STUDIES ON THE DEVELOPMENT AND CONTROL OF DESICCATION CRACKS IN COMPACTED CLAY LINER SOILS

A Thesis

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Declaration

I, Sobha Cyrus hereby declare that the work presented in the thesis entitled "**Studies on the Development and Control of Desiccation cracks in compacted clay liner soils**", being submitted to Cochin University of Science and Technology for the award of Doctor of Philosophy under the Faculty of Engineering, is the outcome of original work done by me under the supervision of Dr.Babu T.Jose,Emeritus Professor, School of Engineering, Cochin University of Science and Technology,Kochi-22.This work did not form part of any dissertation submitted for the award of any degree,diploma,associate ship or other similar title or recognition from this or any other institution

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Certificate

Certified that this thesis entitled " **Studies on the Development and Control of Desiccation cracks in compacted clay liner soils**", submitted to Cochin University of Science and Technology,Kochi for the award of Ph.D. Degree ,is the record of bonafide research carried out by Smt.Sobha Cyrus, under my supervision and guidance at School of Engineering, Cochin University of Science and Technology,. This work did not form part of any dissertation submitted for the award of any degree, diploma, associate ship or other similar title or recognition from this or any other institution

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'In His time He makes all things beautiful' (The Bible - Eccl 3:11)

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SOBHA CYRUS

In the present day scenario of rapid urbanization and industrialization, large quantities of wastes in different physical forms are generated. Despite all efforts to minimize waste and to neutralize it, the requirement for storage or disposal still exists for which landfills form one of the solutions.

The modern landfill is a very complex structure engineered to protect the environment from the liquid that seeps from the waste called leachate. This leachate tends to percolate downward into the ground as well as from the products of decomposition that takes place in the waste which may last for decades. An important component of a land fill is a layer of compacted, low permeability soil that is intended to act as a hydraulic barrier and minimize infiltration of water into the mass, when it is part of a cover system, or prevent the leachate from contaminating the ground water, when the soil is part of a liner system. According to regulations laid down by Environmental Protection Agency, the soil liners shall ensure that the hydraulic conductivity is equal to or less than 10^{-9} m/sec, which obviously is the prime criterion in the selection of liner material. During certain stages in the life of a landfill, it could be subjected to seasonal changes, resulting in significant variation of water content leading to the desiccation of clay liner material and thus posing a major threat to the integrity of the system as a hydraulic barrier.

The application of bentonite is currently the most accepted practice for lining purposes. The ideal bentonite sand combination, which satisfies the liner requirements is 20% bentonite and 80% sand, was selected as one of the liner materials for the investigation of development of desiccation cracks. Locally available sundried marine clay and its combination with bentonite were also included in the study. The desiccation tests on liner materials were conducted for wet/dry cycles to simulate the seasonal variations.

Digital image processing techniques were used to measure the crack intensity factor (CIF), a useful and effective parameter for quantification of desiccation cracking. The repeatability of the tests could be well established, as

the variation in CIF values of identical samples had a very narrow range of 0 to 2%. The studies on the development of desiccation cracks showed that the CIF of bentonite enhanced sand mixture (BES) was 18.09%, 39.75% and 21.22% for the first, second and third cycles respectively, while it was only 9.83%, 7.52% and 4.58% respectively for sun dried marine clay (SMC). Thus the locally available, alternate liner material suggested, viz SMC, is far superior to BES, when subjected to alternate wet/dry cycles.

Further, the improvement of these liner materials when amended with randomly distributed fibre reinforcements was also investigated. Three types of fibres ,namely nylon fibre, polypropylene monofilament and polypropylene fibre mesh were used for the study of fibre amended BES and SMC.The influence of these amendments on the properties of the above liner materials is also studied. The results showed that there is definite improvement in the properties of the liner materials when it is reinforced with discrete random fibres. The study also proved that the desiccation cracks could be controlled with the help of fibre reinforcement.

Chapter 1

INTRODUCTION

Industrialization brought forth with it the associated problems. The industrial activities generated large quantities of wastes. Part of these wastes in different physical forms such as solids liquids and gases turn as pollutants in due course. Based on the safety level, these wastes can be hazardous or non hazardous. Wastes can be controlled by different options such as waste reduction at source, resource recovery through separation and recycling, resources recovery through waste processing, waste transformation and environmentally sustainable disposal on land. Despite all efforts, to minimize waste, and to neutralize it, the requirement for storage or disposal still exists. The most frequently used disposal option for solid waste in the land fill because of its low cost and efficiency. The landfill plays a vital role in the whole waste treatment/disposal process.

The basic philosophy of all engineered landfill should revolve around environmental protection through containment and controlled release, physical compatibility of the final landform to the surroundings, longevity for the design period, appropriateness to the type of waste and cost effectiveness. The main components of an engineered landfill are a liner system at the base and sides of the landfill which prevent migration of leachate or gas to the surrounding soil, a leachate collection facility which collects and extracts the leachate from within and from the base of land fill and then treats the leachate, a gas control facility which collects and extracts gas from within and from the top of the landfill, a final cover system which enhances surface drainage intercepts infiltrating water and supports surface vegetation. The final cover system can comprise of multiple layers of soils.

Environmental protection Agency (EPA) has laid down norms for the selection of suitable standard liner materials. With regard to the long term satisfactory performance of land fills, the liners covers play a very vital role. The most common land fill liner materials are clayey soils of low permeability, since they satisfy the EPA standards. Clayey soils pose numerous problems to geotechnical engineers because of their high compressibility and poor shear strength. The high compressibility of clays which leads to large scale volume changes is always a cause of concern to the field engineer. However there are occasions when clayey soils can be utilized such as control of permeability in clay liners, stabilization of bore holes etc.Here the geotechnical engineer is posed with the problem of suppressing the negative aspects of the clayey soils to the minimum while utilizing the positive aspects to the maximum.

Volume changes in clayey soils occur in many ways. One is due to the expulsion of pore water from the voids upon static surcharge. This behaviour, termed as consolidation is a well known and well defined phenomenon. The other volume change is due to the shrinking of clay soils during drying. Desiccation is the continuous process of pore water loss from a soil exposed to a warm environment. In response to drying, soil water volume decreases and in consequence the soil shrinks. If shrinkage is restrained, soils can crack during desiccation as and when the tensile stresses that develop within the soil exceed the tensile strength of soil. The desiccation cracking of a clay mass can have a significant impact on the performance of clayey soils in various geotechnical, agricultural and environmental applications. Cracks affect the compressibility of the soil, its time rate of consolidation, its strength and the rate at which water can re-enter. Thus, several geotechnical constructions are affected directly or indirectly by the presence of cracks in a soil mass.

Many earth structures are constructed of clayey soils which have a tendency to shrink and swell when subjected to cycles of drying and wetting. During dry periods, clay near the surface of the slope shrinks, resulting in desiccation cracks. Deep cracks expose the interior of the soil mass, thus allowing further cracking to occur. When subsequent wetting of the slope occurs during rainfall, the extensive network of cracks and fissures created during shrinkage, permits rapid percolation of rain water into the soil mass. As the cracks fill with water, the exposed clay surface along the crack swells, the clay softens resulting in loss of strength along the cracks and fissures. Over a period of time, the seasonal shrinking and swelling may result in further propagation of the cracks and ultimate failure of the slope.

Cracks create zones of weakness in a soil mass and cause reductions in the overall strength and stability as well as increase in the compressibility of the soil. Structures that are constructed over fine grained soils such as foundations and embankments can be affected by mechanical changes caused by cracking. Cracks are also a possible precursor for inception of failure surface at the top of dams and embankments.

Desiccation cracking is a common phenomenon in clayey soils and can change the hydraulic conductivity of soil. Drying fractures strongly affect permeability and may compromise such structures as clay buffers for nuclear waste isolation, barriers such as landfill liners, top covers, etc. Compacted clay liners are essential components of both municipal and hazardous waste land fills and their design has typically been based on the premise that little leakage will occur if the soil has a laboratory measured hydraulic conductivity of less than $1x10^{-9}$m/s. Hydraulic conductivity of clay liner material may increase from $1x10^{-9}$m/s for wet and intact soil, to $1x10^{-6}$m/s for the material after cracking.

Desiccation of landfill clay liners is a major factor affecting the performance of landfills. Desiccation leads to the development of shrinkage cracks. Cracks provide pathways for moisture migration into the landfill cell which increases the generation of waste leachate, and ultimately increases the potential for soil and ground water contamination.

A variety of research efforts have attempted to address the problem of desiccation cracking. Some have considered the use of surface moisture barrier above the clay liner, but case histories show that repeated cycles with seasonal temperature changes result in significant desiccation of the clay layer and associated cracking. Use of fibres to reduce the desiccation cracking in compacted clay has caught the attention of geotechnical engineers. An attempt is therefore made herein to study the development of cracks in clay liner materials and to control the desiccation cracks by use of randomly distributed discrete fibres.

The contents of various chapters of this thesis are briefly described below.

Chapter 1 presents the significance of landfill liners in the present day scenario and highlights the need of the study of desiccation induced cracks in landfill liners. The brief outline of the problems associated with desiccation cracks is also given.

Chapter 2 critically reviews the earlier efforts in the related fields in the literature. Details of the different liner systems and liner materials used for land fills and the EPA standards for the selection of landfill liner materials are discussed. The earlier studies on marine clay are also included. The scope of the work and the objectives are also discussed.

Chapter 3 deals with a description of the soils and amendments used in the study. The various innovative testing procedures and equipments designed and developed exclusively for the investigations are also dealt with in detail.

Chapter 4 gives the detailed account of the tests carried out to determine the development of desiccation cracks in liner soils when subjected to alternate wetting and drying. The determination of crack intensity factor using digital image processing is also dealt with.

Chapter 5 presents the investigations on the use of random discrete fibre to control the desiccation cracks on liner materials. Three types of fibres namely nylon, poly propylene mono filament and poly propylene fibre mesh were selected for the study. Tests were carried out to find the influence of these fibres on the compaction characteristics, hydraulic conductivity, shear strength and tensile strength characteristics.

Chapter 6 presents the conclusions derived from the detailed investigations carried out.

Chapter 2

REVIEW OF LITERATURE

2.1 INTRODUCTION

Since the 1970's, awareness of landfill problems and their solutions has increased to a point where waste disposal by landfill has become a technology in its own right (Kalteziotes et al, 1994). The disposal of waste-municipal, industrial, hazardous—in an acceptable manner has become a challenge. Waste disposal methods include deep well injection, incineration and landfills. The most frequently used disposal option for solid waste is the landfill because of its low cost and efficiency. The landfill plays a vital role in the whole waste treatment/disposal process. This is because most ashes produced by incineration and many sludges from water treatment plants end up in landfills. It is therefore, important to focus attention on landfills, which go a long way to reducing public health risk and protecting the environment. The most common landfill lining materials are clayey soils of low permeability although there is a rapidly growing use of man made synthetic membrane lines (geomembranes). Nearly all geomembranes used in landfills are thin sheets of flexible thermoplastic or thermoset polymeric materials.

2.2 DIFFERENT LANDFILL LINERS

According to Koerner (1984), the landfill liners for both bottom and side use is constructed from a wide range of materials. The liners are classified as:

2.2.1 Rigid liners

Shotcrete or Gunite Liners type of liners a mixture of cement, sand and water is blown under a pressure on to the prepared bottom and sides of the waste pit. For landfills, the technique is not often used because of its high cracking potential for all but the most rigid of foundations, eg: a landfill built on rock.

In concrete liners, the aggregate is stone and sand, which is then mixed with cement to which water is added to make the final product. It can be reinforced with wire mesh or reinforcement bars depending on conditions of sub grade. Its strength is greater than that of shotcrete, but it suffers a drawback in that construction and expansion joints must be incorporated in the pour, both of which are subject to leaks. This might not be compatible with the leachate that will eventually develop in the landfill. The paving of side slopes with angles greater than 30° to the horizontal presents major construction problems when using concrete, and the materials high cost greatly limits its use.

Soil-Cement Liners are meant to duplicate the qualities of concrete as closely as possible but more cheaply by using on site materials and construction methods. The major problem in this type of liners is the final homogeneity of the material.

Bituminous Concrete (asphalt) has been used often for landfill liners. The technology follows asphalt pavement procedures as used in highway engineering and construction but places greater emphasis on permeability considerations. In this type of liners, the major consideration is the chemical compatibility between the landfill contents and the bituminous material used in the liner of lesser concern, but still problematic are the accelerated aging of bituminous liners and the weed growth that can occur through them producing preferential seepage paths for leachate and other landfill contents.

2.2.2 Flexible liners

2.2.2.1 Natural Materials

The different natural materials used as liner materials are as follows:

Compacted Soil Liners: Under advantageous conditions, naturally occurring fine grained soils may themselves be used for the liner. The key parameter is the soil permeability which should be 10^{-5} to 10^{-7} cm/s, depending upon the contained material. This puts it in to the class of fine grained soils were the montmorillonite clays pocess the lowest values. This Puts it into the class of fine-grained soils, where the montmorillonite clays pocess the lowest values. This includes bentonite clays. Bentonite, which to montmorillonite, types which have least permeability.

Chemically Treated Soils

The interaction of chemicals with soils can drastically alter their behavior and there fore suitably chemically treated soils can be used. This will be economically viable only with cheap chemicals. These materials either infiltrate into the voids of a soil or form a crust on its surface, there by blocking off the voids. In these types or liners, the problem is to achieve uniformity of treatment. There is no guarantee against irregularly treated zones having high permeability's.

Bentonite

It is obviously a very complex material, the structure of which is most important, the resulting particles are flake like, with their lengths and widths much greater than their thickness. This comes about because of the structural sharing of oxygen atoms forming silica tetrahedral and aluminum octahedral sheets. Water can easily penetrate between the sheets or it can easily be adsorbed bed onto their surfaces. The surface adsorption is very important because bentonites are strongly anionic, ie, they carry a negative electrical charge due to substitutions which occur within their crystalline structure. When water in the soil voids is confronted with this situation, the water being polar liquid aligns itself in an orderly fashion, to help satisfy the charge of the bentonite sheets and the cations in the water, which are usually only partly hydrated, also move to the clay's surface. These changes result in an adsorbed water layer containing entrapped cations, which is bound to the surface of bentonite sheets. This forms the adsorbed water layer, which protrudes, into the soil voids drastically affecting the soils permeability.

This situation has long been recognised by engineers looking for liners to retard seepage from landfills of all types.

2.2.2.2 Synthetic liners

Geomembranes (Plastic and Elastomers) from synthetic materials (plastic and rubber) began with the introduction of polyvinyl chloride (PVC) and butyl rubber. Geomembranes are best categorized by their properties. The EPA

(Environmental Protection Agency) standards require that there be two liners underlying the unit and a leak detection system between the two liners.

2.2.2.3 Composite liners

According to Daniel (1993), Composite liners are in fact combination of flexible membrane liners (FML) and pure clay. Leachate from top of the clay liner will percolate down through it at a rate controlled by the hydraulic conductivity of the liner, the head of the leachate on top of the liner and the liner's total area. With the addition of FML placed directly on top of the clay and sealed up against its upper surface, leachate moving down through a hole or deflect in the FML does not spread out between FML and the clay liner. The composite liner system allows much less leakage than a clay liner alone because the area of flow through the clay liner is much smaller. The lamellar arrangement of the liner materials prevent the direct permeation of the leachate from the top even in case of an accidental leakage.

2.3 CLAY LINERS FOR WASTE CONTAINMENT

The containment of hazardous wastes is certainly one of the most urgent problems faced by civil engineers. Among these materials, compacted clay liners have the longest history of successful applications (Kays, 1986), although concretes, asphalts, soil, cement and more recently polymeric membranes have all been widely used. Of the different liner materials presently in use to contain hazardous wastes, the local availability of suitable clayey soils in many regions make use of the soil liner as an economically attractive design alternative. There are various types of liner materials available for the construction of impermeable barriers like compacted fine grained soils, admixtures, polymeric membranes, sealants etc. Easy availability and economic feasibility make clayey soils the most preferred liner materials. Natural clay deposits can provide an effective barrier in many situations and are used for disposal of municipal wastes and sometimes even for the hazardous wastes (Cherry, 1987 and Fetter, 1990). However, they may not be suitable for all the situations, i.e. they may not be having sufficient thickness to prevent migration of contaminants. Hence in such case, compacted clays can be used as liner materials. Compacted clays comprise of naturally occurring clayey soils, mixes of clayey soils or mixes of processed clay mineral with soils. The compacted clay liners are usually 0.3 to 1.2m thick with coefficient of permeability less than 10^{-7} cm/sec. The clay material owing to their low permeability and economic viability are often used as impervious liner for waste impoundment sites. The clay liner prevents pollution of surrounding environment by (i) controlling the amount of seepage and thereby mitigating pollution by dilution (ii) delaying pollution by containment for certain prescribed period and (iii) causing a temporary or permanent decrease in the solute concentration by undergoing physico-chemical interaction with solute (Folkes, 1982).

According to Kays (1986) and Daniel (1993), even though, questions have been raised about clay waste interactions which may degrade clay soils and possibly increase their hydraulic conductivity, current EPA guidance allow single and double liners of compacted clays across sections to be installed at hazardous waste disposal sites. Soil liners are preferred because of their low cost, large leachate attenuation capacity and resistance to damage and puncture. Clays also possess sorptive and or attenuative capacity and reduce the concentration of contaminants in leachate. This capacity relies on chemical composition and on mass of the liner. Soils generally have large capacity to sorbs materials of different types, but some soils do not provide an impermeable boundary. These properties can be enhanced by the use of soil additives. Of the two types of liners viz. soil and synthetic liners commonly used in waste disposal facilities, soil liners seems to be indicative of the extensive use of clay soils as pollution barriers.

2.3.1 Compacted clay liners

Kim (1992) explained the effects of freezing on hydraulic conductivity of compacted clay as the effectiveness of system is controlled by the hydraulic conductivity of the compacted clay. The hydraulic conductivity of specimens compacted dry of optimum increased approximately two to six times their original values. The hydraulic conductivity of specimens compacted wet of optimum increased about two orders of magnitude. He also explained that the freeze-thaw would likely damage compacted clay and increase its hydraulic conductivity.

According to Shelly (1993), Shakoor and Cook (1990), for gravel contents less than 60%, it has the beneficial effects of slightly lowering the hydraulic conductivity of the kaolinite and simultaneous broadening of the range of molding water content by which minimal hydraulic conductivity was achieved. The objective of compaction is to remould clods of the soil into a homogeneous mass that is free of large continuous inter-clod voids. The water content of the soil, method of compaction and compactive effort has major influence on the hydraulic conductivity of compacted soil liners.

Soil liners have traditionally been compacted in the field to a minimum dry weight over a specified range in water content. With soil liners, hydraulic conductivity is usually paramount importance. Daniel & Benson (1990), presented data to show that the water content – dry density criterion for compacted soil liners can be formulated in a manner that is different from the currently used approach in which the adequate strength and permissible compressibility is ensured. The approach recommended by them is based on defining water content – dry density requirements for a broad, but representative, range of compactive energy and relating those requirements to hydraulic conductivity.

2.3.2 Requirements of clay liners

Clay is the most important component of soil liners because the clay fraction of the soil ensures low hydraulic conductivity. According to EPA (1989), the soil liners be built so that the hydraulic conductivity is equal to less than $1x10^{-7}$ cm/sec. To meet this requirement certain characteristics of soil material should be met. First, the soil should have at least 20% fines. Secondly, the plasticity index should be greater than 10%. Soils with very high plasticity index, greater than 30 to 40% are sticky and are difficult to work with. Also high plasticity index soils form hard lumps when the soils are dry and difficult to breakdown during compaction. Thirdly, the coarse fragments should be screened to not more than about 10% gravel size particles. Soils with a greater percentage of coarser fractions can contain zones of gravel, which will have high hydraulic conductivity. Finally, the material should not contain soil particles or chunks of rock larger than 1 to 2 inches in diameter, which may form a permeable window through a layer.

According to Daniel (1993), for any material to be used as a liner, it should have the following properties:

(i) The fluid transmission capability of a soil is defined as the permeability of the soil. Permeability, which is also a measure of the materials ability to contain the leachate. A low permeability generally $10\text{-}8^9$ m/sec. is required.

(ii) Durability and resistance to weathering is the quality of the material to withstand the forces of alternating wet/dry and freeze/thaw cycle.

(iii) Constructability, which means the material, should be reasonably workable in terms of placement and compaction under field conditions.

(iv) Compatibility with leachate: the liner material must maintain its strength and low permeability even after prolonged contact with leachate.

2.4 PROPERTIES OF BENTONITE AND ITS APPLICATIONS AS LINER MATERIAL

Bentonite is a montmorillonite clay containing small quantities of inert mineral grains. It is a very highly plastic, swelling and clay material used for a variety of purposes (Mitchell, 1976). Montmorillonite consists of three blocks in the arrangement Si:G:Si with common oxygen atoms and hydroxyl groups on both adjacent of Al planes and tops of the Si tetrahedrons. The bond on these planes result from van der walls forces and cations, which may be present to equilibrate missing ions in the structure. These weak bonds are easily broken by splitting or by adsorption of water molecules, which produces a high swelling potential on wetting (Nonveiller, 1989).

According to Kays (1986), the clay material used as a liner may be the native clay soil or a bentonite – sand mixture. Bentonite slurry is often used as a sealant. Edil and Muhanna (1992) observed that the ability of a bentonite slurry to rapidly form a filter cake of low permeability on a porous formation is not only important for general stability, but also for sealing qualities.

The use of bentonite alone or amended with natural soils for construction of liners for water – retention and waste containment facilities is common. The importance of bentonite content in reducing hydraulic conductivity of liners is well recognized. Sivapullaiah et al. (2000) illustrated the role of the size of the coarser fracton in controlling the hydraulic conductivity of the clay liner. It has been shown that at low bentonite contents the hydraulic conductivity of the liner varies depending on the size of the coarser fraction apart from clay content. At given clay content, the hydraulic conductivity increases with an increase in the size of the coarser fraction; the hydraulic conductivity is controlled primarily by clay content alone.

2.4.1. Volume change behaviour of bentonite

When montmorillonite clays are in contact with water or water vapour, the water molecules penetrate between the unit layer. This inter layer swelling is evident from an increase of the basal spacing of the clays to definite values of the order of 12-20A depending upon the type of cation. The amount of water held between the layers varies from one to four molecular layers depending on the type of cation and the vapour pressure (van Olphen, 1963). When four layers of water molecules are adsorbed on the surfaces of dry clay, the volume of dry clay can almost get doubled.

For the combination of sodium montmorillonite and fresh water, the fluid that enters the particles, forms a thick viscous diffuse double layer around the sheets, causing the montmorillonite particles to swell, possibly to the extent of complete separation of the sheets. For the combination of dry sodium bentonite and a saline solution, less fluid is required to neutralize the negatively charged sheets. If the ion concentration is large or the valences of the cations are large, the separation distance between sheets will remain small. The fabric of bentonite in this case will consist of swollen but intact montmorillonite particles, surrounded by thin viscous diffuse ionic layers (Fernandez and Quigley, 1988; Kenney et a., 1992).

Numerous investigators have attempted to explain the volume change behaviour of bentonite. Sridharan et al. (1973) recognized that there are basically two mechanisms controlling the volume change behaviour of clays. In mechanism 1, the volume change is controlled by shearing resistance at the interparticle level. In mechanism 2, primarily by the long range diffuse double layer repulsive forces. They also observed that mechanism 1 governs the volume change behaviour of non-expanding lattice type clays like montmorillinite.

El-Shoby and Rabba (1981) from their investigations, arrived at the conclusions that the influence of initial water content on swelling is not appreciable for values of it below the shrinkage limit of the soil. However, as the values of initial water content exceed the shrinkage limit, the influence is significant. The swelling potential of a clay is determined by the activity of the clay fraction. The higher the value of the initial dry density, the greater is the swelling potential. El-Shoby and Mazen (1983) also confirmed the influence of the mineralogical composition as a controlling factor governing the swelling behaviour of expansive clayey soil.

The swelling of montmorillonite soil was explained by Ranganathan (1987) with the diffuse double layer theory. The clay particles with charge excess sites on the surface cause ion concentration in the diffuse double layer to exceed that in free water. The difference in ion concentration results in the diffusion of water those forces the clay particles apart causing swelling.

Basma and Al-Sharif (1994) studied treatment of expansive soils and concluded that initial water content of the soils had a great influence on the amount of swelling. The swelling properties are reduced drastically when the initial compaction water content is increased. They also found that the dry unit weight of specimens is a very important parameter affecting the swelling properties. They proved that increase in initial dry unit weight of the specimens resulted in decrease in the swelling properties. The presence of salt in pore fluid resulted in the reduction in swelling. However, it is also obvious that after reaching certain salt concentration within the pore fluid, no reduction in swelling characteristics is recorded.

As per the observations made by Katti and Katti (1994), the causes of swelling are clay mineral with an expanding lattice and the dipolar nature of water. High liquid limit, plasticity index and low shrinkage limit may be attributed to high base exchange capacity in the montmorillonite clay mineral.

Individual clay-sized particles in expansive soils are anisotropic in nature with respect to swelling.

Studies conducted by Sivapullaiah et al. (1996) on the swelling behaviour of mixtures of bentonite and non-swelling coarser fractions of different sizes and shapes revealed that generally, the swelling of soils occurs in three distinct phases: intervoid swelling, primary swelling and secondary swelling. The intervoid swelling is due to finer expansive clay present in the voids, which in turn created by coarser non-swelling particles. But, this does not contribute to total volume increase. At the end of intervoid swelling, a large increase in swelling occurs and constitutes about 80% of the total swelling. The slow and continued swelling with time after primary swelling is called secondary swelling.

2.4.2 Bentonite-sand mixtures

Chapuis (1990) presented numerous results of laboratory permeability tests for soil-bentonite mixtures used as impervious liners. The hydraulic conductivity is shown to be correlated to porosity individually, bentonite content or total fines content. The hydraulic conductivity is shown to be correlated to an efficient porosity n*, which corresponds to the pore space available for seepage of the fast moving water.

Bentonite-sand mixtures are comprised of two truly contrasting soils with regard to grain size, permeability, chemical activity and strength which when combined in optimum proportion, can form an excellent seepage barrier that is dimensionally stable and possess a low hydraulic conductivity. Bentonite is contained within the voids between sand particles and in the presence of water hydrates and swells. When the void ratio of bentonite is less than its free-swell capacity, bentonite completely fills the space and presses lightly against the sand particles. If the void spaces in the sand exceeds free-swell capacity of the bentonite, the space will not become completely filled with hydrated bentonite, and so, will contain free water, forming holes or if connected to other water filled holes, channels (Kenney et al., 1992).

Ideal mixtures which contain sufficient bentonite to fill all space and in which bentonite4 is uniformly distributed, have their permeabilities controlled by the permeability properties of bentonite and the sand particles are impervious inclusions in the matrix of hydrated bentonite. In the actual practice, it frequently happens that mixtures either do not contain sufficient bentonite to fill all the voids in the sand framework or the bentonite is inadequately distributed. In either case, permeability of the mixture is increased by water filled defects in the bentonite matrix and as will be reported, might reach a value closer to that of sand than that of bentonite. Therefore, to obtain a low permeability mixture it requires both adequate content of bentonite and proper distribution of bentonite within the mix.

The hydraulic conductivity of a bentonite-sand mixture is dependent on the fabric of bentonite in the mixture. Conditions at the time of mixing determine the fabric of bentonite in the mixture and this fabric controls the permeability of bentonite and influences changes in permeability that accompany any later changes in the chemical nature of the pore-fluid.

Kenney at al. (1992) recommended that in the absence of impervious soils, compacted mixtures of bentonite and sand have been used to form barriers to fluids. Low hydraulic conductivity requires continuity of the bentonite matrix within the mixtures and this in turn requires both adequate bentonite distributions. In well-compacted mixtures containing upto 20% bentonite in dry mass, sand forms the load-supporting framework and gives the mixtures dimensional stability at the macro level.

Bentonite slurry is often used as a sealant. Edil and Muhamma (1992) observed that the ability of a bentonite slurry to rapidly form a filter cake of low permeability on a porous formation is not only important for general stability, but also for sealing qualities. Low permeability is required to prevent migration of water.

Pandian et al. (1995) suggested that, in seepage control, clay cores and cut off walls should be compatible with the surrounding material in terms of compressibility and strength. Bentonite-sand mixtur4es and bentonite-soil mixes, if properly proportioned, can satisfy this requirement. When pure bentonite is used as slurry, it is subjected to considerable volume changes. In such cases, coarse sand or soil available in the vicinity can be added in suitable proportions.

2.5 CRACK FORMATION IN SOILS

Shrinking soils often crack hen they dry. Cracking occurs under different conditions and it cannot always be explained in the same way. Thus, the paste-like muds and sediments that are left behind after flooding or from puddles, crack on drying into a mosaic of polygons. Clods lying on the surface of the soil after tillage often breakdown into smaller fragments on drying. In hot dry regions, where drying can be rapid and intense, many clays do this naturally—the so called self mulching soils. Peds are by definition units of soil separated from each other by surfaces of weakness which are actual or incipient cracks. Long meandering vertical cracks often appear across fields during particularly dry seasons.

In some of the examples mentioned above, shrinkage sets up tensile forces that must exceed the tensile strength of the soil for cracking to occur. In other differential shrinkage, through for example non-uniform drying, sets up complex patterns of shear stress in which the shear stress must exceed the shear strength for cracking. In yet other examples, the cracks may be permanent features that simply open and close in response to drying and wetting, and do not involve strength at all. Cracking is an important component of the soil processes that are vital in various geotechnical, agricultural and environmental applications.

2.5.1 Types of cracks in soil

Gray (1989) discussed the influence of shrinkage cracks on the hydraulic conductivity of compacted clays. He suggested that in the construction of a clay liner, volume stability may prove to be equally as important as hydraulic conductivity. He classified cracks which occur in soil liners into two groups according to their mechanism of occurrence: mechanically induced cracks and physico-chemically induced cracks. Mechanically induced cracks include those which occur as a result of poor construction caused by compaction on weak subgrade or laterally unconfined areas, settlement cracking, and compaction cracking (lift planes) induced by poor compaction and lack of adequate bonding between successive lifts of compacted soil. Mechanically induced cracking can be minimized by proper construction methods and adequate construction quality assurance (CQA).

Physico-chemically induced cracking may be divided into three groups: syneresis cracks, cracks induced by freeze thaw cycles and cracks induced by desiccation. Syneresis cracks are induced by changes in the inter-particle forces resulting from replacement of interstitial water with a low dielectric organic solvent or highly saline aqueous solution (Brown and Anderson, 1983). Cracks formed as a result of freeze thaw cycles result in crack formation and have the net result of increasing the hydraulic conductivity of the soil.

