HYDRAULIC AND CONTAMINANT TRANSPORT PERFORMANCE OF COMPACTED LATERITIC SOIL– BENTONITE MIXTURES PROPOSED AS WASTE CONTAINMENT BARRIER

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APPROVAL/CERTIFICATION

This dissertation "HYDRAULIC AND CONTAMINANT TRANSPORT PERFORMANCE OF COMPACTED LATERITIC SOIL – BENTONITE MIXTURES PROPOSED AS WASTE CONTAINMENT BARRIER" by AMADI, Agapitus Ahamefule, has been read and approved as having satisfied the conditions for the award of Ph.D. degree of the Department of Civil Engineering, Faculty of Engineering, Ahmadu Bello University, Zaria, Nigeria.

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DECLARATION

I declare that this report entitled **Hydraulic and Contaminant Transport Performance of Compacted Lateritic Soil – Bentonite Mixtures proposed as Waste Containment Barrier** is the result of my research study. All sources of information are duly acknowledged by means of references.

DEDICATION

This work is dedicated to my little baby, Miss Amarachi, Chika Amadi whose birth brought renewed inspiration and motivation to complete the project.

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ABSTRACT

Contamination of the geoenvironment due to unsafe waste disposal practices and leakage from waste repositories is a well-known and widespread problem. Emphasis has therefore been put on the hydraulic properties and contaminant transport capability of materials proposed for use as barriers in such facilities. Consistent with the current approaches for evaluating clayey soil liners, data on geotechnical, hydraulic and contaminant transport performance in addition to other relevant engineering properties of lateritic soil associated with bentonite treatment were developed and analyzed. Evaluation of lateritic soil - bentonite mixtures with bentonite content up to 10% has been carried out with respect to hydraulic conductivity, unconfined compression strength, volumetric shrinkage, unsaturated flow behaviour, compatibility with the municipal solid waste MSW leachate at the various compaction states (dry, wet and optimum water content) and compactive efforts (reduced British Standard Light, British Standard Light, West African Standard or 'intermediate' and British Standard Heavy). The diffusion properties of inorganic chemical species in MSW leachate through lateritic soil – bentonite mixtures has been characterized in terms of effective diffusion coefficients. Results indicate that unconfined compressive strength together with hydraulic conductivity of compacted samples of the mixtures decreased with increase in the amount of bentonite while volumetric shrinkage strain increased with increase in bentonite content for all the efforts. Acceptable ranges of compaction water content with the corresponding dry unit weight that will provide best performance with respect to the design parameters were delineated in an envelope on the moisture – density plane for the various soil - bentonite mixtures. At the various dosage levels of bentonite, the unsaturated hydraulic conductivity computed using the close form solutions of van Genuchten – Mualem (VGM) and Brooks and Corey - Burdine (BCB) models was essentially the same for specimens prepared either dry or wet of optimum water content. Hydraulic conductivity tests carried out to evaluate compatibility of specimens with MSW leachate showed increase in

values by a factor of 1.2 to 1.7 at the end of test duration. These increases from a practical standpoint did not jeopardize the use of the mixtures since hydraulic conductivity at the end of the test was still below 1.0×10^{-9} m/s for mixtures that hitherto met this threshold value. Adsorption of metals followed the expected trend of increased metal adsorption characteristics with higher bentonite content. For the range of conditions examined, K cations were found to diffuse faster than the other two cations. The effective diffusion coefficients of the cations were found to be in the order of $K^+ > Na^+ > Pb^{2+}$ and diffusion was seen to decrease as the bentonite content increased for all the cations.

Evaluation of the influence of model parameters on the reliability associated with a laboratory based hydraulic conductivity model (a dominant failure mode for clay liner materials) for lateritic soil bentonite mixtures showed that reliability index decreased with increasing coefficient of variation of initial saturation, plasticity index and clay content. Both normal and lognormal probability distribution functions of hydraulic conductivity yielded approximately similar trends in reliability – coefficient of variation relationships. The change in reliability levels for variability in these parameters suggests that effective quality control measures must be put in place during construction in order to achieve the specified values for these variables.

The result of this study highlights the capability of compacted lateritic soil – bentonite mixtures to function effectively both as hydraulic and contaminant barrier when used alone or in a composite construction in waste containment applications. It is expected to serve as a valuable tool for designers, researchers and regulators as information provided in this work may be helpful for a better understanding of soil materials of similar physical and engineering characteristics to be used as hydraulic barriers.

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SYMBOLS AND NOTATIONS

A	Cross sectional area of soil
AAS	Atomic Absorption Spectrometer
AASHTO	American Association of State Highway and Transportation Official.
ASTM	American Society of Testing and Materials
BCB	Brooks-Corey-Burdine function
BOD	Biological Oxygen Demand
BSH	British Standard Heavy
BSL	British Standard Light
c	Mass concentration of solute based on volume of solution in soil
c_o	Initial mass concentration of solute in source solution
CEC	Cation Exchange Capacity
COD	Chemical Oxygen Demand
COV	Coefficient of variation
D*	Effective diffusion coefficient of solute
DDL	Double Diffuse Later
D_h	Hydrodynamic dispersion coefficient
D_m	Mechanical dispersion coefficient
D_o	Free solution diffusion coefficient
DTA	Differential Thermal Analysis
DUW	Dry Unit Weight
E	Compactive Energy
E_h	Redox Potential
FORM	First order reliability method
GCL	Geosynthetic Clay Liner
J	Mass flux vector
J_D	Diffusion mass flux
K	Hydraulic conductivity
K_d	Distribution coefficient
K_p	Partition coefficient
K_r	Relative hydraulic conductivity
K_{sat}	Saturated hydraulic conductivity
LL	Liquid Limit
Meq	Milliequivalent
MSW	Municipal Solid Waste
MWC	Moulding Water Content
n	Soil porosity
n_e	Effective soil porosity
NGSRB	Nigerian General Specifications for Roads and Bridges
OMC	Optimum Moisture Content
P_f	Probability of failure
pH	Positive Hydrogen ions
PI	Plasticity Index
PL	Plastic Limit
PPM	Part Per Million
q_m	non zero positive root

R^2	Coefficient of determination
R_d	Retardation factor
RBSL	Reduced British Standard Light
SB	Soil Bentonite
SD	Standard deviation
Si	Initial Degree of Saturation
SWCC	Soil Water Characteristic Curve
TOC	Total Oxygen Content
UCS	Unconfined Compressive Strength
v_s	Seepage velocity
VGM	van Genuchten – Mulam function
VOC	Volatile Organic Compound
WAS	West African Standard Compaction
W_{opt}	Optimum water content
ZAV	Zero Air Void
α	Fitting parameter in van Genuchten function
m	Fitting parameter in van Genuchten function
γ_d	Dry unit weight
θ	Volumetric water content
θ_m	Maximum volumetric water content
θ_r	Residual volumetric water content
θ_s	Saturated volumetric water content
λ	Pore size distribution factor in Brooks-Corey function
Ψ	Matric suction
Ψ_a	Air entry pressure
τ	Tortousity factor

CHAPTER ONE

INTRODUCTION

1.1 General

Rapid industrialization and unsafe waste management practices in developing countries like Nigeria have resulted in complex and challenging problems in the surface and subsurface environment. Uncontrolled dumps which are the dominant route for solid waste disposal in these countries (examples in Plates 1A,B,C and D), constitute a major threat to groundwater resources which serve as a primary source of drinking water for many communities. This is of major concern because these sites receive a wide range of wastes possibly including toxic compounds. Invariably, leachate from such sites contain a wide range of organic compounds some of which are toxic at low concentrations, in addition to inorganic compounds, heavy metals, pesticides etc. Once in soil environment, the leachate will migrate and contaminate the geoenvironment and impair the use of groundwater as the natural (indigenous) barrier at these sites cannot be relied on (Amadi, 2007).

Increased waste generation in our cities is an inevitable repercussion of human activities arising from industrialization. This is more worrisome now that waste composition is changing from vegetable based refuse to manufactured/synthetic materials resulting from new packaging techniques and consumption pattern. These wastes cannot be ignored, cities have to accept responsibility for the waste arising from within them by properly and safely disposing them in acceptable engineered structures in order to protect not only the surface but also the geoenvironment.

Engineered landfill is one environmental option that has been found to work in the industrialized countries of Europe and America. Landfill, accounts for over 75% of solid

waste disposal in United state of America, 65% in Canada, 82% in United Kingdom, etc (William, 1997; Edil, 2003) It is an engineering project whose objective is to contain the waste in a manner that is protective of human health and the environment. It requires highly sophisticated technology employing engineering methods in which the waste is deposited on excavated and prepared cells, compacted to the smallest practical volume and covered with a layer of soil at the end of each day's operation (Daniel, 1993a; Cernica, 1995).

Research has shown that the impact of a waste disposal facility on groundwater quality will depend principally on the nature of barrier which is intended to minimize and control contaminant migration (Rowe *et al*., 1995). The primary objective of engineered components in containment facilities is to decelerate contaminant migration into the underlying subsurface during both the active disposal period and the post closure period. Barrier system is therefore one of the most important elements of a modern engineered landfill. The technology has evolved significantly in the last decades in countries where landfill has been in use. The potential materials are compacted clay, geomembrane, geosynthetic clay liners (GCLs), recycled or reused waste. The first consideration is always a suitable natural clayey soil when such a soil is readily or economically available. Compacted soil is widely used to line landfills and waste impoundments, to cap new waste disposal units and to close old ones (Daniel and Benson, 1990).

Due to their wide occurrence, lateritic soils may be considered for use as barrier in waste disposal facilities. Laboratory tests have shown that when lateritic soils is used alone or stabilized with fly ash or foundry slag as barrier material it proved favourable in terms of chemical resistance, better workability, less susceptibility to desiccation induced shrinkage, shear strength, hydraulic conductivity, compatibility and diffusion parameters (Amadi, 2003; Eberemu, 2003; Nwaiwu, 2004). However, further research in the improvement of hydraulic

performance may be justified bearing in mind that the predominant clay mineral is the inert and non - expanding kaolinite often mixed with quartz.

1.2 Statement of Problem

Notwithstanding the widespread use of natural soil materials in barrier systems (i.e. liners and covers), questions and issues persist relative to their use. Their reliability has been a contentious issue following a number of failures (eg. Goldman *et al.*, 1986; Daniel and Brown, 1988). This has led to great suspicion and to regulations by some agencies involving the use of composite barriers made of compacted fine-grained soil overlain by geosynthetics and recently geosynthetic clay liners (GCLs). These may be due to increase in hydraulic conductivity, k or low resistant to changes in hydraulic conductivity, k (Rowe *et al*., 1995).

Lateritic soil, a residual soil formed under the tropical and subtropical climatic whose predominant clay mineral, kaolinite is generally inactive, non – expanding and suffers from the problem of obtaining low hydraulic conductivity values. This soil is also distinguished from other soils by the presence of high proportion of sesquioxides of iron (Fe_2O_3) and aluminum (Al_2O_3) relative to other chemical components. The sesquoxide gel within the fine fraction of the soil tends to coat the surface of individual soil particles. This coating reduces the surface activity of the clay mineral and by extension the ability of the clay mineral to absorb water. It also causes a physical cementation of adjacent grains that result in the aggregation of particles to form coarser particles. Both factors reduce plasticity (Maigien, 1966; Townsend *et al.*, 1971) and invariably the ability of the soil to achieve low hydraulic conductivity. The aforementioned properties give an indication of their limitations when lateritic soils are to be used in hydraulic containment applications and are of concern to the geoenvironmental engineer. Modification with bentonite that is widely used in containment application will ameliorate to a large extent these deficiencies to ensure better hydraulic

performance while utilizing the relatively high attenuative potential of the soil (D'Appolonia, 1980; Ryan, 1987; Huang and Wong, 1992; Kenny *et al.*, 1992; Daniel, 1993; Koerner, 1994; O'Sadnick *et al.*, 1995; Evans, 1993; Osinubi and Amadi, 2003; Osinubi and Nwaiwu, 2005).

1.3 Aim

To evaluate by laboratory testing and numerical analysis the performance of lateritic soil - bentonite mixtures with respect to geotechnical, hydraulic and contaminant transport properties of compacted mixtures proposed for use as barrier material in waste containment facilities and to develop recommendations for further research in improving the performance of the material as liner systems.

1.4 Objectives:

The specific objectives of this study are itemized as follows:

(1) To evaluate and develop data on the hydraulic conductivity, k, volumetric shrinkage, and shear strength of compacted lateritic soil – bentonite mixtures.

(2) To assess improvement in unsaturated flow behaviour through compacted lateritic soil treated with bentonite.

(3) To investigate the sorption/attenuative capabilities and chemical compatibility characteristics of lateritic soil bentonite mixtures with MSW leachate.

(4) To investigate the various diffusion parameters of some municipal waste contaminants in compacted mixtures.

(5) To predict using a reliability approach, the performance of lateritic soil – bentonite mixtures from a regression-based model developed from laboratory hydraulic conductivity data.

1.5 Justification

Given the apparent increasing use of compacted fine-grained soils as barriers, which primarily are based on consideration of availability of material and cost and the favourable results obtained by Osinubi and Nwaiwu (2004) on the use of laterite soil as hydraulic barrier material, considering methods of achieving a greater level of hydraulic performance with this locally abundant soil will be a worthwhile engineering research effort.

Secondly, most barrier technologies presently in use were developed in the temperate regions, adopting such technologies for application in the tropics may not be practical and cost effective as there are significant differences in geological, environmental and material properties in the two regions. In order to develop or reliably adopt suitable and cost effective liner technology for tropical environment, geoenvironmental assessment of tropical soils such as lateritic soils and stabilized mixtures has to be undertaken.

