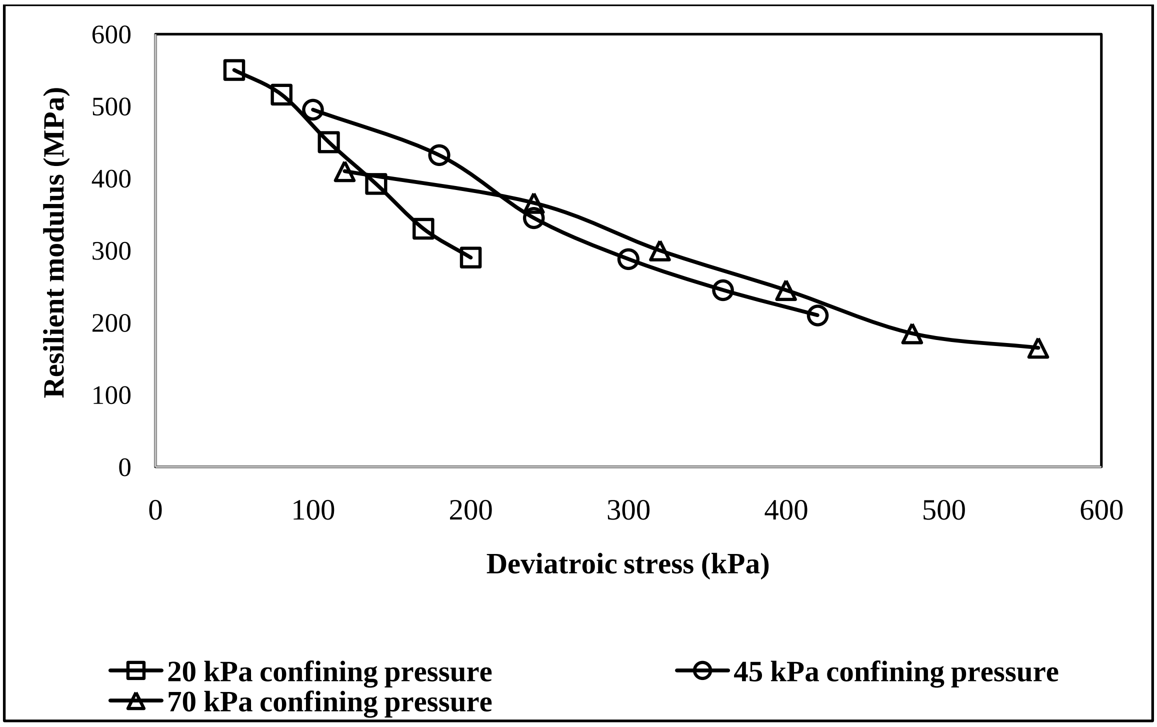

Figure 7.8 Variation of resilient modulus with confining pressure and deviatoric stress levels for natural aggregates

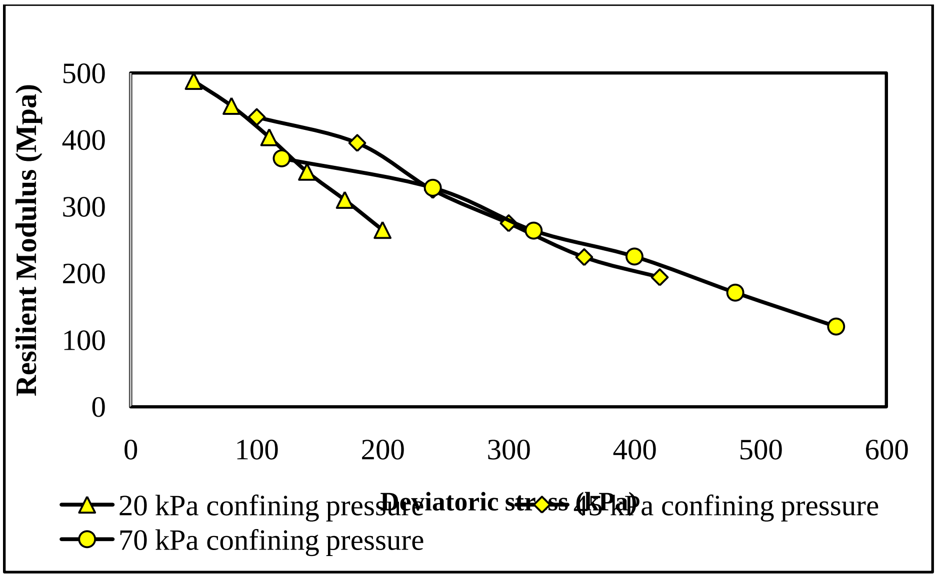

Figure 7.9 Variation of resilient modulus with confining pressure and deviatoric stress levels for Recycled aggregates