Desiccation cracks are induced by evaporation of water and the consequent shrinkage. Hvorslev (as referenced by Lambe, 1958) subjected soil samples with different loading conditions to drying and found that the pattern of cracking in soil upon drying tended to be perpendicular to the direction of applied force. He concluded that cracking was parallel to particle orientation.

2.5.2 Mechanism of crack formation

George (1968) concluded that as a result of shrinkage, tension stresses will be set up in the soil. He found that the shrinkage stress reaches its maximum value in the early stage of drying when it is highly localized at the exposed surface, and decreases rapidly with depth. This localized stress can be relieved only by surface cracking or plastic flow in materials. Thus the mechanism of desiccation cracking, as he proposed, may be explained as the failure of the material in tension when the tensile stresses developed in the material due to shrinkage exceed the tensile strength of soil.

According to G.D.Towner (1987), when drying clays are prevented from shrinking in one direction, they do not crack until the stress induced in that direction is equal to or greater than the corresponding tensile strength. The tensile stress and the tensile strength increase with decreasing water content (ie., increasing suction), at least when the clay is saturated. The magnitude of the induced stress is equal to that of the change on soil water suction for isotropic shrinkage, but considerably less for enforced anisotropic shrinkage. Furthermore, the induced stress in the constrained direction is approximately a function only of the water content, independent of the state of anisotropy.

Morris et al. (1992) reported that macro cracks were produced by the growth of micro cracks under tensile loading at the crack tips as a result of increased pore water suction. The pore water suction is inversely proportional to the radius of the capillaries and hence to the particle size. The capillary forces associated with soil moisture loss to the atmosphere cause a soil mass to shrink. He also showed that soil macro cracks due to pore water suction are more readily produced in fine grained soils than in coarse grained soils. This is because fine grained soils have smaller particle size and hence smaller inter-granular voids. The smaller voids support large pore water suction. They also reported that conditions for crack propagation are more favourable at the ground surface where pore water suctions are generally largest and self-wt stresses are zero. The depth to which the crack extends is ultimately constrained by the increasing stresses due to self-weight of soil, and the planar length of the crack is limited by intersection with other cracks.

Fang (1994) reported that when water is lost from surface soil mass, tensile forces are established in the drying surface layer. Due to the water loss, soil loses its ability to relieve tensile forces. These stresses are finally relieved by the formation of shrinkage cracks. As soil particles move closer, the surface layer is broken up into pieces of more or less distinct geometric shapes. This geometric shapes of the cracks depend on the clay mineral composition, the heating process, and the pore fluids. Mi (1995) studied the surfacial characteristics of cracks and showed that crack geometry was a function of duration of desiccation and corresponding soil moisture suction.

2.5.3 Effect of Desiccation Cracking in Compacted Clay Liner

Kleppe and Olson (1985) conducted on Shrinkage tests on mixtures of highly plastic clay and sand, with clay contents ranging from 12% to 10%. Cylindrical specimens were used to determine shrinkage of the sand-clay mixtures on a percentage basis. The severity of cracking for each mixture was determined by preparing flat plates of the sand clay mixtures, and rating the size and number of cracks on a 0 to 4 point scale (4 being the most severely cracked). Kleppe and Olson (1985) observed that shrinkage strain increased with increasing clay content and with increasing compaction water content, but was independent of compactive effort. Several specimens were soaked after compaction and prior to drying. Shrinkage strains for the specimens increased relative to the strains for specimens dried immediately relative to the strains for specimens dried immediately after compaction, and shrinkage strain became nearly independent of compaction water content. Results of the plate tests indicated that the severity of cracking with increasing clay content, and increases if the soil is soaked prior to drying. Comparison of cracking severity and shrinkage strain for each mixture showed that shrinkage strains >50% may produce cracks in compacted soils, and shrinkage strains >10% are likely to produce severe cracking

Daniel and Wu (1993) conducted shrinkage tests when developing an acceptable zone for compaction following methods described by Daniel and Benson (1990). Specimens of highly plastic clay were prepared at compaction water contents between 10% and 20%, using three compactive efforts. Their results indicate that shrinkage increases with increasing compaction water content, but the relationship between compactive effort and shrinkage strain is less clear. At low compaction water contents, shrinkage decreased with increasing compactive effort. No clear trend was apartment at higher water contents.

Boynton and Daniel (1985) performed hydraulic conductivity tests on desiccated clay. Specimens were trimmed from plates of compacted soil prepared at three water contents, and plates in flexible- wall permeameters for testing. A specimen that had not been desiccated was also tested. At low effective stresses the hydraulic conductivity of the desiccated specimens was typically one-half to one order of magnitude greater than the hydraulic conductivity of the undesiccated specimen. The hydraulic conductivity of the desiccated specimens decreased rapidly with increasing effective stress (up to 546 kPa), presumably due to the closure of cracks. The hydraulic conductivity decreased more gradually as the effective stress was raised beyond 56 kPa. At the highest effective stress, the hydraulic conductivity of each desiccated specimen was still greater than the hydraulic conductivity of the undesiccated specimen.

Sims e al. (1996) obtained similar results from tests on specimens collected in thin-walled sampling tubes from a desiccated natural deposit of clay. They also reported that the hydraulic conductivity decreased rapidly with increasing effective stress up to 120 kPa, and attributed the reduction in hydraulic conductivity to closure of cracks.

Benson et al. (1993) performed hydraulic conductivity tests on two low-plasticity clays (Live Oak and Wenatchee clays) subjected to four cycles of wetting and drying. Specimens of each soil were prepared at 3% dry of optimum water content. Optimum water content, and 3% wet of optimum water content. Specimens of Wenatchee clay compacted dry of optimum and at optimum water content showed no increase in hydraulic conductivity when desiccated, but the hydraulic conductivity of the specimen prepared wet of optimum water content in creased by a facto of three. For the Live Oak clay, the hydraulic conductivity of all specimens increased one order of magnitude within the first two wet-dry cycles, but ceased increasing thereafter.

Phifer et al. (1994) and Phifer4 et al. (1995) conducted tests on processed kaolinite and a natural kaolinitic soil to assess how desiccation affected hydraulic conductivity. Desiccation caused the specimens to shrink significantly (volumetric shrinkage strains – 20%) and the dry unit weight to increase. However, no cracks formed, and decreases in hydraulic conductivity were observed. Subsequently, Phifer et al. (1995) conducted laboratory scale lysimeter tests to determine if similar behaviour would be observed at larger scales (Drumm et al. 1997). Results of the lysimeter tests showed that the soil cracked when desiccated, and that the bulk hydraulic conductivity (i.e., crack and matrix flow) increased as much as two orders of magnitude.

Day (1997) performed hydraulic conductivity tests on a specimen prepared from a mixture of montmorillonite and sand, and a specimen of highly plastic clay from a natural deposit. Both specimens were subject5ed to five of wetting and drying. At the end of one day of permeation, the hydraulic conductivity of each specimen was observed to be essentially the same as its initial hydraulic conductivity.

2.6 CASE HISTORIES ON DESICCATION INDUCED CRACKING ON COMPACTED CLAY LINERS

Montgomery and Parsons (1989), Corser an Cranston (1991), Benson and Khire (1995), Albrecht (1996), Khire et al. (1997), and elchior (1997) have all shown that cracks often form in compacted clay barriers used in covers when insufficient protection is provided for the compacted clay. These studies also show that desiccation cracks typically penetrate the entire thickness of the clay barrier and that the cracks remain pervious. The crack aperture tends to be very small (<0.5 mm), which prevents infilling by most particles under field gradients. Hydrological data from these studies also show that the presence of desiccation cracks results in preferential flow.

Montgomery and Parsons (1989) describe two test sections constructed near Milwaukee, Wisconsin, that simulated landfill covers comprised of a clay barrier layer overlain by a vegetated surface layer. Both had a compacted clay layer 122 mm thick; in the other the surface layer was 450 mm thick. Four years after construction, test pits showed that desiccation cracks penetrated the clay barriers. Percolation measurements made using lysimeters installed beneath the test sections indicated that percolation was transmitted from the base of the cover shortly after precipitation events, which is a strong indication that preferential flow through cracks was occurring.

Corser and Cranston (1991) describe three sections constructed in southern California, two of which simulated final covers. Both test sections contained a compacted clay layer 915 mm thick. In one test section, the clay barrier was overlain by a geomembrane and an additional 610 mm-thick layer of cover soil. The other test section was similar, except the clay barrier was not covered by a geomembrane. Inspection of the clay barriers several months after construction showed that the clay barrier overlain by a geomembrane was unaffected. In contrast, the other clay barrier was extensively cracked. An inspection three years later showed similar results, except that the cracks penetrated completely through the clay barrier that was not covered by a geomembrane (Patrick Corser, personal communication, 2000). Infilling of the cracks was not evident.

Benson and Khire (1995) describe cracking of a clay barrier in a cover in southern Wisconsin The transect was exposed during construction of a lateral expansion. Despite the presence of a surface layer 150 mm thick and a protective layer 600 mm thick, the clay barrier was extensively cracked less than five years after construction. Moreover, none of the cracks were filled with particles. In fact, the only material found in the cracks was a network of fine plant roots, the presence of which is a clear indication that the cracks were a source of water.

Albrecht (1996) investigated how desiccation affected the clay barrier in a cover in central Wisconsin. The cover consisted of 600 mm of compacted clay overlain by a gegetated surface layer 150 mm thick. Test pits were excavated in the cover and undisturbed samples of the clay barrier were retrieved in thin-wall sampling tubes and as large (300-mm-diam) blocks. The clay was hard and dry, and had a blocky structure created by numerous cracks. Fine roots were also present within the cracks. Hydraulic conductivity tests conducted on the undisturbed specimens yielded hydraulic conductivities of 7.2×10^{-7} cm/s (sampling tube) and 6.9×10^{-5} cm/s (block), whereas the hydraulic conductivity of the clay barrier was 1×10^{-8} cm/s when constructed. No infilling was observed in the cracks because the cracks had very small apertures.

Khire et al. (1997) present hydrological data from a final cover test section constructed in central Washington. The cover consisted of a compacted clay barrier 600 mm thick overlain by a vegetated surface layer 150 mm thick. Two years after construction, the percolation rate abruptly increased by a factor of 5 and pulses of percolation were observed soon after precipitation events. The increase in percolation rate and the pulses of percolation were attributed to preferential flow through desiccation cracks. Exhumation of the clay barrier showed that it contained numerous desiccation cracks with very fine apertures. No infilling of the cracks was observed. However, mold was present in some crack surfaces, indicating that flow through the cracks had been occurring.

2.7 REINFORCED EARTH

Soil is a relatively cheap and abundant material, which makes it ideal for use in construction. Soil is capable of providing very high strength in compression, but virtually no strength in tension. In civil engineering applications soil usually fails in shear. Like other construction materials with limited strength, soil can be reinforced with foreign material to form a composite material that has increased shear strength and some apparent tensile strength.

The term 'Reinforced soil' refers to soil that has been strengthened by placement of reinforcing material within the soil mass in the form of strips, bars, sheets or grids. When load is applied to this soil mass, these materials resist the tensile stresses which develop within the soil mass.

The concept of reinforcing soil with tensile resisting elements has been widely accepted in engineering practice. This concept is used to improve the soil properties such as bearing capacity, shear strength etc. This concept was first developed by Henry Vidal, by which he demonstrated that the introduction of the reinforcing elements in a soil mass increases the shear resistance of the medium. Reinforced earth technique is considered to be an effective ground improvement method because of its cost effectiveness, and easy adaptability.

Reinforced soils can be obtained by either incorporating continuous reinforcement inclusions within a soil mass in a certain pattern or mixing discrete fibres randomly with a soil. Randomly distributed fibre reinforced soils have recently attracted increased attention in many geotechnical engineering application.

In comparison with systematically reinforced soils, randomly distributed fibre reinforced soils exhibit some advantages. Preparation of randomly distributed fibre reinforced soils is similar to soil stabilization by admixture. Discrete fibres are simply added and mixed with the soil, much like cement, lime or other additives. Randomly distributed fibre reinforced soil offer strength isotropy and limit potential planes of weakness that can develop parallel to oriented reinforcement.

2.7.1. Reinforcement with Discrete Random Fibres

Experimental results reported by various investigators (Verma and Char 1978; Hoare 1979; Radoslaw and Jan Cermak 1986; Gopal Ranjan et al. 1994; Gray and Al-Refai 1986; Venkatappa Rao and Balan 2000; Prabhakar and Sridhar

2002; Arvind Kumar et al. 2005 etc.) have shown that fibre reinforcement causes significant improvement in the strength and stiffness of soil.

Verma and Char (1978) performed triaxial compression tests on sands reinforced with solid plates, perforated plates, continuous wire, and rectangular shaped fibres comprised of aluminium or mild steel. Comparison of cohesion and angles of internal friction for unreinforced and reinforced samples indicated that the shear strength increased with increasing percentage of reinforcement up to a point, after which further addition of reinforcement resulted in only a small increase or no increase in shear strength. The effective stress friction angle (ϕ')increased from 36º to 45º when sand was reinforced with 2% aluminium perforated circular plates. When the quantity of aluminum plates was increased to 4%, ϕ' was 44º.

Andersland and Khattak (1979) performed consolidated-drained (CD) and consolidated-undrained (CU) triaxial tests on kaolinite reinforced with pulp fibres 1.6 mm long and 0.02 mm in diameter. fibres were added at 16% and 40% by weight. As the fibre content increased, the mode of failure changed from brittle to plastic. For 16% fibre content, the peak deviator stress increased 43%. Unreinforced kaolinite reached peak deviator stress at 8% axial strain whereas the fibre-reinforced specimens continued to increase in strength beyond 20% axial strain. For the CD tests, ϕ' was 20% for unreinforced kaolinite and 31º when reinforced by fibre. CU tests yielded ϕ' of 20º for unreinforced kaolinite and 39º for fibre-reinforced kaolinite.

Hoare (1979) performed triaxial compression tests on dry angular crushed sandy gravel reinforced with two materials: strips of polypropylene/nylon fabric (66 mm long x 7 mm wide) and twisted polypropylene chopped staple fibre (5 cm long). Results of the tests showed that reinforced specimens were significantly stronger than unreinforced specimens compacted to the same porosity using the same method. However, specimens compacted with the same effort decreased in strength when the amount of reinforcement was increased.

Gray and Ohashi (1983) developed a mechanistic model to characterize the effects of reinforcement. They proposed that strips develop tension in the shear zone as a result of anchorage occurring at the soil reinforcement interface outside the shear zone. The tension in the strips adds to the shear strength of the soil. Furthermore, after peak strength is reached, the strips remain in tension. Consequently, the post peak loss in strength is smaller than would be observed for unreinforced specimens.

Gray and Ohashi (1983a) performed direct shear tests on sand reinforced with natural and synthetic fibres and compared the results to predictions made with the model. The test results were consistent with behaviour predicted by the model. Gray and Ohashi also observed that Mohr-Coulomb failure envelopes for reinforced soil were biliner. Reinforcement increased the friction angle of the sand until a 'critical confining stress' $\left(\sigma_c^{'}\right)$ was achieved. At confining stresses beyond $\sigma_c^{'}$, the failure envelope was parallel to the envelope for unreinforced sand.

McGown et al. (1985) studied the influence of randomly distributed polymeric mesh elements on the strength of granular soil. Mid-Ross sand was used for all experiments and pieces of a poly-propylene-based mesh were used as reinforcement. CD triaxial tests were run on unreinforced sand and sand reinforced with 0.18% of mesh by dry weight of the soil. Failure envelopes showed that the shear strength of the sand was increased by adding the mesh. The percentage increase in strength was found to be the most significant at low confining pressures.

McGown et al. also conducted tests using model footings to determine if the strength increases it would be realized under more realistic loading conditions. The results showed, for the same amount of settlement, that footing placed on reinforced sand carried twice the vertical stress of the same footing placed on unreinforced sand. During unloading, settlements in the reinforced sand were recovered.

Radoslaw and Jan Cermak (1986) based on the triaxial drained compression tests on fibre reinforced sand, found out that a substantial increase in shear strength can be gained with a fibre concentration of 2% by volume (compared to unreinforced sand). This increase can be as much as 70% whereas when the fibre concentration is 0.5%, the reinforcement effect drops to about 20% or less.

Donald H. Gray and Al-Refai (1987) compared triaxial compression test results on dry sand reinforced with randomly distributed discrete fibres and oriented continuous fabric layers. They found that increasing the amount of reinforcement or number of fabric layers increased the peak strength and for, dense sand, reduced the post-peak loss in strength. For the same area ratio and weight fraction, rougher reed fibres were found to be more effective than smoother glass fibres. Test results showed that both types of reinforcement systems increased the strength and modified the stress deformation behaviour of sand in a significant manner. Apparently, the rough surfaces produced a better grip between the reinforcement and the soil. Gray and Al-Refai also found that the failure envelopes exhibited the biliner shape observed by Gray and Ohashi (1983).

Setty and Rao (1987) and Setty and Murthy (1990) carried out triaxial test, CBR test and tensile strength tests on silty sands and black cotton soil, respectively reinforced with randomly distributed polypropylene fibres. The test results indicated that both the soils showed significant increase in cohesion intercept and a slight decrease in angle of internal friction (ie. overall effect is to increase shear strength), with an increase in fibre content upto 3% (by weight).

Arenicz and Chowdhury (1988) conducted tests using a shear box and model earth walls constructed with reinforced beach sand. Randomly oriented aluminium particles, disks, fibres, and cast-iron particles were used as reinforcement.

Results of tests conducted in the shear box indicated that quantity and shape of the reinforcing elements affected shear strength. Bulky shapes were least effective in increasing strength, whereas fibrous shapes were most effective. Random reinforcement used as backfill material increased the critical height of the model walls. The increase in critical height ranged from 1.5% (cast-iron particles to 12.6% (aluminium fibres).

Shewbridge and Sitar (1989) performed direct shear tests on sands reinforced with six types of reinforcement to study the effect of concentration, stiffness and bond strength on the deformation of sand. Reinforcing materials included bungee cord, parachute cord, wood dowelling, aluminium rods, and steel rods. A large rectangular direct shear box (0.76 m x 0.40 m) was used. Results of the tests showed that reinforcement increased the strength of the sand. However, the increase in strength was not linerly related to the percentage of reinforcement in the shear zone, as mechanist models developed by others suggest [e.g. Gray and Ohashi (1983)]. The rests also showed that the reinforced soils had a larger volume of soil active in shear and that the changes in volume are not spatially homogeneous.

Maher and Gray (1990) performed triaxial compression tests on sands and glass spheres of varying angularity and gradation that were reinforced with discrete randomly oriented fibres. They also developed a stochastic model to predict increases in strength obtained randomly oriented fibres. The experimental and modeling results agreed reasonably well. Results of the tests showed that the failure envelopes for both angular and smooth particles were biliner. The transition in slope near the critical confining stress was gradual for smooth particles and abrupt for angular particles. The critical confining stress decreased as the gradation, angularity of particles, and aspect ratio of the reinforcement increased. However, changes in particle size and content of reinforcement did not affect the critical confining stress.

Fatani et al (1991) conducted direct shear tests to evaluate the strength behaviour of soil by reinforcing with aligned and randomly oriented metallic fibres. It was observed that reinforcing a silty sand with rigid, semi rigid, and flexible inclusions improved the shear strength considerably when the inclusions ere inclined at certain orientations to the shear plane.

Leung Christopher (1992) has provided a derivation of fibre-bond strength. His study focused on 'debonding' of fibres from a matrix by two theories of failure. He reported that the two modes of failure, fracture-based and strength-based, may both occur in a given specimen, but may differ in degree. For fracture-based failure, the debonded region was treated as a crack that will lengthen if a critical toughness was exceeded along the fibre-matrix interface. Leung believed that the two modes of failure can only be distinguished in a given material by varying the size or volume of fibres through a testing program. His study was limited to singular fibres, of predetermined orientation, in matrices of both mortar and epoxy resin.

Lawton et al (1993) used discontinuous multi-oriented polypropylene elements to reinforce compacted and rained specimens of sand. Triaxial quantity of reinforcement, surface roughness and orientation of the reinforcing elements. Result of the tests showed that reinforcement increased the peak deviator stress as much s 106%. Smooth reinforcing elements generally resulted in a larger peak deviator stress but lower stiffness than was obtained with rough elements. The results also showed that reinforcement was most effective when placed in layer and enough reinforcement was added to cover the entire cross section of the specimen.

Gopal Ranjan et al (1994) conducted a series of triaxial compression tests to study stress-strain behaviour and increase in shear strength of sand due to fibre inclusion. Test results show that fibre reinforcement increases shear strength and stress-strain behaviour significantly. They used plastic fibres as reinforcement. They found that the finer sand size particles had significantly greater fibre-bond strengths, thus they were less likely to fail by conditions of slippage than the coarser grained soils. Silts, being even smaller than fine-grained sands, might then be expected to achieve a stronger bond with fibres.

Al Wahab and El-Kedrah (1995) used fibres to reduce tension cracks and the amount of shrink/swell in compacted clay. Polypropylene fibres were used. The investigators used a soil with a liquid limit of 54%, a plasticity index of 26%, and an optimum ,moisture content of 20,5%. fibre contents of 0%, 0.2%, 0.4%, and 0.8% were studied. An optimum fibre length of 12.7mm was selected. The researchers defined crack index as the ratio of area of cracks that are deeper than 2mm to total surface area of soil sample. The area of crack is equal to product of its length and width. The results of this study showed that the fibre content did not effect the compaction characteristics, but reduced the amount of shrink/swell by additional 30 to 35%. The reduction in crack index was about 25 to 45%.

Michalowski and Zhao (1996), based on triaxial test results, indicated that the steel fibres led to an increase in the peak stress, and the stiffness prior to reaching failure. They also reported that polyamide fibres produced an increase in the peak shear stress for large confining pressures, but the effect was associated with a considerable loss of stiffness prior failure and a substantial increase of the strain to failure.

Yildiz Wasti and Mustafa (1996) conducted laboratory model tests on a strip footing supported by sand reinforced by randomly distributed polypropylene fibres and mesh elements in order to compare the results with those obtained from unreinforced sand. For the mesh elements there appeared to be an optimum inclusion ratio, whereas fibres exhibited a linerly increasing trend on the basis of n increasing trend on the basis of an increase in ultimate bearing capacity for the range of reinforcement amounts employed. Also, the effectiveness of randomly distributed discrete reinforcing inclusions in improving the properties of sand depends on the quantity as well as the shape of the inclusions.

Kumar et al (1999) based on their laboratory investigations conducted on silty sand and pond ash specimens reinforced with randomly distributed polyester fibres concluded that the fibres increased the peak compressive strength, CBR value, peak friction angle and ductility of the specimens. They also reported that the optimum fibre content for both silty sand and pond ash was approximately 0.3%-0.4% of dry unit weight.

Kaniraj and Havangi (2001), conducting unreinforced fly ash soil mixture, concluded that randomly oriented polyester fibre inclusions increased the strength of the raw fly ash soil specimens as well as that of the cement stabilized specimens and changed their brittle behaviour to ductile behaviour.

Prabhakar and Sridhar (2002) conducted compaction test and undrained triaxial compression test to study the effect of sisal fibres. It was concluded that the shear stress increased nonlinerly with increase in length upto 20mm and beyond, whereas an increase in length reduces the shear stress. The increase in fibre length and fibre content also reduces the dry density of the soil. The nonliner variation of shear strength with fibre content leads to the conclusion that the behaviour of the fibre included soil may be nonliner in high stress regions.

Tingle et al (2002) concluded from full-scale field tests that fibre-stabilised sands were a viable alternative to traditional road construction materials for temporary or low-volume roads.

S.K.Dash et al (2003) conducted laboratory model tests to study the relative performance of different forms of reinforcement (ie., Geocell, planar and randomly distributed mesh elements) in sand beds under strip loading. The results demonstrated that the geocell reinforcement is the most advantageous soil reinforcement technique among those investigated. For the case with randomly distributed reinforcement, failure was recorded at a load of around 1.8 times the ultimate capacity of the unreinforced case and a settlement of about 10% of the footing width.

Aravind Kumar et al (2005) conducted unconfined compression tests on soft clay mixed with various percentages of polyester fibres. It was observed that unconfined compression strength of clay increases with the additional fibres and it further increases when fibres are mixed in clay sand mixture.

2.8 STUDIES ON MARINE CLAYS

2.8.1 Properties of Marine Clays

Marine deposits can be found all along the coastal belt of Indian Peninsula. Narasimha Rao and Kondandaramaswamy (1984), based on investigations on samples from Cochin and Madras, have drawn some useful conclusions on Indian Marine Clays. Indian Marine clays are deposited at high water content close to liquid limit giving rise to poor consistency and high void ratio. The soils have high colloidal activity and are low to medium sensitive. One of the notable features of the marine clay of Eastern Canada, was an unusually sharp change in compressibility at the preconsolidation load, which results in extremely high compression indices. Investigation by Hamilton and Crawford (1959) from their experiments concluded that flocculated samples have steep stress-strain curves and develop peak strength at low strains. Also dispersed samples have flat stress-strain curves and continue to increase in strength even at high strains. They found that flocculated structure develops less pore water pressure on shearing than dispersed structure.

Quigley andThompson (1966) produced the evidence of the change of undrained shear strength of samples in which ionic replacement is effected without disturbing the sample. Nambiar et al. (1985) made an extensive study on the engineering behaviour of fine-grained carbonate soil from the west coast of India.

Mathew and Rao (1997) carried out tests with homo-ionized systems. They bring out the marked influence of valence and hydrated radii of the adsorbed cations on the consistency limits of marine clay. The results show that with the increase in valence there is a significant reduction in liquid limit and the plastic limit is marginally affected. It has also been observed that by changing the valence of the cations from monovalent to divalent or trivalent, the system changes from CH (clays of high compressibility) toCI (clays of low compressibility).

2.8.2 Physical Properties of Cochin marine clays

Careful examination of X-ray diffraction indicated that the primary mineral is montmorillonite in Cochin marine clay. Narain and Ramanathan (1970) were perhaps the earliest to observe the physical properties of marine clays in Kerala. According to them, the marine clays of this region undergo irreversible changes in plasticity characteristics. Air drying was found to cause formation of aggregates, which was considered responsible for the change in plasticity. The phenomenon of reduction in plasticity on drying has been attributed to dehydration of hydrated iron/aluminium sesquioxides and of the halloysite mineral, by Frost (1967). He brings out the significant changes during overdrying and indicates the importance of carrying out tests on soils in its natural state. Organic matter and carbonates are recognized as major cementing agents contributing to particle aggregation and reduction in Atterberg limits upon drying (Rao et al., 1989). They indicate the dominance of calcium and magnesium ions in the exchange sites and presence of a high salt concentration (9.4g/l), which facilitates strong interparticle attraction, and a close spacing of the particles.

Sridharan and Rao (197) discussed in detail the mechanisms controlling the liquid limit of clays. The results obtained by Cassagrande and Norman (1958) by the use of direct shear and vane shear apparatuses, respectively, indicate that the strength at liquid limit is of the range of 15-30 g/cm^2. Narain and Ramanathan (1970), while discussing the geotechnical properties of the marine clays from Kuttanad area, focuses attention on the peculiarity of the soil where there is variation in properties caused by air drying. They established that air drying caused formation of aggregates and this was responsible for the irreversible reduction in plasticity. In the case of marine clays, the electrolyte concentration of free pore water can vary.

Detailed investigations by Jose (1988) on Cochin marine clays have drawn the following conclusions.

The Cochin marine clays are susceptible for significant changes in test results depending upon their initial conditions. The natural 'moist' clay when air/oven dried gets aggregated. When tests are performed on these dried samples for index and other properties they will show significant changes depending upon the degree of aggregation. The aggregation is almost irreversible even if the soil is soaked for period of two years. Initially oven dried samples showed reduction in liquid limit to the extent of 60% when compared with the values obtained for natural moist samples. While in the case of plastic limit, the reduction is marginal, and the shrinkage limit registered marginal increase for both the samples whether they air/oven dried initially. The free swell index is reduced to one-third of its original values in certain cases. Since all the clay fractions in marine clay are likely to exist as flocs, defloculation is important compared to other clayey soils. It is also noticed that physical properties of marine clay improve considerably by lime treatment.

2.8.3 Compressibility characteristics

According to Jose (1988), the C_c (compression index) value obtained for Cochin marine clay is considerably high. The resistance to compressibility was found to increase with duration of the load. Consolidation tests with different pore fluids indicate that oven-dried sample behaves like montmorillonite clay.

According to the studies carried out by Benny (1993), on Cochin marine clays, the following findings were obtained, The C_v (the co-efficient of consolidation) increases with load increment ratio upto 1.5 beyond which it tends to decrease. Similarly the values of t_{50} and t_{100} are very high for lower load increment ratios. Both the parameters tend to be constant for values of load-increment ratio higher than 1.0. Considerable improvement in the compressibility was noticed when marine clays are subjected to repeated loading and unloading. Further, compaction studies carried out on air-dried samples of Cochin marine clay, showed considerable improvement in compressibility.

According to Aniamma (1998), for the same length of drainage path, the higher the load increment ratio, the higher would be the values of coefficient of consolidation. From the studies on the effect of drainage on compressibility characteristics, it was concluded that though t_{100} or t_{50} for single drainage was always greater than that of double drainage, c_v values for single drainage was always found more than that of double drainage condition. It was also observed that c_v values increase linerly with the length of drainage path on a log-log plot.

2.8.4 Shear strength characteristics

Compaction studies carried out on air dried samples of Cochin marine clay (Benny, 1993), showed considerable improvement in shear strength. The CU tests showed that the rate of development of pore pressure decreases at higher values of σ_3. This can be attributed to dilation at higher stress levels. The effective stress paths are inclined almost at 45° in the initial stages, which show that the development of pore pressure takes place at a lower rate. This is indicative of higher fabric strength in marine sediments. The unconsolidated undrained tests showed that the samples grow stiffer with increase in σ_3. He found that drying process gave better results.

2.8.5 Retention studies

Solly (2004) conducted studies on the performance of marine clay as liner material in comparison with bentonite clay. Retention studies to check the compatibility with chemicals and studies on physical properties of bentonite when amended with sand and sundried marine clay (SMC) were done. The studies concluded that sundried marine clay with 20% sand or 20% bentonite can meet all the regulations by EPA. Indeed the results of this work motivated for the present study.

2.9. SCOPE OF WORK

A critical review of the literature showed that desiccation induced crack is a serious problem affecting the hydraulic conductivity of any earthen structure, especially compacted clay liner. Bentonite enhanced sand mixtures (BES) are mostly used as liner materials, but only limited studies have been conducted on the desiccation of BES mixtures. Apart from the standards laid down by regulatory agencies, the selection of liner material is usually governed by the availability of materials either at site or near by areas. An assessment of costs will generally indicate whether a liner system should incorporate natural soils, which are available within a reasonable distance of the site, or whether synthetic materials are needed. India has a very long coastline and hence marine clay is abundantly available. In this context, the studies on the performance of marine clay when subjected to alternate wetting and drying is was taken up. The effect of randomly distributed discrete fibres on the properties of standard liner material and the proposed alternative material- sundried marine clay, also formed part of the investigation. So far no studies have been conducted on the desiccation of sundried marine clay.