Another source of concern is the apparent increase in construction cost arising from the use of geosynthetic polymers. This consideration suggests that evaluation of alternative materials merit scrutiny.

Lastly, most modern solid waste containment facilities employ a composite liner system consisting of a geomembrane or geosynthetic clay liners overlying a compacted clay liner. This is because geosynthetic clay liners and geomembranes when used alone suffer from high diffusion flux and as such do little to inhibit the transport of volatile organic compounds (VOCs), which diffuse readily through geomembrane polymers (Rowe *et al*., 1995; Edil, 2003). Furthermore, a number of house hold chemicals such as margarine, vinegar, ethyl alcohol, shoe polish, peppermint oil, etc commonly found in municipal solid waste degrade

the geomembranes and geosynthetics, making it soft, brittle and susceptible to cracking and eventual strength lost.

Given that the clay component usually provides a diffusion barrier that controls the rate of transport of the VOCs, success in partly replacing the costly composite system with readily, and economically available lateritic soil – bentonite system will provide a back up for synthetic barriers hence a cost effective means of gaining the environmental protection required.

While bentonite controls soils hydraulic conductivity by manipulating exchangeable ions, lateritic soils on the other hand on the other hand provides greater chemical resistance, better workability, less susceptibity to desiccation induced cracking in addition to high sorption/attenuative propensity. These inherent properties of lateritic soils are due to the presence of organic matters, organic carbon content, oxides and hydroxides and thus have great potentials for good compatibility characteristics (Mitchell and Jaber, 1990; Rouse and Pyrih, 1993; Shackelford, 1994; Shackelford and Jefferis, 2000; Tipping, 2002).

1.6 Dissertation outline

Chapter two of this dissertation presents the literature review on solid waste management/waste management technologies, soil contamination, lateritic soil, bentonite and its uses in geotechnical/geoenvironmental applications as well as leachate generation and characteristics.

The theoretical and general background information on containment system design, geotechnical design parameters for liners and covers, unsaturated flow behaviour in compacted soil, contaminant transport processes and reliability as a probability concept are reviewed.

Chapter four presents the experimental programme, the materials and the test methods for the various experiments adopted to achieve the stated objectives.



A B



C D

Plates 1A, B, C and D. Official refuse disposal sites in some Nigeria cities (Amadi, 2007).

Results of the physical/characterization tests as well as the geotechnical design parameters and the various factors affecting the engineering behaviour of these parameters were discussed with graphical illustrations. Long term hydraulic conductivity tests for evaluating the chemical compatibility, sorption performance of the various mixtures, diffusion test results as well as the reliability estimates are presented in chapter five.

Chapter six summarized the research findings and provides conclusions as well as recommendations for further work.

CHAPTER TWO

LITERATURE REVIEW

2.1 Elements of Solid Waste Management

The problems associated with the management of solid wastes in today's society are complex. The activities involved from the point of generation to final disposal are grouped into six functional elements:

2.1.1 Waste generation

Waste generation encompasses those activities in which materials are identified as no longer being of value and are either thrown away or gathered together for disposal (Oweis and Khera, 1998; Cheremisinoff, 2002). Factors influencing the quantity of municipal waste generation include: geographic location, season of the year, extent of salvaging and recycling, changing life style, changing consumer habits, collection systems and legislation. The amount of waste generated and the types of hazardous materials present in the waste stream increase with increasing industrialization of a country. In many cases, generation data are either not available or not reliable. In some part of the world, a significant proportion of the waste in rural areas is burnt in backyards. This practice while now terminated in many developed countries is still in use in some developing countries. Other materials are dumped on waste ground. Collected waste, therefore only represents a proportion of generated waste and varies widely between the different municipalities, towns and even villages within the same district (NEST, 1991; Oyeshola, 1995; Ekenta, 2001; Ogunbiyi, 2001). The annual production of domestic solid waste in Nigerian cities and towns exceed 22 million tons, approximately 0.5kg per person per day (Ogunbiyi, 2001; Amadi, 2007).

2.1.2 Waste categories

Waste may take the form of solids, sludges, liquids, gases or any combination thereof. It is generally accepted that the total waste stream generated within a community comprises three major categories, namely:

- Municipal waste
- Industrial Waste
- Hazardous Waste

2.1.2.1 Municipal waste

Municipal waste can generally be categorized as all waste under the control of local authority or agents acting on its behalf. Typically, it consists of heterogeneous mixtures of non hazardous wastes that are primarily of residential and commercial origin such as petrescible wastes, garden wastes, and uncontaminated biosolids as well as chemicals and related wastes. It is recognized that municipal waste is likely to contain a small proportion of hazardous waste (e.g. cleaners, automotive products, paints, e.t.c) from households and small commercial premises that standard waste screening procedures will not detect (Oweis and Khera, 1998). However, this quantity should not generally exceed 200ml/tonne or 200g/tonne.

2.1.2.2 Industrial waste

Industrial process wastes include a very wide range of materials and are produced in industries such as chemical manufacturing and processing, food processing, iron and steel manufacturing, petroleum refining and the manufacturing of plastics and resins, paper and pulp e.t.c. Most of these wastes contain somewhat higher levels of contaminants such as heavy metals, organic and inorganic chemicals than municipal wastes and needs to be managed with environmental controls appropriate to the specific wastes being land filled

(Oweis and Khera, 1998). Examples found under this heading include general factory rubbish, organic wastes from food processing industries, acids, alkalis and metallic sludges.

2.1.2.3 Hazardous waste

Industrial processes are the major sources of hazardous wastes. These wastes possess a present or future threat to humans or the environment as a result of one or more of the following characteristics:

- Explosiveness
- Flammability
- Capacity to oxidize
- Corrosiveness
- Toxicity and/or
- Eco-toxicity

Hazardous waste are generated in nuclear power plants, mining sites, metal smelting, chemical manufacturing, oil production etc and contain contaminants such as heavy metals and human made chemicals at levels high enough to require treatment to render them safe before landfill disposal (Tchobanoglous, 1977; Krieger *et al.*, 2000).

2.1.3 On site handling, storage and processing

On site handling refers to the associated activities prior to the storage of waste in containers for collection. The heterogeneous waste generated in residential areas must be removed within some days due to the fact that storage space is usually limited and that such wastes are partly biodegradable. Factors that must be considered in the on site storage of solid waste include (Robinson, 1986; Cheremisinoff, 2002; Kreith and Tchobanoglous, 2002):

- the type of container to be used
- the container location

- public health and aesthetic consideration
- the collection method to be used.

2.1.4 Collection

Collection as a functional element includes not only the gathering of solid wastes, but also the hauling of wastes after collection to the location where the collection vehicle is emptied. This location may be a transfer station, a processing plant or a landfill disposal site. Collection of solid wastes is one of the most costly functional elements and refers to the types of collections services, the types of collection systems and the collection routes (Tchobanoglous, 1977; Headly, 1994; Krieger *et al*., 2000). Based on their mode of operation, collection systems are classified into two categories namely:

- hauled container systems and
- stationary container systems.

2.1.5 Transfer and transport

This involves two steps:

- transfer of wastes from smaller collection vehicles to large transport equipment and
- the subsequent transport of the wastes, usually over long distances to the disposal site.

The transfer usually takes place at a transfer station. Depending on the method used to load the transport vehicle, transfer stations may be classified into three types (1) direct discharge, (2) Storage discharge and (3) Combined direct and storage discharge.

In a direct discharge transfer station, the wastes from the collection vehicles are usually emptied directly into the vehicle used to transport them to the place of final disposal. These are usually constructed in a two level arrangement and employ stationary compactors.

In a storage discharge transfer station, wastes are emptied either into a pit storage or onto a platform from which they are loaded into the transport vehicles by various types of auxiliary

equipment. Some transfer stations use both direct discharge and storage discharge methods as multipurpose facilities designed to serve a broader range of users.

2.1.6 Processing and recovery

Processing and recovery include the range of activities, techniques, equipment and facilities used both to improve the efficiency of solid waste disposal systems and the recovery of usable materials, conversion of products or energy from solid wastes. Important processing techniques used routinely in municipal solid waste management systems include compaction, thermal volume reduction (incineration) and manual separation of waste components. In the recovery of materials, separation operations for example, have been devised to recover valuable resources from the mixed solid wastes delivered to transfer stations or solid waste processing plants. These operations include size reduction and density separation by air classifiers, magnetic devices for iron, separation for aluminum and screens for glass. In the selection of a recovery processes, the cost of operation must be weighed up against the value of the recovered materials or products.

2.1.7 Waste minimization/source reduction

During the past few years, intense efforts have been made to prevent or at least minimize generation of wastes through the use of benign materials and innovative manufacturing processes (Reddi *et al.*, 2000). Waste minimization is an increasingly essential element of solid waste management. Not only does it make good environmental sense, it makes good economic and business sense. It is aimed at reducing the production of waste through education and improved production process rather than aiming to increase the technology to improve treatment of waste (Yong, 2000). Minimizing waste generation has the potential to reduce costs or increase profits by maximizing the use of resources and by reducing the

amount of waste to be disposed hence the cost of waste management is also decreased (Robinson, 1986; Cheremisinoff, 2002; Kreith and Tchobanoglous, 2002).

2.1.8 Waste containment

Wastes are created that require disposal despite the best waste management practices. Such wastes include household garbage, mine refuse, highly toxic industrial byproducts and nuclear wastes. Proper disposal of these wastes in engineered waste containment facilities is crucial to protect public health and environment. Containment facilities for liquid wastes are known as surface impoundments or more commonly, lagoons or ponds. These impoundments have to be lined properly at the bottom to prevent infiltration into the subsurface of their chemical constituents. In the past, because regulations on such linings did not exist, linings were not provided, resulting in several contaminated sites.

Containment facilities for solid wastes are known as landfills. Newly generated liquid and solid wastes are required to be disposed of in engineered impoundments and landfills. All these containment facilities require liner systems that perform as both hydraulic and chemical barriers. In addition, these facilities must be located where hydrogeological conditions are favourable (Rowe *et al*, 1995; Daniel, 1993a). Upon reaching their waste storage capacity, the containment facilities should be covered properly to isolate the waste and to prevent infiltration of precipitation. The mechanical stability of liners and covers should also be insured.

2.2 Solid Waste Management Technologies

2.2.1 Recycling

Waste recycling is the breaking down of materials from waste streams into raw materials, which are then reprocessed either into the same material or a new product (Robinson, 1986;

Cheremisinoff, 2002). Recycling efforts have been increased to divert wastes from disposal in containment facilities. Similar to source reduction in its impact on reducing the waste stream, recycling involves recovery of materials that have already been discarded or are unusable in their current form. Recyclable materials include a wide range of household and manufacturing materials that are no longer usable such as papers, plastic, glass, tires, metals e.t.c. Recycling efforts have been fairly successful and large quantities of materials have been collected.

2.2.2 Incineration

This is the basic method of reducing municipal solid wastes to a minimum volume for disposal in landfills. The bulk volume reduction by incineration is about 90% (Heeramum, 1993; Kreith and Tchobanoglous, 2002). The end products of incineration are: the cleaned gases that are discharged to the stake, ashes and unburnt materials that are conveyed to the ash pit for final disposal in a landfill while the excess heat generated by the combustion process can be recovered as steam for direct use or can be converted into electricity and marketed. Problems associated with incinerators concern mostly emission of dioxin and furan (Heeramum, 1993). Incinerators therefore require sophisticated air pollution control equipment.

Capital expenditure and maintenance costs are high, which must be a major consideration in justifying the choice of the incineration method. Incineration ash may be frequently contaminated with the residues of the incinerated waste or may contain new contaminants formed during the process and must therefore handled with care.

2.2.3 Composting

This is the process involving the separation and biological decomposition of the organic constituents of solid wastes under controlled conditions. Most composting operations involve the following three basic steps (Robinson, 1986; Cheremisinoff, 2002; Kreith and Tchobanoglous, 2002):

- Preparation of the solid wastes: This involves receiving, storing, separation, size reduction and addition of moisture and, if possible, nutrients.
- Decomposition of the solid wastes: Several techniques have been developed. In the windrow system, the prepared solid wastes are placed in elongated piles, called windrows, in an open field. Aeration is accomplished by periodically turning the piles in such a manner that all particles are exposed to comparable conditions at some time during the active period of the composting process.
- Preparation of the end product for marketing. This step may include fine grinding, blending with various additives, granulation, bagging, storage and in some cases, direct marketing.

Maintaining strict hygienic conditions can eliminate odour problems – usually such plants have to stop work for one day at regular intervals especially for cleaning purposes.

2.2.4 Landfilling

Although source reduction, reuse, recycling and composting can divert large portions of municipal solid waste for disposal, some waste still must be placed in landfills.

Modern landfills are well engineered facilities for the disposal of municipal solid waste designed and operated to minimize public health and environmental impacts. Waste landfills must be designed to protect the environment from contaminants which may be present in the waste stream. The landfill-siting plan which prevents the siting of landfills in environmentally sensitive areas provide additional safeguards. The typical landfill consists of

several cells in which the waste is systematically placed. The landfill base consists of a liner that minimizes the leakage of liquid waste materials from the landfill into the groundwater system. As the waste is built up in layers, it is covered daily by a thin layer of soil to prevent paper, dust or odour from escaping into the environment. The entire system is a sophisticated facility that requires huge sums of money to build and the services it provides can hardly be quantified.

2.2.4.1 Municipal solid waste landfill

Often referred to as sanitary landfill, municipal solid waste landfill is used for the disposal of a combination of commercial and residential waste generated within urban, suburban and rural areas. It can also receive non-hazardous sludge, industrial solid waste as well as construction and demolition debris. Some common household items such as paints, cleaners/chemicals, motor oil, batteries and pesticides may be banned from disposal in municipal solid waste landfills. Leftover portions of these products are called household hazardous waste. These products, if mishandled, can be dangerous to health and the environment.