Figure 7.10 represents variation of resilient modulus for the natural and recycled aggregates under three different confining pressures. It can be noted from figure 7.10 resilient modulus decreased nearly 20-50% if natural aggregates completely replaced recycled aggregates. Variation of resilient modulus between natural aggregates and recycled aggregates increases with increasing confining pressure.

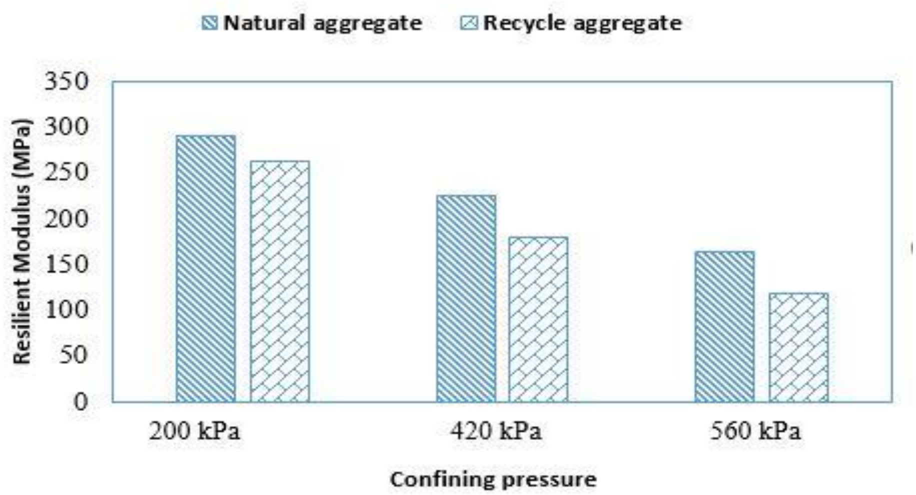

Figure 7.10 Variation of resilient modulus of natural aggregates and recycled aggregates at different confining pressure.

7.5 Laboratory Pavement Model

Five model tests were performed with different base and subgrade layers as shown in table 7.3.

Table 7.3. Pavement model tests

1	Pavement model test 1	Natural aggregate
		Red soil (OMC)
2	Pavement model test 2	Natural aggregate
		Black cotton soil (OMC)
3	Pavement model test 3	Natural aggregate (Saturated)
		Red soil (Saturated)
4	Pavement model test 4	Natural aggregate (Saturated)
		Black cotton soil (Saturated)
5	Pavement model test 5	Recycled aggregate (Saturated)
		Black cotton soil (Saturated)

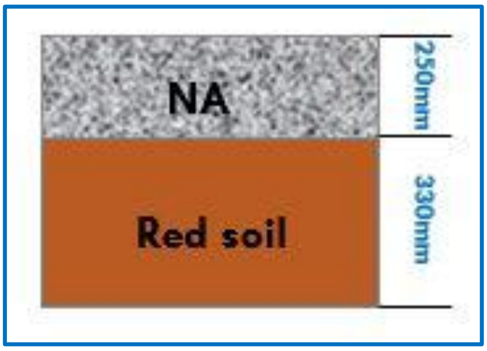

Figure 7.11 Pavement model test 1

Model test 1 consist of Red soil as a subgrade and base layer consist of natural aggregate, both layers are compacted with maximum dry density and moisture content which were obtained from modified proctor test. Each model is applied 550 kPa stress.