2.10. OBJECTIVES

1. To develop techniques for the accurate assessment of desiccation cracks in clayey soils when subjected to alternate wetting and drying.
2. Evaluate the change in strength characteristics of the various liner soils with the addition of randomly distributed discrete fibres.
3. Evaluate the influence of random discrete fibre on the tensile strength of liner materials.
4. To determine the effect of fibre inclusion on the hydraulic conductivity of the clay liner soils.
5. To propose the criteria for the liner soils under consideration based upon the parameters like hydraulic conductivity, compressive and tensile strength, reduction in cracks etc.

Chapter 3

MATERIALS AND METHODS

3.1 INTRODUCTION

A review of the available literature strongly points to the need of a proper design procedure for the compacted clay liner and cover which are subjected to seasonal variations to meet the requirements laid down by relevant regulatory agencies. The present chapter deals with a description of the soils and amendments used in the study. The various innovative testing procedures and equipments designed and developed exclusively for the investigations are also dealt with in detail.

3.2 MATERIALS

3.2.1 Soils used in the study

As per the standards prescribed by Environmental Protection Agency the material for compacted clay liners should necessarily satisfy the following norms for the co-efficient of permeability/hydraulic conductivity, plasticity index, minimum fines content, and maximum gravel content. Obviously, only clayey soils will satisfy the above conditions.

As per the current practice, bentonite enhanced by sand is the most popular liner material. The present study aims at investigating the properties of bentonite sand mixture vis-à-vis crack development and propagation along with ways and means to control them. A review of the literature indicated that marine clays can act as a substitute to bentonite sand mixture. Thus two soils were selected for the present investigations on clay liner.

3.2.1.1 Bentonite

The bentonite used in this study is commercially available, highly expansive clay. The percentage of water present in bentonite sand clay liner varies depending upon the climatic conditions, since bentonite, which belongs to the montmorillonite group, has great affinity for moisture. Hence, the bentonite, which was procured for the complete investigations, was thoroughly mixed for uniformity and then preserved in double layer of polythene bags. These bags were stored in airtight bins. The properties of bentonite used are given in Table 3.1.

3.2.1.2 Marine clay

Marine clay used in the study was collected from Mundamveli, in Cochin on the Western coast of India. The earlier investigation reports show that in almost all locations in Greater Cochin area, thick uniform layers of marine clays could be obtained after 3 to 9 meters. Hence, bulk samples of the clay were collected by bore holes advanced by shell and auger method. Bore holes were taken to the clay layers for collection of samples. The boring operations were carried out as per the direction given in IS:1892-1979 (Code of practice for surface investigation for foundations). Care was taken not to include bentonite slurry during the boring operations as it might contaminate the soil samples.

For uniformity among the samples collected from different bore holes, representative samples collected from same depth but different bore holes at various locations of the same site were pooled together and mixed thoroughly into a uniform mass and preserved in polythene bags. This is designated as the moist sample of Cochin marine clay. However, this moist-marine clay does not satisfy the requirement with regard to plasticity index. Earlier researchers have shown that sundried marine clay will satisfy the EPA norms.

Sun dried marine clay (SMC) specimens were prepared by spreading the representative samples of moist clay in large trays and exposing to sunlight and drying to constant weight. The lumps formed during drying were broken by a wooden mallet. The samples towards the end of drying were pulverized using a heavy hammer and passed through 425μ sieve without any loss of material. All the clay samples so prepared were kept in polythene bags. The properties of the marine clay used for the study is given in Table 3.1.

Table 3.1 Properties of soils used

Sl.No.	Property	Bentonite	Moist Marine clay	Kaolinite soil
1.	Specific gravity	2.8	2.67	2.64
2.	Liquid limit (%)	362	102	52
3.	Plastic limit(%)	45	41	30
4.	Plasticity index(%)	317	61	22
5.	Shrinkage limit(%)	1.34	20.3	26
6	Grain size distribution			
	Sand (4.75-0.75mm) (%)	0	17.5	40
	Silt size (0.002-0.075mm) (%)	29.5	41	23
	Clay size (<0.002mm) (%)	70.5	41.5	37

3.2.1.3. Soil of Kaolinite group

Investigations were carried out to compare the behaviour of the montmorillonite group with kaolinite. The kaolinitic soil used in the present study was collected from Kalamassery, about 12 km from Cochin. The samples were collected by open excavation. They were dried, sieved through 425μ sieve and stored in polythene bags. The properties of this kaolinite soil are given in Table 3.1.

3.2.1.4 Sand

Even though, bentonite is impermeable as a liner material, the volume change is of a very high order. Earlier investigators have added sand in different proportions to bentonite to reduce the volume change, and have suggested that a combination of 80% sand to 20% bentonite satisfies the liner requirements. The sand used in the study was collected from the bank of river Periyar in Kerala. The sand was dried and sieved, the portion passing 425μ sieve and retained in 75μ sieve was collected and kept in air tight bins/plastic bags. The properties of the sand used are given in Table 3.2.

Table 3.2 Properties of Sand used

Sl.No.	Property	Value
1.	Specific Gravity	2.64
2.	Effective Size D_{10}(mm)	0.128
3.	D_{30} (mm)	0.17
4.	D_{60} (mm)	0.22
5.	Uniformity Co-efficient	1.72

3.2.2 Amendments used in the study

After confirming from our studies, that the currently accepted practice of using sand bentonite mixtures as clay liner has serious limitations, due to development of cracks on desiccation, attempts were made to effectively control this tendency. The cracks were controlled by reinforcing the soil with the help of randomly distributed discrete fibres.

3.2.2.1 Polypropylene fibre

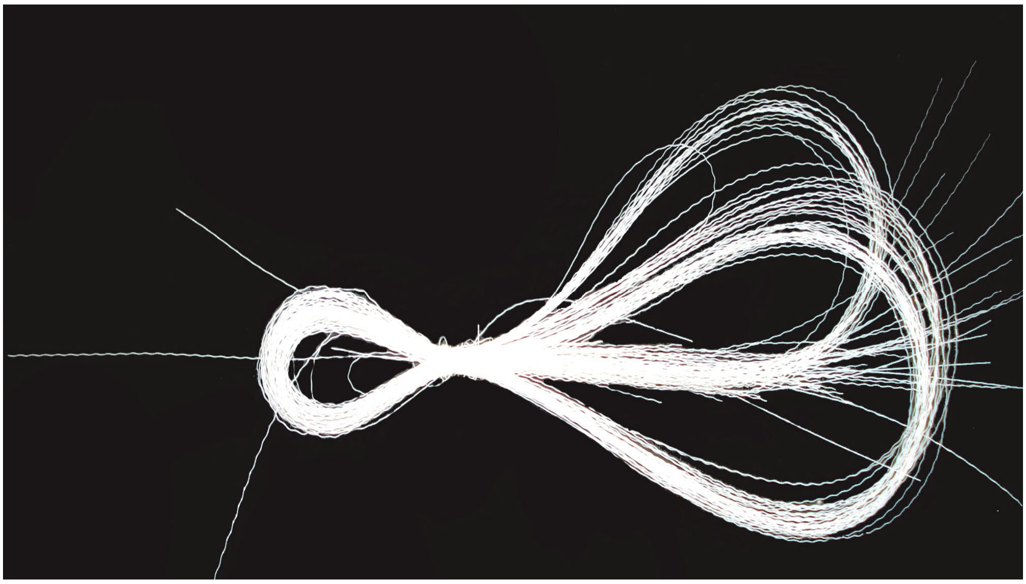
Polypropylene is the most common synthetic material used to reinforce concrete and soil. The primary attraction of this material is its relatively low cost. It easily mixes with soil and has a relatively high melting point which makes it possible to place the specimens of the fibrous soil in the oven and conduct the tests for moisture content. Also, polypropylene is hydrophobic and it is un-affected by the presence of salts in soil or by biological and ultraviolet degradation. These properties of polypropylene make it an excellent fibre that can be used in clay liners, whose expected life span is around 100 years.

Crimped monofilament polypropylene and collated fibrillated polypropylene fibre mesh were used in the study. The monofilament polypropylene fibre was supplied by Garware Synthetic Ltd., Mumbai, and the polypropylene fibre mesh by Taian Modern Plastic Co. Ltd, China. The monofilament fibres comprise of single, individual strands. The collated fibrillated bundles have a lattice structure which opens out during mixing to form a three dimensional network. This fibre network provides phenomenal increase in mechanical bonding under tension as compared with monofilament fibre's due to stress transfer throughout the fibrils, rather than solely at the fibre /matrix interface. The properties of the fibres used in the study is given in Table 3.3.

Table 3.3 Properties of fibres used in the study

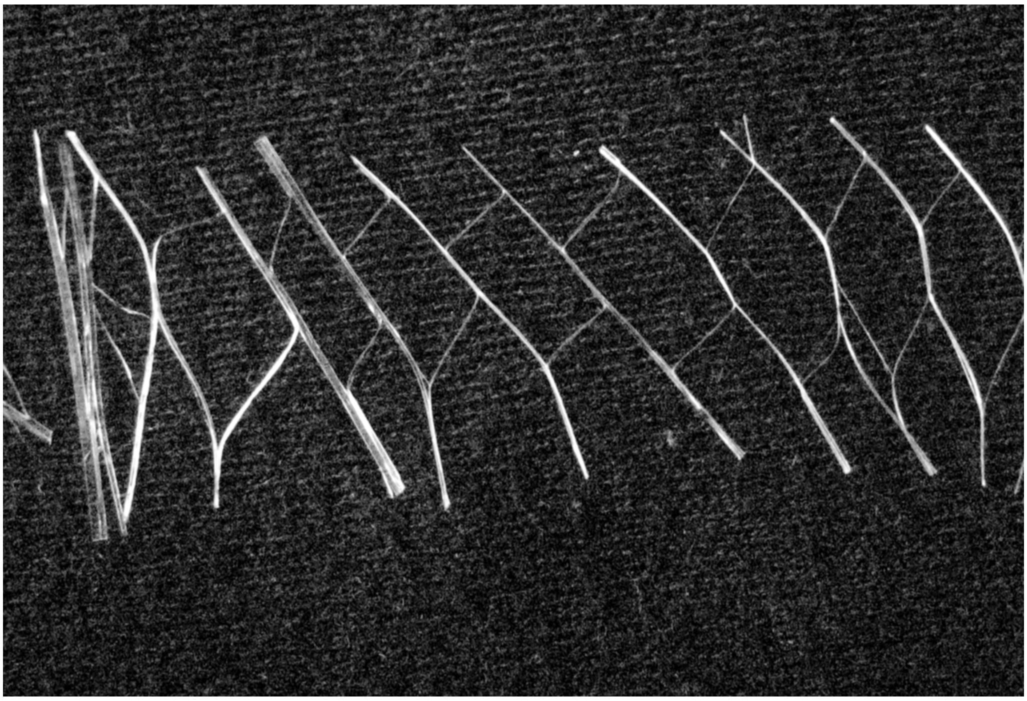
Particulars	Nylon	Polypropylene Monofilament	Polypropylene mesh
Average Diameter (mm)	0.78	0.5	0.1
Acid Resistance	Strong	Strong	Strong
Alkali Resistance	Strong	Strong	Strong
Tensile Strength (MPa)	196	250	>450
Melting Point (^{o}C)	220	163	160-170
Water Absorption (%)	8.63	Nil	Nil

(a)



(b)

(c)



(d)

Fig: 3.1 Photographs of the Fibres Used

a. Nylon Fibre
b. Polypropylene Monofilament
c. Polypropylene Fibre Mesh
d. Fibre Mesh in Expanded form

3.2.2.2 Nylon fibre

Another polymeric fibre, nylon was used as random discrete fibre to reinforce the soils. The properties of the nylon fibre used in the study is given in Table 3.3. Like the polypropylene fibres, the nylon fibre has good acid and alkali resistance. The nylon fibres used in the study was obtained from M/s. Apollo Tyres Ltd., Perambra, Kerala.

3.3 TEST PROCEDURES ADOPTED

3.3.1 Experimental program

Physical experiments were performed in two phases. The former involved the behaviour of the two fine grained soil, when subjected to alternate wetting and drying techniques. Techniques were developed to measure the cracks developed due to desiccation with electronic methods. In the latter, attempts were made to control crack development and propagation using reinforcements inorder to maintain the integrity of the compacted clay liner.

3.3.2 Preparation of samples

In the preparation of all specimens air dried soil was used as compaction is an important variable controlling the hydraulic properties of the soil liner materials. Soils compacted at water contents dry of optimum tend to have a relatively high hydraulic conductivity, whereas soils compacted at a water content wet of optimum tend to have a lower hydraulic conductivity. A typical construction specification requires that the soil, compacted over a specified range of water content, shall not to exceed the optimum water content by 4%, and a minimum dry unit weight of 95% of the maximum dry unit weight obtained from Standard Proctor compaction.

In this study, the test specimens were compacted at their respective maximum dry density unless otherwise specified, and 3% above optimum moisture content (OMC). The fibre content is defined as follows:

$\rho_f = \frac{W_f}{W_s}$ where W_f is the weight of fibre, W_s is the weight of dried soil.

In the preparation of all specimen types, if amendments were not used, the air dried soil was mixed with an amount of water depending upon the OMC of the soil. When fibre was used as amendment, the prescribed content of fibres were first mixed into the air dried soil in small increments by hand, making sure all the fibres were mixed thoroughly to achieve a fairly uniform mixture, and then the required amount of water was added. Afterwards, the soils were sealed in plastic bags and allowed to hydrate for 48 hours prior to compaction. All mixing was done manually and proper care was taken to prepare homogeneous mixtures, at each stage of mixing.

3.3.3 Drying procedure

The heating system for the drying of the soil specimens was developed with the objective of simulating the slow rate of drying that occurs in the field, while maintaining reasonable test times. Two heating systems were designed with the above objective.

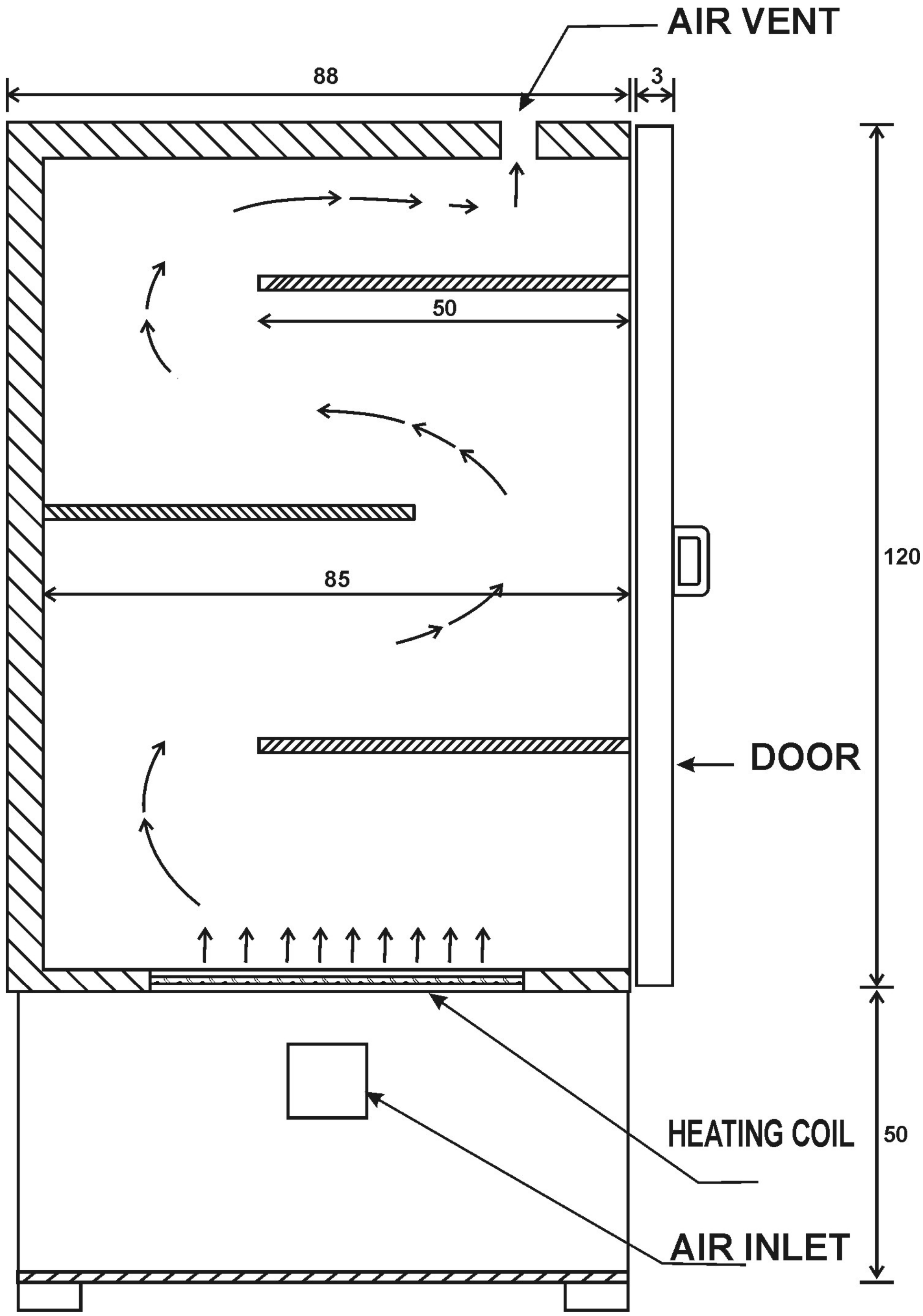


Fig 3.2 Drier

The first heating system is a drier which is designed in such a way that a draft hot air is circulated through stacks as shown in Figure 3.2. Air let in at the bottom, was heated by heating coils and allowed to circulate through moulds stacked in three tiers. Through several revisions of design and fabrication, the temperature in the drier could be maintained upto a maximum of 80^0C with a variation as small as $\pm$ 1.5^0C from bottom to top. This ensured that all the specimens were subjected to the same temperature selected for drying which was 40^0C.

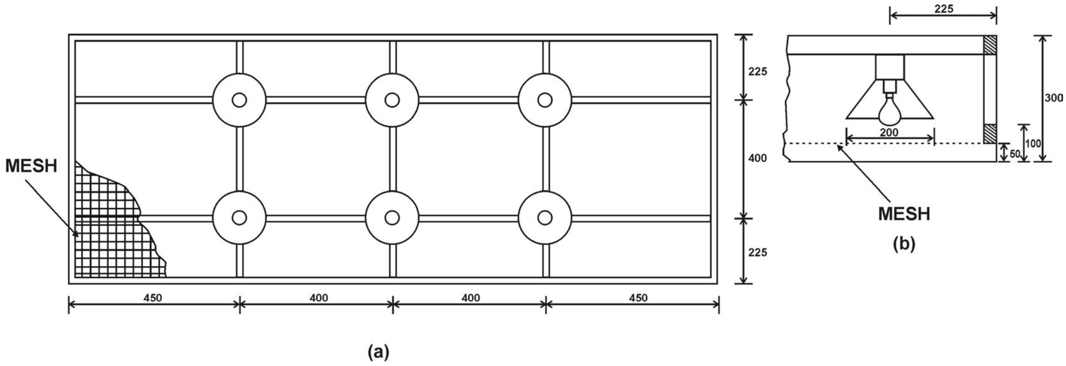


Fig 3.3 Hood System for Drying Of Soils

a) Top View b) Sectional View

The second system shown in Fig.3.3 consists of a hood made of thick ply wood to which, 6 heat lamps of 100W light bulbs, is connected. The mesh provided at the end of the hood ensured uniform distribution of heat. The temperature was controlled by raising or lowering the hood by using a pulley system.

The two heating systems were calibrated so that the same amount of heating energy could be delivered for all the soil specimens. Further all the test specimens, subjected to alternate wetting and drying, was kept in the respective moulds and insulated in the sides using insulating material so that only the top surface was exposed to the heat and drying occurred only from the top. The type of drying adopted is believed to replicate the field situation.

3.3.4 Desiccation tests

3.3.4.1 Experimental set up

The equipment used for this study includes soil moulds, Marshall compactor and a drier for drying the specimens. The mould used in this study was 30 cm in diameter and 10 cm in height. A 5cm height collar was attached to the mould during compaction. Marshall compactor used for compacting the soil sample had a dynamic weight of 4.52kg under a free fall of 45.6 cm. falling onto a 9.8 cm diameter base.

Standard Proctor compaction effort was used for compaction in this study. Taking into account, the difference in size of the mould, a modification to the number of blows per layer of soil was made. The number of blows was calculated using the equation

$$\text{No. of Blows} = \frac{\text{Compaction effort x Volume of mould}}{\text{Wt. of hammer x Ht.of fall x No. of layers}}$$

The soil was compacted in two layers with 103 and 104 blows of the hammer described above.

The compacted soil sample, together with mould is placed in the drier for drying. The mould was kept in ply wood box packed with insulating material to ensure that drying of the soil was only from the top. This arrangement is expected to simulate a better field situation than keeping the mould alone in the drier.

3.3.4.2 Determination of duration and number of wet/dry cycles

During the life of landfill system, the clay cover liner will be exposed to numerous climatic cycles. These cycles include repetitious dry and wet seasons. During the wet season, a significant fraction of the run off that is generated on the land fill surface infiltrates through inter connected conduits to the topmost layer of the compacted clay. This causes the moisture in the clay liner to increase. During the dry season, the excessive moisture in the clay cover liner will be reduced and desiccation cracking may occur. The fluctuation in the degree of saturation of the clay cover liner during its life time was simulated in this study by alternate wet/dry cycles.

Most of the construction activities of liner inevitably are carried out during the dry season, as the onset of monsoons can disrupt construction activities. It is expected that after construction, some desiccation of the liners and covers will take place before the rains will gradually wet and eventually saturate the compacted soil. Hence the investigation of wet-dry cycling effects was carried out in the following sequence: drying of compacted soil, then wetting to saturation, followed by drying and so on. This sequence can be expected to simulate adequately the field conditions.

Several tests were conducted using soil samples to determine the duration of drying and wetting periods. The soil was compacted in the mould and kept for drying. The soil sample was weighed every 24 hours. It was found that the change in weight was insignificant after 5 days. Mi (1995) has showed that the crack propagation ceased after approximately 120 hrs from the start of the drying process. Hence the first drying period starting from the end of compaction was kept as 5 days, the drying period after complete saturation of the sample was kept as 7 days, based on similar tests.

In order to determine the duration of wetting period two soil samples were compacted, dried for 5 days and then water was ponded on top of the sample by placing the collar to the mould. Every two days, soil specimens were removed from the mould for the determination of the water content at the base of the mould. It was found that after 6 days of wetting, the soil was fully saturated and therefore a 7 day period was taken for the wetting period. The experiments were conducted for 3 wet / dry cycles for unamended soils and for amended soils it varied between 1-3 wet/dry cycles.

3.3.5 Measurement of Cracks

Crack dimensions are usually measured using approximate methods. In most cases, qualitative descriptions are provided to estimate the extent of cracking. The irregular shape and complex geometry of cracks prevent accurate measurements of length, width and depth. The width and depth of a crack are not uniform along the length of a crack. Maximum length, width and depth are commonly recorded using measurements with rulers and/or thin gauge wires. Kleppe and Oslon (1985) developed a scale that ranged from 0 to 4 to describe severity of cracking. A crack severity number of 0 indicates absence of cracking, whereas, cracks with width >20mm and with substantial depths are described by a crack severity number of 4.

Al Wahab and El-Kedrah (1995) developed a cracking index to quantify the amount and severity of cracking on a dried sample by introducing a parameter called crack index (CI) , which is given by

Crack index, $CI = \left(\sum_{1}^{n} B.L \right) / A$

Where

n = number of cracks deeper than 2mm on the sample surface area

B = width of cracks

L = Length of cracks and

A = Total surface area of the sample.

Based on the above definition of CI, the width and length of cracks, 2 mm deep or more, were measured at the end of shrink cycles. Al Wahab and El-Kedrah (1995) did not present methods for the determination of the length and width of cracks. This potentially leads to overlooking the effects of the irregular shape of cracks in the calculation of the cracking index.

Mi (1995) and Miller et al. (1998) adopted a similar approach. The surfacial cracked area was used to determine the crack intensity factor (CIF). Crack intensity factor is defined as the percentage of the cracked area to the total surface area of the sample The cracks that were included in the CIF analysis, were those whose widths were more than 1 mm. Miller et al. (1998) has not presented the determination of CIF.

3.3.5.1 Determination of crack intensity factor by digital image processing

Of the two parameters, mentioned above namely crack index and crack intensity factor, the latter has greater acceptability. Thus attempts were made to measure CIF using the techniques of digital image processing. To understand, the digital measurement method for evaluating CIF, a brief introduction to Digital image Processing is given in this section.

The important aspect in digital image processing is image representation. Any monochrome image can be represented by means of a two dimensional light intensity function f(x,y), where x and y denotes spatial co-ordinates and the value of x at any point (x,y) is the gray level or the brightness of the image at that point. The original is taken at the top left corner of image and the horizontal line and the vertical line through the origin are taken as y and x axis, respectively.

The monochrome image f(x,y) is discretized both in spatial co-ordinates and gray level values to obtain the digital image. A digital image can be represented as a matrix whose rows and columns are used to locate a point in the image and corresponding element values give the gray level at that point. Each element in this matrix/digital array is called as picture elements or pixels. A typical digital image of size MxN is represented as given in the following equation

$$f(x,y)=\begin{bmatrix} f(0,0) & f(0,1) & -- & f(0,N-1) \\ f(1,0) & f(1,1) & -- & f(1,N-1) \\ - & - & -- & - \\ f(N-1,0) & f(N-1,1) & -- & f(N-1,M-1) \end{bmatrix}$$

A pixel is the smallest part of a digital image which can be assigned a value. The physical size of the piece of photograph represented by each pixel is determined by how precisely we wish to record the data.

To produce the digital version of a photograph, we take samples of its brightness at regular intervals. The process involves laying a sampling grid over the data and then measuring the average brightness in each square of the grid. There are 16 rows of samples and each row contains 16 samples. The sampling process itself takes the average brightness of the picture underneath each square in the sampling grid and assigns it a value. That value is placed in an array of numbers at the location corresponding to the position of the square in the grid. The values used correspond to the brightness. In this case, 0 represents black and 255 represents the brightest area which the sampling system can record. This image is called the grey scale for the image. 255 may seem like a strange choice for the maximum brightness. It is, in fact, the largest value which can be represented in a single byte of computer storage. A byte consists of 8 bits. Each bit can take the value 0 or 1. Using standard binary number representation the values 0 to 255 can be encoded in a single byte. A byte is normally the smallest unit on which a general-purpose computer can conveniently perform standard arithmetic operations. Brightness values between the extremes of 0 and 255 are represented on a liner scale.

The term resolution is used to describe the level of spatial detail captured by the digital image. Any measure of resolution is specified only in terms of number of pixels. The density of pixels in an image reflects the resolution of the image. As the resolution increases, the storage requirement also increases.

Different steps involved in Digital Image Processing

There are a number of well defined processes which go to make up a typical image application. Some or all of these steps are needed by just about every application which involves image processing. The steps are:

Acquisition—The capture of the image in digital form.

Enhancement—A rather non-specific improvement of the perceived quality of the image.

Restoration—Quantitative correction of the image to compensate for degradations introduced during acquisition. Restoration is used in preference to enhancement when there is sufficient information available about the degradations.

Segmentation—Breaking up the image into areas which correspond to physically meaningful objects.

Analysis—Identification and measurement of the various objects apparent in the image.

Each step in a typical application might involve use of a number of image processing functions.

The steps involved in the digital image analysis of desiccated samples for determination of CIF consists of image acquisition and image enhancement. The image acquisition is done using a digital camera, SONY DSCW35. Image enhancement technique is defined as a process of an image processing such that the result is much more suitable than the original image for a 'specific' application. The word specific is important because the method that is more useful for an application may not be suitable for another application. Enhancements are often used to try to improve the detectability of certain features in an image, so that a human observer will be able to see them more easily. Sometimes, it may be necessary to change the overall appearance of the image dramatically to allow the important features to be seen. The term enhancement may seem a strange one to apply in this case, since the result is quite different from the original. In this case we can think of the technique as enhancing the detectability of the features of interest, rather than enhancing their perceived quality.

The image enhancement approaches can be put into two broad categories and they are spatial domain approach, and frequency domain approach. In the spatial domain approach, the pixels of an image are manipulated directly. The frequency domain approach is mainly based on modifying the Fourier transform of an image. Spatial domain approach was used for image enhancement in this study.

3.3.5.1.2 Analysis by MATLAB Version 7

The software 'MATLAB' version 7was used to analyze the digital images. This software has several features in digital image processing to display, edit, enhance and analyze digital images at different resolutions. It also supports standard digital image processing functions such as the enhancement of contrast smoothening, sharpening, threshold and edge detection.

The hardware requirements for this determination are minimal since any Pentium computer has the capability to perform digital image studies. A digital camera was used to capture images of the desiccated samples. The Image Acquisition was done using SONY DSCW 35, model (7.2 mega pixels) digital camera. The size of the image varied between 3072x2304 to 1632x1224.

Crack Intensity Factor (CIF) is defined as the ratio of the area of cracks (A_c) to the total surface area (A_t) of a drying soil mass. The areas were determined using photographs of a desiccating soil. Cracks appear darker than remaining uncracked soil surface in photographs of a drying soil. The contrast in grey level between cracks and uncracked soil was sufficiently high, so that the obtained digital images could be segmented into cracks and uncracked soil using a simple grey threshold. If the input images is $f(x,y)$ and the threshold image is $g(x,y)$, the equation of the thresholding operator is given by:

$$g(x,y) = 0 \text{ if } T_1 \le f(x,y) \le T_2$$

$$g(x,y) = 255 \text{ if } f(x,y) > T_2$$

The number of pixels having the range of values between T_1 & T_2 is obtained. Scanned photographs of soil surfaces were analysed using MATLAB to determine CIF. Determination of CIF for a desiccated sample is given is Section 4.4.3.1.2.

3.3.6 Compaction Test

Since compaction has significant effect on the hydraulic conductivity of compacted clay liners, the maximum dry density and the optimum moisture content for the different soil samples were obtained by conducting the Standard Proctor compaction test. The test was carried out according to IS 2720 (Part VII)-1985. For the tests, dynamic compaction was given using automatic compacting machine. Currently, there are no standards that deal specifically with fibrous soil. Hence the mixing and compaction procedures related to amended soil was carried out similar to the unamended one.