2.2.4.2 Bioreactor landfills

These are municipal solid waste landfills that are designed to quickly transform and degrade organic waste. The increase in waste degradation and stabilization is accomplished through the addition of liquid and in some cases, air to enhance microbial processes. Bioreactors are a new approach to landfill design and operation that differ from the traditional “dry tomb” municipal landfill approach (Benson, 1999).

2.2.4.3 Industrial waste landfills

These landfills are designed to accept predominantly industrial waste. In many cases industrial waste landfills are monofills, associated with a specific industry or industrial location (eg mining, forestry, smelting, etc) and are designed and operated in accordance with the specific waste targets. Design, operation and monitoring requirement may be more stringent than for municipal solid waste landfills. An industrial waste landfill may also receive municipal solid waste and inert waste depending on the design.

2.2.4.4 Hazardous waste landfills

This type of landfill accepts wastes formally defined as “hazardous waste” in statutory instruments or as specifically determined through any special requirements that may be set by the regulatory agency. Sitting, design, operation and monitoring requirements for landfills accepting hazardous waste will be considerably more stringent than for landfills accepting only municipal wastes.

2.2.4.5 Selection of landfills sites

A landfill site has to meet several locational and geotechnical design criteria and be acceptable to the public. In selecting a location, the first concern should be to protect humans and the environment from the adverse effects of toxic and hazardous waste exposure. The area selected for the facility should be a remote area and/or one with a low population density. The site considerations should also include where the hydrogeology minimizes possible migration and where the slope of the site itself restricts possible migration of leachate or other waste residues. Public health and welfare, water quality standards and land use plans should all be evaluated.

Both landfill operating cost and hauling cost are taken into consideration while developing the optimal service region. Since acceptability to the public is crucial to the landfill siting process, the citizens affected (host community) should be informed regarding the site selection process as early as possible. Locational criteria dictated by the regulatory body must also be studied.

Usually a landfill cannot be sited within a certain distance of the following: Lakes, ponds, rivers, wetlands, floodplains, highway, critical habitat areas, water supply well, active fault areas and airports. In addition, landfill siting is not allowed in areas where a potential for contamination of groundwater or surface water bodies exists. Landfill siting should be avoided in unstable areas and seismic impact zones (Robinson, 1986; Tchobanoglous, 1997; Krieger *et al.*, 2000; Cheremisinoff, 2002; Kreith and Tchobanoglous, 2002).

2.2.4.6 Final land use consideration

As property values continue to increase in densely populated areas, constructing structures over sanitary landfills becomes more tempting. Some of the major problems that must be taken into account when considering the use of a completed landfill sites are: fill settlement, allowable foundation loads, corrosive nature of fill materials, explosive hazards associated with landfill gas and excavation of the decomposing refuse in the fill. Most of the natural settling occurs within five years after the refuse is placed. Additional settling may result from activities associated with the final use of the landfill.

The most appropriate uses for completed landfill sites are open space and recreational areas. Parks, playgrounds, golfcourses etc are deal for installation on these sites. In general, land use planning must be compatible with any applicable zoning regulations. It may be true that the

ultimate use of the site will not be realized for many years, but it may be a selling point that can be used to gain local support.

2.3 Cost Components in Solid Waste Disposal Systems

Cost components in a solid waste disposal system are divided into:

2.3.1 Fixed costs

Fixed costs are those upfront costs associated with the purchase and installation of the system being constructed. These are

- Onsite construction cost: These include the following cost categories
    - equipment
    - foundation (knowledge of soil and foundation characteristics of the site).
    - Buildings
    - Utilities (e.g development required for the distribution of electricity and water for the provision of storm drainage and sewage for a treatment system for sanitary sewage and process waste water) and
    - Site development costs.
- Offsite construction costs: Providing for the necessary roadway and utilities.
- Land Acquisition cost: Land costs are both system and site dependent. A sanitary landfill may require several hectares.
- Professional services: Several professional services are required to assemble a project of any size e.g construction plan and specifications and construction supervision.
- Financing costs: This category includes cost of selling bonds as well as funds needed to meet the financial commitments until a revenue stream is developed.

2.3.2 Recurring costs (operation and maintenance costs)

These are variable costs that depend on the way in which the facility operates. Many of these costs are in direct proportion to the quantity of material processed. They include the following categories:

- Labour cost – the operating and maintenance personnel are specific for each facility.
- Utilities – Utility costs for the needs of the employees e.g energy cost
- Supplies – a variety of operating supplies ranging from process chemicals and fuel for vehicles collecting wastes.
- Administrative cost – personnel including wages and fringe benefits, insurance, legal and fiscal costs, taxes includes property tax and income tax.

2.4 Liquid Waste Management

Liquid waste is wastewater generated by individual households, shops and institutions like hospitals, industries and other establishments. Sewage and industrial waste form the principal components of liquid wastes. Most times, many companies simply discharge liquid wastes from washing processes such as collieries, metal manufacturing and processing, formation water produced during onshore exploration and exploitation of oil and gas, refinery effluent and sludge, etc untreated into surface impoundments (evaporation lagoons) or rivers. The use of these impoundments for liquid wastes allow for significant accumulations of sediments and sludges which contain hazardous materials (Tchobanoglous, 1994).

Generally, regulatory agencies in most countries specify minimum requirements for treated effluents to be discharged into streams and rivers. Typical effluent stream requirements for treated effluent from petroleum refinery are, 15ppm oil, with phenols < 0.2ppm and Sulphides < 0.5ppm (Overcash and Pal, 1979). In Nigeria, produced water onshore and

offshore during oil exploration and exploitation are treated to meet Department of Petroleum Resources' 20ppm and 48 ppm oil water respectively before discharge.

2.5 Relevant Environmental Laws and Regulation

Since the beginning of the 1980s waste disposal has become a particularly sensitive issue. This has led to the development of new legislation in both the European community, America and indeed internationally which resulted in an array of regulations concerning landfill liner design. The Federal government of Nigeria has promulgated various laws and regulations to safeguard the environment. One of such laws is the Federal Environmental Protection Act of 1988 (FEPA). The following regulations were made pursuant to the FEPA act:

(1) National Environmental protection (Effluent Limitation) regulations:

   i. National Environmental protection (pollution abatement in industries and facilities generating wastes) regulations and

   ii. National Environmental protection (Management of solid and hazardous wastes) regulations. This provides that the collection, treatment, transportation and final disposal of wastes shall be the responsibility of the industry or facility generating the waste.

(2) Environmental Impact Assessment Act of 1992 (EIA Act).

(3) Harmful wastes (Special criminal provisions etc) act of 1988 (Harmful wastes act)

The Federal Ministry of Environment administers and enforces environmental laws in Nigeria. FEPA was absorbed and its functions taken over by Federal Ministry of Environment in 1999.

2.6 Soil Contamination

There is now an international recognition of the problems of environmental damage, especially to soil. Past and continuing abuses exert high cleanup tolls in terms of technical,

financial and social resources. Whether driven by government mandate, social responsibility, economies or other forces, remediation of contaminated sites is a necessary exercise in our present environmentally conscious society (Yong, 2000). Contamination has resulted from industrial activities such as mining and smelting of metalliferous ores, electroplating, gas exhaust, energy and oil exploration and exploitation, fertilizer and pesticide application and generation of municipal wastes.

Some of the major sources of contamination are indiscriminate, inadvertent and uncontrolled discharge of liquid wastes, poor management and control of chemicals and other processes, inadequate and improper liquid runoff collection systems, disposal and improper storage or containment of hazardous wastes, runoff from non point source pollution surfaces and spills and illicit dumping (Colten, 1990; Yong, 2000; Coduto, 2003; Yong and Thomas, 2004). Considering pollutants as those contaminants deemed to be threats to human health and the environment, it is important for us to be aware of the fate of these pollutants in the soil strata underlying the soil surface.

Usually, contaminants in the soil are physically or chemically attached to soil particles or become trapped in the small spaces between soil particles. The threats posed by contaminants in contaminated soil are normally considered in relation to their impact on the health of biotic receptors and the environment. Contamination of soil can lead to severe consequences such as loss of ecosystem and agricultural productivity, diminished food chain quality, tainted water resources, economic loss and human and animal illness (Doran and Parkin, 1994).

The degree of threat (risk) posed by pollutants traveling along the pathways, and the processes affecting the fate of pollutants on these pathways will be some of the many key factors that will determine the course of action required to minimize or eliminate the threat (Yong, 2000). Several options present themselves if one recognizes that total removal of

contaminants is not always required – so long as one can eliminate the health and environmental threats posed by these contaminants. These include (Rouse and Pyrih, 1993; Hansen, 1997):

- Immobilizing the contaminants so they stay fixed to the soil particles (soil solids) and cannot migrate or diffuse into the surroundings.
- Ensuring that the immobilized contaminants do not produced leachate harmful to the environment and the biotic receptors.
- Reducing the concentration, volume and mass of the contaminants.
- Reducing the toxicity level of the pollutants
- Eliminating the contaminant path in the source – receptor path chain of communication.

The net result of these actions will establish

(2) Avoidance of groundwater contamination

(3) Minimization of contaminant migration

(4) Elimination of interaction of contaminants (source) and biotic receptors

(5) Rehabilitation of the contaminated land to a level consistent with land use requirements.

2.6.1 Contaminants of major concern

2.6.1.1 Toxic inorganic contaminants

Heavy metals in sludges and soils arise from domestic, municipal, industrial and road run off input. The environmental problems with heavy metals is that they can be considered conservative because they are not decomposed in the environment (in contrast to organic matter that can be transformed by incineration or biodegradation and become less toxic over time) and that most of them have toxic effects on living organisms when the concentration exceeds a certain threshold value (Bruce, 1977). Furthermore, some heavy metals are subject to bioaccumulation and may pose a risk to human health when transferred to the food chain

(Amadi, 2007). However, in the soil environment many metals especially redox sensitive elements such as Arsenic and Chromium can undergo transformation or sorption reaction that alter both mobility and relative toxicity (Wild, 1993). These affect how the total metal content of a soil is distributed into fractions that can be found in the water phase - the mobile phase, or the solid phase - the immobile phase respectively. The processes that determine the speciation of a heavy metal in a soil environment are:

- precipitation and dissolution
- sorption and desorption
- complexation with organic compounds
- complexation with inorganic compounds

The processes that will control the speciation of any metal i.e that will dominate over the other processes, depends on the chemical characteristics of the metal and properties of the soil environment.

In most soil environment, sorption is the dominating speciation process and thus, the largest fraction of heavy metal in a soil is associated with the solid phase of that soil. Pollution problems arise when heavy metals are mobilized into the soil solution and taken up by plants or transported to the surface or ground waters. The properties of the soil are thus very important in the attenuation of heavy metals in the environment.

2.6.1.2 Radionuclides

The radionclides in soils that have caused concern have come from testing of nuclear weapons, accidents at nuclear power stations such as that which occurred at Chernobyl, Ukraine and poor waste storage and disposal. A radionuclide undergoes the same reaction in soil as the non radioactive isotopes. Its adsorption depends on the following: (Wild, 1993) (i) The properties of the ion in solution and especially whether it is positively or negatively

charged or uncharged and whether it forms complex ions in solution (ii) The amounts in soil of the oxides of iron, manganese and aluminum and of the different clay aluminosilicates and (iii) the amount and nature of organic matter that is present. Reducing conditions and pH affect adsorption properties of soil, and also affect the ionic form of some radionuclides in solution. The greater the retention of a radionuclide by soil, the less it is taken up by plants and the less it is washed into drainage water (Wild, 1993).

2.6.1.3 Toxic organic contaminants

Organic contaminants constitute an important threat to environmental quality of soil and groundwater and are composed primarily of carbon and hydrogen, they may also contain small amounts of chlorine, nitrogen, sulphur and phosphorous. There are thousands of organic contaminants sources and pollutants types, but the following list is illustrative:

1 Formation water produed when separating oil, gas and water during the exploration and exploitation of oil and gas.

2 Petroleum hydrocarbons from rupture of underground storage tank (benzene, ethylbenzene, toluene, xylene, alkanes, alkenes).

3 Spillage or leakage of solvents and dry cleaning agents (acetones, trichloroethylene, formaldehyde and perchloroethylene).

4 Percolation into soil from pesticides and herbicides – lindane, organchlorines, organophosphates e.t.c.

5 Leakage of transformers - the polychlorinated biphenyls (PCBs).

Current concerns are focused more on organic compounds that can be acutely toxic added inadvertently to soil which includes the polychlorinated and polybrominated biphenyls, polyhalogenated aromatic hydrocarbons, a group of chemicals used in paints, electrical transformers, insulators, e.t.c.

2.7 Lateritic Soils

The fundamental characteristic of these soils is the nature and constitution of the mineral soil mass. Generally they are surface formations in tropical and subtropical areas which are enriched in sesquioxides of Iron (Fe_2O_3) and or Aluminum (Al_2O_3) and develops by intensive and long lasting weathering of the underlying parent rock. This enrichment/concentration may be by residual accumulation or by solution movement or chemical precipitation (Maignien, 1966; Gidigasu, 1976; Charman, 1988; Fookes, 1997). In all cases, it is the result of secondary physico-chemical processes and not the normal primary process of sedimentation, metamorphism, volcanism or plutonism. The accumulated hydrated oxides are sufficiently concentrated to affect the character of the deposit in which they occur. They may be present alone in an unhardened soil, as a hardened layer or as a constituent such as concretionary nodules in a soil matrix or a cemented matrix enclosing other materials (Charman, 1988).