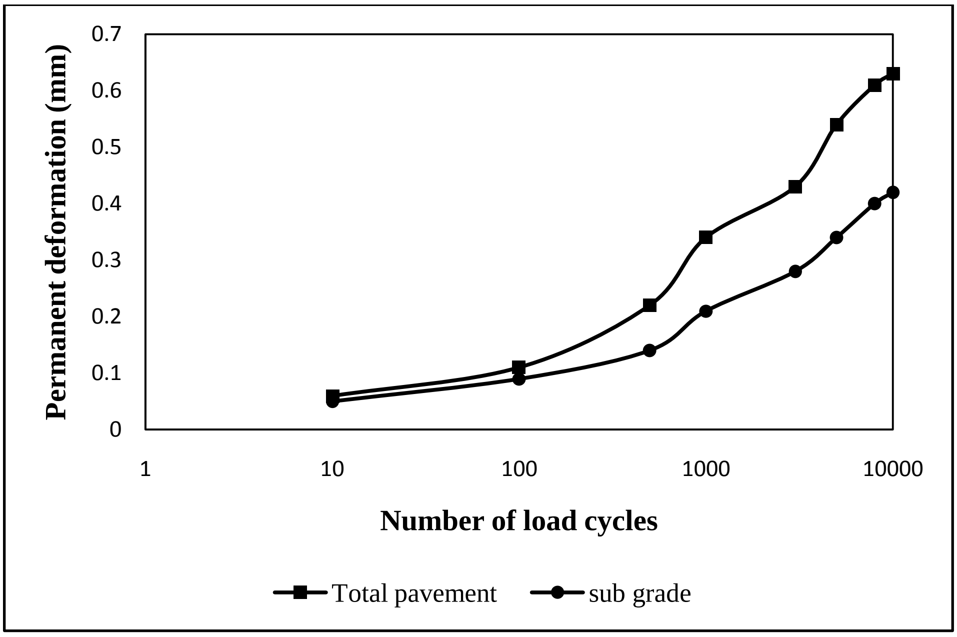

Figure 7.12 Variation of permanent deformation of pavement model test 1

Figure 7.12 represents the variation of permanent deformation with number of load cycles for model pavement 1. From Figure 7.12, 0.63mm permanent deformation was observed for total pavement after 10,000 cycles whereas for subgrade, it is 0.42. Permanent deformation of total pavement was measured using LVDT 1 and subgrade permanent deformation was measured using LVDT 2. In the pavement model test 1, for both the total pavement section and subgrade level, the permanent deformation shows very narrow hysteresis loops, which indicates less energy dissipation. The loops are very narrow and almost closed indicating an elastic shakedown.

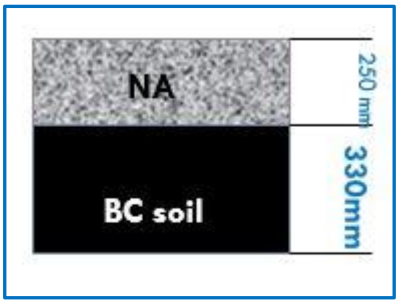

Figure 7.13 Pavement model test 2

Model test 2 consists of Black cotton soil as a subgrade and base layer consisting of natural aggregate where both layers are compacted at maximum dry density and moisture content, which were obtained from modified proctor test. Each model is applied Pressure of 550 kPa

