UTILISATION OF COIR GEOTEXTILES FOR UNPAVED ROADS AND EMBANKMENTS

A THESIS

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Dedicated to

My parents

Krishnan & Kallyani

CERTIFICATE

This is to certify that the thesis entitled **"UTILISATION OF COIR GEOTEXTILES FOR UNPAVED ROADS AND EMBANKMENTS"** is a report of the original work done by Shri. K. K. Babu, under my supervision and guidance in School of Engineering. No part of this thesis has been presented for any other degree from any other institution.

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DECLARATION

I hereby declare that the work presented in the thesis entitled **"UTILISATION OF COIR GEOTEXTILES FOR UNPAVED ROADS AND EMBANKMENTS"** is based on the original work done by me under the supervision and guidance of Dr. K. S. Beena, Reader in Civil Engineering, School of Engineering, Cochin University of Science and Technology, Kochi, 682022. No part of this thesis has been presented for any other degree from any other institution.

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ABSTRACT

KEY WORDS: Bearing capacity, California Bearing Ratio, coir fibre, coir geotextile, embankment, ground improvement, interface friction, soil- fibre composite, unpaved roads, vertical drains.

The increasing tempo of construction activity the world over creates heavy pressure on existing land space. The quest for new and competent site often points to the needs for improving existing sites, which are otherwise deemed unsuitable for adopting conventional foundations. This is accomplished by ground improvement methods, which are employed to improve the quality of soil incompetent in their natural state. Among the construction activities, a well-connected road network is one of the basic infrastructure requirements, which play a vital role for the fast and comfortable movement of inter- regional traffic in countries like India.

One of the innovative ground improvement techniques practised all over the world is the use of geosynthetics, which include geotextiles, geomembranes, geogrids, etc.. They offer the advantages such as space saving, environmental sensitivity, material availability, technical superiority, higher cost savings, less construction time, etc.. Because of its fundamental properties, such as tensile strength, filtering and water permeability, a geotextile inserted between the base material and subgrade can function as reinforcement, a filter medium, a separation layer and as a drainage medium. Though polymeric geotextiles are used in abundant quantities, the use of natural geotextiles (like coir, jute, etc.) has yet to get momentum. This is primarily due to the lack of research work on natural geotextiles for ground improvement, particularly in the areas of unpaved roads. Coir geotextiles are best suited for low cost applications because of its availability at low prices compared to its synthetic counterparts. The proper utilisation of coir geotextiles in various applications

demands large quantities of the product, which in turn can create a boom in the coir industry. The present study aims at exploring the possibilities of utilising coir geotextiles for unpaved roads and embankments.

The properties of coir geotextiles used have been evaluated. The properties studied include mass per unit area, puncture resistance, tensile strength, secant modulus, etc.. The interfacial friction between soils and three types of coir geotextiles used was also evaluated. It was found that though the parameters evaluated for coir geotextiles have low values compared to polymeric geotextiles, the former are sufficient for use in unpaved roads and embankments. The frictional characteristics of coir geotextile - soil interfaces are extremely good and satisfy the condition set by the International Geosynthetic Society for varied applications.

The performance of coir geotextiles reinforced subgrade was studied by conducting California Bearing Ratio (CBR) tests. Studies were made with coir geotextiles placed at different levels and also in multiple layers. The results have shown that the coir geotextile enhances the subgrade strength. A regression analysis was performed and a mathematical model was developed to predict the CBR of the coir geotextile reinforced subgrade soil as a function of the soil properties, coir geotextile properties, and placement depth of reinforcement.

The effects of coir geotextiles on bearing capacity were studied by performing plate load tests in a test tank. This helped to understand the functioning of geotextile as reinforcement in unpaved roads and embankments. The performance of different types of coir geotextiles with respect to the placement depth in dry and saturated conditions was studied. The results revealed that the bearing capacity of coir-reinforced soil is increasing irrespective of the type of coir geotextiles and saturation condition.

The rut behaviour of unreinforced and coir reinforced unpaved road sections were compared by conducting model static load tests in a test tank and also under repetitive loads in a wheel track test facility. The results showed that coir geotextiles could fulfill the functions as reinforcement and as a separator, both under static and repetitive loads. The rut depth was very much reduced while placing coir geotextiles in between subgrade and sub base.

In order to study the use of coir geotextiles in improving the settlement characteristics, two types of prefabricated coir geotextile vertical drains were developed and their time - settlement behaviour were studied. Three different dispositions were tried. It was found that the coir geotextile drains were very effective in reducing consolidation time due to radial drainage. The circular drains in triangular disposition gave maximum beneficial effect.

In long run, the degradation of coir geotextile is expected, which results in a soil – fibre matrix. Hence, studies pertaining to strength and compressibility characteristics of soil – coir fibre composites were conducted. Experiments were done using coir fibres having different aspect ratios and in different proportions. The results revealed that the strength of the soil was increased by 150% to 200% when mixed with 2% of fibre having approximately 12mm length, at all compaction conditions. Also, the coefficient of consolidation increased and compression index decreased with the addition of coir fibre.

Typical design charts were prepared for the design of coir geotextile reinforced unpaved roads. Some illustrative examples are also given. The results demonstrated that a considerable saving in subase / base thickness can be achieved with the use of coir geotextiles, which in turn, would save large quantities of natural aggregates.

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NOMENCLATURE

Symbols

The notations listed below are for general reference. Symbols, which do not appear here, are explained in the text where they first occur.

A Tyre contact area (mm^2)

a Regression coefficients of independent variables

a_0 Regression constant

a_s Fraction of grid surface area (mm^2)

B Width of footing (mm)

C Traffic in terms of coverage

Cc Compression index

Cds Coefficient of direct sliding

Cu Undrained cohesion (N/mm^2)

Cv Coefficient of consolidation (mm^2/s)

c Cohesion intercept(N/mm^2)

D Depth of coir geotextile from surface (mm)

Es Secant Modulus of Coir Geotextile (N/m)

e Void ratio of soil

f Fibre content (%)

H Total height of CBR sample (mm)

h Thickness of aggregate base with geotextile using quasi – static analysis (mm)

h' Aggregate thickness in base course for the reinforced condition (mm)

h_0' Thickness of aggregate layer, without geotextile when traffic is considered (mm)

h_0 Thickness of aggregate base without geotextile using quasi- static analysis (mm)

L Stationary length of geotextile (mm)

L_f Length of coir fibre (mm)

N_i Number of passes of any axle load

N_s Number of passes of standard axle load

$N\gamma$ Bearing capacity factor

n Number of observations

P Axle load (kN)

P_c Tyre inflation pressure (kN/m^2)

P_i Any axle load (kN)

P_s Standard axle load (kN)

p Stress on the soil subgrade with geotextile (N/mm^2)

p_{ec} Wheel load pressure (kN/m^2)

p_g Function of tension in geotextile

p_0 Stress on the soil subgrade without geotextile (N/mm^2)

q Load per unit area for unreinforced soil (N/mm^2)

q_R Load per unit area of reinforced case at any settlement level (N/mm^2)

q_u Ultimate bearing capacity of unreinforced case (N/mm^2)

$q_{u(R)}$ Ultimate bearing capacity of reinforced case (N/mm^2)

R Hydraulic radius (m)

R^2 Coefficient of determination

Rds Maximum shear resistance (N/mm^2)

r Rut depth (mm)

S Settlement under the wheel (mm)

$S\gamma$ Shape factor

Su Ultimate settlement in unreinforced case (mm)

$Su_{(R)}$ Ultimate settlement in reinforced case (mm)

Tv Time factor

x Independent variable

y Height from top of CBR mould where geotextile is placed (mm)

Z Depth of sand bed (mm)

z Depth from top where geotextile is placed for plate load test (mm)

α Dispersion angle with geotextile (degrees)

α_0 Dispersion angle without geotextile (degrees)

α_{ds} Direct Sliding Coefficient (theoretical)

γ Unit weight of soil/ aggregate (kN/m^3)

Δh Reduction in aggregate thickness (mm)

δ Interfacial friction angle (degrees)

ε Strain

κ Dependent variable

σ Normal stress (N/mm^2)

σ_N Effective normal stress (N/mm^2)

σ_S Strip tensile strength of coir geotextile (kN/m)

σ_W Wide width tensile strength of coir geotextile (kN/m)

τ Shear strength of soil (N/mm^2)

Φ Angle of internal friction of soil (degrees)

Abbreviations

ASTM American Society for Testing Materials

BCR Bearing Capacity Ratio

BCRu Bearing Capacity Ratio (ultimate settlement)

BCRs Bearing Capacity Ratio (any settlement)

BIS Bureau of Indian Standards

BS British Standards

CBR California Bearing Ratio

CBR_M Modified CBR

CCRI Central Coir Research Institute

CRRI Central Road Research Institute

CVPD Commercial Vehicles Per Day

ESWL Equivalent Single Wheel Load

FEM Finite Element Method

HDPE High Density Poly Ethylene

IRC Indian Roads Congress

IS Indian Standards

LED Light Emitting Diode

LVDT Linear Variable Displacement Transducer

MDD Maximum Dry Density

MORD Ministry of Rural Development, Government of India

msa Million Standard Axle

NW Non Woven coir geotextile

OMC Optimum Moisture Content

PVC	Poly Vinyl Chloride
PVD	Prefabricated Vertical Drains
RH	Relative Humidity
SPSS	Statistical Package for Social Sciences
UCC	Unconfined Compressive strength
WBM	Water Bound Macadam
WMM	Wet Mix Macadam

CHAPTER 1

INTRODUCTION

1.1 GENERAL

The development of transportation infrastructure is the key to overall development of a country. For countries like India, where resources are limited, the importance of rural / unpaved roads is to be highly emphasised. The subgrade, which is the bottom most layer of the pavement, is made up of compacted soil and so also for the highway and railway embankments. The road alignment is decided based on many factors of which one is the availability of good soil along the proposed alignment. In early days, areas having weak soil deposits were avoided while fixing up the alignment. But with scarcity of land and other resources, we do not have the choice of land and hence roads and embankments have to be built on weak soil deposits. These problematic soils have one or more of the short comings viz., low shear strength, high compressibility, low hydraulic conductivity, swelling and shrinkage, susceptibility to frost action etc., and hence are associated with problems such as low bearing capacity, high settlement, high seepage loss, liquefaction during earthquake and instability of foundation excavation. In such cases, it is often impossible to build a stable base course over soft subgrade, without loosing expensive base material which penetrate into the soft subgrade soil and hence a ground improvement method has to be resorted to.

Ground improvement is a general term used for the modification of soil to enhance the strength and other engineering properties. There are many methods of ground improvement such as using additives (like cement, lime etc.), compaction (both static and dynamic), thermal stabilisation etc.. One of the methods, which got momentum in recent years, is the concept of reinforced soil. Though the principle was not clearly enunciated, people have used techniques of reinforcing earth for centuries. The modern concept of reinforced soil was however coined by Henri Vidal in 1963 (Vidal, 1969). The reinforced earth system proposed by Vidal used metal strips as reinforcements. With the development in the field of polymer technology, a wide variety of geosynthetic materials have come up. Geosynthetics, both natural and polymeric, establish a family of geomaterials, which are used in a wide variety of civil engineering applications.

According to ASTM D4439 (2004) a geosynthetic is defined as "a planar product manufactured from polymeric material, used with soil, rock, earth or other geotechnical related materials, as an integral part of a human made project, structure or system". There are eight types of geosynthetics, namely, geotextiles, geogrids, geonets, geomembranes, geosynthetic clay liners, geopipes, geofoams and geo composites (Koerner, 2005). These products generally have a long life and do not undergo biological degradation, but are liable to create environmental problems in long run.

Geotextiles form one of the largest groups of geosynthetics. Its growth rate in the industry during the past fifteen years has been nothing short of awesome. They are indeed textiles in the traditional sense, consisting mainly of synthetic fibres, though natural fibres are also used for manufacturing. They can be Woven or Non-woven type. There are enormous specific application areas for geotextiles, even though the fabric always performs at least one of the five discrete functions, viz., separation, reinforcement, filtration, drainage and moisture barrier (when impregnated). One of the most popular applications of these materials is in the construction of pavements and embankments on soft soil.

The consumption of polymeric geotextiles in India is insignificant compared to the worldwide consumption. The main factor inhibiting the use of geotextiles on a large scale in India is their high cost (Rao and Balan, 1994). In addition to the low cost of natural fibres, the growing concern over the impact of the use and disposal of synthetic materials has recently led to a renowned interest in the possible advantages of natural geotextiles. In many ground-engineering problems, geosynthetics are mainly required to perform its function in full capacity, only for a limited duration: for example, within temporary haul roads, basal reinforcements for new embankments, vertical drainage to increase shear strength, etc.. In most of the cases, the geosynthetic capacity is surplus to the requirements during the later periods of the working life of such systems. In such situations, the deliberate and designed use of a geosynthetic system, which has a predictable reduction of capacity with time, is a good engineering practice. Natural geotextiles made of coconut fibre, jute fibre, sal, etc. can be used as an alternative to polymeric geosynthetic materials. It is even possible to have tailor made composites of natural fibres to produce a material with required strength - time profile.

Coir geotextiles with Indianised connotation "Coir Bhoovastra", a generic member of the geosynthetic family, is made from coconut fibre extracted from the husks of coconut fruit. Like their polymeric counter parts, coir geotextiles can also be synthesised for specific applications in civil engineering like erosion control, ground improvement, etc. (Rao and Balan, 2000). The use of biodegradable natural materials is gaining popularity in rehabilitation of areas damaged either by natural or industrial causes, especially in the light of growing awareness of sustainable development throughout the world.

1.2 MOTIVATION

India is one of the leading coir producing countries. Coir industry provides employment to people belonging to weaker sections of the society in rural and coastal areas. To protect the traditional coir industry and to make it possible to meet the challenges in structured development of the nation, the development of new products and new horizons of varied applications of the existing products is necessary. At present, coir geotextiles account for only a fractional share of the global market of geotextiles. While the world focus is shifting to natural geotextiles, India as a producer of coir geotextiles, has much to gain by using it for meeting the domestic as well as global demands. The country's limited exposure to engineering projects using geotextiles, limited eco sensibility, priority resource constraints for environmental issues are reasons for coir geotextiles not being consumed for engineering applications in the country. The potential end users, designers and, rather, the decision makers are not fully aware of the product availability and its applicability in different areas. More research works need to be carried out to explore the possibilities of utilising coir fibre based products. Coir geotextile is one among them, which has wide and versatile application in civil engineering and infrastructure development. The proper utilisation of coir geotextiles in various applications demands large quantities of the product, which in turn, can create a boom in the coir industry.

Though huge amount of research work has been done and reported in the area of unpaved roads and embankments using polymeric geotextiles, only very limited work has been reported in the area using natural geotextiles. Even in the area of natural geotextiles, work utilizing coir geotextiles are comparatively less. The majority of works carried out in the field of coir geotextiles are related to erosion control and watershed management. Only a few works have been reported regarding the utilisation of coir geotextiles for roads and embankments and a systematic research work in this area is lacking.

The present work aims at establishing the potentiality of coir geotextiles for the construction of unpaved roads and embankments, by analysing its various engineering properties, including the strength and drainage aspects.

1.3 ORGANISATION OF THE THESIS

In chapter 1, a brief introduction is presented where the state-of- the art technology of using geotextiles is highlighted. The motivation behind the work is also discussed.

Chapter 2 discusses the objectives and scope of the investigations. The significance of the work also forms a part of this chapter.

Chapter 3 presents a comprehensive summary of the literature associated with the present study. Use of geotextiles, both polymeric and natural, for pavements and embankments are reviewed in detail. Analytical and experimental investigations dealing with frictional characteristics of coir geotextiles, strength aspects in terms of California Bearing Ratio, unconfined compressive strength, bearing capacity, drainage aspects and behaviour of soil- fibre composites are given due attention.

In chapter 4, a detailed description of coir geotextiles is given. All aspects of production, properties and application areas of coir geotextiles are discussed.

The property characterisation of the various materials used in the study is discussed in chapter 5. The properties of soils, coir geotextiles and aggregates are also detailed.

Chapter 6 presents the details of the laboratory experimental works conducted to study the interface friction characteristics of coir geotextiles with different subgrade materials used in unpaved road construction. The test set-up developed and fabricated for the present study is well explained. The procedure for experiments conducted is also explained in detail.

The experimental works to obtain the strength parameters, particularly CBR, of the coir-reinforced soil under varied conditions are detailed in chapter 7. A regression model is built to estimate the modified CBR of the subgrade soil, in terms of the strength characteristics of the soil and the coir geotextiles, which is also discussed here.

Chapter 8 presents the study related to the bearing capacity of coir reinforced soil. Details of the experimental set - up, the test procedure and the results in terms of bearing capacity ratio in dry and saturated conditions are discussed.

In chapter 9, the rut behaviour of coir reinforced unpaved road sections are dealt with. Effects of wheel loads were studied by conducting tests on water bound macadam sections with coir geotextiles at subgrade - base interface. The experimental set - ups for both static and repetitive loading conditions along with discussion of the results are given.

Preparation of design charts and design methods of unpaved roads using coir geotextiles are described in chapter 10. Design charts are prepared for the design of unpaved roads for different rut depths. Step by step procedure for three design methods are explained with illustrative design examples.

Effects of coir geotextiles on the drainage aspect in road embankments in the form of vertical drains are discussed in chapter 11. Fabrication and installation of two different types of drains are discussed along with different configurations, their time - settlement behaviour and efficiency.

Chapter 12 contains the details of the experimental work done on soil - coir fibre composites, which forms the part of the subgrade after degradation of coir geotextiles. Effects of fibres on strength and compressibility characteristics of soil are studied and the results are discussed.

Major conclusions drawn from the present investigations, and a mention about the scope of future works are presented in chapter 13. This chapter is followed by references and bibliography.

CHAPTER 2

OBJECTIVES AND SCOPE OF THE INVESTIGATION

2.1 GENERAL

Availability of good subgrade soil is of primary concern in the design and construction of highway projects. Indian Roads Congress (IRC: 37-2001) specifies that the subgrade soil should have a California Bearing Ratio (CBR) of minimum two per cent. Also, where the California Bearing Ratio of the subgrade is less than two per cent, a capping layer of 150mm thickness of material with a minimum CBR of ten per cent shall be provided in addition to the sub base layer. Ground improvement technology has played a very important role in solving many of the major geotechnical problems in highway constructions as well as in other civil engineering fields. The uses of geosynthetic reinforcement as a basal reinforcement in the construction of embankments over soft soils and basal mattresses for the construction of roads over fills and shallow soft deposits have been well established. Over the last two decades, the use of geotextiles has received a tremendous application in the highway construction in many developing and developed nations of the world. The main functions which geotextile serve in highway construction are separation, reinforcement and filtration / drainage. The major disadvantage with polymeric geotextiles is that they are liable to pose environmental problems in the long run.

The availability and low cost of coir fibre make it an eco-friendly material, which suits geotechnical applications. To protect this need based / appropriate technology for rural development, more research work in this area with well-documented laboratory studies are warranted.

A number of published works are available which deal with different types of geosynthetics being used for separation, filtration, reinforcement, etc.. But the data related to natural geotextiles is only very limited. This is particularly true with respect to the application of coir geotextiles used in road construction. Most of the works with coir geotextiles consist of applications in slope protection and erosion control. Hence there is a need for conducting studies to exploit the potential use of coir geotextiles as a highway construction material. The objectives and scope of the present study have been outlined in the following sections.

2.2 OBJECTIVES

The objective of the present study is to explore the possibility of utilising coir geotextiles for the construction of unpaved roads and embankments, after studying the functions and mechanism of coir geotextiles as separators, reinforcement, and for filtration / drainage.

This is achieved by conducting extensive laboratory investigations, which include the following:

- Estimation of the interface friction characteristics of coir geotextiles when placed in subgrade.
- Study of the strength behaviour of coir geotextile reinforced subgrades.
- Study of the effect of coir geotextiles on bearing capacity of soil.
- Analysis of the rut behaviour of coir geotextile reinforced unpaved road sections under static as well as repetitive wheel loads.
- Study of the performance characteristics of coir geotextile vertical drains.

- Evaluation of the strength and compressibility characteristics of coir fibre reinforced soil.

2.3 SCOPE

The present study focuses mainly on the applicability of coir geotextiles in satisfying the different functions of coir geotextiles, with respect to the unpaved roads and embankments. Coir geotextiles are widely used for improving the slope stability and slope protection of embankment. However, this aspect is not considered in the present investigation since sufficient studies have already been reported in this area.

The scope of the study is limited to the following with respect to materials used :

- The study is restricted to the use of three types of coir geotextiles designated as H2M6, H2M8 (both Woven type) and AGL C/201 (Non - woven type) procured from M/s Aspinwall Pvt. Ltd, Alappuzha, Kerala.

- Four types of soil (red earth from Palakkad and Kochi, clayey silt from Kochi and river sand from Pattambi), granite aggregates and screenings from a local quarry in Palakkad were used for the experiments.

CHAPTER 3

REVIEW OF LITERATURE

3.1 INTRODUCTION

The advent of geosynthetics in the recent past has brought in new dimensions to the solutions of various difficult problems in the area of geotechnical engineering. The majority of geosynthetics, covering a wide variety of woven and non - woven geotextiles, geogrids, geonets, geomembranes, geopipes, geofoams, geocomposites and geosynthetic clay liners are mainly polymeric, although natural materials can also be used. The polymeric materials used in the manufacture of geosynthetic are polypropylene, polyester, polyethylene and polyamide. They are manufactured from the byproducts of petroleum, a raw material that might become scarce with the passage of time. These products generally have long life and do not undergo biological degradation, but are liable to create environmental problems in the long run. However, ecological and environmental engineering considerations have imposed restrictions on the extensive use of geosynthetics. Coir, Bamboo and Jute are some of the natural materials exploited as an alternative to polymeric geosynthetic materials.

Kaniraj and Rao (1994) reviewed the trends in the use of geotextiles and the related products in civil engineering applications in India. The potential for growth in the use of geotextiles in India has been highlighted by discussing five themes viz., manufacture, properties and testing equipment, specifications, research and applications, and the use of geotextiles made of natural fibres. The development and potential of jute geotextiles has been detailed by Ranganathan (1994).

In most of the cases, geotextiles, be it polymeric or natural, their function as separators, as reinforcement and for drainage / filtration remains the same for a particular project. These functions are examined in this work, in relation to unpaved roads and embankments, while exploring the possibility of utilizing coir geotextiles. The literature pertaining to the present work is reviewed in this chapter.

3.2 INTERFACIAL FRICTION

Geotextiles can fulfill its functions (reinforcement, separation, filtration etc.), only if the force in the soil is transferred to the material and vice versa. This is achieved by the interfacial friction acting in the plane of the material. This transfer of load is a prerequisite for satisfactory functioning.

In addition to laboratory testing, finite element studies such as the ones performed by Burd and Brocklehurst (1990) and subsequently Burd and Brocklehurst (1992) have emphasised the importance of the friction shear characteristics in soil / geosynthetic friction tests. Their work highlighted the importance of these parameters in overall behaviour of reinforced unpaved road.

Experimental studies on friction of granular material along an interface have been reported by Paikowsky and Player (1994). The test results showed that the grain shape and surface roughness, rather than the grain size, are the primary parameters controlling the interfacial shear at a given stress level. The use of a modified direct shear box for the evaluation of interfacial friction seems to be influenced by the boundary condition resulting in interface friction angles exceeding those that would develop along unrestricted interface.

A critical examination of the past studies on the interfacial friction in terms of data generation techniques used and the conclusions drawn, have been made by Subba Rao et al. (1996). Two types of situation exist in practice: the structure is placed on the free surface of a prepared fill (type A) and the fill is placed against the material surface (type B). The friction angle depends on the surface roughness of construction material. It was pointed out that in type A situation, it is independent of density of fill and its limiting maximum value is the critical state friction angle, whereas in type B situation, it is dependent on density of fill and its limiting value is the peak angle of internal friction of the fill. Subba Rao et al. (1998) again reported that type B apparatus has the advantage of yielding friction angle values applicable to both type A and type B situation.

A comparison of displacement-induced mobilisation of frictional resistance between different types of reinforcements on coarse and fine sand has been reported by Jayadeep (1997). The results indicated that mobilised friction parameters of reinforced soil system are lower than those of soil itself and are affected by surface texture, thickness and type of fabrics.

It is reported that the ratio of interfacial friction angle to the angle of internal friction of the soil is independent of over consolidation ratio and this ratio increases with roughness (Subba Rao et al., 2000). The ratio of interface friction angle to the angle of internal friction is unity for very rough surfaces.

Studies on the geotextile - soil interface shear behaviour carried out by Mahmood et al. (2000) concluded that the shear strength of organic clay - geotextile interfaces were increasing with the increase of geotextile tensile strength. Similar findings were obtained by Burd (1995) while performing FEM analysis on the frictional behaviour of soil - geotextile interface.

Different types of apparatus used for the interfacial friction measurement reported in the literature have been reviewed in detail by Subba Rao et al. (2001). The factors affecting interfacial friction were also reviewed.

Estimates of interfacial friction angle, coefficient of shear stress interaction and coefficient of direct sliding are necessary for the design of retaining structures and deep foundations and in many geotechnical problems involve estimation of stresses transferred along the interface between soils and structures.(Koerner, 2005)

Interface friction depends on many parameters such as pressure, grain size, shape, surface roughness of geotextile and presence of water. The amount of friction mobilised between the soil and reinforcement has a significant influence on the internal and external stability of mechanically stabilised earth structures (Tateyana, 1999).

3.3 GEOTEXTILES FOR UNPAVED ROADS

Providing good road network is very essential for the development of any country. The rural roads are the basic infrastructure required for the development of rural areas. A road continuously deteriorates under the combined action of traffic loading and the environment. The ability of the road to satisfy the demands of traffic and the environment over its design life is known as its performance. Various researchers studied the use of geosynthetics in improving the performance of roads in different angles. In general geosynthetics perform at least one or more of the functions viz., separation, reinforcement and filtration / drainage in improving the road performance.

The benefit of using geosynthetics in flexible pavement depends largely on the quality and thickness of the granular base and location of the geosynthetics within the pavement structure (Chan et al., 1989).

3.3.1 Reinforcement Mechanisms

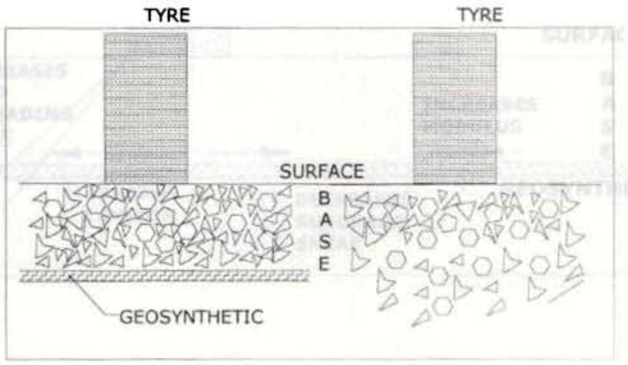
Three fundamental reinforcement mechanisms have been identified involving the use of geotextiles in unpaved road applications. These are: a) separation, b) lateral restraint, and c) tensioned membrane effect (Perkins and Ismeik, 1997).

3.3.1.1 Separation

One of the reasons for distress or failure of roads is migration or mixing of fines from the subgrade to overlying granular layer. High wheel load stresses acting on the road surface combined with a weak/saturated subgrade, typically cause a base and subgrade mix. This mixing causes a reduction in the effective base thickness by reducing the actual modulus of the granular base as well as its physical thickness. Mixing is best pictured as granular material is pushed down into the soft subgrade and/or soft subgrade is pumped up into the overlying granular layer (Tensar Design Manual, 1998). Fig. 3.1 shows the schematic diagram that represents both the mixed and separated situations.

3.3.1.2 Lateral restraint

Lateral restraint reduces the horizontal deformation of the base course and the subgrade, when these are in contact with the geosynthetic. It has been reported that geosynthetics hold the base material and the subgrade together by developing friction forces between it and the other two materials. This action of the geosynthetic and the base material is referred to as base restraint and that between the geosynthetic and



Separated (unmixed) Mixed

Fig. 3.1. Schematic representation of separation function

subgrade as subgrade restraint. This is composed of four related mechanisms that combine to provide better overall pavement performance. Initially, a shear force is generated at the base of the granular layer, as the material would like to move down and out from wheel loads on the surface. This shear stress is absorbed by the geosynthetic, thus reducing the lateral strain in the upper granular layer. Simultaneously, this induces a slightly more lateral stress in the lower portion of the granular layer, thus leading to higher elastic modulus for the granular layer due to the slight increase in the confining stress. Therefore, the granular layer with a greater modulus spreads the surface load over a wider area, thus decreasing the intensity of vertical stresses, and vertical strains in layers above and below the geosynthetic. Finally, shear stress being absorbed by the geosynthetic transfers only less intense shear stress to the subgrade (Tensar Design Manual, 1998). Illustration of this is shown in Fig.3.2.

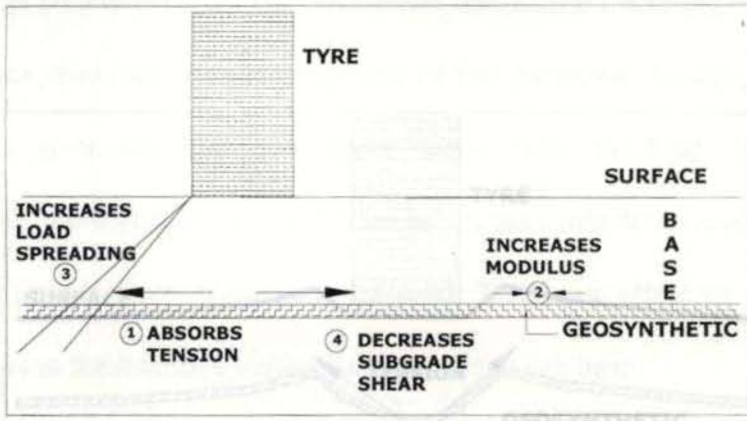


Fig.3.2. Lateral restraint function showing four mechanisms of improvement

3.3.1.3 Tensioned membrane effect

Membrane reinforcing is mobilised when the subgrade deforms. As the subgrade deforms under loading, the geosynthetic material stretches like a membrane. The loading is distributed over a wider area as a result of the vertical component of the tension, which develops in the material. This was first described by Giroud et al. (1981) and is applicable to cases where rut depth is more like in unpaved roads. Fig.3.3 schematically shows the mechanisms. The tension in a highly distorted membrane at the base of an overlying granular layer provides a reaction with a vertical component that contributes to support the wheel load at the surface and confines the soft subgrade below.

3.3.2 Laboratory and Field Studies

From the plate load tests conducted on highway pavement, Mc Leod (1956) developed a method for the design of unpaved roads based on deflection criteria. He considered 30 cm diameter standard plate and a deflection of 0.5cm at ten repetitions as standard for the preparation of design curves.

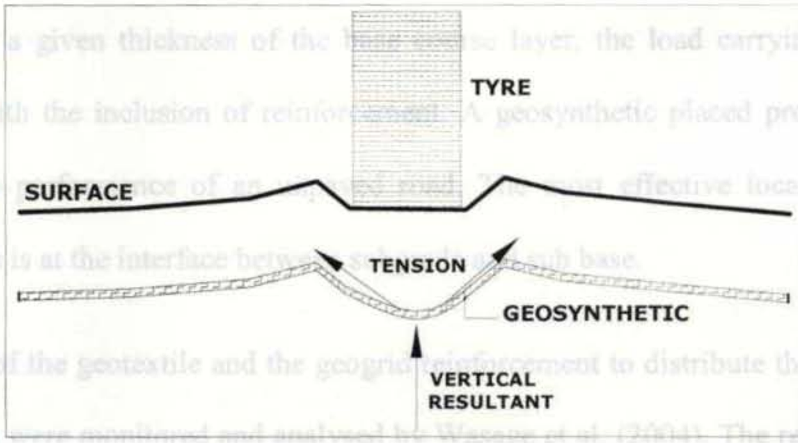


Fig.3.3. Tensioned membrane function showing displacement and resultant

Milligan et al. (1989a, 1989b) developed a method for the design of reinforced unpaved roads where, only small rutting is permitted. Their work was based on the concept that the principal function of the geotextile reinforcement is to carry shear stresses that would otherwise be applied to soft subgrade.

A full-scale field test was carried out by Sigurdsson and Fannin (1997) to investigate the performance of different types of geosynthetics in unpaved road construction over soft ground. Experiments were carried out on control sections, sections with geotextiles and sections with geogrid. It was reported that greatest improvement in the reinforced sections occurs on the thinnest base course layers. Separation of the subgrade and base course layer appears to be important for thinner layers whereas the reinforcement of the base course layer is important in thicker layers.

A series of small-scale plane strain model tests were done by Jayaganesh (2002) to study the load deformation characteristics of a two layer base course and subgrade system. It was concluded from the test results that as the thickness of the aggregate layer increases, the load carrying capacity of the pavement system also increases. Further, for a given thickness of the base course layer, the load carrying capacity increased with the inclusion of reinforcement. A geosynthetic placed properly does improve the performance of an unpaved road. The most effective location of the geosynthetic is at the interface between subgrade and sub base.

The ability of the geotextile and the geogrid reinforcement to distribute the load over a wider area were monitored and analysed by Wasage et al. (2004). The results of the reinforced specimens were compared with unreinforced specimens to assess the advantages of using geosynthetics in increasing the rut resistance of flexible pavement. The test results obtained from this experimental research programme demonstrated that a geogrid reinforcement placed at the surface course / base course interface effectively increased the rutting resistance of flexible pavement, hence would be beneficial in increasing the service life of the flexible pavement. The results also showed that the base course aggregates were found to intrude into the subgrade layer causing stone losses in the specimens if geotextiles were not placed at the subgrade- sub base interface.

Koerner (2005) proposed a laboratory method for modeling the field situation to arrive at the influence of the geotextile. The method involves conducting CBR test on specimen of soil and on soil-aggregate system. Reinforcement ratios were defined as the load resisted by the unreinforced and the reinforced specimens. Maximum reinforcement ratio multiplied by the actual CBR was termed as modified CBR. US Army Corps of Engineers used this modified CBR to find the thickness of flexible pavement

Edil (2006) conducted large-scale experiments on working platforms of crushed rock overlying simulated soft subgrade. Tests were conducted with and without geosynthetic reinforcement to evaluate the deflection characteristics of the working platform. Four types of geosynthetics were tried and it was found that the reinforced working platforms deformed at a slower rate, and in most of the cases this deformation ceased after nearly 200 loading cycles.

3.3.3 Theoretical Studies

Though many design methodologies have emerged which address geotextiles in pavement, two theories based on original work contributed greatly to a better understanding of geosynthetic applications for pavements. These methods were suggested by Barenburg et al. (1975) and Giroud and Noiray (1981).

3.3.3.1 Barenburg method

Barenburg et al. (1975) presented a method, which make use of different bearing capacity factors for the unpaved roads application with and without geotextiles. Lateral restraint action of the geotextiles is the backbone of this theory. Soft cohesive soils were used as subgrade, and load repetitions less than 100 were assumed. The work based on small-scale laboratory test showed that the bearing capacity factors (Nc) of 6.0 and 3.3 were appropriate for loading with and without the inclusion of geotextile respectively.

3.3.3.2 Giroud and Noiray method

Giroud and Noiray (1981) presented the first method that utilises geotextiles' modulus and tensile strength. They considered undrained soft saturated clay subgrade. Giroud and Noiray initially followed the lateral restraint theory by Barenburg (1975), but added the tensioned membrane concept later. Details of this method are given in chapter 10.

3.3.3.3. Other design methods

Sellmeijer et al. (1982) developed an analytical model for the design of reinforced unpaved roads. They assumed, the same pyramidal distribution of load as was assumed by Giroud and Noiray (1981) but followed the principle of limit equilibrium to derive their analytical equation.

Haliburton and Baren (1983) developed a pavement design for unsurfaced roads using geosynthetic reinforcements. They observed that thc optimum depth of aggregate placed on geotextile significantly increases the strength and deformation resistance of aggregate cover.

Holtz and Sivakungan (1987) developed design charts to determine the aggregate thickness for geotextile-reinforced roads using the Giroud and Noiray (1981) approach. They prepared charts for rut depths of 75mm, 100mm, 150mm, 200mm and 300 mm for an axle load of 80kN and with tyre pressures of 480MPa and 620MPa.

A new method for the design of reinforced unpaved roads without considering tension membrane effect has been proposed by Houlsby and Jewell (1990). The role of shear stress on reinforcement is given primary importance. Design charts were presented which give the necessary depth of granular fill and the required reinforcement tension.

Dixit and Mandal (1993) applied variational method to determine the bearing pressure of geosynthetic soil. The shape of the failure surface and distribution of normal stresses were obtained using minimizing theorems of variational calculus.

A new analytical design model has been proposed by Burd (1995). This new model is based on a membrane reinforcement mechanism and is appropriate for cases of large surface deformations as is the case of unpaved roads.

A theoretical design method to find the thickness of the base course of unpaved roads has been developed by Giroud and Han (2004a). This method considers distribution of stress, strength of base course materials, interlock between geosynthetic and base course material, and geosynthetic stiffness in addition to the conditions stipulated in the earlier study (Giroud and Noiray, 1981) viz., traffic volume, wheel loads, tyre pressure, subgrade strength, rut depth and influence of the presence of a reinforcing geosynthetic on the failure mode of the unpaved road. The advantage of this method is that the thickness can be obtained by using a unique equation. The equation was developed for geogrid reinforced unpaved roads, but can be used for roads reinforced with geotextiles, with appropriate modifications for the relevant parameters. The calibration of this design method has been done (Giroud and Han, 2004b) using data from field wheel load tests and laboratory cyclic plate loading tests on unreinforced and reinforced base courses.

3.4 GEOTEXTILES FOR FOUNDATIONS AND EMBANKMENTS

In many places the formation of embankment and its foundation beneath becomes an integral part of the construction of unpaved roads especially through swampy areas. Hence the role of geotextile in these aspects should be considered. The previous investigations conducted in these lines are reviewed in this section. Mainly two aspects of ground improvement are considered in this context: one, the bearing capacity and the other, the stabilisation by accelerating the drainage using vertical drains.

3.4.1 Bearing Capacity

Binquet and Lee (1975a, 1975b) proposed an analytical approach for the design of footing on sand deposits. Mathematical models for bearing capacity were developed by Hopkins (1991) and Slepak and Hopkins (1993 and 1995a, 1995b). Analytical research works have also been reported by Beena (1993), Yetimuglu et al. (1994), Otani et al. (1998), and Kumar and Saran (2003a and 2003b).

Using the laboratory model tests, Guido et al. (1986) has compared geogrid and geotextile reinforced earth slabs and it was observed that bearing capacity was increased directly with increase in tensile strength of the geotextile.

The applicability of predicting the bearing capacity increase in reinforced sandy ground was examined by Haung and Tatsuoka (1990) using tests performed under various conditions.

The study conducted by Mandal and Sah (1992) pertains to the efficiency of horizontal geogrid reinforcement in improving clay subgrades. The maximum percentage reduction in settlement with the use of geogrid reinforcement below compacted saturated clay was reported as 45% and it occurs at a distance of one fourth of the width of foundation from its base.

Khing et al. (1993) studied the variation of bearing capacity ratio with respect to ultimate bearing capacity at levels of limited settlements of foundations by conducting laboratory model tests on strip foundation supported by sand layer reinforced with layers of geogrids. They found that reinforcement founded at a depth greater than 2.25 times the width of foundation did not contribute to any increase in bearing capacity.

Also, the bearing capacity ratio calculated on the basis of limited settlement appears to be about 67% – 70% of the ultimate bearing capacity ratio.

Based on experimental tests Omar et al. (1993) showed that for the development of maximum bearing capacity, the effective depth of reinforcement is about twice the foundation width for strip footing and about 1.4 times the footing width in the case of square foundation. It was also showed that the mean depth of the placement of first layer of geogrid should be less than approximately the foundation width to take advantage of reinforcement.

In another laboratory study, Omar et al. (1993) postulated that, for certain type of geogrid, and for a given sand at a given density of compaction, the critical depth of reinforcement for mobilisation of maximum possible ultimate bearing capacity ratio decreases with the width to length ratio of the foundation.

Potential benefits of geosynthetic reinforced soil foundation have been investigated using large-scale model footing tests by Adams and Collin (1997). It was observed that maximum improvement in bearing capacity at low strains occurs at a depth of one-fourth width of the footing from the bottom of footing.

Shin and Das (2000) conducted reduced scale model tests on strip foundation supported by medium dense sand reinforced by multiple layers of geogrid. The study revealed that for a given thickness of reinforcement zone, the bearing capacity ratio increases when the foundation is placed at a depth below ground level. Critical reinforcement ratio is approximately two for multiple layers of geogrid reinforcement.

Gabr and Hart (2000) used data from plate load test to determine the elastic modulus of sand reinforced bed. Comparison between the performance of reinforced and unreinforced cases has also been made. The study focused mainly on estimating the elastic modulus, rather than the bearing capacity of the soil.

A new method of ground improvement by using semi flexible vertical reinforcing element was studied by Puri et al. (2005). It was found that, for significant improvement in the ultimate bearing capacity of loose and medium dense sand, spacing of vertical reinforcement used should be about 0.15 to 0.2 times the width of the footing. Length of the reinforcement should be at least equal to the width of the footing and preferably 1.5 times the footing width.

3.4.2 Geotextiles for Drains

The rapid increase in population and associated developmental activities has resulted in the scarcity of good sites for construction. The activities are now forced to concentrate in low land areas having very low strength and high compressibility. Thus it is very essential to modify the soft soil before commencing construction activities, to prevent undue settlement and in turn, the failures - both functional and structural. Ground improvement by drainage has been found to be the most economical for soft deposits, but this approach still requires considerable time for preloading and subsequent consolidation.

In many cases to pace with the speed of construction activities, preloading, which is a well-established method of ground improvement, may not be always a viable solution. In such cases, the presence of vertical drain can greatly reduce the preloading period. Installation of vertical drains results in the reduction of the length of drainage path in radial direction. Since the consolidation time is inversely proportional to the square of the length of drainage path, the period of loading required to achieve stabilisation in soft soil is considerably reduced.

The function of Prefabricated Vertical Drains (PVD) is to allow drainage to take place in the horizontal (radial) direction over a much shorter drainage path so that the rate of consolidation can be accelerated and thereby the time for consolidation can be greatly reduced. However, when PVDs are installed, there is a chance that the soil surrounding the PVDs is disturbed after the installation of PVDs. As a result, the permeability of the soil in the disturbed zone called smear zone is reduced (Hausmann, 1990).

The first vertical drain, developed by Kjellmann (1948) made of corrugated card board covered by a paper sheath measuring 100mm x 3mm, was found to be convenient and more effective than sand drains, and gradually replaced them. Subsequent developments in materials and manufacturing processes led to the evolution of synthetic drains in 1970's made of polymeric materials. PVDs gained popularity due to their ease in storage and transport, rapid installation, lighter installing equipment, high discharge capacity and non clogging potential (Hausmann, 1990; Holtz et. al.,1991)

Holtz et al. (1991) reported that the discharge capacity of prefabricated vertical drains could vary from 100 to 800m^3/year. The discharge capacity of PVD is a function of its filter permeability, core volume or cross sectional area, lateral confining pressure, and drain stiffness controlling its deformation characteristics. (Hansbo, 1979 and Holtz et al.,1991). For long vertical drains that are vulnerable to well resistance, Hansbo (1981) pointed out that in the field, the actual reduction of the discharge capacity can be attributed to: (a) reduced flow in drain core due to increased lateral earth pressure, (b) folding and crimping of drain due to excessive settlements, and (c) infiltration of fine silt or clay particles through filter.

A numerical method was developed to predict the consolidation behaviour of very soft soil with horizontal drains under the action of the gravity pressure by Jang et al. (1999). They compared the numerical analysis for the soil with horizontal drains with the case of self-weight consolidation. The influence of design factors, such as lateral spacing and the depth of installation of drains, on consolidation process were studied. As a result of the analysis, it was reported that the time to reach 95% degree of consolidation using horizontal drains takes only nine times less than that of self-weight consolidation. The settlement of clay with drains occurred more than three times compared with the case without drains. Hawlader et al. (2002) conducted a parametric study to identify the influence of viscosity and smear on the consolidation process of clays with vertical drain.

Chai et al. (2004) studied long term drainage capacities of four types of PVDs and three types of prefabricated horizontal drains. It was found that the rate of water flow per unit drainage area increases with hydraulic radius of the drainage channel approximately with $R^{1.2}$ and $R^{1.7}$ for elapsed time of 1 and 3 months respectively. For all geosynthetic drains, the drainage capacity ratio was increased almost linearly with the hydraulic radius.

Chew et al. (2004) has conducted some laboratory tests and field trials with electric vertical drains. The field trial has shown that the soft clay, beneath the sand fill, can be reached effectively by PVD.

More than 140 million meters of prefabricated vertical drains have been installed for the soil improvement work for the Changi East Land Reclamation Project in Singapore. Chu et al. (2004), described the factors controlling the selection of PVDs, quality control test for them, selection of design parameters, adaptability of some of the design criteria, smear effect etc., based on experience of Changi East Land Reclamation and other soil improvement projects in Singapore. It was concluded that, the success of a soil improvement project using prefabricated vertical drains depends not only on the design calculations, but also on controlling the quality of the drain, the selection of soil parameters and the method of installation.

Studies carried out by Lorenzo et al. (2004) on embankments at different locations of the site of the second Bangkok International Airport and at the campus of Asian Institute of Technology, Thailand, confirmed that PVD installation not only accelerated and facilitated uniform consolidation, but also aided in recharging the subsoil.

Consolidation process of geotextile tube filled with fine-grained materials has been reported by Shin and Oh (2004). They examined several issues associated with the prediction of the tube shape and mechanical properties of the filling materials inside the tube during and after the consolidation process. The variations of mechanical properties of internal filled material during the consolidation process are also reported.

Analysis of field performance of embankments constructed on clay deposits with and without PVDs, conducted by Shen et al. (2005), showed that PVDs increased bulk vertical hydraulic conductivity of soft subsoil by about 30 times. Discharge capacity is an important parameter that controls the performance of prefabricated vertical drains. Only PVDs with sufficient discharge capacity can function well. Bo (2004) developed a single basic equation to obtain the required average discharge capacity and found that the required discharge capacity was in the order of 10^{-6} m/s for 100 mm width drain.

3.5 SUMMARY

In the preceding sections, application of geosynthetics in the context of unpaved roads and embankments were discussed by reviewing the investigations conducted by different researchers in this field. The increasing application of geosynthetics, however, has the disadvantage of having high cost and of being petroleum based. They are not eco friendly also (Lee et al., 1994). With the increasing environmental awareness and sustainability, along with the high cost of petroleum products, the developing countries have lead to investigations of substitutes using natural products.

The utilisation of coir geotextiles in unpaved roads and embankments, being the topic of research, additional importance is given to review the works on coir geotextiles right from its manufacturing. Hence a separate chapter, "Coir geotextiles", which follows, is provided to review its production, properties and its general application in civil engineering projects.

CHAPTER 4

COIR GEOTEXTILES

4.1 INTRODUCTION

Coir is derived from the exocarp of the fruit of the coconut tree "*Cocos nucifera Lynn*" grown in the tropical countries mainly for the high oil content of the endosperm (*copra*). Large production areas, in particular, can be found along the coastal regions in the wet tropical areas of Asia, in the Philippines, Indonesia, India, Sri Lanka, and Malaysia. Total world production of coconut increased substantially from nearly 35 million tons in 1980 to more than 50 million tons today. Yield varies from region to region with an average of 70 to 100 nuts and a maximum of 150 nuts per tree per year. The kernel (copra, coconut water and shell) comprises 65% of the total weight, while the husk contributes only 35%. Despite their low trade value, the fibres provide significant economic support to populations especially to weaker sections in specific areas of the coir producing countries, for example in southern states of India viz., Kerala, Karnataka, Tamilnadu, Andhrapradesh and also in the west and south of Sri Lanka.

Coir being a biodegradable and environment friendly material is virtually irreplaceable by any of the modern polymeric substitutes. With the diversification of the products and evolvement of new technologies for the production of fibres, the export of coir products has been increased tremendously. Though the demand for coir geotextiles is increasing, the total coir exports from India comprises only less than 3% of it. The close involvement of the local governments, with the support of the public research institutions and private enterprises is required for innovation, manufacturing and marketing of coir.

4.2 COIR FIBRE

Coir fibres are extracted from the husks surrounding the coconut . There are two distinct varieties of coir fibre based on the extraction process viz., white coir and brown coir. The average fibre yield depends on geographical area and the variety of coconut tree. In southern states of India and in Sri Lanka, where the best quality fibres are produced, the average yield is 80 to 90 grams per husk. Husks are composed of 70% of pith and 30% of fibre on a dry weight basis. The maximum total world production of coir fibre is estimated to be between 5 and 6 million tons per year (Dam, 1999).

4.2.1 Composition of Coir Fibre

Cellulose fibres are obtained from fruit (e.g. coir), seed (e.g. cotton), stem (e.g. sisal), leaf (e.g. banana and pineapple) and so on. Coir – the 'golden fibre', is a 100 per cent organic fibre. Coir is a strong cellulose fibre with high lignin content. It is a multi - cellular fibre containing 30-300 or more cells in its cross section, which is polygonal and round in shape. Each cell is made of concentric layers consisting primary wall, outer secondary wall, middle secondary wall and inner secondary wall. In between the primary cell walls is situated the intercellular cementing non-crystalline material comprising of lignin, pectines and hemi cellulose which holds the cells together. (Kulkarni et al. 1983).

Coir fibre is hard and tough and its length ranges from 150mm to 280mm and the diameter from 0.1mm to 0.5mm. It is one of the hardest natural fibres because of its high lignin content (CSIR, 1960). The chemical composition of coir is given in Table 4.1.

Table 4.1 Chemical composition of coir*

Content	Percentage
Lignin	45.84
Cellulose	43.44
Hemi cellulose	0.25
Pectin and related compounds	3.00
Ash	2.22
Water soluble	5.25

(* Sarma, 1997)

4.2.2 Properties of Coir Fibre

A fibre material would be suitable for geotextile production when it has reasonably good mechanical properties and resistant to microbial attack. Coir fibres are of different types and are classified according to varying degree of colour, length and thickness. The decomposition of coir fibre is generally known to be much less than that of jute due to high lignin content. The engineering properties of coir fibre are given in Table 4.2.

4.2.3 Fibre Production

The traditional production of fibres from husk is a laborious and time-consuming process. After manual separation of the nut from the husk, the husks are processed by various retting techniques. This is generally done in ponds of brackish waters for three to six months or in salt backwaters or lagoons for 10 to 12 months. By retting the fibres are softened and can be decorticated and extracted by beating, which is usually done by hand. After hacking, washing and drying in shade the fibres are loosened manually and cleaned. Traditional practices of this kind yield the highest quality of white fibre for spinning and weaving

Table 4.2 Engineering properties of coir fibre *

Property	Value
Length (mm)	15 - 280
Density (g/cc)	1.15 - 1.4
Tenacity (g/tex)	10.0
Breaking elongation (%)	30.0
Diameter (mm)	0.1 –1.5
Rigidity modulus (dynes/cm^2)	1.8924
Swelling in water (diameter)-(%)	5.0
Moisture at 65% RH (%)	10.5
Specific gravity	1.15
Young's modulus (GN/m^2)	4.5
Specific heat	0.27

(* Ayyar et al., 2002.)

Alternatively, mechanical process using either defibering or decorticating equipment needs only five days of immersion in water to process the husks. Crushing the husks in a breaker opens the fibres. By using revolving drums the coarse long fibres are separated from the short woody parts and the pith. The stronger fibres are washed, cleaned, dried, hackled and combed. The quality of the fibre is greatly affected by these processes.

New environment friendly novel methods of fibre production have developed by Central Coir Research Institute (CCRI) using a biotechnological approach with specific microbial enzymes. This has substantially reduced the retting time to as low as three to five days. High quality production has been maintained (Coir Board, 1996).

4.3 COIR GEOTEXTILES

Coir geotextiles with its Indianised connotation "Coir Bhoovastra", a generic member of the geosynthetic family, are made from the coconut fibre extracted from the husk of the coconut fruit as explained in the following section. Like their polymeric counter parts, coir geotextiles can be synthesised for specific applications in geotechnical engineering practice. Coir geotextiles is not a consumer product, but a technology based product. A range of different mesh matting is available, meeting varying requirements.

Coir fibres can be converted into fabric both by woven and non-woven process. Coir mesh matting of different mesh sizes is the most established coir geotextiles. Mesh matting having different specifications is available under quality code numbers H2M1 to H2M10. These qualities represent coir geotextiles of different mesh sizes ranging from 3.175mm to 25.4mm. Several types of non-woven geotextiles also exist. Most of the non-woven mats are made from loose fibres, which are interlocked by needling or rubberising. Non-woven geotextiles are available in several dimensions and have a minimum thickness of 2mm.

4.3.1 Terminology

Cross machine direction: Direction of the geosynthetic in a direction perpendicular to its long manufacturing or machine direction.

Ends: The threads, which lie along the length of woven fabric

Machine direction: Direction of the geosynthetic in a direction of its long manufacturing

Permittivity: the amount of water moving across a geotextile in unit time through unit area at unit head.

Picks: Weft or filling yarn, which lies across the length of a fabric

Runnage: It is the length of the yarn in metres to weigh one kilogram

Scorage: It is the indication of thickness or thinness of yarn. It is the number of strands that can be accommodated in a span of (900mm) 36 inches without overlapping divided by 20

Secant modulus: The ratio of change in load per unit width to a stated value of strain, usually 10%

Staple: Short fibres in the range of 7mm to 70mm

Tenacity : The fibre strength as force per linear density

Tex: This is the universal unit for yarn count; it is the weight in grams per kilometer of yarn

Transmittivitty: the product of water permeability along the geotextile plane and thickness of the geotextile.

Warp: Set of yarns running length-wise of a fabric

Weft: Set of yarns running width-wise of a fabric

4.3.2 Production

After fibre is produced, the process of spinning extracts yarns. This can be done by wheel spinning, by mechanical spinning or by hand spinning. Wheel spin yarns are of uniform good quality. The quality of yarn is judged by the thickness, colour, appearance, uniformity in twist, strength, fineness, texture, etc.. Yarns are named after the places of production like Anjengo, Vycome, Aratory, etc..