2.7.1 Formation of lateritic soils.

Lateritic soils are formed in hot, wet tropical regions with an annual rainfall between 750mm to 2000mm (usually in areas with a significant dry season) on a variety of different types of rocks with high iron content. The location on the earth, that characterize these conditions fall between latitude 35^oS and 35^oN (Maigien, 1966; Newill and Dowling, 1970; Gdigasu, 1975; Fookes, 1997).

Laterization involves physico-chemical alteration of primary rock forming minerals into materials rich in 1: 1 lattice clay minerals (kaolinite) and laterite constituents (Fe, Al, Ti, Mn). In the first place Ca, Mg, Na and K are released, leaving behind a siliceous framework consisting of silica tetrahedral and alumina octahedral. Silica which is soluble at all pH values, will be leached slowly while as alumina and ferri sesquioxides (Al_2O_3, Fe_2O_3 and

TiO_2) remain together with kaoline as the end product of clay weathering. The end result is a "reddish matrix" made from kaolinite, goethite and fragments of the pisolitic iron crust (Maignien, 1966; Gidigasu, 1976; Charman, 1988; Fookes, 1997).

Two aspects of the parent rock affect the formation of laterite. One is the availability of iron and aluminum minerals. These are more readily available in basic rocks. The other is the quartz content of the parent rock. Where quartz is a substantial component of the original rock, it may remain as quartz grains. Laterite profiles occur on flat slopes in the terrain where runoff is limited. On the level ground, where drainage is poor, expansive clay dominate at the expense of laterite. From the above, three major stages have been identified in the process as follows (Maignien, 1966; Gidigasu, 1976):

- **Decomposition**:- Physico-chemical breakdown of primary minerals and the release of constituent elements (SiO_2, Al_2O_3, Fe_2O_3, CaO, MgO, K_2O, Na_2O etc) which appear in simple ionic forms.
- **Laterization**:- Leaching under appropriate conditions of combined silica and bases and the relative accumulation or enrichment of oxides and hydroxides of sesquioxides (Fe_2O_3, Al_2O_3 and TiO_2). The soil conditions under which the various elements are rendered soluble and removed through leaching or combination with other substances depend mainly on the pH and Eh of the groundwater and the drainage conditions. The level to which the second stage is carried depends on the nature and the extent of the chemical weathering of the primary minerals. Under conditions of low chemical and soil forming activity, the physico-chemical weathering does not continue beyond the clay forming stage and tends to produce end products consisting of clay minerals predominantly represented by kaolinite and occasionally by hydrated or hydrous oxides of iron and Aluminum.

- **Desiccation or Dehydration**:- The partial or complete dehydration (sometimes involving hardening) of the sesquoxide rich material and secondary minerals. The dehydration of colloidal hydrated iron oxide involves loss of water and the concentration and crystallization of the amorphous iron colloids into dense crystals in the sequence; limonite, goethite, with haematite to hematite. Dehydration may be caused by climatic changes, upheaval of the land, or may also be by human activities for example by clearing of forests.

2.7.2 Global distribution of lateritic soils

Laterites and lateritic materials are widely distributed throughout the world but occur more frequently in the tropics and subtropics of Africa, Australia, India, South-east Asia and South America (Maignen, 1966). The global distribution of laterite and associated materials is broadly governed by world climatic zones. Lateritic formation requires conditions of temperature and rainfall similar to those of the humid tropical and subtropical zones. There are other factors peculiar to particular regions which govern the type of laterite which may be found, whether an indurated hardpan laterite, a nodular laterite or some other form. These factors include local variations in climate and the geology as well as the geomorphology associated with the weathering and soil development stages of landscape formation (Gidigasu, 1975).

2.7.3 Chemical and mineralogical composition

Clay mineralogical constitution of this soil is principally kaolinite often mixed with quartz.

The higher proportion of sesquioxdes of iron (Fe_2O_3) and aluminum (Al_2O_3) relative to other chemical components is a feature characteristic of all grades of lateritic soils. Those groups in which the iron oxide predominates are called ferruginous laterite soils and those in which alumina predominates - aluminous laterite soils. Iron is present usually as oxide minerals

notably haematite (Fe_2O_3) and also as hydrated oxide – goethite (FeOOH) or as limonite (an amorphous mixture of hydrated oxides which retain various amounts of water). Aluminum occurs as its hydrated oxides gibbsite ($Al_2O_3.3H_2O$) and/or boehmite ($Al_2O_3H_2O$). It is also contained within the lattice structure of kaolinite as an Aluminum silicate. Lateritic soils may contain significant amounts of manganese often identifiable as black nodules or concretions while titanium occur in limited quantities as titanium oxides. Zinc, chromium, nickel, cobolt, molebdenium, vanadium and other trace elements have been identified. Free silica is present as quartz inherited from the parent material. The predominant clay mineral is well-crystallized kaolinite.

2.7.4 Colour

Lateritic soils have characteristic reddish shades, which appear to be due to the various degrees of iron oxides – goethite and hematite, titanium and manganese hydration. The shades also reflect the degree of maturity. Generally, lateritic soils derive their colour from two sources.

- From organic matter: Black, brown, grey
- From mineral composition:
  (a) Iron: red, orange, yellow, brown, blue and green.
  (b) Calcium, Magnesium, Sodium and Potassium: White
  (c) Aluminum: White
  (d) Manganese: Black, Brown.

2.7.5 Geotechnical properties of lateritic soils.

Geotechnical characteristics and field performance of lateritic soils as well as their reaction to different stabilizing agents may be interpreted in the light of all or some of the following parameters.

- Genesis and pedological factors (parent material, climate, topography, vegetation, period of time in which the weathering processes have operated).
- Degree of weathering (decomposition, sesquoxide enrichment and clay size content, degree of leaching).
- Position in the topographic site and
- Depth of site in the profile (Gidigasu, 1976).

2.7.5.1 Particle size distribution

The particle size distribution of the soil may provide the following information- (1) A basis for identification and classification of the soil

(2) The compactibility characteristics

(3) Permeability

(4) Sweallability and

(5) A rough idea of deformation characteristics of the soil mass.

Texturally, lateritic soils are very variable and may contain all fraction sizes; boulders, cobbles, gravel, sand, silt and clay as well as concretionary rocks. Quartzitic gravels which are formed from the alteration of quartz rich parent rocks are generally well graded with 20% of silt and clay – size fraction. Concretionary laterites have a higher content of fines ranging between 35 - 40%. Foot slope concretionary laterite gravels are coarse and gap graded (less sand), compared to high level gravels (Gidigasu, 1976).

2.7.5.2 Plasticity characteristics of lateritic soils

The interaction of the soil particles at the microscale level is reflected in the consistency limits or Atterberg limits of the soil. Knowledge of the Atterberg limit may provide the following information.

- A basis for identification and classification of a given soil

- Texture
- Strength and compressibility characteristics
- Swell potential of the soil or the water holding capacity.

Plasticity may be affected by pretest preparation, degree of moulding and time of mixing, drying and rewetting and irreversible changes in plasticity on drying. Drying drives off adsorbed water which is not completely regained on rewetting (this is the case in both oven and air drying, Fookes, 1997). Soil which contain hydrated oxides of iron and aluminium may become less plastic on drying. This is partly because dehydration of sesquoxides creates a stronger bond between the particles which resists penetration by water. Studies on the relationship between the natural moisture content and liquid limit and plastic limit have shown that generally the natural moisture content is less than the plastic limit in normal lateritic soils (Vergas, 1953). However, the lateritic soils from higher rainfall areas may have moisture contents as high as the liquid limit (Ackroyd, 1971).

2.7.5.3 Compaction characteristics of lateritic soils

The compaction characteristics are determined by their grading characteristics and plasticity of fines. These in turn can be traced to genetic and pedological factors. A significant characteristic of lateritic soils is the influence of the strength of the concretionary coarse particles on compaction. Most lateritic soils contain a mixture of quartz and concretionary coarse particles, which may vary from very hard to very soft. The strength of these particles has major implications in terms of field and laboratory compaction results and their subsequent performance in service. Weak coarse fractions breakdown under load with a resulting increase in fines of the soil (Ackroyd, 1971). The degree to which the materials breakdown is related to the content of iron oxide and the degree of dehydration. The higher the iron oxide content and the more the degree of dehydration, the harder the concretionary particles become. Placement variables (moisture content, amount of compaction, and type of

compaction effort) also influence the compaction characteristics. Varying each of these placement variables has an effect on permeability, compressibility, swellability, strength and stress – strain characteristics (Lamb, 1958).

2.7.5.4 Strength properties of lateritic soils

Lateritic soils are weathered under conditions of high temperatures and humidity with well-defined alternating wet and dry seasons and continually leached by rainwater causing a tendency for deterioration of its strength characteristics. Shear strength characteristics of these soils have been found to depend significantly on the parent materials and the degree of weathering (ie degree of decomposition, laterization and desiccation) which is a function of the position of the sample in the soil profile and the compositional factors (Lohnes *et al*., 1971; Wallace, 1973). The higher the degree of laterization, the more favourable are the shear strength parameters (Baldovin, 1969). Furthermore, the structural elements in the soils are often a less stable coarse-grained aggregation of variable strength which may break down in performance (Maigien, 1966; Charman, 1988) in addition to their varying silt and clay content which often render them moisture sensitive. (Ola, 1978; Nicholson *et al*., 1993). The aforementioned properties give an indication of their engineering limitations that restrict its use and of such sites to minor engineering projects.

2.7.5.5 Permeability of lateritic soils

Lateritic soils are distinguished from other soils by the presence of high proportion of sesquioxides of iron (Fe_2O_3) and aluminum (Al_2O_3) relative to other chemical components. The sesquoxide gel within the fine fraction of the soil tends to coat the surface of individual soil particles. This coating reduces the surface activity of the clay mineral (principally kaolinite often mixed with quartz) and by extension the ability of the clay mineral to absorb water. It also causes a physical cementation of adjacent grains that result in the aggregation of

particles to form coarser particles and a reduction in surface activity (15 m^2/g) and ability of the clay particles to absorb water. This factor also contributes significantly in lowering the plasticity of the soil and the inability of the soil to achieve very low hydraulic conductivity. The Base Exchange capacity of the clay mineral is usually very low (~5 meq/100g). Both factors reduce plasticity (Maigien, 1966; Townsend *et al*., 1971; Charman, 1988; Rowe *et al*., 1995) and invariably the ability of the soil to achieve low hydraulic conductivity.

2.7.6 Stabilization of lateritic soils

Stabilization may be defined as any processes by which a soil material is improved and made more stable. The goals of stabilization are therefore to improve the soil strength, to improve the bearing capacity and durability under adverse moisture and stress condition, and to improve the volume stability of a soil mass.

The tendency for lateritic gravels to be gap graded with depleted sand fraction and to contain a variable quantity of fines as well as to have coarse particles of variable strength which breakdown, limits their usefulness in some engineering projects such as road pavement (Charman, 1988). To ameliorate the above deficiencies and consequently to improve on their field performance characteristics, they need to be stabilized.

2.7.6.1 Mechanical stabilization

Strength and durability of soils can be improved by mechanical means using compaction without the addition of any foreign matter. Water is added during compaction to displace air and facilitate movement of fine grained particles past one another so as to achieve densification.

2.7.6.2 Cement stabilization

Most soils including tropical and lateritic soils can be stabilized with cement. Depending on the soil type, 4% to 10% of cement by weight of dry soil is mixed with the soil to cause it to harden into a compact mass, which will not soften in the presence of water. Cementation is based on hydration of cement. The major hydration products are a series of calcium silicate hydrates (CSH) and hydrated lime $Ca(OH)_2$. Reactions of the soil with cement include replacement of Ca^{++}, adsorption of $Ca(OH)_2$ by particles and cementation at interparticle contacts by the CSH gel. When clay graded minerals are present in excess of about 30%, it is more difficult to achieve economic stabilization by use of cement due to great difficulty in pulverizing and mixing (Gillot, 1968). The success of stabilization depends on intimate mixing of the additive with the soil. For lateritic soils, thorough pulverization is important because the sesquoxides effectively coat the surfaces of particles and can prevent the necessary chemical reaction taking place.

2.7.6.3 Lime stabilization

Lime is generally considered more appropriate for the stabilization of more clayey soils, but is less effective in organic soils. Lime stabilization is achieved with calcium oxide (quicklime) or calcium hydroxide (slaked lime). The stabilization mechanism of lime is similar to that of cement. Lime acquires silica or other pozzolans in the soil to form CSH gel. Lime reduces plasticity by replacing troublesome cations such as sodium with calcium. At the same time, coagulation takes place and the coarser lime particles absorb a good deal of free water resulting in dehydration. Lime treatment of 3% to 8% by weight of dry soil are typical for improvement of plastic and expansive soils.

2.7.6.4 Bitumen stabilization

In bituminous stabilization, bitumen materials (hot cutback and emulsions) or road tars are mixed with soil so as to water proof the particles and/or provide the additional cohesion necessary for stabilization. The amount of binder normally ranges between 4 and 7%.

2.7.7 Utilization of lateritic soils in civil engineering

Originally, lateritic soil utilization was discussed in connection with mining of solid minerals such as Iron, Aluminum, Bauxite, and Manganese (Maignien, 1966). The civil engineering aspect has also been the subject of numerous studies and has been successfully used in various civil engineering construction projects as foundation materials, highway embankments, earth dams, low cost (mud) houses and in making bricks, etc. In the tropics and subtropics, the most common pavement material is ironstone lateritic gravel (Osinubi and Katte, 1997). Sometimes these soils require improvement in their engineering properties before they can be used in any form of construction. In such cases, lateritic soils can be chemically treated with suitable stabilizing agent.