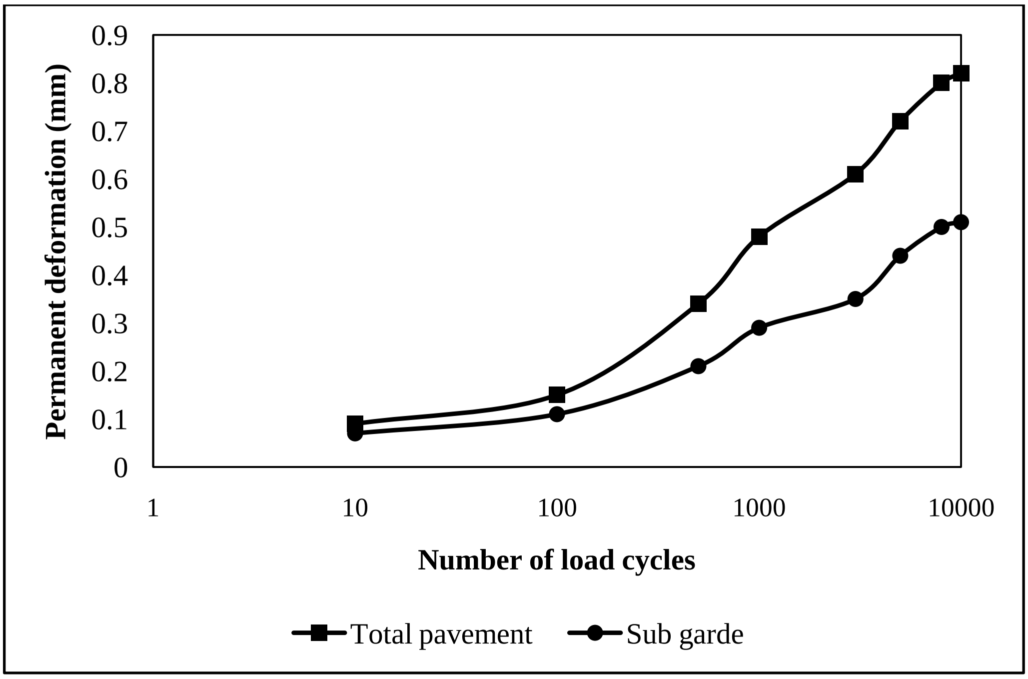

Figure 7.14 Variation of permanent deformation of pavement model test 2

Figure 7.14 depicts the variation of permanent strain with number of load cycles for the pavement model test 2. From figure 7.14, 0.82 mm permanent deformation was recorded for total pavement after 10,000 cycles whereas for subgrade it was 0.51. From the Figures 7.12 and 7.14, nearly 22-30% increment was observed in accumulation of permanent strain and deformations were measured using LVDT. Both subgrade and total pavement initially shows accumulation of permanent strain whereas after that the rate of accumulation of permanent strain becomes negligible. This stage refers elastic shakedown stage according to shakedown theory.

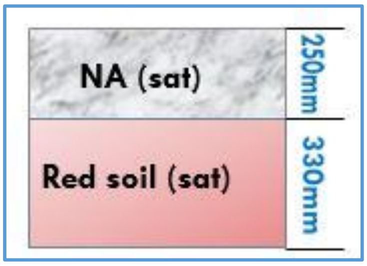

Figure 7.15 Pavement model test 3

Model test 3 consists of Red soil as a subgrade and base layer consisting of natural aggregate where both layers are compacted at maximum dry density which was obtained from modified proctor test and sample was cured up to 100% saturation level.

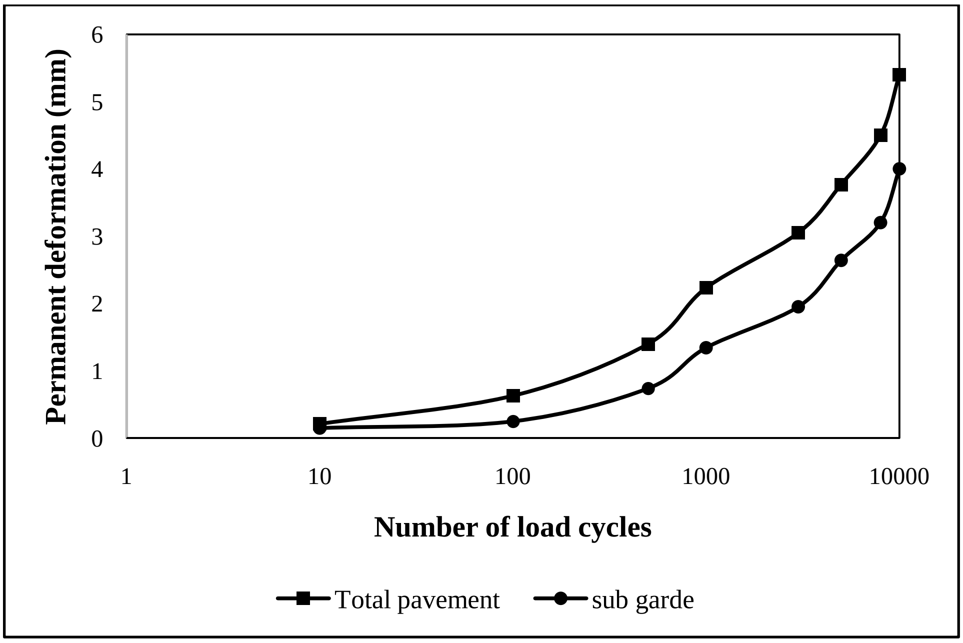

Figure 7.16 Variation of permanent deformation of pavement model test 3

Figure 7.16 exhibits the accumulation of permanent strain with number of load cycles for the pavement model test 3. From the Figure 7.16, it can be noted that 5.4mm deformation was occurred in total pavement after 10,000 load repetitions whereas for subgrade 4.0mm deformation was recorded. From figures 7.12 and 7.16, it can be observed that by enhancing the moisture content from optimum moisture content to saturation level, nearly 6.7 times increment in permanent deformation was observed in total pavement section whereas for subgrade 8.5 times increment in permanent deformation was observed due to the lubricant effect. The accumulation of permanent strain in pavement model shows that it is in plastic shakedown stage according to shake down theory.