Coir geotextiles are manufactured from mainly four types of coir yarn viz., Aratory, Anjengo, Vycome and Beach. The yarn is wound on bobbins and transferred to a creel. Warping is done between sticks or by means of a peg board, the yarn from the bobbins being passed on to the warping drum and the requisite width is prepared by warping the sections on a weavers beam. Weaving is similar to the pit loom weaving without the fly shuttle arrangement. Two treadle, three treadle, four treadle or multi treadle weaving can be done. The photograph of a fully automatic power loom is shown in Fig. 4.1. Constructional details of coir geotextiles are given in Table. 4.3.

Fig. 4.1 Fully automatic power loom

Table 4.3 Constructional details of coir geotextiles*

Designation	Type of warp yarn	Scorage of warp yarn	Ends Per dm	Types of weft yarn	Picks Per dm	Mass (g/m^2)	Mesh size (mm x mm)
H2M1	Anjengo	14	9.0	Vycome	8	650	-
H2M2	Beach	9	8	Beach	7	700	10 x 10
H2M3	Aratory	15	14	Aratory	14	875	-
H2M4	Anjengo	12	19	Aratory	11	1400	-
H2M5	Vycome	13	9.0	Vycome	8	740	9 x 9
H2M6	Vycome	12	4.6	Vycome	4	400	20 x 20
H2M7	Beypore	12	4.0	Beypore	6	1250	15 x 15
H2M8	Anjengo	12	11	Aratory	7	700	7 x 10
H2M9	Anjengo	11	13	Aratory	7	900	-
H2M10	Anjengo	11	18	Anjengo	9	1300	-

(*Coir Board)

4.3.3 Properties of Coir Geotextiles

Testing and evaluation of coir geotextiles is a key issue, which can answer the question of successful performance in the field. Most of the properties of coir geotextiles are obtained in the same way as that of polymeric geotextiles. No separate testing procedures have evolved so far. Again, though coir geotextiles are classified based on the type of yarn and other parameters, standardisation of coir geotextiles is yet to be evolved. The properties of geotextiles can be grouped into five categories as given below (Mandal and Divshikar, 2002).

1. Physical Properties

   - Mass per unit area

- Thickness
- Specific gravity

2. Mechanical Properties

    - Strip tensile strength
    - Wide width tensile strength
    - Trapezoidal tear strength
    - Grab tensile strength
    - Drop cone penetration resistance
    - Puncture resistance
    - Burst strength
    - Interface friction
    - Pull out resistance
    - Sewn seam strength

3. Hydraulic Properties

    - Cross plane permeability (Permittivity)
    - In plane permeability (Transmittivity)
    - Apparent opening size
    - Porosity

4. Endurance Properties

    - Creep
    - Gradient ratio (Clogging)

5. Degradation Properties

    - Biological degradation
    - Ultraviolet degradation

Among the above properties, physical, mechanical and hydraulic properties play major role in design of reinforced soil structures.

4.4 APPLICATION OF COIR GEOTEXTILES

Coir geotextiles find application in a number of situations in geotechnical engineering practice. Coir geotextiles can be used as an overlay or interlay, the former protecting the surface from run off and the latter performing the functions of separation, reinforcement, filtration and drainage. Soil bio - engineering with coir geotextiles finds effective application in the following field situations.

- Separation application in unpaved roads, railways, parking and storage areas
- Shore line stabilisation
- Storm water channels
- Slope stabilisation in railway and highway cuttings and embankments
- Water course protection
- Reinforcement of unpaved roads and temporary walls
- Providing sub base layer in road pavement
- Filtration in road drains and land reclamation
- Mud wall reinforcement
- Soil stabilisation

Some of the major application areas are detailed below.

4.4.1 Unpaved Roads

Unpaved roads are mainly low volume roads constructed in rural areas. The unsatisfactory performance of roads arises mainly from two factors, namely, the poor quality of subgrades and the insufficient thickness and quality of sub base and base courses. All these factors can be mitigated by the use of coir geotextiles either alone or in conjunction with other products / materials. In cohesionless soil lateral confinement by coir geotextiles can improve the shear resistance and the bearing capacity of the subgrade soil and consequently reduces the thickness of the pavement material. In cohesive soils adequate drainage of the subgrade can be achieved by depressing the water table by use of coir geotextile drains. In very poor soil the use of coir geotextile composite blankets, and strip drains can help in quickening the consolidation of non-expansive clays and reducing the construction time of high embankments. Coir geotextiles can also be used in pavement layer to reduce thickness, increase fatigue resistance and reduce reflection cracking due to traffic.

4.4.2 Embankments and Slopes

Constructions of bunds in marshy areas pose the problem of inadequate shear strength of soil to support the soil fill for the required height. Coir geotextiles can be used both for the foundation support and also within the fill, particularly for filter and separation function, so that the erosion of the sides can be prevented. Use of coir geotextile in protecting natural slopes is well established. Coir fibres are effective in preventing failures due to reversal of pore pressures, through drainage without removal of soil particles. Also with the provision of reinforcements, the stabilisation of the side faces can be improved which otherwise would be very difficult to maintain the slope.

4.4.3 Retaining Walls

Retaining walls are conventionally built to withstand lateral pressure of soil fill through the action of gravity, which involves additional vertical force on soil. This necessitates a strong foundation or large base width and hence costly. Coir geotextiles can be used in the fill itself so that no additional facing wall is required to resist the lateral pressure. This is particularly suited to walls having low height and where foundation soil is weak. The use of coir geotextiles is not advisable for construction of tall retaining walls because of their low tensile strength.

4.4.4 French Drains

These are drainage measures for subgrade soil to lower the water table to protect road formations without the use of slotted pipes to take the collected water. Coir geotextiles with high transmittivity like needled felt with mesh core can be used in the place of pipes and thicker layers can be used to reduce the quantity of pervious sand surrounding the drain.

4.4.5 Vertical Drains

Construction of embankment over soft and sensitive clays requires accelerated consolidation. Several methods such as sand drains, metal drains, geosynthetic prefabricated vertical drains (PVD) etc., are used for this purpose. Instead of this, coir geotextile drains can be used. Two types of such drains are investigated and documented in this report. In such cases, even if the coir decays in course of time, the initial period helps in consolidation and long-term stability is not affected due to the presence of fibres.

4.5 REVIEW OF PREVIOUS INVESTIGATIONS

Coir geotextiles can perform functions viz., separation, reinforcement, filtration, erosion control, pore fluid transmission and dissipation of pressure. Most of the previous works in the field deals with bearing capacity and slope stability / erosion control aspects. A few studies have been reported in the area of subgrade stabilisation and durability aspects also.

Ramaswamy and Aziz (1982 and 1983) have conducted some studies on jute geotextiles and their applications. The laboratory test results conclusively showed that the stress- strength characteristics of the soil are better with the jute fabric than without it. The study also showed the beneficial effects of natural jute geotextiles for subgrade stabilisation.

Prasad et al. (1983) have studied treatment of coir fibres for coir polyester composites. It was reported that the tensile strength of fibres was increased by 15% when fibres were soaked in 5% sodium hydroxide aqueous solution at 28^0C for 72 to 76 hours and thereafter showed a reduction in strength.

Beena (1986) and Ayyar et al. (1988) conducted model studies on reinforced sand bed using coir rope reinforcement along with bamboo strips as anchorages. A parametric study was conducted to determine the effect of horizontal spacing, number of reinforcing layers, region of confinement and pattern of reinforcement. Depending on the arrangement, it was found that BCR could be increased up to 2.5.

The properties and durability of coir geotextiles can be increased by several means. Datye (1988) has reviewed the various methods of treatments such as: a) impregnation with water and oil preservatives, b) impregnation with synthetic polymers, c) coating with cold setting liquid resins, d) coatings with synthetic melts, and e) encasement in concrete when used for reinforcement.

With regard to natural geotextiles, durability is not a matter of concern where a short service life is required, as for drainage and consolidation of soft compressible deposits (Datye and Gore, 1994). Usually, if the geotextiles survives the construction-induced stresses, it will also withstand the in-service stresses (Bonaparte et al, 1988).

A geosynthetic vertical drain, using organic fibres from jute and coir, to improve thick soft clay deposits, known as fibre drain has been developed by Lee et al. (1994). In addition, it was required to withstand the application of high-energy impact on surface fills constructed as part of improvement works. The study showed that the axial and filter permeability of this drain were more than 10^{-4} m/s and 10^{-5} m/s, respectively, for consolidation pressure up to 100 kN/m^2.

Banerjee (1996) has developed a machine to manufacture strip drains by braiding jute yarns enclosing coir yarns. An important feature of the braided jute sheath is its swelling nature resulting in a clog resistant drain. The discharge capacities of these drains were less than that of synthetic drains, however, it does satisfy the requirements.

Coir geotextiles are produced from naturally occurring coir fibre, which is available at relatively low cost in tropical countries. These are found to last for four to six years within the soil environment depending on the physical and chemical properties of the soil and pore fluid (Ramakrishna, 1996).

Ayyar and Dipu (1997) conducted studies on the effect of coir composites on the bearing capacity of sand. It was shown that the coir fabrics could be used economically for improving bearing capacity of sand subgrade significantly.

A study on coir reinforcement for stabilising soft soil subgrade has been carried out by Rajagopal and Ramakrishna (1998). The test results clearly indicated the capability of coir geotextiles in improving the stiffness and bearing capacity of soft subgrade. They concluded that the coir geotextiles are suitable for cost effective field applications.

Use of coir geotextiles in erosion control measures has been reported by Cammack (1988) and Lekha (2004). North American experiences with coir geotextiles for bank stabilisation have reported by Lanier (1991) and White (1991).

In a study carried out by Ayyar and Girish (2000) for finding the possibility of improving the durability of soil reinforcement system, load penetration test and plate load test were done on sand beds reinforced with coir-needled felt, with and without treatments. It was observed that there exist increased strength and durability with the use of coir.

The use of coir products on ground improvement have been studied by Sheeba et al. (2000). Two aspects were mainly considered in their work, one was to examine the load deformation behaviour of needled felt in clays, and the other was to examine the durability of coir in sandy subgrade. While the cement-coated non-woven felt is seen to be the most effective among fabrics studied, even the plain-needled felt improved the resistance of clay slope.

A comprehensive summary of the production, properties and applications of coir geotextiles were provided by Rao and Balan (2000) and Ayyar et al. (2002). The biggest advantage of coir geotextiles is its availability, economic price range, and eco compatibility. It can be tailor made to end users' technical requirements like porometry, permittivity, strength, etc. (Rao and Balan, 2000).

Sampath Kumar et al. (2000) reported the development of jute – coir braided PVD. It was reported that the core prepared out of more number of thin coir yarns showed better performance than a smaller number of thick yarns.

Coir is an abundantly available and renewable resource, which is more durable than jute as its lignin content is higher. Under water, coir has been shown to retain its strength for about 8 to 10 years. Though the survivability concept is more important in the case of geotextile functions, studies indicate that, in most of the separation applications, the critical period in the life of a geotextile is during the construction, rather than during the service life (Koerner, 2005).

4.6 CASE STUDIES

Though less in number compared to polymeric geotextiles, coir geotextiles have been tried for different civil engineering applications. A few case studies are described below.

4.6.1 Protection of Mine Waste Dumps in Goa

Mine waste dumps of iron ore mines in Goa are a perennial problem faced by all mines in Goa. Severe surface erosion takes place along the open mine waste dumps during the monsoon season and creates a lot of environmental problems in the surrounding area. Hillocks are being made, using the mine waste dump of very loose density. Once rainfall starts, small gullies will be formed along the slope, which will lead to large gullies in the subsequent rain, and even deep-seated slope failure may take place. In order to prevent the surface erosion and to increase the slope stability of the dumps coir geotextiles were tried (Ayyar et al., 2002).

Usually the erosion problems in mine waste dumps were addressed by traditional bio-engineering techniques such as planting of acacia plants or cashew plants over a small cover of lateritic soil. However this traditional solution becomes very difficult owing to the high transportation cost for movement of lateritic soils to the dump areas.

Application of new generation coir erosion control blankets with special design features could be an effective alternative to provide solutions and to speed up the vegetation process.

Non-woven coir geotextiles with medium thick polypropylene net on top and bottom were used to protect the surface of the dump. The tensile strength of the non-woven coir geotextile was 3.5 kN/m. Geotextiles were kept in position using wooden planks of 25mm thick and having a length of more than 1000 mm (Fig. 4.2). Length of the planks was so selected that it has to cut the probable slip circle at top and bottom region of the slope. The sites treated with the blankets are performing satisfactorily with stabilisation of the slopes, controlling the soil loss and reduction in pollution.

Fig. 4.2 Mine waste dumps in Goa

4.6.2 Pullangode Estate Erosion Control

One of the important field studies carried out successfully using coir fabrics has been reported by Rao and Balan (2000). The work was carried out in 1994 and was stable enough for vegetation to grow till the matting degrades, which was expected in one year. The location of the site was a rubber plantation near Nilambur in Kerala. The site consists of an area of $583m^2$ abandoned plantation over a length of 50m with side slopes of 49^0- 66^0 suffered severe erosion with formation of wide gullies presented an ideal area for the study.

Coir mattings were chosen and two varieties of white coir yarns manufactured by Aspinwall, H2M8 and H2M6, were used. Rolls of the coir matting were first anchored in the top trench and then unrolled along the slope. Overlaps of 15cm minimum between adjacent ones were given. The anchoring of the matting was made with mild steel staples spaced to form a grid of 2m x 2m. Coir ropes of 20mm diameter were used to tie the coir matting in a criss cross pattern at 90^0, making a grid of 1m x 1m size. Steel staples were driven at each joint in the rope. Type A matting ($915g/m^2$) was used on the upper half of the slope of 66^0 while type B matting ($440g/m^2$) was used for the lower half of the slope of 49^0 since the thicker matting helped in preventing rain splash.

The highlight of this study is the fact that the soil protected is lateritic. Peniseltum purpureum grass was adopted, which is suitable for high elevations and steep slopes. Also coir mesh matting of smaller aperture was more effective than the areas with coarse openings.

4.6.3 Muvattupuzha Canal Protection

Sarma (1997) reported the details of this project which is situated on the 23.20 km of the left bank main canal of the Muvattupuzha Valley Irrigation Project (MVIP) near the main central road crossing between Muvattupuzha and Koothattukulam in Kerala state. This stretch of canal was one of the most highly eroded stretches due to high stream velocity in the major rainy season. Turfing grass protection was unsuccessful since the time taken for grass to take root was more than successive monsoon periods.

The soil is also acidic with p^H 4.3. The details of the coir netting are not reported but it is mentioned that there is sufficient space for proper growing of grass. Possibly it is a coarser net and the monitoring of the strength of net showed 50% decrease in six months. It was assumed that complete degradation would take place in five years. Lemon grass having roots 45cm long was found to be the choice of vegetation in the area and the coir fabric was very conducive to its growth. The case has been reported as a success story for control of erosion. The photograph of a short stretch is shown in Fig. 4.3.

Fig. 4.3 Muvattupuzha canal protection

4.7 NEED FOR PRESENT STUDY

The review of literature shows that the polymeric geotextiles is a versatile material with attractive characteristics and advantages, and as a result, this material is now being used abundantly all over the world. At the same time, these materials have got the disadvantages that it is non-biodegradable, petroleum based and is costly. The review here also shows that, the use of natural geotextiles has not gained popularity, even though some studies have been reported in this area. Though India produces large quantities of coir geotextile, their use for geotechnical and highway engineering applications has not gained confidence due to limited studies undertaken. An elaborate and systematic study, covering different aspects and functions of coir geotextiles, in the context of unpaved roads and embankments is lacking and so also the design methods and procedures for the range of materials having properties that of coir geotextiles. It is in this context, that an in-depth study in the utilisation of coir geotextiles for unpaved roads and embankments are deemed necessary and hence attempted herein.

CHAPTER 5

MATERIAL CHARACTERISATION

5.1 INTRODUCTION

The subgrade soil and other materials, which we use for the construction of roads and embankments, play major roles in deciding the method of design and construction. It is always recommended to use the locally available materials if their strength and hydraulic characteristics permit. Use of geotextiles for improvement of engineering properties of soil is a proven method, which allows the utilisation of local materials to a certain extent, even though the available materials are not so competent in view of design considerations. This can be achieved only at some additional cost and the relation between price and performance is significant in the choice of the material. A geotextile can perform one or several functions to improve the mechanical and / or hydraulic behaviour of the structure in which it is incorporated. The utility of coir geotextiles for performing different functions to improve the engineering behaviour of soil and the structure as such is studied here.

The soil available in nature varies widely in its physical and engineering behaviour. In a study like this, it becomes necessary to limit the analysis to certain types of soil. Two types of red soil (collected from two different sources), a soil of type clayey silt and river sand are the basic materials selected at different stages for the investigations on soil-geotextile matrix. Similarly, out of the different varieties of coir geotextiles available, only three types of them (Woven - H2M6 & H2M8 and Non-woven – AGL C/201) are selected for this study, which are expected to be more suitable for the use in roads and embankments. Other construction materials used herein are granite aggregates and granite screenings for Water Bound Macadam (WBM) road, and rice husk for the construction of vertical drains in embankments. The laboratory evaluation of properties of these materials is summarised in this chapter.

5.2 MATERIALS USED

5.2.1 Soil

Herein, the Red soil obtained from the campus of N.S.S College of Engineering, Palakkad, Kerala (designated as Soil-1), Red soil procured from Kakkanadu in Kochi, Kerala (designated as Soil - 2), Clayey silt soil from Marad area in Kochi, Kerala (designated as Soil - 3) and River sand from Pattambi in Palakkad, Kerala (designated as Soil - 4) were used for the experimental investigation at various stages. In addition, the interfacial frictional characteristics of rock dust (designated as Soil – 5) were studied, which was used as filler material in WBM sections. Table 5.1 summarises the selection of soils with respect to their inherent properties and the requirements for various experimental programmes.

Table 5.1 Selection of soils for different experimental programmes

Sl. No.	Investigation	Soil Used				
		Red Soil (Soil-1)	Red Soil (Soil – 2)	Clayey Silt (Soil – 3)	River Sand (Soil – 4)	Rock Dust (Soil – 5)
1	Studies on interfacial friction	X	-	-	X	X
2	Reinforced subgrade CBR	X	-	X	-	-
3	Bearing capacity studies	-	-	-	X	-
4	Reinforced unpaved roads	X	X	-	-	X
5	Prefabricated vertical drains	-	-	X	-	-
6	Soil - fibre matrix	X	-	X	-	-

The relevant properties of all the soils selected were determined in the laboratory as per current Bureau of Indian Standard Specifications (IS: SP: 36(Part – 1), 1987). Table 5.2 summarises the properties of Soil-1, Soil-2 and Soil-3. Soil-4 being river sand the physical properties of which are reported in Table 5.3 along with rock dust. Grain size distribution curves for the soils are shown in Fig. 5.1

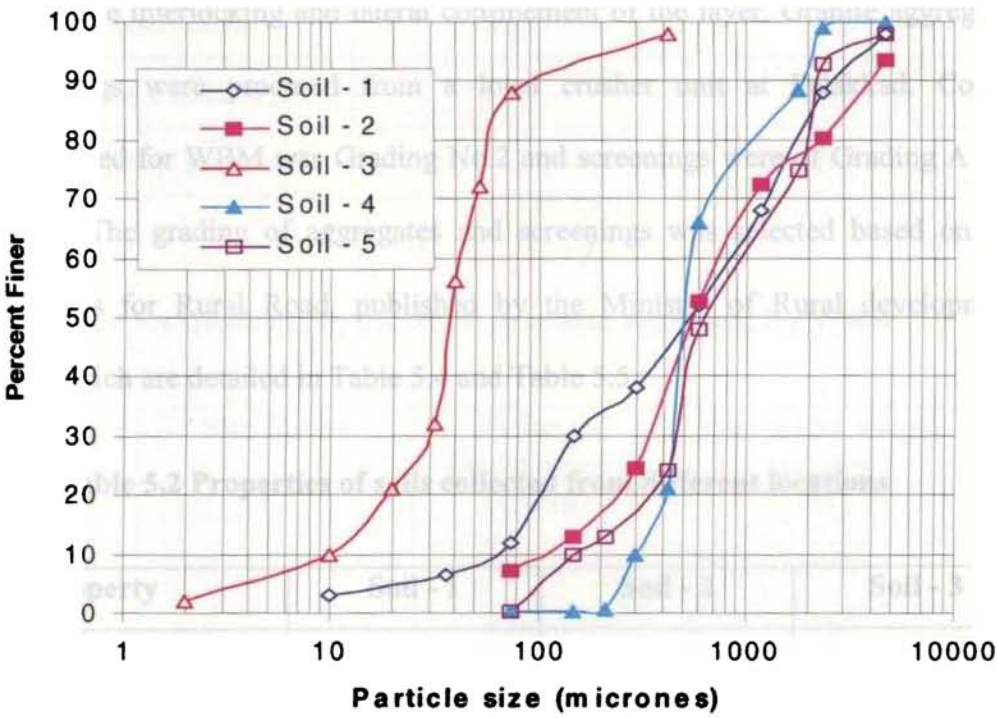


Fig. 5.1 Particle size distribution curves

5.2.1.1 Rock dust

These are waste materials obtained during the crushing of rock masses and are available in different sizes and gradations. This material was used as a filler material while modeling the WBM sections. In the present investigation rock dust was procured from a nearby rock-crushing unit at Palakkad, the properties of which are given in Table 5.3.

5.2.2 Aggregates and Screenings

These are used to simulate sub base and base courses in unpaved roads. In India sub base and base courses are made of Water Bound Macadam (WBM) or Wet Mix Macadam (WMM). The properties of sub base and base courses are very significant, as they have to bear the stresses induced by moving traffic. The strength of this layer depends on the interlocking and lateral confinement of the layer. Granite aggregates and screenings were procured from a local crusher unit at Palakkad. Coarse aggregates used for WBM was Grading No.2 and screenings were of Grading A and Grading B. The grading of aggregates and screenings was selected based on the Specifications for Rural Road, published by the Ministry of Rural development (MORD), which are detailed in Table 5.4 and Table 5.5.

Table 5.2 Properties of soils collected from different locations

Property	Soil - 1	Soil - 2	Soil - 3
Type	Red Soil	Red Soil	Clayey Silt
Classification	CI	CH	ML
Specific gravity	2.62	2.5	2.76
Liquid limit (%)	51	73	32
Plasticity index (%)	20	23	10
Shrinkage limit (%)	22	35	18
Free swell index (%)	0	0	8.25
Soaked CBR (%)	3.35	1.50	0.95
OMC (%)	17	15.8	24
Max. dry density (kN/m^3)	11.7	14.6	15.2

Table 5.3 Properties of sand (soil – 4) and rock dust (soil – 5)

Property	Soil – 4 (River Sand)	Soil – 5 (Rock Dust)
Classification	SP	SW
Specific gravity	2.72	2.01
Effective size (microns)	300	170
Uniformity coefficient	1.83	5.59
Coefficient of curvature	1.17	1.25
Angle of friction (Degrees)	40.0	43.7

Table 5.4 Grading requirements of coarse aggregate for WBM*

Grading No.	Size Range	IS Sieve Designation	Percent by weight passing
1	90mm to 45mm	125mm	100
		90mm	90-100
		63mm	25-60
		45mm	0-15
		22.4mm	0-5
2	63mm to 45mm	90mm	100
		63mm	90-100
		53mm	25-75
		45mm	0-15
		22.4mm	0-5
3	53mm to 22.4mm	63mm	100
		53mm	95-100
		45mm	65-90
		22.4mm	0-10
		11.2mm	0-5

*(as per MORD)

The engineering properties of aggregate samples used in the study were evaluated in the laboratory as per IS-2386 (1963) and are summarised in Table 5.6. The aggregate samples seemed to satisfy the requirements laid by MORD.

Table 5.5 Grading of screenings*

Grading No.	Size of Screening	IS Sieve Designation	Percent by weight passing
A	13.2 mm	13.2mm	100
		11.2mm	95-100
		5.6mm	15-35
		180 microns	0-10
B	11.2 mm	11.2 mm	100
		5.6 mm	90-100
		180 microns	15-35

*(as per MORD)

Table 5.6 Properties of aggregates used for WBM

Property	Value
Aggregate impact value	32%
Los Angeles abrasion value	24%
Deval attrition value	26%
Water absorption	2%
Flakiness index	26%
Elongation index	13%

5.2.3 Coir Geotextiles and Coir Fibres

Natural geotextiles (synthesised by man), such as, coir meshing and jute fabrics are being used in many civil engineering projects including roads and railways. These geotextiles are generally classified by manufacturing process and by the type of yarn / fibre used, and is often separated into two sub categories, namely, Woven and Non-woven. For the present work three types of coir geotextiles, two woven types (designated as H2M6 and H2M8) and one variety of Non-woven (designated as AGL C/201) were used which are shown in Fig. 5.2. Mixed fibres having different lengths were used for making soil – fibre composites for studying consolidation and strength characteristics of soil –fibre matrix.

(a) H2M8 **(b) H2M6** **(c) Non-woven**

Fig. 5.2 Types of coir geotextiles used

5.2.3.1 Evaluation of properties

The properties of coir geotextiles were determined as per standard specifications (BS: 6906 and ASTM, 1993) in the geotextile testing facility available in the Geotechnical engineering laboratory of Cochin University of Science and Technology. A brief outline of the test procedure followed is given below.

The basic property of the geotextile, mass per unit area, was determined by accurately weighing four pieces of coir geotextile having size 200mm x 200mm. The average weight in g/m^2 was reported as the mass per unit area.

The thickness of the coir geotextile was measured using a thickness gauge, which consists of a base plate, a surcharge plate and dial gauges. Four specimens of 200mm x 200mm size were taken. The overall thickness was found out under a surcharge load of 0.02 kg/cm^2. From the total thickness the average thickness of one piece of coir geotextile was found out.

In order to determine the puncture resistance, the coir geotextile specimen was properly clamped on the top side of the CBR mould by means of the clamping strip. The mould was kept in a compression testing machine and the load was applied through a plunger of 50mm diameter. The rate of deformation was kept at 60 mm per minute. The puncture resistance was reported as the peak value of load in Newton.

The cross plane permeability was measured as in the case of soil, using a constant head assembly. The specimen was clamped in the permeability cell properly. The discharge was noted and permeability was calculated.

The tensile strength of coir geotextile was determined using a wide specimen of 200mm width for wide width tensile test and a narrow specimen of 100mm width for strip tensile test. The specimens were cut both in the machine direction and cross machine direction. The specimens were connected by means of the clamping arrangement devices and adapter in a tension testing machine. The distance between the top and bottom clamping was kept respectively as 75mm and 100mm for strip test and wide width test. The loading rate was 300mm/minute for strip test and

10mm/minute for wide width test. The tensile strength was calculated as the peak load per width of the strip, expressed in kN/m.

The secant modulus was determined as the slope of the line joining the origin and the point corresponding to 10% strain on the stress - strain curve. This was expressed in kN/m width.

The physical and engineering properties of the coir geotextiles were determined as detailed above and the results are summarised in Table 5.7.

Table 5.7 Properties of coir geotextiles used

Properties	**H2M8**	**H2M6**	**Non-woven**
Mass/unit area (g)	690	365	908
Thickness at 2 kPa (mm)	7.47	7.03	9.96
Puncture resistance (N)	500	440	50
Cross plane permeability (cm/s)	-	-	0.025
Mesh opening (mm x mm)	7 x 10	20 x 20	-
Construction (warp x weft /dm)	10 x 7	4.5 x 4.5	-
Yarn runnage (m/kg)	220	210	-
Strip tensile strength (kN/m) Machine direction	17.80	05.00	-
Strip tensile strength (kN/m) Cross machine direction	12.80	04.80	-
Wide width tensile strength (kN/m) Machine direction	16.00	05.60	0.78
Wide width tensile strength (kN/m) Cross machine direction	12.80	05.60	0.75
Secant modulus (kN/m) Wide strip	64.00	15.00	9.20

5.3 SUMMARY

Determination of physical and engineering properties of different basic materials used in the investigation were described in this chapter. The results of further investigation would certainly depend on the properties determined in this section. Due to various constraints, the study was limited to the use of four types of soil and three varieties of coir geotextiles even though the types and properties of soil encountered in nature vary vastly. However, an assessment of general behaviour could be obtained by studying these typical materials.

CHAPTER 6

INTERFACE FRICTION CHARACTERISTICS OF COIR GEOTEXTILES

6.1 INTRODUCTION

Reinforcement is one of the most important functions of geotextiles in mechanically improving the soil properties, whether it is used in slopes, embankments, retaining walls, or pavements. Reinforcement achieves this mechanical improvement by withstanding tensile axial force and thereby enhancing the shearing resistance of the soil. The reinforcement acts efficiently when it is oriented in the direction in which tensile strain develops in the deforming soil. The benefit of reinforcement is derived from the tangential and normal components of the tensile reinforcement force acting on the shear surface.

When reinforcement is placed in soil it can develop bond through frictional contact between the soil particles and the planar surface areas of the reinforcement, and from bearing stresses and transverse stresses, which exist in grids or ribbed strips. Deformation in the soil mobilises tensile or compressive force in the reinforcement depending on the inclination of the latter and is ultimately limited by the available bond between soil and reinforcement. The stiffness properties of the reinforcement also influence the soil shear deformation of the composite material, which is required to mobilise the reinforcement force. Hence the shear frictional behaviour of soil - geotextile interfaces plays a pivotal role in the overall performance of geotextile reinforced constructions.

The interfacial friction depends upon a large number of parameters such as pressure, grain size and shape, surface roughness of geotextile, etc.. The frictional resistance mobilised between the soil and the reinforcement has a significant role while analysing the internal and an external stability of the mechanically stabilised earth structures. Hence the properties of the interaction between soil and reinforcement such as coefficient of friction must be determined as an indispensable factor along with the individual properties of the soil and reinforcement, in order to arrive at the load conditions on the geotextile and for the determination of design factors such as spacing and extent of reinforcement.

There are two limiting modes of interaction viz., *direct sliding*, in which a block of soil slides over a layer of reinforcement, and *pull out*, in which a layer of reinforcement pulls out from the soil once its maximum available bond stress is overcome. Modified direct shear tests are suitable for measuring the coefficient of direct sliding between soil and any type of reinforcement materials and pull out tests to model the development of bond stresses. It is reported that the results of the pull out test are difficult to interpret and can be greatly influenced by the conditions in the test, even though special apparatus is used for modeling (Palmeira and Milligan, 1989). For design, it is usually sufficient to calculate the bond coefficient from the theoretical analysis (Jewell, 1996). Hence in this present study it is limited only to direct sliding tests.

Five series of modified direct shear tests on the soil – coir geotextile interfaces were conducted under monotonic loading. The purposes of the tests were to examine the behaviour of interfaces between the four types of soil and three types of coir geotextiles under different test conditions. The results would provide a better understanding of the shear frictional mechanism of soil coir geotextile interface to use in the design of unpaved roads and embankments utilising coir geotextiles. Details of the testing programme, test results and discussions of the test results are presented in this chapter.

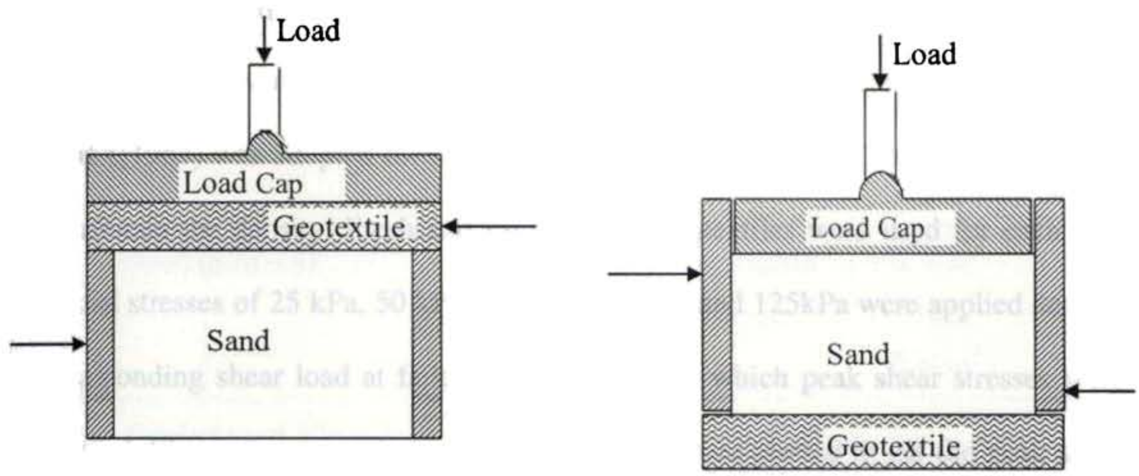
6.2 TEST DESCRIPTION

The direct shear test apparatus consisted of 60mm x 60mm x 40mm deep box which can be split horizontally at mid height with displacement controlled loading system. The rate of shear displacement was 0.02mm /second. The constant normal stress was applied by dead load.

The size of the direct shear box in this study was relatively small. The boundary effects could affect the test results to some degree. However test results with the 60mm square direct shear box were expected to have insignificant boundary effects for two reasons. First, the dimensions of the direct shear box were approximately hundred times the mean grain size of the soil specimen. This was in the range recommended by ASTM D3080 and by other researchers (Jewell and Wroth, 1987; Palmeira, 1988). Second, it was confirmed by O'Rourke et al. (1990) that when the 60mm square direct shear apparatus was used for Ottawa sand and High Density Poly Ethylene (HDPE) to find the interface friction, it gave results similar to those obtained from large size direct shear apparatus.

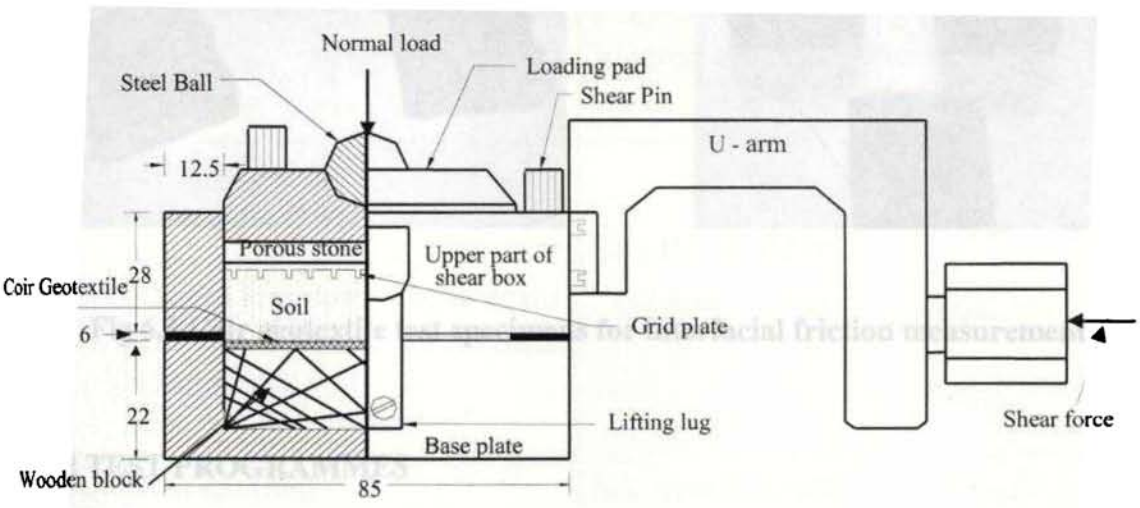
In order to determine the interfacial friction, several modifications have been made by different researchers (Subba Rao et al., 1996). Basically two types of arrangements have been tried. The solid material can be placed over prepared soil bed (type A mode of shear) or the soil can be prepared over the solid material (type B mode of shear).

Schematic diagram of type A and type B modes of shear is shown in Fig 6.1. It is reported that type B apparatus has the advantage of yielding friction angle values applicable to both type A and type B situations (Subba Rao et al., 1998). In the present investigation type B apparatus was used. Fig 6.2 shows the schematic diagram for the test set - up for the present study. The range of normal stress applied was 25 kN/m^2 to 125 kN/m^2.



(a) Type A **(b) Type B**

Fig. 6.1 Type A and Type B modes of shear



Sectional view **Front view**

Fig. 6.2 Schematic diagram for test set - up

6.3 SPECIMEN PREPARATION AND TEST PROCEDURE

Wooden blocks were cut to size so as to fit into the bottom half of the direct shear box. Coir geotextiles cut into 60mmx 60mm sizes were glued in the top surface of the rigid wooden block (Fig. 6.3). The rigid wooden block with coir geotextile was fitted inside the lower half of the direct shear box. The upper part of the shear box was placed over the lower part and pins were placed at the corners to keep the two parts intact. Calculated quantity of soil was placed in the upper part of the shear box and tampings were given to get the required density. The test procedure laid in IS: SP: 36 - Part 1(1987) was adopted for the entire series of experiments. Great care was taken to maintain the density. Fresh soil samples and geotextiles were used for each test. Normal stresses of 25 kPa, 50 kPa, 75 kPa, 100 kPa and 125kPa were applied and the corresponding shear load at failure was noted from which peak shear stresses were obtained. Shear stress versus normal stress graphs were plotted to get the peak angle of internal friction.

Fig 6.3 Coir geotextile test specimens for interfacial friction measurement

6.4 TEST PROGRAMMES

Five series of experiments were done using three types of soil (sand, rock dust, and red earth) and three types of coir geotextiles (woven – H2M6 and H2M8 and Non - woven). 68 sets of experiments were done in different combinations to get frictional characteristics of different interfaces. Interface friction characteristics of sand and rock dust in natural and in different graded states were studied in addition to the red soil. The effect of water content was studied in red soil and in all other cases the material was kept in dry state. Test programmes for the direct shear test are presented in Table 6.1.

Table 6.1 Details of test series

Series	Type of Subgrade soil	Type of Geotextile	Unit weight/ Water content
I	Sand (Soil – 4)	Nil, H2M6, H2M8 and Non - woven	$15 kN/m^3$ $16 kN/m^3$ $17 kN/m^3$
II	Graded sand (2mm to 4.75mm)	Nil, H2M6, H2M8 and Non - woven	$16 kN/m^3$
	Graded sand (1mm to 2mm)		
	Graded sand (1mm to 425 microns)		
III	Rock dust (Soil – 5)	Nil, H2M6, H2M8 and Non - woven	$15.1 kN/m^3$ $16.5 kN/m^3$ $17.6 kN/m^3$
IV	Graded rock dust (2mm to 4.75mm)	Nil, H2M6, H2M8 and Non - woven	$16.5 kN/m^3$
	Graded rock dust (1mm to 2mm)		
	Graded rock dust (1mm to 425 microns)		
V	Red soil (Soil – 1)	Nil, H2M6, H2M8 and Non - woven	$15.0 kN/m^3$ 10%, 15% and 20%

6.5 RESULTS AND DISCUSSION

The results of the direct shear tests conducted are presented and discussed in the following sessions. The relationship between peak shear stress and normal stress was plotted for different soil and coir geotextile interfaces. Failure state was defined as the peak shear stress. Values of direct sliding coefficient (C_{ds}) were calculated as:

$$C_{ds} = \frac{R_{ds}}{L\sigma_N \tan\phi} \qquad \text{--------------------(6.1)}$$

where, R_{ds} = Maximum shear resistance in kN/m,

L = Stationary length of geosynthetic in m,

σ_N = Effective normal stress in kN/m^2, and

ϕ = Effective soil friction angle in degrees.

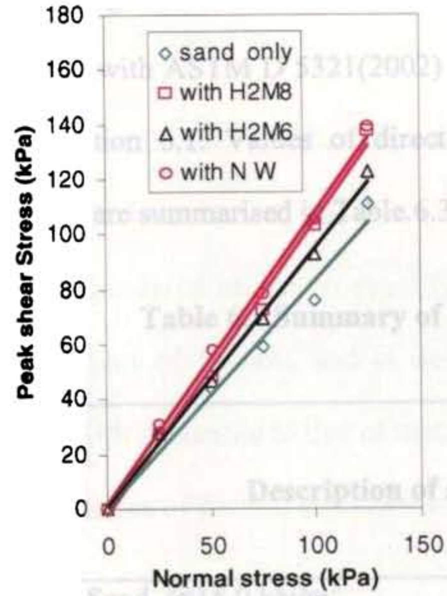
The values of interface friction angles (δ) obtained for different series of experiments are summarised in Table 6.2 in order to have a quantitative analysis.

6.5.1 Shear Stress - Normal Stress Relationship

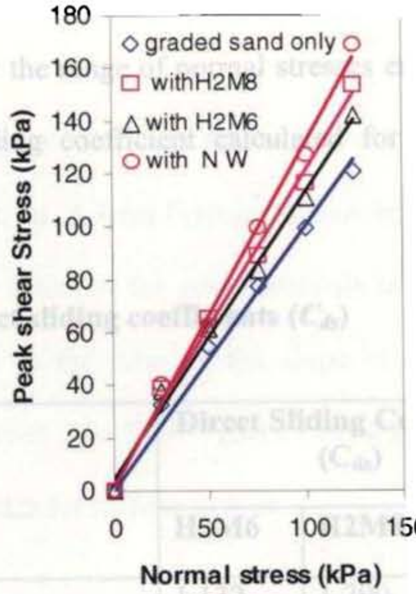
The shearing characteristics of different materials used in unpaved roads and embankments show vast variations. Hence it is required to understand the shear behaviour when coir geotextiles have to perform in conjunction with different materials during and after the construction. Fig.6.4 shows the failure envelopes obtained by plotting peak shear stresses and normal stresses for typical cases. Analysing the similar plots it can be observed that the introduction of geotextile increases the shear resistance invariably in all cases studied. It could also be seen that the maximum shear strength by way of interfacial friction is developed with soil having larger particle size at high density, even though the normal stress – shear stress relation was identical for all the cases of soil - coir geotextile interfaces studied.

Table 6.2 Summary of interface friction angles

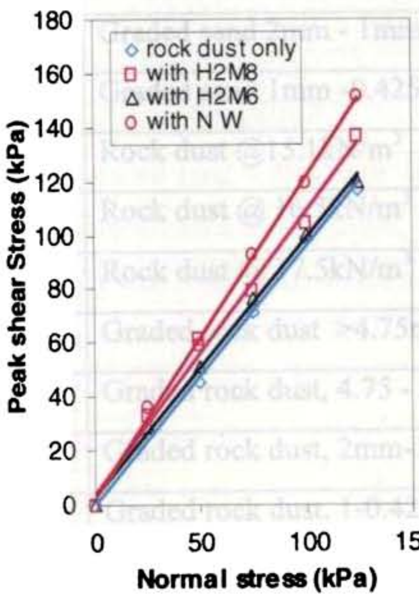
Test Series No.	Description	Angle of internal friction (Φ) (degrees)	Interface Friction angle (δ) (degrees)		
			H2M6	H2M8	NW
1	Sand @15.0 kN/m^3	36.98	38.79	41.26	43.35
2	Sand @16.0 kN/m^3	41.00	43.63	44.65	47.12
3	Sand @17.0 kN/m^3	44.94	46.88	50.02	52.32
4	Graded sand 4.75 –2mm @16.0kN/m^3	45.19	47.46	49.50	52.45
5	Graded sand 2mm - 1mm, @ 16.0kN/m^3	44.97	46.85	48.47	49.43
6	Graded sand 1mm -0.425mm, @ 16.0kN/m^3	39.63	44.90	46.33	46.97
7	Rock dust @15.1kN/m^3	39.63	40.56	41.48	44.17
8	Rock dust @ 16.5kN/m^3	43.69	43.80	46.33	50.00
9	Rock dust @ 17.5kN/m^3	46.72	47.77	50.39	52.98
10	Graded rock dust >4.75mm, @16.5kN/m^3	57.28	57.92	59.26	59.48
11	Graded rock dust, 4.75 - 2mm, @16.5kN/m^3	54.90	55.98	56.86	57.23
12	Graded rock dust, 2mm - 1mm, @16.5kN/m^3	50.02	51.23	52.07	53.08
13	Graded rock dust, 1 - 0.425mm, @16.5kN/m^3	46.82	47.68	50.18	50.05
14	Graded rock dust, <0.425mm, @16.5kN/m^3	42.52	42.63	43.84	45.00
15	Red soil @15.0kN/m^3 w/c =10%	42.49	44.31	47.91	49.21
16	Red soil @15.0kN/m^3 w/c = 15%	41.98	42.36	39.05	43.48
17	Red soil @15.0kN/m^3 w/c = 20%	30.95	34.47	38.18	38.77



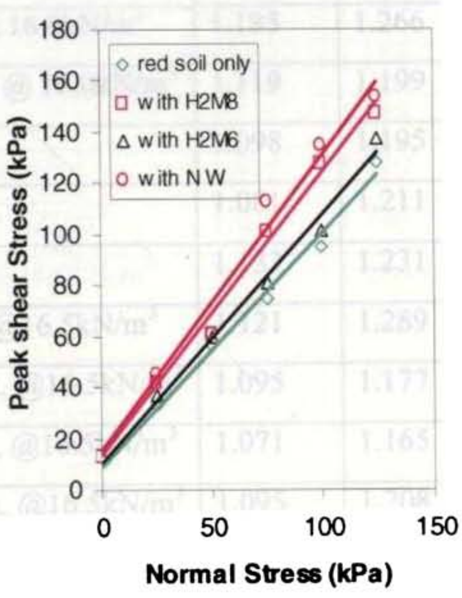
(a) Sand at 16.0 kN/m^3



(b) Graded sand (4.75mm to 2mm size) at 16.0kN/m^3



(c) Rock dust at 16.5 kN/m^3



(d) Red soil at 10% water content at 15.0kN/m^3

Fig. 6.4 Variation of peak shear stress with normal stress for coir geotextile interface

6.5.2 Direct Sliding Coefficient and Friction Coefficient

Direct sliding coefficient or Coefficient of Direct Sliding (Cds) was determined in accordance with ASTM D 5321(2002) over the range of normal stresses encountered using equation 6.1. Values of direct sliding coefficient calculated for the cases considered are summarised in Table.6.3.

Table 6.3 Summary of direct sliding coefficients (C_{ds})

Test Series No.	Description of soil	Direct Sliding Coefficient (C_{ds})		
		H2M6	H2M8	NW
1	Sand @15.0 kN/m^3	1.132	1.290	1.311
2	Sand @16.0 kN/m^3	1.165	1.226	1.332
3	Sand @17.0 kN/m^3	1.128	1.195	1.245
4	Graded sand 4.75 –2mm @16.0kN/m^3	1.219	1.285	1.354
5	Graded sand 2mm - 1mm, @ 16.0kN/m^3	1.185	1.266	1.316
6	Graded sand 1mm -0.425mm, @ 16.0kN/m^3	1.119	1.199	1.412
7	Rock dust @15.1kN/m^3	1.098	1.195	1.223
8	Rock dust @ 16.5kN/m^3	1.081	1.211	1.323
9	Rock dust @ 17.5kN/m^3	1.133	1.231	1.294
10	Graded rock dust >4.75mm, @16.5kN/m^3	1.121	1.289	1.315
11	Graded rock dust, 4.75 - 2mm, @16.5kN/m^3	1.095	1.177	1.190
12	Graded rock dust, 2mm-1mm, @16.5kN/m^3	1.071	1.165	1.223
13	Graded rock dust, 1-0.425mm, @16.5kN/m^3	1.095	1.208	1.254
14	Graded rock dust, <0.425mm, @16.5kN/m^3	1.052	1.189	1.236
15	Red soil @15.0kN/m^3 w/c =10%	1.279	1.473	1.559
16	Red soil @15.0kN/m^3 w/c = 15%	1.290	1.415	1.472
17	Red soil @15.0kN/m^3 w/c = 20%	1.915	2.135	2.168

Based on the specification for geosynthetic used as soil reinforcement in mechanically stabilised earth-retaining structures, the minimum direct sliding coefficient values shall not be less than 0.8.

The interface friction coefficient was calculated as the tangent of the interfacial friction angle between soil and coir geotextiles. A term Friction Enhancement Factor (FEF) is considered herein, to quantify the effect of the coir geotextile in increasing the coefficient of friction, and is defined as the ratio of the slope of the failure envelope with geotextile to that of unreinforced case [FEF = (tan δ / tan Φ)]. Table 6.4 gives the values of friction enhancement factor for different cases.

Table 6.4 Friction Enhancement Factor

Test Series No.	Description of soil	FEF = tan δ / tan Φ		
		H2M6	H2M8	NW
1	Sand @15.0 kN/m^3	1.067	1.165	1.253
2	Sand @16.0 kN/m^3	1.136	1.178	1.283
3	Sand @17.0 kN/m^3	1.146	1.195	1.298
4	Graded sand 4.75 –2mm @16.0kN/m^3	1.089	1.170	1.300
5	Graded sand 2mm - 1mm, @ 16.0kN/m^3	1.077	1.140	1.180
6	Graded sand 1mm -0.425mm, @ 16.0kN/m^3	1.120	1.126	1.129
7	Rock dust @15.1kN/m^3	1.033	1.068	1.173
8	Rock dust @ 16.5kN/m^3	1.004	1.097	1.247
9	Rock dust @ 17.5kN/m^3	1.038	1.138	1.249
10	Graded rock dust >4.75mm, @16.5kN/m^3	1.025	1.124	1.137
11	Graded rock dust, 4.75 - 2mm, @16.5kN/m^3	1.029	1.076	1.092
12	Graded rock dust, 2mm - 1mm, @16.5kN/m^3	1.044	1.076	1.116
13	Graded rock dust, 1 - 0.425mm, @16.5kN/m^3	1.021	1.125	1.120
14	Graded rock dust, <0.425mm, @16.5kN/m^3	1.004	1.047	1.090
15	Red soil @15.0kN/m^3, w/c =10%	1.065	1.208	1.265
16	Red soil @15.0kN/m^3, w/c = 15%	1.013	0.918	1.054
17	Red soil @15.0kN/m^3, w/c = 20%	1.144	1.320	1.339

The variations of direct sliding coefficient (C_{ds}) and interface friction coefficient ($tan\delta$) are shown in Fig. 6.5 and Fig.6.6 respectively. It could be observed in all cases that, the values of direct sliding coefficient was greater than the minimum recommended value of 0.8 for all soil coir geotextile interfaces. Here the values obtained were in between 1.0 to 1.5. The values were minimum for H2M6 interfaces and maximum for Non-woven interfaces. Also, it was observed that graded soil with higher particle size fetch high values of direct sliding coefficient and interface friction coefficient.

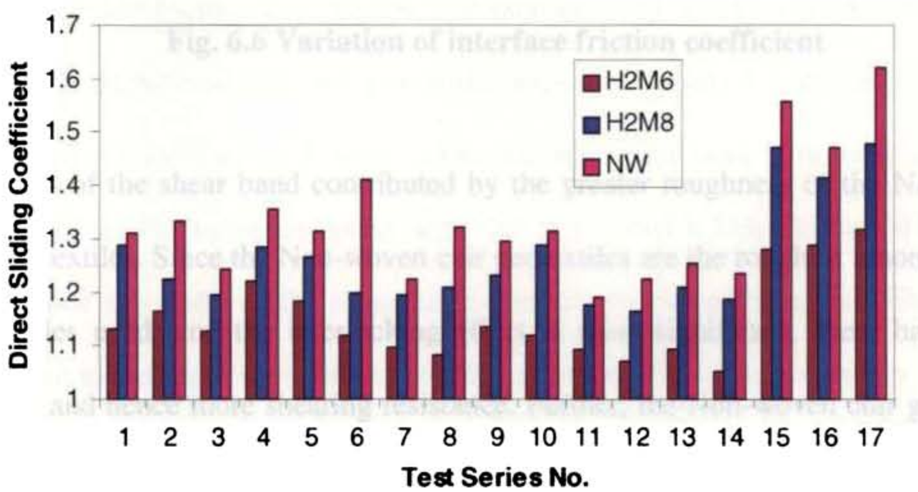


Fig. 6.5 Variation of direct sliding coefficient

6.5.3 Effect of Types of Coir Geotextile

The behaviour of interface depends on the characteristics of reinforcing material, the coir geotextiles, and the soil properties. It could be seen that the interfacial friction is more with Non-woven coir geotextiles irrespective of the type of soil and its density, which can be observed from Table 6.2 and Fig. 6.4. This may be due to the larger

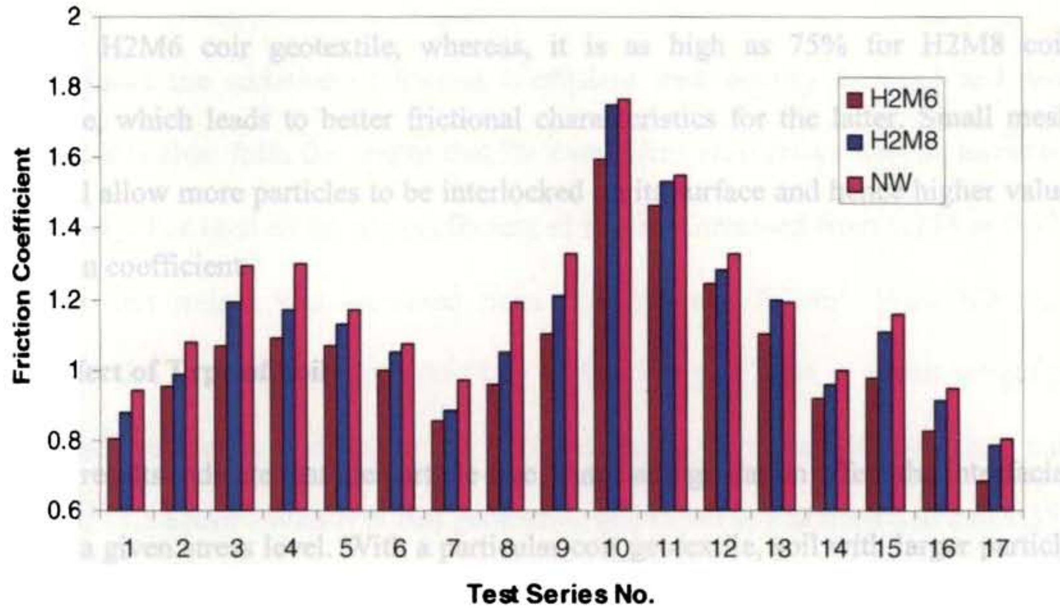


Fig. 6.6 Variation of interface friction coefficient

thickness of the shear band contributed by the greater roughness of the Non-woven coir geotextiles. Since the Non-woven coir geotextiles are the roughest among the coir geotextiles used, and the interlocking effect is most significant, shear band is the thickest and hence more shearing resistance. Further, the Non-woven coir geotextiles are in full contact with the soil, and hence more soil grains are mobilised in the shearing process. Comparing the performance of H2M8 with Non-woven, it could be seen that the developed interface friction angle is more or less the same in the case of graded rock dust.