2.8 Bentonite

These are processed, very plastic swelling clay of the smectite mineral group whose principal clay mineral is montmorillonite. The major characteristics of this mineral group include large cation exchange capacity, weak interlayer bond, large specific surface ($800m^2/g$), high swelling potential, Activity of 1.5 – 7 and low hydraulic conductivity k, to water (Grim, 1968; Grim and Guven, 1978; Egloffestein, 1995; Gleason *et al.*, 1997; Shackelford *et al*., 2000). Most bentonites are either sodium or calcium bentonite.

Bentonite is widely used for the following hydraulic containment applications:

(1) As a soil amendment for compacted soil barriers in landfills liners and cover systems (Huang and Wong, 1992; Kenny *et al.*, 1992; Daniel, 1993; O'Sadnick *et al*., 1995)

(2) As a low hydraulic conductivity component in geosynthetic clay liners (Daniel, 1993; Koerner, 1994). Natural, sodium activated, and chemically resistant bentonites have been used in the manufacture of geosynthetic clay liners (GCLs) – a thin factory manufactured clay liners 10 mm thick that are filled with bentonite (~5kg/m^2) sandwiched between two geotextiles or bonded to a geomembrane (Shackelford *et al.*, 2000). At present, there is little field evidence regarding their durability and effectiveness at limiting contaminant migration.

(3) Soil bentonite, cement bentonite or bentonite water slurries have been used in trench cut off walls in the remediation of contaminated sites or to maintain trench stability (D'Appolonia, 1980; Ryan, 1987; Evans, 1993).

The cations in the adsorbed layer of natural bentonite may be of varied species such as Ca, Mg, Na, K, Al, Si. The relative abundance of the cations to a large extent influences the thickness of the adsorbed layer and in that case, the bentonite derives its name from the dominant and major exchangeable cation and this varies with the source of bentonite. There are two major types in use – sodium and calcium bentonite. Sodium bentonite is used more extensively than calcium bentonite because of its superior swelling capacity and low hydraulic conductivity to water.

2.8.1 Soil - bentonite mixtures in hydraulic applications.

The need for barrier systems in landfills that are more cost effective and efficient in service in terms of performance and integrity are some of the major reasons for the growing interest in alternatives to existing ones. Soil, or sand blended with bentonite is one of such research efforts. Compacted sand – bentonite (S-B) mixtures have been used for hydraulic containment applications in cases where suitable natural clayey soils are not readily or economically available (eg Daniel, 1987; Garlanger *et al.*, 1987; Chapuis *et al.*, 1992; O'Sadnick *et al.*, 1995; Gleason *et al.*, 1997; Howell and Shackelford, 1997).

Daniel (1987) presented graphically the effect of percent of bentonite on hydraulic conductivity for sand – bentonite mixtures. He found that hydraulic conductivity decreased from 10^{-4} to 10^{-8} cm/s as the percentage of bentonite increased from 0 – 8%. At higher bentonite contents however, little reduction in hydraulic conductivity was observed. Similar results were observed by Kenny *et al.* (1992) who noticed significant decrease in the hydraulic conductivity of S-B as the bentonite content increased from 0 – 8% at various moulding water contents. Extensive studies on how water content and compactive effort affect the hydraulic conductivity of S-B mixtures have been carried out. Haug and Wong (1992) reported that moulding water content is not a critical factor in the design and construction of S-B barrier layers, as water is not required to break down clods and facilitate remolding as is required for clayey soils. Kenny *et al.* (1992) however explained that at low water contents, the moulding water content can influence the hydraulic conductivity of S-B mixtures because it affects the distribution of the bentonite. Kraus *et al.* (1997) also inferred that the hydraulic conductivity of S-B mixtures is not particularly sensitive to compactive effort. A similar result was reported by Chapuis (1990) who observed lack of correlation between porosity and hydraulic conductivity of S-B mixtures.

Similarly, local soils blended with bentonite to achieve low hydraulic conductivity have been investigated for use as barrier in waste containment facilities. D'Appolonia (1980) evaluated how variation in fines content (particles finer than 200 sieve) affected the hydraulic conductivity of soil – bentonite slurries. He observed that hydraulic conductivity decreased as the percentage of fines increased even when the percentage of fines was substantial (>30%). He also found that slurries containing plastic fines typically had hydraulic conductivities one order of magnitude lower than slurries containing mainly non-plastic fines. Ryan (1987) reported similar results. These results are in agreement with changes in hydraulic conductivity, k obtained in S-B mixtures (Daniel, 1987 and Kenny *et al.*, 1992).

The addition of bentonite to either soil or sand to achieve low k has been justified based on the high swelling potential of bentonite in the presence of water. The large specific surface ($800m^2/g$) of the predominant clay mineral – montmorillonite combined with its large negative charge results in adsorption of a large number of hydrated cations as well as water molecules and interlayer separation during hydration (Grim, 1968 and Mitchell, 1993). These adsorbed water molecules and cations can occupy a significant fraction of the pore space and are essentially immobile (Mesri and Olson, 1971; Mitchell, 1993; Shang *et al*., 1994). As a result, the portion of the pore space occupied by bulk water ie non adsorbed water that is free to flow is relatively small and the pathways formed by the bulk water have irregular shapes, (Mesri and Olson, 1971) which is responsible for the very low hydraulic conductivity of mixtures containing bentonite to water. Hence the hydraulic conductivity of bentonite is particularly sensitive to changes in the chemistry and composition of the pore fluid that influences the thickness of the adsorbed layer. In particular, liquids that cause the adsorbed layer to expand and the bentonite to swell such as water, results in relatively low hydraulic conductivity. Liquids that cause the adsorbed layer to collapse and the bentonite to shrink cause the hydraulic conductivity to increase.

2.8.2 Bentonite and bound water

The correlation between hydraulic conductivity of bentonite and the swelling of montmorillonite particles is generally attributed to the volume of water molecules that are bound to the clay surface. These molecules are considered immobile relative to those in bulk pore water, and act similar to the solid phase in terms affecting flow. When the volume of the bound water molecules increases, the fraction of the pore space comprised of freely flowing bulk water decreases and pathways for flow become smaller and more tortuous. At the microscale, an increase in the volume of bound water is manifested as an increase in the swell volume and a decrease in hydraulic conductivity (Mesri and Olson, 1971). The fraction of the

total volume of water that is immobile is proportional to the number of interlayer surfaces between the 2:1 montmorillonite layer. In the geotechnical literature, the volume of bound water and the interaction between particles has traditionally been described in the context of the diffuse double layer using the Stern-Gouy theory (Mitchell, 1993; Shackelford *et al*., 2000).

2.9 Leachates

Leachate is a contaminated liquid from decomposed waste in a landfill or waste dump. Most of the solid waste in a municipal landfill ultimately become liquid waste called leachate. If not controlled with a leachate collection system and a liner, leachate may leak into aquifers or drinking water wells. Much like the source material, leachate composition varies from state to state, city to city, site to site as well as from season to season. Leachate flux is characterized by a slow, unsaturated type of flow. It is unsteady, non uniform and sometimes discontinuous (Fang, 1997).

2.9.1 Generation

The composition of an average municipal refuse may vary from place to place and will depend on the local cultural background of the generating community. Leachate is produced when water or another liquid comes into contact with waste. Source of water in a containment landfill include rainfall, snow, surface or groundwater intrusion, water in the waste itself and the recirculation of leachate or irrigation of the final cover. As water percolates through the fill, it leaches out soluble salts and biodegraded organic products. Thus percolating water gradually deteriorates in quality resulting in a polluted liquid, which may vary in colour from light brown to nearly black usually with a foul and sickly smell.

2.9.2 Composition and characteristics

Leachate through a decomposing landfill contains a range of inorganic and organic chemicals. These chemicals impact the groundwater quality beneath the landfll facility or the dumpsite as the case may be.

The main components of leachates are (Judith and Eduljee, 1994):

- Major ions – Calcium, Magnesium, Potassium, Iron, Sodium, Ammonium, Bicarbonate, Sulphate and Chloride.
- Trace metals e.g Magnesium, Zinc, Copper, Chromium, Nickel, Lead and Cadmium.
- A wide variety of organic components that are usually measured as Total Organic Carbon (TOC) or Chemical Oxygen Demand (COD) or Biological Oxygen Demand (BOD). Individual components hazardous at very low concentrations may also be of concern e.g. pesticides, benzene, phenol.
- Microbiological components. While the leachate quality and strength is affected by the extent of rainfall infiltration into the site, the nature of waste, the rate and nature of waste degradation, the method of operation, the method of operation of the site and also the measures taken for leachate management, the quality at any time is primarily dependent upon the balance between the acetogenic and methanogenic phases of degradation. The acetogenic leachate produced during the early stages are of high organic strength whereas during the later methanogenic phase, the organic compounds are actively converted to landfill gases, leaving a residue of poorly degradable humic-type material.

Leachate quality is however site specific and it is very difficult to predict the timescale over which the different changes will take place. What is clear is that a large high-density municipal waste containment landfill will continue to produce strong and potentially polluting leachate over a very long time (Robinson and Gronow, 1992)

2.9.3 Interaction between bentonite and municipal solid waste leachate

The hydraulic conductivity of bentonite and soils containing bentonite changes in accordance with the volume of water bound to the mineral surface, which is affected by concentration, cation valence and pH of the permeant solution (Shackelford *et al.*, 2000; Egloffestein 2001) The Na^+ ions initially existing on and between the montmorillonite layers can be replaced by multivalent cations (e.g., Ca^{2+} and Mg^{2+}) in the permeant solution, a process referred to as cation exchange (Grim, 1968; van Olphen, 1977; McBride, 1994). The rate at which the ion exchange process occurs is controlled primarily by the rate at which cations diffuse into and out of the interlayer region between the montmorillonite layers (McBride 1994; Jo *et al*., 2004).

- When multivalent cations replace the Na^+ ions, the volume of water bound to the montmorillonite surface (i.e. immobile water) decreases and the volume of free (mobile) water increases (van Olphen, 1977; McBride, 1994), resulting in shrinkage of montmorillonite aggregates. Under modest confinement, this change in aggregate size results in larger interaggregate pores (i.e. the pores conducting flow) and higher hydraulic conductivity (Mesri and Olson, 1971; Jo *et al.*, 2001). Conversely, if the permeant solution contains only monovalent cations, or if there is no cation exchange, the volume of bound water increases or decreases depending on the cation concentration in the permeant solution. This effect causes expansion or contraction of the montmorillonite aggregates or interaggregate pores, resulting in changes in hydraulic conductivity. A variety of investigators studying the hydraulic conductivity of GCLs on salt solutions have observed these effects (Petrov and Rowe 1997; Shackelford *et al.*, 2000; Egloffestein 2001; Jo *et al.*, 2001; Vasko *et al.*, 2001).

CHAPTER THREE

THEORETICAL BACKGROUND

3.1 Elements of Containment System Design

3.1.1 Liner systems

Containment facilities require liner systems that perform as both hydraulic and chemical barriers. The primary objective of engineered components in containment facilities is to prevent migration of leachate directly into the underlying subsurface during both the active disposal period and the post closure period. Liner systems is therefore one of the most important elements of a modern engineered landfill (Benson, 2000). The technology has evolved significantly in the last decades in countries where landfill has been in use (Amadi, 2007). The potential materials are compacted clay, geomembrane, geosynthetic clay liners (GCLs), recycled or reused waste. The first consideration is always a suitable natural clayey soil when such a soil is readily or economically available. Compacted soil is widely used to line landfills and waste impoundments, to cap new waste disposal units and to close old ones (Daniel and Benson, 1990). A more sophisticated and effective lining system incorporates a composite liner comprised of a geomembrane or geosynthetic clay liners overlying a compacted clay liner. Composite liners are now the most common liners used for municipal solid waste (MSW) landfills in North America, Europe, Australia, New Zealand and many Asian countries (Kortegast *et al*., 1997; Benson, 2000; Katsumi *et al*., 2001). Regulations for hazardous waste landfill often require composite liners. Redundant lining systems incorporating one or more liners are also used primarily for sites where risks are greater (e.g hazardous waste landfills or corrective action management unit at remediation sites).

3.1.2 Cover systems

Upon reaching their waste storage capacity, the containment facilities should be covered properly to isolate the waste and to prevent the generation of leachate by minimizing the amount of precipitation percolating through the waste during the inactive period. Cover systems are a critical component in the overall process of managing liquid and gas movement into and out of the underlying contaminated material.

The amount of percolation through the cover P_c is calculated from consideration of the water balance for the cover expressed as follows

$$P_c = P_r - R_s - S_w - ET - L_f \qquad (3.1)$$

where P_r = precipitation, R_s = surface runoff, S_w = soil water storage, ET = evaporation = transpiration (evapotranspiration) and L_f = intra-layer lateral flow. Aside from precipitation, each component of the water balance depends on the hydraulic conductivity, k_h, of the soils used in the cover.

Two possible soil cover scenarios are commonly evaluated (Benson and Khire, 1995; Khire *et al*., 1997; Khire *et al*., 2000). Resistive cover and capillary cover. Resistive covers employ a conventional hydraulic barrier (compacted clay layer, geosynthetic layer or composite clay – geosynthetic barrier) that limits flow by its imperviousness, while a capillary barrier results when unsaturated flow occurs through a relatively fine layer overlying a relatively coarse layer (e.g clay over sand, silt over gravel and sand over gravel). In a capillary barrier, only a small fraction of the flux associated with the wetting front through the finer layer is transmitted into the underlying coarse layer.