3.3.7 Hydraulic conductivity tests

The testing equipment was a modification of conventional consolidometer equipment (Mitchell and Madsen, 1987; Kenney et al. 1991) as shown in Fig.3.3. Samples were prepared at a water contents of 3% above OMC and at maximum dry density. They were contained in rigid wall consolidation cells and loaded in conventional loading frames. The top cap was fitted an O-ring to prevent loss of fluid between the top cap and the wall of the cell, allowing the use of fluid back pressure if required. Top cap and base plate each had two drainage ports to enable the porous stones to be flushed. Soil specimens had cross sectional areas equal to 50cm^2 and an initial thickness of 20mm. Each specimen was consolidated using a load increment ratio equal to 1. Between consolidation steps, falling head permeability tests were conducted. Time taken to perform a falling head permeability test was between several days and several weeks depending on the magnitude of permeability of the soil.

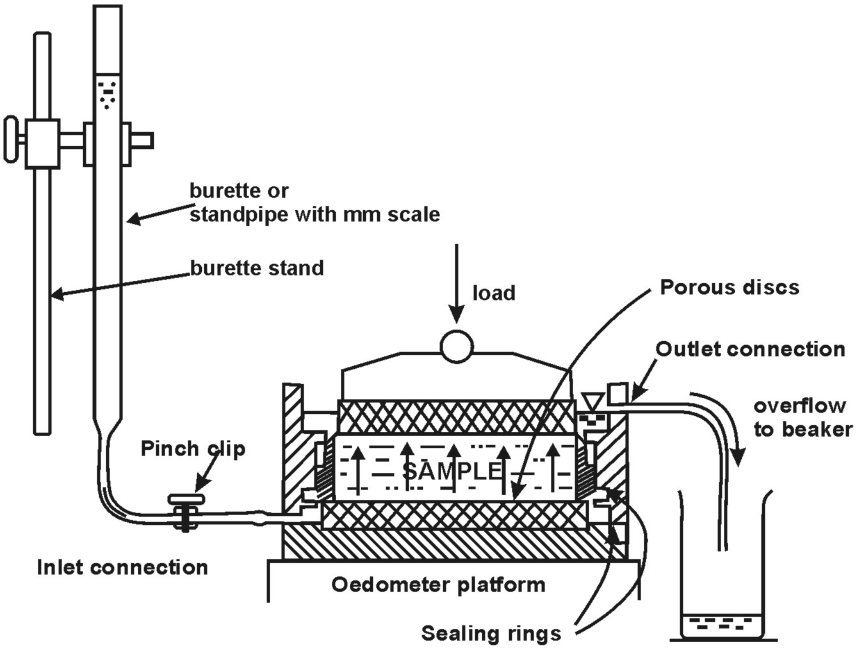


Fig 3.4 Arrangement for Falling Head Permeability Test in Oedometer Consolidation Cell

3.3.8 Tension tests

The tension test assembly fabricated consists of a specially designed fixture installed in the direct shear device as shown in Fig.3.5. Soil fibre mixture was compacted into the fixture at maximum dry density.

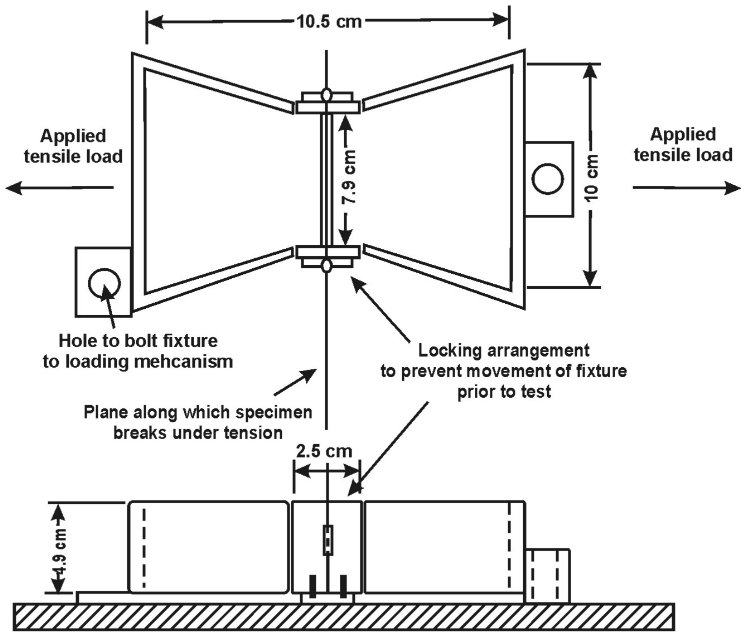


Fig: 3.5 Tension test fixture installed in the direct shear device

Tension tests were conducted at a displacement rate of 0.25 mm/minute. In the above mentioned tests uniform fibre distribution could be confirmed by visual inspection of the fibres at the broken faces of the specimen. The specimen had a length of 10.5cm, thickness of 4.85 cm. The width of the specimen at the ends was 10cm and that at the centre was 7.5cm. This means the cross sectional area subjected to tension for failure was 7.5cm x 4.5cm.

3.3.9 Unconfined compression test

The conventional unconfined compression apparatus was employed in the tests. The tests were conducted at optimum moisture content and maximum dry density for both unamended and amended soils. In order to ensure uniform compaction, the required quantity of sample was placed inside the mould in five steps and proper compaction given.

Chapter 4

DEVELOPMENT OF DESICCATION CRACKS IN CLAY LINER SOIL

4.1 INTRODUCTION

The modern landfill is a very complex structure engineered to protect the environment from the liquid that seeps from the waste, called leachate. This leachate to percolate downward into the ground as well as from the products of decomposition that takes place in the waste which may last for decades. An important component of a landfill is a layer of compacted, low permeability soil that is intended to act as a hydraulic barrier and minimize infiltration of water into the mass, when it is part of a cover system, or prevent the leachate from contaminating the ground water, when the soil is part of a liner system. According to EPA (1989) the soil liners shall ensure that the hydraulic conductivity is equal to or less than 10^{-9} m/sec. which obviously is the prime criterion in the selection of liner material. During certain stages in the life of a landfill, it could be subjected to seasonal changes, resulting in significant variation of water content leading to the desiccation of the clay liner materials and thus posing a major threat to the integrity of the system as a hydraulic barrier.

4.2 SELECTION OF LINER SOILS FOR THE STUDY

4.2.1 Introduction

For any material to be used as a liner, it should have low permeability and should be compatible with the leachate generated in the landfill. Apart from the requirement of hydraulic conductivity laid down by the regulatory agencies, certain additional norms should also be met by the soil. First the soil should have at least 20% fines. Secondly, the plasticity index should be greater than 10%. Thirdly, the coarse fragments should be screened to not more than about 10%

gravel size particles. The norms for clay liners are given in detail in Chapter 2. Based upon these requirements, the following clay liner materials were selected for the study.

4.2.2 Bentonite enhanced sand mixture

The application of bentonite is currently, the most accepted practice for lining purposes. Even though, bentonite is impermeable, its compatibility to leachate, and volume change behaviour are susceptible. In order to increase the strength and volume stability it should be blended with coarser particles for use as liner material (Kleppe and Olson, 1985). Sand is the material most commonly blended with bentonite, the proportioning of sand and bentonite in the mixture is done depending on the specified hydraulic conductivity. The ideal bentonite sand combination (the mixture which satisfies the liner requirements) is 20% bentonite and 80% sand, which was selected for the study. The properties of the bentonite and sand used are given in Chapter 3. This liner soil is termed as bentonite enhanced sand mixture (BES). The properties of BES are given in Table 4.1.

Table 4.1 Properties of BES and BSMC used for the study

Property	BES	BSMC
Specific gravity	2.67	2.67
Liquid limit (%)	60.8	69.7
Plastic limit (%)	23.8	31.5
Plasticity index (%)	37	38.2
Shrinkage limit (%)	16	15.39
Grain Size Distribution		
Sand (%)	80	37
Silt (%)	1	29
Clay (%)	19	34

4.2.3 Locally available materials

In addition to the requirements of clay liners already stated, the selection of liner material will usually be governed by the availability of materials, either at site or in nearby areas. An assessment of costs will generally indicate whether a liner system should incorporate natural soils, which are available within a reasonable distance of the site, or whether synthetic materials are needed. India has a very long coastal line and hence marine clay is abundantly available. It is in this context locally available material—marine clay was considered. The samples for the studies were collected as given in Chapter 3.

4.2.3.1 Sun dried marine clay

Marine clays, formed by the sedimentation of clayey soils in marine environment, exhibit many unusual physical properties. They possess high liquid limit and their natural moisture content is close to the liquid limit values. The most striking feature of the physical properties is the phenomenal changes that are caused by drying of the soil. So, a preliminary study on the effect of drying on the physical properties of marine clay has been presented. Table 4.2 presents the physical properties of marine clay samples in two different initial conditions viz., moist and sun dried.

Table 4.2 Variations in properties of marine clay on drying

Property	Initial conditions	
	Moist	Sun dried
Liquid limit (%)	102	50
Plastic limit (%)	41	27
Plasticity index (%)	61	23
Shrinkage limit (%)	20.4	15.8
Grain Size Distribution		
Sand (%)	17.5	42
Silt (%)	41	28
Clay (%)	41.5	30

From the data given in Table 4.2, it can be seen that sun drying significantly reduces the Atterberg limits. According to Jagdish Narain and Iyer (1967) there was significant reduction in the liquid limit of Kuttanad clays on air drying. Jose et al. (1989) has reported the reduction in these properties for samples collected from different locations of Cochin area. The variations in their Atterberg limits are due to aggregation of finer particles to coarser ones, during drying the clay content reduces and this in turn brings down the Atterberg limits.

The drastic reduction in plasticity index to 23%, leads to the possibility of marine clay for use as a liner material. The studies of earlier workers on liner material, Bagchi et al. (1989) recommend a plasticity index 10-30%. A requirement to satisfy EPA criteria of hydraulic conductivity of soils is that the plasticity index should be greater than 10%, and the soils should have atleast 20% fines (silt + clay). Hence all these requirements for liners are satisfied by the sun dried marine clay. In this study sun dried marine clay (SMC) is taken as one of the liner soil.

4.2.3.2 Sun dried marine clay as an additive to bentonite

Earlier studies on marine clay conducted by Solly et al. (2004) have shown that the sun dried marine clay can perform equally or even better than sand as an additive to bentonite. Their studies have concluded that 80% sun dried marine clay is a suitable additive to bentonite. Hence, 80% sun dried marine clay added to 20% bentonite was also selected for the investigations in connection with development of desiccation cracks. The properties or bentonite amended with sun dried marine clay (BSMC) are presented in Table 4.1.

4.2.4 Kaolinite soil

Though kaolinite soil is not a liner material, it has been included for comparative purposes in the study related to behaviour of soil when subjected to alternate wetting and drying. The properties of Kaolinite soil are given in table 3.1.

4.3 COMPACTION STUDIES ON LINER SOILS

The purpose of compaction is to reduce the hydraulic conductivity of the soil liner. The quality control procedure for the installation of clay liners usually focuses on the energy delivered to the soil, water content at compaction, and the dry unit weight. Water content at compaction is an important parameter which can influence the hydraulic properties of the soil liner materials. Soils compacted at water contents dry of optimum tend to have a relatively high hydraulic conductivity whereas soils compacted at a water content wet of optimum tend to have a lower hydraulic conductivity (Bagchi, 1994). A typical construction specification requires that the soil be compacted over a specified range of water content not to exceed the optimum water content by 4%, and a minimum dry unit weight of 95% of the maximum dry unit weight from standard proctor compaction (Bagchi, 1994).

Standard Proctor compaction tests were conducted on the four selected soils—bentonite enhanced sand mixture (BES), sun dried marine clay (SMC), bentonite with 80% sun dried marine clay (BSMC) and Kaolinite soil. The compaction characteristics and the compaction curves of the study soils are given in Table 4.3 and Fig.4.1.

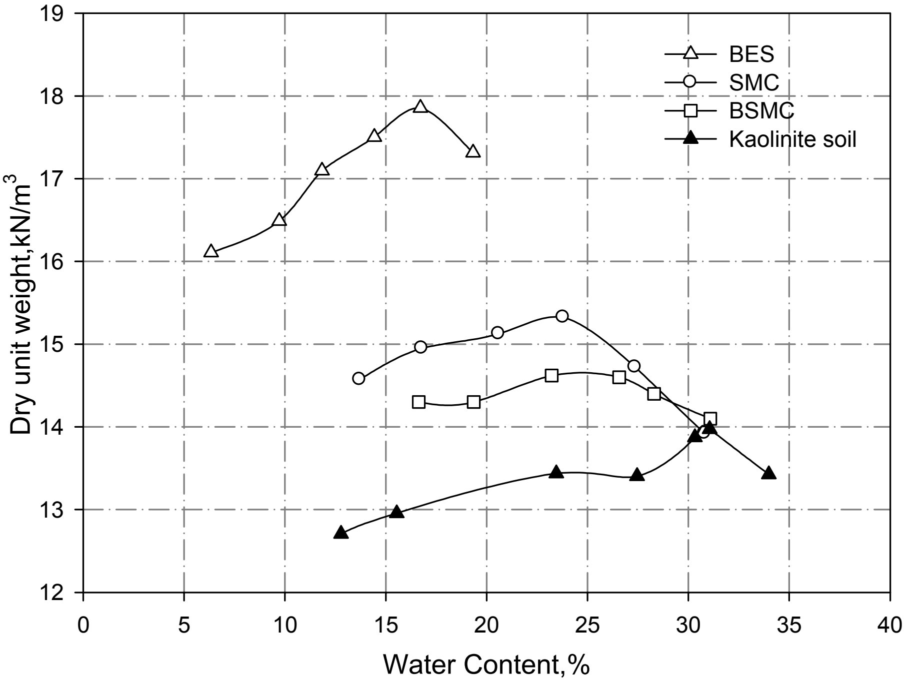


Fig 4.1 Compaction curves of the liner soils used for the study

Table 4.3 Compaction characteristics of liner soil used in the Study

Particulars	Soils			
	BES	SMC	BSMC	Kaolinite soil
OMC (%)	16.7	23.8	26.6	30.7
Maximum dry unit weight (kN/m^3)	17.58	15.3	14.6	14.0

4.4 DESICCATION TESTS

4.4.1 Introduction

Desiccation tests were carried out in large moulds of size 30 cm in diameter and 10 cm in height. The details of the experimental set up, the compaction procedure adopted etc. are explained in detail in section 3.3.5.1. As outlined in section 3.3.5.2, the fluctuations in the degree of saturation of the clay liner during its life time were simulated in this study by wet/dry cycles. The liner soils were compacted at 3% wet of optimum and 95% of maximum dry density as per the construction specifications. The compacted soil was then kept for drying in the drier for 5 days, which marks the end of compaction-drying cycle. This was followed by wet/dry cycle, which is started by ponding the sample removed from the drier, with water. A collar of 5 cm height was used for this purpose. The duration of the wet cycle was 7 days and that of the dry cycle was also 7 days. The determination of the wet/dry cycles, duration and number of cycles are explained in detail in section 3.3.5.2. Thus the first wet/dry cycle is completed at the end of 19 days after compaction. The total test duration for the second and third wet/dry cycles was 35 days and 49 days respectively.

4.4.2 Behaviour of the liner soil after compaction-drying cycle and wetting period

Photographs were taken at the end of compaction-dry cycle for all the four soil samples. As could be observed, except for BSMC, there were no cracks developed in the other samples. Hair line cracks could be seen in the BSMC sample. After the compaction dry cycle, the samples were ponded with water. The samples were fully saturated at the end of the wetting period.

4.4.3 Development of desiccation cracks

At the end of first wet/dry cycle desiccation cracks developed in all the soil samples except the Kaolinite soil. The surficial area was used to determine the crack intensity factor (CIF) which is defined as the percentage of the cracked area to the total surface area of the sample.

4.4.3.1 Desiccation cracks in BES

A combination of bentonite with 80% sand is currently considered to be ideal liner material having hydraulic conductivity and volumetric shrinkage within acceptable limits. The behaviour of this ideal material when subjected to alternate wetting and drying was astonishing and shocking. Fig.4.2 shows the desiccation cracks that had developed in the sample. The sample which was intact at the end of compaction-drying cycle, swelled when subjected to wetting. Since the sample was fully saturated the height to which swelling had occurred could not be measured. During the drying period, shrinking of BES has taken place, leading to the formation of desiccation cracks. The photograph taken at the end of the first wet/dry cycle shown in Fig.4.2 was analyzed using MATLAB, for determination of the CIF and length of cracks.

Fig: 4.2 Desiccation cracks in BES sample (Cycle 1)

4.4.3.1.1 Quantification of cracks by digital image processing

The image acquisition was done by using a digital camera SONY DSC W35, 7.2 mega pixels. Colour image of the desiccated sample is taken. The details regarding image acquisition is given in Chapter 3.

Determination of crack intensity factor

The steps involved in the determination of crack intensity factor is shown as a sequence in Fig.4.3(a-d). The colour image of the desiccated sample was converted to monochrome using Matlab function[Fig.4.3(b)].

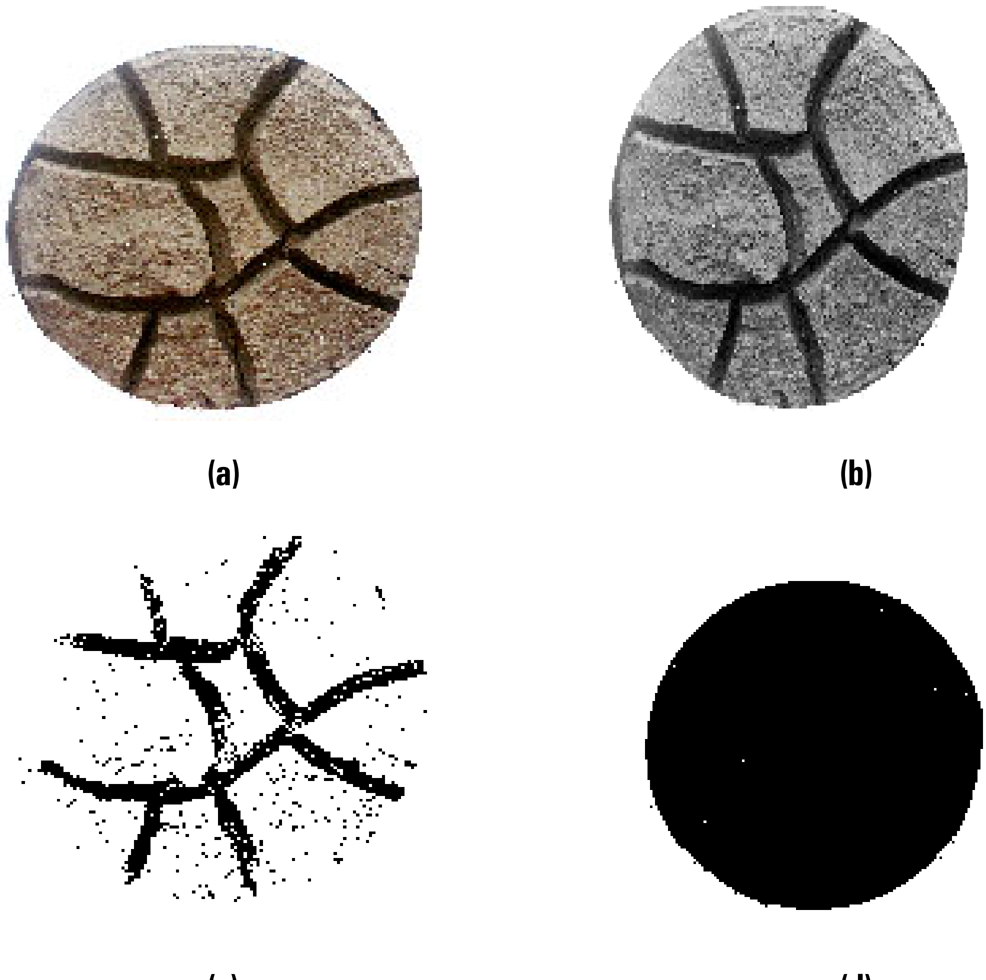


Fig: 4.3 Sequence of analysis carried out to find crack intensity factor

Cracks appear darker than remaining uncracked soil surface in photographs of a drying soil. The contrast in grey level between cracks and uncracked soil was sufficiently high, so that the obtained digital images could be segmented into cracks and uncracked soil using a simple grey threshold. If the input image is f(x,y) and the thresholded image is g(x,y), the equation of the thresholding operator for the image under consideration is given by

$$g(x,y) = 0 \text{ if } 1 \leq f(x,y) \leq 62$$

$$g(x,y) = 255 \text{ if } f(x,y) > 62$$

The number of pixels, having the range of intensity values given by the thresholding operator, was found out using the program. Thus the cracks were extracted from the image. [Fig.4.3(c)]. From the image surface area of the sample was extracted [Fig.4.3(d)]. Crack intensity factor, which is the area of cracks to the total area of the sample is obtained as (c/d). For the image under consideration the CIF obtained is 18.

Determination of length of crack

The number of crack lines in each sample was determined by an interactive Matlab function. The total length of crack is obtained in pixels, from the program. From the known diameter of the sample and the number of pixels, it corresponds to, the total length of crack in a desiccated sample is found out. This is shown in Fig.4.4. Digital image studies were considered for the quantification of cracks in soil after reviewing the methodologies explained in section 3.3.5.2.

Fig: 4.4 Determination of length of crack

4.4.3.1.2 Effect of number of wet/dry cycles on BES

The same BES sample was subjected to 3 wet/dry cycles. The photographs taken at the end of the second wet/dry cycle (Cycle 2) and at the end of third wet/dry cycle (Cycle 3) are presented as Fig.4.5. A visual interpretation of the Fig.4.5(a) shows that at the end of cycle 2, the number of surficial cracks have increased and the number of cracked cells which was 9 in cycle 1 (Fig.4.2) has increased to numerous cells. The depth of the crack was measured using thin wire gauges and width by vernier calipers. A visual comparison of Fig.4.5(a) and (b) shows that the surficial cracks have reduced in Cycle 3. The data obtained from the analysis of the photographs of all the cycles of desiccated samples of BES is presented in Table 4.4.

(a)

(b)

Fig: 4.5 Desiccated Samples of BES

a) Cycle 2 b) Cycle 3

Table 4.4 Desiccation crack details of BES for various cycles

Cycle No.	Length of crack (cm)	Maximum Depth of crack (cm)	Crack intensity factor (%)
1	106.58	4.2	18.09
2	243.57	5.0	39.75
3	230.83	5.1	21.22

It can be understood from Table 4.4 that the crack intensity factor (CIF) and length of crack increases to a maximum value in cycle 2 and then decreases.

4.4.3.2 Desiccation cracks in sun dried marine clay

Figure 4.6 shows the photograph of desiccated sample of sun dried marine clay for cycle 1. The desiccation cracks varied from hair line ones to that having a maximum widths of 1.4 cm. Comparison of the Figs.4.2 and 4.6 itself shows that, when subjected to alternate wetting and drying, the amount of surficial cracking in SMC is considerably less than BES, which points to the possible use of sun dried marine clay as an alternate liner material which is locally available.

Fig: 4.6 Desiccated Sample of Sundried Marine Clay (Cycle 1)

The photographs of the SMC sample when subjected to the next cycles of wetting/drying are given in Fig.4.7. The details obtained from the analysis of the photographs and the other crack details are shown in Table 4.5.

(a) (b)

Fig: 4.7 Desiccated samples of SMC (Cycle 2 & Cycle 3)

Table 4.5 Desiccation crack details of sun dried marine clay

Cycle No.	Crack details			Crack intensity factor (%)
	Length (cm)	Maximum Width (mm)	Maximum Depth (cm)	
1	133	3.6	0.7	9.83
2	177	4.5	1.7	7.52
3	114	4.58	2.0	4.58

4.4.3.3 Development of desiccation cracks for different soils

Samples of bentonite with 80% sun dried marine clay (BSMC) and kaolinite soil were subjected to two cycles of wetting and drying. The photographs taken after the end of each cycle were analyzed to find out the crack intensity factor and length of the crack, Table 4.5 a gives the details of CIF and length of crack.

Table 4.5 (a) Desiccation crack details of BSMC and Kaolinite soil

Type of soil	Cycle No.	Length of crack (cm)	Crack Intensity factor (%)
BSMC	1	88.0	3.39
	2	83.99	7.43
Kaolinite	1	0	0
	2	0	0

Swelling of the soil sample was noticed for BSMC during wetting, while kaolinite soil did not exhibit any swelling. The kaolinite soil shrunk as a whole mass and no desiccation cracking was observed. The photographs of the desiccated samples of the four liner soils for cycle 1 and cycle 2 are presented in Fig.4.8 and Fig.4.9 respectively. A comparison of the crack intensity factor and length of crack for the four soils is presented in Table 4.6.

Table 4.6 Desiccation crack details of the liner soils for cycle 2

Type of soil	Length of crack (cm)	Maximum Depth of crack (cm)	Crack intensity factor (%)
BES	243.57	5.1	54.62
SMC	177	1.7	7.52
BSMC	83.99	3.2	7.43
Kaolinite	0	0	0

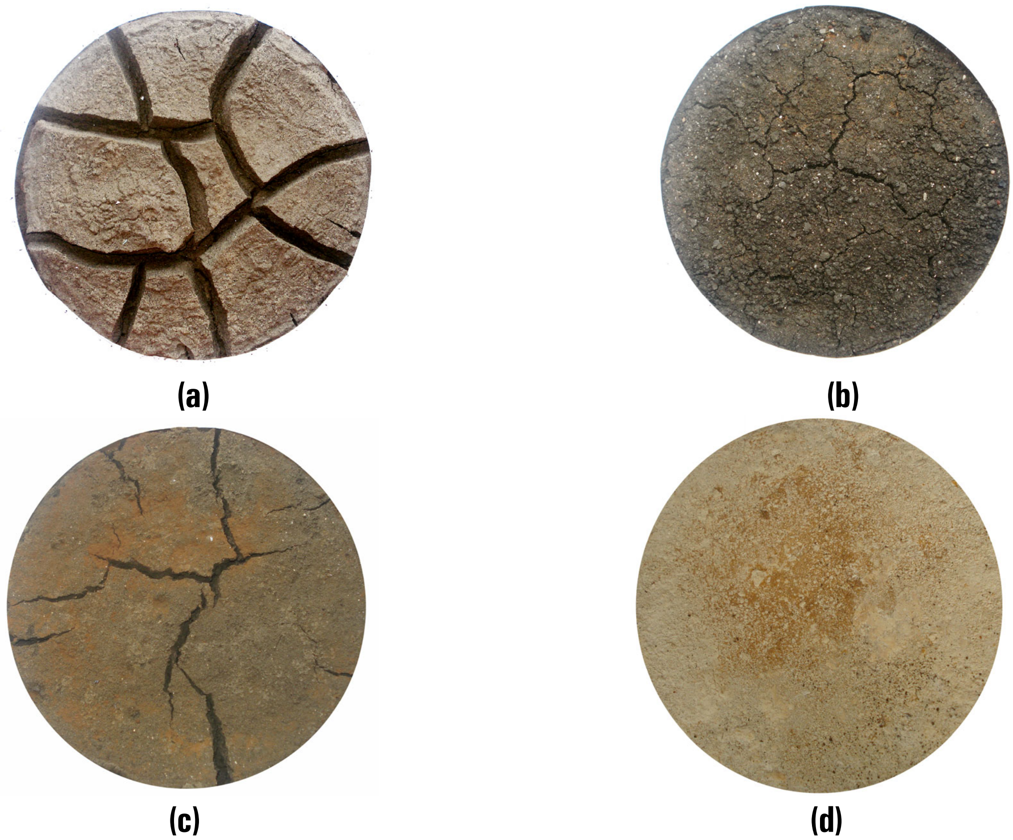


Fig: 4.8 Desiccated samples of liner soils for Cycle 1

a) BES b) SMC
c) BSMC d) Kaolinite

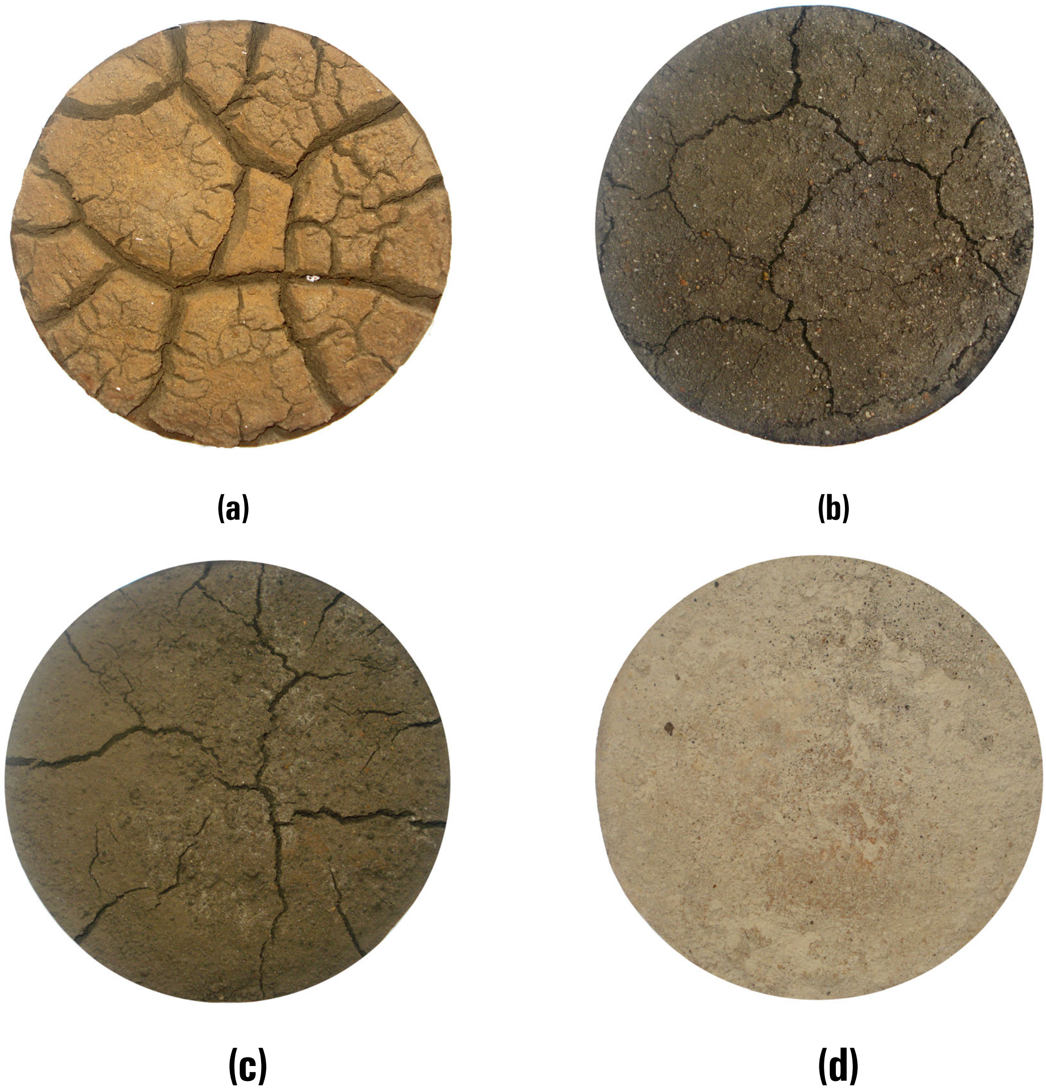


Fig: 4.9 Desiccated samples of liner soils Cycle 2

a) BES b) SMC

c) BSMC d) Kaolinite

It is clear from the Figs.4.8 and 4.9 and from the table that the ideal liner material BES has failed miserably. The liner materials locally available SMC and bentonite blended with SMC proved to be superior when subjected to alternate cycles on wetting and drying. In the case of SMC and BSMC, the crack intensity factors for cycle 2 are nearly similar as indicated in Fig.4.10. An observation into the crack details show that the length of the crack of SMC, though spread along 177 cm is penetrated to a maximum depth of 1.7 cm, while the crack in desiccated samples of BSMC has penetrated to a maximum depth of 3.2 cm. As already pointed out BSMC swelled during the wetting process. Another aspect is that when locally available material, SMC is available in plenty it is not necessary to go in for bentonite amended sun dried marine clay.