From Fig. 6.5 it can be seen that, the direct sliding coefficient for Non-woven coir geotextile is consistently higher than other geotextiles in all cases followed by H2M8 type. Among woven coir geotextiles, H2M6 and H2M8, it could be seen that H2M8 coir geotextile performs far better than H2M6 coir geotextile in all soils tested as can be seen from the Table 6.2. This may be due to the lesser contact area of H2M6 coir geotextile which in turn depends on the grid surface area available for sliding and hence the mesh size opening. The effective grid surface area available for sliding is 42% for H2M6 coir geotextile, whereas, it is as high as 75% for H2M8 coir geotextile, which leads to better frictional characteristics for the latter. Small mesh sizes will allow more particles to be interlocked on its surface and hence higher value of friction coefficient.

6.5.4 Effect of Type of Soil

The test results indicate that the particle size, shape and gradation affect the interfacial shear at a given stress level. With a particular coir geotextile, soil with larger particle size gave higher friction. For example, for sand at a unit weight of 15 kN/m^3, the C_{ds} with H2M6, H2M8 and NW coir geotextiles were respectively 1.132, 1.290 and 1.311 whereas, for rock dust with lesser effective size than sand at a unit weight of 15.1kN/m^3, the values were respectively 1.098, 1.195 and 1.223. This could be due to the fact that the width of the shear band was affected by particle size. The area of shear zone increases as the particle size increases. Larger amount of energy is needed to transform the larger area. The frictional resistance was more in the same order of area of sheared zone, that is, in the same tone of the grain size.

The interfacial friction coefficients for sand at a unit weight of 15.0 kN/m^3 were 0.804, 0.879 and 0.944 for H2M6, H2M8 and Non-woven respectively, whereas the corresponding values for rock dust at a comparable unit weight were 0.856, 0.884 and 0.972 respectively. This shows that the interface friction coefficient of rock dust is higher than that of sand.

6.5.5 Effect of Density

Fig.6.7 shows the variation of friction coefficient with density for sand and rock crushing. It is clear from the graphs that the coefficient of interface friction increases with density. For sand alone, the coefficient of friction increased from 0.753 to 0.998 when its unit weight was increased from 15kN/m^3 to 17kN/m^3. With NW coir geotextile, this increase was from 0.944 to 1.295. For rock dust, at a unit weight of 15.1kN/m^3 the friction coefficient was 0.83, which was increased to 1.062 at a unit weight of 17.5 kN/m^3. With NW coir geotextile, this increase was from 0.972 to 1.33.

The percentage increase in interface friction coefficient for sand-NW interface with respect to sand-sand interface varies from 25.3% to 29.3% as unit weight of sand changes from 15kN/m^3 to 17kN/m^3. In the case of rock dust similar variation was from 17.4% to 24.9%. For rock dust the friction coefficient was increased by 28% as unit weight increased from 15.1kN/m^3 to 17.5kN/m^3 whereas the increase was 37% for rock dust- NW coir geotextile interface.

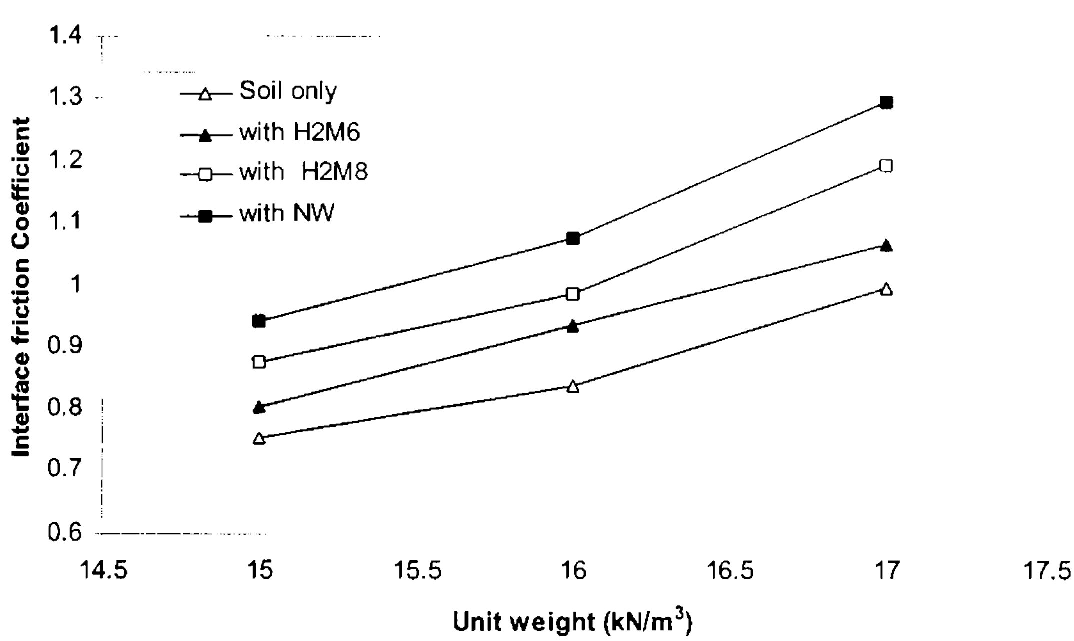


Fig.6.7 (a) Variation of friction coefficient with density for sand

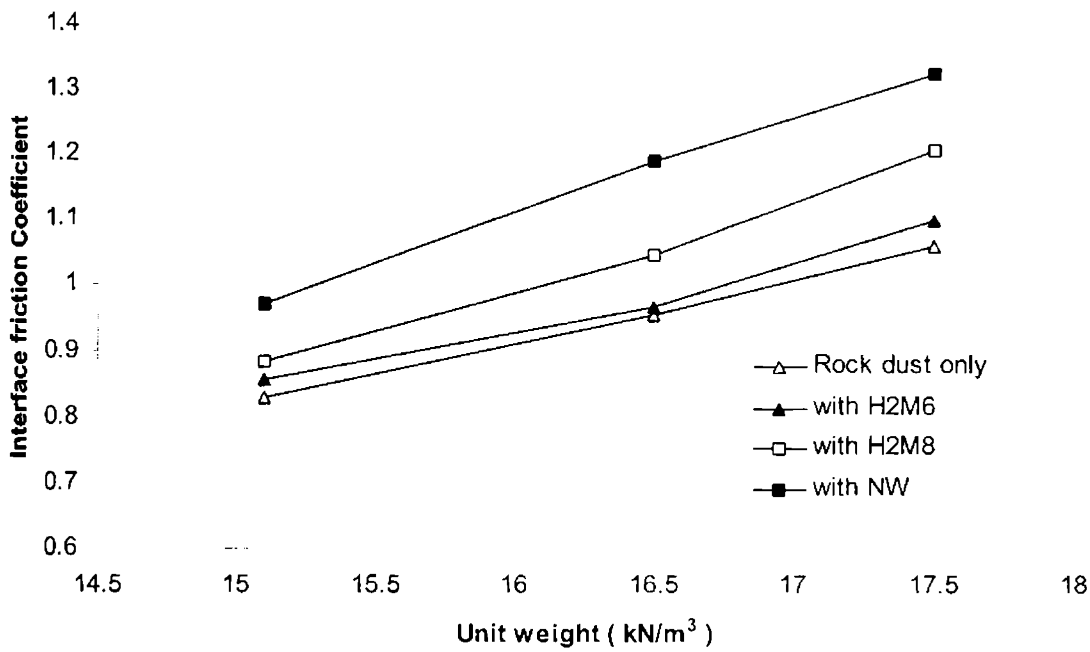


Fig. 6.7(b) Variation of friction coefficient with density for rock dust

6.5.6 Effect of Water Content

Fig.6.8 shows the variation of interface friction coefficient with water content in the case of red soil. Experiments were done keeping unit weight constant at 15.0 kN/m^3 (corresponding to 90% of OMC) with varying water content. It was observed that the coefficient of friction was reduced when water content was increased. The behaviour was identical for all types of coir geotextiles. For soil without geotextile the coefficient of friction was 0.92 at 10% water content, which was reduced to 0.89 at 15% and then to 0.6 at 20% water content. With NW coir geotextiles the interface friction coefficient was reduced to 0.8 from 1.16 when water content was changed from 10% to 20%. The variation in C_{ds} was only marginal when water content was in the range of 15% to 20% with average value of C_{ds} as 1.0 for H2M6, 1.06 for H2M8 and 1.16 for Non-woven.

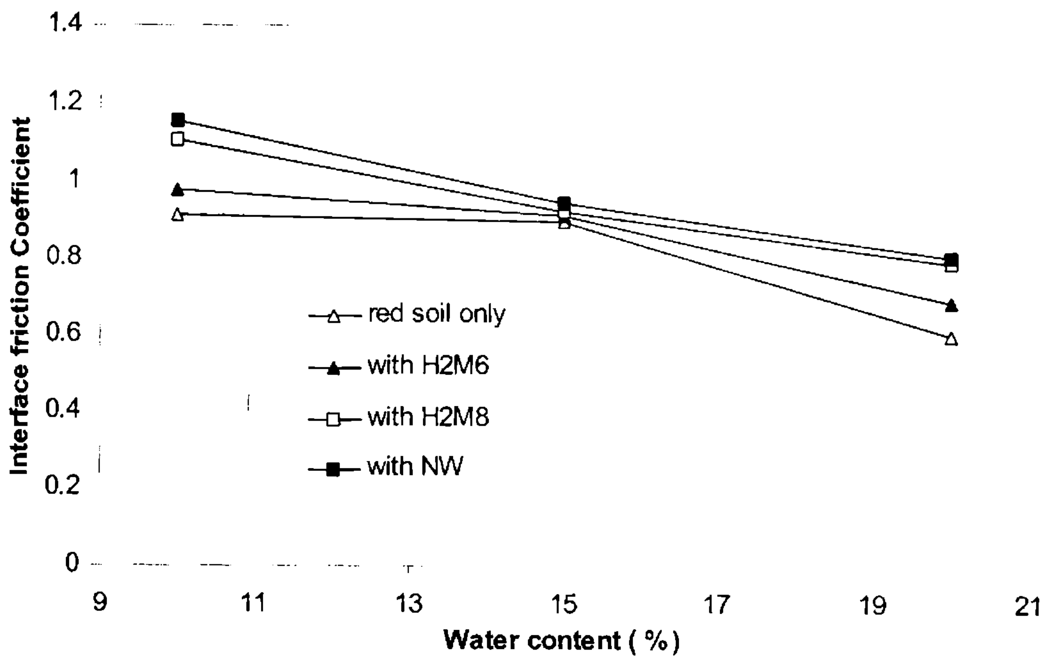


Fig 6.8 Variation of friction coefficient with water content

6.6 THEORETICAL ANALYSIS

As can be observed from the previous discussion, the frictional characteristics of soil geotextile interface depends on many factors such as type of geotextile, type of soil, density of soil, etc. While designing the geotextile-reinforced structures, the overall direct sliding resistance is an important characteristic and is measured in terms of direct sliding coefficient. Theoretical expression for direct sliding coefficient is available for polymer or synthetic type of materials (Jewell, 1996) as

$$\alpha_{ds} = a_s\left(\frac{\tan\delta}{\tan\phi}\right) + (1 - a_s) \qquad \text{(6.2)}$$

where,

α_{ds} = direct sliding coefficient,

a_s = fraction of grid surface area,

δ = interface friction angle, and

ϕ = friction angle for soil.

Here an attempt is made to compare the values obtained theoretically using the above equation with the actual direct sliding coefficient values obtained from the experiment for different cases as shown in Fig. 6.9. It can be observed that both the experimental and theoretical values compare well within marginal variation, and hence it can be safely state that the formula holds good in the case of coir geotextile also.

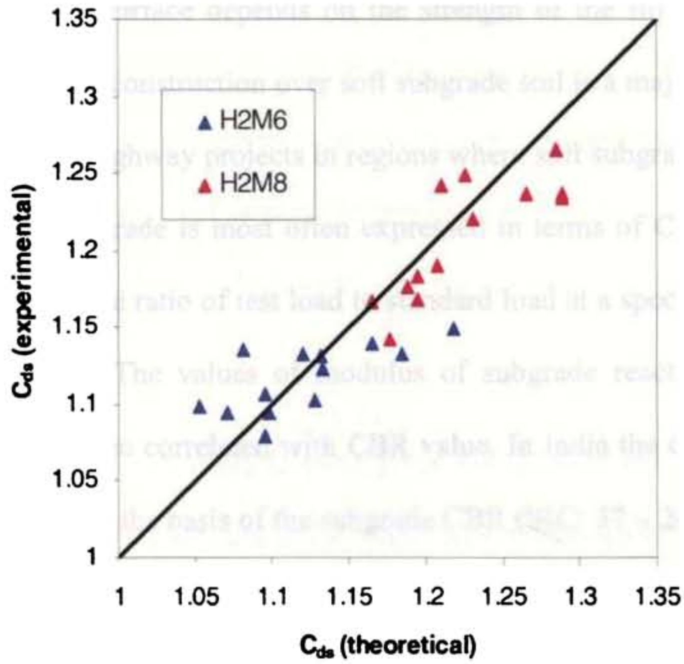


Fig. 6.9 Comparison of theoretical and experimental values of direct sliding coefficient

6.7 SUMMARY

The experimental studies revealed that the interfacial friction characteristics of coir geotextiles are sufficient to fulfill its functions such as reinforcement, separation, etc.. The values of interface friction characteristics of coir geotextiles obtained were much more than the values specified by the International Geosynthetic Society for use in different applications. Due to the provision of coir geotextiles, the friction coefficient was found to increase by as much as 30% to 40%.

CHAPTER 7

STRENGTH BEHAVIOUR OF COIR GEOTEXTILE REINFORCED SUBGRADE

7.1 INTRODUCTION

The behaviour of road surface depends on the strength of the fill material and the subgrade below it. Road construction over soft subgrade soil is a major issue affecting cost and scheduling of highway projects in regions where soft subgrades are common. The strength of the subgrade is most often expressed in terms of California Bearing Ratio (CBR), which is the ratio of test load to standard load at a specified penetration, by a standard plunger. The values of modulus of subgrade reaction and resilient modulus of soil have been correlated with CBR value. In India the design of flexible pavement is primarily on the basis of the subgrade CBR (IRC: 37 – 2001).

The CBR method is one of the earlier empirical methods for pavement design developed during 1928-1929 (Ullidtz, 1986). This method involves determination of CBR value of subgrade for the most critical moisture condition. The design thickness of pavement is read against the CBR value of the subgrade from the design charts, which were developed from experience on pavement performance after noting and analysing a number of failed pavement sections and the corresponding subgrade CBR values. Based on this apparent relationship between the CBR value and thickness of pavement, the US Corps of Engineers in 1940 adopted the CBR method of design for airfield pavements (Horonjeff and Mckelvey, 1983). It may be noted that this attempt at designing pavement did not initially involve the number of load repetitions that a pavement can sustain and hence later, correction factors related to the number of load repetitions were also introduced.

Many techniques have been evolved to strengthen the highway soil subgrade. Most of them primarily involve stabilisation using chemical admixtures. One of the recent techniques is the use of geotextiles. Geotextiles can be placed within subgrade to strengthen the subgrade and also can be placed at the interface between subgrade and sub base. Since subgrade CBR is taken as the criterion for the design of flexible pavements, the thickness of the component layers (sub base and base course) will be reduced when the subgrade CBR is high. In this section, the results of studies on the performance characteristics of the subgrade with the provision of coir geotextiles are reported.

Two subgrade soils: Red soil (Soil-1) and Clayey silt (Soil-3) and three varieties of coir geotextiles (H2M6, H2M8 and NW) were used in the study. CBR tests were conducted with coir geotextiles at depths of H/2, H/3 and H/4 from the top surface of soil where, H is the depth of CBR specimen. Experiments were also conducted with multiple coir geotextile layers and with aggregate layer above the coir geotextile layer. Using multiple linear regression analysis a mathematical model for modified CBR was obtained in terms of original CBR of the subgrade soil, and properties and depth of placement of coir geotextile. The reduction in the required thickness of the base layer can be computed using US Army method, and IRC method, which is reported elsewhere in this thesis.

7.2 EXPERIMENTAL PROGRAMME

The objective of this study was to find out the increase in strength mobilisation in terms of CBR values, by conducting CBR tests on the subgrade soil when reinforced with coir geotextiles placed at different positions. It was also aimed at in assessing the saving in aggregate thickness due to the use of coir geotextiles in unpaved roads. The details of the experimental programme are given in Table 7.1.

Table 7.1. Parameters varied in laboratory experiments

Parameter	Variables
Subgrade material	Clayey silt – with and without aggregate layer
	Red soil – with and without aggregate layer
Soaking condition	Soaked
	Unsoaked
Type of coir geotextile	Woven – H2M8
	Woven – H2M6
	Non-woven – AGL C/201
Position of goir geotextile	No geotextile – Control section
	At H/2 from surface
	At H/3 from surface
	At H/4 from surface
	Multiple layers – at H/3 and H/2

7.2.1 Preparation of Specimen

The experimental set - up consisted of that required for the standard laboratory CBR test. It consists of a cylindrical mould of 150 mm inner diameter and 175mm height and a cylindrical plunger of 50mm diameter. For CBR test on the remoulded sample, soil is compacted in the CBR mould with moisture content corresponding to OMC.

Compaction of the soil sample was done by static method. In this method, a known quantity of air-dried soil passing 20mm sieve was taken and mixed with requisite water to get OMC. Calculated quantity of wet soil was transferred to mould and compacted to the required height statically using hydraulic jack. Top surface was scratched and the coir geotextile specimen, cut to the inside dimension of the CBR mould, was placed over it. Soil was put in the next layer and compacted as in the previous case. A filter paper was placed on the top of the specimen and then the surcharge disc over it. Schematic representations of the test specimens are shown in Fig.7.1.

In order to obtain the soaked CBR values, the specimens were soaked for 96 hours before loading the specimen. To soak the specimen, the specimen in the mould and surcharge weight was wrapped in a gunny bag and kept immersed in water. The mould, after four days of soaking is taken out and water is allowed to drain off. The sample, along with the surcharge, is then subjected to loading.

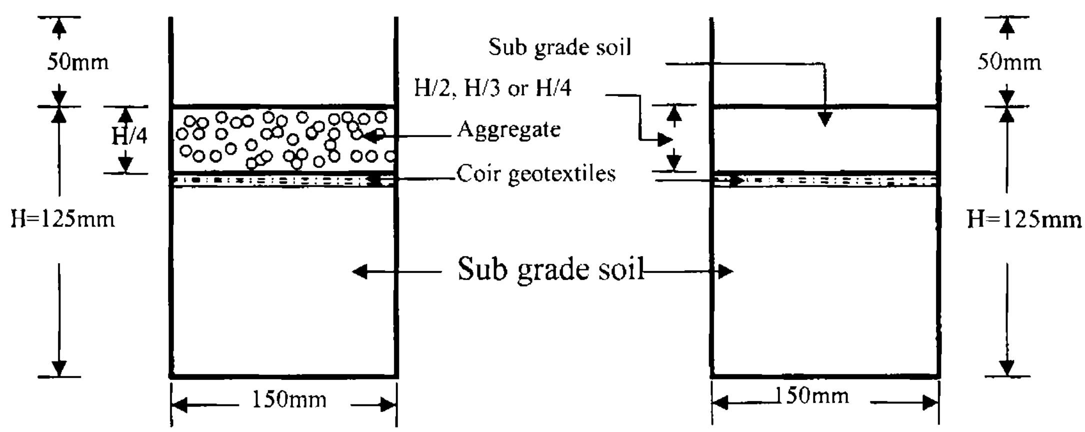


Fig 7.1 Schematic representation of CBR test samples

7.2.2 Testing

Specimens were tested in a load frame with an electronic outfit, which gives LED display of loads and penetrations. A standard plunger of 50mm diameter was penetrated into the soil at the rate of 1.25mm/minute. The load values corresponding to penetrations of 0.5mm, 1.0mm, 1.5mm, 2.0mm, 2.5mm, 3.0mm, 4.0mm, 5.0mm, 7.5mm, 10mm and 12.5mm were noted. From the load - penetration graphs, CBR value was calculated as the highest value obtained from the ratio of test load divided by the standard load and expressed in percentage.

$$CBR\ (\%) = [Test\ load\ /\ Standard\ Load] \times 100 \qquad \text{----------------} \ (7.1)$$

The standard loads are 13.7kN, 20.55kN, 26.3kN, 31.8kN,and 36.0kN for 2.5mm, 5.0mm, 7.5mm, 10.0mm, and 12.5mm respectively.

7.3 RESULTS AND DISCUSSION

7.3.1 General

CBR values expressed in percentage for different cases are summarised in Table 7.2, in which the initial CBR refers to percentage CBR obtained for soil alone without any coir geotextiles. The experimental results give a clear indication that the presence of coir geotextiles influences the California Bearing Ratio (CBR) of the soil. The improvement in strength of soil due to the placement of coir geotextiles is a function of interaction of coir geotextiles with the soil. It was observed that there exists interaction between soil and coir geotextile in soaked and unsoaked condition.

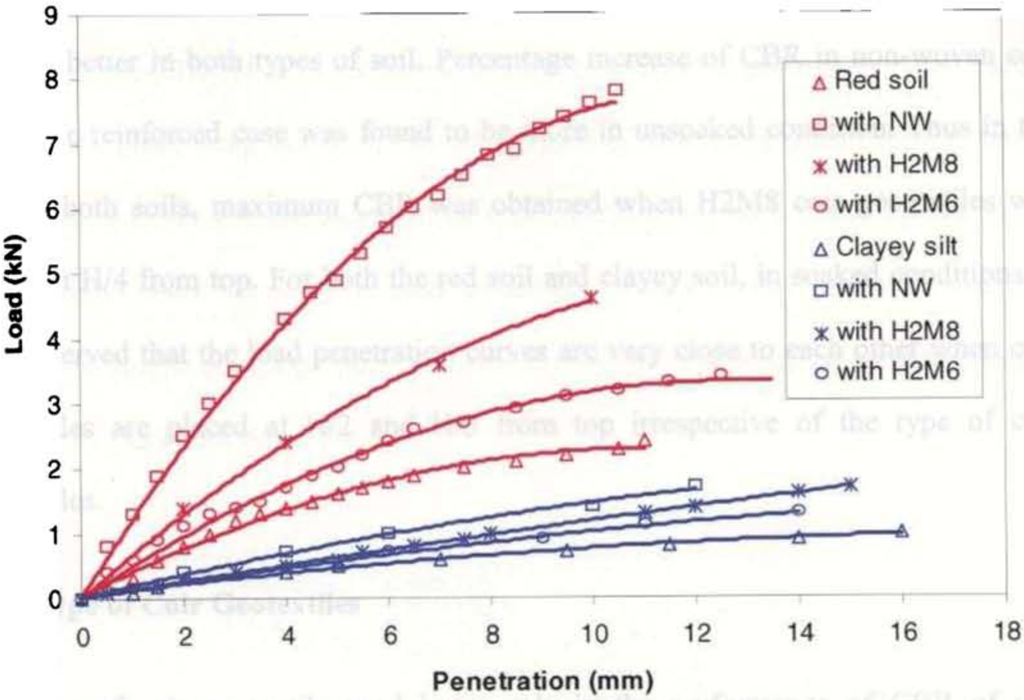
7.3.2 Type of Soil

Fig.7.2 shows the load penetration curves for unsoaked and soaked conditions for three types of coir geotextiles (H2M8, H2M6 and NW) placed at a depth of H/4 from top surface for two types of soils tested. It can be observed that for both the soils tested, the behaviour was identical when coir geotextiles were placed at H/4 from top in the soil mass and tested in soaked and unsoaked conditions. Penetration of the plunger was more for soil without geotextile in all cases and CBR values were less in soaked condition. Between the two soils, it was found that the percentage increase in

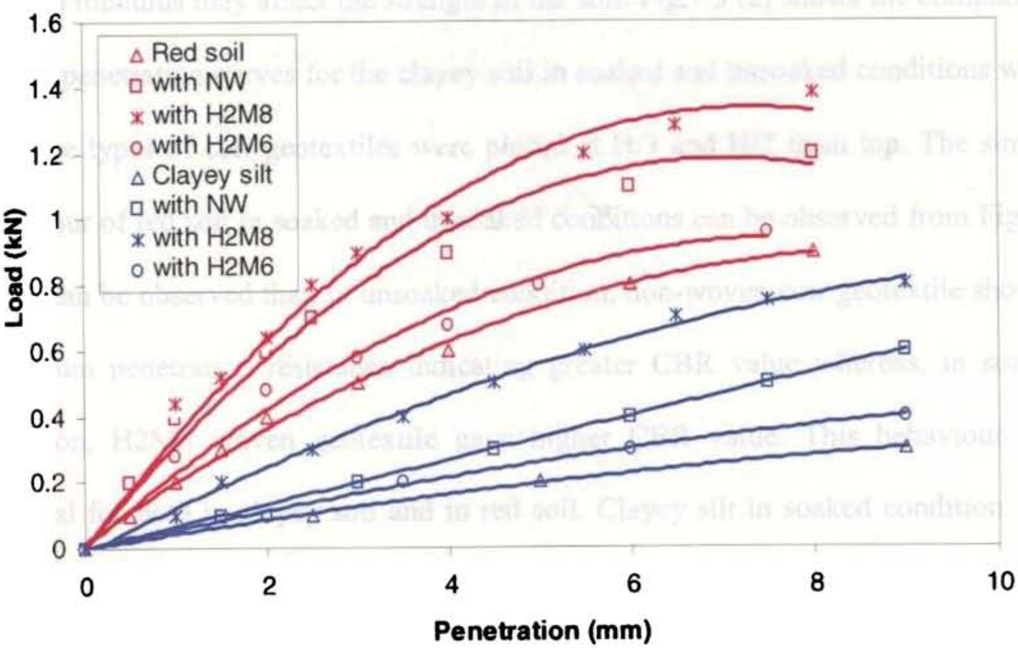
CBR for red soil in unsoaked condition was higher than that for clayey silt. On the contrary, it can be observed that, the coir geotextiles perform much better in clayey soil in soaked condition, the corresponding percentage increase being 100%, 250% and 43% for clayey silt and 36%, 50% and 9% for red soil respectively for Non-woven, H2M8 and H2M6 coir geotextiles.

Table 7.2 Summary of CBR test results

1. Clayey Silt Unsoaked: Initial CBR = 2.43%			
Depth of placement from top	Type of Coir Geotextile		
	Non-woven	Woven H2M8	Woven H2M6
H/4	4.25	3.54	3.04
H/3	3.72	3.04	2.89
H/2	3.08	2.93	2.78
2. Clayey Silt Soaked: Initial CBR = 0.95%			
Depth of placement from top	Type of Coir Geotextile		
	Non-woven	Woven H2M8	Woven H2M6
H/4	1.90	3.35	1.36
H/3	1.56	2.74	1.08
H/2	1.44	2.58	1.06
3. Red Soil Unsoaked: Initial CBR = 7.98%			
Depth of placement from top	Type of Coir Geotextile		
	Non-woven	Woven H2M8	Woven H2M6
H/4	25.09	14.52	10.27
H/3	14.83	10.65	9.31
H/2	9.89	9.50	8.67
4. Red Soil Soaked: Initial CBR = 3.35%			
Depth of placement from top	Type of Coir Geotextile		
	Non-woven	Woven H2M8	Woven H2M6
H/4	4.56	5.02	3.65
H/3	4.11	4.26	3.50
H/2	3.61	3.88	3.42



(a) Unsoaked condition



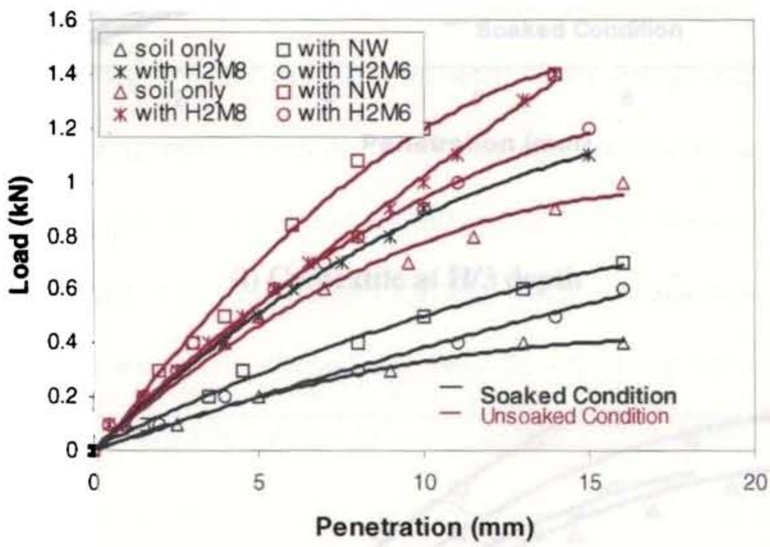
(b) Soaked condition

Fig. 7.2 Load - penetration curves with coir geotextiles placed at H/4 from top

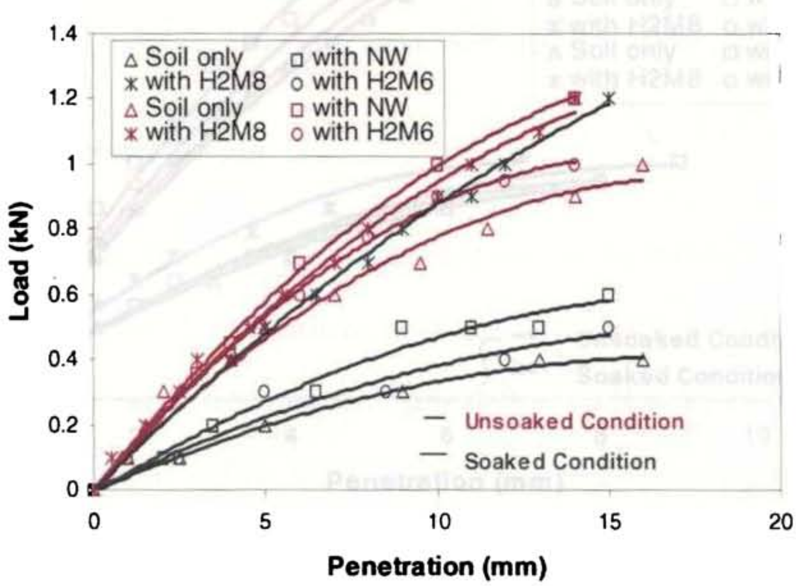
Considering different types of coir geotextiles, H2M8 and Non-woven geotextile perform better in both types of soil. Percentage increase of CBR in non-woven coir geotextile reinforced case was found to be more in unsoaked condition. Thus in the case of both soils, maximum CBR was obtained when H2M8 coir geotextiles was placed at H/4 from top. For both the red soil and clayey soil, in soaked conditions, it was observed that the load penetration curves are very close to each other when coir geotextiles are placed at H/2 and H/3 from top irrespective of the type of coir geotextiles.

7.3.3 Type of Coir Geotextiles

The type of coir geotextile used has a role in the performance of CBR of coir geotextile reinforced soil. The properties of coir geotextile viz,. its mass density, mesh size and modulus may affect the strength of the soil. Fig.7.3 (a) shows the comparison of load penetration curves for the clayey soil in soaked and unsoaked conditions when the three types of coir geotextiles were placed at H/3 and H/2 from top. The similar behaviour of red soil in soaked and unsoaked conditions can be observed from Fig.7.3 (b). It can be observed that, in unsoaked condition, non-woven coir geotextile showed maximum penetration resistance indicating greater CBR value whereas, in soaked condition, H2M8 woven geotextile gave higher CBR value. This behaviour was identical for both in clayey soil and in red soil. Clayey silt in soaked condition with geotextiles at H/4 from top, the CBR values obtained were 1.9% and 3.35% for NW and H2M8 respectively. But for the same soil in un-soaked condition, the CBR values were respectively 4.25% and 3.54% for NW and H2M8. Similarly, for red soil, the CBR values with geotextile at H/4 from top were 4.56% and 5.02% for NW and H2M8 respectively in soaked condition and were 25.09% and 14.52% respectively in un-soaked condition. For all placement depth this behaviour was similar. In all cases it could be seen that the performance of H2M6 in terms of improving the CBR values, is only marginal when compared to other types of coir geotextiles. Thus it can be concluded that for the enhancement of CBR, H2M8 will be a better option, as the CBR values were more in soaked condition, which is the condition normally considered for the design purpose. Non-woven coir geotextile can be considered when more or less dry conditions prevail.

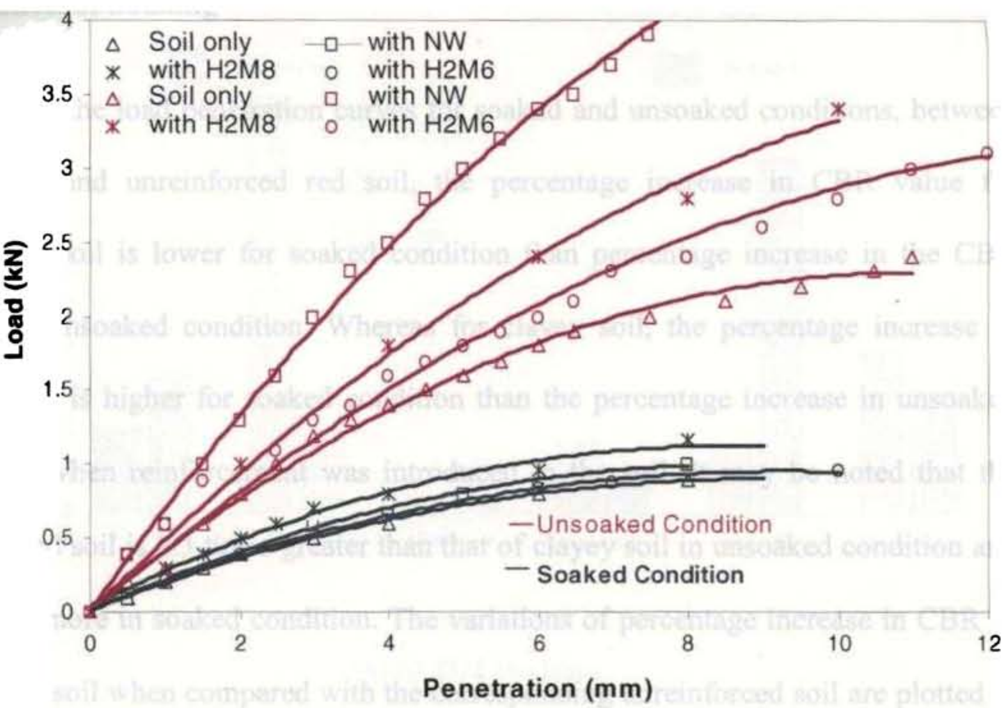


(i) Geotextile at H/3 depth

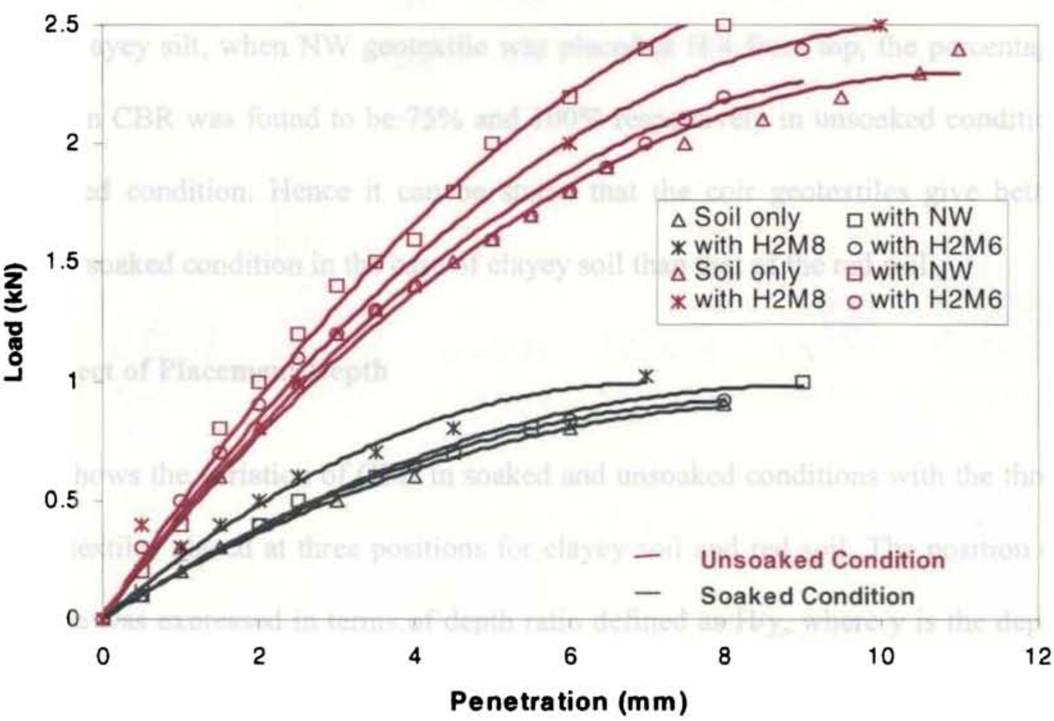


(ii) Geotextile at H/2 depth

Fig. 7.3 (a) Effect of type of coir geotextile for clayey silt



(i) Geotextile at H/3 depth



(ii) Geotextile at H/2 depth

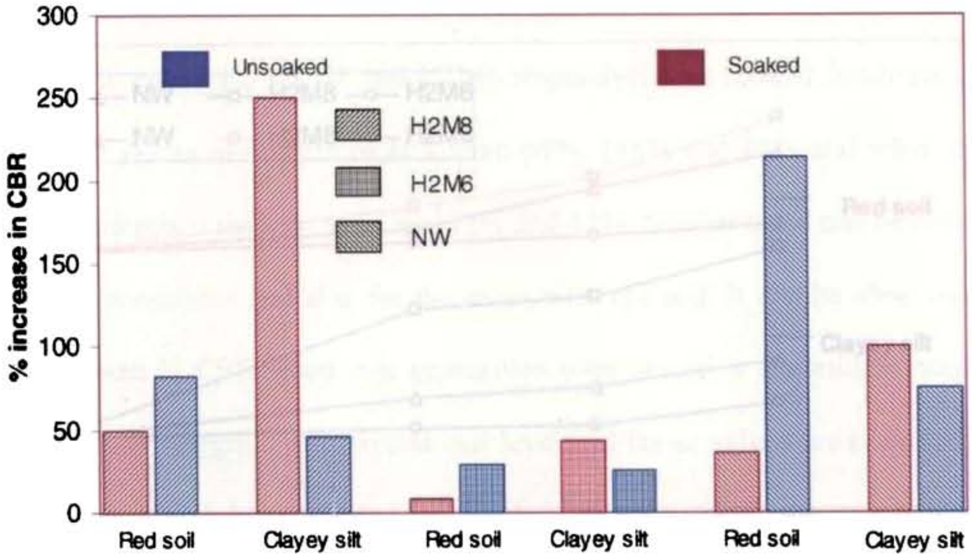
Fig. 7.3 (b) Effect of type of coir geotextile for red soil

7.3.4 Effect of Soaking

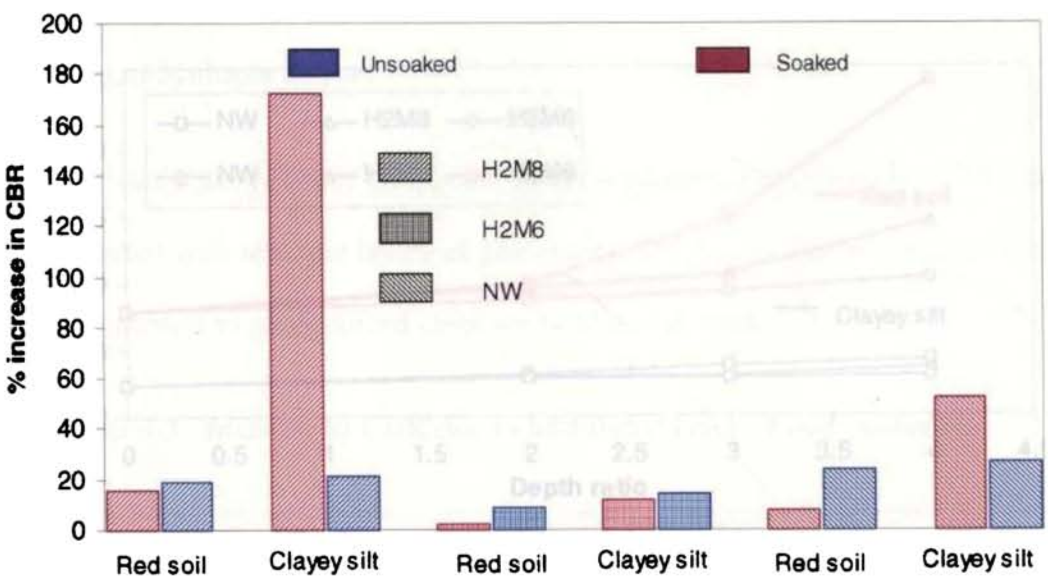
Comparing the load penetration curves for soaked and unsoaked conditions, between reinforced and unreinforced red soil, the percentage increase in CBR value for reinforced soil is lower for soaked condition than percentage increase in the CBR value for unsoaked condition. Whereas for clayey soil, the percentage increase in CBR value is higher for soaked condition than the percentage increase in unsoaked condition when reinforcement was introduced in the soil. It may be noted that the CBR of red soil is 3.3 times greater than that of clayey soil in unsoaked condition and 3.5 times more in soaked condition. The variations of percentage increase in CBR of reinforced soil when compared with the corresponding unreinforced soil are plotted in a chart form in Fig. 7.4. It can be observed that, in the case of red soil, when NW coir geotextile was placed at H/4 depth from the top, the percentage increase in CBR in unsoaked condition was 214% whereas this value in soaked condition was only 36%. But for clayey silt, when NW geotextile was placed at H/4 from top, the percentage increase in CBR was found to be 75% and 100% respectively in unsoaked condition and soaked condition. Hence it can be stated that the coir geotextiles give better results in soaked condition in the case of clayey soil than that of the red soil.

7.3.5 Effect of Placement Depth

Fig.7.5 shows the variation of CBR in soaked and unsoaked conditions with the three coir geotextiles placed at three positions for clayey soil and red soil. The position of geotextile was expressed in terms of depth ratio defined as H/y, where y is the depth of reinforcement from the surface and H is the total depth of the sample in the CBR mould, denoting the depth ratio as zero for unreinforced case for the purpose of comparison. It can be clearly seen from the graph that due to the placement of coir

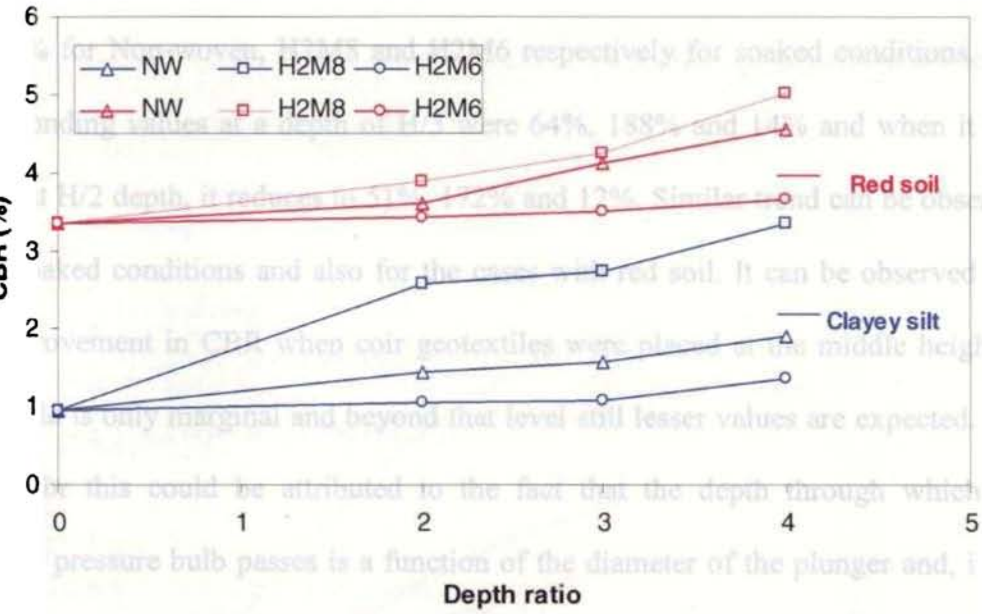


(a) @ H/4 Position

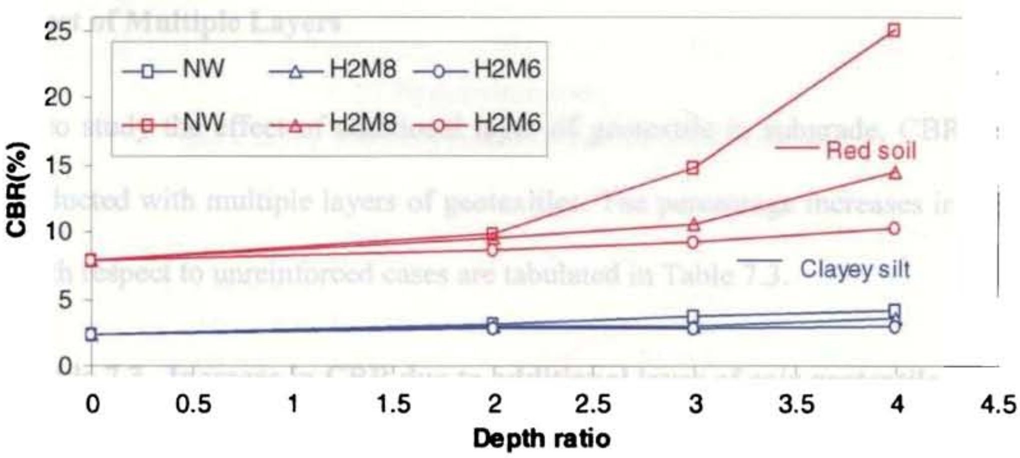


(b) @ H/2 Position

Fig. 7.4 Effect of soaking on geotextiles placed at H/4 and H/2 from top



(a) Soaked CBR



(b) Unsoaked CBR

Fig. 7.5 Effect of placement depth of coir geotextiles

geotextiles, the CBR value is increased irrespective of type of coir geotextiles and placement depth, but the quantum of increase depends on type of coir geotextile and placement depth. It is observed that, though the CBR values were increased in all cases, the percentage increase was found to be much higher when Non-woven coir geotextile was placed in the upper one-third region. For example, when coir geotextile was placed at H/4 depth, the percentage increase in CBR values were 100%, 252% and 43% for Non-woven, H2M8 and H2M6 respectively for soaked conditions. The corresponding values at a depth of H/3 were 64%, 188% and 14% and when it was placed at H/2 depth, it reduces to 51%, 172% and 12%. Similar trend can be observed for unsoaked conditions and also for the cases with red soil. It can be observed that the improvement in CBR when coir geotextiles were placed at the middle height of the mould is only marginal and beyond that level still lesser values are expected. The reason for this could be attributed to the fact that the depth through which the effective pressure bulb passes is a function of the diameter of the plunger and, if the geotextile is inserted at depths greater than the depth of pressure bulb, no significant improvement can be witnessed.

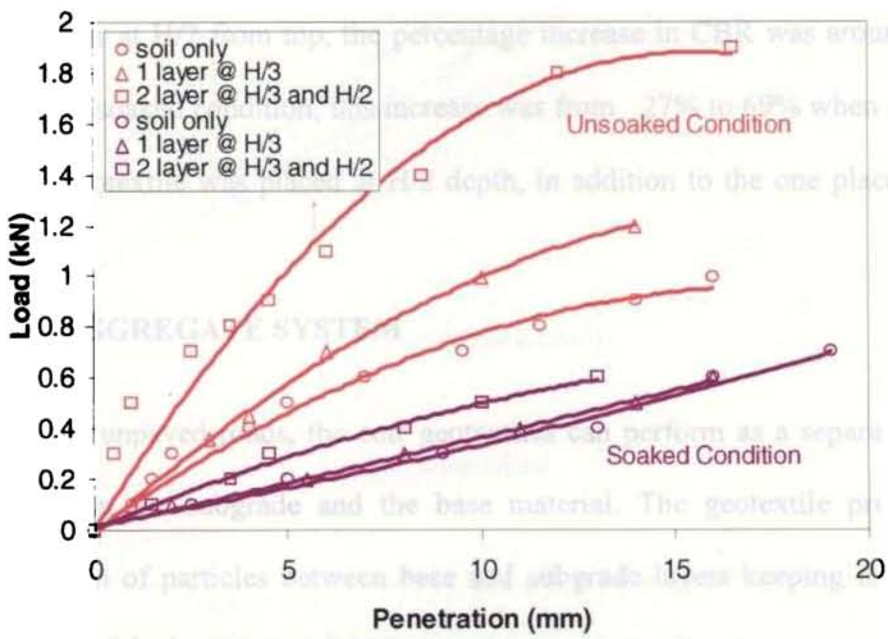
7.3.6 Effect of Multiple Layers

In order to study the effect of additional layer of geotextile in subgrade, CBR tests were conducted with multiple layers of geotextiles. The percentage increases in CBR values with respect to unreinforced cases are tabulated in Table 7.3.

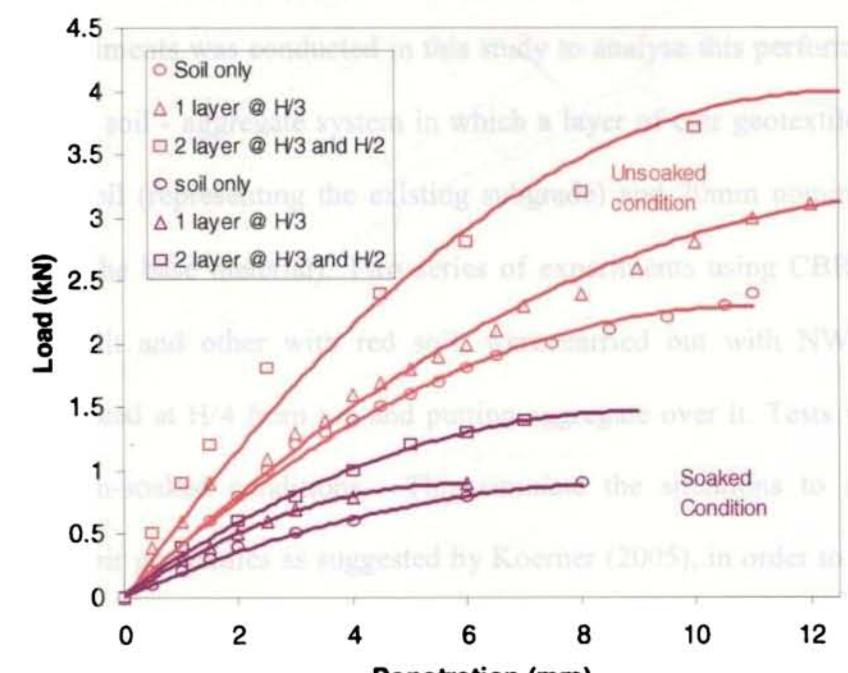
Table 7.3 Increase in CBR due to additional layer of coir geotextile

Soil	Type of geotextile	CBR (%)				
		Without geotextile	With 1 layer at H/3	Percentage increase	With additional layer at H/2	Percentage increase
Clayey silt Soaked	NW	0.95	1.56	64	1.98	108
Clayey silt Un-soaked	NW	2.43	3.72	53	5.25	116
Clayey silt Soaked	H2M8	0.95	2.74	188	3.42	260
Red Un-soaked	NW	7.98	14.83	86	19.80	148
Red Un-soaked	H2M8	7.98	10.65	33	12.74	59
Red Soaked	H2M8	3.35	4.26	27	5.84	69

The effects of multiple layers of coir geotextiles were studied in few cases. The variations in CBR values due to the provision of additional reinforcement are shown in Fig. 7.6 for red soil and clayey silt.



(a) Clayey silt with non-woven coir geotextile



(b) Red soil with H2M8 coir geotextile

Fig. 7.6 Effect of Multiple layers of coir Geotextiles

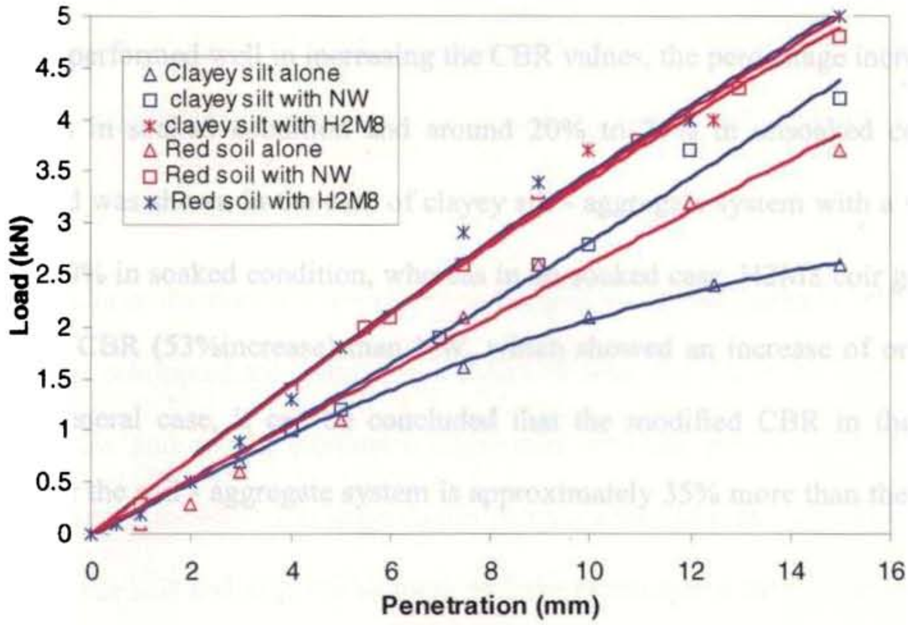
It can be seen that by providing one additional layer of geotextile the increase in CBR was roughly doubled. For instance, the percentage increase in CBR when Non-woven coir geotextile was placed at a depth of H/3 from top in clayey silt was 64%. With one additional layer at H/2 from top, the percentage increase in CBR was around 108%. For red soil in soaked condition, this increase was from 27% to 69% when a layer of H2M8 coir geotextile was placed at H/2 depth, in addition to the one placed at H/3 location.