3.1.3 Leachate collection and removal system (LCRS)

The collection of liquids in waste containment systems and their proper removal represents an important element in the successful functioning of waste containment facilities (Koerner *et al*., 1994; Oweis and Khera, 1998; Giroud *et al*., 2001). The purpose of a leachate collection

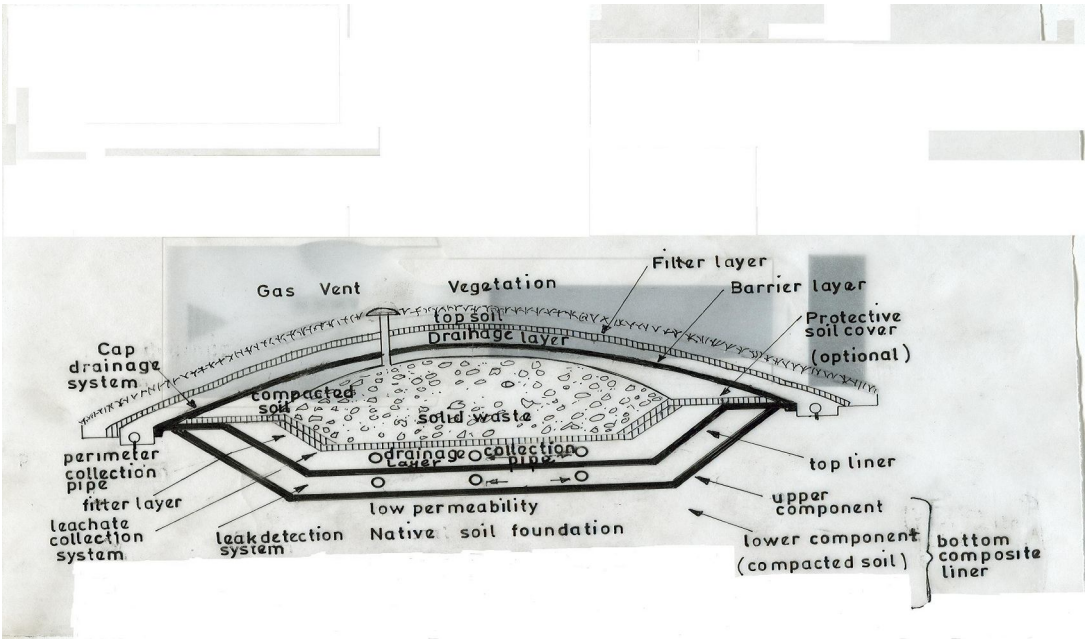


Fig. 3.1 Schematic representation of typical liner and cover cross-section for controlled solid waste landfill (not to scale).

system is to remove leachate collecting on top of the liner in order to reduce the leachate head acting on the base of the landfill, the hydraulic gradient through the underlying barrier and the Darcy velocity out of the landfill can be reduced to acceptable levels. By removing contaminant from the landfill, the mass of contaminant available for transport into the hydrogeological system will be reduced.

To collect and prevent the accumulation of leachate water, a pipe network is installed at the bottom of cells. The system consists of perforated pipes made of High Density Polyethylene (HDPE). These pipes are installed in ditches covered with protective filter layer (Koerner, 1993) (e.g gravel and a filtration geotextile or geonet) to minimize possible risk of

obstruction. Submersible pumps are commonly used to pump the collected leachate to trucks, storage tanks, reservoirs or directly to treatment facilities adjacent to the landfill site.

3.1.4 Gas collection and removal system

For the collection and removal of landfill gases, composed mainly of methane and carbon dioxide (Barlaz and Ham, 1993; Tchobanoglous *et al.*, 1997), engineered landfill sites are equipped with a network of vertical wells regularly spaced in the interior of the cells and are prolonged up until the final covering layer. These wells are connected to horizontal tubes that lead the landfill gas to an energy station if such facility is economically viable (Oweis and Khera, 1998).

3.1.5 General stability

Although most of the focus on landfill design relates to controlling contaminants, it would be a mistake to overlook the importance of geotechnical considerations such as slope stability (Rowe *et al.*, 1995). This is particularly true with the growing trend to both larger new landfills and vertical and lateral expansion of existing landfills.

A landfill can represent a major loading on the underlying soil. Just as with the design of any structure on soil, care should be taken to ensure the overall stability of the landfill under both static and, where appropriate, seismic conditions. This involves an evaluation of the geotechnical conditions and appropriate stability analyses to identify the most critical conditions in the development of the landfill. These analyses should consider (Rowe *et al.*, 1995):

1. Stability of the excavated trenches for the landfill
2. Stability during development of the landfill
3. Stability of the completed landfill

Particular care is required not to underestimate the load that can be applied by partially saturated waste. Landfill failures have occurred due to inadequate assessment of overall stability (Rowe *et al.*, 1995).

3.2 Design Parameters for Liners and Covers

3.2.1 General barrier specifications

The suitability of any material for the construction of a barrier system depends on a number of factors:

- Low hydraulic conductivity, generally less than 10^{-9} m/s as required by most regulatory agencies (Daniel and Benson, 1990 as well as Daniel and Wu, 1993).
- Adequate strength to support the overburden pressure imposed by the waste body. Daniel and Wu (1993) used minimum unconfined compression strength of 200kN/m^2 to represent adequate shear strength for compacted soil liner and cover.
- Durability and long term stability of the material is necessary to withstand the destructive forces of alternating wet/dry cycles (Edil *et al.*, 1992).
- Constructibility, that is, material should be workable in terms of placement and compaction under field construction conditions.
- Minimum potential for shrinkage and cracking (natural or compaction induced) upon drying. Maximum allowable value of 4% volumetric shrinkage of compacted cylinder upon drying has been adopted by some investigators – (Daniel and Wu, 1993; Albrecht and Benson, 2001; Osinubi and Amadi, 2006).
- Compatibility with the leachate to be contained, which means the strength, low hydraulic conductivity and shrinkage behaviour should be maintained after prolonged exposure to leachate and should not release harmful chemicals from its matrix to the leachate but rather should have an adsorptive or attenuative capacity for critical pollutants, such as heavy metals (Edil *et al.*, 1992 as well as Rowe *et al.*, 1995).

- A minimum of 15 – 30% of the soil with a particle size smaller than 2μm, plasticity index greater than 7% and an activity of 0.3 or greater is specified because low hydraulic conductivity is normally associated with the presence of clay minerals. Alternatively, a minimum cation exchange capacity (CEC) of about 10 milliequivalent (Meq) per 100g of soil may be specified (Daniel, 1993; Rowe *et al.*, 1995).
- For a municipal waste facility, minimum thickness of compacted barrier is generally about 0.9 – 1.0m. However, this is highly variable from jurisdiction to jurisdiction and in some cases is reduced to 0.6m for composite liner systems. For industrial/toxic facilities, the minimum thickness of compacted liner is generally 3 – 4m, although some jurisdiction require up to 15m or multiple liner systems (Rowe *et al*., 1995; Benson *et al*., 1999).

While the foregoing represents common specifications of clayey barrier systems, the design of a barrier is site specific and its characteristics must be selected to suit the site such that the system will provide adequate control on leachate migration.

3.2.2 Hydraulic conductivity

The hydraulic conductivity of soils is of great importance in relation to some geotechnical and geoenvironmental problems including the determination/transport of contaminants through barriers in waste repositories. The primary function of compacted clay liners in municipal and hazardous waste containment systems is to minimize the flow of leachate. Factors affecting the hydraulic conductivity performance of a liner material will depend on a number of important factors (Lamb, 1954; Mitchell *et al*., 1965; Garcia-Bengochea *et al*., 1979; Benson and Daniel, 1990). The critical variables identified include: moulding water content, method of compaction, compactive effort and degree of saturation.

Benson *et al.*, (1994) reported that hydraulic conductivity of compacted soil specimens depend greatly on the moulding water content and the dry unit weight achieved during compaction. Soils compacted dry of optimum water content tend to have relatively high hydraulic conductivity whereas soils compacted wet of optimum water content tend to have low hydraulic conductivity. Specimens compacted at combinations of water content and dry unit weight yielding higher degree of saturation (at compaction) have lower hydraulic conductivity.

Hydraulic conductivity was also seen to be sensitive to the Atterberg limits and particle size distribution. Soils that are more plastic (higher liquid limit or higher plasticity index or containing a greater quantity of clay size particles) have lower hydraulic conductivity.

Generally, compacted clayey soils used as barriers and covers in waste containment systems are expected to have low hydraulic conductivity. According to Daniel and Benson (1990), the hydraulic conductivity must be less than or equal to $1x10^{-9}$ m/s for soil liners and covers used to contain hazardous waste, industrial and municipal solid waste.

3.2.3 Volumetric shrinkage

Clay liners are always subjected to drying during construction, and cracking due to shrinkage is a potential problem. In current geoenvironmental practice, clay barriers of waste containment facilities have been constructed using natural clays, processed clays and native soils mixed with bentonite (Daniel, 1993; Benson *et al.*, 1999; Shackelford *et al.*, 2000).

Since hydraulic conductivity is a direct control of the performance of landfill barriers (Daniel, 1993; Shackelford, 1994; Chen *et al.*, 2000; Shackelford *et al.*, 2000), the design of the liner is based on achieving a material with the desired hydraulic conductivity. However, the reduction in hydraulic conductivity is often accompanied by an increase in volume change due to the susceptibility of clay to desiccation-induced shrinkage. The consequence of this is

that the properties and the structure of a compacted barrier can change with time resulting in a reduction of the effectiveness of the barrier. The deterioration of the barrier can be the result of volumetric changes imposed by climatic wet-dry or volumetric changes associated with the interactions of clay particles with leachate.

The engineering design of landfill barriers with such mixes is a complex problem requiring a good understanding of the constitutive behaviour in order to predict accurately the performance of this material and to take into account the critical parameters relevant to the proper functioning of the facility under service conditions.

The designer has practically no control over the magnitude of the changes that will be imposed on the barrier i.e the degree of desiccation the barrier will undergo. However, the designer can influence the formation and the extent of shrinkage by manipulating the composition and the compaction conditions (moulding water contents and compaction energy) in order to reduce the shrinkage potential of the compacted barrier to standard specification i.e maximum of 4% (Khire *et al*., 1997; Stewart *et al*., 1999).

3.2.4 Shear strength

In order to ensure the structural integrity of a constructed landfill facility compacted soils used for the liner and cover systems must have adequate strength (Daniel and Wu, 1993).

A soil liner or cover system should have adequate strength to support the overburden pressure imposed by the waste body. The strength of compacted clay depends on the density as well as water content. Clays compacted dry of optimum are initially stronger than when compacted wet of optimum. Compacted soil strength can be evaluated in terms of unconfined compression strength. Daniel and Wu (1993) selected minimum unconfined compression strength of 200kN/m^2 (or 200kPa) to represent adequate shear strength for compacted soil

liner and cover. However, each project would have to be evaluated to determine minimum strength requirements.

3.2.5 Delineation of acceptable zones

In current geoenvironmental practice, design engineers usually require that clayey liners be compacted within a specified range of water content and to a minimum dry unit weight. This specification is based primarily upon the need to achieve a minimum dry unit weight and a range of water content that will produce compacted soil mixtures with acceptable hydraulic conductivity, strength and shrinkage strain. These three components are tied directly or indirectly to compaction.

Water content and dry unit weight values can greatly affect a soils ability to impede the flow of fluids. The design parameters of a clayey material such as hydraulic conductivity, shear strength and volumetric shrinkage are greatly influenced by water content. Little variations in water content and dry unit weight may result in tremendous changes in these parameters (Mitchell, 1965). If the soil is too dry at the time of compaction, low hydraulic conductivity may be difficult to attain. On the other hand, if the soil is too wet, a number of problems such as weak soils, slope instability caused by low strength and workability problems in addition to desiccation-induced shrinkage/cracks may arise. Thus, ranges of water contents within which the compacted soil will exhibit specification requirements of $k \leq 1x\ 10^{-9}$ m/s, volumetric shrinkage $\leq$ 4% and unconfined compressive strength $\geq$ 200kN/m^2. The recommended procedure involves establishing water content – dry unit weight, γ_d ranges needed to achieve the required hydraulic conductivity, shear strength, volumetric shrinkage and then modifying these ranges to account for other factors (Daniel, 1993; Daniel and Benson, 1990).

The acceptable zone for hydraulic conductivity is delineated by an envelop drawn to define the zone where moisture – density combinations produce acceptable hydraulic conductivities. The acceptable zone based on strength criteria defines the minimum dry unit weight and range of water content that will produce compacted soil with acceptable strength. The zone for shrinkage is defined in terms of the upper limit of the water content. The upper boundary of all limits is the zero air void line. When these plots are superimposed, the zone where all the permissible ranges overlap is the overall acceptable zone.

3.3 Unsaturated Hydraulic Properties

Interest in unsaturated (vadose) zone has dramatically increased in recent years because of growing evidence of public concern that the quality of the subsurface environment is being adversely affected by industrial, municipal and agricultural activities that introduce chemicals into the subsurface. The unsaturated zone provides the best opportunities to limit or prevent groundwater pollution since before reaching the water bearing saturated zones, these chemicals migrate through the unsaturated zone (van Genuchten *et al*., 1991).

In geoenvironmental engineering practice, the purpose of the final cover system in waste repository is to prevent the generation of leachate by minimizing the amount of precipitation percolating through the waste postclosure period (Chiu and Shackelford, 1998). Proper evaluation of the hydrology of compacted soil covers should be based on consideration of unsaturated flow since soil covers are compacted in an unsaturated condition (Khire *et al*., 1994, 1995, Chiu and Shackelford, 1998; Miller *et al*., 2002). Thus cheap and more expedient methods for estimating hydraulic properties are needed if improved practices for managing water and chemicals in the unsaturated zone are to be implemented.