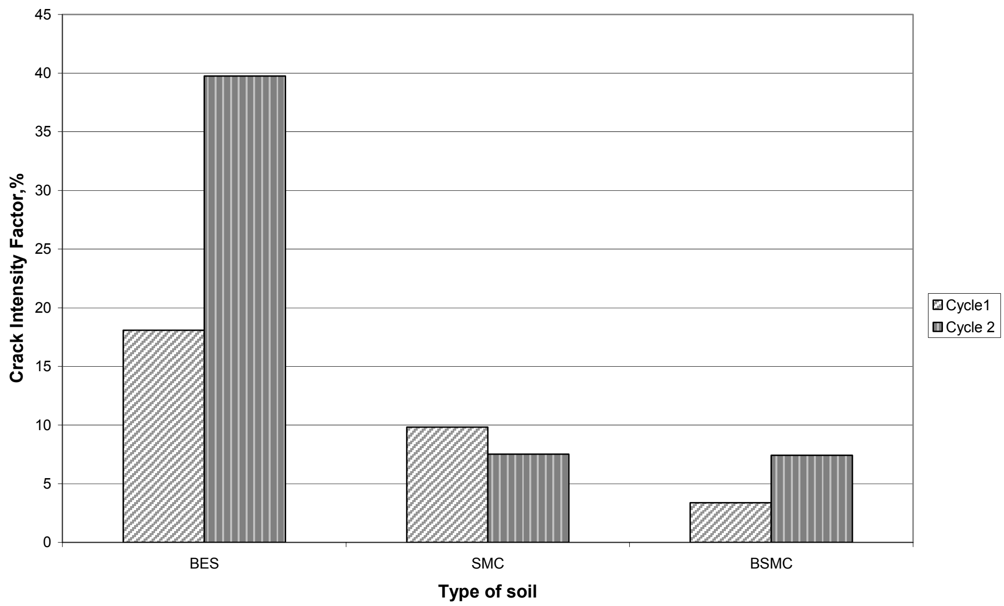


Fig: 4.10 Variation Of Crack Intensity Factor For Different Soils

4.5 SUPERIORITY OF SUN DRIED MARINE CLAY OVER BENTONITE ENHANCED SAND MIXTURE

Notwithstanding the fact that a combination of the 80% sand and 20% of bentunite is the currently accepted and most popular liner material for engineered land fills, whose importance is steadily growing owing to the rapid urbanization, the series of experimental results presented in this chapter has clearly brought out that there are some serious and glaring limitations for BES as liner material. Fig 4.2 itself is a shocking revelation of this fact.

The introduction of the crack intensity factor and its possibility of an accurate assessment of crack configuration and CIF with the new technique of digital image processing, has now facilitated a comparative study of BES with other possible liner materials.

Fig: 4.11 and 4.12 presents a comparison of the desiccated samples of BES and SMC after first, second and third cycles of wetting and drying.

(a) (b)

Fig: 4.11 Comparisons of Desiccated Samples (Cycle 1)

a) BES (b) SMC

The enormity of the susceptibility acceptability of BES compared to the newly proposed sun dried marine clay in comparison is clearly brought out emphatically. The superiority of SMC over BES is too obvious, from the figures.

(a) (b)

(c) (d)

Fig: 4.12 Comparisons of designated samples of BES and SMC

a) BES Cycle 2 b) SMC Cycle 3

c) BES Cycle 2 d) SMC Cycle 3

Fig: 4.12 shows the crack patterns for the two samples after the second and third cycle of wetting and drying.

Table 4.7 Details of Desiccation cracks of BES and SMC

Cycle No	BES		SMC	
	Total length of crack (cm)	Maximum depth of cracks(cm)	Total length of crack (cm)	Maximum depth of cracks(cm)
1	106.58	4.2	133	0.7
2	243.57	5.0	177	1.7
3	230.83	5.1	114	2.0

Table 4.7 provides a comparative study of the crack details of BES and SMC. After the first cycle, the total length of crack 106.58cm for BES and 133cm for SMC. But after the second cycle, the crack is length in BES increases to 243.57cm an increase by 128%. In SMC this increase is only 33%. For the third cycle the increase is 116% for BES. But SMC registers a reduction in total crack

length from 177cm to 114cm which is 14% less than the original total length of 133cm.

The threat from desiccation cracks emanate from the fact that the crack openings reduces effective thickness of the clay liner and facilitates percolation of leachate. Thus the clay liner becomes more vulnerable.

Here again BES suffers in comparison. While the crack developed in BES had a maximum depth of 5.1cm, that in SMC was only 2 cm – about 40%.

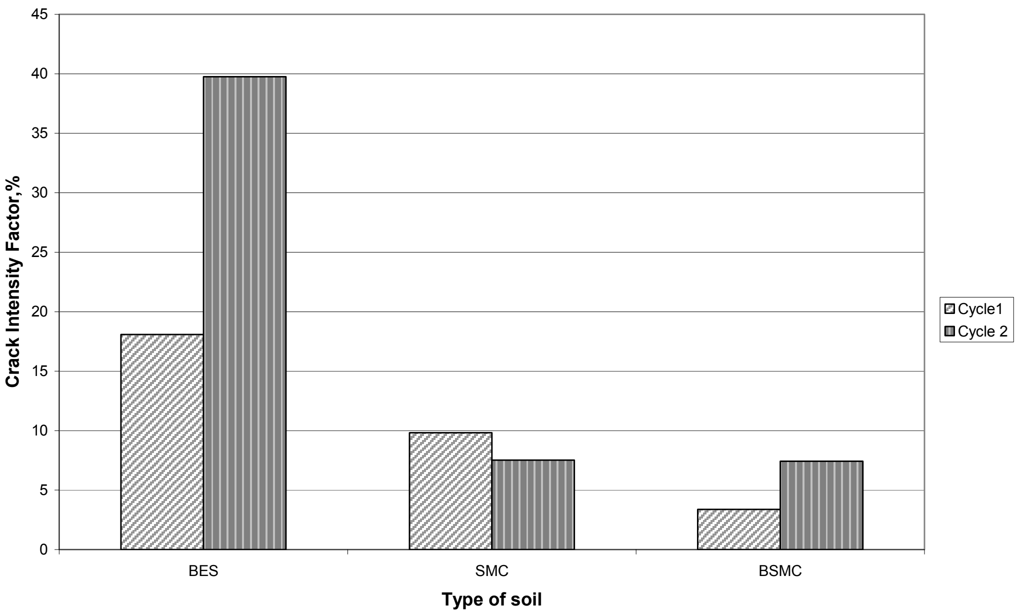


Fig 4.13 Variation of CIF for desiccated samples of BES and SMC for different cycle

Fig: 4.13 shows that the crack intensity factors of BES and SMC for the three cycles of wetting and drying.

Table 4.8 Crack intensity factor for BES and SMC

Cycle No	Crack Intensity Factor (CIL)		Ratio of CIFs
	BES	SMC	
1	18.09	9.83	1.84
2	39.75	7.52	5.29
3	21.22	4.58	4.63

Table 4.8 gives the crack intensity factor for BES and SMC for three cycles. CIF increaseS form 18.09 to at 39.75 cycle 2 and then reduces to 21.22 for cycle 3.

In case of SMC, the CIF decreases steadily with each cycle of wetting and drying – from 9.83 for the first cycle to 4.58 for the third cycle. The wide variation observed in the crack patterns in fig: 4.11 and 4.12 is quantitatively brought out by the ratio of crack intensity factor of BES to CIF of SMC shown in table 4.8. The values are 1.84, 5.29 and 4.63 for first, second and third cycle respectively.

These ratios provide a good comparison between the present liner material of Bentonite enhanced clay and sun drained marine clay and the superiority of the latter cannot be emphasized.

Chapter 5

EFFECTS OF FIBRES AND ADMIXTURE ON THE DESICCATION CRACKS IN CLAY LINER SOIL

5.1 INTRODUCTION

The studies conducted on typical clay liner materials viz., bentonite enhanced sand mixture and locally available marine clay as an alternative material, point to the fact that desiccation cracking is a problem encountered in compacted clay liners, when they are subjected to alternate wetting and drying, which are inevitable due to seasonal variations. With regard to the long term performance of the clay liners, the desiccation risk of compacted clay liners is of cardinal relevance as desiccation will induce cracks in the liner and will reduce the sealing effect of the cover system significantly. A variety of research efforts have attempted to address the problem of desiccation cracks. Some have considered the use of surface moisture barriers above the soil layer while a few others have considered the use of fibres and soil additives like lime, sand and cement. However attempts to quantify the distribution and depth of cracks and to control the crack potential by suitable amendments have been limited. The investigations presented in this chapter focus on the use of random discrete fibres to reduce the crack intensity factor of the liner materials. The influence of the amendments used in the study, on the properties of these liner materials is also discussed.

5.2 RANDOMLY DISTRIBUTED FIBRE REINFORCEMENT

In randomly distributed fibre reinforcement, discrete fibres are added and mixed with soil. The main advantage of randomly distributed fibre is the ease in mixing, maintenance of strength isotropy and absence of potential planes of weakness. In this study, three types of fibres have been used, viz., nylon fibre, polypropylene monofilament and polypropylene fibre mesh the details of which have been given in chapter 3. Table 5.1 gives the details of the aspect ratios (l/d) of the fibres used in the study. Fibre length of 25 mm has been used for nylon fibre and polypropylene monofilament, unless otherwise specified.

Table 5.1 Aspect Ratio of the Fibres used

Particulars	Aspect Ratio (l/d)			
Length of fibre	10	15	20	25
Nylon	12.8	19.2	25	32
Polypropylene monofilament	20	30	40	50
polypropylene fibre mesh			200	

5.2.1Compaction characteristics

The water content at placement and the maximum dry density achieved in the field, play a vital role in the hydraulic conductivity and volumetric shrinkage of the clay liner. In this context, it was necessary to investigate the effect of fibre reinforcement on the compaction behaviour of the liner soils. A series of standard Proctor compaction tests were conducted with nylon fibre of length 25mm at fibre contents 0.15%, 0.3%, 0.6%, 1.0% and 1.2% on bentonite enhanced sand mixture (BES) and sun dried marine clay (SMC). The results obtained from the standard Proctor compaction tests for BES and SMC amended with nylon fibres are presented in Table 5.2 and Fig.5.1,respectively

Table 5.2 Compaction characteristics of BES with nylon fibres

Fibre content	Maximum dry density (kN/m^3)	Optimum moisture content (%)
0	17.85	16.8
0.15	17.64	16.72
0.3	17.88	16.9
0.6	18.01	17.63
1.0	18.12	17.72

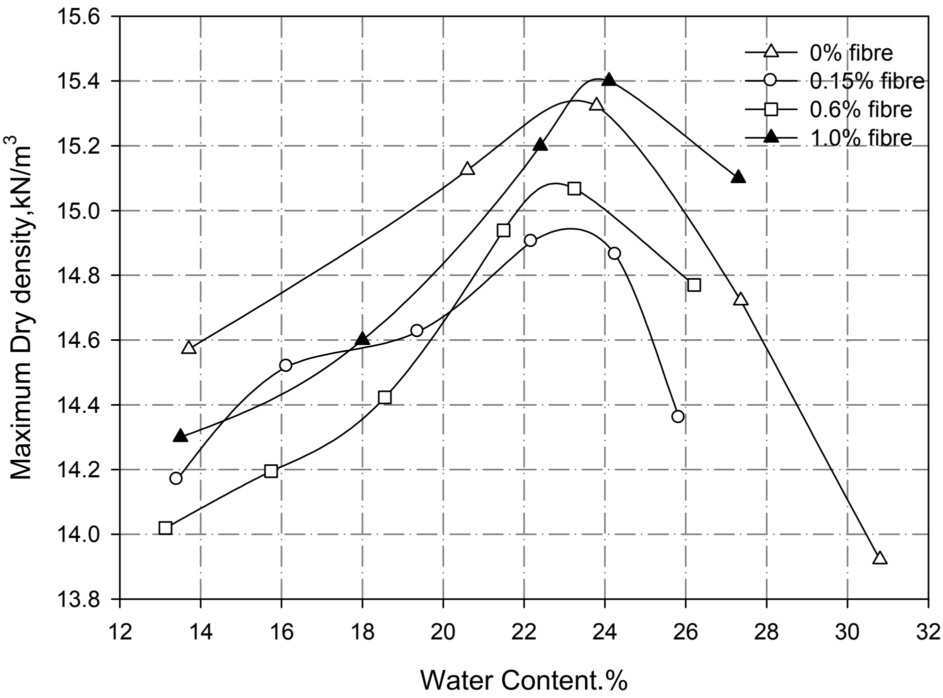


FIG.5.1 Compaction characteristics of sun dried marine clay amended with nylon fibre.

The similarity in the compaction curves, at all fibre contents, indicates that fibrous soil exhibits the same type of compaction behaviour as the corresponding plain soils, both for BES and SMC. Soil amendment with fibre resulted in changes in both the optimum water content and the maximum dry density. It is marginal quite evident from Table 5.2 and Fig.5.1, inclusion of fibre in the soil does not have any significant effect on the maximum dry density as well as on the optimum moisture content. The variation in optimum moisture content (OMC) for the nylon fibre amended soil is between 0.08% below OMC and 0.85% above OMC compared to unamended BES soil. These variations are 0.5% and 5% of the OMC of the unamended soil and hence considered insignificant. The maximum dry density varied between -0.32kN/m^3 to + 0.21kN/m^3 compared to that of unamended BES. These variations are 1.18% and 1.79% from the maximum dry density values of unamended BES mixture. According to Al-Wahab and El-Kedrah (1995), the difference in unit weights upto 5% is insignificant for two reasons: (1) a borrow soil itself could have variations in the maximum unit weight as high as 5% due to variations in soil type, moisture, compactive effort and field measurements of unit weight, and (2) soils are generally compacted on the wet side by atleast 2% above the optimum moisture content for different reasons, in which case the maximum difference in unit weights between the fibrous and the plain soil is less than 2%. Therefore the optimum moisture content of 16.8% and maximum dry unit weight of 17.85 kN/m3 of unamended BES mixture have been taken as the compaction characteristics of nylon amended BES mixture.

The optimum moisture content of the sun dried marine clay amended with nylon fibre varied between 0.6% below and 0.3% above OMC of unamended sun dried marine clay (SMC), which is 1.26% and 2.5% of that of SMC. Similarly, the variation in maximum dry unit weight of nylon fibre amended SMC is between 0.2kN/m^3 to 0.1kN/m^3, which again is less than 1.5% of that of unamended SMC. Hence based on the arguments stated earlier, these variations are insignificant in all further studies. There for SMC amend with nylon fibre, the optimum moisture content and maximum dry unit weight have been taken as 23.8% and 15.3kN/m3 respectively which pertain to that of unamended SMC.

A comparison between the results of the compaction characteristics of bentonite enhanced sand mixture and sun dried marine clay, points to the fact that the inclusion of randomly distributed fibre in the soil did not have any significant change in the compaction behaviour.Similar results have been reported from compaction studies in the literature (Maher and Ho, 1994; Al-Wahab and El-Kedrah, 1995; Miller C.J. and Rifai S, 2004). Their results were obtained from tests performed on soils that have different plasticity indices, different fibre, fibre lengths and different compactive efforts. Based on these findings, for bentonite enhanced sand mixture and sun dried marine clay amended with nylon fibre and the findings from literature, it was concluded that there was no need to study the changes in compaction behaviour of bentonite enhanced sand mixture and sun dried marine clay amended with polypropylene monofilament fibre and polypropylene fibre mesh. It is expected that the soils amended with the polypropylene monofilament fibre and polypropylene fibre mesh will exhibit similar type of compaction behaviour as that of the corresponding unamended soils.

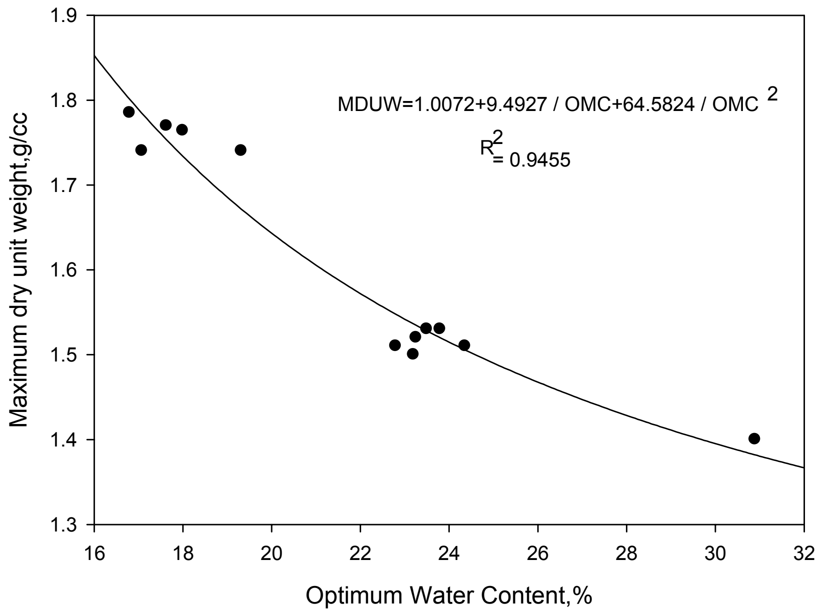


Fig 5.2 Correlation of maximum dry unit weight and optimum moisture content of the amended and unamended soil used in the study

The data obtained from the proctor compaction test results of the amended and unamended soil samples used in the study, was correlated between maximum dry unit weight (MDUW) and optimum moisture content. Good correlation exists between the maximum dry unit weight and OMC for all soils. Figure 5.2 shows the relationship between maximum dry unit weight and OMC for various soils including amended and unamended soil samples used in the study.

5.2.2 Hydraulic conductivity

A prime criterion for the acceptability of any soil—unamended or amended—as laid down by EPA is that the hydraulic conductivity should be less than $1x10^{-9}$ m/sec. Hence it is essential to verify and ensure that the amended soils do not cross the prescribed limit. The order of hydraulic conductivity is an essential characteristic to judge its acceptability for containment structures such as landfill covers and bottom liners. Therefore, the effect of fibre inclusion on the hydraulic conductivity of the soils selected for the study is of primary concern and has to be evaluated. The hydraulic conductivity tests were carried out at different consolidation pressures, as explained in chapter 3. The unamended and amended soil samples for the tests were prepared under standard compactive effort at 3% wet of optimum moisture content.

5.2.2.1 Effect of fibre inclusion on the hydraulic conductivity of bentonite enhanced sand mixture

The hydraulic conductivity tests were conducted on unamended BES and BES amended with nylon, polypropylene fibre and monofilament fibre. The fibre contents of 0.15%, 0.3%, 0.6%, 1.0% and 1.2% were used in the study for BES amended with nylon and polypropylene monofilaments. Beyond a fibre content of 1.0%, the mixing of the random fibres especially polypropylene monofilament fibres caused clumping or 'balling' of fibres, and it was extremely difficult to get a uniform fibrous soil. Polypropylene fibre mesh could not be used beyond 0.8%, because of the 'balling' of fibres and difficulty encountered while mixing with higher percentage of fibres. Thus, the fibre contents used for polypropylene fibre mesh were limited to 0.15%, 0.3%, 0.6% and 0.8%.

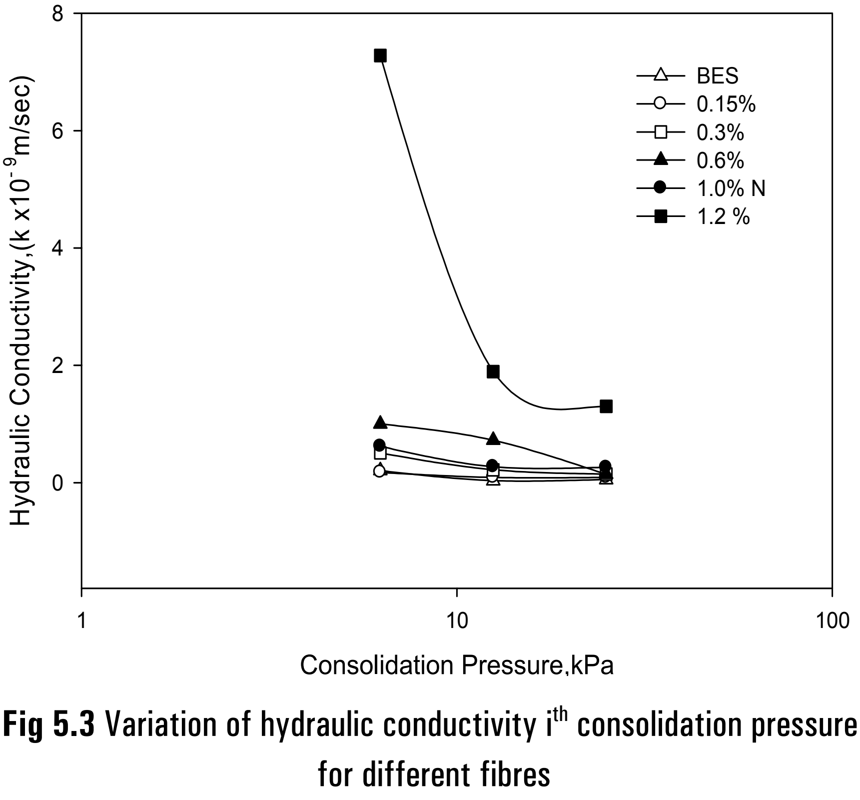


Fig 5.3 Variation of hydraulic conductivity i^th consolidation pressure for different fibres

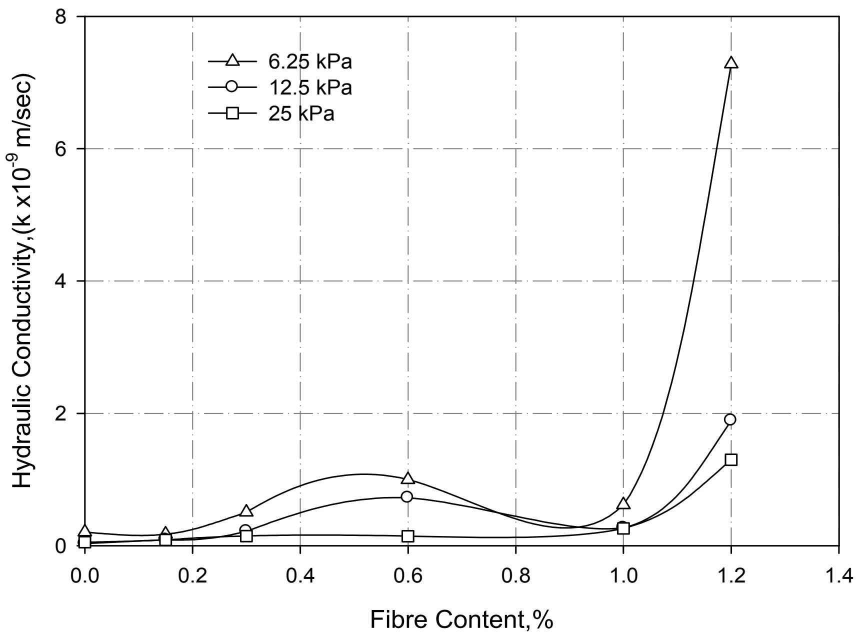


Fig 5.4 Hydraulic Conductivity of BES amended with Nylon fibres at different consolidation pressure

The variation of hydraulic conductivity of BES amended with nylon fibre for different consolidation pressures is given in Fig.5.3. It can be seen from the figure that even at a low vertical stress of 6.25kPa, the hydraulic conductivity of samples of BES amended with nylon fibres are within the accepted limit of $1x10^{-9}$m/s for all fibre contents except 1.2%. It is also to be noted that there is slight decrease in hydraulic conductivity as the fibre content is increased from 0% to 0.15%. As the consolidation pressure is increased, the same trend is maintained, and as expected the hydraulic conductivity decreased with increase in consolidation pressure. The experiments were conducted till 25kPa. The magnitude of hydraulic conductivity increased with increase in fibre content as shown in Fig 5.4. The results obtained from hydraulic conductivity tests for nylon fibre amended BES is tabulated in Table 5.3. The results indicate that the inclusion of nylon fibre upto 1.0% in BES, would satisfy the EPA criteria for hydraulic conductivity (ie. $1x10^{-9}$m/sec.).

Table 5.3 Hydraulic conductivity for BES amended with nylon fibres

Fibre content	Hydraulic conductivity (m/sec.)		
	Consolidation Pressure (kPa)		
	6.25 kPa	12.5 kPa	25 kPA
0	$2.06x10^{-10}$	$3.3x10^{-11}$	$5.24x10^{-11}$
0.15	$1.75x10^{-10}$	$8.85x10^{-11}$	$8.8x10^{-11}$
0.3	$5.06x10^{-10}$	$2.19x10^{-10}$	$1.45x10^{-10}$
0.6	$1.0x10^{-9}$	$7.23x10^{-10}$	$1.46x10^{-10}$
1.0	$6.19x10^{-10}$	$2.7x10^{-10}$	$2.6x10^{-10}$
1.2	$7.28x10^{-9}$	$1.89x10^{-9}$	$1.3x10^{-9}$

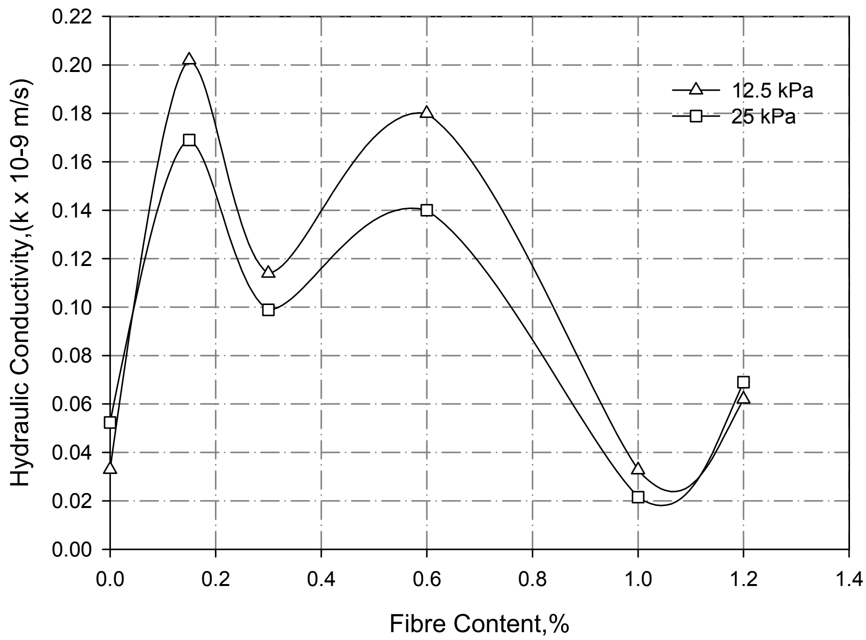


Fig 5.5 Variation of Hydraulic conductivity BES amended with polypropylene monofilament fibre for different consolidation pressure

The hydraulic conductivity test results of polypropylene monofilament fibre amended BES is given in Table 5.4 and Fig.5.5.

Table 5.4 Hydraulic conductivity for BES amendedwith polypropylene monofilament fibre

Fibre content	Hydraulic conductivity (m/sec.) Consolidation Pressure (kPa)	
	12.5 kPa	25 kPA
0	3.3×10^{-11}	5.24×10^{-11}
0.15	2.02×10^{-10}	1.69×10^{-10}
0.3	1.14×10^{-10}	9.89×10^{-11}
0.6	1.8×10^{-10}	1.4×10^{-10}
1.0	3.28×10^{-11}	2.16×10^{-11}
1.2	6.2×10^{-11}	6.9×10^{-11}

It is evident from the test results, that polypropylene monofilament fibre inclusion, even up to 1.2% in BES, gave hydraulic conductivity values well within the permissible limits of 1×10^{-9}m/s. In the case of polypropylene monofilament fibre, an irregular trend is noted. As the fibre content is increased from 0 to 0.15%, there is an increase in hydraulic conductivity, but from 0.15% to 0.3%, there is a decrease in hydraulic conductivity.

Bentonite enhanced sand mixture was amended with polypropylene fibre mesh at fibre contents of 0.3%, 0.6% and 0.8% and hydraulic conductivity tests were performed. Table 5.5 shows the test results obtained at 12.5 kPa and 25 kPa. It could be concluded from the test data, that a fibre content of 0.6%, BES amended with polypropylene fibre mesh could be used as a liner material with respect to the hydraulic conductivity criteria.

Table 5.5 Hydraulic conductivity of BES amended with polypropylene fibre mesh

Fibre content (%)	Hydraulic conductivity (m/sec.) Consolidation Pressure (kPa)	
	12.5	25
0	3.3×10^{-11}	5.24×10^{-11}
0.3	2.4×10^{-10}	8.56×10^{-10}
0.6	1.5×10^{-9}	9.2×10^{-11}
0.8	3.78×10^{-9}	1.29×10^{-9}

5.2.2.2 Effect of fibre inclusion on the hydraulic conductivity of sun dried marine clay

Based upon the hydraulic conductivity test results obtained for BES amended with fibres, and taking into consideration the test durations involved and time limitations, it was decided to limit the hydraulic conductivity tests for two fibre contents- 0.8% and 1.2%. The lower fibre contents were ignored for the test, since it was definite from the test data on BES, that a hydraulic conductivity within the permissible limits could be possible till a fibre content of 0.6%. Table 5.6 gives the hydraulic conductivity test results of sun dried marine clay amended with nylon and polypropylene monofilament fibres.