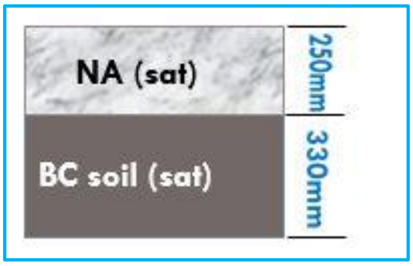

Figure 7.17 Pavement model test 4

Pavement model test 4 consists of Black cotton soil as a subgrade and base layer consisting of natural aggregate where both layers are compacted at maximum dry density which was obtained from modified proctor test and sample cured up to 100% saturation level.

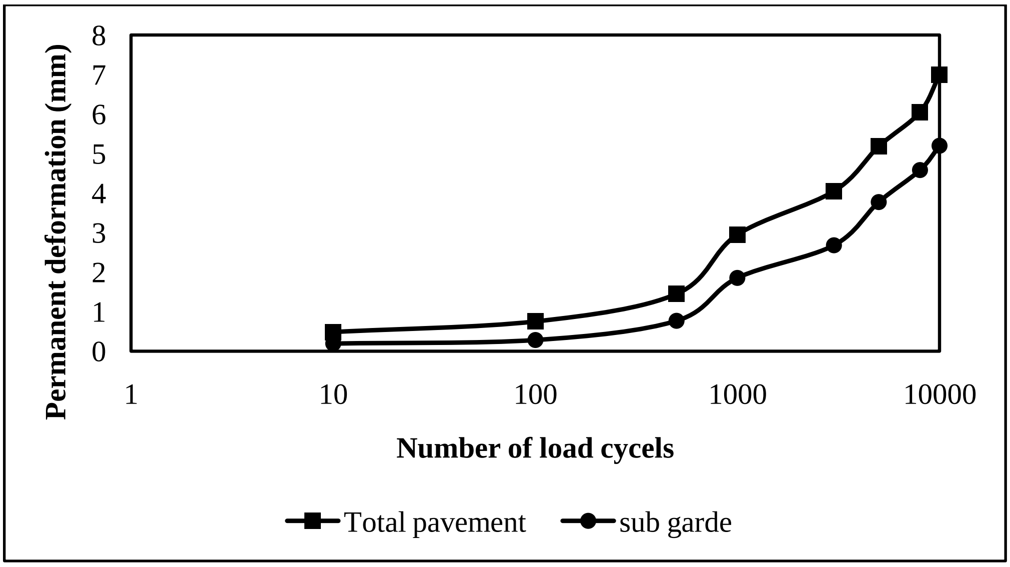

Figure 7.18 Variation of permanent deformation of pavement model test 4

Figure 7.18 depicts the variation of permanent strain with number of repetitions for model pavement 4. From the Figure 7.18, 7mm deformation can be observed after 10,000 load cycles for total pavement section and 5.4mm deformation was recorded for subgrade section. For the pavement model test 4, accumulation of permanent strain is constant in initial cycles whereas after that, the rate of permanent strain is constant throughout 10,000 cycles. This stage refers to plastic creep stage according to shake down theory.

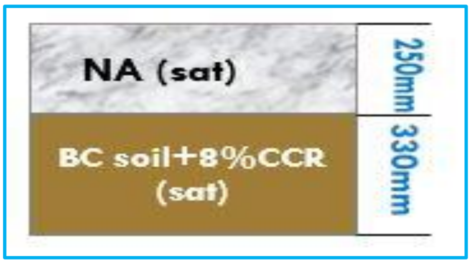

Figure 7.19 Pavement model test 5

Model test 5 consists of CCR stabilized Black cotton soil as a subgrade and base layer consisting of natural aggregate where both layers are compacted at maximum dry density, which was obtained from modified proctor test, and sample cured up to 100% saturation level.