Unsaturated hydraulic conductivities are commonly derived from measured saturated hydraulic conductivity and soil water characteristic curves (SWCCs) determined in the laboratory. The SWCC which defines the soil's ability to store and release water play a fundamental role in the design of waste containment facilities. Therefore accurate prediction quantification of the SWCC is important for better prediction of performance (Miller *et al*., 2002). Two experimental paths can be followed in order to establish SWCC, namely drying (desorption) and wetting (adsorption) methods. In this study, the sequential drying path approach was followed (see Fig. 3.2).

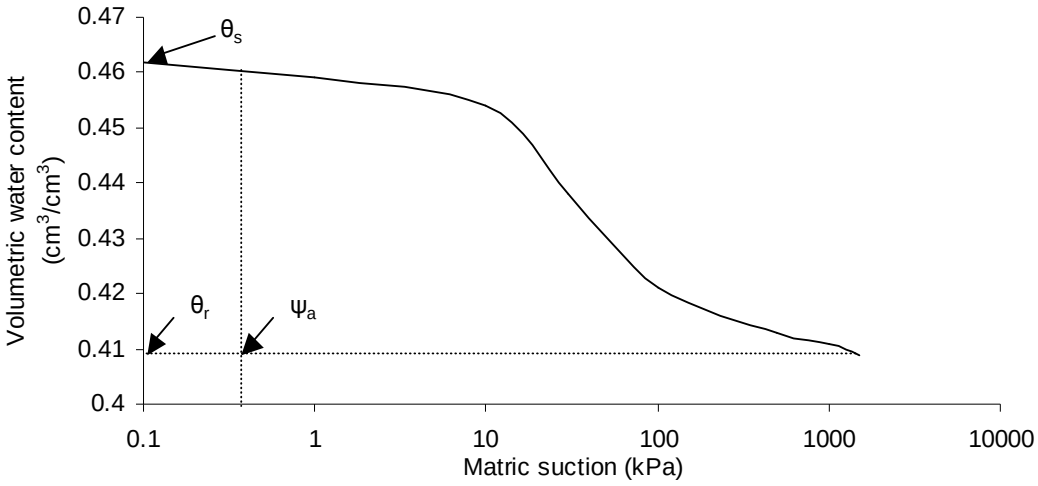


Fig. 3.2. Typical Soil Water Characteristic Curve

3.3.1 Soil water retention models

Several functions have been proposed to empirically describe the trends in soil water characteristic curve (SWCC) data. The commonly used functions include the Brooks-Corey (Brooks and Corey, 1964), van Genuchten, (van Genuchten, 1980) and Fredlund and Xing (Fredlund and Xing, 1994) equations.

The Brooks-Corey model which is later referred to as BC equation is given as

$$\frac{\theta_w - \theta_r}{\theta_s - \theta_r} = \left(\frac{\psi_a}{\psi}\right)^{\lambda} \tag{3.2}$$

where the optimized parameters are θ_r , ψ_a and λ. θ_r, θ_s are the residual and saturated water contents, respectively; ψ_a is the air entry value which is the matric suction when air starts to enter the largest pores in the soil, while λ is an empirical parameter representing the pore size distribution index and is related to the slope of the curve.

The van Genuchten model is expressed as:

$$\frac{\theta_w - \theta_r}{\theta_s - \theta_r} = \frac{1}{\left[1 + \left(\frac{\psi}{\alpha}\right)^n\right]^m} \tag{3.3}$$

where the optimized parameters are θ_r, α, n, and m.

The parameter α is related to the air entry value and the n parameter is related to pore size distribution of the soil that controls the slope of the SWCC while m is related to asymmetry of the model curve.

The Fredlund and Xing four parameter model is expressed as:

$$\frac{\theta_w}{\theta_s} = \frac{1}{\left\{\ln\left[e + \left(\frac{\psi}{a}\right)^b\right]\right\}^c} \tag{3.4}$$

where the optimized parameters are a, b, and c. The parameters a, b, and c of the Fredlund and Xing model are similar to the parameters α, n, and m in the van Genuchten model, respectively. Application of this model assumes that θ_r is small enough that it can be neglected.

The Fredlund and Xing five-parameter model is given as:

$$\frac{\theta_w - \theta_r}{\theta_s - \theta_r} = \frac{1}{\left\{\ln\left[e + \left(\frac{\psi}{a}\right)^b\right]\right\}^c} \tag{3.5}$$

where the optimized parameters are θ_r, a, b, and c.

The soil water characteristic functions presented by Brooks and Corey and van Genuchten were considered for use in this study.

3.3.2 Unsaturated hydraulic conductivity

Although direct measurement of unsaturated hydraulic conductivity (k) is possible, the associated problems make it unattractive. Predictions of unsaturated k can be made based on either the Brooks – Corey – Burdine (BCB) (Brooks and Corey, 1964) or van Genuchten – Mualem (VGM) (van Genuchten, 1980) relative hydraulic conductivity functions. Either or both of these relative hydraulic conductivity functions typically are required in unsaturated flow modeling (Chiu and Shackelford, 1998). Common to all of these functions is the use of SWCC in conjunction with saturated hydraulic conductivity k_{sat}.

The relative hydraulic conductivity, K_r, is defined as the hydraulic conductivity of the soil, $K[=f(\psi_m)]$, relative to the saturated hydraulic conductivity K_{sat} of the soil or

$$K_r = \frac{k}{k_{sat}} \tag{3.6}$$

The BCB relative hydraulic conductivity function is given as:

$$K_r = \begin{cases} \left(\frac{\psi_a}{\psi}\right)^{2+3\lambda} & \psi_a > \psi \\ 1 & \psi \le \psi_a \end{cases} \tag{3.7}$$

where λ is the pore size distribution index (Corey, 1994).

The VGM relative hydraulic conductivity function is given as:

$$K_r = \frac{\left\{1-(\alpha\psi)^{n-1}\left[1+(\alpha\psi)^n\right]^{-m}\right\}^2}{\left[1+(\alpha\psi)^n\right]^{\frac{m}{2}}} \qquad (3.8)$$

where α, m and n are as defined in eqn. (3.3)

The BCB function is based on the assumption that the porous medium consists of a bundle of capillaries, whereas the VGM function considers the porous medium to consist of a randomly distributed pore – size distribution (Chiu and Shackelford, 1998).

3.4 Contaminant Transport Processes and Governing Equations

3.4.1 Preamble

Once a chemical enters the groundwater regime, several transport mechanisms are present that may aid in the spreading of the contamination. These mechanisms include, advection, diffusion, mechanical dispersion, hydrodynamic dispersion e.t.c.

3.4.2 Advection

Advection involves the movement of dissolved solute (contaminant) in response to a gradient in total hydraulic head. The amount of contaminant being transported is a function of its concentration and the quantity of fluid flowing. Due to advection, conservative solutes (i.e solutes which are not subject to chemical or biological reactions) are transported at an average rate equal to the seepage velocity of the groundwater, or

$$v_s = \frac{v}{n} \qquad (3.9)$$

where v_s is the seepage or average linear velocity of the water, n is the total porosity of the porous material, and v is the flux of water (i.e quantity of flow per unit area per unit time). The flux is given by Darcy's law which, for one dimensional flow can be written as

$$v = \frac{Q}{A} = -K\frac{\partial h}{\partial x} = Ki \qquad (3.10)$$

Where Q is the volumetric flow rate of the water, A is the total cross sectional area (solid plus void) perpendicular to the direction of flow, K is the hydraulic conductivity, h is the total hydraulic head, x is the direction of flow, and i is the dimensionless hydraulic gradient.

3.4.3 Diffusion

Diffusion is a transport process in which a contaminant or chemical species in water flows in response to a gradient in its concentration. Diffusion will occur as long as a concentration gradient exists even if the fluid is not moving and as a result, a contaminant may spread away from the place where it is introduced into a porous medium. Diffusion may also occur when the concentration of a contaminant is higher in one stratum than in an adjacent stratum provided that the adjacent stratum has the prerequisite porosity. The resulting mass flux of the solute usually is assumed to be directly proportional to the concentration gradient in accordance with Fick's first law, or:

$$J = -D_o\frac{\partial c}{\partial x} \qquad (3.11)$$

Where J is the mass flux (mass transported per unit time per unit cross sectional area), c is the concentration of the solute in solution, x is the direction of transport, and D_o is the constant of proportionality known as the free-solution diffusion coefficient. The negative sign in equation indicates that diffusion occurs in the direction of decreasing concentration. Equation represents the one-dimensional form of Fick's first law describing diffusion in aqueous solution (i.e no soil).

For diffusion in saturated soil, a modified form of Fick's first law applies,

$$J = -\tau D_o n \frac{\partial c}{\partial x} \qquad (3.12)$$

or

$$J = -D^* n \frac{\partial c}{\partial x} \qquad (3.13)$$

Where τ is a dimensionless tortuosity factor and D^* is the effective diffusion coefficient. Since the pathways for diffusion in soil are longer and more tortuous than they are in aqueous solution, D^* values are lower than D_o values and diffusion in soil is slower than diffusion in aqueous solution.

Fick's first law describes steady-state flux of solutes. For time independent (transient) transport of solutes in soils, Fick's second law applies:

$$\frac{\partial c}{\partial t} = D^* \frac{\partial^2 c}{\partial x^2} \qquad (3.14)$$

There are several mathematical solutions to this equation, depending on the initial and boundary conditions which describe the concentration of a solute as a function of time and space (Crank, 1975; Shackelford, 1991).

As a steady state equation, Fick's first law is analogous to Darcy's law describing the steady state transport of a fluid in response to a hydraulic gradient. As a transient equation, Fick's second law is analogous to the time dependent consolidation equation where c represents the excess pore water pressure (u) and D^* represents the coefficient of consolidation (c_v).

3.4.4 Mechanical dispersion

This is a mixing process that occurs due to local variations in the flow velocity that causes the solute front to spread out or disperse as it flows through the porous medium. Groundwater flow through different porous media occurs at variable rates due to the arrangement of

particles in the subsurface. As contaminated groundwater travels through the stratum, the fluid does not all travel at the same velocity, and as a result, mixing occurs along flow path. This mixing is termed mechanical dispersion and it results in a dilution of the contaminant at the advancing edge of flow (Shackelford, 1993; Rowe *et al.*, 1995). Mechanical dispersion is composed of two-contaminant front mixing methods – longitudinal and transverse dispersion. Longitudinal dispersion occurs along the direction of the flow path, while transverse dispersion occurs normal to the flow path and causes lateral spreading of the contaminant (Freeze and Cherry, 1979). Mechanical dispersion and diffusion combine in groundwater flow to create hydrodynamic dispersion, hence the two processes are often lumped together as a composite parameter, D, called the coefficient of hydrodynamic dispersion represented by:

$$D_h = D_m + D_o\tau \qquad (3.15)$$

where D_m is the mechanical dispersion coefficient; D_o is the free solution (aqueous) diffusion coefficient of the solute and τ is the dimensionless tortuosity factor. Values of D_o for most common inorganic chemical species typically range between ~$7x10^{-10}$ to ~ $2.0x10^{-9}$ m^2/ s (Shackelford and Daniel, 1991).

The tortuosity factor, τ, is purely geometric and accounts for the increased distance of transport and the more tortuous pathways experienced by solutes migrating through the soil. Tortuosity is expressed as

$$\tau = \left(\frac{L}{L_e}\right)^2 \qquad (3.16)$$

where L is the macroscopic, straight-line distance between two points defining the flow path, and L_e is the actual, microscopic or effective distance of transport between the same two points. Since $L_e > L$ for porous media, $\tau < 1$. Values of τ are reported to be in the range from 0.01 to 0.84 for saturated soils and from 0.025 to 0.57 for unsaturated soils (Freeze and Cherry, 1979; Daniel and Shackelford, 1988; Shackelford and Daniel, 1991).

The mass flux under the combined effects of hydraulic and concentration gradient is then given by

$$J = nvc - nD\frac{\partial c}{\partial x} \qquad (3.17)$$

where J s the mass flux; n is the total porosity of the soil (dimensionless); v is the seepage velocity of the fluid; c is the concentration of the solute in the liquid phase of the soil; x is the direction of transport; and D is the hydrodynamic dispersion coefficient.

For typical problems involving low permeability earthen barriers used for waste containment facilities, mechanical dispersion does not appear to be significant, i.e $D_h \approx D^*$ (Rowe, 1987). In such cases, the total solute flux, assuming coupled flow processes are negligible, becomes:

$$J = nv_s c - nD_o\tau.\frac{\partial c}{\partial x} \qquad (3.18)$$

When there is only concentration gradient, v_s and, therefore, D_m are zero and eqs. 3.15 and 3.18 reduces to the following form of Fick's first law for one dimensional diffusive transport in saturated soil:

$$J_D = -D_o\tau n.\frac{\partial c}{\partial x} \qquad (3.19)$$

where J_D designates the diffusive flux.

3.4.5 Sorption

Sorption process is defined as the attraction of aqueous species to the surface of a solid.

In soil medium, it is the partitioning of contaminant between pore water and solid phase. Adsorption results in partial removal of solute i.e it is partitioned between the soil solids. For instance, replacement of one or more types of cations on the soil surface with other cations is a form of adsorption. The mass of solute, S, adsorbed by the soil is a function of chemical concentration, c:

$$S = K_f c^n \quad (3.20)$$

Where K_f and n are constants determined by plotting adsorption data on a log-log scale. At low concentration (as is usually the case for municipal landfills) n ≈ 1. Then equation … becomes

$$S = K_d c \quad (3.21)$$

Where K_d is the distribution coefficient. A sorption or distribution coefficient (K_d) is commonly used to quantify the amount of contaminant in the soil system. The K_d is simply a ratio of the sorbed phase concentration to the solution phase concentration at equilibrium.