It is quite evident from Table 5.6 that the addition of fibres to sun dried marine clay even upto 1.2%, does not render the soil unsuitable for liner application. The addition of polypropylene fibre mesh to SMC also follows the same trend. The hydraulic conductivity criteria for the landfill liner soils is satisfied upto 0.8% of polypropylene fibre mesh addition to SMC. The results of hydraulic conductivity tests performed with polypropylene fibre mesh amended SMC is tabulated in Table 5.7.

Table 5.6 Hydraulic conductivity of SMC amended with nylon and polypropylene fibres

Type of fibre	fibre content (%)	Hydraulic conductivity (m/sec.) Consolidation Pressure (kPa)	
		12.5	25
Nylon	0	$8x10^{-10}$	$3.8x10^{-10}$
	0.8	$2.7x10^{-10}$	$1.6x10^{-10}$
	1.2	$1.5x10^{-10}$	$9.2x10^{-11}$
Polypropylene monofilament	0	$8.18x10^{-10}$	$3.8x10^{-10}$
	0.8	$3.128x10^{-10}$	$2.77x10^{-10}$
	1.2	$9.86x10^{-11}$	$9.2x10^{-11}$

Table 5.7 Hydraulic conductivity test results of SMC amended with polypropylene fibre mesh

Fibre content (%)	Hydraulic conductivity (m/sec.) Consolidation Pressure (kPa)		
	6.25 kPa	12.5 kPa	25 kPA
0	$4x10^{-9}$	$8.18x10^{-10}$	$3.85x10^{-10}$
0.3	$1.3x10^{-9}$	$1.06x10^{-10}$	$7.277x10^{-10}$
0.8	$1.65x10^{-9}$	$2.58x10^{-10}$	$1.42x10^{-10}$

A comparison of the performance of the inclusion of fibres in BES and SMC enable us to draw some useful conclusions. The inclusion of all the fibres, selected for the study, in the liner soils, BES and SMC does not make it unsuitable for liner applications. In the case of BES, nylon fibres upto 1.0% and polypropylene monofilament upto 1.2% satisfy the hydraulic conductivity requirements of clay liner soils. fibre contents upto 1.2% could be used with sun dried marine clay. Polypropylene fibre mesh amended BES and SMC satisfy the hydraulic conductivity criteria upto fibre inclusion of 0.6% and 0.8% respectively.

5.2.3 Shrinkage limit of amended soil

Shrinkage limit, which is the lowest water content at which the sample can remain in a saturated state, is an important parameter in the identification of clay water electrolyte systems. The shrinkage limit of soil samples amended with fibres was done similar to that of unamended soil samples.

5.2.3.1 Effect of fibre inclusion on the shrinkage limit of BES mixture.

The soil samples (both amended and unamended) are prepared at 95% maximum dry density and 3% above optimum moisture content. Nylon and polypropylene monofilament fibres of length 25 mm were used in different proportions as indicated in the table below. Table 5.8 gives the shrinkage limit of BES mixture amended with nylon and polypropylene monofilament fibres.

Table 5.8 Shrinkage limit of BES mixture amended with fibres of 25 mm length.

Fibre content (%)	Shrinkage limit (%)	
	Nylon	Polypropylene
0	16	16
0.15	16.6	15
0.3	17.3	16
1.0	16.2	16.2

It was found that the fibre inclusions of nylon and polypropylene monofilament do not have significant influence on the shrinkage limit of BES mixture. Polypropylene fibre mesh was also used in the study. The variations in shrinkage limit of BES mixture when polypropylene fibre mesh is used as the amendment is given in Table 5.9.

Table 5.9 Shrinkage limit of BES mixture amended with polypropylene fibre mesh

Fibre content (%)	Shrinkage limit (%)
0	16.046
0.15	15.34
0.3	19.75

In comparison with the other fibres, polypropylene fibre mesh amended BES mixture gives a higher value of shrinkage limit. The shrinkage limit of BES mixture increased by about 23% when polypropylene fibre mesh is used, while it was only 1% and 7.66% respectively, for polypropylene monofilament fibre and nylon amended BES mixture.

The elevated shrinkage limit of the soil with polypropylene fibre mesh is beneficial to the volumetric shrinkage of the soil during desiccation process. In the field, during desiccation process, such soils reach the shrinkage limit early, and thus reducing the chance of further volume change. When desiccation occurs in the field, the polypropylene fibre mesh amended BES mixture with high shrinkage limit will have less change in volume than the other fibre amended BES mixture and unamended BES.

5.2.3.2 Effect of fibre inclusions on the shrinkage limit of sun dried marine clay.

Shrinkage limit tests were conducted on SMC and SMC amended with nylon fibre, polypropylene monofilament and fibre mesh for different percentages of fibres and the results are tabulated in Table 5.10 and Table 5.11. From the tables, it is quite clear that the inclusion of polypropylene fibre mesh and nylon fibre in SMC improved the shrinkage characteristics of SMC much better than the polypropylene monofilament fibre inclusions.

Table 5.10 Shrinkage limit of SMC amended with fibres of length 25 mm

Fibre content (%)	Shrinkage limit (%)	
	Nylon	Polypropylene
0	15.8	15.8
0.15	16.1	16.3
0.3	17.3	16.3
1.0	15.4	16.4

Table 5.11 Shrinkage characteristics of SMC amended with polypropylene fibre mesh

Fibre content (%)	Shrinkage limit (%)
0	15.77
0.15	13.4
0.6	17.3

5.2.4 Strength characteristics

Compacted soils used for liner and cover systems must have adequate shear strength, for maintaining structural integrity during construction and operation. In this context, unconfined compression tests were conducted on BES and SMC amended with fibres. The tests were performed on specimens prepared at standard Proctor compactive effort and 3% above OMC, for various fibre contents and lengths.

5.2.4.1 Strength characteristics of fibre amended BES

5.2.4.1.1 Effect of fibre inclusions on the unconfined compressive strength of BES

The unconfined compressive strength of samples of BES and BES amended with nylon and polypropylene monofilament fibre of length 25 mm was found out by conducting UCC tests. The fibre contents used in the study were 0.15%, 0.3%, 0.6%, 0.8%, 1% and 1.2%. Figures 5.6 and 5.7 show the stress strain characteristics of BES amended with nylon and polypropylene monofilament fibres respectively.

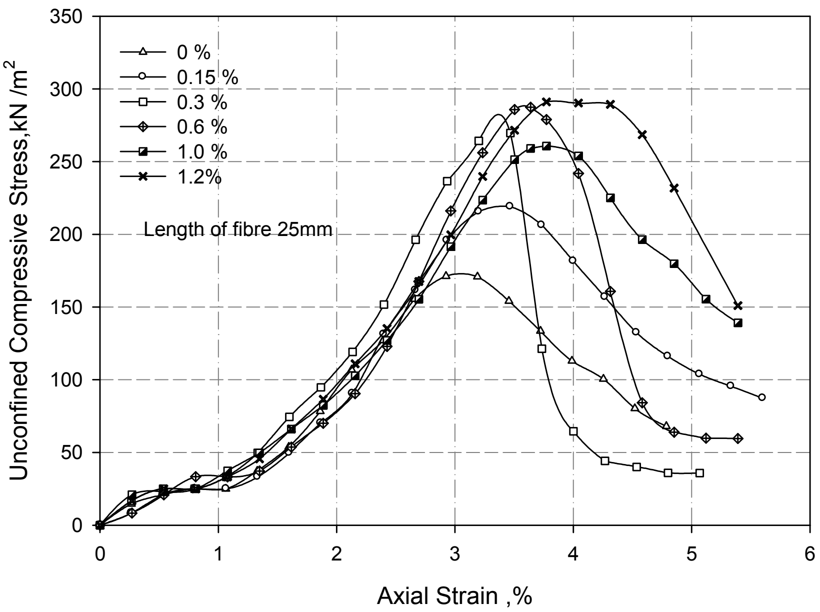


Fig: 5.6 Stress Strain Characteristics of nylon fibre amended Bentonite enhanced sand mixture

Figure 5.6 shows the fibre content vs. unconfined. Strength of nylon amended BES. It can be seen that a fibre content of 0.3% is able to give a strength of 270.5 kN/m2 The increase in strength for a fibre content of 1.2 is only 291 kN/m2 which is marginal compared to the increase in fibre content. The percentage increase in strength for fibre content higher than 0.6% is not significant.

From the figures, it can be observed that the unconfined compressive strength of BES increases with increase in fibre content. After reaching the peak stress, BES amended with nylon and polypropylene fibre shows a sudden and drastic reduction in strength. Another point, to be noted is that though only nominal improvement is observed upto 2% strain, the strain at failure is greater than the unamended soil. Tables 5.12 and 5.13 show the unconfined compressive strength, strain at failure and increase in strength of BES amended with nylon and polypropylene monofilament.

Table 5.12 Variation of unconfined compressive strength of nylon amended BES with fibre content

Fibre content (%)	Unconfined compressive strength (qu)kN/m^2	Strain at failure (%)	Percentage increase in strength (%)
0.3	171.28	2.9	0
0.15	218.99	3.45	27.86
0.3	270.05	3.3	57.67
0.6	287.41	3.6	67.80
0.8	231.09	3.49	34.92
1.0	260.7	3.77	52.21
1.2	291.0	3.77	69.9

Table 5.13 Variation of unconfined compressive strength of polypropylene monofilament fibre amended BES

Fibre content (%)	Unconfined compressive strength (qu)kN/m^2	Strain at failure (%)	Percentage increase in strength (%)
0	171.28	2.9	0
0.15	219.76	3.13	28.3
0.3	238.58	3.74	39.29
0.6	253.606	5.67	48.07
0.8	272.65	4.77	59.18
1.0	254.06	4.01	48.33
1.2	355.94	5.33	107.81

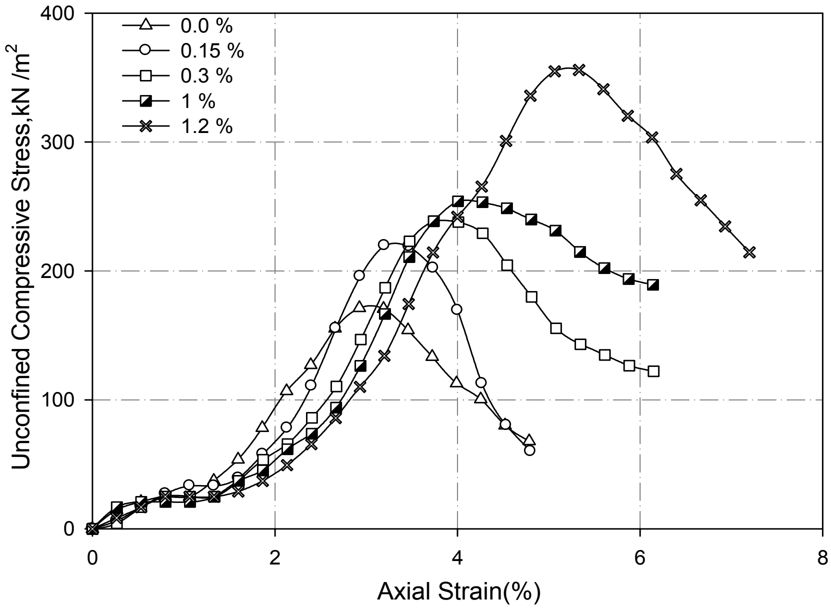


Fig 5.7 Stress strain charactertics of BES amended with polypropylene monofilament fiber

Fig.5.7 shows the stress-strain curves for BES amended with polypropylene monofilament fibre. The behaviour is more or less similar to BES with nylon fibres. The stress decrease very fast after the peak stress. The increase in strength for fibre contents from 0.3 to 1.0% is marginal ie 10%.

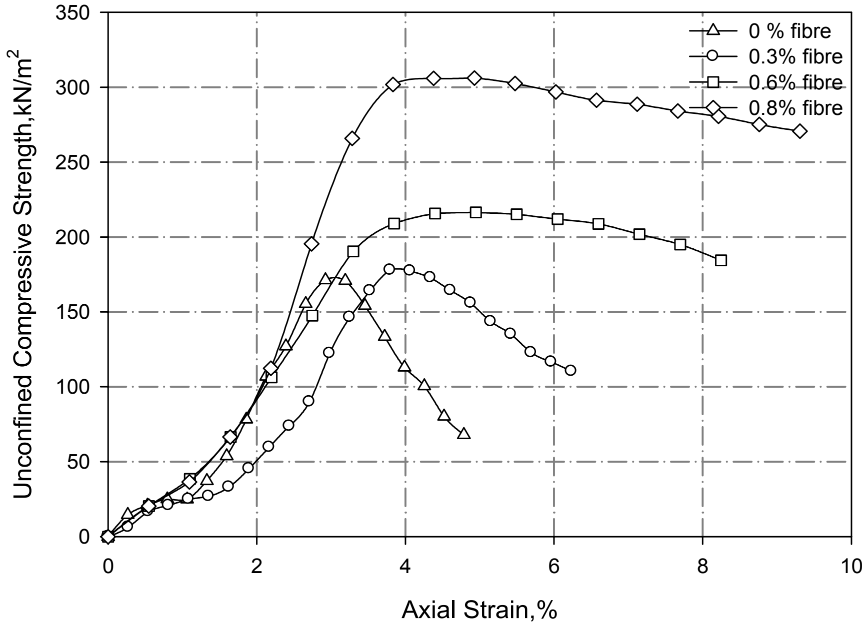


Fig: 5.8 Stress strain charactertics of BES amended with Fibrillated Polypropylene Fiber Mesh

The influence of fibrillated polypropylene fibre mesh on the unconfined compressive strength of BES is depicted in Fig.5.8. As stated in previous section, fibre contents only upto 0.8% have been used for polypropylene fibre mesh, due to the 'balling' effect of the fibre. The unconfined compressive strength of BES increased with increasing fibre content.

The residual strength after attaining the peak stress is considerable for higher fibre content. The variation of unconfined compressive strength with fibre content for polypropylene fibre mesh amended BES shown in Table 5.14. However since 0.6 to 0.8% can provide adequate strength required by the liner and as higher fibre content leads to balling effect as mentioned earlier, higher fibre contents were not attempted.

Table 5.14 Variation of unconfined compressive strength with fibre content for polypropylene fibre mesh amended BES.

Fibre content (%)	Unconfined compressive strength (qu)kN/m^2	Strain at failure (%)	Percentage increase in strength (%)
0	171.28	2.9	0
0.3	177.8	3.8	3.8
0.6	217	4.7	26.69
0.8	307	4.9	79.24

5.2.4.1.2 Effect of type of fibre

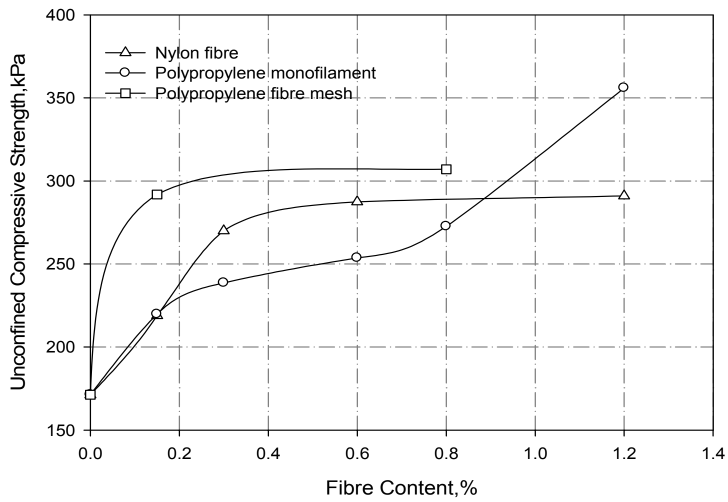


Fig: 5.9 Variation of Unconfined Compressive Strength of BES amended with different fibers

Figure 5.9 shows the effect of type of fibre on the unconfined compressive strength of amended BES. The figure clearly indicates that the contribution of the fibre mesh is significantly high compared to the previous two fibres. The strength steadily increases with fibre content.

It could be seen that for all fibres used in the study, the unconfined compressive strength of amended BES increased with increase in fibre content. For all fibre contents upto 0.8%, the unconfined compressive strength of BES amended with polypropylene fibre mesh is higher than BES amended with other fibres. There is a pronounced residual strength for fibre mesh amended BES at higher percentages and a more ductile behaviour as compared to other fibres in the study. A comparison of the improvement in shear strength of BES amended with different fibres at a fibre content of 0.8% is given in Table 5.15

Table 5.15 Comparison of unconfined compressive strength of fibre amended BES for a fibre content of 0.8%

Type of fibre	Unconfined compressive strength, kPa	Strain at peak stress (%)	Increase in strength (%)
Nylon	231.09	3.49	34.9
Polypropylene monofilament	272.65	4.55	59.18
Polypropylene fibre mesh	307	4.9	79.24

5.2.4.1.3 Effect of fibre length

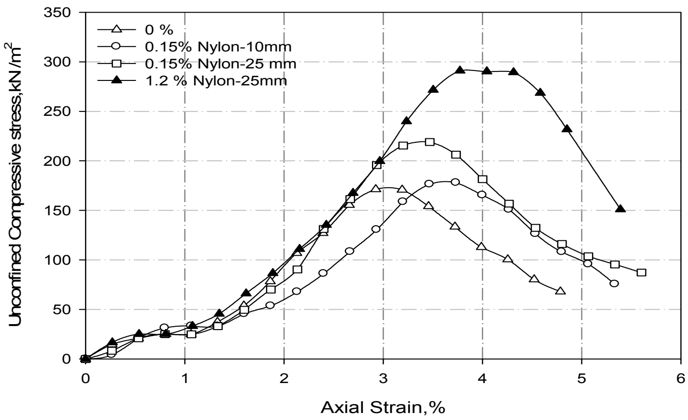


Fig: 5.10 Stress Strain Characteristics of Bentonite Sand mixture amended with Nylon fibre of different lengths

The length of fibre used in the study is an important parameter, like the fibre content. In order to find out whether the length of fibre contributes to the shear strength of fibre amended soil, unconfined compression tests were conducted for fibre lengths of 10 mm, at varying fibre contents. Nylon and polypropylene monofilament fibres were used in the study. Figure 5.10 shows the stress strain characteristics of BES amended with nylon fibre of length 10 mm and 25 mm. The unconfined compressive strength of amended BES increases with increase in length, but the percentage increase in strength is not considerable. The unconfined compressive strength for BES amended with nylon and polypropylene monofilament fibres for lengths of 10 mm and 25 mm at various fibre content is tabulated in Tables 5.16 and 5.17 respectively. It is quite clear from the tables, for a 150% increase in length, the gain in strength is only nominal. Hence, it was concluded that, the effect of length on the unconfined compressive strength of BES amended with nylon and polypropylene fibre is not considerable.

Table 5.16 Unconfined compressive strength for BES amended with nylon fibre of length 10 mm and 25 mm.

Fibre content (%)	**Unconfined compressive strength (kPa)**		**Increase in UCS (%)**
	10mm Length	25mm Length	
0.15	177.83	218.99	23.15
0.6	237.71	287.41	20.9
0.8	219.73	231.09	5.17
1.0	181.56	260.7	79.14
1.2	247.31	291.0	17.8

Table 5.17 Unconfined compressive strength for BES amended with polypropylene monofilament fibre of length 10 mm and 25 mm.

Fibre content (%)	**Unconfined compressive strength (kPa)**		**Increase in UCS (%)**
	10mm Length	25mm Length	
0.15	171.29	219.76	28.30
0.6	287.05	253.61	-11.65
0.8	233.8	272.65	14.25
1.0	299.89	254.06	-15.28
1.2	315.55	355.94	12.8

5.2.4.1.4 Effect of aspect ratio

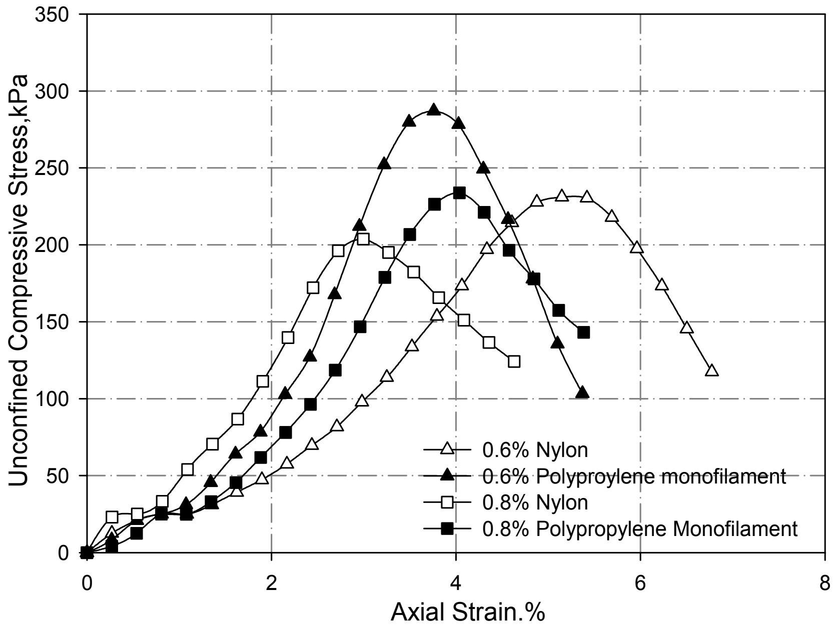


Fig: 5.11 Stress Strain Curves for fibre amended BES at an aspect ratio of 20

The aspect ratio (l/d) of a fibre would give us an idea of how both the parameters—length and diameter, influence the strength characteristics. The details of the aspect ratio considered in the study is given in Table 5.17. The effect of l/d ratio on the UCS of nylon amended BES is given in Table 5.18 and Fig.5.11 for a fibre content of 0.6%. This percentage was selected, since it gave the maximum strength with 10 mm and 25 mm length fibre. It could be seen that as aspect increases, the percentage increase in strength also increases.

Table 5.18 Effect of aspect ratio on nylon amended BES.

Aspect ratio (l/d)	Unconfined compressive strength (kPa)	Increase in strength over amended soil (%)
13	237.71	38.78
19.2	231.12	34.94
25	231.05	34.90
32	287.41	67.80

5.2.4.2 Strength characteristics of Sundried Marine Clay amended with Fibres

5.2.4.2.1 Effect of fibre inclusion on the Unconfined compressive Strength of SMC

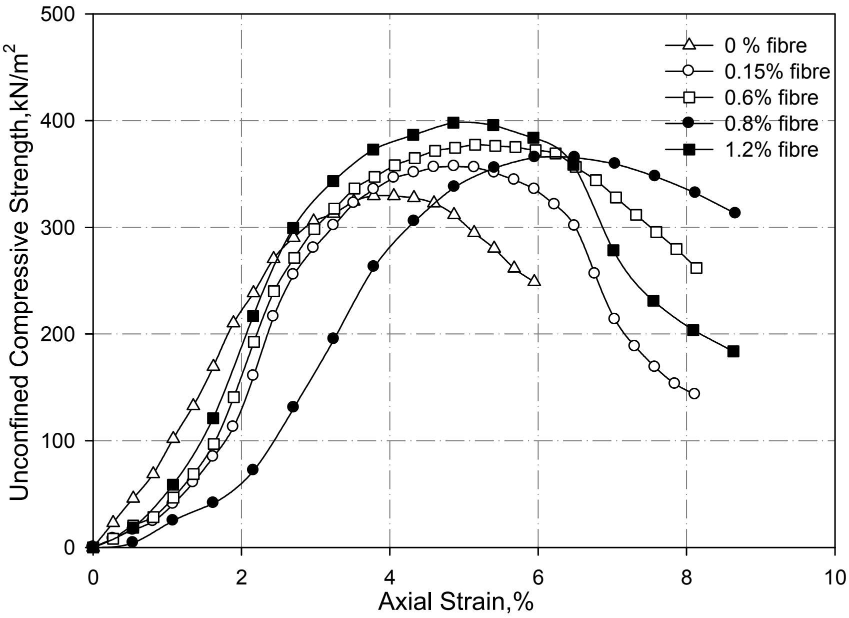


Fig 5.12 Stress Strain Characteristics of Sun dried clay amended with nylon fibres of length 25mm

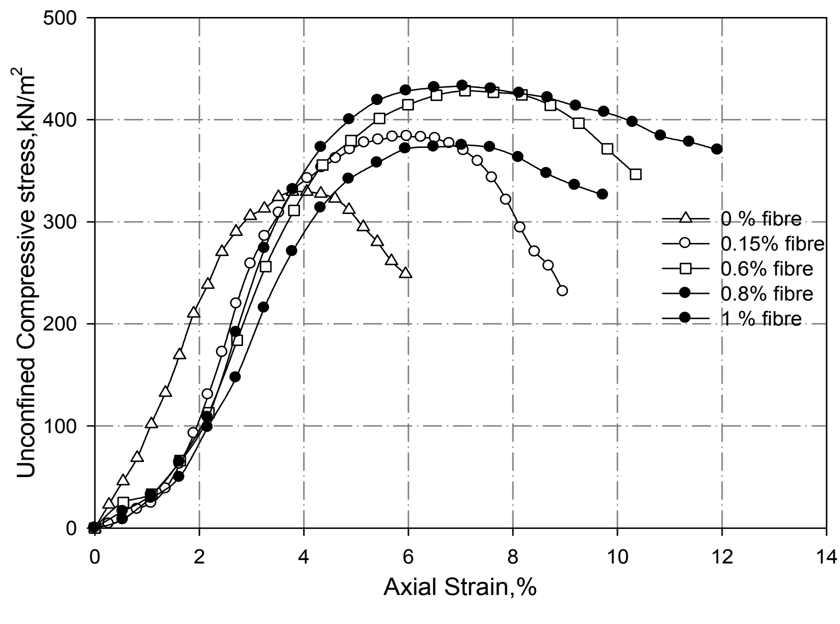


Fig: 5.13 Stress Strain Characteristics of Sun dried Marine clay amended with polypropylene Monofilament fibre

Unconfined compression tests were conducted on samples of SMC and SMC amended with nylon and polypropylene monofilament fibres of length 25mm. The fibre contents used in the study ranged from 0.15% to 1.2%. Figures 5.12 and 5.13 show the stress strain characteristics of SMC amended with nylon and polypropylene monofilament fibres respectively. It can be seen from the stress strain curves that the unconfined compressive strength increases with increase in fibre content, but the percentage increase in strength is very low. Tables 5.19 and Table 5.20 show the unconfined compressive strength, strain at failure and increase in strength of SMC amended with nylon and polypropylene respectively.

From Fig. 5.12 and Table 5.19, it can be seen that the percentage increase in strength is very low. The maximum increase observed is only 21.06% ie from 330 kN/m^2 to 399 kN/m^2. Through the increase in strength is nominal, the increase in strain is very high when nylon fibres are included in SMC.

Table 5.19. Variation of unconfined compressive strength of nylon amended SMC .

Fibre Content	Unconfined Compressive strength (qu) kN/m^2	Axial strain at failure (%)	Percentage increase in Strength (%)
0	329.5	4.05	0
0.15	357	4.6	8.35
0.6	377.34	5.15	14.52
0.8	366.36	6.23	11.19
1.2	398.9	5.13	21.06

Figure 5.13 shows the stress strain curves of SMC amended with polypropylene monofilament fibre. The behaviour is more or less similar to BES with Nylon fibre. The strength increases from 330 to 430 kN/m^2, ie an increase of 30% the increase being directly proportional to fibre content.

Table 5.20 Variation of unconfined compressive strength of SMC with polypropylene monofilament.

Fibre Content	Unconfined Compressive Strength(qu) kN/m^2	Axial strain at failure (%)	Percentage increase in Strength (%)
0	329.5	4.05	0
0.15	382.6	6.25	16.12
0.6	428.4	7.9	30.02
0.8	374.95	7.03	13.79
1.0	433.35	7.32	31.52

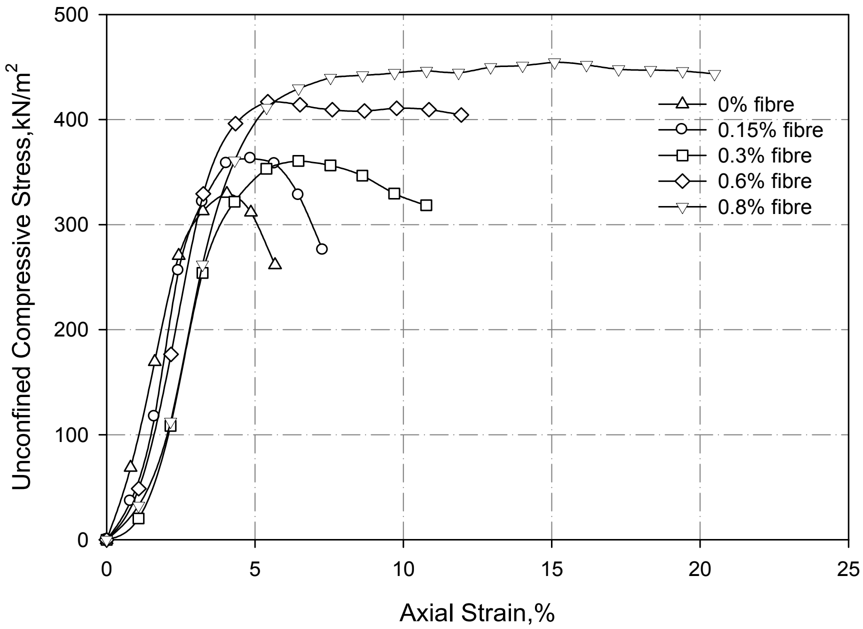


Fig 5.14 Stress Strain Curves for Sun dried Marine clay amended with polypropylene Fiber Mesh

The influence of polypropylene fibre mesh on the unconfined compressive strength of SMC is shown in Fig.5.14. Table 5.21 gives the variation of unconfined compressive strength of SMC with polypropylene fibre mesh. As evident from the figure, the fall in strength after peak value, decreases with fibre content. For 0.6% and 0.8%, there is virtually no fall in strength, which ensures that maximum strength is available for considerable axial strain. It can be seen from Table 5.21, that the axial strain at failure increases with increase in fibre content. For a fibre content of 0.8%, the strain at failure is 11.32%, an enormous increase of 64%.