7.4 SOIL - AGGREGATE SYSTEM

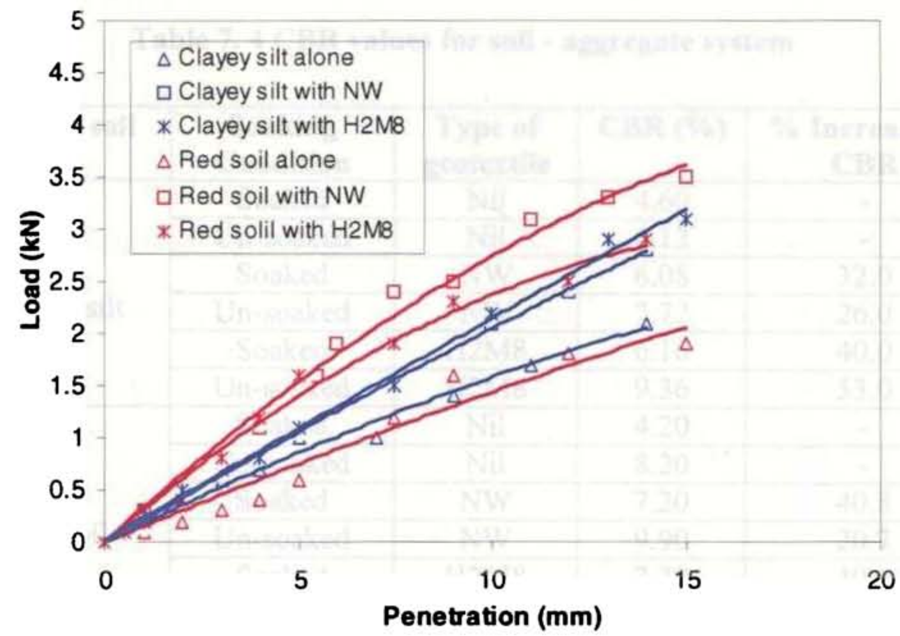
In the case of unpaved roads, the coir geotextiles can perform as a separator if it is placed between the subgrade and the base material. The geotextile prevents the interpenetration of particles between base and subgrade layers keeping the strength and thickness of the base course intact.

A set of experiments was conducted in this study to analyse this performance of coir geotextile in a soil - aggregate system in which a layer of coir geotextile was placed between the soil (representing the existing subgrade) and 20mm nominal aggregate (representing the base material). Two series of experiments using CBR mould, one with clayey silt and other with red soil, were carried out with NW and H2M8 geotextiles placed at H/4 from top and putting aggregate over it. Tests were done in soaked and un-soaked conditions. This simulate the situations to arrive at the influence of coir geotextiles as suggested by Koerner (2005), in order to calculate the thickness of pavement by US Army Corps Method.

Experimental results on soil - aggregate system have shown definite increase in strength in terms of CBR value. Fig. 7.7 shows load penetration curves for red soil -



(a) Unsoaked



(b) Soaked

Fig. 7.7 Effect of coir geotextiles in soil - aggregate system

aggregate system and clayey silt - aggregate system with NW and H2M8 coir geotextile placed at interface in soaked and un-soaked conditions. From the figure it can be seen that in the case of red soil - aggregate system, both NW and H2M8 geotextiles, performed well in increasing the CBR values, the percentage increase was around 41% in soaked condition and around 20% to 25% in unsoaked condition. Similar trend was shown in the case of clayey silt - aggregate system with a variation of 32% to 40% in soaked condition, whereas in un-soaked case, H2M8 coir geotextile gave higher CBR (53%increase) than NW, which showed an increase of only 26%. Still as a general case, it can be concluded that the modified CBR in the soaked condition for the soil - aggregate system is approximately 35% more than the original CBR.

CBR values and percentage increase in CBR in relation to control section CBR were calculated. These values are summarised in Table 7. 4.

Table 7. 4 CBR values for soil - aggregate system

Type of soil	Soaking Condition	Type of geotextile	CBR (%)	% Increase in CBR
Clayey silt	Soaked	Nil	4.60	-
	Un-soaked	Nil	6.12	-
	Soaked	NW	6.08	32.0
	Un-soaked	NW	7.72	26.0
	Soaked	H2M8	6.16	40.0
	Un-soaked	H2M8	9.36	53.0
Red soil	Soaked	Nil	4.20	-
	Un-soaked	Nil	8.20	-
	Soaked	NW	7.20	40.8
	Un-soaked	NW	9.90	20.7
	Soaked	H2M8	7.20	40.8
	Un-soaked	H2M8	10.0	22.0

7.5 CBR PREDICTION MODEL

7.5.1 General

In India the thickness of flexible pavement is designed on the basis of projected number of standard axle loads during the design life, which is obtained using the current commercial vehicles per day (CVPD) and its growth rate, along with the subgrade strength in terms of CBR. In rural roads, the top 30cm of the cutting or embankment at the formation level shall be considered as depth of subgrade (IRC: SP: 20-2002).

From the results of the detailed laboratory investigations on the strength behaviour of coir geotextile reinforced subgrade soil, it could be seen that the strength in terms of the CBR of the soil would increase considerably with the positions of geotextile. Also, the strength mobilisation depends on many factors like the inherent strength properties of the soil and also the strength and the placement depth of the geotextile reinforcement. Hence it is advantageous to develop a model to predict the modified CBR of the reinforced subgrade, which would respond to the changes in the properties of the soil and that of the reinforcement.

Modified CBR = ƒ (Soil characteristics, Reinforcement characteristics, Placement depth) ----------(7.2)

This can be achieved through the principles of multiple linear regression analysis. The model thus developed can be effectively used for the design of coir reinforced unpaved roads. The details of design procedure are given in Section 10.3.

7.5.2 Multiple Linear Regression Analysis (Johnson, 2001)

Multiple linear regression analysis is a statistical technique frequently used to develop prediction equations to establish the relationship for a variable, which is known to respond to changes in two or more other variables. The variable, which is known to respond (κ variable) is commonly called the dependent variable and the other variables influencing it are called the independent variables (x variables). The function will be of the following form:

where, x_1, x_2, x_3-----x_m = m independent variables,

a_0 = regression constant, and

a_1,a_2,---a_m = regression coefficients of the m independent variables.

The regression coefficients are determined from a given set (n) of observed values of κ and x_1,x_2---,x_m by the method of least squares.

The analysis of variance approach is used to test the predictor equation. The total sum of squares of deviations of the 'n' observations from the mean is a measure of the degree to which the 'n' observations are spread around their average value. Smaller the standard error, the better will be the model.

Coefficient of determination (R^2) is another indicator of the strength of relationship. It is the ratio of regression sum of squares to total sum of squares. R^2 lies between 0 and 1. The closer it is to 1, the better is the equation.

The F ratio gives another indication of the adequacy of the equation. It is the ratio of mean sum of regression squares to mean sum of residual squares. If the F ratio obtained is greater than the values given in the F tables, then the model is significant.

7.5.3 Development of Model

The most important property of the subgrade soil considered for the design of pavement is the CBR value. Hence the original CBR for the remoulded soil at OMC and MDD is taken as a property representing the strength of the soil. The different properties of coir reinforcement considered are those which can be evaluated in the laboratory or supplied by the manufacturers viz., mass / unit area, puncture resistance, strip tensile strength, wide tensile strength, secant modulus and mesh size (picks/dm and ends/dm). CBR tests were conducted by placing coir geotextiles at three depths - H/2, H/3 and H/4, where H is the height of the specimen for CBR test and these heights were taken as the placement characteristics of the geotextile.

Multiple Linear regression analysis was done using SPSS software inputting the laboratory test data obtained. The dependent variable is the modified CBR. The independent variables considered for regression analysis were:

i) Original CBR of the soil,

ii) Mass per unit area of the geotextile,

iii) Puncture resistance of geotextile,

iv) Strip tensile strength in machine direction of the geotextile,

v) Wide tensile strength in machine direction of the geotextile,

vi) Secant modulus of the geotextile,

vii) Picks/dm of the geotextile,

viii) Ends/dm of the geotextile, and

ix) Depth of placement of reinforcement (H/2, H/3 and H/4).

Analyses were done using CBR data separately for soaked and unsoaked conditions and also putting together. During analysis some of the variables representing the properties of geotextiles were found insignificant. The equations for modified CBR values (CBR_M) obtained for different conditions are given below in which,

CBR_M = Modified CBR in % (reinforced condition),

CBR = Original CBR in % (unreinforced condition),

D = Depth of coir geotextiles from surface,

(H/2 =6.25cm, H/3 = 4.17cm and H/4 = 3.125cm)

σ_S = Strip Tensile Strength of Coir geotextile in kN/m,

σ_W = Wide width tensile strength of Coir geotextile in kN/m, and

Es = Secant Modulus of Coir Geotextile in kN.

σ_S, Es and σ_W being independent variables, are selected as significant variables in the analysis independently, which give rise to the following three sets of equations in each case of analysis:

i) Both soaked and unsoaked CBR data

a) $CBR_M = 0.550 + 1.249\ CBR - 0.363\ D + 0.156\ \sigma_S$ -------------------(7.4)

$(R^2 = 0.936)$

b) $CBR_M = 0.916 + 1.249\ CBR - 0.363\ D + 0.0255\ E_S$ ----------------(7.5)

$(R^2 = 0.936)$

c) $CBR_M = 0.626 + 1.249\ CBR - 0.363\ D + 0.12\ \sigma_W$ --------------------(7.6)

$(R^2 = 0.936)$

ii) Soaked CBR data only

a) $CBR_M = 0.649 + 0.803\ CBR - 0.178\ D + 0.162\ \sigma_S$ ----------------(7.7)

$(R^2 = 0.944)$

b) $CBR_M = 1.029 + 0.803\ CBR - 0.178\ D + 0.0264\ E_S$ ---------------(7.8)

$(R^2 = 0.944)$

c) $CBR_M = 0.728 + 0.803\ CBR - 0.178\ D + 0.124\ \sigma_W$ ----------------(7.9)

$(R^2 = 0.944)$

iii) Unsoaked CBR data only

a) $CBR_M = 0.930 + 1.342\ CBR - 0.549\ D + 0.150\ \sigma_S$ -------------------(7.10)

$(R^2 = 0.937)$

b) $CBR_M = 1.284 + 1.342\ CBR - 0.549\ D + 0.0246\ E_S$ ---------------(7.11)

$(R^2 = 0.937)$

c) $CBR_M = 1.005 + 1.342\ CBR - 0.549\ D + 0.116\ \sigma_W$ ------------------(7.12)

$(R^2 = 0.937)$

Equation 7.4 uses all data and gave higher F values and hence this equation is recommended for the prediction of the modified CBR for design of coir geotextile reinforced unpaved roads. The linear scatter diagram using this equation is shown in Fig. 7.8, which shows that the deviated points lie on the safer side in terms of design consideration.

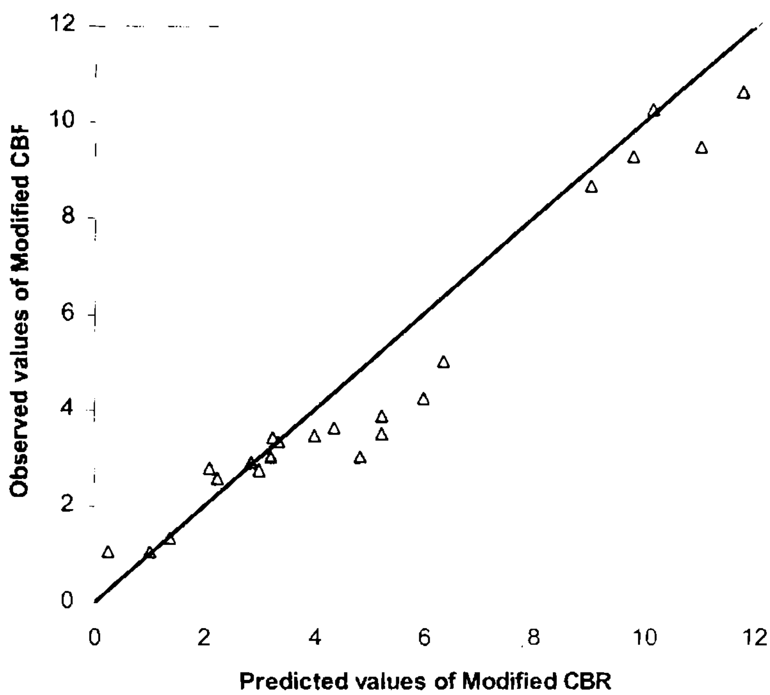


Fig. 7.8 Linear scatter diagram for the CBR prediction model

7.6 SUMMARY

Elaborate experimental studies were carried out to understand the strength behaviour of coir geotextile reinforced subgrade in terms of California Bearing Ratio. The CBR values were found to change with the varying location of reinforcement in the subgrade soil and also with respect to the type of soil and type of reinforcement. It was observed that beyond H/2 depth, effects of coir geotextiles were nominal. Effects of multiple layers of coir geotextile reinforcements were also studied and found that with an additional layer, the percentage increase in CBR was very high.

Depending on the type of soil, type of coir geotextile and soaking conditions, the percentage increase in CBR varies. In general, while it can be stated that H2M8 and non-woven coir geotextiles perform better, the performance of latter being found to be better in unsoaked clayey silt soil. In soaked condition, it was found that H2M8 performs better than Non-woven.

With the help of a large number of experimental results, an equation for modified CBR was formulated, which correlated the properties of soil, properties of geotextiles and placement depth of coir geotextiles.

CHAPTER 8

EFFECT OF COIR GEOTEXTILES ON BEARING CAPACITY

8.1 GENERAL

One of the important functions of geotextiles is to increase the bearing capacity of the soil. The reinforcement effect is achieved either by micro reinforcement or by macro reinforcement. In the former, the reinforcement of the soil is achieved by mixing it with randomly oriented fibres. This aspect has been discussed in detail in Chapter 12. Macro reinforcement on the other hand, consists of placing reinforcing elements such as strips, bars, sheets, grids, cells, etc. in the soil. This can be placed in single layer or in multiple layers. In many cases roads and embankments are subjected to submergence and the soil will be in saturated conditions. To know the effectiveness of coir geotextiles in such situations it is necessary to carry out experimental studies in saturated conditions also. In the present investigation the possibility of utilizing coir geotextiles as reinforcement is explored by conducting studies on square footings resting on coir geotextile reinforced sand beds, both in dry and saturated conditions.

8.2 BACKGROUND

Consider a square footing of size B x B resting on coir geotextile reinforced sand bed, which is subjected to an intensity of loading 'q' as shown in Fig. 8.1. The depth of the sand bed is 'Z' and the coir geotextile is placed at a depth 'z' below the footing. The ultimate bearing capacity of foundation can be given by the following equation (Vesic, 1973)

$$q_u = 0.5\gamma B N_\gamma S_\gamma \qquad \text{---------------------- (8.1)}$$

where, γ is the unit weight of soil, N_γ is the bearing capacity factor and S_γ is the shape factor, which may be expressed as

$$S_\gamma = 1 - 0.4\left(\frac{B}{L}\right) \qquad \text{--------------------- (8.2)}$$

Typical nature of settlement curves for unreinforced and reinforced cases is shown in Fig. 8.2

The improvement in ultimate bearing capacity of a foundation due to soil reinforcement is generally expressed in a non-dimensional form called Bearing Capacity Ratio, BCR_u (Binquet and Lee, 1975) defined as

$$BCR_u = q_{u(R)} / q_u \qquad \text{--------------------- (8.3)}$$

where, $q_{u(R)}$ is the ultimate bearing capacity with soil reinforcement and q_u is the ultimate bearing capacity without reinforcement. In practice, most of the shallow foundations are designed for limited settlement. Hence it is essential to determine the BCR at various levels of settlement. Referring to Fig.8.2, the BCR at a settlement level $S \leq S_u$ can be defined as

$$BCR_s = q_{(R)} / q \qquad \text{------------------- (8.4)}$$

where, $q_{(R)}$ is the load per unit area of foundation for reinforced case and q is the load per unit area for unreinforced soil at a settlement level S.

Eight series of experiments comprising of two plate load tests on unreinforced soil and 16 plate load tests on coir reinforced soils were carried out. From the observed data, applied pressure versus settlement curves were plotted and presented for each test. Variations of BCR for different coir geotextiles placed at different levels under dry and saturated conditions were studied.

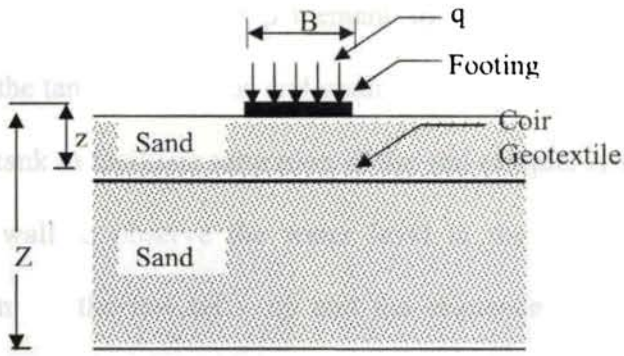


Fig. 8.1 Square footing supported by coir geotextile reinforced sand bed

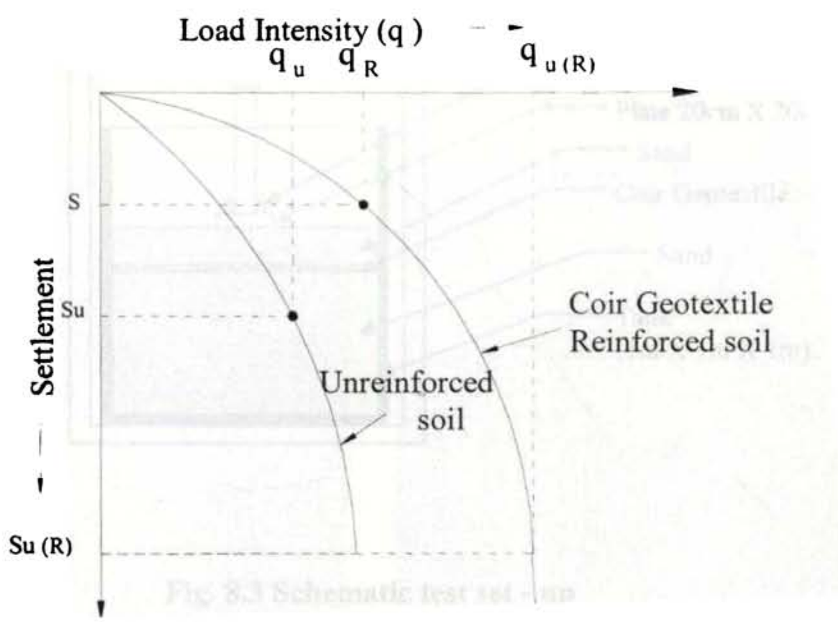


Fig. 8.2 Typical pressure settlement curves for unreinforced and coir geotextile reinforced sand supporting a square footing

8.3 EXPERIMENTAL SET - UP

A series of model footing tests were conducted in a cubical steel tank measuring 1m x 1m x 1m made up of mild steel sheets and angles, one side of which was constructed with perspex sheet. The metallic sidewalls were braced with stiffeners to avoid any possible lateral yielding during the placement of soil and loading of the model footing. Inside of the tank was painted and graduated. Control valves were provided at the bottom of the tank to facilitate saturation of the soil sample. A standpipe was fixed outside the tank wall to observe the water level in the soil Fig. 8.3 shows the schematic diagram for the test set – up and the sequence of the test procedure is shown in Fig. 8.4.

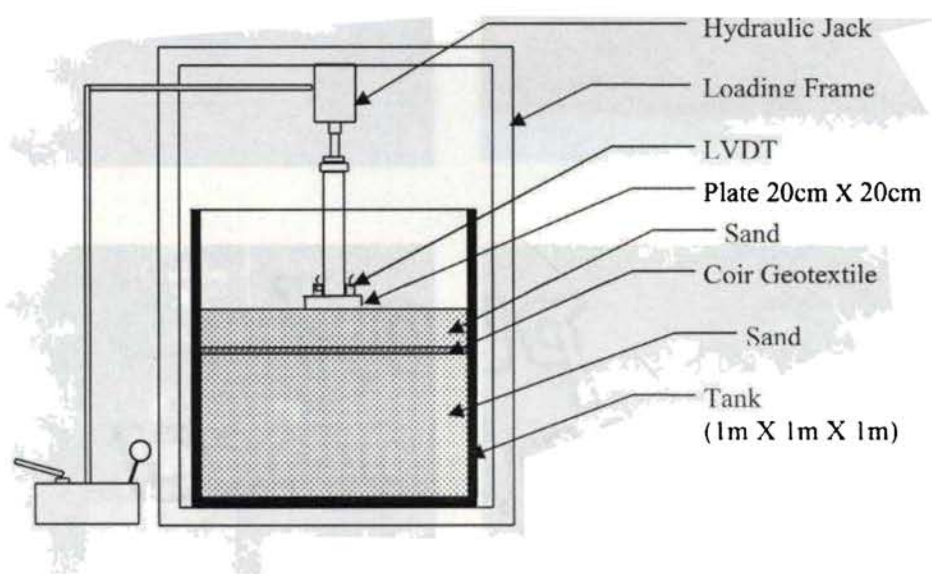


Fig. 8.3 Schematic test set - up

Model footing used was 25.4 mm thick mild steel plate measuring 200mm x 200mm, so that there is minimum dimensional effect. Static loads were applied using 200 kN capacity hydraulic jack, jacked against a reaction frame fabricated using mild steel I-sections. Settlements were measured at four corners of the footing using LVDTs.

8.4 PREPARATION OF TEST BED

The test sand used in this investigation was river sand with coefficient of uniformity of 1.83, coefficient of curvature = 1.34, effective particle size = 300 microns and specific gravity 2.72. All the tests were conducted at a dry unit weight of 15 kN/m^3. Three types of coir geotextiles (H2M6 and H2M8 and Non-woven) were used as reinforcements. Experiments were conducted in dry and saturated conditions. To

Fig. 8.4 Sequence of load test

achieve the desired density, quantity of sand required for 100mm lift was calculated. Sand bed was formed by putting calculated amount of sand in layers and compacted to 100mm thick each. The compaction was done using a wooden mallet to avoid crushing of sand particles. To saturate the soil, water was allowed to flow in the upward direction by operating the control valves provided at the bottom of the tank. Water level in the soil could be seen from the standpipe provided outside the tank wall. Coir geotextiles cut in the form shown in Fig.8.5 was placed at different levels in each case viz. at 100mm, 200mm and 300mm from the top corresponding to z/B ratios 0.5, 1.0 and 1.5 respectively. These z/B values were chosen based on the results of the past studies reported in literature.

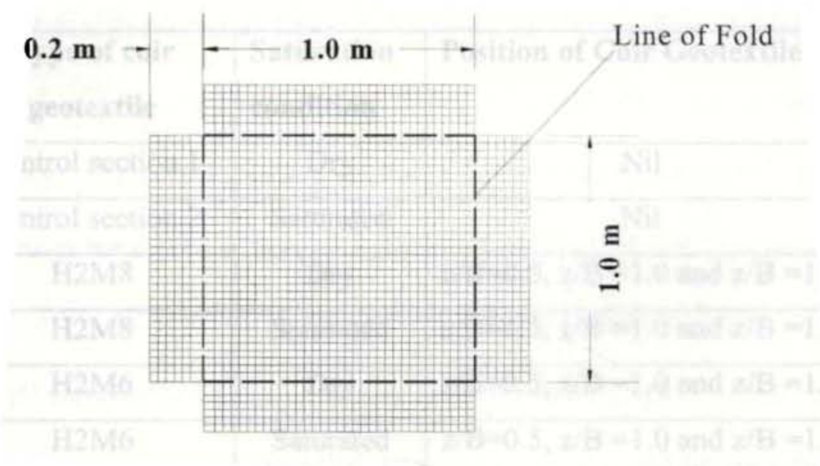


Fig. 8.5 Coir geotextile specimen for plate load test

8.5 TESTING PROCEDURE

After filling the tank to the desired height where coir geotextile is to be placed, the fill surface was levelled and the coir geotextile was placed. Sand was again added in layers to reach the full height. The footing was placed on a predetermined alignment so that the loads from the loading jack would be transferred concentrically to the model footing. Footing settlements were measured through LVDTs placed at four corners of the testing plate. The average settlement corresponding to the four LVDT readings were calculated and was reported as the settlement for each load increment.

The footing was loaded at a constant loading rate until ultimate state was reached. It may be noted that the tests were conducted under stress-controlled conditions, and the post failure behaviour was not recorded. Before starting a new test entire sand in the tank from the previous test was removed and fresh sand was placed in lifts of 100mm. While conducting tests under saturated conditions, the soil bed was saturated and kept for three days before testing. Details of the model tests carried out are given in Table 8.1.

Table 8.1 Details of model tests conducted

Series	Type of coir geotextile	Saturation condition	Position of Coir Geotextile
1	Control section 1	Dry	Nil
2	Control section 2	Saturated	Nil
3	H2M8	Dry	z/B=0.5, z/B =1.0 and z/B =1.5
4	H2M8	Saturated	z/B=0.5, z/B =1.0 and z/B =1.5
5	H2M6	Dry	z/B=0.5, z/B =1.0 and z/B =1.5
6	H2M6	Saturated	z/B=0.5, z/B =1.0 and z/B =1.5
7	Non-woven	Dry	z/B=0.5, z/B =1.0 and z/B =1.5
8	Non-woven	Saturated	z/B=0.5, z/B =1.0 and z/B =1.5

8.6 TEST RESULTS AND DISCUSSION

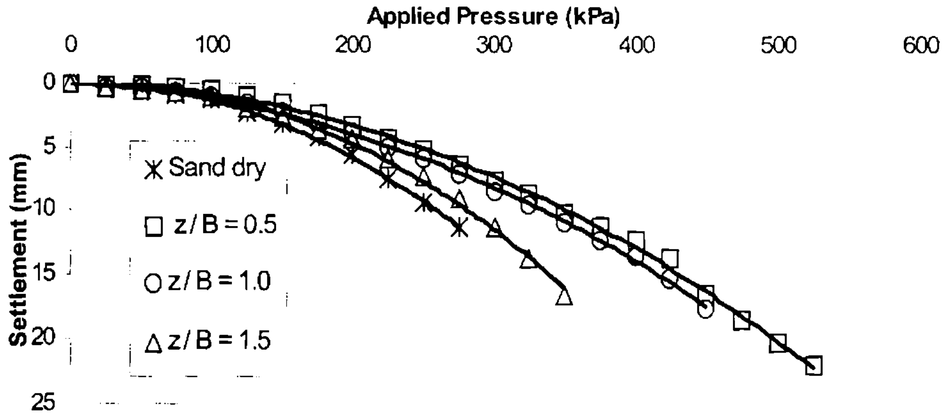
The improvement in bearing capacity due to the provision of coir geotextile reinforcement is represented using a non dimensional parameter, BCR_s which is defined as the ratio of footing pressure with reinforcement at a given settlement (q_R) to the corresponding pressure on the unreinforced soil (q) at the same settlement. The results of experiments using three types of coir geotextiles placed at three embedment levels in dry and saturated conditions are discussed below. In general it was observed that the deformation reduced considerably by the use of coir geotextile both in dry and saturated conditions.

8.6.1 Variation of Bearing Capacity with z/B

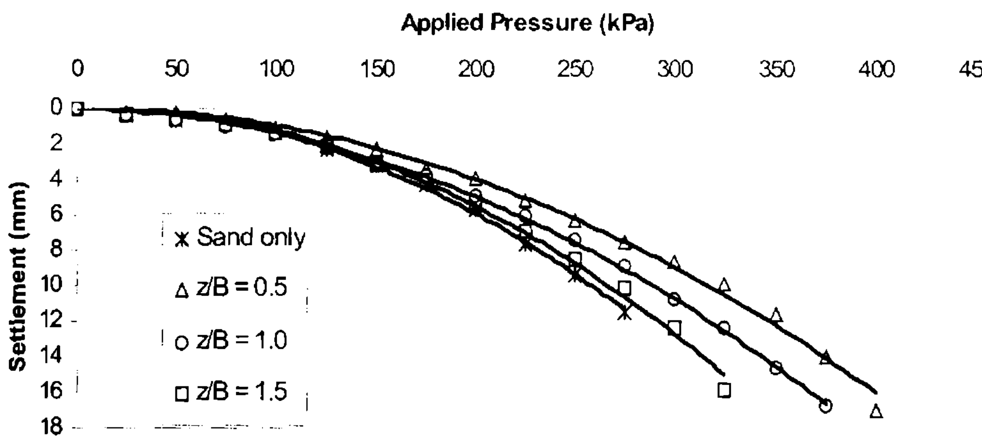
The variations of the applied pressure with settlement of the footing for various values of z/B obtained from the laboratory model tests on coir geotextile reinforced sand bed in dry condition are shown in Fig. 8.6.

It could be observed from Fig. 8.6 that the load carrying capacity of sand reinforced with H2M8 coir geotextile placed at 100mm, 200mm and 300mm from the top (z/B = 0.5, z/B = 1.0 and z/B = 1.5) increased by 36%, 30% and 9% respectively for a settlement of 10mm. The corresponding increases in the bearing capacity with non-woven coir geotextile were 80%, 75% and 32% and those with H2M6 coir geotextile, this increase was 22%, 12% and 3% respectively. In dry condition it was observed that the percentage increase in the load carrying capacity was increased in the order of 22% to 80% for z/B value of 0.5 with different geotextiles. It could be seen from the plots that for tests conducted on unreinforced sand and reinforced sand with z/B ≥ 1, the magnitudes of settlement in reinforced and unreinforced cases were approximately equal. However for z/B < 1, the foundation settlement at the ultimate load was reduced by 1.5 to 2.0 times that obtained from the test on unreinforced soil.

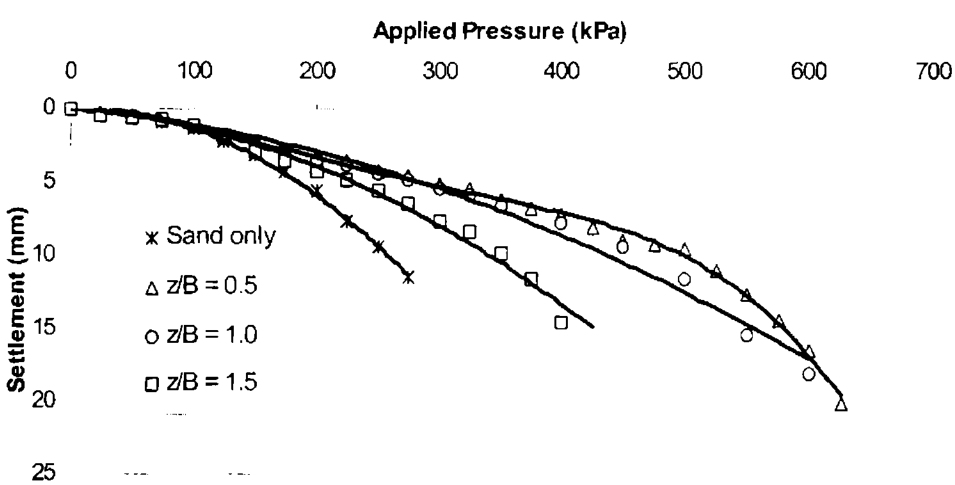
The pressure deformation response of tests conducted with coir geotextiles in saturated condition is shown in Fig. 8.7 for H2M8, H2M6 and NW. The pressure settlement plot for the model foundation supported by unreinforced sand is also shown in these figures. In saturated condition it was observed that the pressure settlement plots were almost coinciding, showing only nominal increase in the load



(a) Using H2M8

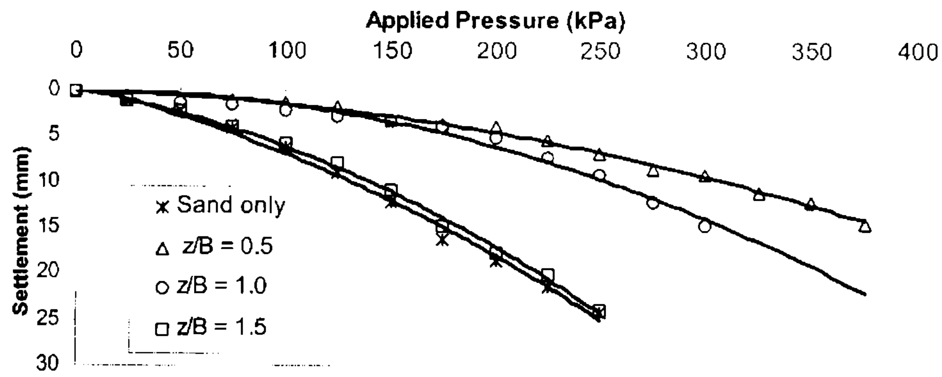


(b) Using H2M6

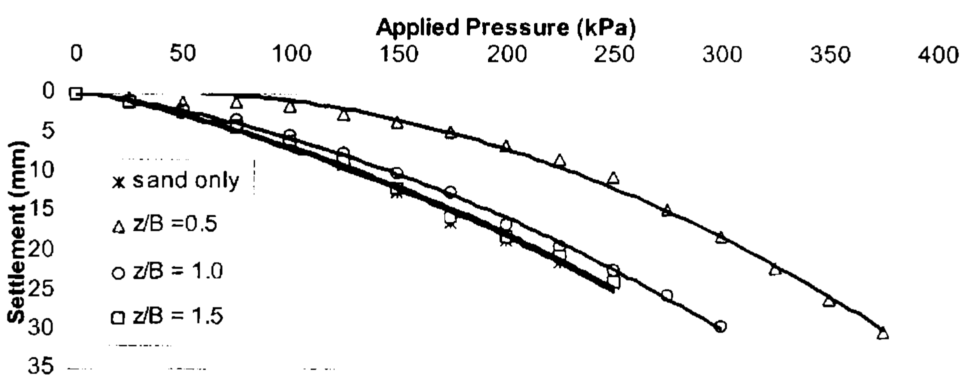


(c) Using Non-woven

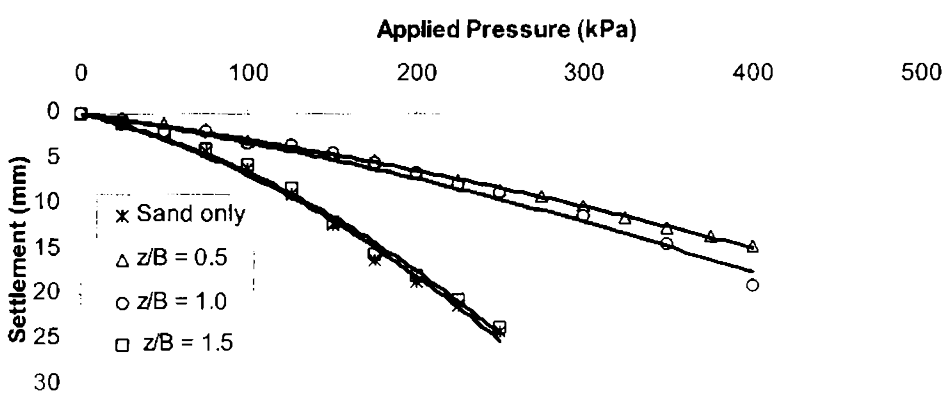
Fig. 8.6 Settlement behaviour of coir geotextile reinforced sand bed in dry condition



(a) Using H2M8



(b) Using H2M6



(c) Using Non-woven

Fig. 8.7 Settlement behaviour of coir geotextile reinforced sand bed in saturated condition

carrying capacity when coir geotextiles were placed at Z/B = 1.5

Fig. 8.8 shows the response curves showing the variation of BCR_s with z/B ratio calculated at settlement levels of 5mm, 10mm and 15mm for dry sand reinforced with H2M8 coir geotextile. It can be observed from the figure that BCR_s values corresponding to 15mm settlement showed higher values compared to those for other settlements. This indicates that reinforcements are more effective in higher strain levels owing to high mobilised friction.

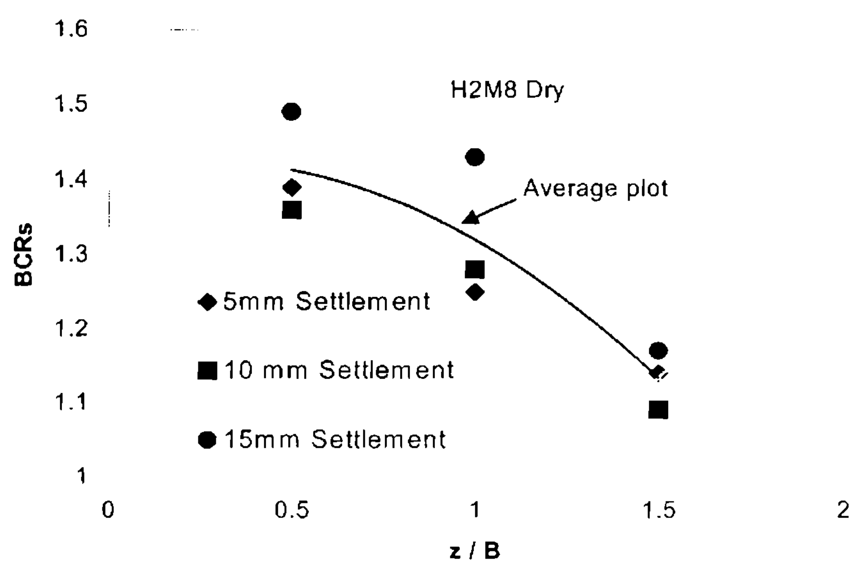


Fig. 8.8 Variation of BCR_s with z/B (H2M8 – dry condition)

It may be noted that a single average curve for the variation of Bearing capacity ratio (BCR) with z/B, irrespective of settlement values, can be plotted which is also shown in Fig. 8.8. Similar curves can be plotted for different coir geotextile reinforced sand bed in dry and saturated conditions (Fig. 8.9) in order to get bearing capacity ratio (BCR) corresponding to any settlement.

The variation of ultimate bearing capacity ratio (BCRu) with z/B is shown in Fig. 8.10. Comparing Fig. 8.9 and Fig. 8.10 it can be noted that the average plot of BCR falls below the plot of BCR_u. This indicates that values taken from the average BCR variation curve for design purpose bears a higher factor of safety.

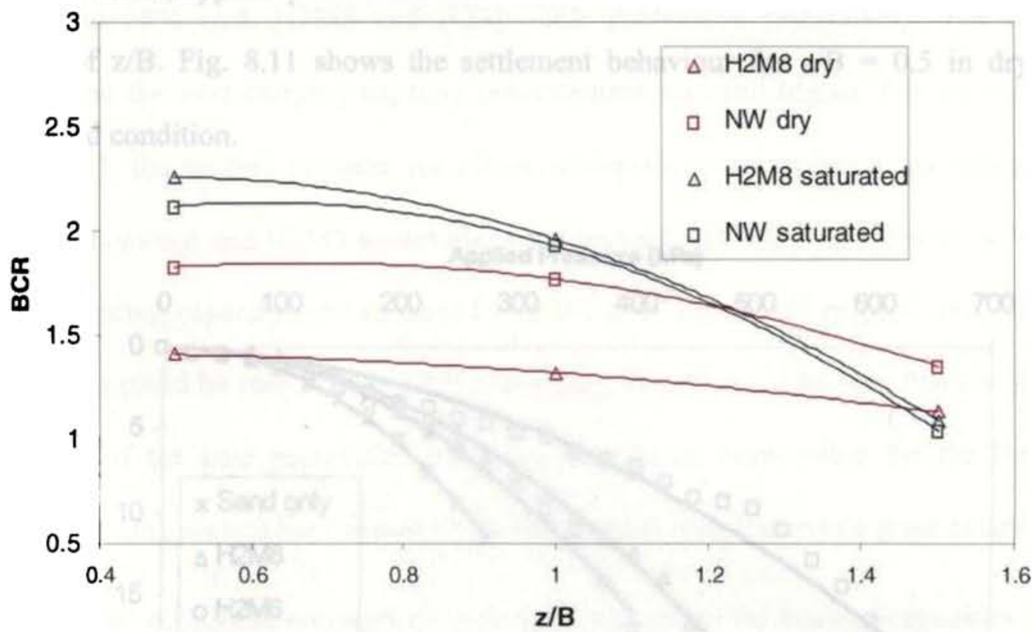


Fig. 8.9 Variation of average BCR with z/B

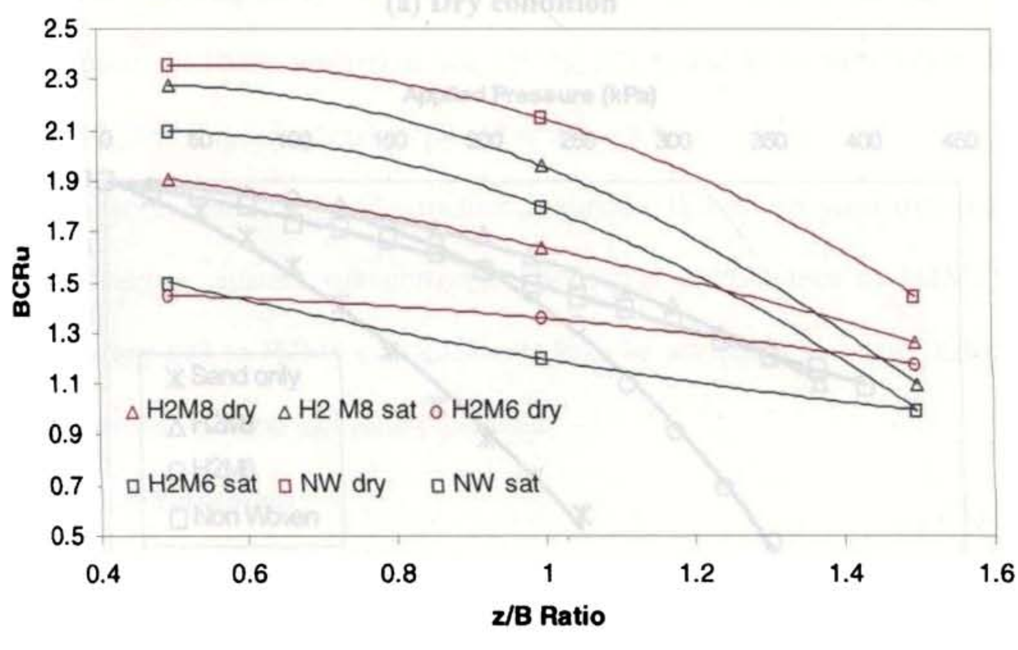
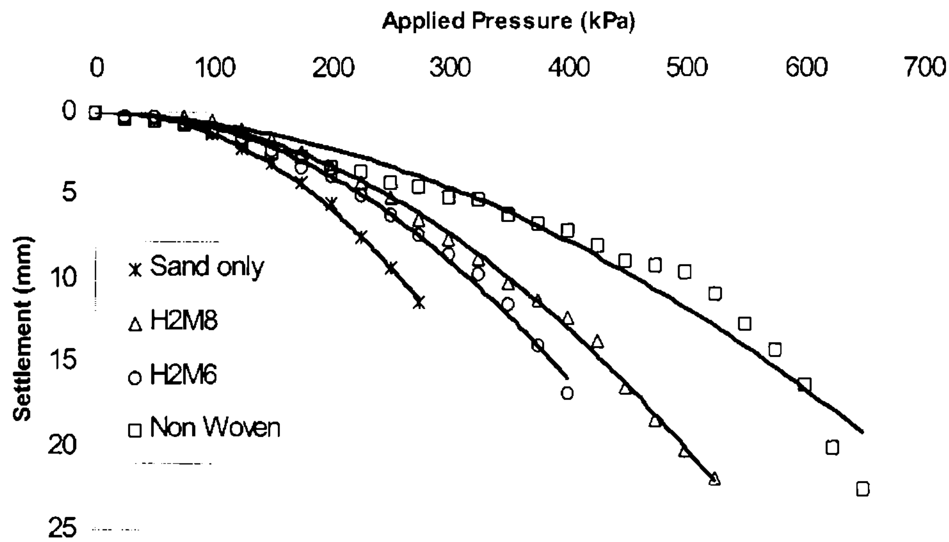


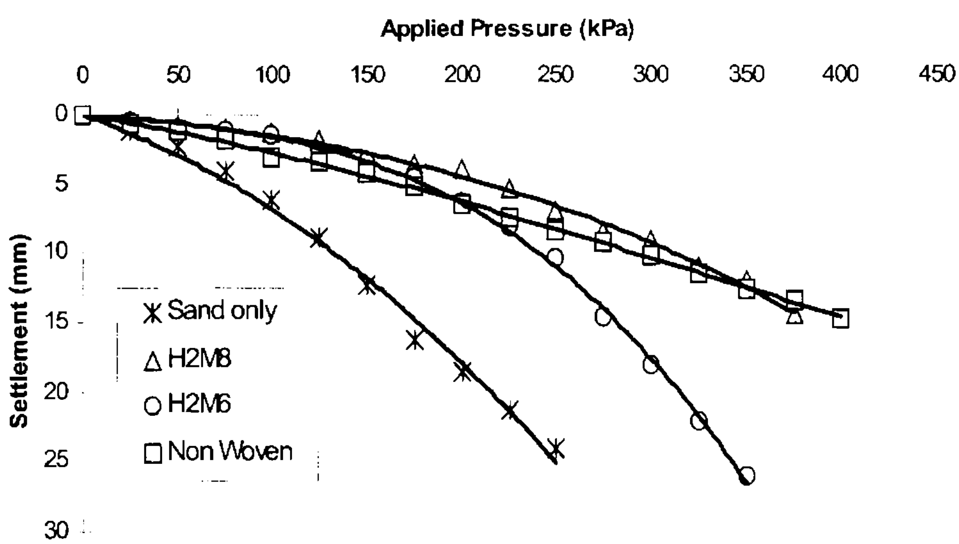
Fig. 8.10 Variation of BCRu with z/B

8.6.2 Effect of Types of Coir Geotextile

In order to understand the beneficial effect of different types of coir geotextile as reinforcement, applied pressure versus settlement graphs were plotted for a specified value of z/B. Fig. 8.11 shows the settlement behaviour for z/B = 0.5 in dry and saturated condition.



(a) Dry condition



(b) Saturated condition

Fig. 8.11 Effect of types of coir geotextile on settlement behaviour (z/B = 0.5)

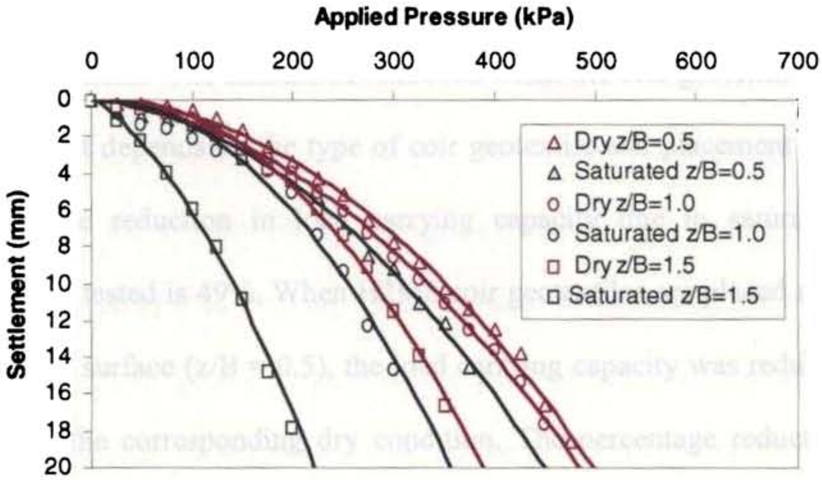
From Fig. 8.11 it could be seen that when coir geotextiles were placed at level corresponding to z/B = 0.5, the applied pressure for 5mm settlement with non-woven coir geotextile was 312.5kPa showing an increase of 74 %. But this increase was only 39% and 25% with H2M8 and H2M6 coir geotextiles respectively. For higher settlement the load carrying capacity enhancement was still higher. For example, at z/B = 0.5, the applied pressure for 10mm settlement to take place when reinforced with Non-woven and H2M8 geotextiles were respectively 452.5 kPa and 350kPa, the corresponding capacity enhancement factor being 1.77 and 1.37 respectively. Similar variations could be seen for other z/B ratios also. Thus it could be seen that the tensile strength of the coir geotextile is not the only factor responsible for the bearing capacity improvement but the interfacial friction also contributes to a great extent.

In saturated condition, corresponding to 5mm settlement the bearing capacities were 210kPa, 177.5kPa and 165kPa respectively with H2M8, Non-woven and H2M6 coir geotextiles placed at z/B = 0.5. The bearing capacity improvements is 162 %, 122 % and 106 % respectively for H2M8, Non-woven and H2M6. Similarly, the increases in bearing capacity at 10mm settlement was 139 %, 123 % and 85 % with H2M8, Non-woven and H2M6 respectively when placed at z/B = 0.5.

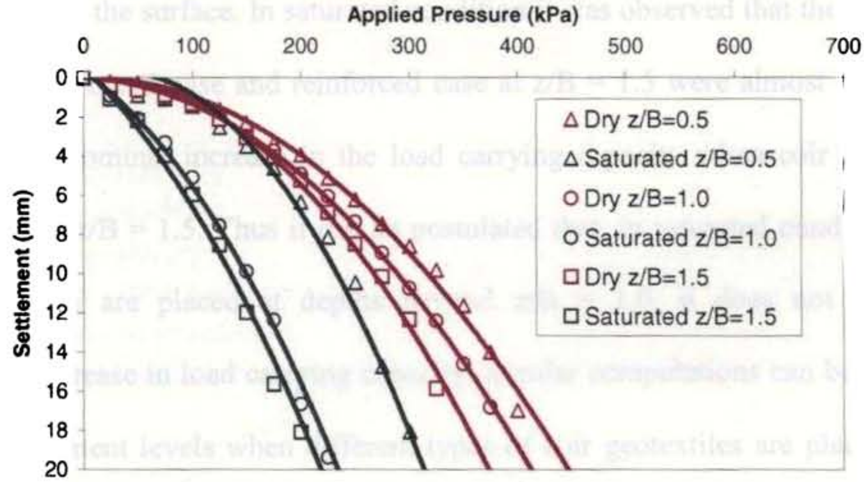
For all z/B ratios, both in dry and saturated conditions, H2M6 coir geotextile showed minimum bearing capacity enhancement. The higher performance of H2M8 coir geotextile compared to H2M6 coir geotextile may be attributed to higher values of stiffness, interface friction and secant modulus.

8.6.3 Effect of Saturation

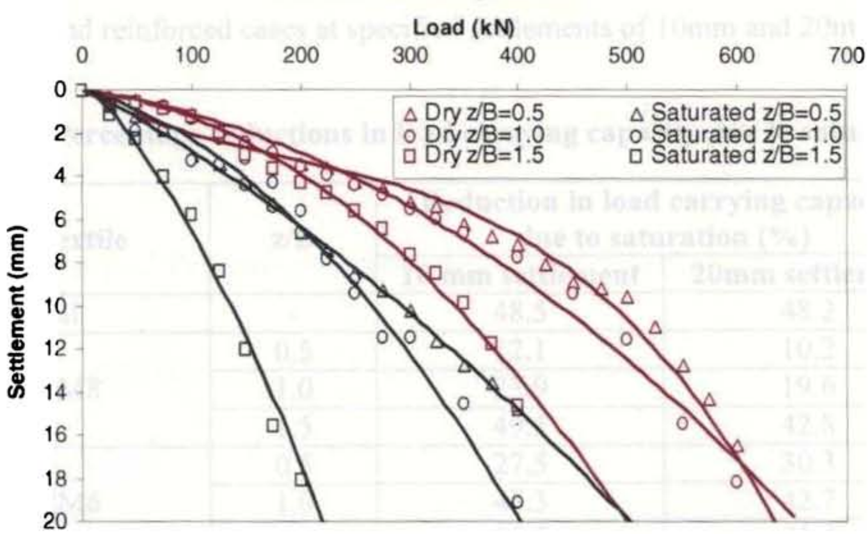
Experiments were done with coir geotextiles placed at different levels both in dry and saturated condition. Fig. 8.12 shows the effect of saturation when coir geotextiles were placed corresponding to z/B values of 0.5, 1.0 and 1.5.



(a) H2M8 coir geotextile



(b) H2M6 coir geotextile



(c) Non-woven coir geotextile

Fig.8.12 Effect of saturation on coir geotextile reinforced sand

It is clear that the bearing capacity reduces upon saturation in all cases with and without reinforcement. The amount of reduction when the coir geotextiles are placed is not uniform but depends on the type of coir geotextile and placement depth (z/B). For example, the reduction in load carrying capacity due to saturation of the unreinforced soil tested is 49%. When H2M8 coir geotextiles are placed at a depth of 100mm from the surface (z/B = 0.5), the load carrying capacity was reduced by 12% with respect to the corresponding dry condition. The percentage reduction in load carrying capacity was 24% when H2M8 coir geotextile was introduced at 200mm (z/B = 1.0) from the surface. In saturated condition it was observed that the settlement plots for unreinforced case and reinforced case at z/B = 1.5 were almost coinciding, showing only nominal increase in the load carrying capacity when coir geotextiles were placed at z/B = 1.5. Thus it can be postulated that, in saturated condition when coir geotextiles are placed at depths beyond z/B = 1.0, it does not fetch any appreciable increase in load carrying capacity. Similar computations can be made for different settlement levels when different types of coir geotextiles are placed. Table 8.2 gives the percentage reduction in load carrying capacity due to saturation for the unreinforced and reinforced cases at specified settlements of 10mm and 20mm.

Table 8.2 Percentage reductions in load carrying capacity due to saturation

Geotextile	z/B	Reduction in load carrying capacity due to saturation (%)	
		10 mm settlement	20mm settlement
Nil	-	48.5	48.2
H2M8	0.5	12.1	10.2
	1.0	23.9	19.6
	1.5	49.5	42.8
H2M6	0.5	27.5	30.3
	1.0	48.3	42.7
	1.5	50.5	41.4
Non-woven	0.5	35.8	20.6
	1.0	43.3	38.5
	1.5	61.3	60.0

A comparison of the load carrying capacity due to placement of coir reinforcement for dry and saturated conditions is shown in Fig. 8.13. Analysing the test results, it could be observed that the presence of water reduces the load carrying capacity of the soil, in general, but by the introduction of coir geotextiles, this can be improved and the percentage reduction of load carrying capacity due to saturation can be minimised. In dry condition NW coir geotextile gave maximum load carrying capacity and H2M6 the minimum, whereas in saturated condition the performances of NW and H2M8 were almost same.