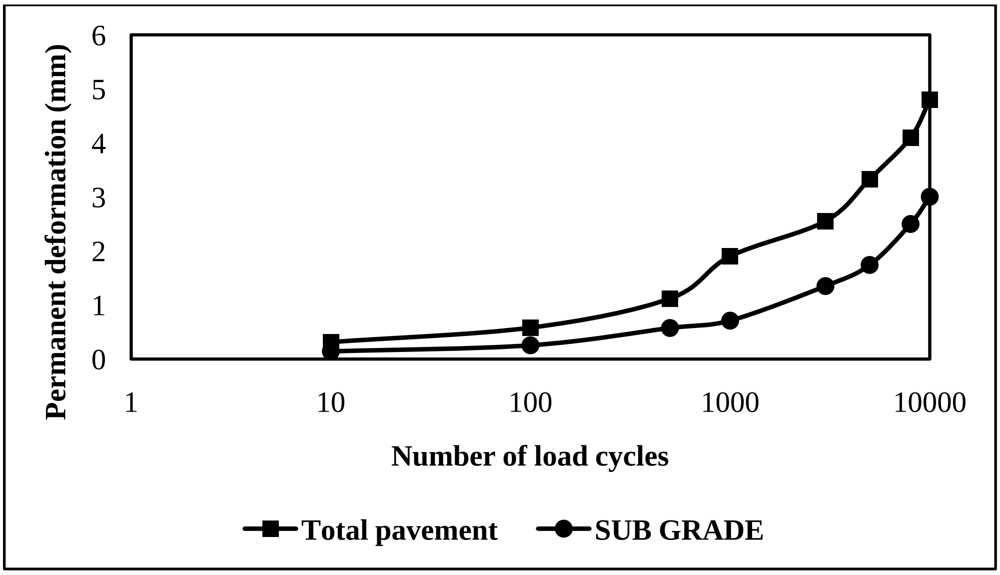

Figure 7.20 Variation of permanent deformation of pavement model test 5

Figure 7.20 represents the variation of permanent strain with number of load cycles for the pavement model 5. From Figure 7.20, it can be observedthat 4.8mm deformation occurred in the total pavement after 10,000 load repetitions whereas for subgrade, 3mm deformation was recorded. From Figures 7.18 and 7.20, it can noted that due to calcium carbide residue stabilization, nearly 75% reduction was observed in accumulation of permanent strain. The accumulation of permanent strain in pavement model shows that it is in plastic shakedown stage according to shake down theory.

7.6 Summary

The experimental program can be divided into two phases. In the first phase, resilient modulus and permanent deformation of natural and recycled aggregated are determined. In the second phase Pavement model tests were carried out on Black cotton soil, Red soil and CCR stabilized Black cotton with and without saturation in order to study the effect of stabilization and water content on permanent deformation behaviour of Black cotton soil

In phase 1, Considering range B as a stable state for this material, the plastic shake down limits were found to be 200 and 360kpa for 20kpa and 45kpa confining pressures respectively and more than 400kpa for 70,100 and 150kpa confining pressures. For recycled aggregates the plastic shake down limits were 200,300,400kpa for confining pressures of 20, 45 and 70 kpa respectively. The Power Law model was found to be a good fit for the data yielding very high coefficient of determination and VTT model was also found to be suitable to predict the permanent deformation except for the stress levels that correspond to the incremental collapse state. Resilient modulus increases with increase confining pressure and decreases with the deviatory stress for both Natural & Recycled aggregates. The universal model and NCHRP model fitted well for this variation.

Permanent deformation of pavement structure, which is having subgrade layer as CH soil, produces 30% more permanent deformation compared to CI soil as subgrade layer. Increase in the moisture content from OMC to saturation, results in nearly seven times increment in permanent deformation increment observed for CH soil, whereas, for Red soil in decreases nearly 5.8 times. Addition of 8% CCR to CH soil causes 45% reduction in plastic strain under saturated condition.