Table 5.21. Variation of unconfined compressive strength of Polypropylene fibre mesh amended SMC

Fibre Content	Unconfined Compressive Strength (kN/m^2)	Axial strain at failure (%)	Percentage increase in Strength (%)
0	329.5	4.05	0
0.15	362.9	4.86	10.14
0.6	361.6	5.7	9.74
0.8	417.7	6.2	26.77
1.0	447.3	11.32	35.77

5.2.4.2.3 Effect of Type of Fibre

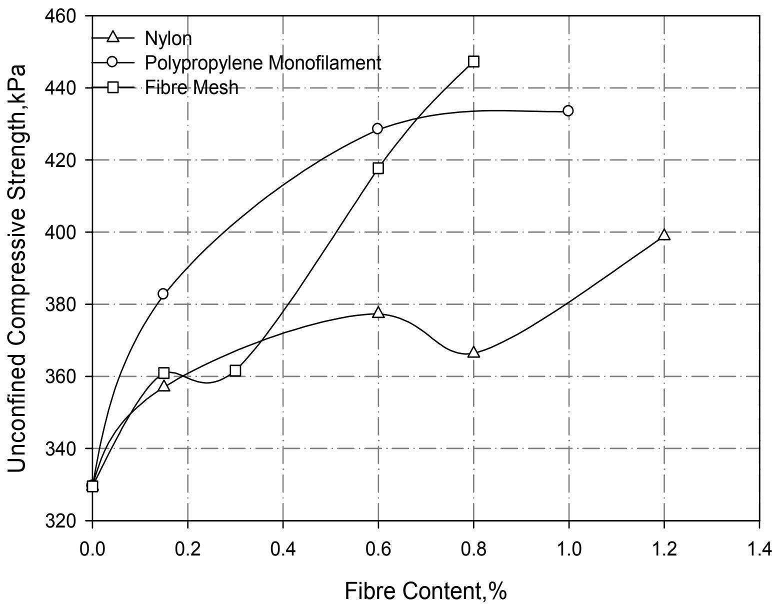


Fig 14(a) Variation of UCC strength of SMC amended with different fibres

The effect of type of fibre on the unconfined compressive strength of SMC is given in Fig.5.14(a). The figure clearly indicates that the contribution of fibre mesh is high compared to the other two fibres. Table 5.22 gives a comparison of the unconfined compression strength of the three fibres amended with SMC.

Table 5.22. Comparison of Unconfined Compressive Strength of Fibre amended SMC at a Fibre content of 0.8%

Fibre Content	Unconfined Compressive Strength kPa	Strain at peak Stress (%)	Increase in Strength (%)
Nylon	366.36	6.23	11.19
Polypropylene Monofilament	374.95	7.03	13.79
Polypropylene Fibre Mesh	447.3	11.32	35.75

5.2.5 Superiority of Sundried marine clay over bentonite enhanced Sand Mixture

A comparative study of the various figures and tables presented earlier clearly shows that sundried marine clay is superior to bentonite enhanced sand mixture.

Table 5.23 Comparison of the Unconfined Compressive Strength of BES and SMC for different Fibres

Type of Fibre	Unconfined Compressive Strength kN/m^2		Ratio of Strength SMC/BES
	BES	SMC	
No Amendment	171.28	329.5	1.9
Nylon Fibre	231.09	366.27	1.585
Polypropylene monofilament	272.65	374.95	1.375
Fibre Mesh	307	447.3	1.457

Fig. 5.15 shows the stress strain curves of BES and SMC amended with polypropylene fibre mesh at a fibre content of 0.8%

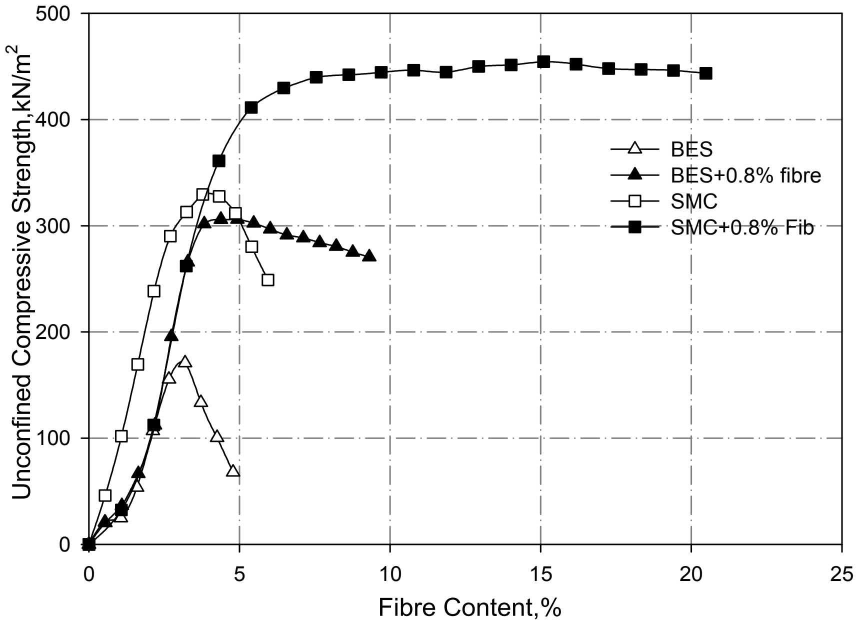


Fig 5.15 Stress Strain Curves for BES and SMC amended with polypropylene Fibre Mesh

5.2.6 Tensile strength characteristics of liner soils amended with fibres

When clay liner soils are subjected to drying, the soil water volume decreases and in consequence, the soil shrinks. If shrinkage is restrained, soils can crack during desiccation when the tensile stresses that develop within the soil exceed the tensile strength of soil. Random discrete fibres when included in soil can impart improved tensile strength to the soil. The tensile tests on BES and SMC amended with fibres were performed to understand whether inclusion of discrete fibres contributed to the tensile strength of soil and if so to what extent.

The tension tests were conducted on a specially designed fixture installed in the direct shear apparatus, the details of which are included in chapter 3. The tests were performed at a rate of 0.25 mm/mt. Based on the shear strength studies explained earlier, the length of the fibre for both nylon and polypropylene monofilament for tension tests was selected as 25 mm.

5.2.6.1 Tensile strength characteristics of fibre amended BES

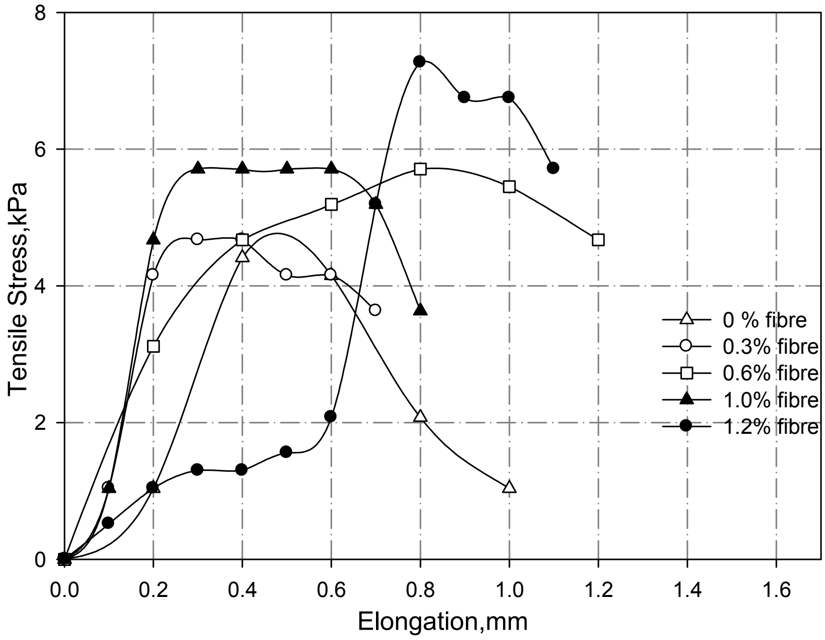


Fig: 5.16 Tension Test result for BES amended with Nylon fibres

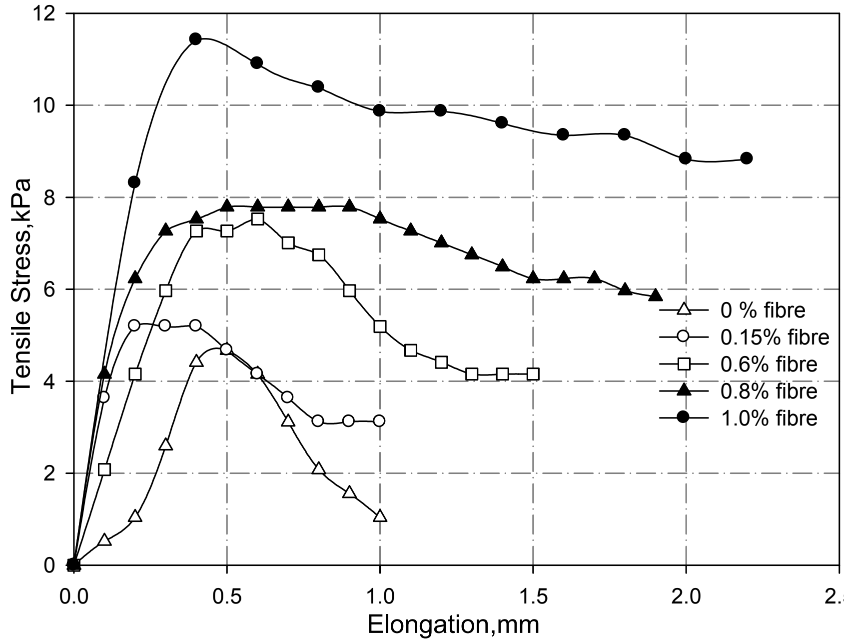


Fig: 5.17 Tension Test Results for BES amended with polypropylene Monofilament fibers

Tension tests were conducted on samples of unamended BES and BES amended with nylon and polypropylene monofilament fibre. The fibre content used in the study were 0.3%, 0.6%, 0.8%, 1.0% and 1.2%. The results of the tension test on BES amended with nylon and polypropylene monofilament fibres are summarized in Tables 5.24 and 5.25 respectively. It can be seen that the inclusion of fibre increased the tensile strength of soil. The tensile stress-displacement relationships of the BES amended with fibres is given in Fig.5.16 and Fig.5.17.

Table 5.24 Tension test results of BES amended with nylon fibres

Fibre content (%)	Tensile strength (kPa)	Elongation at peak tensile stress (mm)	Residual tensile strength (kPa)
0	4.67	0.5	0
0.3	4.67	0.4	3.63
0.6	5.71	06-0.9	4.15
1.0	5.85	0.3-0.6	3.63
1.2	7.27	0.8	5.7

Table 5.24 shows that the tensile strength improves from 4.67 to 5.71 kPa – an increase of 22.2% for a fibre content of 0.6%.More than the improvement in strength, it is the stress-strain behaviour of the amended soil which will help to control the generation and propagation of the desiccation cracks.

From Fig.5.16 and Table 5.24, it is quite clear that BES without any fibre amendment failed immediately after reaching the peak strength. The tensile stress-displacement (elongation) curves of BES amended with nylon fibres show that the peak tensile strength is available over a larger strain zone, which can contribute significantly against propagation of cracks.

Fig.5.16 shows some very interesting tensile stress-elongation characteristics of BES amended with nylon fibres. For a fibre content of 1%, the peak strength is available for an elongation from 0.3 to 0.6 mm. In case of a fibre content of 0.6%, 90% of the peak strength is available for elongations of 0.6 to 1.2 mm. Such a behaviour helps to prevent wide ruptures during crack propogation.

Table 5.25 Tension test results of BES amended with polypropylene monofilament fibres

Fibre content (%)	Tensile strength (kPa)	Elongation at preak tensile stress (mm)	Residual tensile strength (kPa)
0	4.67	0.5	0
0.15	5.19	02-0.4	3.11
0.6	5.19	0.2-0.4	4.15
0.8	7.77	0.5-0.9	5.84
1.0	11.4	0.4-0.5	8.82

Tension test results of BES amended with polypropylene monofilament fibres presented in Fig.5.17 present some very interesting features of the amended soil. Not only there is a significant gain in strength, it considerably alters the stress strain behaviour of the soil. While the unamended soil shows a sharp peak with sudden drop thereafter, the soil steadily grows in ductility and resilience as the fibre content increases.

As Table 5.25 shows an increase of 144% is obtained in tensile strength for a fibre content of 1%. But a comparison of the curves reveals that the soil exhibits greater ductile properties as the fibre content increases. While the unamended soil shows an abrupt drop after peak strength, the curves slowly and steadily flattens after peak stress as per the increasing fibre content. More than 90% of peak strength is available for elongation of 0.5 to 2.0% and 0.4 to 2.25 mm for fibre contents of 0.8 and 1% respectively.

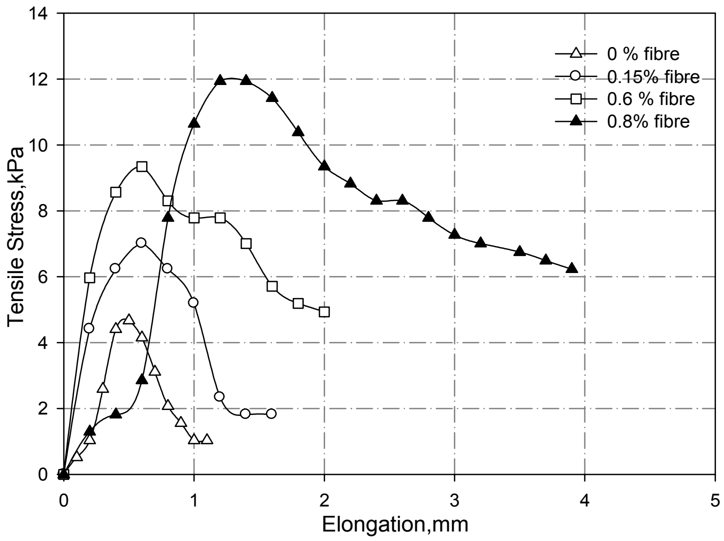


Fig: 5.18 Results of tension tests in BES amended with polypropylene fibrillated Fibre Mesh

The tension test results of BES amended with fibrillated polypropylene fibre mesh follow the same trends as the other fibre amendments especially propylene monofilament fibres. The results are presented in Fig.5.18 and Table 5.26.

Table 5.26 Tension test results of BES amended with polypropylene fibre mesh

Fibre content (%)	Tensile strength (kPa)	Elongation at preak tensile stress (mm)	Residual tensile strength (kPa)
0	4.67	0.5	0
0.15	7.01	0.5-0.6	1.817
0.6	9.34	0.7	4.43
0.8	11-94	1.2-1.5	6.30

The elongation at tensile strength is higher for fibrillated fibre mesh amended BES. This is a positive trend as far as the control of desiccation crack is concerned.

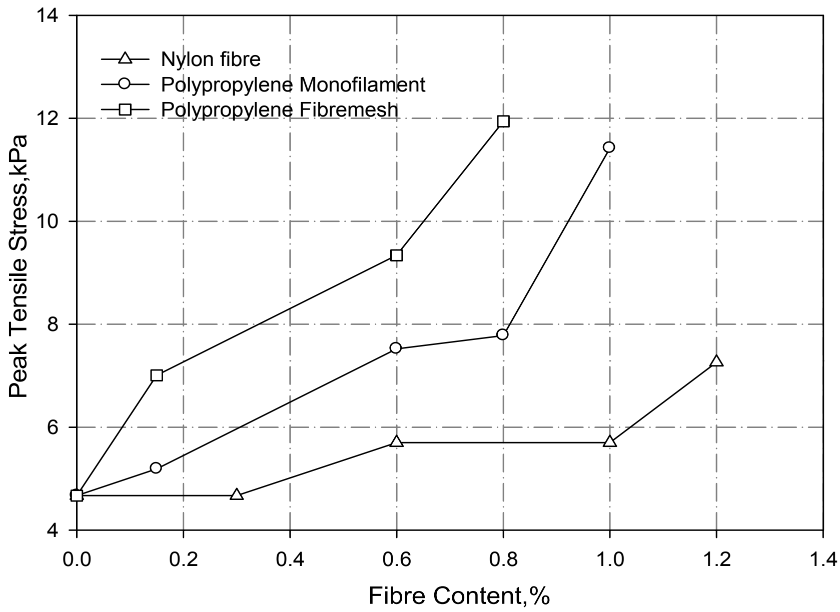


Fig 5.19 Variation of peak Tensile stress with fibre content for BES mixture amended with fibres

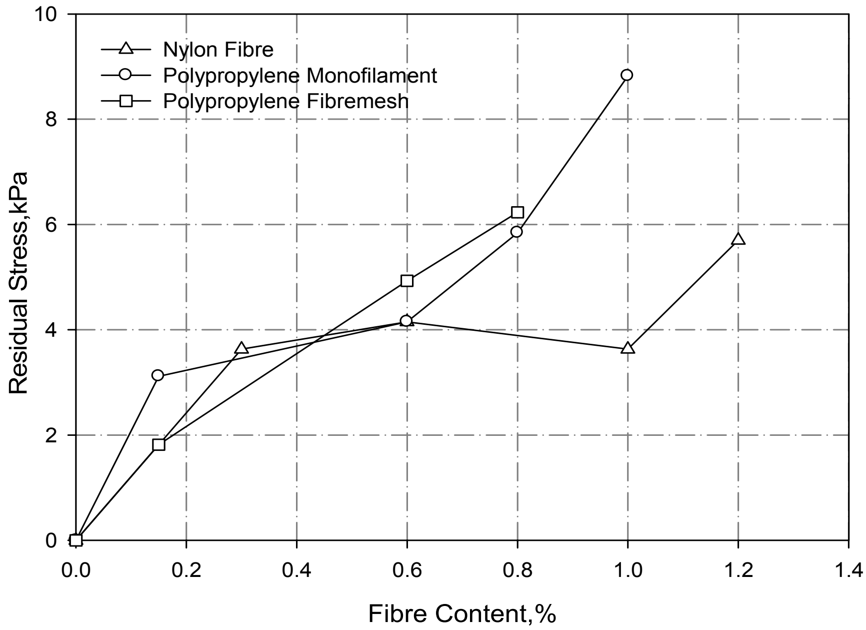


Fig: 5.20 Variation of Residual Tensile Stress with fibre content for BES amended with fibers

The variation of tensile strength and residual strength for BES amended with fibres is shown in Fig.5.19 and Fig.5.20 respectively. The tensile strength of BES amended with polypropylene fibre mesh is greater at all fibre contents than the other fibre amended BES. At a fibre content of 0.6%, the increase in tensile strength of fibre mesh amended BES is 100% compared to 22% and 61% of nylon and polypropylene monofilament fibre amended BES.

The comparison of the residual tensile strength of the amended BES with the fibres present a different picture. Though , as shown in Fig.5.20 the fibrillated fibre mesh gave a high tensile strength, the residual tensile strength after failure is almost similar to that of BES amended with other fibres. The residual tensile strength, at a fibre content of 0.6%, for BES amended with polypropylene fibre mesh is 4.9 times that of unamended BES. The residual tensile strength of BES amended with nylon and polypropylene monofilament fibres is 4.15 times that of unamended BES. Polypropylene monofilament fibre gives a better performance especially at high percentages for bentonite enhanced sand mixtures.

5.2.6.2 Tensile strength characteristics of fibre amended sun dried marine clay

5.2.6.2.1 Effect of fibre inclusion in the tensile strength of SMC

Tension tests were conducted on samples of unamended SMC and SMC amended with nylon and polypropylene monofilament fibre. The fibre content used in the study were 0.3%, 0.6%, 1.0% and 1.2%. The results of the tension tests conducted on SMC amended with nylon and polypropylene monofilament fibres are tabulated and presented in Tables 5.27 and 5.28 respectively, show that there is a definite increase in tensile strength of SMC when it amended with fibres.

Table 5.27 Tension test results of SMC amended with nylon fibres

Fibre content (%)	Tensile strength (kPa)	Elongation at peak tensile stress (mm)	Residual tensile strength (kPa)
0	9.34	0.2-0.3	0
0.3	19.207	0.4	3.63
0.6	25.956	0.8	8.31
1.0	43.09	1.4	14.01

The tensile strength of SMC amended with nylon fibres showed a tensile strength of 43 kPa, at a fibre content of 1%, which is 3.6 times (361%) higher than that of unamended SMC. Similarly, the tensile strength of polypropylene monofilament amended SMC is 29 kPa, which is 2.11 times greater than the tensile strength of unamended SMC.

Table 5.28 Tension test results of SMC amended with polypropylene monofilament fibres

Fibre content (%)	Tensile strength (kPa)	Elongation at preak tensile stress (mm)	Residual tensile strength (kPa)
0	9.34	0.2-0.3	0
0.3	15.574	0.3	3.14
0.6	21.54	0.6	7.26
1.0	24.92	0.6	13.5
1.2	29.07	0.6	13.24

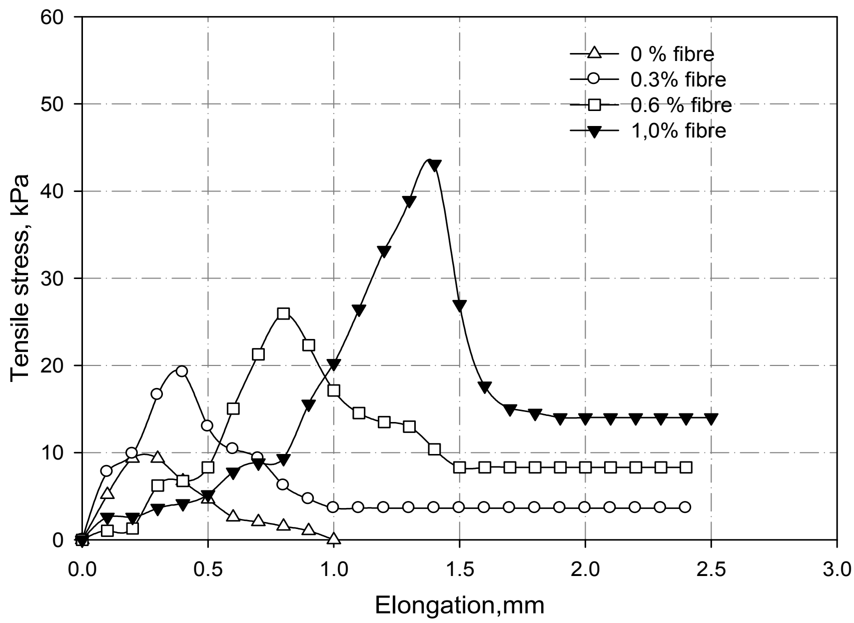


Fig: 5.21 Tension test results of Nylon amended Sun dried Marine Clay

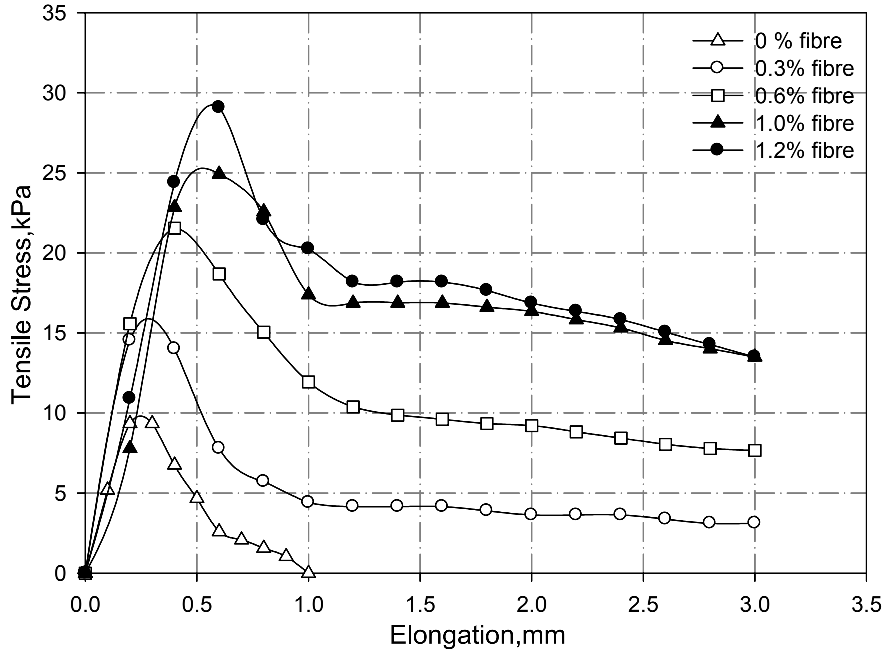


Fig: 5.22 Tension test result on SMC amended with polypropylene Monofilament fibre

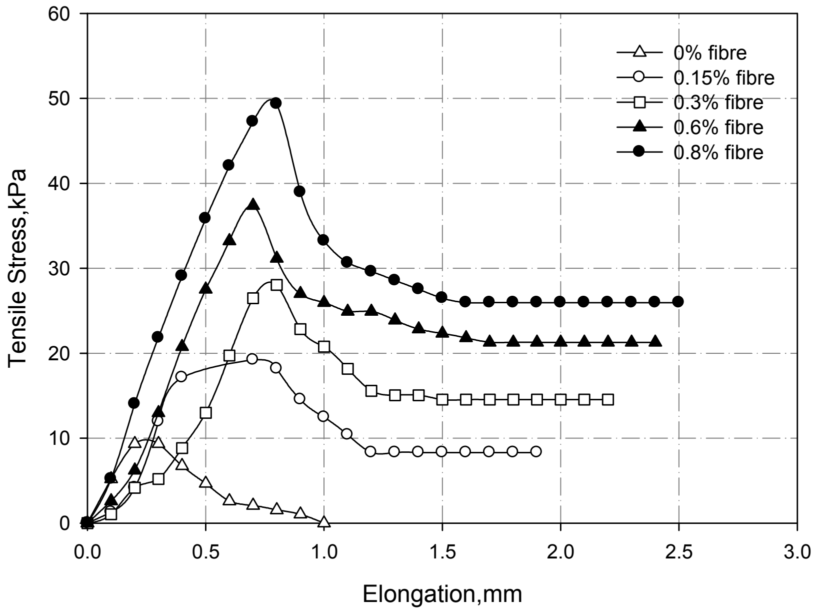


Fig: 5.23 Tension test results for SMC amended with polypropylene Fibre Mesh

Another aspect of fibre inclusion can be seen from the tensile stress-displacement curves of the SMC samples presented in Figs.5.21-5.23, amended with nylon, polypropylene monofilament and polypropylene fibre mesh respectively. These figures show that the fibres contributed to the residual strength of the soil following formation of a tension crack. That is SMC amended with fibres showed ductile behaviour under pure tensile loading while those with no fibres failed immediately after reaching the tensile strength.

5.2.6.2.2 Effect of the fibrillated fibre structure

The effectiveness of the fibre depends upon how the fibres interact with different clays. The mechanism by which the fibres interact with the soil at low normal stresses is through adhesion. When a tensile force needs to be mobilized in the fibres, such as that which occurs as a drying clay shrinks and desiccation cracks tend to develop, only adhesion restrains the fibre from pullout and thus allows for its tensile resistance to develop. The amount of adhesion developed for each fibre depends upon the surface area of the fibre, fibre geometry and type of soil. This adhesion developed will be small for smooth short fibres mixed with clay, especially when compared with the available tensile strength of the actual fibres.

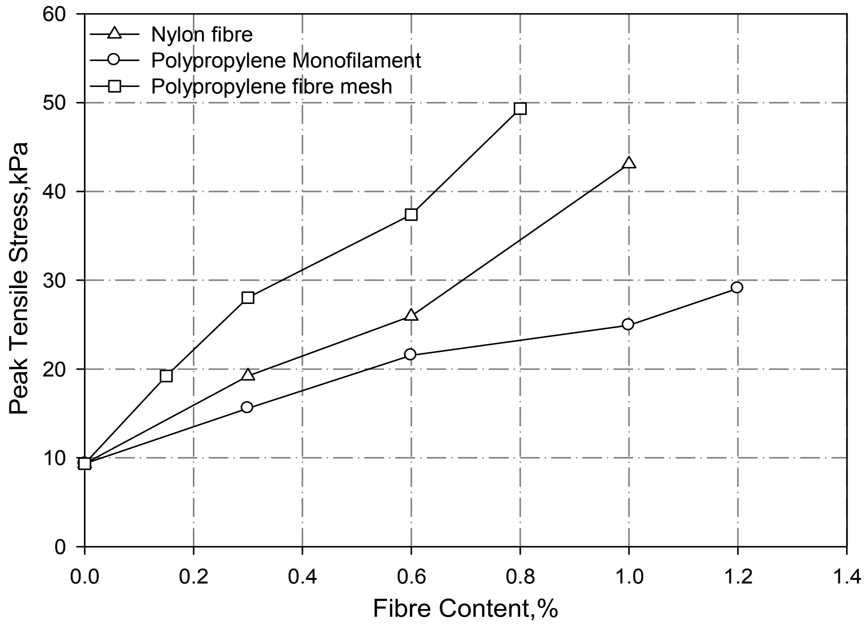


Fig: 5.24 Variation of peak stress with fibre content for SMC amended with different types of fibre

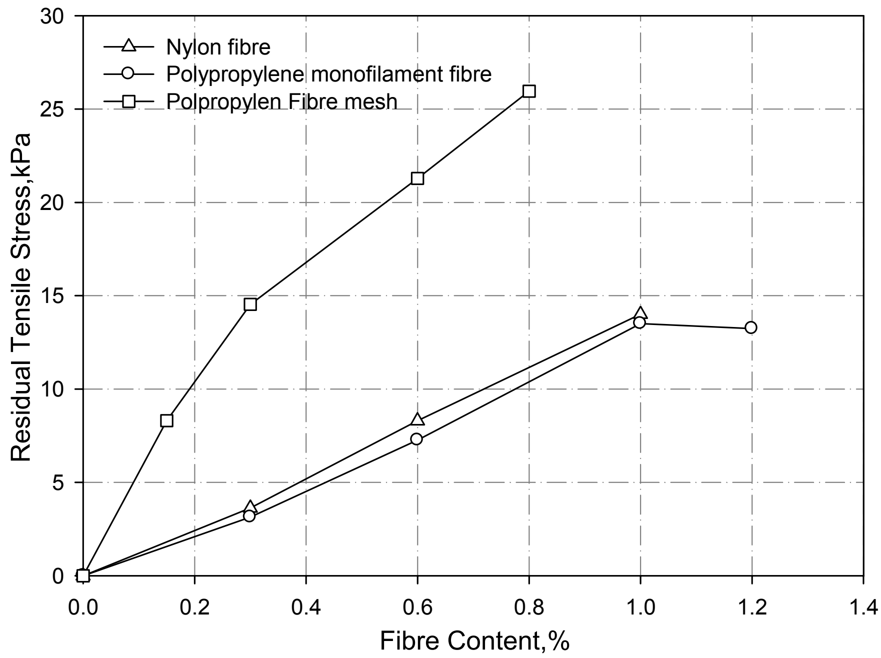


Fig: 5.25 Variation of the residual tensile stress with fiber content for Sun dried Marine Claly amended with fibers

The variation of tensile strength and residual tensile stress with varying fibre content for SMC amended with fibres is shown in Fig.5.24 and 5.25. The same is also tabulated for fibre contents of 0.3% and 0.6% in Table 5.36.