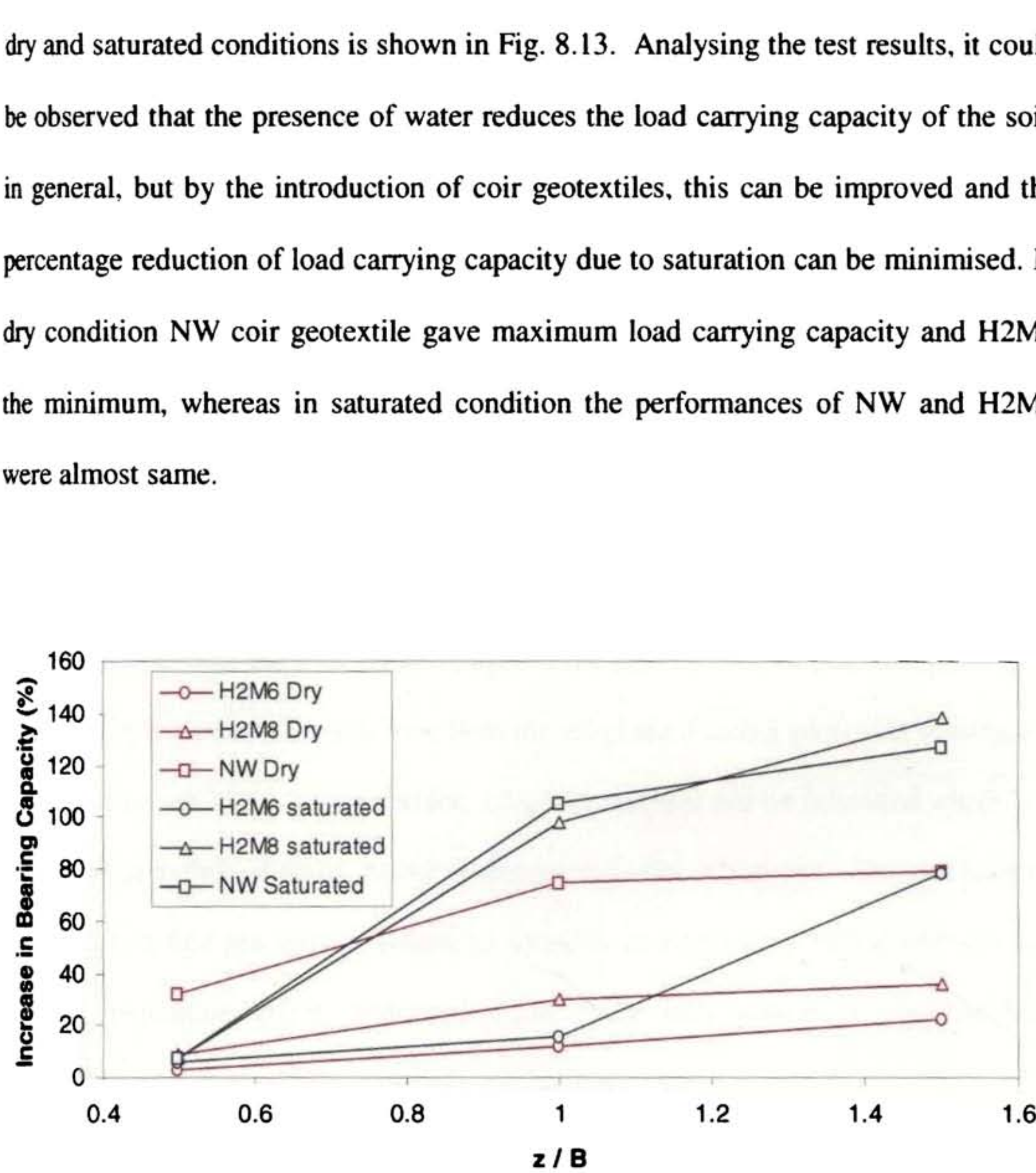


Fig. 8.13 Percentage increase in bearing capacity with z/B

Since the percentage increase in load carrying capacity is more in saturated condition, as can be observed from Fig. 8.13, it could be a better option for use in roads and embankments because it is likely that the formations will be saturated due to rise in water table.

8.7 SUMMARY

Results clearly demonstrate that coir geotextiles, a natural product, can substantially increase the bearing capacity of shallow square footing on sand both in dry and saturated conditions. The performance in saturated condition was even better in some cases. Both H2M8 and NW coir geotextiles were found suitable for increasing the bearing capacity in dry condition and the former was found more effective in saturated condition.

CHAPTER 9

RUT BEHAVIOUR OF COIR GEOTEXTILE REINFORCED UNPAVED ROADS

9.1 GENERAL

A road continuously deteriorates under the combined action of traffic loading and the environment. The most common indicators of pavement performance, the ability of roads to satisfy the demands of traffic and environment over its design life, are surface rutting, fatigue cracking, riding quality and skid resistance.

Geotextiles increase the stability and improve the performance of weak subgrade soils primarily by separating the sub base from the subgrade. Placing geotextile at subgrade - sub base or sub base - base interface, subgrade restraint can be enhanced which will facilitate the mobilization of heavy construction machinery at site. The mechanisms attributing to this are increased bearing capacity in addition to lateral restraint and tension membrane effect. Substantial life cycle cost saving is possible with geosynthetic reinforced aggregate base course in pavements.

Mechanistic method for flexible pavement design is regarded as the most powerful pavement design methodology and is becoming increasingly popular amongst various countries. In India too Indian Roads Congress has updated the specifications for flexible pavement design by changing the design methodology from empiricism to mechanistic design principles. In the mechanistic approach, the two design criteria, the fatigue failure and rutting failure corresponding to the horizontal tensile strain at the bottom of the bituminous layer and the vertical compression strain on the subgrade are considered (Chakroborty and Das, 2003).

Rutting is the permanent deformation along its wheel path. It is a manifestation of two different phenomena: i) densification and ii) shear deformation of pavement layer materials and subgrade (Yoder and Witczak, 1975). Rutting is very important because of its safety implications. The contributions to rutting from various layers could be different. It is reported that 46% of rutting took place from bituminous surface and granular base course, while sub base and subgrade contributed 54% of the total rutting (AASHO, 1962). The vertical strain on the subgrade is assumed as the index of rutting to occur in a pavement.

In the present study, the rut behaviour of unpaved roads with coir geotextile reinforcements placed at subgrade – sub base interface and also between layers of sub base under the action of static loads and repetitive loads were studied. The details of the testing programme and discussion of the results obtained are described in this chapter.

9.2 RUT BEHAVIOUR UNDER STATIC WHEEL LOADS

Plate bearing tests were performed to investigate the behaviour of coir geotextile reinforced unpaved roads under static loads. The test section consisted of 600mm thick subgrade overlain by water bound macadam (WBM) 150 mm thick.

9.2.1 Experimental Set - up

The experimental set - up consisted of a plate load test facility as described in section 8.3. The loading was done with the help of a 200 kN capacity hydraulic jack and self reaction frame made of mild steel I sections. The load was applied through 200mm square mild steel plate, 25.4mm thick to simulate Equivalent Single Wheel Load

(ESWL). Rut measurements were made using LVDTs. The schematic arrangement of the test set - up is shown in Fig. 9.1.

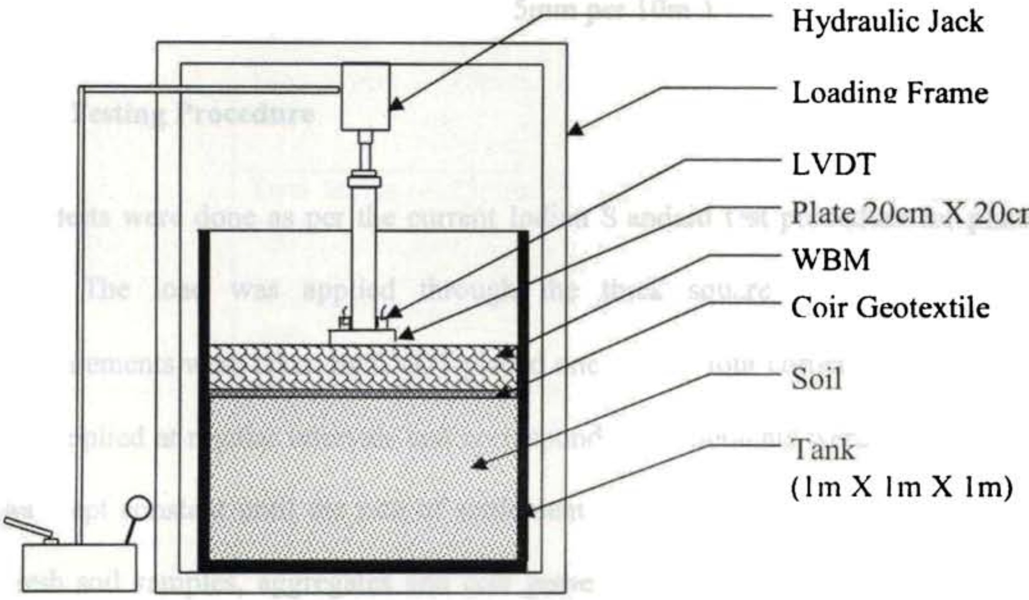


Fig. 9.1 Schematic test set – up to study rut behaviour under static loads

9.2.2 Preparation of Test Bed

For the present study two types of subgrades, red soil (Soil - 1) and Clayey silt soil (Soil - 3) were used. Three types of coir geotextiles (Woven – H2M6 and H2M8 and Non - Woven) were used as reinforcing layer. Water bound Macadam was constructed using granite aggregates and screenings.

The subgrades were prepared at a dry unit weight of 15 kN/m^3 and with a water content of 10% for red soil and 4% for clayey silt soil subgrade. Required quantity of wet soil was prepared by mixing dry soil with water. Soil was filled in the tank in layers of compacted thickness of 100 mm each up to a total height of 600mm in all trials. Coir geotextiles cut to the inside dimension of the tank was placed over the prepared sub grade. Water bound macadam of grade II was laid over the compacted subgrade. The quantities of coarse aggregate and screenings were taken as per MORD specifications ($0.91m^3$ to $1.07m^3$ of coarse aggregate and $0.12m^3$ to $0.13m^3$ of screening for a compacted thickness of 75mm per $10m^2$).

9.2.3 Testing Procedure

The tests were done as per the current Indian Standard test procedure for plate load tests. The load was applied through the thick square mild steel plate. Rut measurements were taken by LVDT placed one each at four corners of the plate. Load was applied at regular intervals and corresponding settlements were noted. Each load was kept constant until the rate of settlement reduces to less than 0.025mm/minute. Fresh soil samples, aggregates and coir geotextiles were used for each testing. The details of the different tests carried out are summarised in Table 9.1.

9.2.4 Results and Discussion

The principal criterion for determining the thickness of flexible pavements is the vertical compressive strain on top of the subgrade imposed by standard axial load. In India the standard axial load is 81.7 kN. Excessive vertical subgrade strain causes permanent deformation in the subgrade, which is manifested in the form of rutting on the pavement surface. Acceptability level of rut depth is different in different countries. IRC: 37-2001 recommended an allowable rut depth of 20 mm to estimate the rutting life of the pavement in terms of standard load repetition. IRC: SP: 20-2002 recommends that the maximum rutting that can be accepted in rural roads may be taken as 50 mm before rehabilitation work is needed.

Table 9.1 Summary details of tests conducted to study rut behaviour

Sl. No.	Type of subgrade soil	Sub base (WBM) thickness	Type of reinforcement	Location of reinforcement
1	Soil - 1	One layer – 150 mm	No reinforcement	-
2		One layer – 150 mm	H2M8	At interface
3		One layer – 150 mm	H2M6	At interface
4		One layer – 150 mm	Non Woven	At interface
5		Two layers – 75mm each	H2M8	At interface and at mid depth of WBM
6		Two layers – 75mm each	H2M6	At interface and at mid depth of WBM
7		Two layers – 75mm each	Non Woven	At interface and at mid depth of WBM
8	Soil - 3	One layer – 150 mm	No reinforcement	-
9		One layer – 150 mm	H2M8	At interface
10		One layer – 150 mm	H2M6	At interface
11		One layer – 150 mm	Non Woven	At interface

9.2.4.1 Rut behaviour of red soil subgrade

Fig. 9.2 shows the performance variation in terms of rut depth due to applied wheel loads for unpaved road sections with and without coir geotextiles placed at subgrade-sub base interface. It could be observed that the control section without coir geotextile reinforcement can sustain a wheel load stress of 192.5kPa for a rut depth of 20 mm and 317.5kPa for a rut depth 50mm. When H2M6 coir geotextile was introduced at subgrade - sub base interface, this load carrying capacity was increased to 262.5kPa and 462.5 kPa respectively. Also these values were respectively 325 kPa and 560 kPa for non-woven coir geotextile and 337.5kPa and 542.5kPa for H2M8 coir geotextiles. The percentage increase in the load carrying capacity is worked out to be 37%, 69% and 75% for H2M6, NW and H2M8 coir geotextiles respectively at 20 mm rut depth and 48%, 77% and 71% at 50 mm rut depth. Again at greater rut depths the percentage increases in stress were found to be still higher. The increase in the load carrying capacity is attributed to the separation of aggregate from the subgrade in addition to the strength gain due to friction or interlock developed between the aggregate and geotextiles. The contribution due to lateral restraint is very less for non-woven geotextile whereas that due to separation and bearing capacity is very high for non-woven coir geotextile.

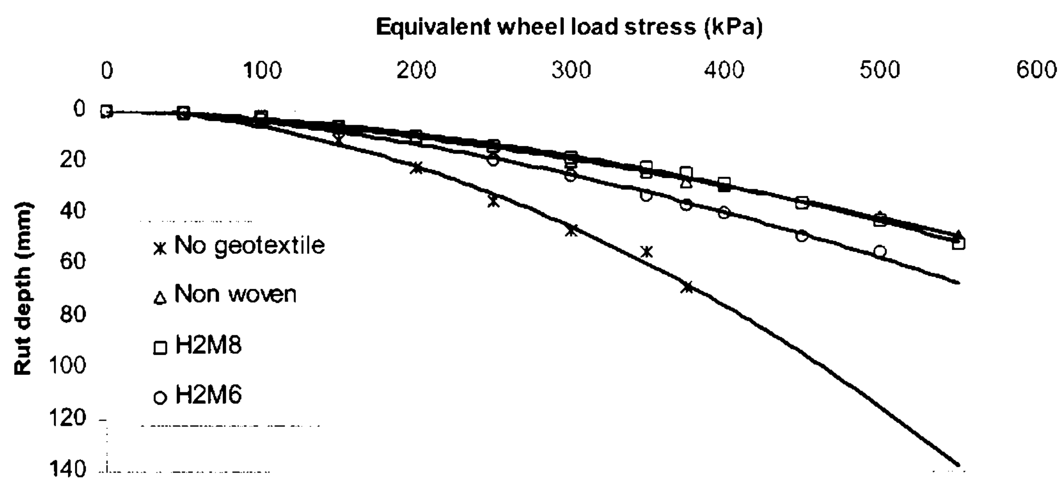


Fig. 9.2 Rut depth due to wheel load stress in WBM with red soil subgrade

9.2.4.2 Rut Behaviour of clayey silt subgrade

The rut depth behaviour for unpaved road section on clayey silt subgrade under static load condition is shown in Fig. 9.3. For the control section, a wheel load stress of 300kPa produced 20 mm rut depth. The wheel load stress for the same rut depth for coir reinforced sections with H2M6, H2M8 and NW were 355 kPa, 450 kPa and 537.5 kPa respectively. For 50mm rut depth the corresponding value for un-reinforced case was 475 kPa and for the reinforced cases the values were 562.5 kPa, 712.5 kPa and 837.5kPa respectively for H2M6, H2M8 and NW coir geotextiles placed at subgrade-sub base interface. In this case the Non-woven coir reinforced unpaved section is found to be superior to H2M8 and H2M6 reinforced one. The subgrade soil, being more clayey in nature is expected to be more flexible and the Non-woven geotextile may be more adaptable to follow the undulations. This may increase the performance of geotextile as separator and as reinforcement due to membrane effect.

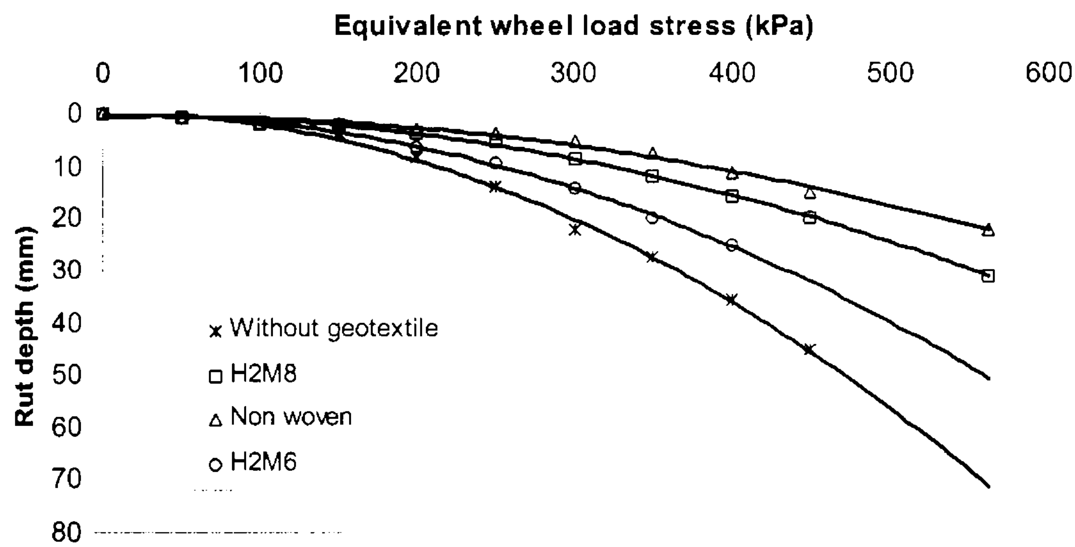


Fig. 9.3 Rut depth due to wheel load stress in WBM with clayey silt subgrade

9.2.4.3 Effect of additional reinforcement layer

In order to explore the possibility of further improving the rut behaviour by using coir geotextiles, an additional layer of coir geotextile was provided within the subbase and analysed. When two layers of coir geotextiles were placed (one at subgrade – sub base interface and other within the sub base itself at mid depth), the load carrying capacity was further improved as shown in Fig 9.4. A comparison between single layer system with coir geotextiles at the interface and two-layer system with an additional reinforcement layer within the WBM section (red soil subgrade) shows that while there was a noticeable difference between reinforced and unreinforced cases, there exists a marginal difference among the single layer and two-layer reinforced cases.

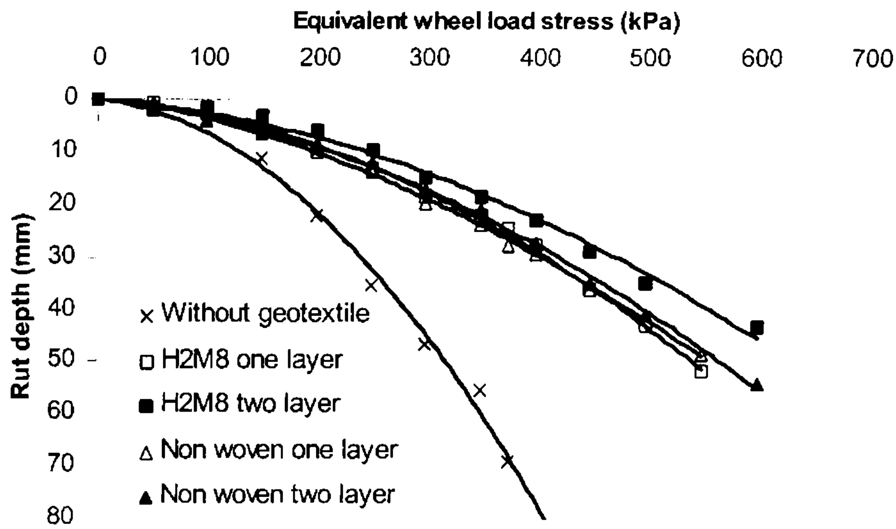


Fig. 9.4 Effect of additional layer of coir geotextile

It could be observed from the test results that, in the case of Non-woven coir geotextiles for a rut depth of 20 mm the carrying capacity was 295 kN/m^2 with single layer reinforcement and 327.5 kN/m^2 with two layers showing only 11% increase due to additional layer of reinforcement. With H2M8 reinforcement the percentage increase in carrying capacity due to additional layer of reinforcement is only 15.5%. For 50mm rut depth, the carrying capacity with single layer reinforcement and two-layer reinforcement was 552.5 kN/m^2 and 562.5 kN/m^2 for NW coir geotextile and 537.5 kN/m^2 and 625 kN/m^2 for H2M8 coir geotextile, inferring a little effect on carrying capacity due to the additional layer of reinforcement in the case of Non-woven geotextiles.

9.2.4.4 Effect of type of coir geotextile

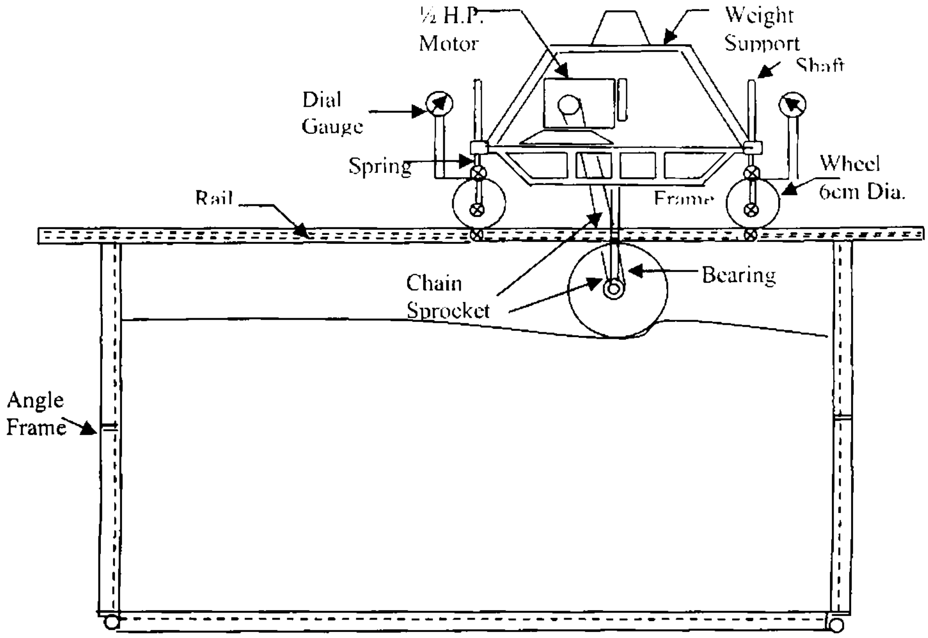
While analysing the rut behaviour of unpaved road section with red soil subgrade (Fig. 9.2) the percentage increase in carrying capacity was 37%, 69% and 75% in the order of H2M6, NW and H2M8 coir geotextiles for a rut depth of 20 mm. For 50 mm rut depth, the corresponding increases were 46%, 77% and 71% respectively. For unpaved section with clayey silt subgrade, the performance was remarkable when H2M8 and NW coir geotextiles were placed at the interface. For 20 mm rut depth the percentage increase in load carrying capacity in relation to unreinforced case was19 %, 50% and 80% with H2M6, H2M8 and NW coir geotextiles whereas at 50 mm rut depth, the percentage increase in carrying capacity was respectively 19%, 50% and 77%. When two layers of reinforcements were placed, H2M8 coir geotextile gave the highest performance and the contribution due to additional layer of NW coir geotextile was meagre.

9.3 RUT BEHAVIOUR UNDER REPETITIVE LOADS

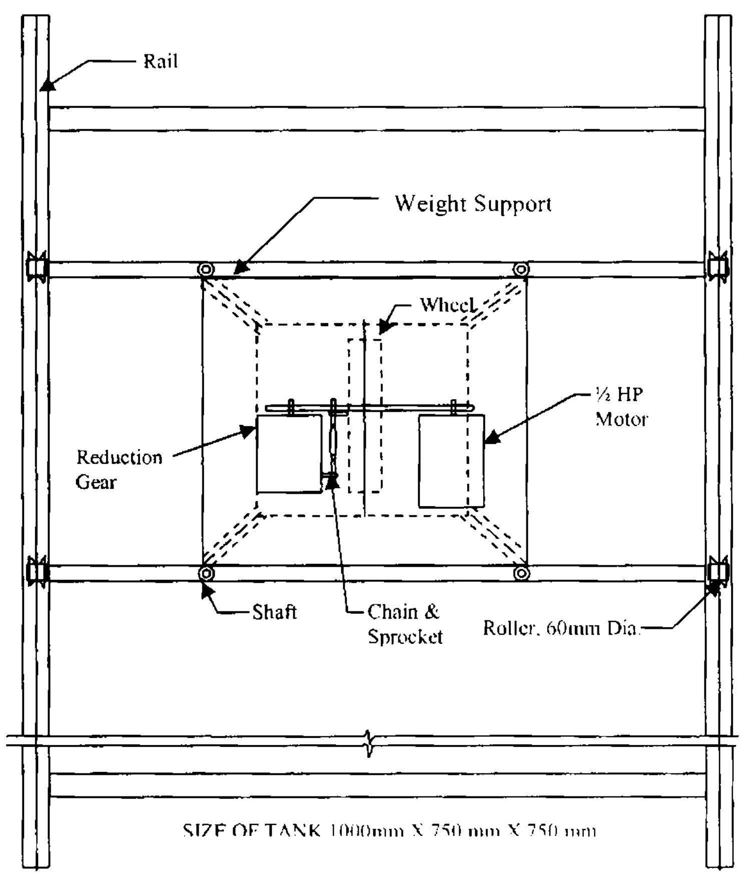
In order to study the benefits of applying coir geotextile reinforcement in improving rutting resistance of unpaved roads, laboratory wheel tracking tests were performed. The details of the study are explained in the following sections.

9.3.1 Wheel Tracking Apparatus

Laboratory wheel tracking tests is the most practical tool to study the rutting behaviour of pavement materials under simulated moving traffic loads (Wasage et al., 2004). A wheel-tracking machine was designed and fabricated in the present work to study the effect of load repetitions. Track bed was made in a steel tank measuring 1.5m x 0.75m x 0.75m. A 0.5 HP constant torque geared motor was used to control the motion of the wheel. The wheel was 200mm diameter and 45mm wide with a rubber ring over it. The motor was mounted on a frame, which moves on four 60mm diameter wheels, on rails provided on the top of two long sidewalls of the tank. The



a. Longitudinal Section



b. Plan

Fig. 9.5. Wheel tracking apparatus

tracking machine had an in built counter that can register the number of passes of the wheel. Arrangements were provided for simulating ESWL by placing steel plates on the frame. Rut measurements were made using a depth gauge. The longitudinal section and sectional plan of the test set - up are shown in Fig. 9.5 and its photograph is given in Fig. 9.6.

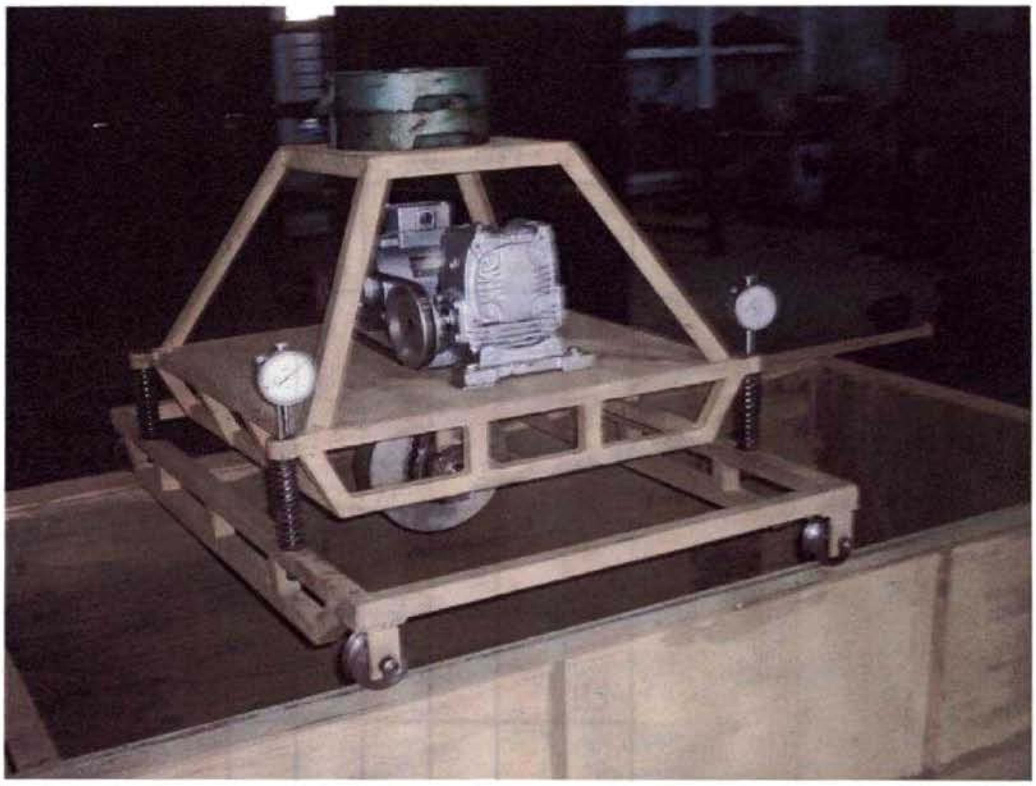


Fig. 9.6 Photograph of wheel tracking apparatus

9.3.2 Testing Programme

To prepare the test bed, the tank was filled with red soil (soil-2) in layers and compacted to a density of 1400 kg/m^3. The subgrade was filled to a height of 600mm. Coir geotextile was placed over the subgrade and then WBM layer was laid over it using aggregates of size 22.4mm down and screenings of grading B. The compacted thickness of the WBM layer was 150 mm. Four series of experiments were done, viz.,

(i) Control test with no geotextiles,

(ii) Test with Non-woven coir geotextile at the subgrade - base interface,

(iii) Test with H2M8 coir geotextile at the subgrade - base interface, and

(iv) Test with H2M6 coir geotextile at the subgrade - base interface.

Rut measurements were taken at the top surface of WBM section after the specified number of wheel passes using the depth gauge at 25 locations which are represented by A1, B1, ……., E5 as shown in Fig. 9.7.

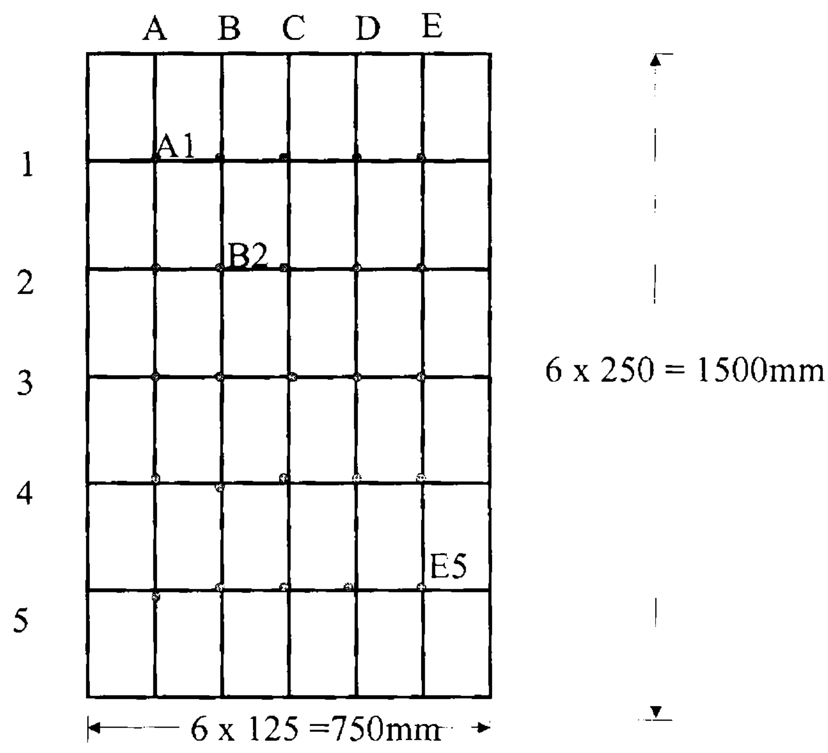


Fig. 9.7 Locations of rut measurement

9.3.3 Test Results and Discussion

From the observed data, rut profiles in the longitudinal direction and transverse direction were drawn for the controlled section and coir-reinforced section for different wheel load passes. Fig 9.8 gives the transverse rut profile at center (through A3, B3, C3, D3 and E3) for the control section. It could be seen that a rut depth of 20mm, which is the allowable rut depth by the IRC, has occurred at about 55 number of wheel passes. The development of rut was very fast in the initial stages of wheel passes and afterwards it was observed that the increase in rut depth was gradual. Accordingly, a rut depth of 29mm, which was obtained for 500 passes became 49mm only, after 1750 number of wheel passes. The soil on either side of the wheel was bulged due to loading, which is manifested as negative settlement in the figure. Fig.9.9 shows the variation of rut depth after 1750 wheel passes along the centre (through A3, B3, C3, D3 and E3) for the reinforced and unreinforced cases. It was observed that due to placement of the coir geotextile the rut depth was reduced considerably. Thus, when rut measurements were taken at C3 location, on the section reinforced with Non-woven geotextile, it was only 2mm, whereas it was 48.5mm for the unreinforced case. Similar effects were noted with other geotextiles like H2M8 and H2M6, which gave maximum rut depths of 24mm and 30mm respectively after 1750 passes of the wheel. The percentage reduction in rut depth amounts to 55%, 50% and 38% respectively with Non-woven, H2M8 and H2M6 after 1750 wheel passes. At B3 location the bulging was eliminated when coir geotextiles were placed. Thus it can be concluded that coir geotextiles function both as separator and as reinforcement in the case of repetitive loads also. From the profiles drawn it was observed that, heaves on both sides of the rut had approximately in equal volume to the volume of rut. This suggests that displacement of the materials rather than densification of the layers contributed to the rut formation. While comparing reinforced and unreinforced cases, it was observed that there existed no heaving in coir-reinforced sections.

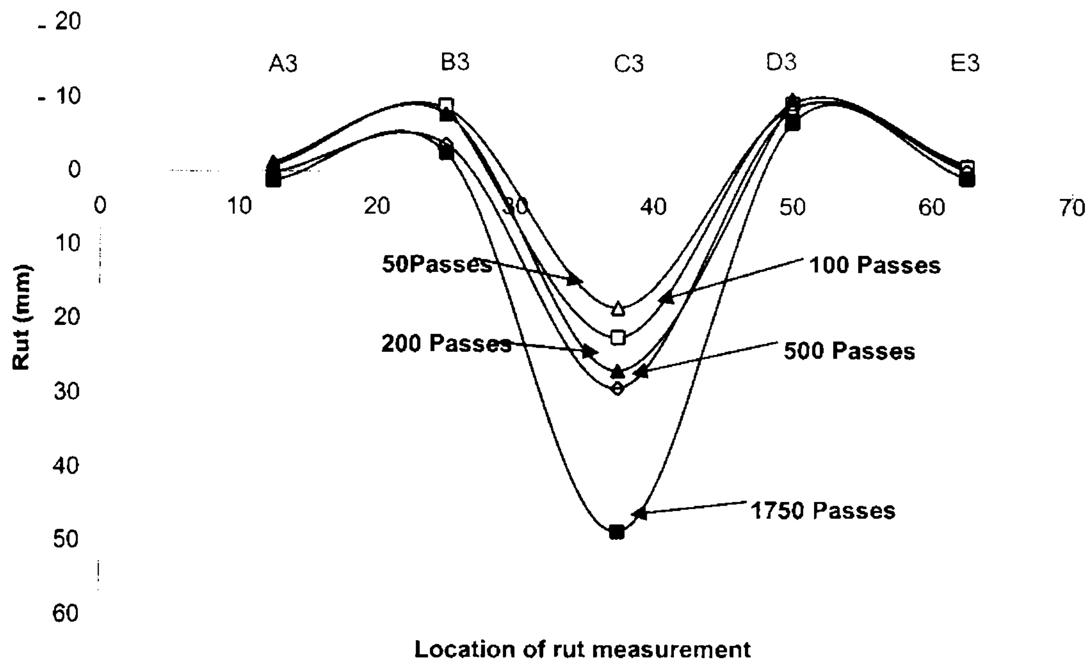


Fig. 9.8 Transverse rut profile for control section

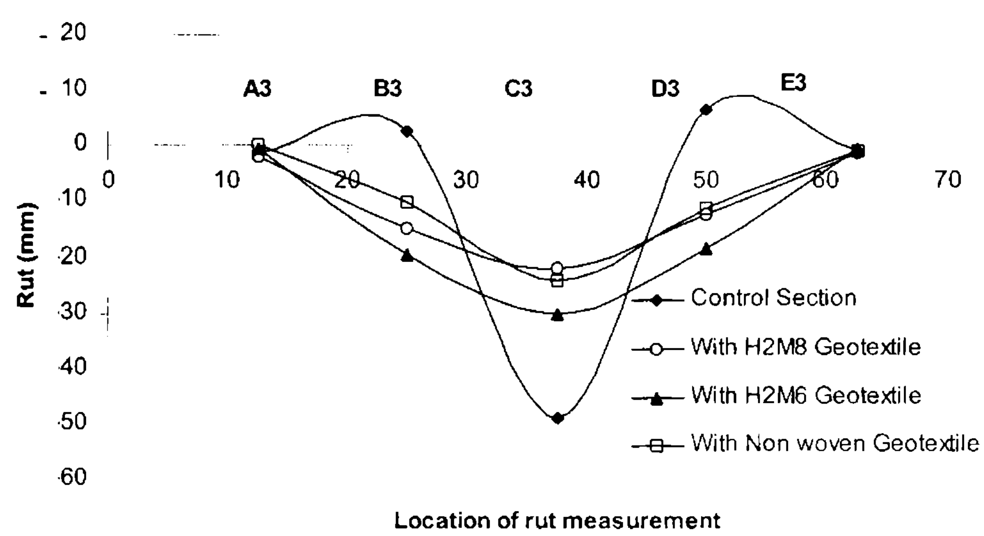


Fig. 9.9 Rut profiles for reinforced sections after 1750 wheel passes

Fig. 9.10 shows the variation of rut depth with the number of wheel passes when coir geotextiles are placed at subgrade - sub base interface. It could be seen that a rut depth of 20mm was produced due to 55 wheel passes in the case of control section whereas coir reinforced section with H2M8 coir geotextile needed 1050 passes for the same rut depth to take place. Also, sections with Non-woven and H2M6 coir geotextiles produced 20 mm rut after 950 and 450 number of passes. Variations in rut depth for 500, 1000 and 1500 passes are compared in Fig. 9.11 for different geotextiles. It is clear from the figure that H2M8 and Non-woven coir geotextile produced similar performance. Fig. 9.12 shows typical graphs for 1750 wheel passes along the centreline and along a path 125mm away from the centreline.

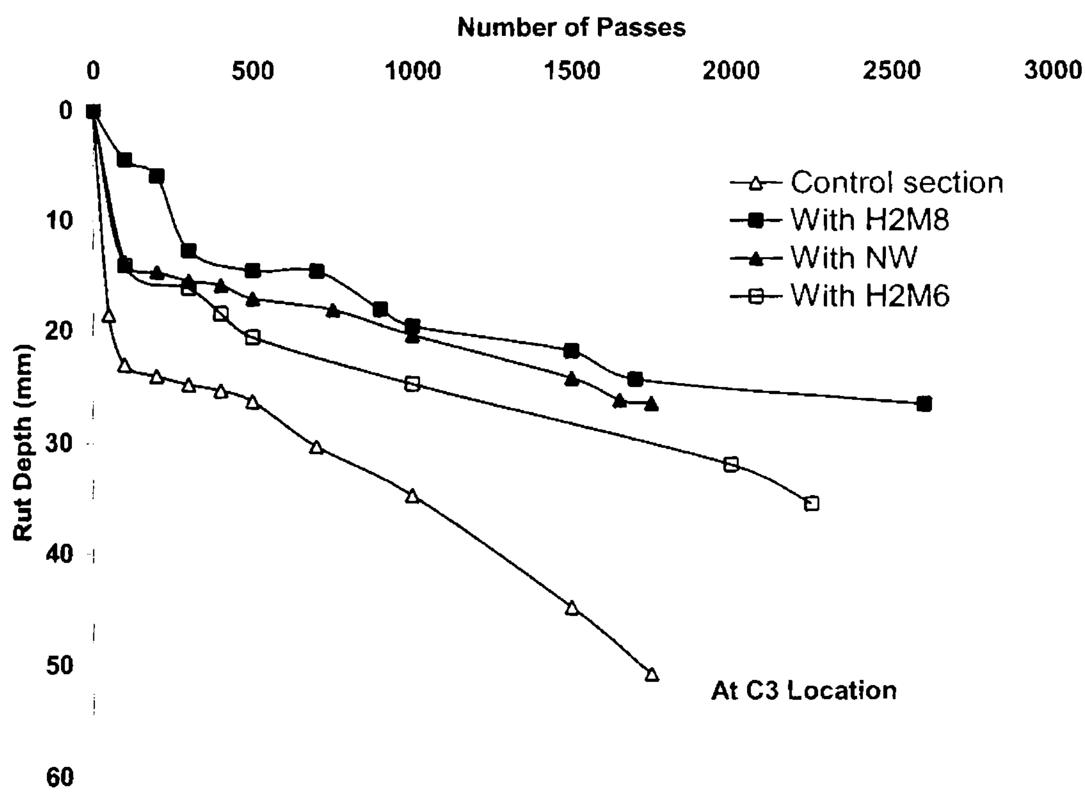


Fig.9.10 Variation of rut depth with number of wheel passes

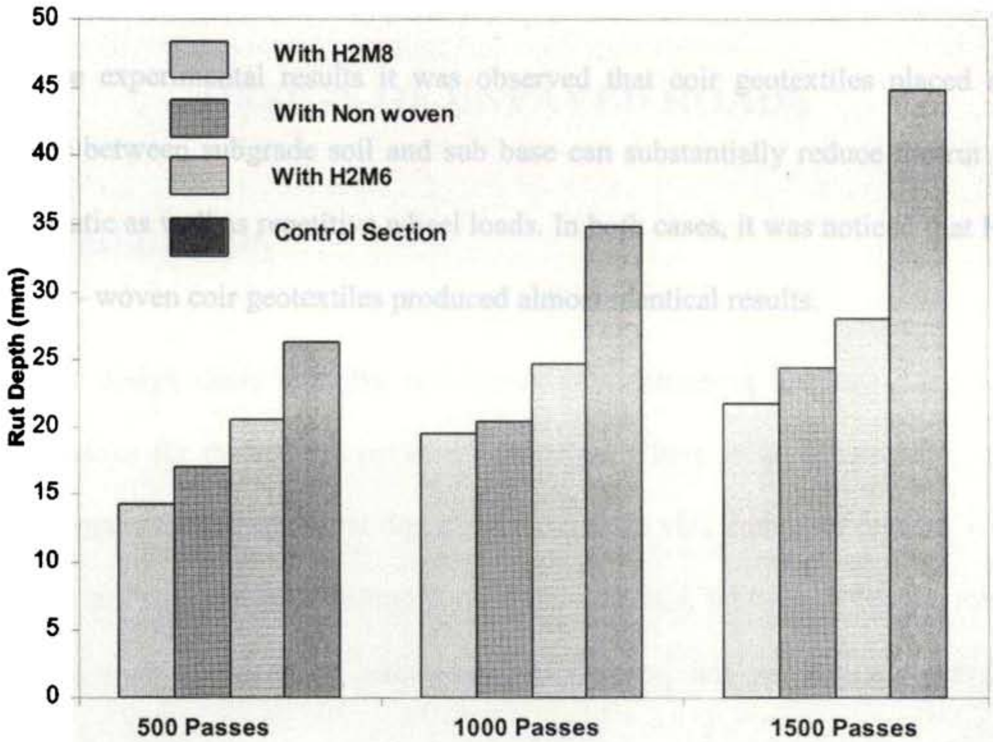
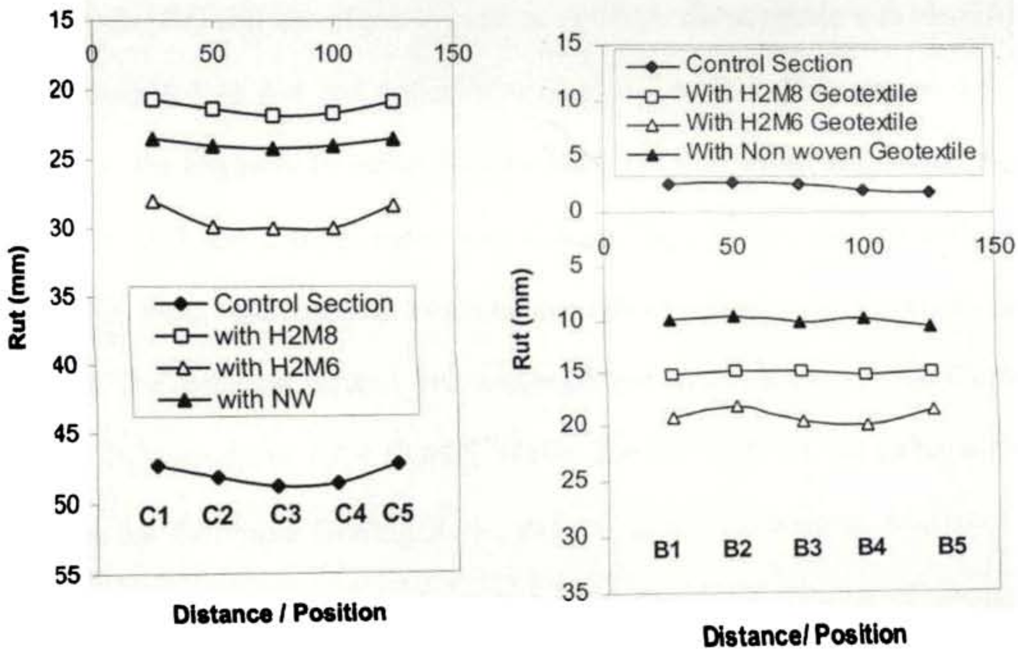


Fig. 9.11 Effect of coir geotextile on rut depth



(a) along centreline **(b) away from centreline**

Fig. 9.12 Longitudinal rut profile

9.4 SUMMARY

From the experimental results it was observed that coir geotextiles placed at the interface between subgrade soil and sub base can substantially reduce the rut depth due to static as well as repetitive wheel loads. In both cases, it was noticed that H2M8 and Non - woven coir geotextiles produced almost identical results.

CHAPTER 10

DESIGN OF UNPAVED ROADS

10.1 INTRODUCTION

Pavement design deals with the techniques of determining thickness, and laying configurations for the chosen pavement materials. There exist a number of design methodologies for the structural design of pavements viz., empirical method with or without a soil strength test, limiting shear failure method, limiting deflection method, regression method based on pavement performance, and mechanistic- empirical method.

The use of empirical method without a strength test dates back to the development of Public Roads (PR) soil classification system, in which the subgrade was classified as uniform from A-1 to A-8 and nonuniform from B-1 to B-3. This system was later modified by the Highway Research Board (HRB, 1945) in which soils were grouped from A-1 to A-7 and a group index was added to differentiate the soil within each group. This index was used to estimate the sub base and total thickness of the pavement. The empirical method with a strength test was first used by the California Highway Department in 1929 (Porter, 1950). The thickness of the pavements was related to the California Bearing Ratio, defined as the penetration resistance of a subgrade soil relative to standard crushed rock. The CBR method of design was studied extensively by the US Corps of engineers during the World War II and became a very popular method of pavement design after the war. The Indian Roads Congress also used this method to determine the thickness of individual layers of pavement. The disadvantage of this empirical method is that it can be applied only to a given set of environmental, material, and loading conditions.

In the limiting shear failure method the thickness of pavement is determined so that shear failures will not occur. The major properties of subgrade soil considered are cohesion and angle of internal friction. Mc Leod (1953) advocated the use of logarithmic spirals to determine the bearing capacity of pavements.

The limiting deflection method is used to determine the thickness of pavements so that the vertical deflection will not exceed the allowable limit. The Kansas State Highway Commission (1947) modified Boussinesq's equation and limited the deflection of subgrade to 0.1 inches (2.54mm). The US Navy (1953) applied Burmister's theory (Burmister, 1943) and limited the surface deflection to 0.25 inches (6.35mm). The use of deflection as a design criterion has the apparent advantage that it can be easily measured in the field. Unfortunately pavement failures do occur due to excessive stress and strain, instead of deflections.

A good example of the use of regression equations for pavement designs is the AASHTO method based on the results of road tests. The disadvantage of the method is that the design equation can be applied only to the conditions at the road test site.

The mechanistic – empirical methods of design are based on the mechanics of materials that relate an input, such as a wheel load, to an output or pavement response such as stress and strain. The response values are used to predict distress based on laboratory test and field performance data. Dependence on observed performance is necessary because theory alone has not proven sufficient to design pavements realistically.

The empirical – mechanistic method is more theoretical in approach, though it needs calibration based upon the performance of in – service pavements. This approach is increasingly popular amongst various countries. In India too, the pavement design guidelines IRC: 37 have updated in 2001 where the design methodology has changed from empiricism to mechanistic pavement design principles. The empirical – mechanistic approach is being successfully used in the design of reinforced sections also, as it tries to relate the stress – strain parameters with the expected life of the pavement.

In the present study three methods of design of unpaved roads reinforced with coir geotextiles are discussed: (a) Giroud and Noiray method for designing road sections with geotextiles placed at subgrade - sub base interface, (b) IRC method for designing road sections using the modified CBR value for the design of road sections with geotextiles placed within the subgrade, and (c) US Army method using modified CBR values from laboratory CBR tests on reinforced soil - aggregate systems.

It is expected that normally rural roads will not have more than 450 commercial vehicles per day. The standard axle load prescribed by the IRC is of 81.7 kN. Since coir geotextiles are used, the values of moduli are less than those of the polymeric geotextiles and as such, graphs for geotextiles in the range (10 kN/m to 100 kN/m) are developed. The limiting strain is taken as 30%. The failure rut depth prescribed by the IRC for the design of flexible pavements is 30mm in general, whereas it is 50mm for rural roads.

10.2 GIROUD AND NOIRAY METHOD

10.2.1 General

Giroud and Noiray (1981) use the geometric model shown in Fig. 10.1 for a wheel load pressure of p_{ec} on an area (B x L), which dissipates through 'h_0' thickness of aggregate base without geotextile and 'h' thickness of aggregate base with geotextile. The geometry indicated results in a stress of 'p_0' (without geotextile) and 'p' (with geotextile) on the soil subgrade as follows:

$$p_0 = \frac{P}{2(B+2h_0\tan\alpha_o)(L+2h_o\tan\alpha_o)} + \gamma h_o \qquad \text{-------------(10.1)}$$

$$p = \frac{P}{2(B+2h\tan\alpha)(L+2h\tan\alpha)} + \gamma h \qquad \text{--------------(10.2)}$$

where,

P = Axle load,

γ = Unit weight of aggregate,

α = load dispersion angle for unreinforced case, and

α_0 = load dispersion angle for reinforced case.

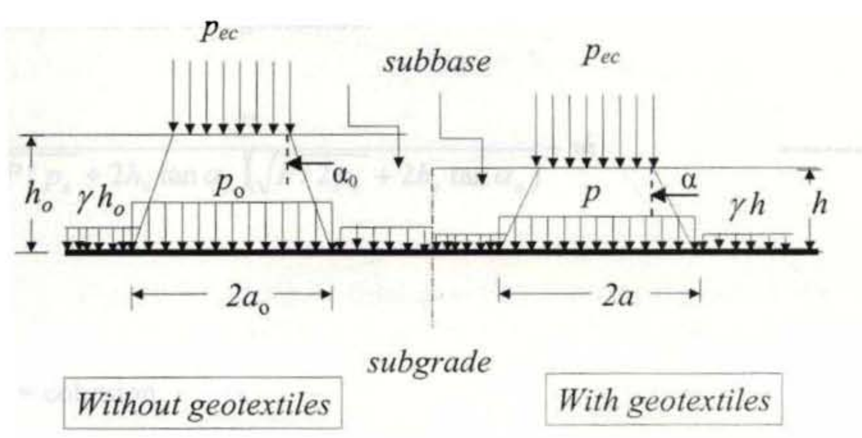


Fig. 10.1 Load distribution through sub base (After Giroud and Noiray, 1981)

Since the pressure exerted by the axle load through the aggregate into the soil subgrade is known, the shallow foundation theory of geotechnical engineering can be utilized. Assuming that the soil subgrade consists of fine-grained silt and clay in saturated condition, shear strength is taken as the undrained cohesion. Without geotextile, it is again assumed that the maximum pressure that can be maintained corresponds to the elastic limit of the soil, that is,

$$p_o = \pi c + \gamma h_0 \qquad \text{-----------------(10.3)}$$

and that with geotextile the limiting pressure can be increased to the ultimate bearing capacity of the soil, that is,

$$p^* = (\pi+2)\, c + \gamma h \qquad \text{------------------(10.4)}$$

10.2.2 Unpaved Road without Geotextile by Quasi-Static Analysis (h_0 vs. CBR)

For the case of no geotextile reinforcement, equation (10.1) and (10.3) can be solved, resulting in equation (10.5), which yields the desired aggregate thickness response curve without the use of a geotextile:

$$c = \frac{P}{2\left(\sqrt{P/p_c} + 2h_0 \tan\alpha_o\right)\left(\sqrt{P/2p_c} + 2h_o \tan\alpha_o\right)} + \gamma h_o \qquad \text{------------(10.5)}$$

where,

c = cohesion,

P = axle load,

P_c = tyre inflation pressure,

h_0 = aggregate thickness, and

α_0=load dispersion angle (assumed as 26 degrees).

The cohesion and CBR (%)of the subgrade soil is related empirically using the equation (IRC: 37-2001) as

$$CBR = 30\,c \qquad \text{-------------------(10.6)}$$

where, c is the cohesion in kPa.