Chapter 8

Conclusions

Present research work is divided into four objectives. First object deals with physical, mechanical, mineralogical and morphological studies on CCR stabilized black cotton soil and red soil. Second object deals with resilient deformation behaviour of clayey soils and CCR stabilized soils. Effect of water content, confining pressure and deviatoric stress levels on resilient modulus of soils are also investigated. Third objective deals with a series of repeated load triaxial tests were carried out on black cotton soil, red soil and CCR stabilized black cotton soil, red soil in order to study the effect of stabilization on permanent deformation behaviour of black cotton soil. The fourth objective can be divided into two phases. In the first phase, resilient modulus and permanent deformation of natural and recycled aggregated are determined. In the second phase Pavement model tests were carried out on black cotton soil, red soil and CCR stabilized black cotton with and without saturation in order to study the effect of stabilization and water content on permanent deformation behaviour of black cotton soil. Following conclusions are drawn from the research.

Conclusions (Objective 1)

This Objective deals with the physical, mechanical, mineralogical and morphological studies on CCR stabilized CH soil and CI soil. The following conclusions can be drawn.

- The results indicated that 4% CCR was found to be optimal for CI soil whereas 8% CCR was required in case of CH soil.

- Stabilization with calcium carbide residue enhanced the unconfined compressive strength and California bearing ratio by 10-14.5 times compared to natural soil for CH soil whereas 7.5-11.8 enhancement was observed for CI soil.

- The BC soil stabilized with both calcium carbide residue obtained 13.5-18 times strength enhancement in CBR tests compared to natural soil.

- The formation of cementing agents such as calcium silicate hydrates and calcium aluminium silicates hydroxides liable for improved strength was evidenced from mineralogical studies using XRD. The formation of pozzolonic compounds and flocculation is evident from the SEM images also.

- The above results affirm that the industrial waste products like calcium carbide residue can be used for stabilizing clayey soil for different geotechnical applications like pavements and foundations.

Conclusions (Objective 2)

The current work examined the effectiveness of calcium carbide residue on enchantment of resilient modulus of two clayey soils.

- The resilient modulus values of samples exhibited 1.9-2.7 times increment with the addition of CCR due to significant increase with the addition of CCR due to presence of calcium in calcium carbide residue leading to flocculation of clay particles there by improving the subgrade characteristics of clayey soils.

- The resilient modulus values were found to increase with increase in confining pressure and reduce with increase in deviatoric stress levels for both the virgin samples and samples treated with CCR.

- The stabilization with CCR was observed to be more effective at higher water contents, while the resilient modulus values of virgin samples markedly reduced in the presence of high moisture content.

- The universal and NCHRP models were found to fit the experimental data for stabilized soil samples very well, using multiple regression analysis with a high coefficient of correlation.

Conclusions (Objective 3)

In the present research work, a series of repeated load triaxial tests were carried out on CH soil, CI soil and CCR stabilized CH soil, CI soil in order to study the effect of stabilization on permanent deformation behaviour of CH soil.

- The elastic shakedown limit of the CH soil sample prepared at its OMC was found to be at the 50% stress level, whereas its plastic creep stage was observed at 60% stress level. For a higher moulding water contents of OMC+2% and OMC+4%, the elastic shake down range was found in between 40% and 50% stress levels and the plastic creep stage commenced at 40% stress level itself.

- The elastic shakedown limit of the CI soil sample prepared at its OMC was found to be at the 30% stress level, whereas its plastic creep stage was observed at 40% stress level. For a higher moulding water contents of OMC+2% and OMC+4%, the elastic shake down range was found to be below 20% stress level and the plastic creep stage commenced at 30% stress level itself.

- It can be observed that with increase in the sub-grade water content, the stress level corresponding to shakedown became low and this lead to rutting under repeated load.

The stabilization was more effective in plastic creep and incremental stage compared to elastic shakedown stage.

- The Power Law model and the VTT model were found to be a good fit for the experimental data for virgin and stabilized soils using multiple regression analysis yielding high coefficient of determination values, except for the stress levels that correspond to the incremental collapse stage.

Conclusions (Objective 4)

The experimental program can be divided into two phases. In the first phase, resilient modulus and permanent deformation of natural and recycled aggregated are determined. In the second phase Pavement model tests were carried out on Black cotton soil, Red soil and CCR stabilized Black cotton with and without saturation in order to study the effect of stabilization and water content on permanent deformation behaviour of Black cotton soil

Phase 1:

- The elastic shake down limit for natural aggregates was found to be 140kpa for 20kpa confining pressure where as it was between 80-140kpa in case of recycled aggregates.
- Considering range B as a stable state for this material, the plastic shake down limits were found to be 200 and 360kpa for 20kpa and 45kpa confining pressures respectively and more than 400kpa for 70,100 and 150kpa confining pressures.
- For recycled aggregates the plastic shake down limits were 200,300,400kpa for confining pressures of 20, 45 and 70 kpa respectively.
- The Power Law model was found to be a good fit for the data yielding very high coefficient of determination and VTT model was also found to be suitable to predict the permanent deformation except for the stress levels that correspond to the incremental collapse state.