Table 5.29 Variation of tensile strength and residual tensile strength for fibre amended SMC.

Fibre Content	0.3%			0.6%		
	Nylon	Poly-propylene monofila ment	Poly-propylene fibre mesh	Nylon	Poly-propylene monofilam ent	Poly-propylene fibre mesh
Tensile strength (kPa)	19.21	15.57	28.03	25.96	21.54	37.38
Increase in peak stress (%)	105.7	66.7	200.1	177.9	130.6	300.2
Residual tensile strength (kPa)	3.63	3.14	14.54	8.31	7.26	21.28

It can be seen from the figures and table, that fibrillated fibre mesh is far superior to the other amendments in SMC. The figures and table also show that the higher surface area contributes to the adhesion and hence an increased tensile strength and residual tensile strength in nylon fibre amended SMC than polypropylene monofilament amended SMC.

The factor which contributes to the higher tensile strength of fibrillated polypropylene fibre mesh is the surface area as well as the geometry of the fibre mesh. As stated earlier in Chapter 3, these fibres formed a miniature mesh with a diamond shaped pattern when stretched perpendicular to the direction of the long chain polymer. This lattice structure opens up during mixing to form a three dimensional network. This fibre network contributes to the improved tensile strength developed by the soil. The fibre network has improved mechanical bonding under tension as compared with monofilaments due to stress transfer throughout the fibrils, rather than solely at the fibre soil interface. For this type of fibre to be pulled out of the clay, the soil in front of the cross pieces would need to be moved in order to allow the longitudinal fibres to move, ie. passive resistance of the soil in front of the cross pieces would also contribute to the resistance to volume change. In this mechanism, therefore passive resistance due to the cohesive strength of the soil is being taken advantage of in addition to the adhesion between the fibres and soil.

Even though, during the mixing process some fibrils are broken, a vast majority opens into a network. Hence it is expected that the above explained mechanism is in operation while conducting the tensile test on SMC amended with polypropylene fibrillated fibre mesh.

In case of SMC, inclusion of random discrete fibres improved the tensile strength of unamended soil significantly than its contribution in BES. In SMC, of the three fibres included in the study, fibrillated fibre mesh gave the best results both in terms of tensile strength, strain at failure and residual tensile strength.

5.2.6.3 Superiority of sun dried marine clay over bentonite enhanced sand mixture

A comparative study of the various figures and tables presented earlier clearly brings out the fact that sun dried marine clay is superior to bentonite enhanced sand mixture.

Table 5.30 Tensile strength of BES and SMC

Type of fibre content	0.6%		1%		Percentage increase		Ratio of strength SMC/BES	
	BES	SMC	BES	SMC	BES	SMC	0.6%	1.0%
No amendment	4.67	9.34	--	--	--	--	--	--
Nylon fibre	5.71	25.95	5.85	43.09	122	277	4.5	7.36
Polypropylene monofilament	5.19	21.54	11.4	24.92	111	230	4.15	2.18
Fibre mesh	9.34	33.5	11.64	49.2 (0.8%)	214	358	3.58	4.22

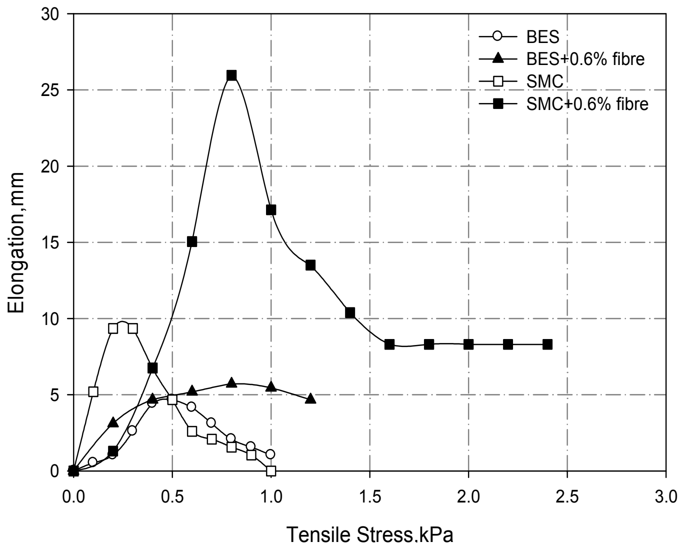


Fig: 5.26 Tension test results for BES and SMC with 0.6% Nylon fibre

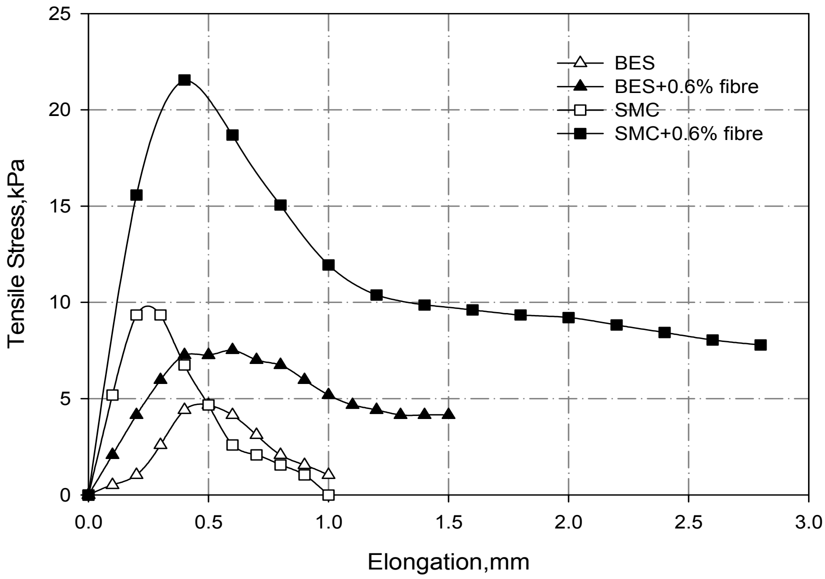


Fig: 27 Tension test for BES and SMC amended with polypropylene monofilament

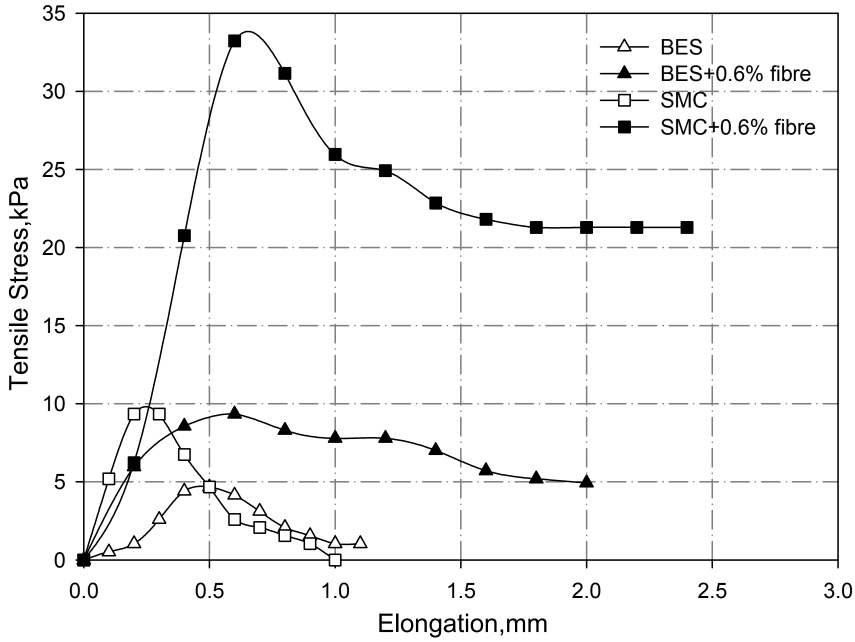


Fig: 5.28 Tensile test results for BES and SMC amended with polypropylene Fiber Mesh

Figures 5.26 – 5.28 shows the tension test results of BES and SMC for samples amended with 0.6% nylone fibre while BES a tensile strength of 5.71 kPa, The corresponding strength in SMC amended with 0.6% fibre content, are 122 and 277% respectively.

In case of polypropylene monofilament the tensile strength increases from 4.67 kPa to 5.19 kPa which is just 111% whereas the tensile strength registered for a fibre content of 0.6% for SMC, is 21.54 kPa which is 230% of the strength for unamended soil.

Since fibre contents beyond 0.8% led to 'balling' effect in soils, especially sundreid marine clay , the tensile strength of SMC amended with 0.8% of fibre mesh only is presented in Table 5.37 a. But this itself produces a remarkable increase in tensile strength which is 49.2 kPa and marks a value improved to 358% of the initial strength of 9.34 kPa.

The ratio of tensile strength of SMC and BES for fibre contents of 0.6 and 1.0% are also presented in the Table. They show that the ratio is normally in the range of 4 to 7, which clearly points to the fact that sundried marine clay is superior to BES, in case of potential for improvement in tensile strength by amendment with fibres.

5.2.7 Desiccation tests of fibre amended Sundried Marine Clay

Through several series of tests, it has now been conclusively established that bentonite enhanced sand has several limitations when it confronts desiccation cracks. With the help of a new technique of digital image processing and crack intensity factor, the disadvantage of BES has been clearly brought out and assessed with greater accuracy.

Eventhough BES satisfies the EPA standards, it fails to perform satisfactorily when the liner material is subjected to desiccation. Several instances of failure of liner have been reported in the literature. The details of this is given in chapter 2. This points to the fact that resistance to desiccation cracking is a parameter of concern if soil liners are to perform under severe seasonal changes. The crack intensity factor, which now can be assessed with sufficient accuracy, should also be included as a norm to be satisfied by the liner material, as in the case of as hydraulic conductivity, minimum fines content etc.

A comparison was drawn between BES and SMC at all stages of the work and Sundreied Marine Clay was found to have a number of advantages over bentonite enhanced sand. This was found superior not only as an unamended soil but the amendments in the form of the three fibers could yield better results with regard to increase in unconfined compressive strength and tensile strength. Both these can contribute to development of resistance to crack formation and propagation. Hence it was decided to carry out a series of tests on fiber amended Sundried Marine Clay.

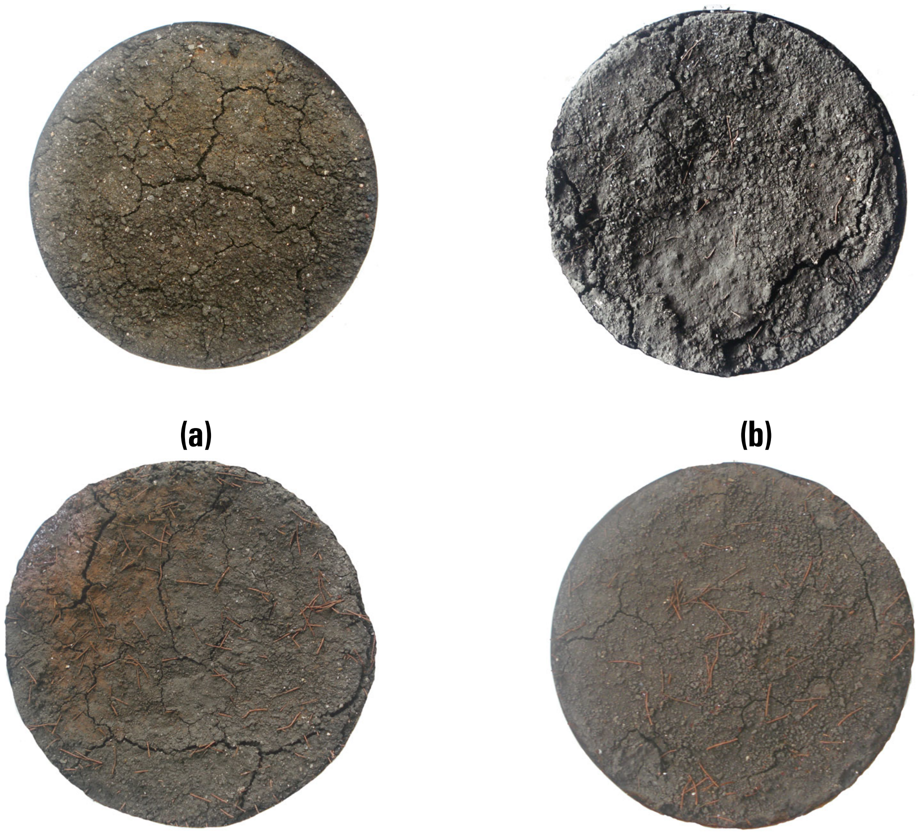
5.2.7.1 Desiccation Tests on Fibre amended Sundried Marine Clay

Sundried marine clay (SMC) was amended with 0.15%, 0.65% of the three fibres selected for the study. Desiccation tests were conducted for two cycles of wetting/dry cycles. Photographs of the desiccated samples were taken after each wet/dry cycle. In addition to the above, desiccation tests were also performed on 1.15% of nylon fibre amended SMC. All the tests performed on SMC were subjected to 3 cycles of wetting/drying. Tests were performed on replicate samples of SMC, (both unamended and amended) for a particular fibre content to check the repeatability of results.

5.2.7.2. Effect of Fibre inclusion in the Control of Desiccation Cracks

In order verify the effect of fibre content as a measure to control cracks. SMC was amended with 0.15%. 0.65% and 1.15% of nylon fiber. The results are presented in Fig. 5.29.

The photographs of the desiccated samples were analysed by using MATLAB. The values of CIF were determined and are presented in Table 5.31. It can be seen from the table that the CIF reduces with increase in fiber content corresponding crack reduction increase. In Fig 5.29, the photographs from (a) to (d) shows a consistent reduction in cracks and the crack pattern in (d) presents an appreciable reduction in cracks, compared to the unamended specimen in (a).



(a) (b) (c) (d)

Fig: 5.29 Photographs of desiccated samples of SMC (Cycle 1)

a) unamended (b) 0.15% nylon

(c) 0.65% nylon (d) 1.15% nylon

Table 5.31 Crack Intensity factor for SMC amended with Nylon Fibre

Fibre Content (%)	Crack Intensity Factor (%)	Crack Reduction (%)
0	9.83	-
0.15	14.72	49.75
0.65	5.69	42.12
1.15	2.97	69.79

Fig. 5.30 show the photographs of SMC samples reinforced with 0.15% and 0.65% of polypropylene fibre and fiber mesh.

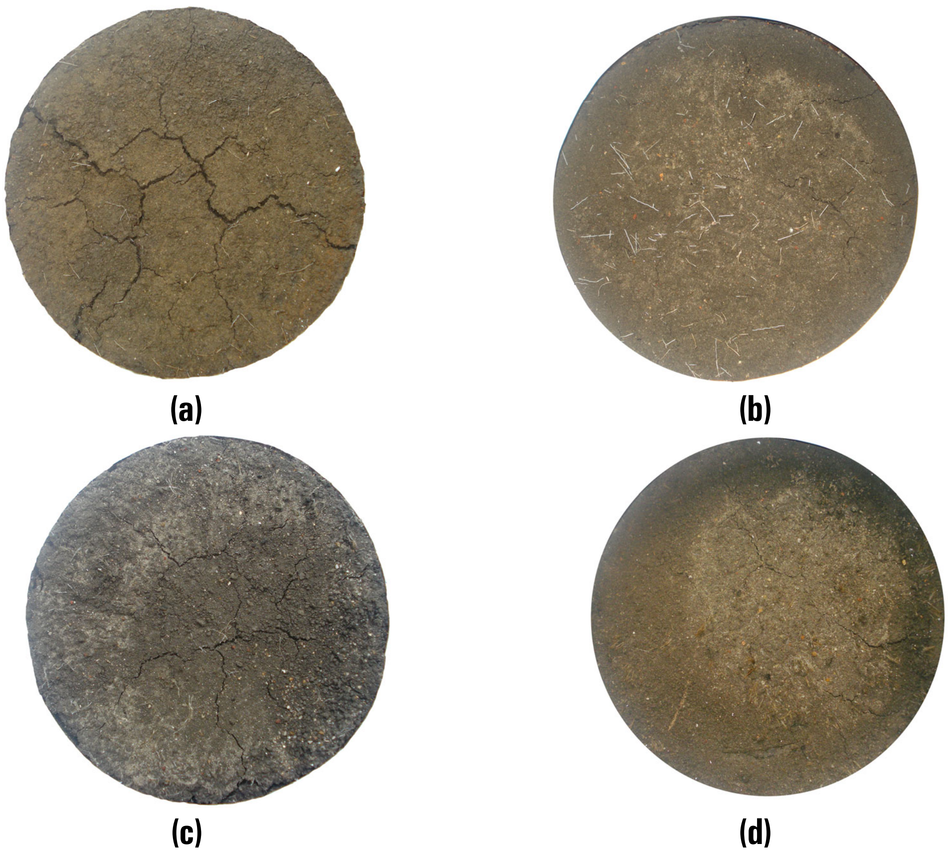


Fig: 5.30 Photographs of desiccated samples of fiber amended SMC (Cycle 1)

(a) 0.15% polypropylene fibre (b) 0.65% polypropylene fibre

(c) 0.15% polypropylene fibre mesh (d) 0.65% polypropylene fibre mesh

It can be clearly seen that for a fiber content of 0.65% of polypropylene fiber and fiber mesh, the cracks formed after one cycle of wetting and drying are considerably low, in number and extent.

The values of CIF for these samples are presented in Table 5.32.

Table 5.32 CIF of SMC Amended with different Fibre for Cycle 1

Fibre Content (%)	Type of Fibre		
	Nylon	Monofilament	Fibre Mesh
0	9.83	9.83	9.83
0.15	14.72	1.94	0.95
0.65	5.69	0.03	0.41

It can be seen that they are fully consistent with the crack configuration in the photographs

A comparative study of the effect of the three fiber used is presented in Fig.5.31. It can be seen that fibrillated fiber mesh is the most effective in reducing the cracks developed by desiccation.

(a) (b)

(c) **(d)**

Fig: 5.31 Photographs of desiccated samples for SMC amended with fibre (Cycle 2)

(a) unamended (b) 0.15% nylon

(c) 0.15% monofilament (d) 0.15% fibre mesh

Thus the series of tests to confirm the efficacy of fiber amendment for reducing the formation of cracks are in full conformity with earlier findings.

Chapter 6

SUMMARY AND CONCLUSIONS

6.1 INTRODUCTION

Modern engineered landfills are designed to minimize or eliminate the constituents released to the environment. Solid and hazardous waste landfills are required by government or local regulations to cover waste materials prior to or as part of final closure. Moreover successful design and construction of soil liners and covers involve many aspects such as selection of material, determination of construction methodology, analysis of slope stability and bearing capacity evaluation of subsidence (settlement), and consideration of environmental factors (Daniel 1987; Daniel and Benson 1990).

Compacted soil is widely used as a material for landfill and waste impoundments. Most regulatory agencies require that the compacted soil liner and cover should be designed to meet the minimum design requirements. The compacted soil liner and cover system may also suffer damages from desiccation cracking and differential settlement problems, consequent increase in the hydraulic conductivity and reduction in the sealing effect of the cover system drastically.

6.2 TECHNIQUES DEVELOPED FOR THE STUDY

Even though it is well known that loss of water content in clay liners, will induce desiccation cracks, research in this area has been taken up only recently. Hence attempts to study the mechanism of development of cracks by simulating desiccation conditions in the laboratory, quantify the extent of crack formation and develop methods to control the formation and propagation of cracks, are limited.

6.2.1 Preparation of Specimens for Tests

When field condition of wetting and drying due to seasonal variations are simulated in laboratory, the time taken for specimens to get either fully saturated or fully dried by ponding water and by heating, both at the top surface only, is important, as this decides the total time taken, for one cycle. Similarly the effect of repetition of such cycles with regard to desiccation cracks is also important as the number of cycles required to get a final configuration of the cracks, is also a parameter to be investigated . The present series of tests in this direction has shown that wetting and drying for 7 days each and subjecting the sample to 2-3 wet/dry cycles will simulate conditions prevailing the field.

6.2.2 Measurement of Cracks by Digital Image Processing

Crack Intensity factor (CIF) is the most useful and effective parameter for quantification of desiccation cracks. Existing practices for determination of CIF, though existing practices, mainly by mechanical measurements, were tried initially and it was found to have serious limitations. The accuracy and reliability of assessing cracks through CIF had serious limitations owing to the following:

[1] The mechanical measurement of width and length of cracks lacked the accuracy required.

[2] The process of measurements interfered with the crack configuration of the samples

[3] Measurement of cracks in the same specimen when it passes through series of wetting and drying was almost impossible as techniques available led to disturbance of crack formation in the samples.

[4] Very fine cracks could not be accounted for, as they were not amenable to mechanical measurement.

[5] Personal errors were unavoidable

In this context, application of the digital image processing technique for measurement of crack was of tremendous advantage over the existing practices.

It was established in this work, in unequivocal terms that digital image processing can be used successfully for measurement of desiccation cracks, in all types of soils tested and for all amendments employed. Since it takes into account, the contrast in pixel intensity, even hair line cracks could be accounted for, which was almost impossible earlier. The repeatability of the tests could also be well established, as the variation in CIF from identical samples had a very narrow range of 0 to 2%. The significance of this technique is further underscored due by the fact that, this is a nondestructive testing procedure. This provides tremendous potential for further work in this direction as field study of performance of clay liners would not have been possible, by practices followed till date.

6.2.3 Experimental Setup Developed

To simulate the field conditions during summer, it was necessary to subject the specimen to a standardized heating, irrespective of the atmosphere conditions in the laboratory. The drier designed and developed, helped to provide a standard drying process for all specimens with very minimal variations in temperature.

Similarly, the tension test assembly developed to determine the tensile strengths of soil was also found effective and sufficiently accurate. Since cracks develop when the tensile stresses exceed the tensile strength, the results of tension tests are significant in the study of desiccation cracks.

6.3 DEVELOPMENT OF DESICCATION CRACKS IN LINER MATERIAL

The present study aimed at critically looking at the current practice of the installation of compacted clay liner using bentonite enhanced sand (BES). However, its behaviour, when subjected to alternate wetting/drying, had not been fully investigated especially with regard to development of desiccation cracks. The possibility of identifying a suitable or better alternative, was also investigated for which sundried marine clay (SMC) was selected.

6.3.1 Desiccation Cracks in Bentonite Enhanced Sand

Since BES apparently satisfied all the norms set by EPA, not much work had been done regarding how this will behave when subjected to series of wetting and drying caused by seasonal variations. The fact that BES is highly susceptible to the development and propagation of cracks, has been clearly brought out through a series of experiments. Determination of crack intensity factor by digital image processing, aided by MATLAB version 7 software, helped to quantity the crack formation and a comparative study was made possible. The crack intensity factor (CIF) which was only 18.09 for the first cycle, increased to 39.75 and 21.22 after second and third cycles respectively. A matter of greater concern was the depth of cracks which was over 5 cm, which brings down the effective thickness of the clay barrier. In the present case, the thickness was reduced to less than 50%. This promotes the permeation of leachate into the body of the liner and thus reduce the thickness further.

It has been established through a series of experiments and reliable measurement techniques, that the prevailing practice of installation of clay liners using bentonite enhanced sand suffers from serious limitations and the practice itself can be questioned. This prompted the studies for an alternative to bentonite enhanced sand

6.3.2 Desiccation Cracks in Sundried Marine Clay

Preliminary studies itself indicated sundried marine clay can be a better liner material. Even a visual comparison of the photographs of the desiccated samples of BES and SMC could lead to this conclusion.

The CIF of sundried marine clay was only 9.83% in the first cycle as against 18.09 of BES, and this steadily decreased with number of cycles unlike BES. The maximum depth of cracks was 0.7cm to 2.0 cm after the first cycle compared to 5 to 5.1 cm observed in BES sample. The ratios of the crack intensity factors BES to SMC are 1.84, 5.29 and 4.63 for the first, second and third cycles respectively.

From all the figures presented above, it could be safely concluded that the proposed alternative, namely the Sundried Marine Clay, is far superior to bentonite enhanced sand in almost all aspects. Yet another combination, 20% bentonite with 80% SMC, was also investigated to a limited extent and the results were encouraging.

6.4 IMPROVEMENT OF LINER MATERIAL BY AMENDMENTS

It has already been established that desiccation cracks is a serious problem faced in engineered land fills.Attempts were made to improve the strength characteristics of the two liner materials selected so as to control the development of cracks. The soil was reinforced with randomly distributed fibre reinforcement. The random distributions helped to maintain strength isotropy and to ensure that potential planes of weakness are absent.

Three types of fibres viz, nylon, polypropylene monofilament and polypropylene fibre mesh were used as soil reinforcement. The fibre contents selected ranged over 0.15% to 1.2% depending upon the ease in mixing and the ‘balling’ effect, experienced when higher fibre contents were tried.

6.4.1 Compaction Characteristics of Amended Soils

A series of standard compaction tests were conducted in soils with different percentage of nylon fibre. It was observed that the variation in OMC was in the range of -0.5% to +0.85% and maximum dry density varied between -0.32 kN/m^3 and+0.21 kN/m^3. Since the changes were marginal, the values of unamended soils as such were used for amended soils also for the further tests.

An equation for maximum dry unit weight (MDUW)could be developed from the test data as given below

$$MDUW = 1.0072 + 9.4927/OMC + 64.5824/OMC^2$$

with $R^2 = 0.9455$

6.4.2 Hydraulic Conductivity of amended Soils

Since EPA insists that the hydraulic conductivity of the liner material shall not exceed 1x10^{-9} m/sec, it had to be verified whether the reinforcements make the soil more permeable.

Hydraulic conductivity was measured with the help of consolidation apparatus, under three consolidation pressures of 6.25 kPa, 12.5 kPa and 25 kPa.

It was observed for BES that incase of nylon fibres, for fibre contents upto 1.0%, values were below 1x10^{-9}m/s. Higher fibre contents can be allowed when polypropylene monofilament fibres are used. However polypropylene fibre mesh reinforcement adversely affected the hydraulic conductivity for fibre contents above 0.6%.

Sundried marine clay liners are not much affected by fibre reinforcement in this aspect. Even for higher fibre contents of 1.2%, the hydraulic conductivity was well within the permissible limit of $1x10^{-9}$ m/sec for nylon and polypropylene monofilament. For the fibre mesh fibre contents around 0.6% were quite acceptable.

In the hydraulic conductivity tests also, SMC proved superior to BES, as higher fibre contents could be permitted in the former, which can considerably improve the strength characteristics of the soil.

6.4.3 Improvement in Strength Characteristics by Amendment

Compacted clay liner should possess adequate shear strength to maintain its structural integrity. The stress-strain behaviour of the liner material can be controlled with the help of fibre reinforcement, which will improve the strength characteristics of the soil. These were established with the help of a series of unconfined compression strength tests and tension tests.

6.4.3.1 Unconfined Compressions Strength of Amended Soil

Unconfined compression strength tests were conducted on BES specimens, reinforced with all the three fibres. Nylon does not influence the strength or stress-strain behaviour of BES. The contribution from polypropylene monofilament fibres is also marginal. Addition of fibrillated fibre mesh influences the strength as well as the stress-strain behaviour of the soil to a great extent. For fibre contents of 0.6% and above, the residual strength of the soil after the peak stress is almost the same for a considerable range of strain. An addition of 0.8% of fibre mesh increases the strength from 171.28 to 307 kN/m^2 - an increase of 79.24%. Thus fibrillated polypropylene fibre mesh is the most ideal reinforcement for BES in case of unconfined compressive strength.

Unconfined compression strength tests on SMC specimens show that the influence of fibre content is not very significant. However, the stress strain curves show that the soil retains residual strength after the peak stress. In case of polypropylene monofilament fibre and fibre mesh the residual strength is almost equal to the peak stress. This indicates that the soil behaves more like a ductile material for fibre contents of 0.6 and 0.8% in case of the above two fibres.

Eventhough both soils do not show any notable increase in unconfined compressive strength, sundried marine clay has certain advantages due to the improved stress-strain characteristics.

6.4.3.2 Tensile Strength Characteristics of Amended soils

When clayey soils are subjected to drying, as in desiccation, soil water volume decreases and soil shrinks. If the shrinkage is restricted, soils crack. Cracks develop when the tensile stress in soil exceeds the tensile strength. Hence attempts were made to increase the tensile strength of the soil and the fibres could make significant contributions in this direction.

As shown by the results presented in the form of tables and graphs, it can be concluded that fibre reinforcement increases the tensile strength of soil. Nylon fibre are not helpful in case of BES. But addition of 1% of polypropylene monofilament fibre increases the tensile strength from 4.67 to 11.4 kPa - an increase of 144%. For fibre mesh the tensile strength increase from 4.67 to 11.94 kPa which gives an increase by 155% for a fibre content of just 0.8%.

In addition to increasing the shear strength, the reinforcement helps to provide a residual strength close to the peak stress for a considerable range of strain. For example, BES amended with 1% polypropylene monofilament fibre retains a residual strength of about 90% of the peak stress for a stretch of 0.4 to 2.25 mm. Such a behaviour helps to prevent propagation of cracks formed at peak stresses. This helps to prevent possible wide ruptures of the compacted clay liners.

In case of SMC also, amendment with fibre helps to increase the tensile strength significantly. For example for a nylon fibre content of 1%, the tensile strength increases from 9.34 to 43.09 kPa- an increase of 361%. For 1% polypropylene fibre content, the increase is 211% and for 0.8% fibre mesh content the tensile strength increass by 421%. These phenomenal increases will definitely delay the formation of cracks during desiccation, which in turn ensures better performance of the clay liner under adverse seasonal changes.

A comparison of the tensile strengths of BES and SMC again asserts the superiority of the latter. A fibre content of 1% of nylon, polypropylene and fibre mesh increases the tensile strength by 25%, 144% and 149% respectively in bentonite enhanced sand. In sundried marine clay, the corresponding values are 361%, 167%, and 426%. Obviously, sundried marine clay will show a lesser tendency to crack, when subjected to desiccation.

6.4.3.3 Desiccation Tests on Sundried Marine Clay Sample

It has been convincingly established that sundried marine clay is superior to bentonite enhanced sand. A few desiccation tests were coducted on SMC for a comparative study of different percentages of fibre contents. Comparing samples amended with different fibre contents, it could be shown that the increase in fibre content reduced the cracks, fully endorsing the earlier findings. With another series of desiccation tests, it could be shown that polypropylene fibre mesh was the most effective out of three reinforcements tried, as shown by the value of crack intensity factor.

Thus the present study has succeeded in bringing out the serious limitations and disadvantages of a liner material i.e. bentonite enhanced sand, which is at present the most accepted material for compacted clay liner. In place of this, it has been shown that a new material viz. sundried marine clay, is far superior and has several advantages over the existing practice, especially when the possibility for desiccation is also taken into consideration.

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