Thus, graphs can be plotted connecting aggregate thickness (h_0) and CBR of the subgrade soil for different values of axle load and tyre pressure. Typical one for a wheel load of 81.7kN with a tyre pressure of 480kPa is shown in Fig.10.2

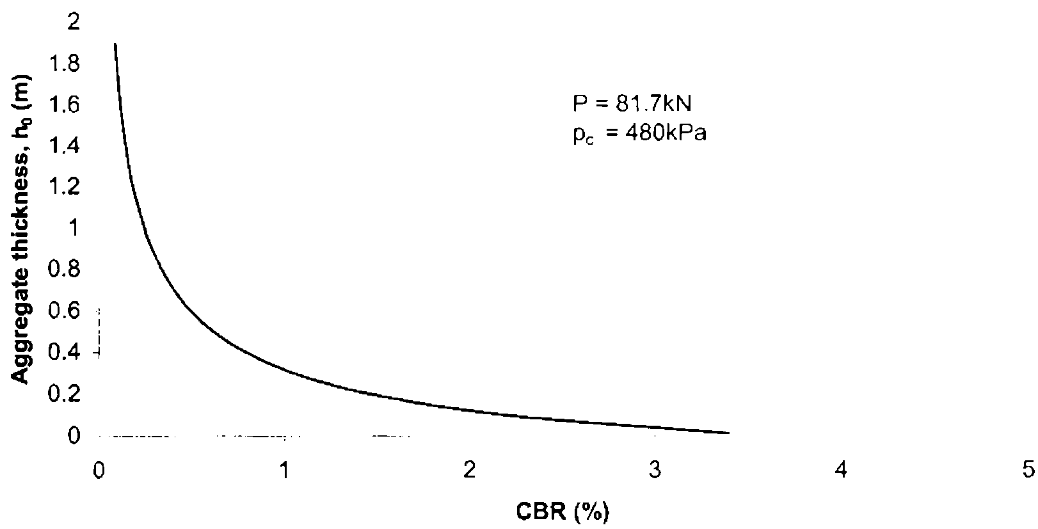


Fig. 10.2 Aggregate thickness (h_0) versus subgrade CBR
(Quasi – static analysis for case without geotextile)

10.2.3 Unpaved Road with Geotextiles by Quasi-Static Analysis (h vs CBR)

For the case where geotextile reinforcement is used, p^* in equation (10.4) is replaced by $(p\text{-}p_g)$, where p_g is a function of tension in the geotextile; hence its elongation is significant. On the basis of the probable deflected shape of the geotextile- soil system p_g is expressed as,

$$p_g = \frac{E_s \varepsilon}{a\sqrt{1+(a/2S)^2}} \qquad \text{---------------(10.7)}$$

where,

Es = Modulus of coir geotextile,

ε = Strain,

a = geometric property (Fig.10.1), and

S = Settlement under the wheel (rut depth).

Combining equations 10.2, 10.4 and 10.7 and using $p^* = p\text{-}p_g$, it gives equation 10.8

$$(\pi+2)c = \frac{P}{2(B+2h\tan\alpha)(L+2h\tan\alpha)} - \frac{E_s\varepsilon}{a\sqrt{1+(a/2S)^2}} \qquad \text{-------(10.8)}$$

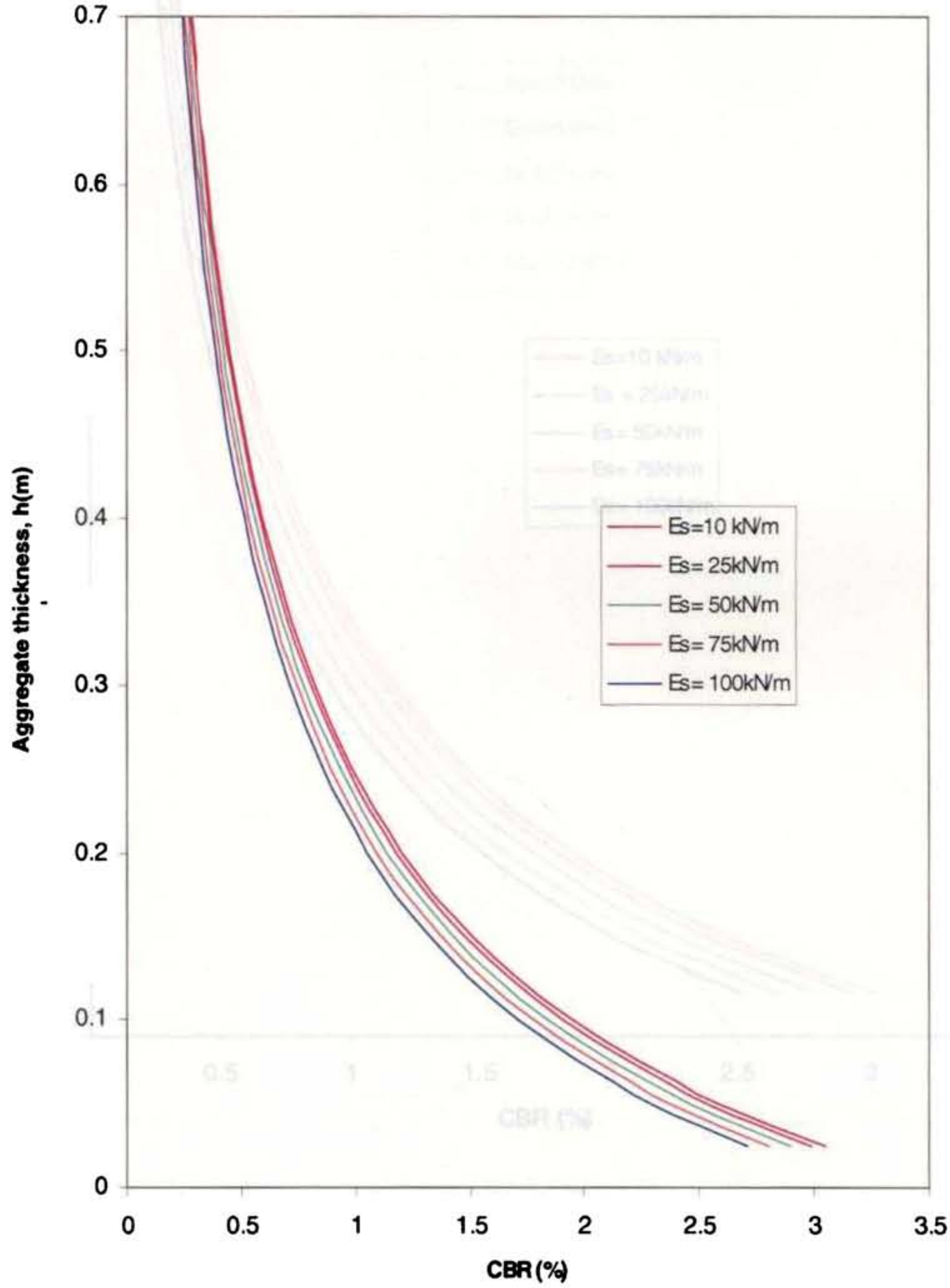
Using the above equation, response curves connecting h and CBR can be drawn for different values of rut depth, modulus of coir geotextile, axle load and tyre pressure. Typical curves for axle load of 81.7kN and tyre pressure of 480kPa for rut depths of 30mm and 50mm are shown in Fig. 10.3.

10.2.4 Reduction in Aggregate Thickness by the Use of Geotextile (Δh vs. CBR)

Having obtained the values of h_o from Fig 10.2 and h from Fig 10.3, the reduction of the base course thickness Δh resulting from the use of geotextile is deduced from quasi-static analysis by

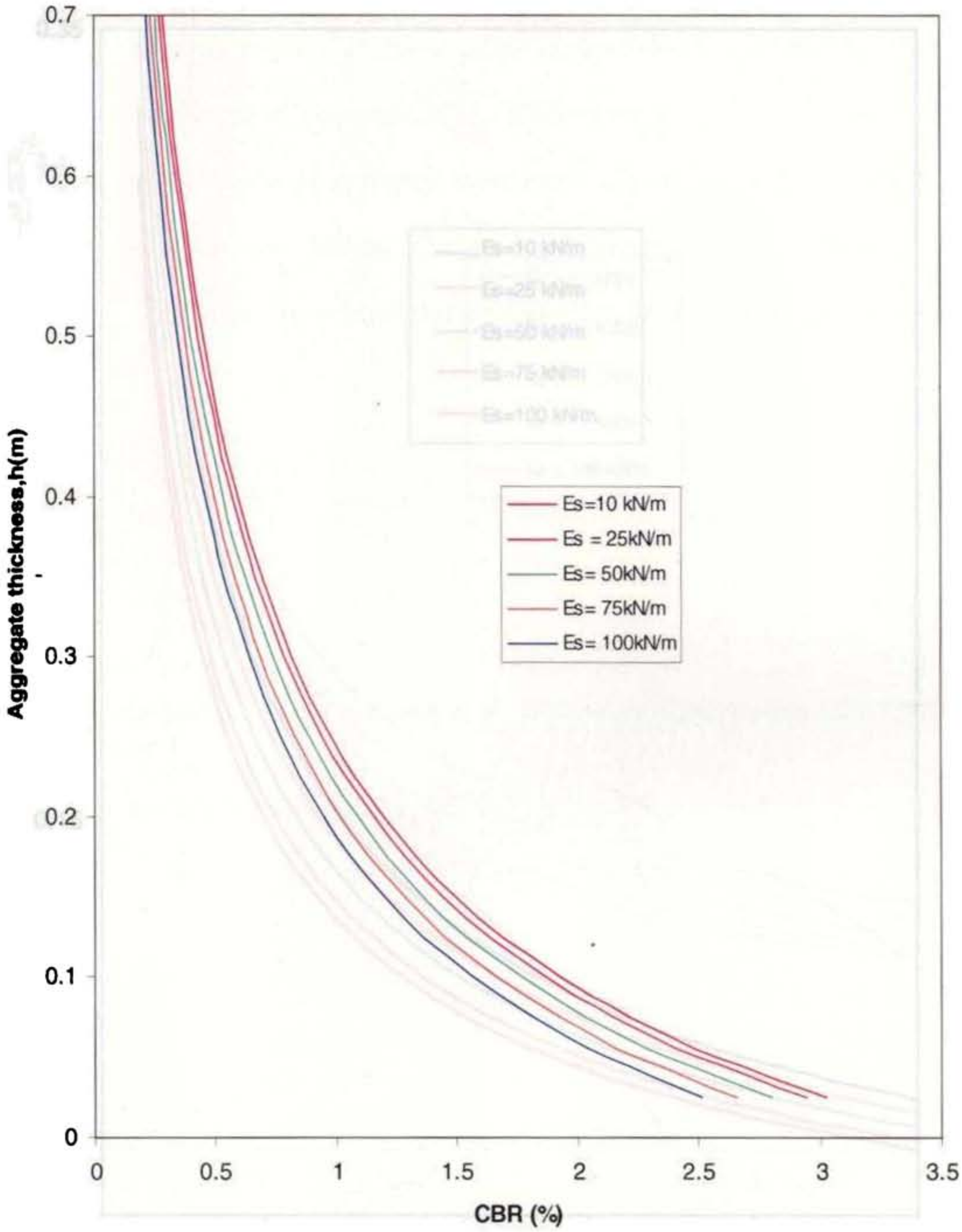
$$\Delta h = h_o - h \qquad \text{------------------ (10.9)}$$

Curves can be plotted relating Δh and CBR. The typical curves drawn for the rut depths of 30mm and 50mm are shown in Fig. 10.4 (a) and (b). It may be noted that *Δh* obtained above doesn't take traffic into consideration.



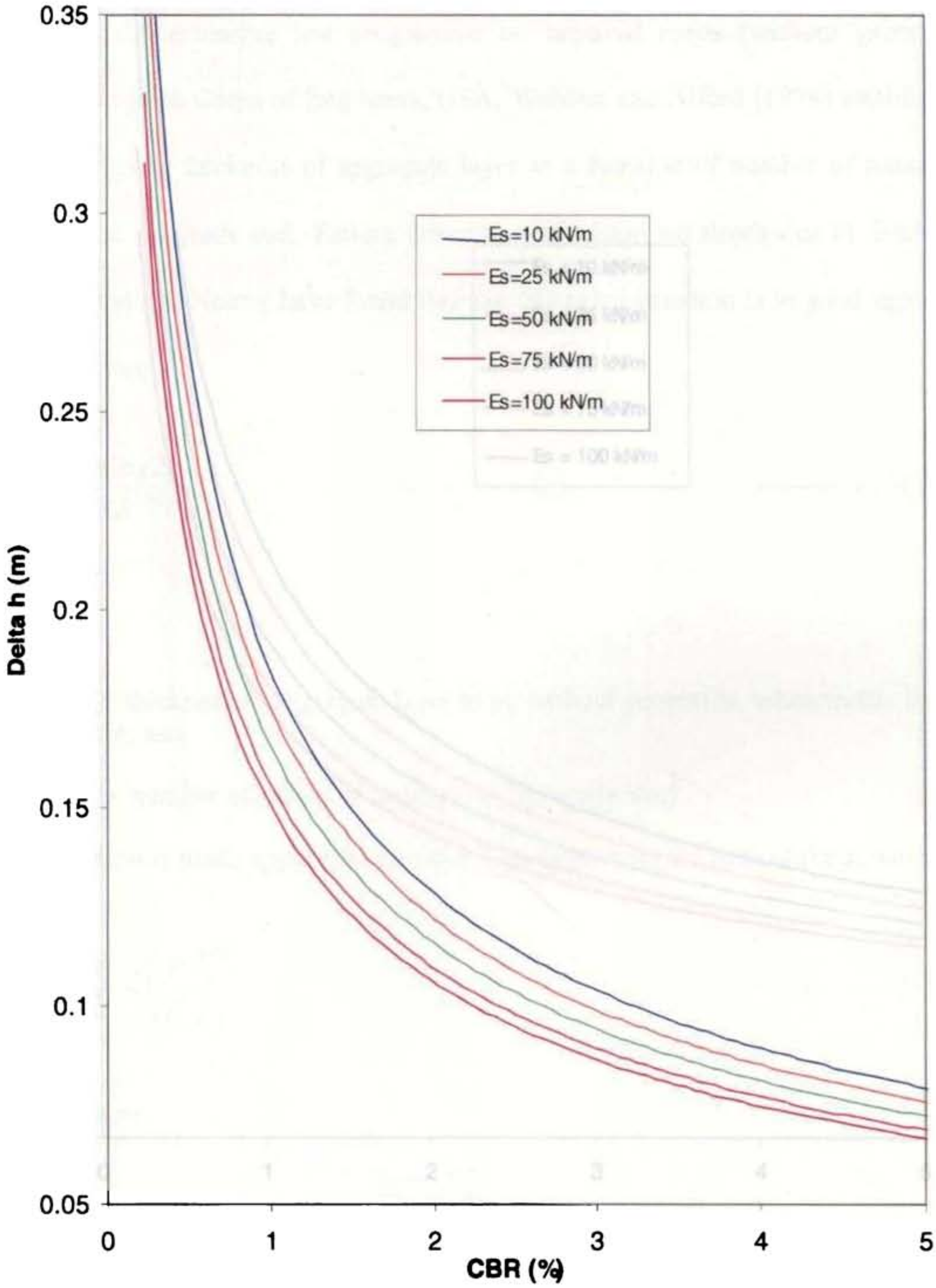
(a) rut depth = 30mm

Fig. 10.3 Aggregate thickness versus subgrade CBR Quasi - Static analysis for case with geotextile)



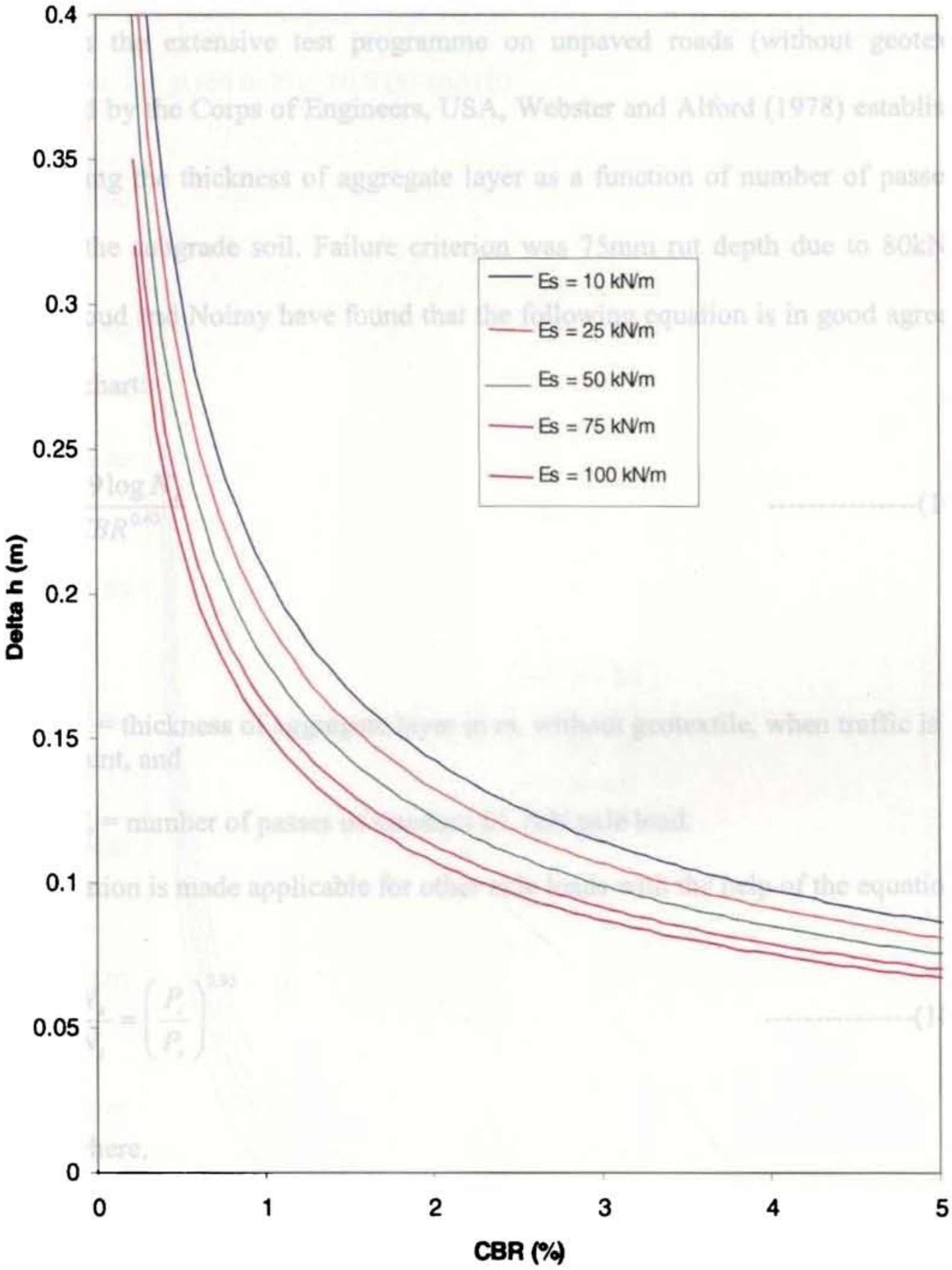
(b) rut depth = 50mm

Fig. 10.3 Aggregate thickness versus subgrade CBR
(Quasi - Static analysis for case with geotextile)



(a) rut depth = 30mm

Fig. 10.4 Variation of reduction in aggregate thickness with subgrade CBR (Quasi - static analysis for the case with geotextile)



(b) rut depth = 50mm

Fig. 10.4 Variation of reduction in aggregate thickness with subgrade CBR (Quasi - static analysis for the case with geotextile)

10.2.5 Unpaved Roads without Geotextiles under Traffic Loading (h_0'vs. CBR)

Based on the extensive test programme on unpaved roads (without geotextiles) conducted by the Corps of Engineers, USA, Webster and Alford (1978) established a chart giving the thickness of aggregate layer as a function of number of passes and CBR of the subgrade soil. Failure criterion was 75mm rut depth due to 80kN axle load. Giroud and Noiray have found that the following equation is in good agreement with the chart:

$$h_o = \frac{0.19 \log N_s}{CBR^{0.63}} \qquad \text{---------------(10.10)}$$

where,

h' = thickness of aggregate layer in m, without geotextile, when traffic is taken into account, and

N_s = number of passes of standard 81.7kN axle load.

This equation is made applicable for other axle loads with the help of the equation

$$\frac{N_s}{N_i} = \left(\frac{P_i}{P_s}\right)^{3.95} \qquad \text{---------------(10.11)}$$

where,

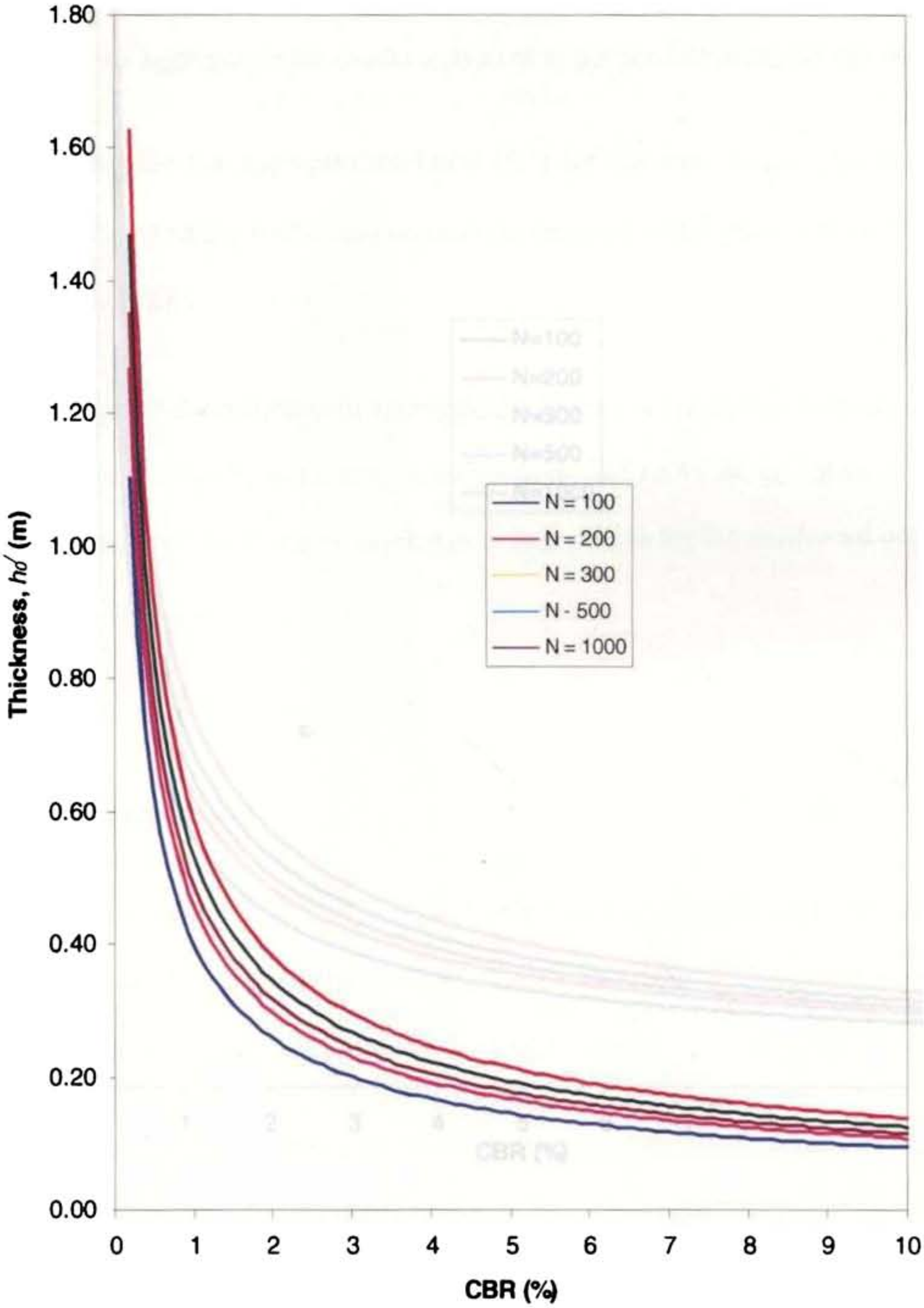
N_s = Number of passages of standard axle load, and

N_i= number of passages of any other axle load Pi.

The terms log N_s in the equation 10.11 can be replaced by (log N_s - 2.34 (r - 0.075)) for any rut values. Thus the equation is modified as

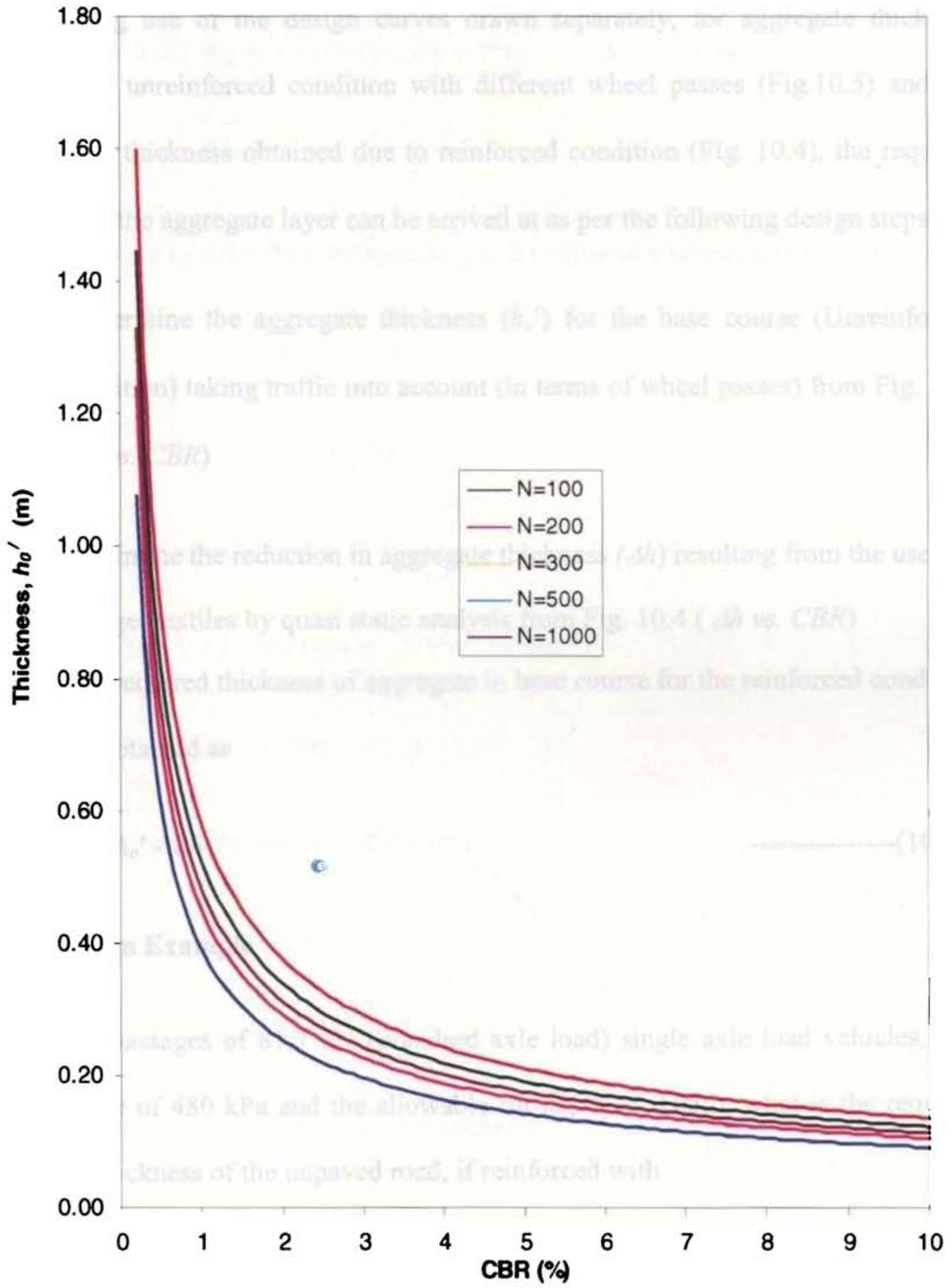
$$h_o' = \frac{0.19\{\log N_s - 2.34(r - .075)\}}{CBR^{0.63}} \qquad \text{-------------(10.12)}$$

Using equation 10.12 charts were prepared for rut depths of 30mm and 50mm for different wheel passes ranging from 100 to 1000, which is the expected CVPD for rural roads, are given in Fig. 10.5 (a) and (b).



(a) rut depth = 30mm

Fig. 10.5 Aggregate thickness h_0' vs. subgrade CBR
(Case without geotextile when traffic is taken into account)



(b) rut depth = 50mm

Fig. 10.5 Aggregate thickness h_0' vs. subgrade CBR
(Case without geotextile when traffic is taken into account)

10.2.6 Design Procedure

Now making use of the design curves drawn separately, for aggregate thickness required for unreinforced condition with different wheel passes (Fig.10.5) and the reduction in thickness obtained due to reinforced condition (Fig. 10.4), the required thickness of the aggregate layer can be arrived at as per the following design steps.

Step 1. Determine the aggregate thickness (h_o') for the base course (Unreinforced condition) taking traffic into account (in terms of wheel passes) from Fig. 10.5 (h_o' *vs. CBR*)

Step 2. Determine the reduction in aggregate thickness (Δh) resulting from the use of coir geotextiles by quasi static analysis from Fig. 10.4 (Δh *vs. CBR*)

Step 3. The required thickness of aggregate in base course for the reinforced condition is obtained as

$$h' = h_o' - \Delta h \qquad \text{---------------(10.13)}$$

10.2.7 Design Example

Given 500 passages of 81.7 kN (Standard axle load) single axle load vehicles, at a tyre pressure of 480 kPa and the allowable rut depth of 0.05m, what is the required aggregate thickness of the unpaved road, if reinforced with

a) Coir geotextile (H2M8) with Es = 64kN/m
b) Coir geotextile (H2M6) with Es = 15kN/m.

Take CBR of subgrade soil as a) 2% and b) 1%

Solution:

Case (a): H2M8 coir geotextile reinforcement

From Fig. 10.5 (b), for N= 500 and CBR = 2%, $h_0' = 0.330$m

and for N = 500 and CBR = 1%, $h_0' = 0.510$m

Similarly from Fig. 10.4 (b) corresponding to Es value of 64kN/m and for CBR = 2%,

$\Delta h = 0.12$m

and for CBR = 1%, $\Delta h = 0.20$m.

Therefore aggregate thickness, $h' = h_0' - \Delta h$

= 0.330 – 0.12 = **0.21m** for CBR = 2% and

= 0.510 – 0.20 = **0.31m** for CBR = 1%

Hence the % saving in aggregate thickness

= 0.12 / 0.330 x 100 = **36%** for CBR = 2% and

= 0.2 / 0.510 x 100 = **39.2%** for CBR = 1%

Case (b): H2M6 coir geotextile reinforcement

From Fig. 10.5 (b) for N= 500 and CBR = 2%, $h_0' = 0.33$m

for N = 500 and CBR = 1%, $h_0' = 0.51$m

Similarly from Fig. 10.4(b) corresponding to Es value of 15kN/m and for CBR = 2%, $\Delta h = 0.10$m and for CBR = 1%, $\Delta h = 0.16$m.

$\therefore h' = h_0' - \Delta h$

$= 0.331 - 0.101 =$ **0.23m** for CBR = 2% and

$= 0.510 - 0.16 =$ **0.350m** for CBR = 1%

The % saving in aggregate thickness

= 0.100/ 0.330 x 100 = **30.5 %** for CBR = 2% and

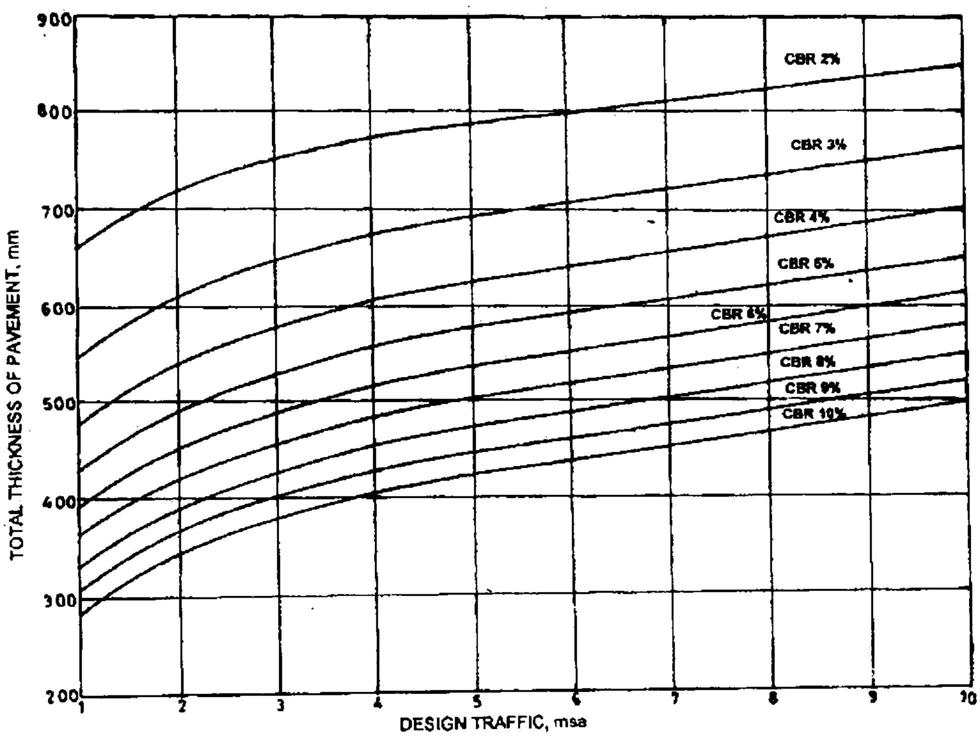
= 0.16/ 0.510 x 100 = **31.4%** for CBR = 1%

Discussion: It could be seen from the design values, aggregate thickness is reduced by about 30% due to the placement of coir geotextiles. Again it was observed that the saving would be more in the case of poor subgrade soils. The H2M8 type geotextiles gave more saving in base course thickness compared to H2M6 coir geotextiles.

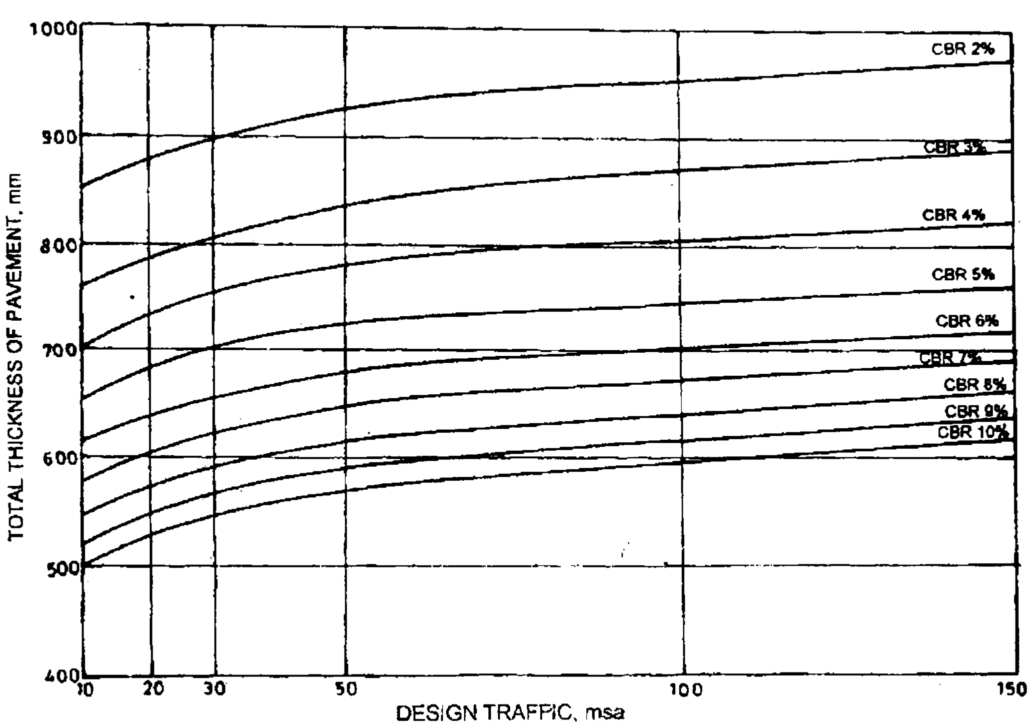
10.3 IRC METHOD

10.3.1 General

The Indian Roads Congress first brought out a guideline for designing flexible pavements in 1970, which was based on empirical method, where the thickness of the pavement was read against the CBR value of the subgrade soil from the design charts. The guidelines were revised in 1984, where traffic was expressed in terms of cumulative standard axle loads. The present design method (IRC: 37 – 2001) is based on the mechanistic pavement design principles, which are evolved from theoretical, laboratory and road performance studies. Design curves and plates are made available corresponding to the CBR of the soil and the traffic conditions (Fig.10.6). For the design of rural roads, the design charts are prepared for low traffic volumes (Fig.10.7)



(a) Traffic 1 – 10 msa



(b) Traffic 10 – 150 msa

Fig. 10.6 Pavement thickness design chart (IRC: 37 – 2001)

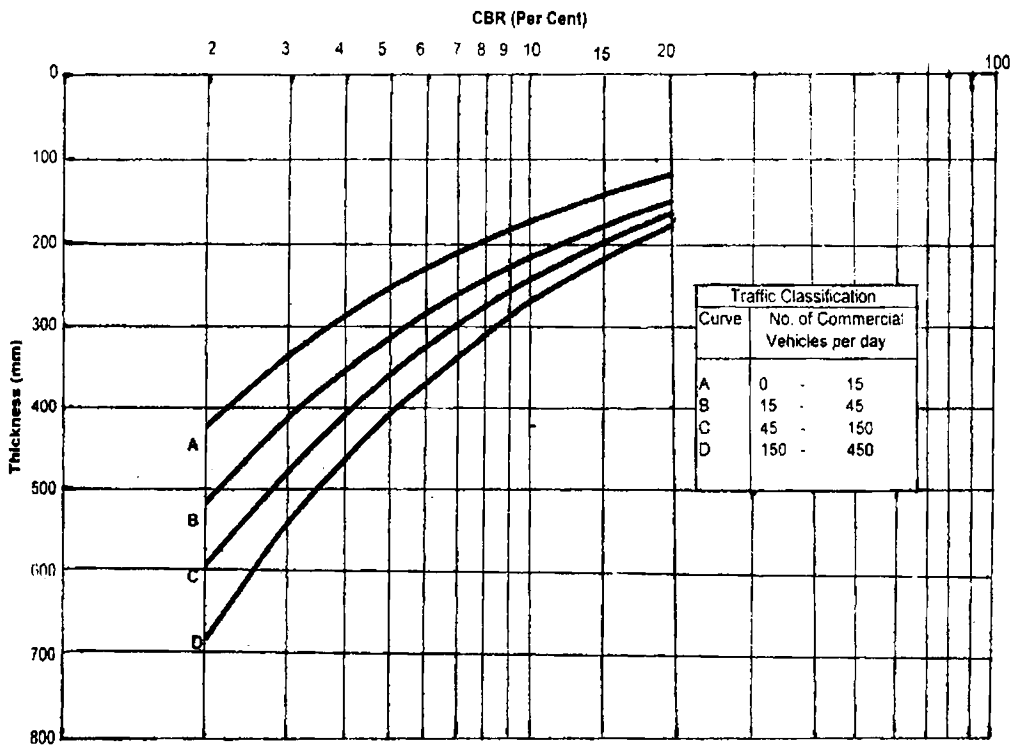


Fig. 10.7 Design curves for rural roads (Rural Roads Manual)

10.3.2 Design Steps

In order to design the roads reinforced with coir geotextiles, which are being placed at subgrade - sub base interface, the CBR of the subgrade soil is determined first. Then the properties of the available coir geotextiles were evaluated. The modified values of CBR due to the provision of coir geotextiles are calculated from the regression equations developed in the present study (section 7.4). The thickness is read from the graphs given in IRC: 37 or Rural Roads Manual.

10.3.3 Design Examples

10.3.3.1 As per IRC: 37 - 2001

Design the pavement for construction of a new by pass with the following data:

Two lane single carriage way

Initial traffic in the year of completion of construction = 400CVPD

Traffic growth rate per annum = 7.5%

Design life = 15 years

Vehicle damage factor = 2.5

Design CBR of subgrade soil = 2%

Type of coir geotextile available = Woven (H2M8 and H2M6) and

NW (AGL C/201)

Design:

Distribution factor = 0.75 (Para 3.3.5 IRC: 37-2001), for two lane single carriage way

Cumulative number of Standard axles to be catered

$$N = 365((1+r)^n - 1)\, Ax\, Fx\, L / r$$

Where,

n = Design life,

r = Traffic growth rate,

A = Present traffic,

F = Vehicle damage factor, and

L = Lane distribution factor

$$N = \{365 \text{ x } [\ (1+0.075)^{15}-1] / 0.075\ \} \text{ x}\{\ 400 \text{ x } 0.75 \text{ x } 2.5\} = 7200000$$

$$= 7.2 \text{ msa}$$

Assuming coir geotextiles are placed at ¼ subgrade thickness,

according to equation 7.3 modified CBR is expressed as

$$CBR_M = 0.550 + 1.249\ CBR - 0.363\ D + 0.156\ \sigma_S$$

With H2M8 coir geotextile, D =3.125cm σ_S = 17.8kN/m

$$CBR_M = 0.550 + (1.249 \times 2) - (0.363 \times 3.125) + 0.156 \times 17.8$$

$$= \mathbf{4.69\ \%}$$

With H2M6 coir geotextile, D =3.125cm and σ_S = 5.0kN/m

$$CBR_M = 0.550 + (1.249 \times 2) - (0.363 \times 3.125) + 0.156 \times 5$$

$$= \mathbf{2.7\%}$$

With NW coir geotextile

$$CBR_M = 1.036 + (1.025 \times 2) - (0.215 \times 3.125)$$

$$= \mathbf{2.4\%}$$

The values of total pavement thickness obtained from Fig. 10.6 (a) for unreinforced and reinforced cases are as follows:

h = **815mm** (without geotextile)

= **625mm** (with H2M8 coir geotextile)

= **750mm** (with H2M6 coir geotextile)

= **765mm** (with NW coir geotextile)

10.3.3.2 As per Rural Roads Manual

A rural road pavement is to be designed for a subgrade CBR of 2% with an initial traffic of 30 commercial vehicles per day in both directions, with a growth rate of 6% per annum for a location having annual rainfall of 1200mm. The design life is 10 years. Determine the thickness of the pavement required if the subgrade is reinforced with i) woven H2M8 ii) woven H2M6 and iii) Non - woven AGL C/201.

Solution:

Initial traffic in both directions = 30 CVPD

Design Traffic = $30 \times (1+0.06)^{10}$ (section 5.2.4 IRC:SP: 20-2002)

= 54 CVPD

For the modified CBR corresponding to original CBR of 2% as obtained in the previous section, using curve C (corresponding to 45 -150 CVPD) of Fig.10.7, the pavement thickness using different coir geotextiles are as follows:

Designed pavement crust thickness = **590mm** (without geotextile)

= **350mm** (with H2M8 coir geotextile)

= **480mm** (with H2M6 coir geotextile)

= **540mm** (with NW coir geotextile)

10.3.3.3 Discussion

In the design using IRC: 37 the allowable rut depth is considered as 30mm. It could be seen from the design example that when H2M8 coir geotextile is used as reinforcement within the subgrade soil at H/4 depth from the top, (CBR = 2%, 400 CVPD) the thickness of pavement can be reduced to 625mm instead of 815mm. This amounts to 24% saving in thickness of aggregates. The corresponding savings when H2M6 and non-woven were used are 8% and 6% respectively, which are also considerable. But, it would be more advantageous in the case of low volume roads ie. as seen from the example given in Section 10.3.3.2 in which the traffic is 30 CVPD and CBR is 2%, the saving when H2M8 reinforcement used is 40% whereas it is only 19% and 9% respectively for H2M6 and Non-woven. Further, it can be concluded that though IRC: 37 – 2001 permits only subgrades with a minimum CBR of 2%, we could easily enhance the CBR value of weaker subgrades by the provision of coir geotextiles. In the case of rural roads, the percentage reduction in thickness seems to be more. With H2M8 coir geotextile the saving in thickness is 41% whereas for H2M6 it is only 19%.

10.4 US ARMY METHOD

10.4.1 General

This method involves the determination of CBR in the laboratory on soil - aggregate system to model the pavement as described in section 7.3. The aggregate thickness is calculated from the equation

$$h = (3.24\log C + 2.21)\left(\frac{P}{36CBR} - \frac{A}{2030}\right)^{0.5} \qquad \text{---------------}10.14$$

where,

h = aggregate thickness in mm,

C = traffic in terms of coverage,

P = ESWL, and

A = tyre contact area in mm^2.

For equation 10.13 design curves were drawn as given in Fig. 10.8.

10.4.2 Design Steps

With the available coir geotextiles placed in between subgrade soil and aggregate layer, CBR of the soil - aggregate system was determined as described in section7.4. From the observations, it was concluded that the modified CBR in the soaked condition for the soil-aggregate system is approximately 35% more than the CBR of the soil and hence the determination of CBR of the soil-aggregate system can be eliminated. For the modified CBR obtained, the thickness of the pavement can be directly read from the graph (Fig 10.8).

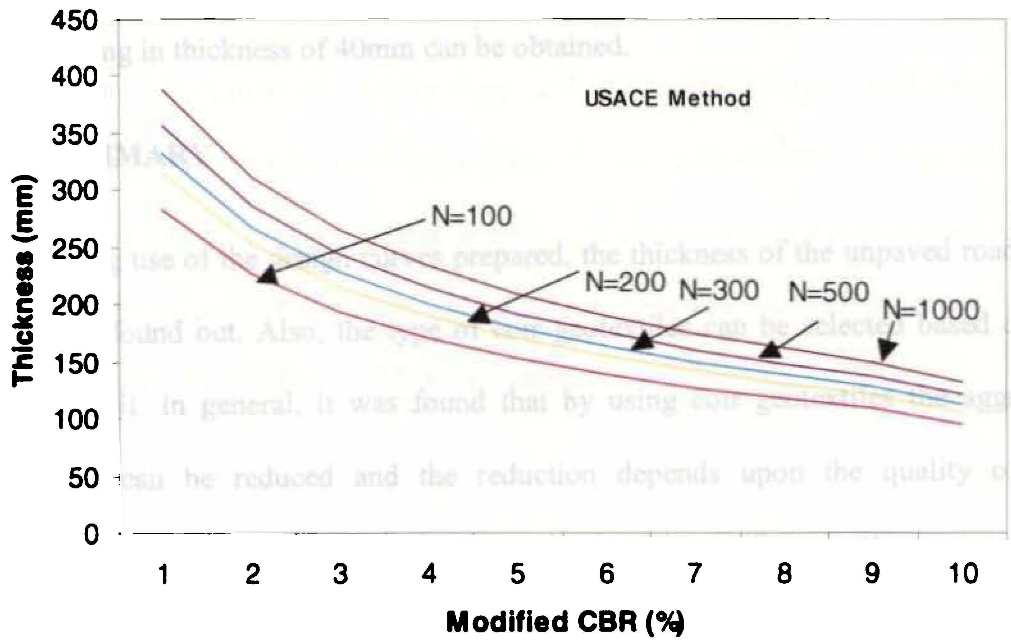


Fig.10.8 Design curves using US Army method

10.4.3 Design Example

Using the US Army Corps of engineers modified CBR design method, calculate the required stone base thickness for an unpaved road carrying 500 coverages of 45kN ESWL using a tyre contact area 450mm x 300mm for

a. Subgrade soil with soaked CBR = 1% with no geotextile reinforcement and

b. The same condition but with a coir geotextile.

Solution:

From Fig.10.8, for CBR = 1%

The thickness of the unpaved road section without reinforcement = 360mm

Modified CBR = 1x 1.35 = 1.35%

The thickness corresponding to modified CBR = 320mm

Thus saving in thickness of 40mm can be obtained.

10.5 SUMMARY

By making use of the design curves prepared, the thickness of the unpaved roads can be easily found out. Also, the type of coir geotextiles can be selected based on the type of soil. In general, it was found that by using coir geotextiles the aggregate thickness can be reduced and the reduction depends upon the quality of coir geotextile, properties of soil and placement depth of the reinforcement.

CHAPTER 11

PREFABRICATED COIR GEOTEXTILE VERTICAL DRAINS

11.1 INTRODUCTION

Construction of unpaved roads in rural areas in many of the cases are accomplished by forming embankments along the banks of the paddy fields or swampy areas and hence the construction of roads and embankments are so closely interconnected that one may not be able to visualise the two constructions independently. One of the major difficulties in these constructions is the presence of saturated soft clay and only very poor soil will be available for construction and hence some sort of ground improvement techniques should be resorted to facilitate speedy and uniform stabilisation of the soft soil.

Of the various methods of ground improvement techniques available, pre-loading is the most successful one. The main disadvantage of this method is that, the time required for consolidation is very long and also the surcharge load required is significantly high. In many cases to pace with the speed of construction activities pre - loading may not be always a viable solution. In such cases, the presence of vertical drain can greatly reduce the pre - loading period. Installation of vertical drains results in the reduction of the length of drainage path in radial direction.

Since it is obvious that the coefficient of consolidation in the horizontal direction is much higher than that in the vertical direction, and that the vertical drains reduce the drainage path considerably in the radial direction, the effectiveness of vertical drains in accelerating the rate of consolidation and improving the strength of soft soil is remarkably improved. In this chapter, the discussion on the use of coir geotextiles in accelerating the consolidation process, by way of prefabricated vertical drains are intended, utilising the drainage and filtration functions of coir geotextile.

The main reason for using the prefabricated vertical drain is its ability to reach the desired degree of consolidation within a specified time period in which both radial and vertical consolidation will be considered in calculating the settlement. Most of the prefabricated vertical drains used for ground improvement applications are of polymeric type. These are costly and not eco- friendly also. The main disadvantage of polymeric type of drains is that its capacity may be effectively surplus to the requirements. In such situations, deliberate and designed use of geosynthetic, which has a predictable reduction in capacity with time, is a good engineering solution. With increasing environmental awareness and sustainability, along with high cost of petroleum products, developing countries lead to investigations of substitutes for polymeric materials, using natural products.

In the present study two types of prefabricated vertical drains using coir geotextiles were developed and their effectiveness were studied by conducting experiments as detailed in the following section.

11.2 APPLICATION OF VERTICAL DRAINS

Vertical drains are artificially created drainage paths which can be installed by one of the several methods and which can have a variety of physical characteristics. The use of vertical drains along with pre - compression has the sole purpose of shortening the drainage path of the pore water, thereby accelerating the rate of primary consolidation. When used in conjunction with pre - compression, the principal benefits of a vertical drain system are:

- To decrease the overall time required for completion of primary consolidation due to pre - loading.
- To decrease the amount of surcharge required to achieve the desired amount of pre compression in the given time.
- To increase the rate of strength gain due to consolidation of soft soils where stability is of concern

The following characteristics of prefabricated vertical drain (PVD) should be considered during the design and construction.

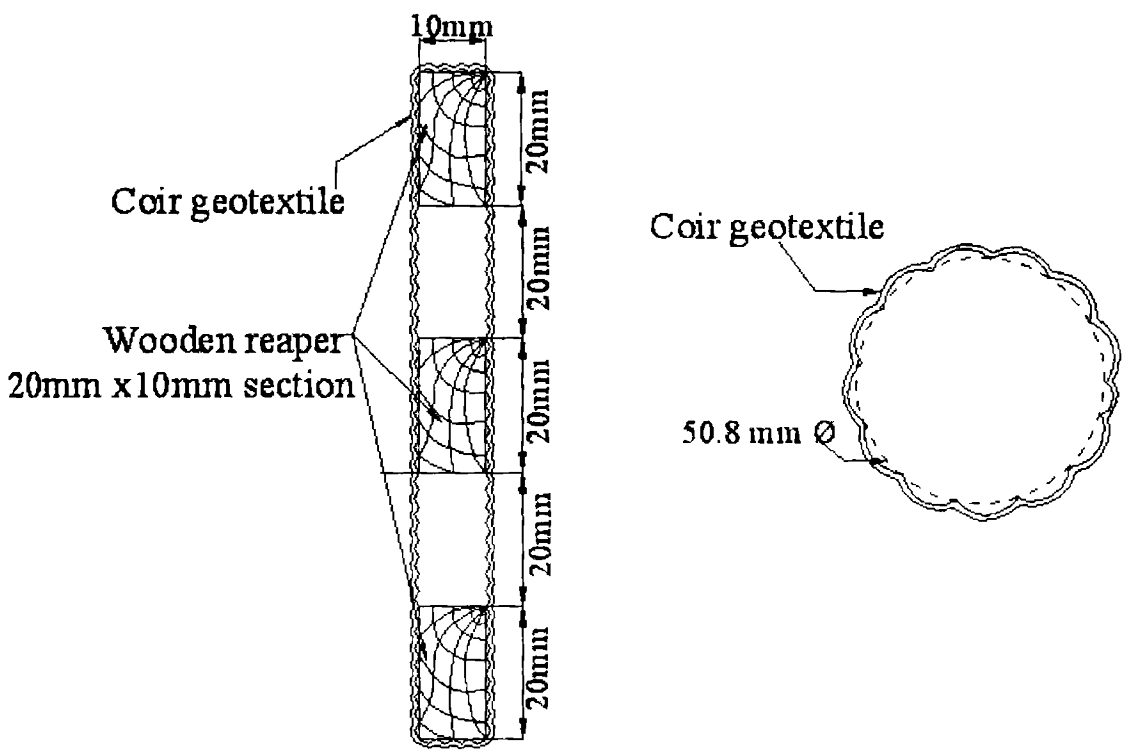
- Ability to be installed vertically into compressible subsurface soil strata under field conditions,
- Ability to permit pore water in the soil to seep into the drain,
- A means by which the collected pore water can be transmitted up and down the length of the drain

The use of PVDs has largely replaced the vertical sand drains for many applications. The important advantages are economic competitiveness, less disturbance and the speed and simplicity of installation.

11.3 PREPARATION OF COIR GEOTEXTILE DRAINS

Two types of coir geotextile drains were developed in the present research work. One is of circular type and the other is of rectangular type. The circular drains were made by wrapping the coir geotextiles twice over 50.8 mm diameter rigid PVC pipes. To keep the geotextiles in position, it is tied by binding wires at 200 mm to 250 mm intervals. For easy penetration of these drains a perforated metallic cone was made.

To make a rectangular type of drain, three wooden reapers of 20 mm x 10 mm (this can be even bamboo strips or waste wood cuttings) are placed at 20 mm clear gap between them. This is glued to coir geotextiles and wrapped all around in four layers. The ends are glued to avoid separation. Perforated metallic V - shaped shoes is placed at the ends at the time of installation to facilitate easy penetration. The cross sections of the drains are shown in Fig.11.1. Drains were made with two varieties of coir geotextiles designated as H2M8 and H2M6.



(a) Rectangular drain **(b) Circular drain**

(dimensions not to scale)

Fig. 11.1 Cross section of coir geotextile drains

11.4 TESTING PROGRAMME

The experimental work is aimed at finding the reduction in time for the consolidation settlement due to the provision of coir geotextile drains in loose and sensitive soils.