- Resilient modulus increases with increase confining pressure and decreases with the deviatory stress for both Natural & Recycled aggregates. The universal model and NCHRP model fitted well for this variation.

Phase 2:

- Permanent deformation of pavement structure which is having subgrade layer as CH soil produces 30% more permanent deformation compared to CI soil as subgrade layer
- Increase in the moisture content from OMC to saturation, results in nearly seven times increment in permanent deformation increment observed for CH soil, whereas, for Red soil in decreases nearly 5.8 times.
- Addition of 8% CCR to CH soil causes 45% reduction in plastic strain under saturated condition.

Recommendations for future study

- The variation of suction in granular materials like the stabilized clayey soil with change in moisture content and its effect on resilient and permanent studies can be studied.
- The regression models for resilient modulus and permanent deformation by including the suction so that they can be implemented in numerical codes like finite elemental analysis and finite difference method.

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List of Publications

Journals:

- ✓ **Venkatesh, N**., Heeralal, M., & Pillai, R. J. (2018). Resilient and permanent deformation behaviour of clayey subgrade soil subjected to repeated load triaxial tests. *European Journal of Environmental and Civil Engineering*, 1-16. **(SCI Indexed)**
- ✓ **Venkatesh, N**., Heeralal, M., & Pillai, R. J. (2018). Permanent deformation behaviour of black cotton soil embedded with calcium carbide residue. *Construction and building materials.*) (*Accepted for publication*). **(SCI Indexed).**
- ✓ **Venkatesh, N**., Heeralal, M., & Pillai, R. J. (2018). Resilient modulus of clayey subgrade soils treated with calcium carbide residue. *International Journal of Geotechnical Engineering*, 1-10. **(SCOUPS Indexed)**
- ✓ **Venkatesh, N**., Heeralal, M., & Pillai, R. J. (2017). Multi-scale laboratory investigation on black cotton soils stabilized with calcium carbide residue and fly ash. *Journal of Engineering Research.*6 (4) **(SCI Indexed)**
- ✓ Heeralal, M **,Venkatesh, N**., Pillai, R. J. & Praveen G V. (2018). A review on permanent deformation of granular material. *Indian Journal of Public Research and development.* 9 (11).**(SCOUPS Indexed).**
- ✓ Heeralal, M **,Venkatesh, N**., Pillai, R. J. & Praveen G V. (2018). A study on engineering properties of black cotton soil mixed with ground granulated blast furnace slag and embedded with polypropylene fibres. *Indian Journal of Public Research and development. 9(11). 2045- 2051(***SCOUPS Indexed).**

National and International Conferences:

- ✓ **N.Venkatesh**, Dr. M.Heeralal, Dr. Rakesh. J.Pillai (2018). " strength and durability characteristic of lime stabilized black cotton soil" Proceedings of the 53 Indian geotechnical conference, Bangalore.

- ✓ **N.Venkatesh**, Dr.M.Heeralal, Dr.Rakesh. J.Pillai (2016). “Influence of Moisture content and stress levels on permanent deformation of subgrade soil” Proceedings of the 51 Indian geotechnical conference, Chennai.

- ✓ **N.Venkatesh**, Dr.M.Heeralal, Dr.Rakesh. J.Pillai (2016) “Effect of stress levels on the resilient modulus of Indian red earth" Proceedings of the International Conference on Advances in Civil Engineering and Sustainable construction (ICACE).

- ✓ **N.Venkatesh**, Dr.M.Heeralal, Dr.Rakesh. J.Pillai (2017). “Stress strain response of subgrade soil under static and dynamic loading”. Proceedings of Research conclave held in NIT Warangal.

- ✓ **N.Venkatesh**, Dr.M.Heeralal, Dr.Rakesh. J. Pillai, Athira Gopinath (2016). Effect of water content and stress level on Resilient modules of subgrade soil. First international conference on recent innovations in engineering and Technology (ICRIEAT-2016) on 22 -23 December-2016.