Though not impossible, it is very difficult to mobilise the equipment and to conduct tests in the field. To by- pass these difficulties and also to have controlled conditions, experiments were done in a test tank fabricated for the purpose in the laboratory. The four series of experiments conducted are tabulated in Table 11.1. The disposition of drains within the test tank is shown in Fig. 11.2.

Table 11.1 Summary of experiments conducted

Series	Pattern of arrangement	Type of Drain	Type of coir geotextile
I	No drain	-	-
II	Single drain at centre	Circular	H2M6
		Circular	H2M8
		Rectangular	H2M6
		Rectangular	H2M8
III	Three drains in Triangular pattern	Circular	H2M6
		Circular	H2M8
		Rectangular	H2M6
		Rectangular	H2M8
IV	Four drains in Rectangular pattern	Circular	H2M6
		Circular	H2M8
		Rectangular	H2M6
		Rectangular	H2M8

A single central drain, three drains in triangular pattern and four drains in rectangular pattern were tried.

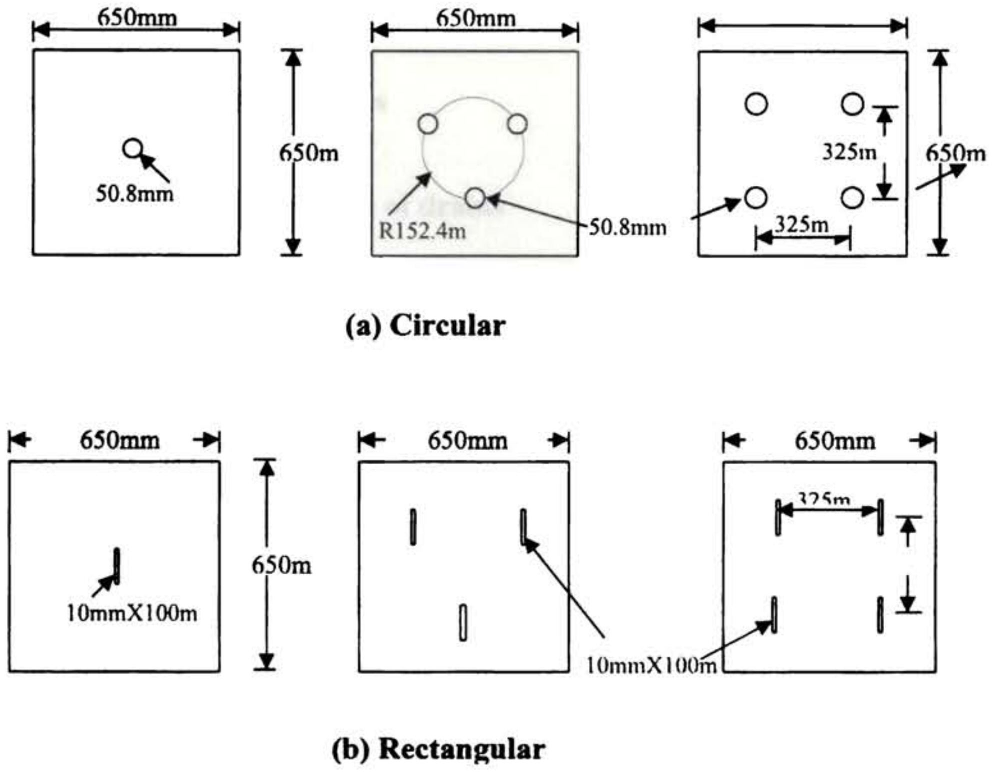


Fig. 11.2 Disposition of coir geotextile vertical drains

11.4.1 Preparation of Test Set - up

A steel tank of size 650mm x 650mm x 850mm was fabricated using mild steel plates and mild steel angles. Bracings were provided on the sidewalls in the diagonal directions to prevent buckling of the plates during loading. Inside of the tank was painted with glossy metallic paint to have a smooth surface, in order to nullify the friction between the soil and inner surface of the tank. Clayey silt was used for this test programme. The soil was soaked in water for 3 days and made into a thick slurry form. The slurry was transferred into the tank in small quantities and was stirred well to remove the entrapped air to the maximum possible. This was continued until soil is filled in the tank to a depth of 700 mm. Three days rest period was given before commencing the experiment.

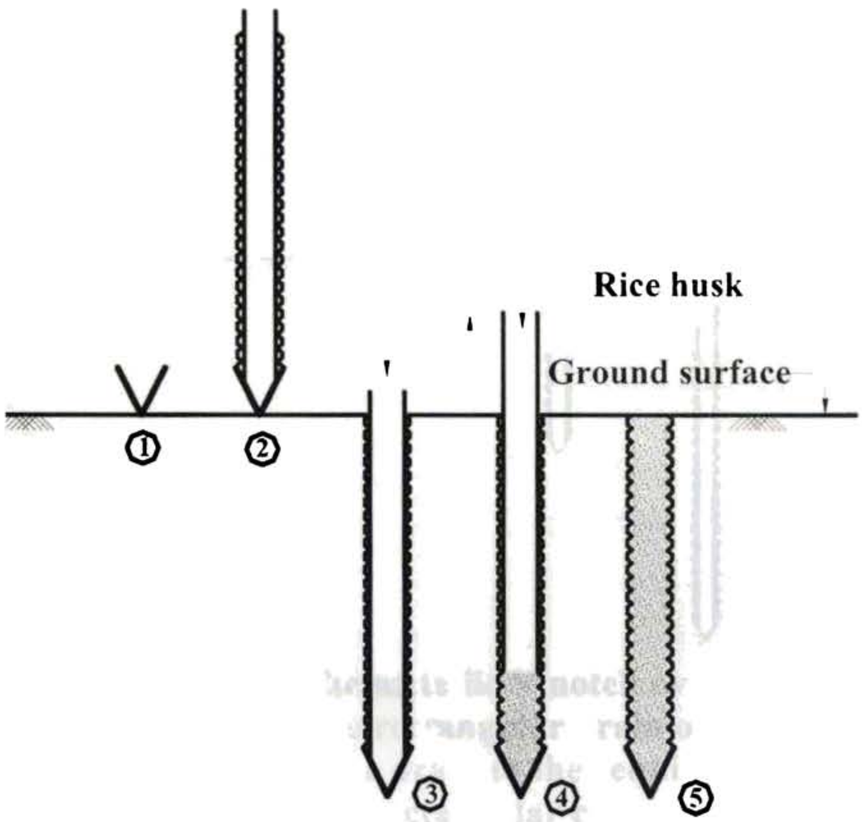
11.4.2 Installation of Vertical Drains

11.4.2.1 Installation of circular type of drains

To install circular type coir geotextile vertical drains, first the perforated metallic cone was placed in the required position and the drain wrapped over PVC pipes were held in vertical position over it. The drains were pressed into the soil to the required depth (50 mm from the bottom of the tank). After reaching the required depth the PVC pipe was withdrawn slowly and simultaneously rice husk was added to the hole. The step-by-step procedure was continued until the drain was installed. Fig.11.3 illustrates the sequence of installation of circular vertical drains.

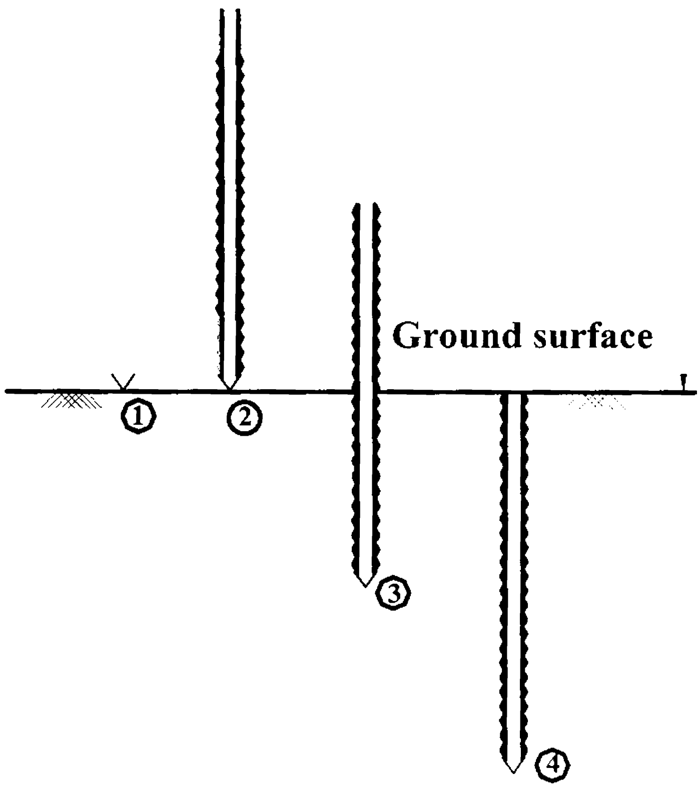
11.4.2.2 Installation of rectangular type of coir geotextile drains

To install the rectangular type of coir geotextile drains, a perforated metallic V - shaped shoe was placed over the soil surface. The drain was placed centrally in the groove and pressed to the required depth. The sequences of installation of rectangular drains are illustrated in Fig.11.4.



1. Placing the metallic cone shoe at the surface.
2. Placing the coir geotextile drain wrapped around PVC pipe centrally over it.
3. Pressing the drain to the required depth.
4. Releasing the PVC pipe and simultaneously adding rice husk.
5. Completed circular coir geotextile drain.

Fig. 11.3 Installation of circular type coir geotextile vertical drains



1. Placing the metallic V notch over the soil surface.
2. Placing the rectangular drain over it.
3. Pressing the drain to the required depth.
4. Completed rectangular coir geotextile drain.

Fig. 11.4 Installation of rectangular type coir geotextile vertical drains

11.4.3 Testing

After installing the drains in the desired configuration the surface of the soil was leveled and a filter paper was placed over it covering the entire soil surface. Above this 5 mm thick perforated steel plate was placed over it. Two layers of coir geotextiles were placed over this, which act as the drainage blanket. Above this a 38 mm thick perforated metallic plate was placed to distribute the load evenly over the surface. The plates were made perforated for the easy escape of water and hence to avoid building up of pore water pressure. Additional load was placed on the top of the steel plates by putting mild structural steel sections. The details of test set-up are shown in Fig.11.5. The tests were performed under a pressure of 10 kPa, for which a total load of 3.60 kN was applied. The loading was done with the help of a tripod and a differential pulley arrangement.

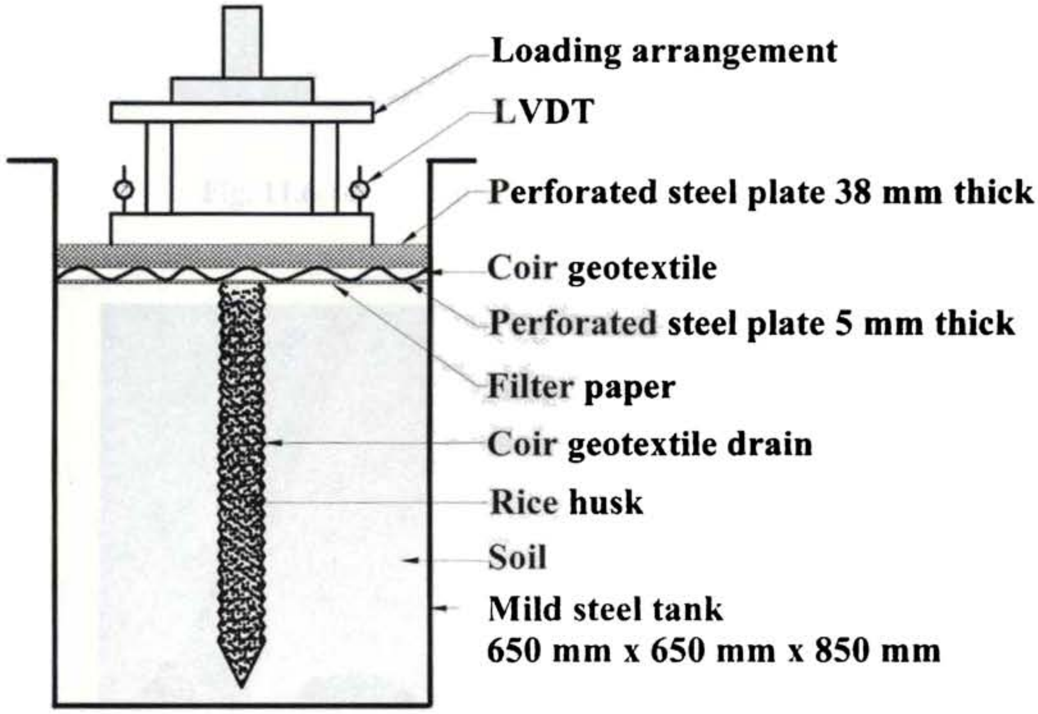


Fig. 11.5 Schematic test set - up for vertical drain

The settlement measurements were taken with the help of four digital displacement sensors having sensitivity of 0.01 mm and the average value is recorded. Settlements were taken at varying time intervals until the settlement is nearly constant. Photographs showing testing sequences are given in Fig.11.6.

Fig. 11.6 (a) tank filled with soil slurry

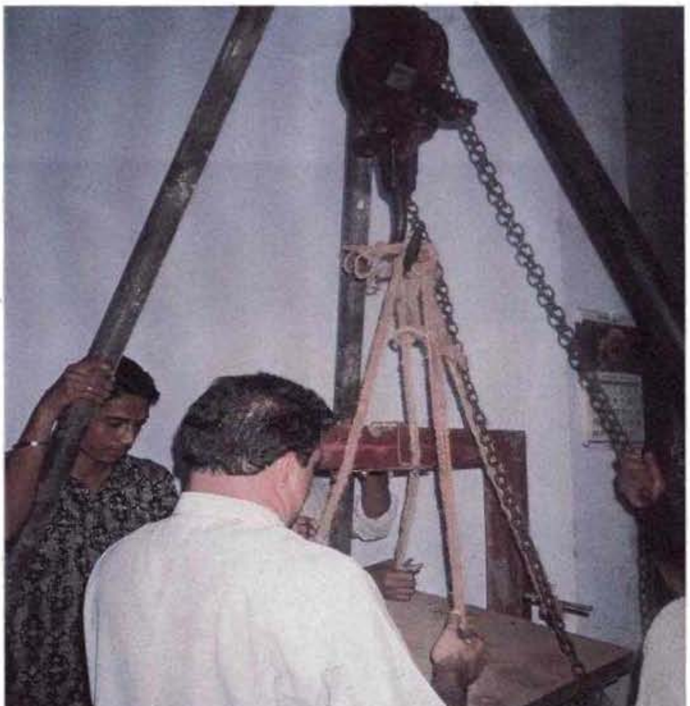


Fig. 11.6 (b) Perforated steel plate 38 mm thick being placed

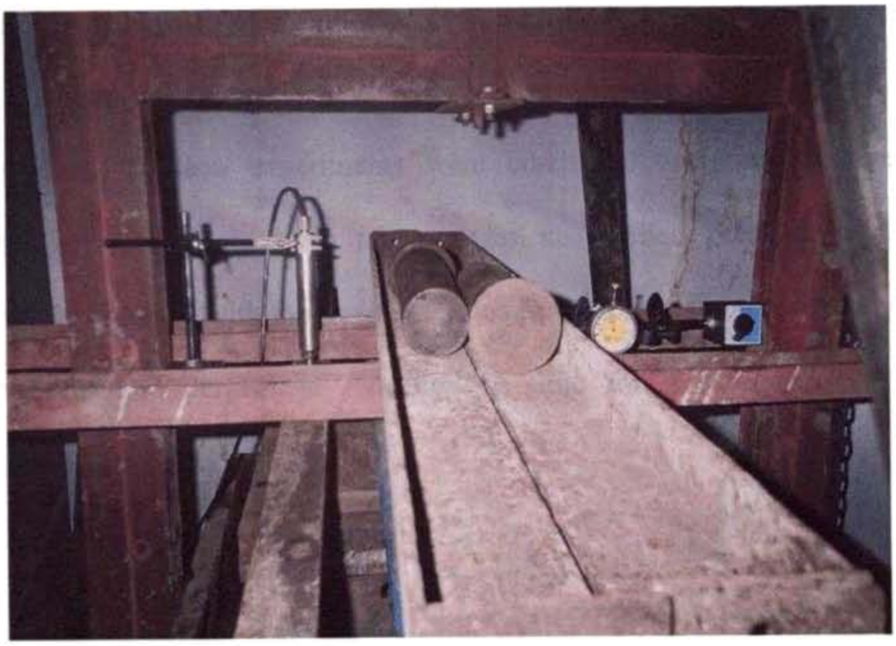


Fig. 11.6 (c) Set - up is ready for taking readings

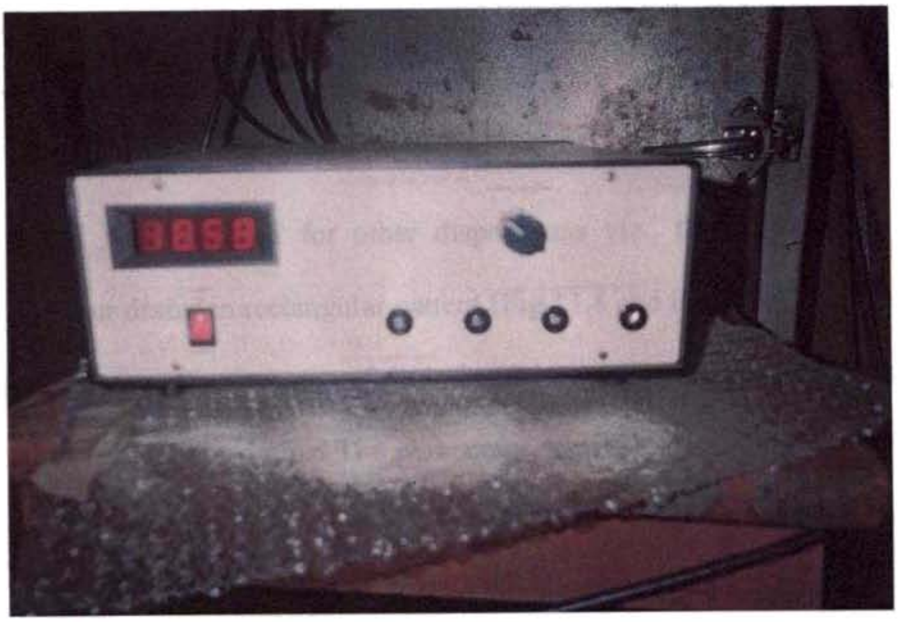


Fig. 11.6 (d) Noting down the readings

Fig. 11.6 Sequence of testing programme

11.5 RESULTS AND DISCUSSION

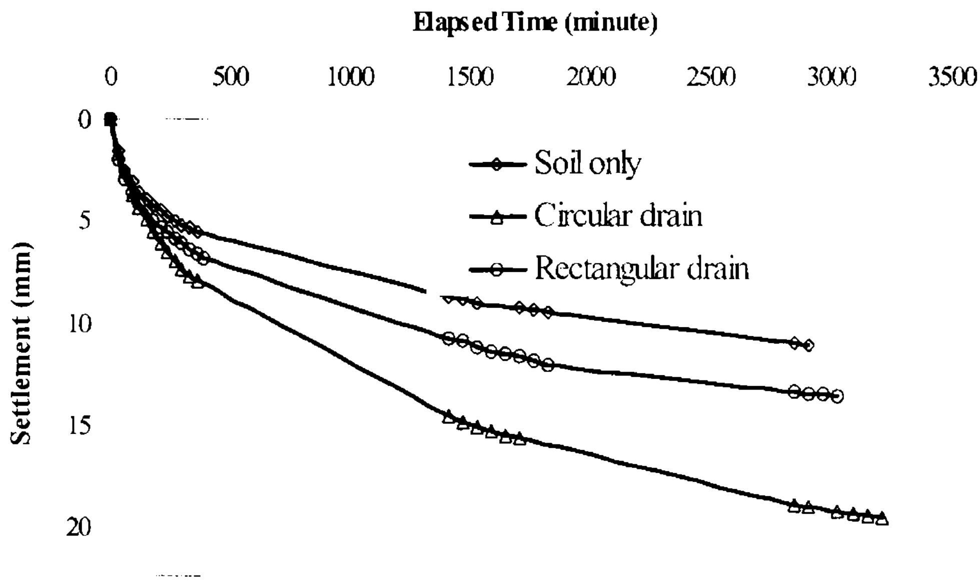
The behaviour of two types vertical coir geotextile drains made of two varieties of woven coir geotextiles in different configurations in reducing the time of settlement were studied. Thirteen experiments were conducted and primary consolidation settlement measurements were taken under an applied pressure of 10kPa.The performance obtained with the provision of coir drains in terms of percentage increase in settlement and percentage reduction in time for a settlement of 10mm is summarised in Table11.2.

11.5.1 Type of Drain

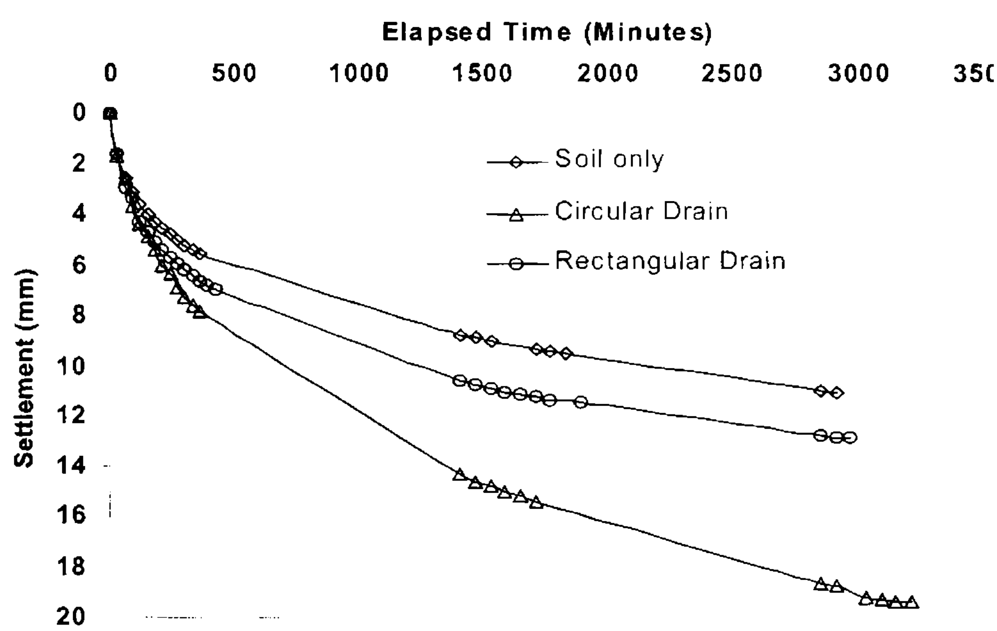
Two types of drains were considered, one is of circular cross section and the other is of rectangular cross section. For the circular type the central core pipe is withdrawn during installation whereas in the case of rectangular drains the full section is driven and kept as such in the soft ground. Fig.11.7 shows the comparison of the time settlement behaviour for single drain (made of H2M8 and H2M6 coir geotextiles) at centre. Here it is seen that the drain with circular cross section is performing better. Similar trend was observed for other dispositions viz., three drains in triangular pattern and four drains in rectangular pattern (Fig.11.8 and Fig.11.9). This may be due to the fact that the effective area of drain is 50 % more in the case of circular drains compared to rectangular drains. The percentage increase in settlement after 48 hours for the circular drains when compared to rectangular drains is 25 to 55%. Another reason which favours the circular type may be that the drain was filled with rice husk which is a free draining material. Again, while making rectangular drains, four wraps were made to get a stable workable drain whereas, two windings only were required in the case of circular drains to get a self-supporting drain. This was possible because rice husk was added while PVC pipe was withdrawn.

Table 11.2 Performance comparison of drains in terms of settlement and time

Expt . No.	Description	% Increase in settlement in 24 hours	% Increase in settlement in 48 hours	% Reduction in time for 10 mm settlement
1	Soil without drain	-	-	-
2	Circular drain (H2M8) – 1 at centre	64.77	72.73	70.23
3	Circular drain (H2M6) – 1 at centre	64.77	68.18	67.44
4	Circular drain (H2M8) – 3 Nos. – Triangular pattern	112.5	122.73	81.86
5	Circular drain (H2M6) – 3 Nos. – Triangular pattern	64.77	78.18	71.63
6	Circular drain (H2M8) – 4 Nos. – Rectangular pattern	76.14	81.82	74.42
7	Circular drain (H2M6) – 4 Nos. – Rectangular pattern	64.77	68.18	66.51
8	Rectangular drain (H2M8) – 1 at centre	23.86	21.82	43.26
9	Rectangular drain (H2M6) – 1 at centre	21.59	17.27	43.72
10	Rectangular drain (H2M8) – 3 Nos. – Triangular pattern	50.00	45.45	69.77
11	Rectangular drain (H2M6) – 3 Nos. – Triangular pattern	40.91	34.54	61.86
12	Rectangular drain (H2M8) – 4 Nos. – Rectangular pattern	42.05	38.18	62.33
13	Rectangular drain (H2M6) – 4 Nos. – Rectangular pattern	40.91	34.54	55.81

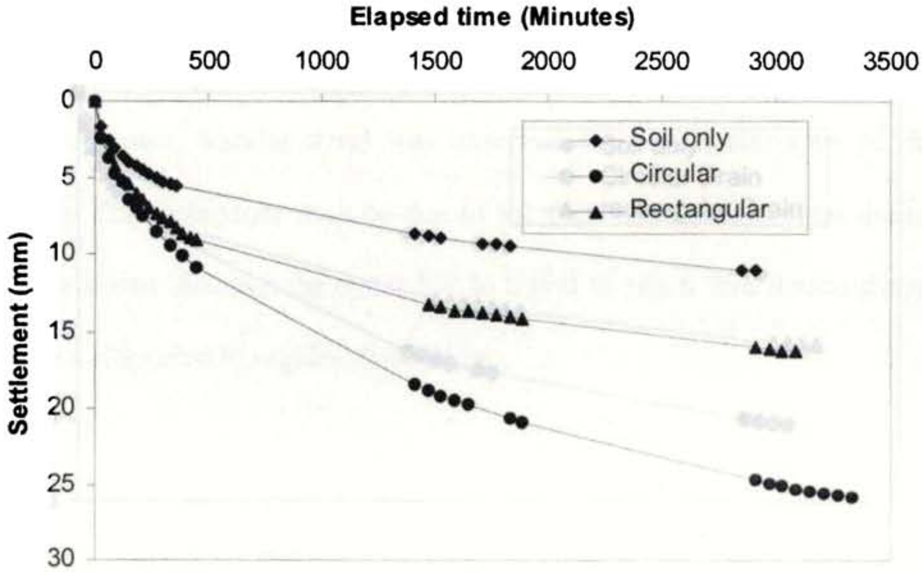


(a) H2M8 geotextile

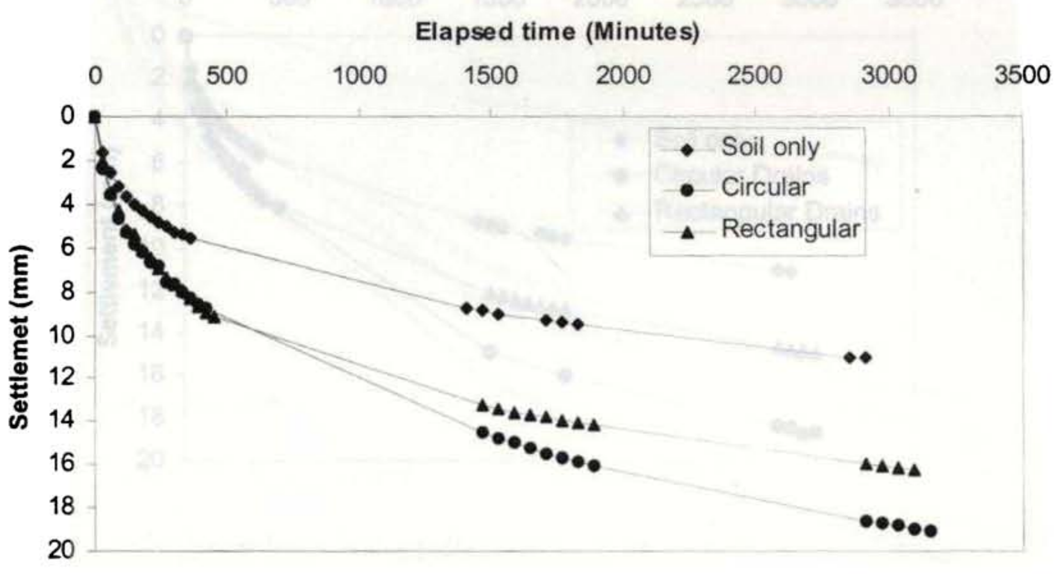


(b) H2M6 geotextile

Fig. 11.7 Effect of type of drain on the time settlement behaviour (single drain at center)

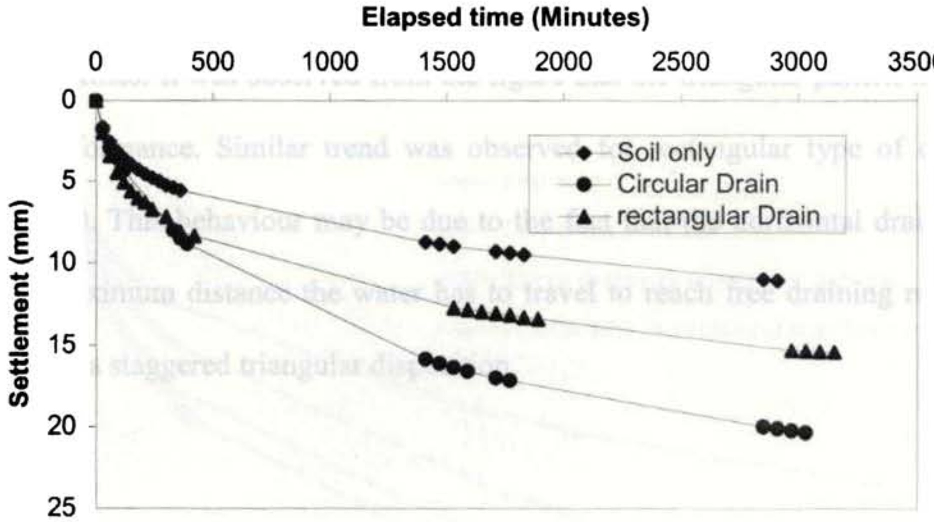


(a) H2M8 geotextile

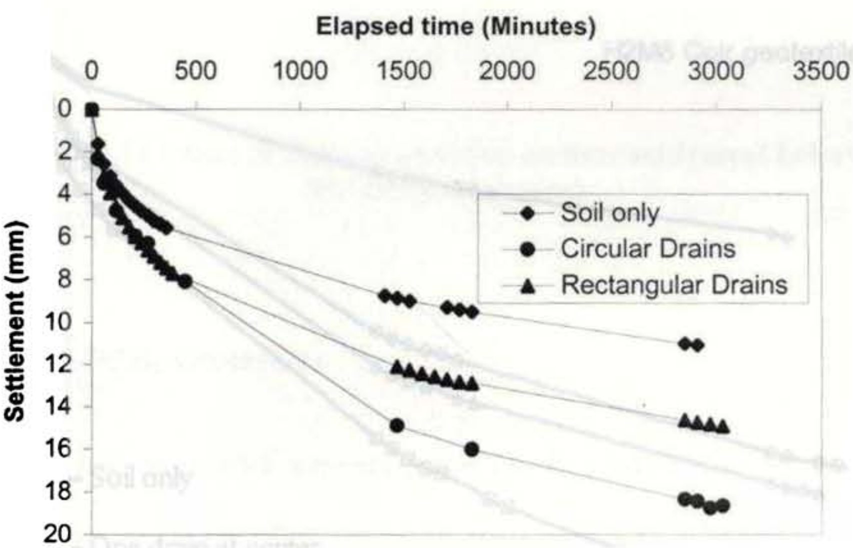


(b) H2M6 geotextile

Fig.11.8 Effect of type of drain on the time settlement behaviour. (Three drains- Triangular disposition)



(a) H2M8 geotextile



(b) H2M6 geotextile

Fig. 11.9 Effect of type of drain on the time settlement behaviour. (Four drains- Rectangular disposition)

11.5.2 Pattern of Arrangement

In order to study the effect of layout of drains, three different patterns were tried, viz., one at centre, three drains in a triangular pattern and four drains in a rectangular pattern. Fig.11.10 shows the results of a typical case of circular drains made of H2M8 coir geotextiles. It was observed from the figure that the triangular pattern is showing better performance. Similar trend was observed for rectangular type of drain also (Fig.11.11). This behaviour may be due to the fact that the horizontal drainage path i.e. the maximum distance the water has to travel to reach free draining medium, is reduced in a staggered triangular disposition.

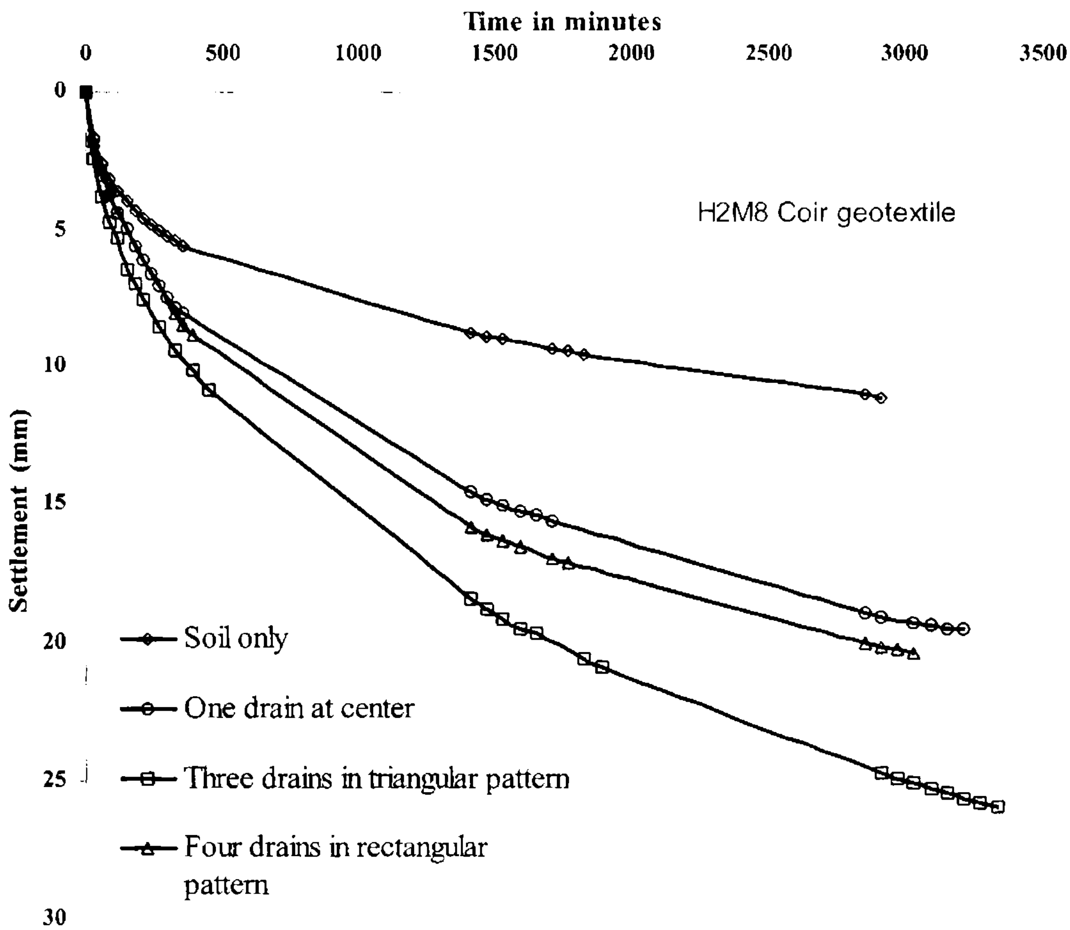


Fig. 11.10 Effect of drain disposition on time settlement behaviour

(Circular drains)

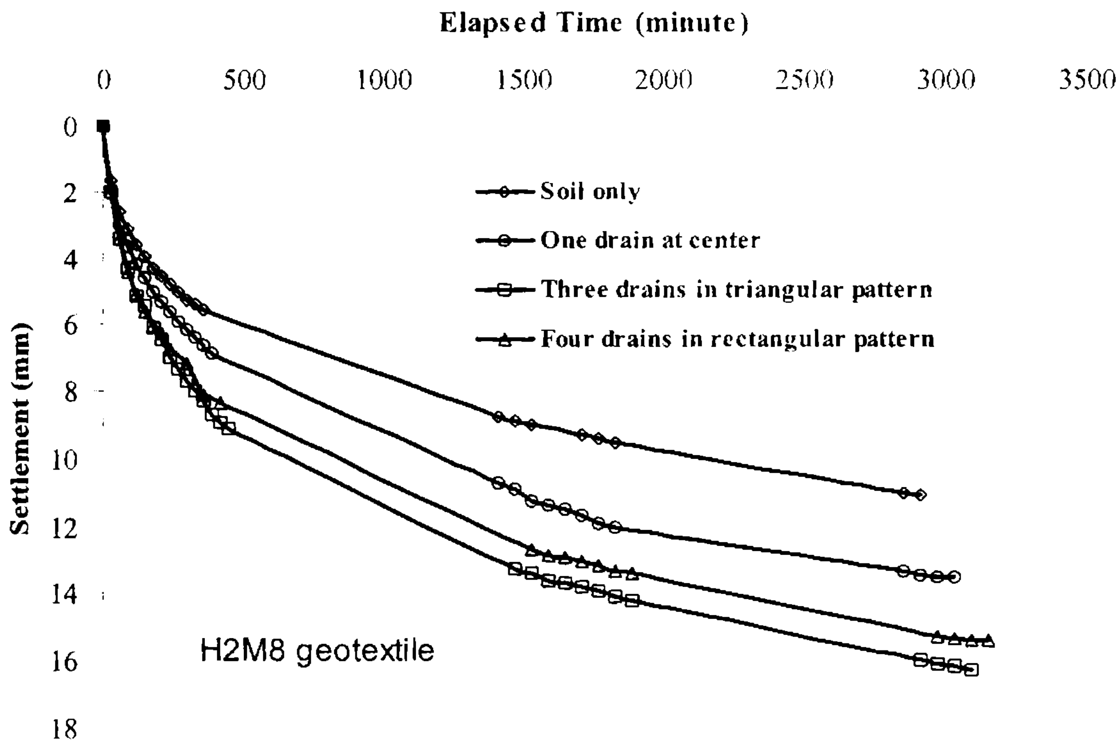


Fig. 11.11 Effect of drain disposition on time settlement behaviour (Rectangular drains)

11.5.3 Types of Coir Geotextiles

Two types of coir geotextiles were employed for the study. A set of six experiments each were done on drains made up of H2M8 and H2M6 geotextiles. Figs. 11.12, 11.13, and 11.14 compares the time settlement behaviour of drains made of two types of coir geotextiles in different dispositions. When single drain is used both H2M8 and H2M6 performed more or less in the similar way as we can see from Fig.11.12. But in all other cases it is seen that drains made of H2M8 is showing better performance in terms of reduction in consolidation time. The H2M6 geotextiles has wide opening and less rigidity. Being more flexible, kink formation may be more while using H2M6 and hence the poor performance.

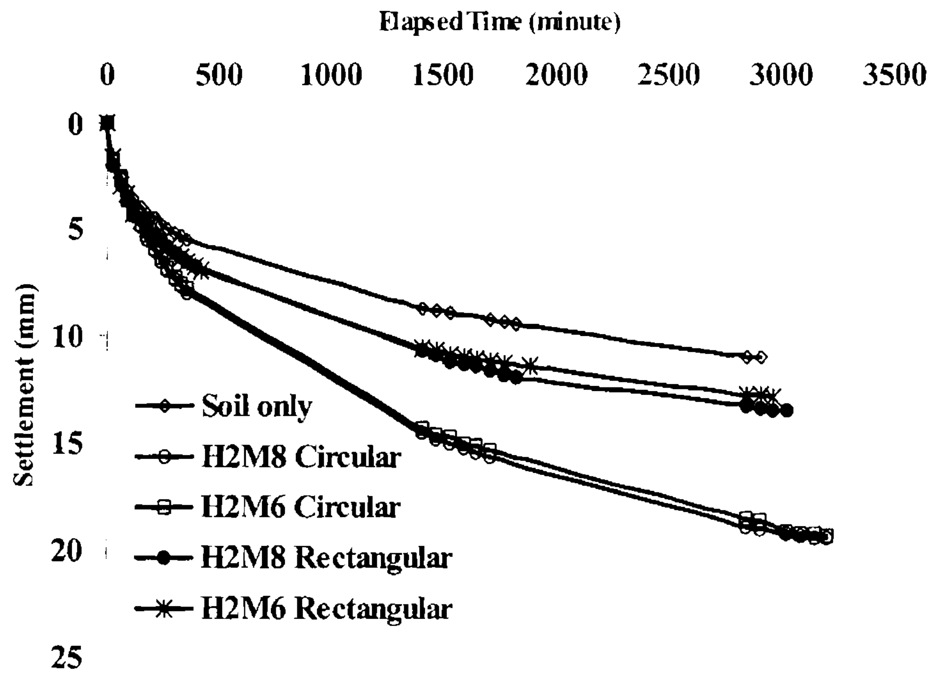


Fig. 11.12 Influence of type of coir geotextiles on the behaviour of drains (One at centre)

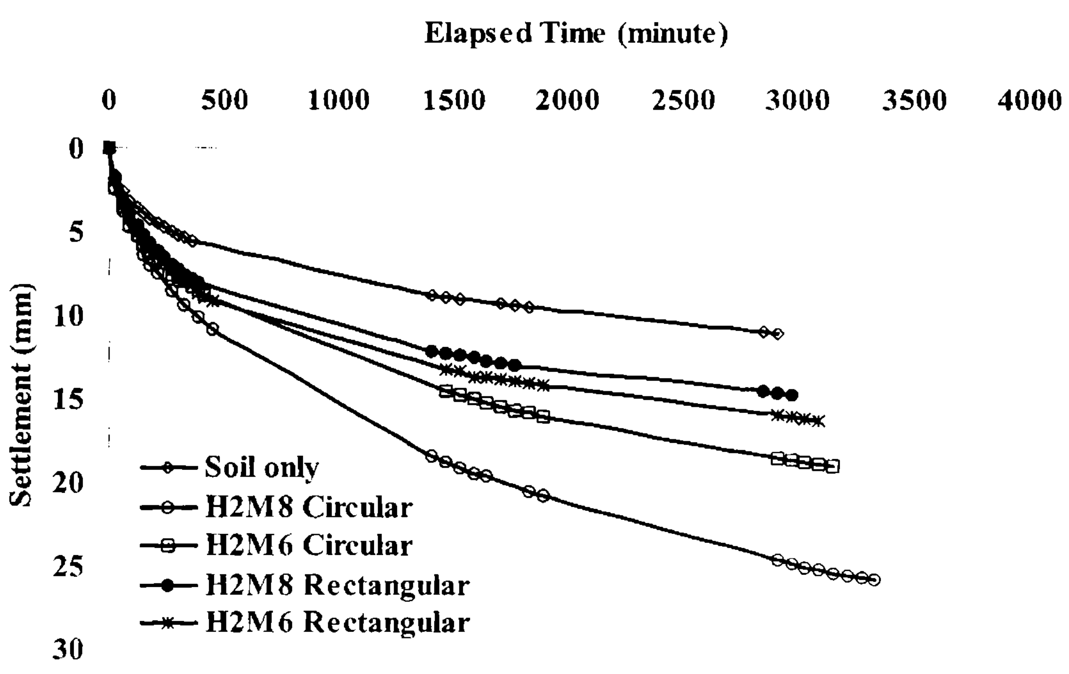


Fig. 11.13 Influence of type of coir geotextiles on the behaviour of drains

(Triangular layout)

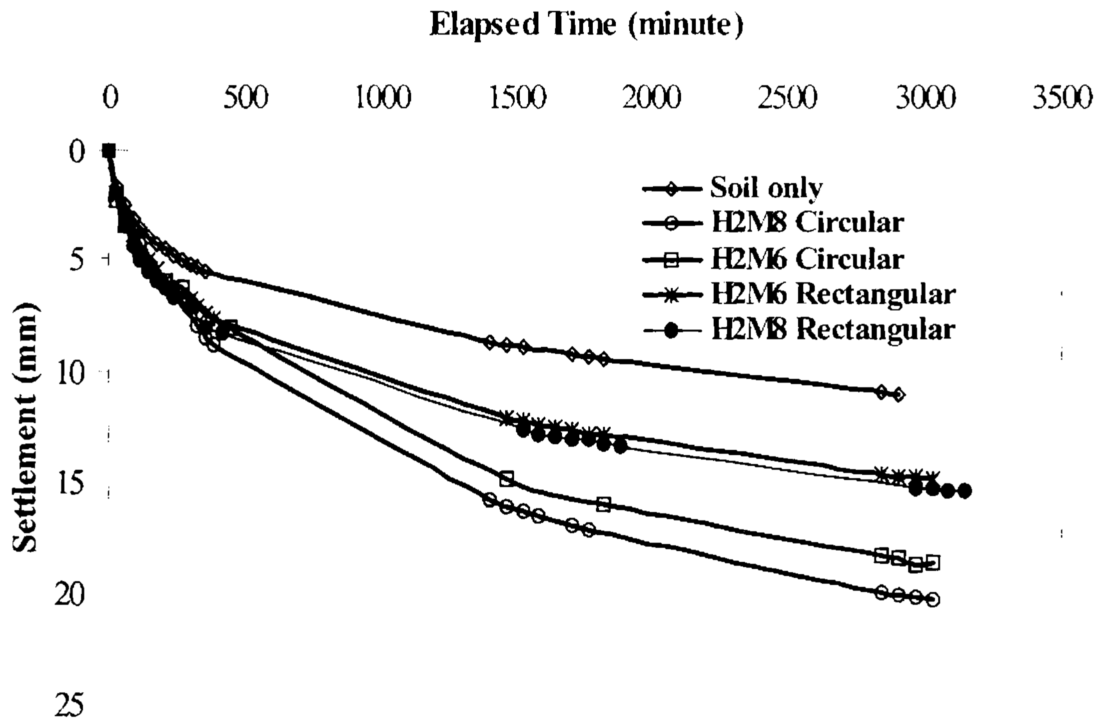


Fig. 11.14 Influence of type of coir geotextiles on the behaviour of drains (Rectangular layout)

11.6 SUMMARY

In general, the time for consolidation is very much reduced due to the provisions of circular and rectangular coir drains in all the configurations tested. While comparing the performance of the types of coir drains, it is seen that drains using H2M8 performed better than that of H2M6 coir geotextiles. Also the triangular configuration is found to be more efficient than the other patterns tried. Much better result was obtained with circular type of drains compared to rectangular type of drains.

CHAPTER 12

CHARACTERISATION OF SOIL - FIBRE COMPOSITE

12.1 GENERAL

When coir geotextiles are used for the ground improvement either for strengthening subgrade or embankment, after its life period of 4 to 5 years, it may be gradually transformed into fibres of varying length due to degradation. The presence of these randomly oriented coir fibres in the soil mass may be considered similar to an admixture for stabilization. More over, these randomly oriented fibres in soil may reduce the drainage path during consolidation process and there by dissipate pore pressure due to the applied load, since the water can find easy way to escape due to the random orientation of the fibres. Hence the study is also aimed to investigate the effect of discrete randomly oriented coir fibres on the performance of pavement supporting soil.

The main idea was to produce a stabilized soil - fibre matrix with enhanced properties relative to virgin soil and establishing the magnitude of inherent variations in strength and compressibility properties. The effect of fibre content and fibre length was investigated by conducting test on soils with and without coir fibres under different moisture and stress environments and to arrive at the optimum values.

Experiments were done with fibre contents up to 2.5% and length up to 20mm. Fibre contents more than 3% was not used in the study due to the difficulty of obtaining an even distribution of the fibre in the soil. Also the amount of fibre that remain in soil after degradation of coir geotextiles is not expected to be more than this value.

12.2 LABORATORY INVESTIGATIONS

12.2.1 Compaction Test

To determine the effect of reinforcing fibres on the moisture – density relationships, standard compaction tests were conducted as per Bureau of Indian Standard specifications on controlled soil (clayey silt and red soil) and soil - fibre mixtures. An even distribution of fibre and soil was achieved by consistent mixing procedure. Compaction was done with the help of an automatic compactor. Specimens were prepared by using both soils mixed with fibre contents of 0%, 0.25%, 0.5%, 1.0% and 2% by weight with different fibre lengths of 10mm, 15mm and 20mm, taking fresh samples each time. It should be noted that the soil - fibre mixes were less workable because of the fibre webbing during mixing.

12.2.2 Unconfined Compression Test

Red soil and Clayey silt soil fractions passing through 425-micron IS sieve were used for the preparation of the soil - fibre composite specimens. Fibres having length 10mm, 15mm and 20mm were mixed with soil in three proportions viz., 0.5%, 1.0% and 2.0% of dry soil by weight. Specimens were prepared at water contents corresponding to 95% of maximum dry density of the soils (dry of optimum and wet of optimum conditions). All specimens tested were of 52 mm diameter and 104 mm height. To prepare the specimens, calculated amount of soil-fibre mixture, were compacted in three equal layers. Before compacting the soil in the mould, the inside of the mould was coated with oil in order to reduce the chance of breaking of specimens during removal from the mould. The soil mixture was pressed in the mould by means of hydraulic jack. Any fibres protruding out at the top and bottom of the specimens were trimmed with scissors.

Loading was done in an AIMIL Unconfined Compression testing machine. The strain was controlled at a rate of 0.5% per minute throughout the testing programme. Peak values of stresses were noted.

12.2.3 Triaxial Shear Test

Soil - fibre mixes were prepared as in the case of UCC test. Specimens of 26mm diameter and 52mm height were prepared in split moulds. Unconsolidated undrained triaxial tests were done at confining pressures of 50 kPa, 100 kPa and 150 kPa. The shear strength parameters (cohesion and angle of internal friction) were found out.

12.2.4 Consolidation Test

Clayey silt soil passing through 425 microns was taken for the test. Dry soil was mixed with known percentage of fibre (0.5%, 1.0%, 2.0% and 3.0%) having specified lengths (10 mm and 15 mm). Water was added and mixed thoroughly to get a uniform randomly oriented saturated soil - fibre mix. The soil - fibre paste was filled in the consolidation ring. Proper care was taken while filling the soil to minimise the entrapped air.

Conventional oedometer tests were done using a standard consolidation ring of 60mm diameter and 20mm height. Readings were recorded before and after placing the load. Each load was kept for 24 hours. Load increment ratio was 1:1. Void ratio for each pressure was calculated and pressure void ratio variations were plotted to study the variations in compression index. Time settlement readings were taken at a pressure of 200 kPa to find Coefficient of consolidation by curve flitting methods.

12.3 RESULTS AND DISCUSSION

All the data obtained from the experimental investigations were analysed to study the magnitude of inherent variations in moisture content-dry density relationship, unconfined compressive strength, triaxial shear strength and compressibility characteristics of soil - fibre matrix. The effect of fibre content and aspect ratio of fibres were analysed to arrive at the optimum values of fibre content and fibre length to get maximum benefit.

12.3.1 Moisture Content Dry Density Relationship

Fig.12.1 shows the variation of optimum moisture content with fibre content for clayey silt soil and red soil blended with coir fibres. It can be seen that, for both soil -

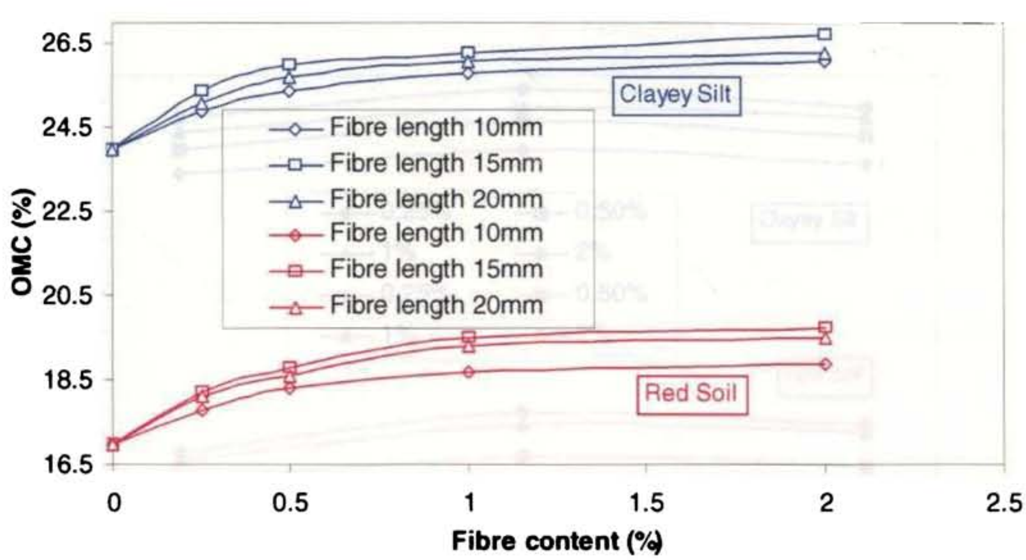


Fig. 12.1 Variation of OMC with fibre content

fibre mixes, as the fibre content increases the optimum moisture content increases irrespective of aspect ratio of fibres. The OMC of clayey silt and red soil without coir fibres was 24% and 17% respectively. Due to addition of 1% fibre the increase in OMC was 7.5%, 9.6% and 8.75% for clayey silt fibre mixes with fibres of length 10mm, 15mm and 20mm respectively. The respective increase in OMC for red soil with 1% fibre content is 10%, 15% and 14% respectively. Though the OMC increases with fibre content for all cases of fibre length considered, it was found that the rate of increase in OMC was not uniform. The maximum increase was for 15mm fibre and the minimum was for 10mm fibre.

Variation of OMC with fibre length for clayey silt and red soil are shown in Fig.12.2. It can be observed that, as the aspect ratio of the fibre increases, OMC increases to a maximum value and then shows a decreasing trend. This behaviour was identical in both soils with all fibre contents.

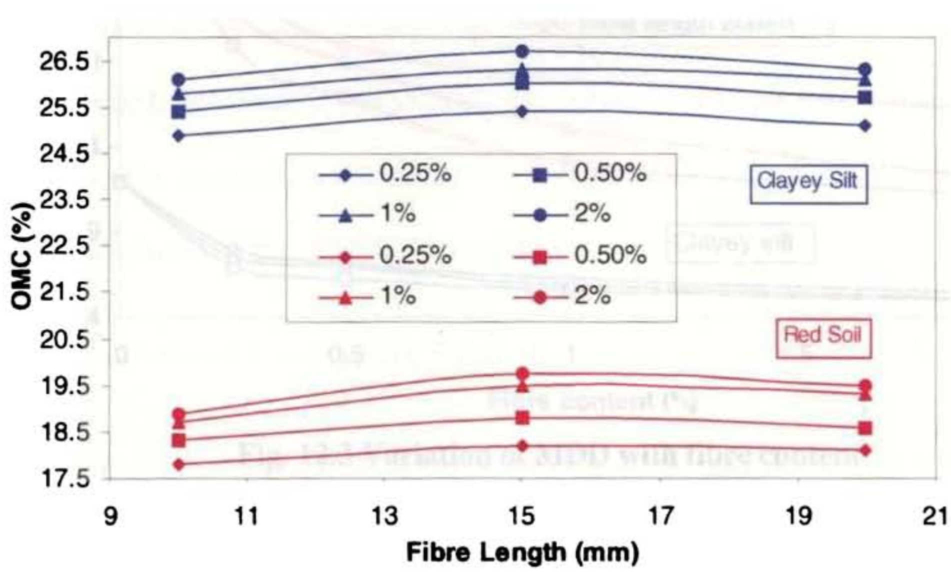


Fig. 12.2 Variation of OMC with fibre length

The variation of MDD with fibre content for clayey silt and red soil is shown in Fig.12.3. As the fibre content increases, the maximum dry density decreases irrespective of fibre length. As can be seen from the plots, MDD decreases sharply with the addition of 0.25% of fibre and thereafter the decrease is at a low rate. For instance for clayey silt, MDD decreases to 14.8 kN/m^3 from 15.2 kN/m^3 with the addition of 10mm fibre and decreases to 14.7 kN/m^3 with the addition of 20mm fibre. The reduction is still more in the case of red soil.

The specific gravity of the coir fibre is 1.15 and that of the soil is 2.73. As the fibre content increases, the lower density fibres replace the higher density soil grains resulting in a lower density for the soil - fibre composite.

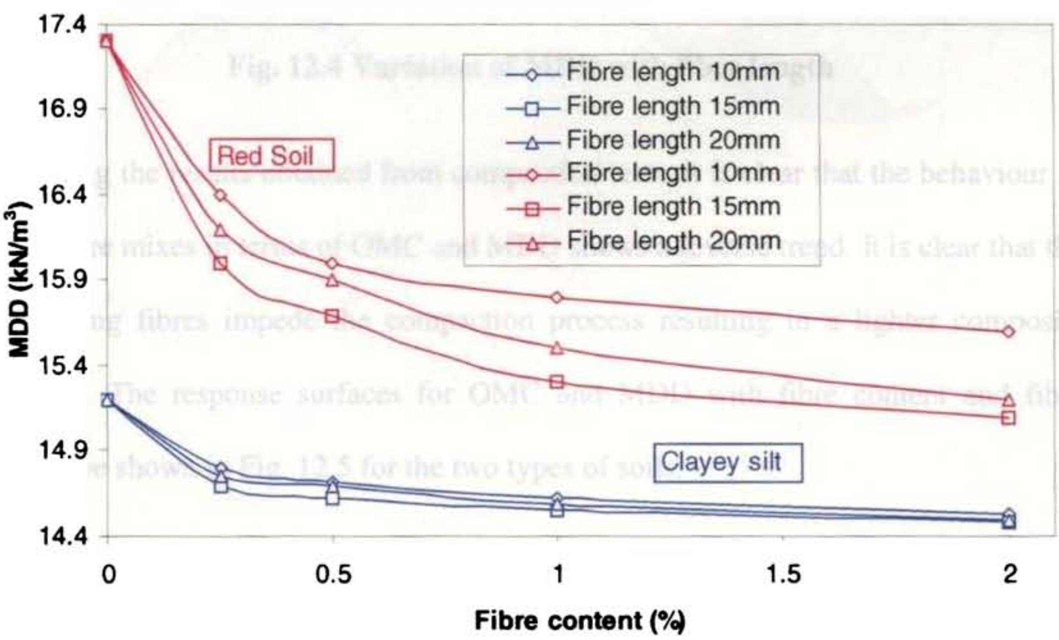
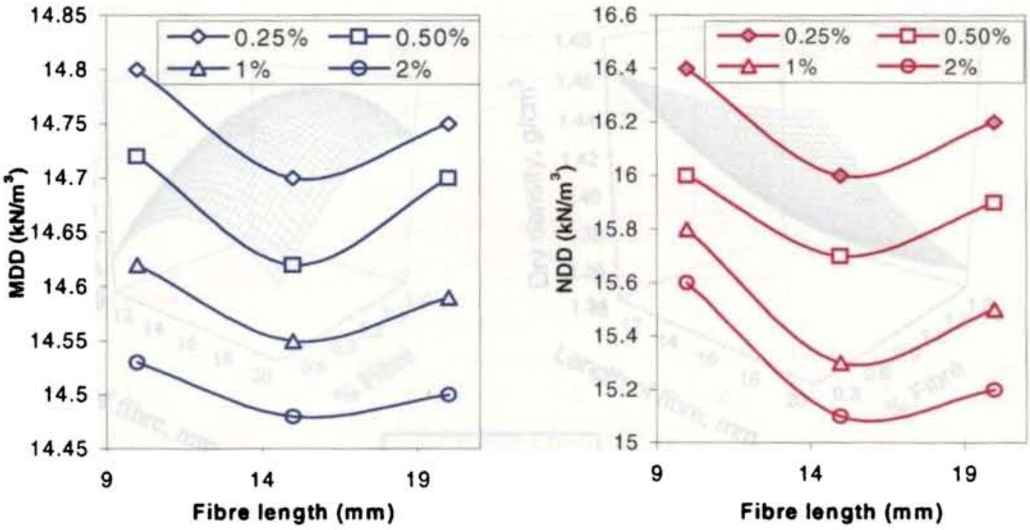


Fig. 12.3 Variation of MDD with fibre content

Fig.12.4 shows the variation of MDD with fibre length. It can be seen that the MDD decreases initially with increase in fibre length and then increases after reaching the minimum value.



(a) Clay **(b) Red Soil**

Fig. 12.4 Variation of MDD with fibre length

Analysing the results obtained from compaction tests, it is clear that the behaviour of soil - fibre mixes in terms of OMC and MDD shows a reverse trend. It is clear that the reinforcing fibres impede the compaction process resulting in a lighter composite material. The response surfaces for OMC and MDD with fibre content and fibre length are shown in Fig. 12.5 for the two types of soils.

12.3.2 Unconfined Compressive Strength

Data were generated at wet of optimum and dry of optimum conditions corresponding to water contents at 95% of MDD of soils to clarify whether the variation is due to lower density and high water content or both. Results of UCC tests showing variations with fibre content for clayey silt soil are shown in Fig. 12.6.

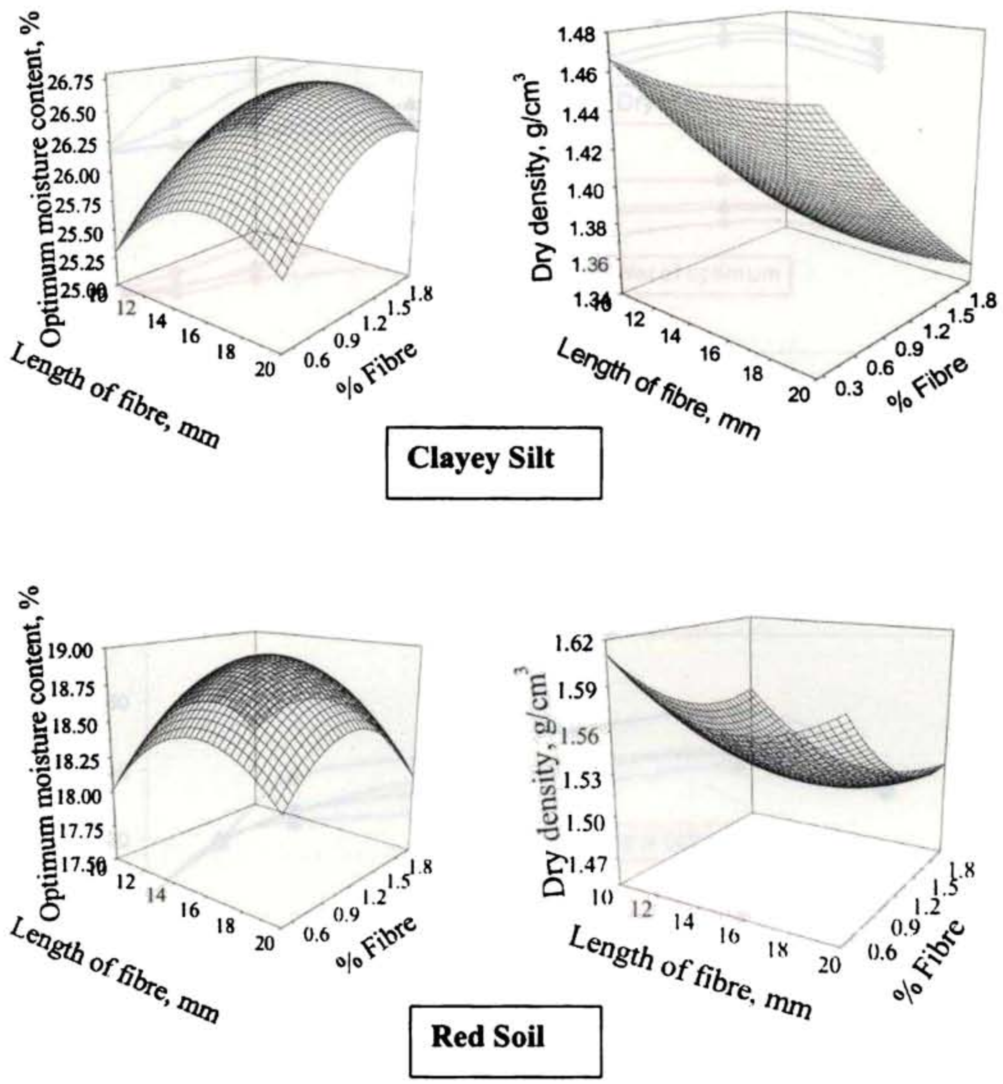
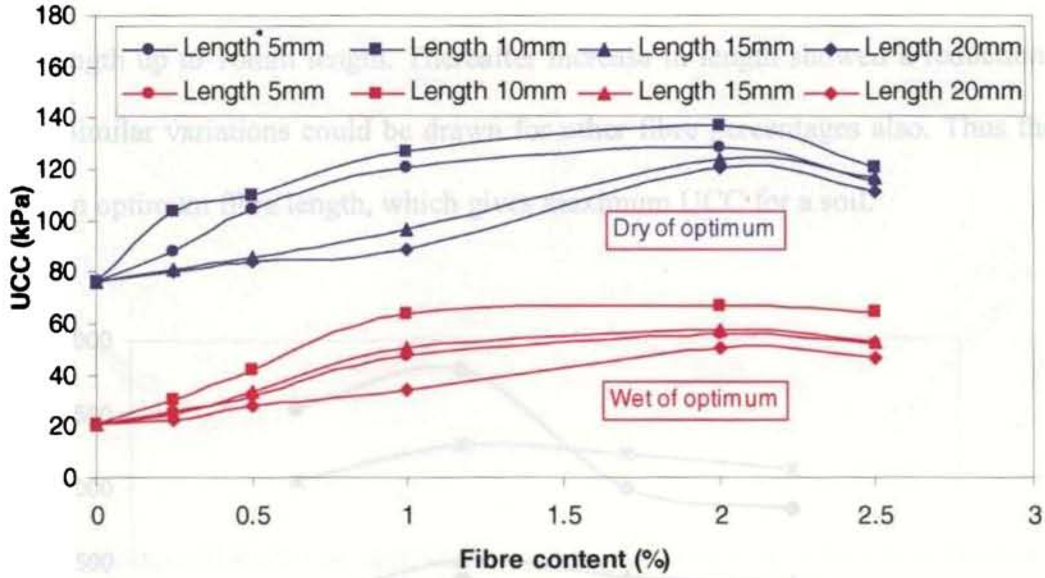
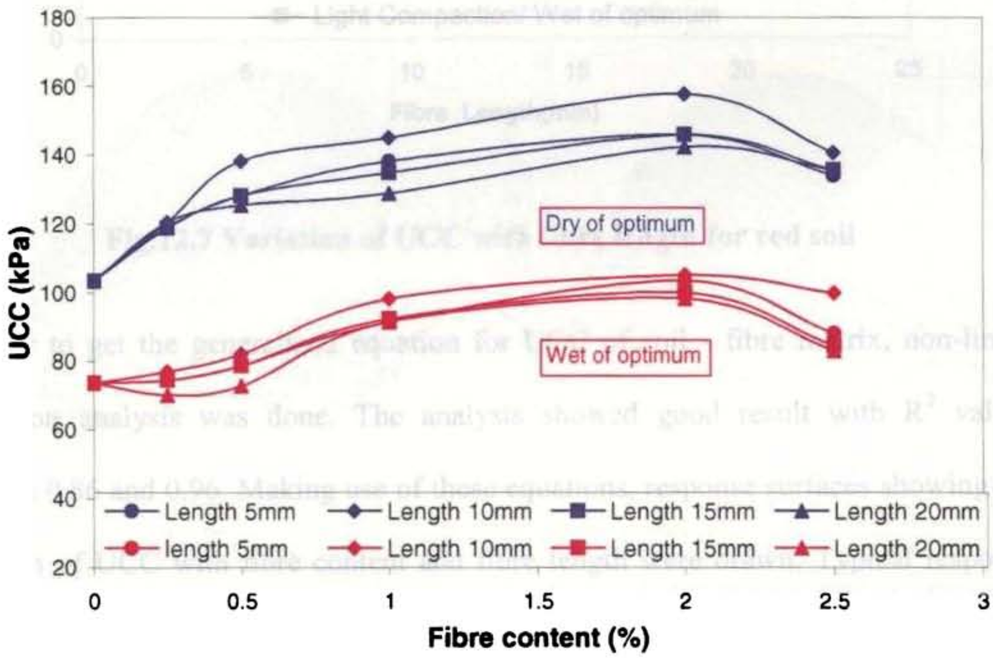


Fig. 12.5 Response surfaces for OMC and MDD

Peak compressive strength was increased up to 2% fibre content and then showed a reduction in strength upon further increase in fibre content. This was attributed to an optimum fibre content. This behaviour was identical in both wet of optimum and dry of optimum conditions in the two soils tested.



(a) Light compaction



(b) Heavy compaction

Fig. 12.6 Variation of UCC with fibre content for clayey silt

When UCC was plotted against fibre length, the variations obtained were similar to variations with fibre content. Fig.12.7 shows the variation of UCC with fibre length for red soil with 1% of fibre content. It was observed that the strength increased with fibre length up to 10mm length. Thereafter increase in length showed a reduction in UCC. Similar variations could be drawn for other fibre percentages also. Thus there exists an optimum fibre length, which gives maximum UCC for a soil.

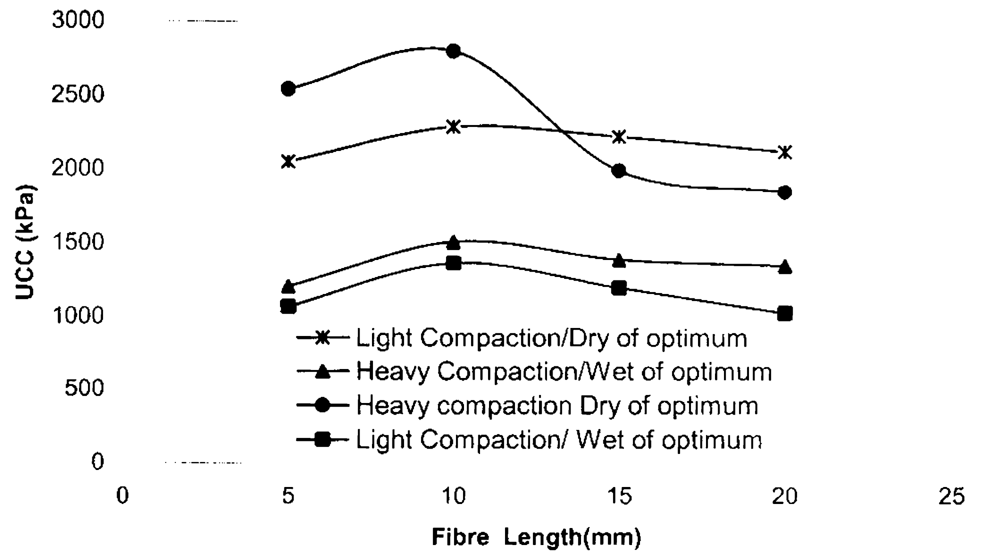
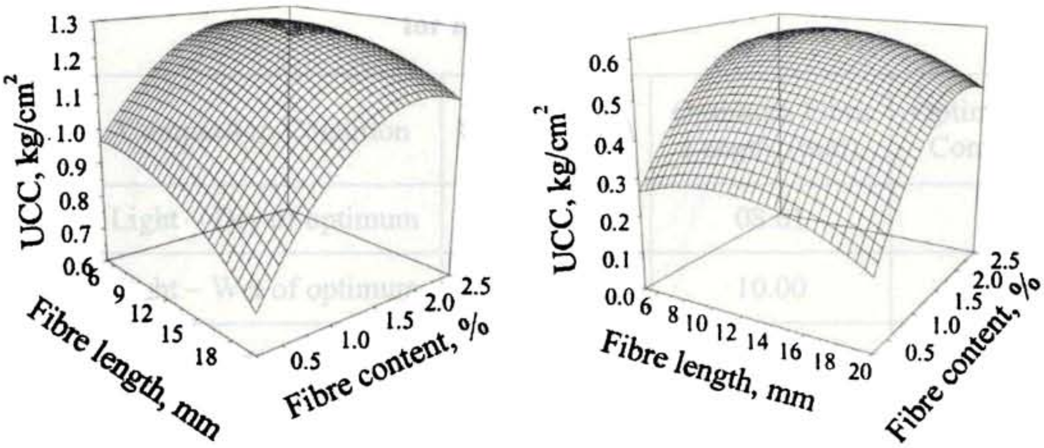


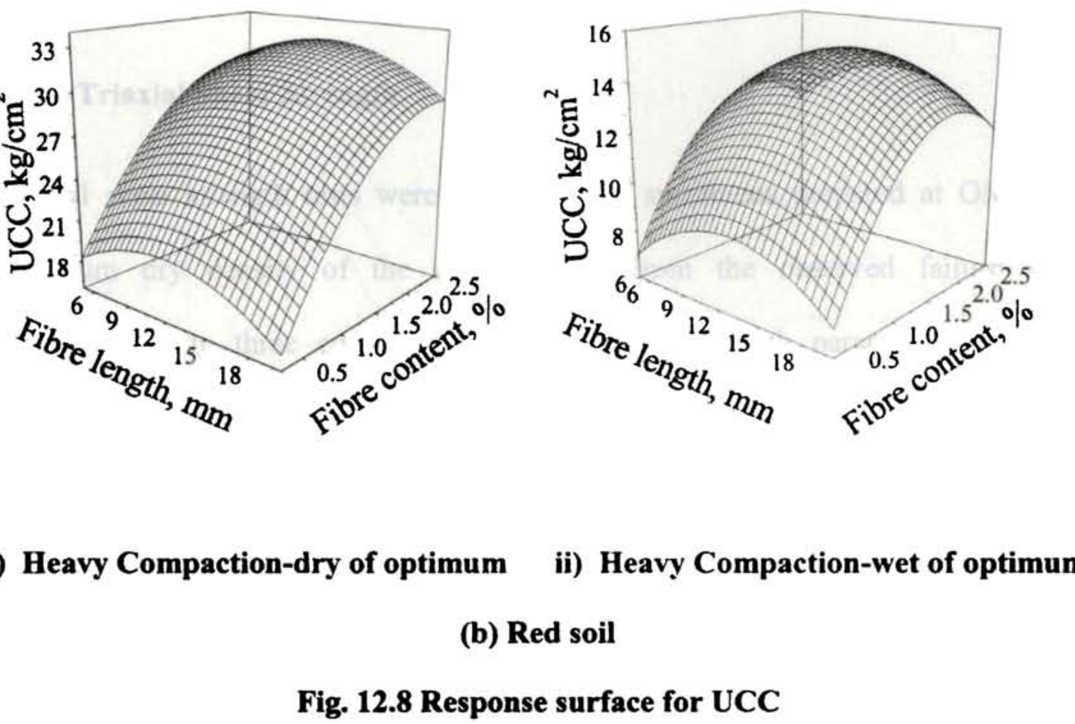
Fig.12.7 Variation of UCC with fibre length for red soil

In order to get the generalized equation for UCC of soil - fibre matrix, non-linear regression analysis was done. The analysis showed good result with R^2 values between 0.86 and 0.96. Making use of these equations, response surfaces showing the variation of UCC with fibre content and fibre length were drawn. Typical response surfaces are shown in Fig. 12.8. It could be easily recognised from the figures that there exist an optimum fibre content and an optimum fibre length, which give a maximum peak compressive strength. The optimum values of fibre content and fibre length along with corresponding UCC values for different cases were determined and the values are tabulated in Table12.1.



i) Light Compaction-dry of optimum **ii) Light compaction-wet of optimum**

(a) Clayey Silt



i) Heavy Compaction-dry of optimum **ii) Heavy Compaction-wet of optimum**

(b) Red soil

Fig. 12.8 Response surface for UCC

Table 12.1 Optimum values of fibre content and fibre length for maximum UCC

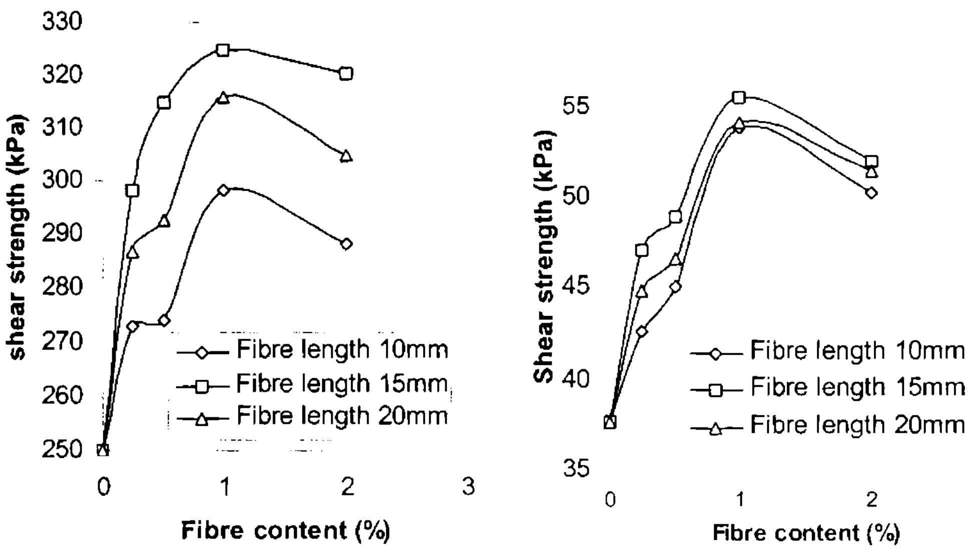
Soil	Compaction Condition	UCC (kN/m^2)	Optimum Fibre Length (mm)	Optimum Fibre Content (%)
Clayey Silt	Light - Dry of optimum	129	08.61	1.85
	Light – Wet of optimum	62	10.00	1.99
	Heavy – Dry of optimum	150	11.31	1.70
	Heavy – Wet of optimum	104	11.48	1.66
Red Soil	Light - Dry of optimum	262	11.44	1.95
	Light – Wet of optimum	186	10.97	1.90
	Heavy – Dry of optimum	265	13.45	1.88
	Heavy – Wet of optimum	184	11.76	1.75

12.3.3 Triaxial Shear Strength

Triaxial shear strength tests were conducted on specimens prepared at OMC and maximum dry density of the control soil. From the observed failure loads corresponding to three confining pressures, shear strength parameters, cohesion intercept and angle of internal friction were found out by drawing Mohr's circles.

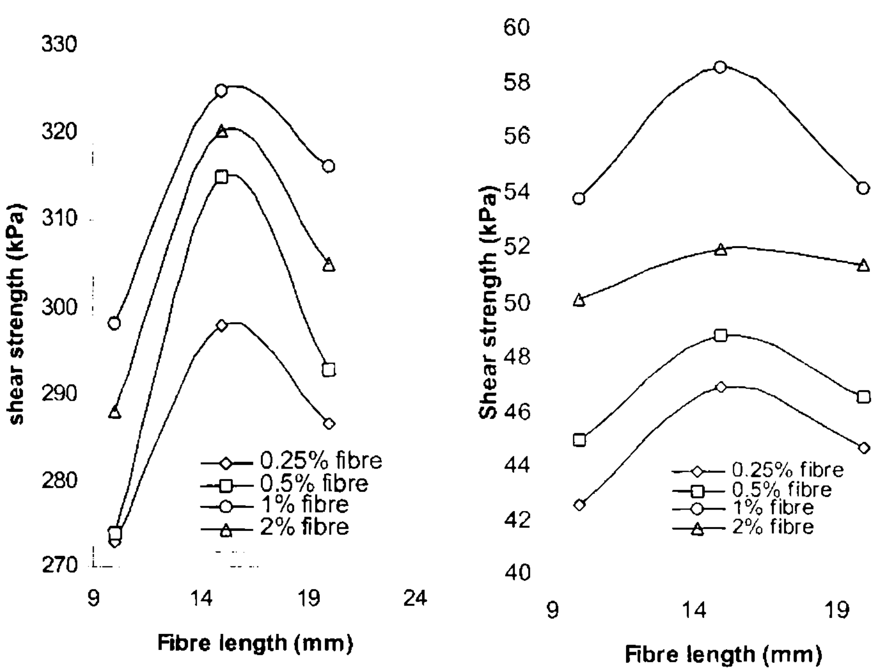
In order to understand the shear strength behaviour, shear strength was calculated as $\tau = c + \sigma \tan\Phi$ for a selected value of σ = 100 kPa. The variations of shear strength with fibre content and fibre length are shown in Fig.12.9 and Fig.12.10 respectively.

It can be observed that the addition of 1% of fibre having 15mm length in clayey silt soil increases the shear strength of the soil by 30%. A fibre length of 15mm showed to produce maximum shear strength. The response surfaces for triaxial shear strength are shown in Fig. 12.11. The optimum values of fibre content and fibre length, which



(a) for Clayey silt soil **(b) for Red soil**

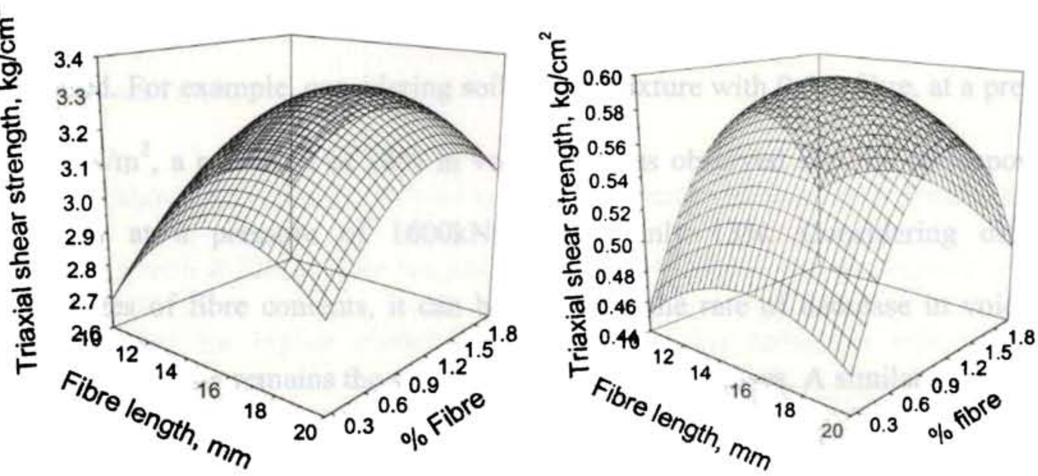
Fig. 12.9 Variation of shear strength with fibre content



(a) Clayey silt **(b) Red soil**

Fig. 12.10 Variation of shear strength with fibre length

gave maximum shear strength for clayey soil and red soil were respectively 1.3% and 1.21% and the corresponding fibre length was 15.8mm and 14.98mm.



(a) Clayey silt **(b) Red soil**

Fig. 12.11 Response surfaces for triaxial shear strength

12.3.4 Volume Change Behaviour

From the load settlement data, void ratios corresponding to each pressure were calculated and the variations were plotted to get the values of compression index. The coefficients of consolidation for each case were obtained for a pressure of 200 kN/m^2 using Casagrande's logarithmic fitting method. All the data were analysed to establish the magnitude of inherent variations in the consolidation properties of fibre soil matrix.

12.3.4.1 Effect of fibre content

Fig.12.12 shows the void ratio – log pressure (e-log p) variation for clay fibre mixture containing 10 mm length coir fibres. Four different percentage fibre content were tried viz., 0.5%, 1%, 2% and 3%. It was found that the addition of fibre resulted in decrease in voids ratio. As the applied pressure is increased, the reduction in void ratio is decreased. For example, considering soil - fibre mixture with 0.5% fibre, at a pressure of 25 kN/m^2, a reduction of 26% in void ratio was observed and the corresponding reduction at a pressure of 1600kN/m^2 was only 13%. Considering different percentages of fibre contents, it can be seen that the rate of decrease in void ratio decreases or rather remains the same as fibre content increases. A similar trend can be observed for soil - fibre mixture with 15mm length fibres. The plots corresponding to 1%, 2% and 3% fibres were observed to be almost parallel exhibiting the same (e-log p) variation.

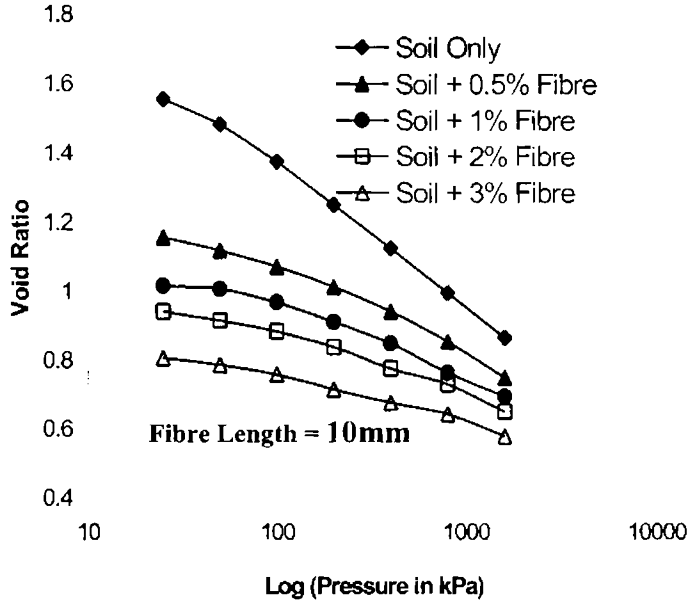


Fig.12.12 (e –log p) curve for soil mixed with 10mm fibre

12.3.4.2 Effect of fibre length

Fig.12.13 shows the comparison of behaviour of soil mixed with fibres having 10mm length and 15mm length. It is observed that at 0.5% fibre content, there is notable change in the reduction of void ratio for soil mixed with 15mm fibre, compared to soil mixed with 10mm fibre. As percentage fibre increased to 3% this variation in void ratio was only marginal.

Fig.12.14 shows the variation in void ratio at different consolidation pressures for soil - fibre mix with different fibre lengths. In general, as the length increases void ratio decreases, but for higher consolidation pressures this reduction was not much prominent.

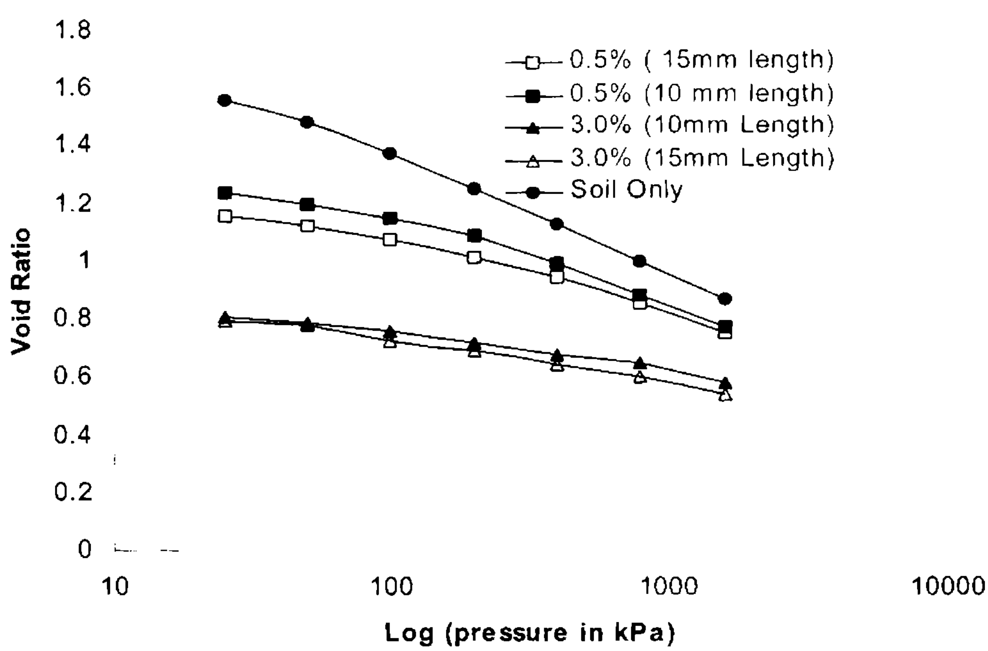


Fig. 12.13 Effect of fibre length on (e –log p) characteristics

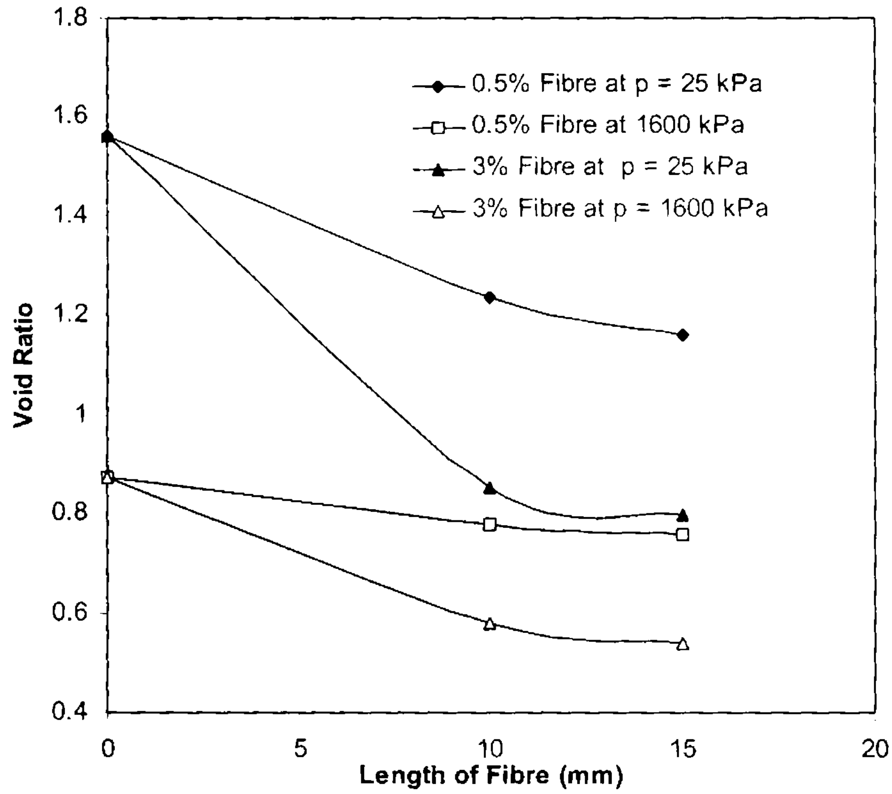


Fig. 12.14 Variation of void ratio with fibre length

12.3.4.3 Variation of compression index

Fig.12.15 shows the variation of compression index (Cc) with fibre content. It can be seen that the addition of fibres in soil reduces the Cc drastically showing 58% and 24% reduction for 10mm and 15mm fibre length respectively corresponding to 0.5% fibre content. The reduction in compression index ultimately leads to the reduction in settlement. As the percentage of fibre content increases, compression index again decreases but rate of reduction in compression index is at a lower pace. At 3% fibre content the percentage reduction in Cc are 66% and 63% for 10mm fibre length and 15mm fibre length respectively. It can also be observed that the percentage reduction in compression index decreases, as the fibre length was increased.

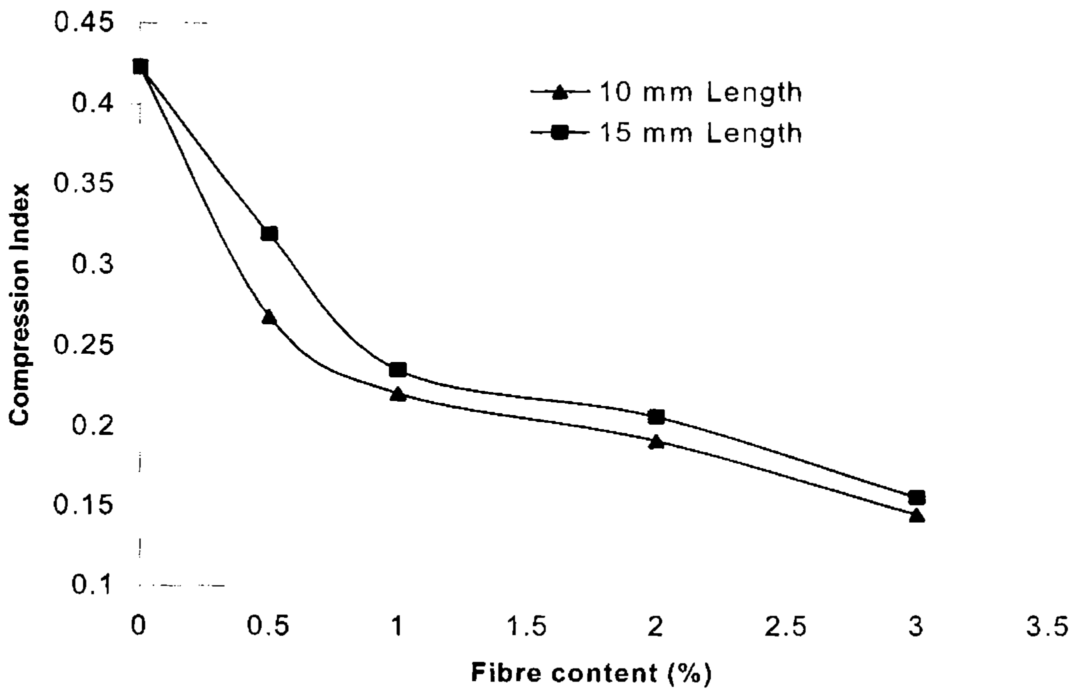


Fig. 12.15 Variation of compression index with fibre content

12.3.4.4 Variation of coefficient of consolidation

The variation of coefficient of consolidation (Cv) with fibre content is presented in Fig. 12.16.The value of Cv increases from 3% to 68% as the fibre content increases from 0.5% to 3% for 10 mm fibre length. The corresponding increase in Cv with 15mm fibres is 21% to 147%. Since the time of consolidation is inversely proportional to the coefficient of consolidation, the addition of fibre in soil reduces the time of consolidation for attaining a specified degree of consolidation. From the test results, it can be observed that, the coefficient of consolidation increases as length of fibre increases. This may be due to the fact that the drainage path becomes more accessible and continuous.

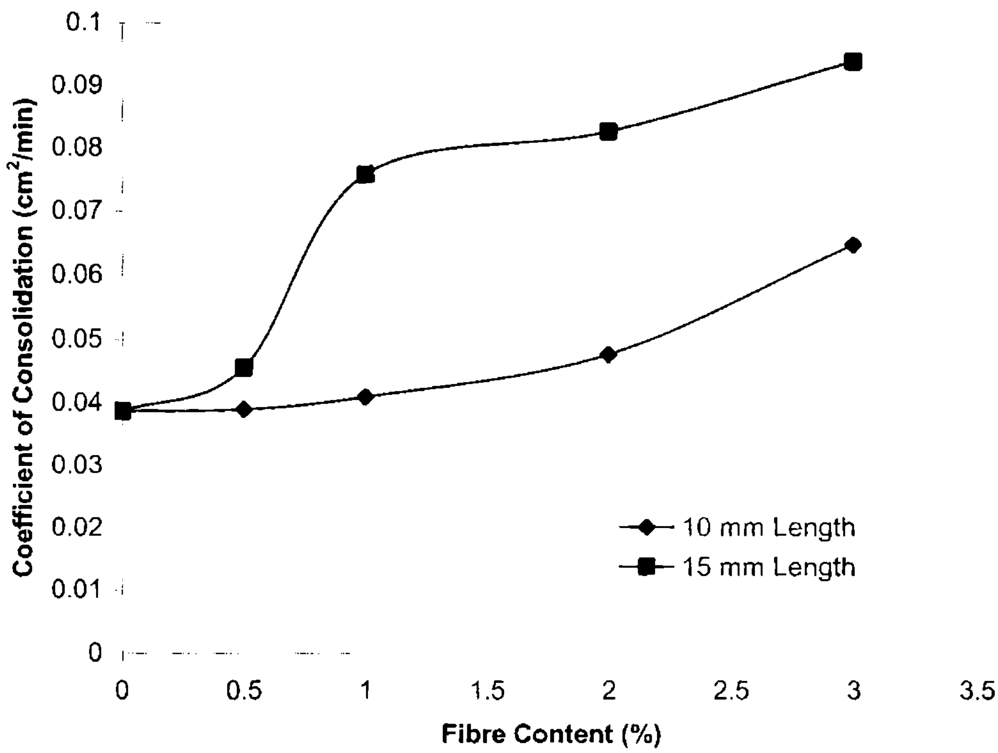


Fig.12. 16 Variation of coefficient of consolidation with fibre content

12.3.4.5 Statistical analysis

From the studies it is observed that the consolidation characteristics of soil blended with coir fibre depends on factors like length of fibre, fibre content and applied pressure. Based on the results of the present experimental investigation, a statistical analysis of the data was performed using SPSS (Statistical Package for Social Sciences). A non-linear regression analysis of the data gave rise to the following equation with R^2 value of 0.93.

$$e = 1.293 - 0.168\,(\log p) - 0.123\,f - 0.13\,L_f - 0.06\,(\log p)^2 \quad \text{--------------} \ (12.1)$$

where,

e = void ratio,

p = pressure in kg/cm^2 (10 kPa),

L_f = length of fibre in cm, and

f = fibre content in %.

12.4 SUMMARY

Two different soils were tested at varying reinforcement - soil ratios and fibre lengths. The results of the tests proved the positive effects of adding coir fibres in enhancing the compressive strength and shear strength if an optimal reinforcement ratio and fibre length is adopted. With the addition of fibres it was observed that, the optimum moisture content of the soil generally increased and maximum dry density decreased resulting in a softer material. The coefficient of consolidation increased considerably with fibre content and the compression index decreased. Both these variations can be viewed advantageous in terms of reduction in consolidation time and the amount of settlement.

CHAPTER 13

CONCLUSIONS AND SCOPE FOR FURTHER RESEARCH

13.1 GENERAL

This thesis is the outcome of the extensive laboratory research work carried out to explore the possibility of utilising coir geotextiles (with Indianised connotation "Coir Bhoovastra"), a natural eco friendly material, for the construction of unpaved roads and embankments in a beneficial manner. Experiments were done to study the four fold applications of coir geotextiles viz., separation, reinforcement, filtration and drainage. This was accomplished by performing elaborate laboratory investigations in different aspects. The main focus in the present investigation was to conduct systematic research work on the use of coir geotextiles, so that new methods of application can be evolved which will pave way for the growth of traditional coir industry.

13.2 CONCLUSIONS

The major conclusions derived from the studies conducted are summarised below in different subsections treating each aspects of geotextile functions separately.

13.2.1 Properties of Coir Geotextiles

Three types of coir geotextiles were used in this research work. Based on the properties evaluated and the studies on interfacial friction characteristics, the following conclusions are made:

1. The puncture resistance and tensile strength of the woven coir geotextiles used in the study satisfy the specifications for geosynthetic fabrics as per US practice for separation purpose and reinforcement functions.

2. Permittivitty of Non-woven coir geotextiles satisfy the requirements of separation function.

3. The experimental results revealed that the interfacial friction characteristics of coir geotextiles are sufficient enough to fulfill its functions such as reinforcement and separation.

4. Friction coefficient of soil - coir geotextile interface is 30% to 40% more than that of soil – soil interfaces.

5. Values of friction coefficient and direct sliding coefficient of coir geotextiles are much more than the minimum values specified by International Geosynthetic Society for use in different applications.

6. Friction characteristics of Non-woven coir geotextiles are better than that of woven coir geotextiles. Among woven coir geotextiles, those with lesser mesh size opening showed better results.

7. The test results indicated that the particle size, shape and gradation affect the interfacial shear at a given stress level.

8. The interfacial friction characteristics increase with increase in density of soil.

9. The higher the water content in soil, the higher the cohesion intercept and the smaller the friction coefficients for the geotextile - soil interface.

10. Comparing with experimental results, it can be observed that, theoretical expressions developed for direct sliding coefficient for polymeric materials also suit for the coir geotextiles.

Coir geotextiles are biodegradable and eco-friendly. They do not pose any environmental problems at a later stage. Coir geotextiles are comparatively economical than polymeric geotextiles.

13.2.2 Strength of Coir Reinforced Subgrade

The strength of the sub grade was studied in terms of California Bearing Ratio, both in soaked and unsoaked conditions. Two types of subgrade soil and three types of coir geotextiles were used in the study in different combinations and layout. Following are the conclusions arrived at:

1. Other conditions remaining the same, the load penetration behaviour of the coir geotextile reinforced soil is identical for all types of soils.

2. Higher CBR was obtained with Non - woven coir geotextiles in unsoaked conditions, and with H2M8 coir geotextiles in soaked conditions.

3. Better results were obtained when coir geotextiles were placed nearer to the surface.

4. For soil - aggregate system, the CBR was increased by 40% to 50% with H2M8 geotextile and 20% to 25% with Non-woven geotextile.

5. Due to the provision of an additional layer of coir geotextile, the CBR was almost doubled.

6. The beneficial effect of coir reinforcement is found to be more with poor soil and that too, in the soaked condition.

7. The CBR of the coir geotextile reinforced subgrade depends on the strength of subgrade, properties of coir geotextile and the placement depth of coir geotextile.

8. The equivalent (or modified) CBR for the coir geotextile reinforced subgrade can be obtained, correlating the properties of soil, properties of coir geotextiles and placement location of geotextile

13.2.3 Bearing Capacity

To evaluate the bearing capacity and settlement characteristics of reinforced soil, investigations were done with three types of natural coir geotextiles in reinforced sand beds. Based on these load tests, the following conclusions are drawn.

1. The results clearly demonstrate that coir geotextiles, a natural product, can substantially increase the bearing capacity of shallow square footing on sand. The behaviour was found to be identical in both dry and saturated conditions. The improved performance can be attributed to increase in shear strength in reinforced soil mass due to the inclusion of coir geotextile reinforcement, owing to high interfacial friction between the geotextile and soil.

2. The bearing capacity of reinforced sand was found to increase up to 80% in dry condition and up to 160% in saturation condition depending on the type of coir geotextiles.

3. The capacity improvement factor increases as z/B values decreases. A value of z/B around 0.5 can be considered as optimum for coir geotextile reinforced soil. It was found that beyond 1.5B the effect of reinforcement is practically zero.

4. Of the three varieties of coir geotextiles used, H2M8 performed better and minimum improvement is obtained when H2M6 is used. This can be attributed to the mesh size and stiffness of the coir geotextiles.

13.2.4 Rut Behaviour of Unpaved Roads

Based on the observations made on unpaved road sections, under static loads and repetitive loads, the following conclusions can be drawn.

1. For the same wheel loads, coir geotextile reinforced unpaved road sections produces lesser rut compared to unreinforced control sections. In other words, for a specified rut depth, coir reinforced sections can sustain higher ESWL.

2. While reinforcing with woven coir geotextiles, those with lesser mesh size (here H2M8) are more beneficial.

3. Non-woven coir geotextile and H2M8 (woven) coir geotextile produced identical effects. In clayey subgrade soil, Non-woven performs even better than H2M8 as reinforcement.

4. Based on the type of geotextiles and the nature of subgrade soil, an additional wheel load of 50% to 90% can be applied on reinforced road sections compared to unreinforced one for a rut depth of 20 mm.

5. From the studies it is observed that an additional reinforcement layer within the subgrade can produce improvement in the performance of the system, only to a limited extent.

6. Coir geotextiles placed between subgrade and sub base can substantially reduce the rut depth due to repetitive wheel loads

7. The rut behaviour under repetitive loads of unpaved road sections reinforced with woven H2M8 and non-woven AGL C/201 were found to be identical.

8. The percentage reduction in rut depth was found to be 50% while using H2M8 or Non-woven coir geotextiles. The corresponding reduction with H2M6 coir geotextile was found to be 38%.

13.2.5 Design of Unpaved Road Sections

Three different approaches of design were considered in the present work. Design charts were prepared for easy determination of pavement thickness. Salient points derived are given below.

1. It was seen that the thickness of the section is reduced by the introduction of coir geotextile, whichever be the method applied for finding the base course aggregate thickness.

2. The reduction in thickness of the aggregate layer is of the order of 40%.

3. If we consider the cost of aggregate saved and cost of coir geotextiles used, coir reinforced sections are more economical than unreinforced sections.

13.2.6 Prefabricated Coir Geotextile Vertical Drains

In order to assess the drainage and filtration characteristics of coir geotextile extensive laboratory investigations were carried out on two types of coir geotextile drains with different configurations and based on that, the following conclusions are made.

1. Of the two types of coir geotextile vertical drains tried, the circular type was found to be more efficient,

2. Installation of the rectangular type is simple compared to circular type because there is no withdrawal of casing pipe and filling of the drains.

3. The triangular disposition was found to be more efficient than central and rectangular dispositions.

4. The performance of drains with H2M8 coir geotextiles was better than that with H2M6 coir geotextiles.

5. Combining all parameters it is seen that circular type coir geotextile drains using H2M8 coir geotextiles, installed in triangular pattern, gives maximum benefit in terms of consolidation settlement.

6. It is seen that for 10mm settlement to take place under a pressure of 10kPa, the time required was reduced by 50% in the case of circular drains in triangular pattern.

7. In general the time for consolidation is very much reduced due to the provisions of the circular and the rectangular coir geotextile drains in all the configurations tested. While comparing the performance of the types of coir drains, it is seen that, the drains using H2M8 performed better than that of

H2M6 coir geotextiles. Again, the triangular configuration is found to be more efficient than other patterns tried. Much better result was obtained with circular type of drains compared to rectangular type of drains.

The coir geotextiles are cheap compared to its polymeric counter parts. Besides, coir geotextiles are eco-friendly and biodegradable and also provide work opportunities to weaker sections of the society. Hence such types of drains are adaptable to countries having large production of coconut.

13.2.7 Soil - Coir Fibre Composites

On degradation of the coir geotextile, the coir fibres remain in soil. This part of the work was aimed at the effect of discrete randomly oriented coir fibres on the performance of pavement supporting soil. The effect of fibre content and fibre length was investigated by conducting experiments on soil fibre blend. The major conclusions from the experimental results are listed below:

1. The UCC increases initially and decreases after a particular value of fibre content. Similar variation was obtained with change in fibre length.

2. It was established that there exists a definite optimum fibre content and fibre length, which give rise to maximum value of UCC. The optimum fibre content is in the range of 1.75% to 2% and fibre length 10 mm to 12mm.

3. Similarly, the optimum fibre length and fibre content to give maximum triaxial shear strength were found to be 15mm and 1.25% respectively for the soils tested at optimum moisture content and maximum dry density.

4. Considering the settlement characteristics, in general, void ratio decreases with the fibre content and this reduction is directly proportional to the applied pressure. It was also observed that the increase in fibre length causes a reduction in void ratio, owing to easy drainage path.

5. In terms of compression index, inclusion of fibre reduces the compression index leading to lesser settlement. Similarly coefficient of consolidation increases as the fibre content increases, which in turn reduces the consolidation time required to achieve a specified degree of consolidation.

6. From the statistical analysis of the results obtained from the laboratory experiments, an expression for void ratio is developed, which can be used for predicting the void ratio in terms of consolidation pressure and fibre properties viz., length and quantity.

13.3 CONCLUDING REMARKS

After considering the above conclusions, the following points in connection with the utilisation of coir geotextiles in unpaved roads and embankments are established.

(i) Reduces the intensity of stress on the subgrade (Function: Separation),

(ii) Reduces the thickness of aggregate required to stabilize the subgrade (Function: Separation and Reinforcement),

(iii) Minimises disturbance of the subgrade during and after construction (Function: Separation and Reinforcement),

(iv) Increases the subgrade strength over time (Function: Filtration),

(v) Minimises the differential settlement of the roadway, which helps to maintain pavement integrity and uniformity (Function: and Reinforcement),

(vi) Prevents subgrade fines from pumping into the base (Function: Separation),

(vii) Prevents contamination of the base materials allowing more open graded, free draining aggregates to be considered in the design (Function: Separation and Filtration),

(viii) Reduces the depth of excavation required for the removal of unsuitable subgrade materials (Function: Separation and Reinforcement), and

(ix) Minimises the maintenance and extend the life of the pavement (Functions: Separation, Reinforcement and Filtration)

This work is concluded with a hopeful note that the results obtained here would lead to more applications of coir geotextiles in the field constructions of unpaved roads with increased level of confidence.

13.4 SCOPE FOR FURTHER RESEARCH

Though exhaustive, the present study was confined to two types of subgrade soils and three types of coir geotextiles. There is substantial scope for carrying out further work in this area. The possible research ideas for future work are summarized below.

1. The present work can be extended considering more varieties of coir geotextiles and soils.

2. In the studies concerned with prefabricated drains, the measurement of pore water pressure and discharge capacity of drains can be made. Again the effect of smear can be studied.

3. Field tests can be carried out to get the long-term effects.

4. Studies related to degradation aspects and their effects can be studied.

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