Isotherms are graphical representations of the mass of contaminants adsorbed per unit dry mass of soil (S) versus the concentration (c) of the contaminant. The Freudlinch isotherm is an equilibrium isotherm that is used most often in real world examples.

The Langmuir isotherm contains two assumptions that make its use usually unfit for the real world, heterogeneous soil case. The assumptions are that the energy of adsorption is constant, and the number of binding sites are finite. The equation of Langmuir isotherm is shown below:

$$S = \frac{abc}{(1+ac)} \quad (3.22)$$

Where a, b are empirical constants; S is the mass of solute removed from solution (sorbed) per unt mass of solid; c s the equilibrium concentration of solutes (mass per unit volume of pore fluid).

Sorption of heavy metals is strongly pH- dependent, the more metal that can be found in solution, the more metal being mobilized. When pH falls to below 5, mobility is enhanced as a result of increased proton concentration. At pH values above 7, some heavy metals tend to form hydroxyl-complexes which will increase the solubility of the metal in question.

3.4.6 Retardation factor : A correction factor known as retardation factor takes into account how much a contaminant's velocity is affected by sorption in the groundwater system. The overall correction factor is a measure of the bulk density of the media, the porosity and the distribution coefficient K_d.

3.4.7 Precipitation and dissolution

These are reactions resulting in the release of contaminants from solids or removal of contaminants as solids. Under various physicochemical conditions, a process of dissolution and precipitation of minerals in water occurs. Dissolution is mainly significant at the source or at the migration front while precipitation is an important mechanism particularly in high pH system (pH > 7). These reactions are more common for the natural systems and play a lesser part in the fate of pollutants (Bohn *et al*., 1985; Wild, 1993; Yaron *et al*., 1996; Shackelford and Rowe, 1998). The extent of the dissolution or precipitation reaction for systems that attain equilibrium can be estimated by considering the equilibrium constant. The precipitation of chemicals can only occur at super saturation since dissolution ends only at water saturation. In the soil medium, the dissolution reaction is a heterogeneous chemical reaction and it is necessary to characterize the solubility in terms of a solubility product with an equilibrium constant which can characterize the solubility as well as predict how solution variable can change the solubility.

3.4.8 Oxidation and reduction (Redox) reactions

Many reactions in soil involve the exchange or transfer of electrons from one element to another or from one compound to another. Usually this transfer of electrons is employed to change the oxidation state of pollutants in order to facilitate its removal from the aqueous phase. For example, ferrous iron Fe(II) is oxidized to ferric iron Fe(III) where $Fe(OH)_3$ is

formed which is removed easily from solution by precipitation. In this case, an electron donor is labeled the reductant whereas an electron acceptor is labeled the oxidant (Wild, 1993).

Redox reactions are among the most important factors controlling contaminant transport and fate in groundwater systems. Redox reactions change the speciation as well as the chemical behaviour of both organic and inorganic soil constituents by affecting solubility, reactivity, mobility, toxicity and bioavailability.

The oxidation – reduction status of the system is measured by the redox potential which is the difference in electrical potential between the half reactions of the coupled reaction. The redox potential will show least change on the addition of or removal of electrons when (Ox) = (Red.); this is the condition of maximum poise and occurs when $E_h = E^o_7$.

E_h = redox potential; E^o_7 = Standard redox potential.

Different oxidation forms of metal behave chemically different. As an example, chromium in the oxidation state (+6) is much soluble and toxic than chromium in the oxidation state (+3).

Organic contaminants can be naturally degraded by redox reactions. Mn and Fe oxides can oxidize aromatic pollutants. The degradation depends on the electron donating power of the functional groups found on the aromatic. The greater the electron donating power, the easier they are oxidized.

3.4.9 Ion exchange

Soil colloids exhibit a net negative charge which attracts and retains cations that are readily exchangeable with other cations. The amount of this negative charge in a soil expresses the soil's ability to adsorb positively charged ions and is called the cation exchange capacity (CEC). Thus, it is a measure of the quantity of sites on soil surfaces that can retain positively charged ions by electrostatic forces. Cations retained electrostatically are easily exchangeable with other cations in the soil solution and are therefore easily available. A high CEC reflects

a soil with high sorption capacity. It is often expressed as centimole charge per kilogram of soil. The negative charge in a clay colloid arises from two sources: From isomorphous substitution in the tetrahedral and octahedral sheets of aluminosilicate clays giving rise to permanent charge on the clay. The second source is the broken edges or the unsatisfied oxygen bonds at the edges of clay particles. Under acidic conditions, these oxygen bonds are satisfied by hydrogen ions forming hydroxyls. At higher pH, hydrogen ions dissociate and give rise to a net negative charge. This type of charge is called pH dependent charge or variable charge.

3.4.10 Volatilization

Volatilization is defined as the transition of chemicals from the solid or liquid phase into the vapour phase. Each chemical has a characteristic saturation vapour pressure or vapour density that varies with temperature. The potential volatility of a chemical is related to its inherent vapour pressure but actual vapourization rates depend on the environmental conditions and all the other factors that control the behaviour of the chemical at the sold-air-water interface. For surface deposits, the actual rates of loss or the proportionality constants relating vapour pressure to volatilization rates are dependent upon external conditions that affect movement away from the evaporating surface such as turbulent, surface roughness, wind speed etc. The rate at which a pollutant moves away from the surface is diffusion controlled. Two distinct processes govern the dispersion of pollutants into the atmosphere. The first is the vapourization of toxic molecules present in the soil medium; second is the dispersion of the resulting vapour into the soil gaseous phase or the overlying atmosphere by diffusion and turbulent mixing.

3.4.11 Hydrolysis

Hydrolysis is one of the most important mechanisms in the environment for the breakdown of a parent chemical. It occurs in soils, water, plants, animals and possibly air. Hydrolysis is the chemical interaction of water with the contaminant which alters the chemical's structure and in turn, alters its transport within the environment. Typically, it makes an organic compound more susceptible to biodegradation and more soluble. Many environmental factors influence the rate of hydrolytic degradation such as temperature, pH, solubility, sunlight, and or absorption and volatility.

3.4.12 Biological process

Biodegradation is the biological process by which microorganisms (aerobic microbes or anaerobic microbes) break down organic chemicals to either a higher or a lower molecular weight chemical(s). This is a very important process by which soil microbes or aquatic microbes can detoxify chemicals, however, the process may also result in the formation of a more toxic chemical. Chemicals that are highly water soluble can biodegrade, but those with low water solubility usually will not. Reduction of the concentration of a particular organic contaminant due to biological transformation does not ensure a reduction in toxicity. Many of the daughter products of transformation processes are as toxic or more toxic than their parent. Biotransformation of trichloroethylene (TCE) for example can result in the formation of vinyl chloride which is more toxic (and stable) than its parent.

3.4.13 Natural attenuation of contaminants in soils

Natural attenuation employs naturally occurring biological, chemical and physical processes to reduce environmental contaminants in soils and groundwater. The results of this interaction can lead to the partial or total immobilization of potential subsurface contaminants. The process of immobilizing and retarding the chemical constituents from

moving with the water phase - the mobile phase, is called attenuation (Shackelford and Jefferis, 2000). The ability of natural geological materials such as soil to immobilize potential groundwater contaminants and attenuate their movement is well known. Under the right conditions, soils and sediments have intrinsic attenuative capacity, thus immobilize potential contaminants through complexation with organic and inorganic compounds, precipitation and adsorption mechanism and prevent their movement (Rouse and Pyrih, 1993; Hansen, 1997). The extent to which this can work depends on a number of factors namely (Headly, 1994; Shackelford and Jefferis, 2000; Amadi, 2007):

- thickness and nature of the strata making up the unsaturated zone
- the physico – chemical attenuation capacity of these strata as a result of chemical reaction and biochemical degradation.
- the pH and Eh conditions that are established during contact of the leachate with the geological material.

Such 'indigenous filters' cannot be relied upon for total contaminant removal because of the uncertainty associated with the integrity of the system. Even when these conditions are locally or extensively favourable, there is likely to be considerable scope for short circuiting by preferential flow through hydraulic defects or imperfections such as fissures, cracks, joints, holes etc in the underlying strata.

3.4.13 Compatibility of barrier materials

Apart from the traditional selection and performance criteria in assessing the technical suitability of materials meant for hydraulic barriers in waste disposal facilities namely low hydraulic conductivity, adequate shear strength and low potential for desiccation cracks and volumetric shrinkage (Klepp and Olson, 1985; Daniel and Benson, 1990; Edil *et al*., 1992; Daniel and Wu, 1993; Rowe *et al*., 1995), one other fundamental design requirement and testing consideration is the compatibility between the material and the fluid to be contained

(Shackelford *et al*., 2000). This interaction is better understood through a compatibility test, which is usually made by means of hydraulic conductivity testing. Compatibility refers to the potential effect of chemical interactions between the permeating liquid and the porous material on the properties of the material. Drastic changes in hydraulic conductivity is an indicator of incompatibility. In cases of adverse results, stabilizing materials such as relatively non-reactive clays, organic modifiers or polymers may be incorporated to prevent drastic increases in hydraulic conductivity (Evans, 1993; Shackelford, 1997; Shackelford and Jefferis, 2000).

There are reported cases of high values of hydraulic conductivity in the field and in some cases failures of containment facilities traceable to compatibility problems (e.g., Goldman *et al*., 1986; Daniel and Brown, 1988). Most geological materials that contain these clay minerals are found to be naturally reactive resulting in greater adsorption characteristics and double layer expansion and this exerts considerable influence on the hydraulic conductivity both in short and long term. This influence has been explained by the double layer theory (Mitchell, 1993; Rowe *et al*., 1995; Shackelford *et al*., 2000). In the interaction between clay particles and the surrounding pore fluid, the net electrical charge on clay particles is negative (Mitchell, 1993) which causes hydrated cations in the surrounding pore water to be attracted to the surfaces of the clay. this layer of water and adsorbed ions that surrounds a clay particle is referred to as the electrical double layer or diffuse double layer (Mitchell, 1993; Shackelford, 1994). A double layer contraction at constant void ratio (flocculation) creates a large increase in free void space, this may cause increases in hydraulic conductivity. Conversely, a chemical change that peptizes, disperses or expands the double layer may eliminate most of the free space and reduce hydraulic conductivity (Rowe *et al*., 1995).

The low hydraulic conductivities often recorded for clayey soils are primarily as a result of restriction of pore space effective in flow by the double layer associated with clay minerals. These soils tend to be sensitive to changes in the chemistry of the pore fluid that influence the thickness of the double layer. Thus, soils containing the more active clay minerals are potentially more vulnerable to chemical attack and incompatibility. The key role the double layer plays in controlling the hydraulic conductivity of clayey soils is demonstrated by Mesri and Olson (1971). In their studies that involved sedimented specimens of kaolinite, illite and montmorillonite with non-polar liquids (Benzene and Carbon tetrachloride, $\varepsilon < 3$), salt solutions (NaCl, $CaCl_2$) and deionized water, Non-polar liquids were used to fully collapse the double layer, whereas salt solutions and deionized water were used to obtain double layer of varying thickness.

Permeating clays with solutions containing aqueous miscible organic solvents has resulted in large increases in hydraulic conductivity (Fernandez and Quigley, 1985; Bowder and Daniel, 1987). This increase in hydraulic conductivity has been attributed primarily to shrinkage and cracking of the clay due to compression of the double layer, allowing clay particles to approach closer to each other by the low dielectric constant ε of pure phase organic compound (typically $\varepsilon < 40$) relative to water ($\varepsilon \sim 80$).

3.5 Reliability Analysis

The first principle of design must be that the design is inherently reliable in the expected environment and taking account of the expected production processes. This requires that the designer in addition to understanding and taking account of the usual design features such as stress, weight, appearance, etc, must also appreciate the range of environmental conditions throughout the life of the product or system and the range of production methods which will be used in its manufacture (O'Connor, 1992; Birolini, 2004). The reliability of a system

(component) is the probability of performing without failure a specified function under given conditions for a specified period. Therefore it represents the probability of survival at time, t. A failure, on the other hand, is the inability of a system (component) to perform its intended function and may occur as a result of defects in the system (component) because of unexpected stresses (Rothbart, 1986; Ireson and Grant, 1988; Oroge, 1991; O'Connor, 1992). Thus, reliability and failure are related (Weissman *et al*., 1994). Reliability analysis embodies the combination of all available probabilistic and statistical techniques as well as the modern science of decision-making. The combination makes the reliability estimate to reflect the quality and quantity of the available information and also provides a consistency with present states of stress analysis, failure analysis and environmental and load analysis (Afolayan, 1992).

3.5.1 Reliability as a probability concept

Reliability is basically a design parameter and must be incorporated in the system at the design stage. One way to quantify reliability during design and to design for reliability is the probabilistic approach. The design variables and parameters are random variables and hence the design methodology must consider them as random variables. The reliability of any system is a function of the reliabilities of its components. In order to analyze the reliability of the system, we have to first understand how to compute the reliabilities of the components.

The basic idea in reliability analysis from the probabilistic design methodology viewpoint is that a given component has certain strength which if exceeded will result in the failure of the components. The factors which determine the strengths of the components are random variables as well as the factors which determine the stresses or loading acting on the component. 'Stress' is used to indicate any agency that tends to induce failure while 'strength' indicates any agency resisting failure. Failure itself is taken to mean failure to

function as intended. It is said to have occurred when the actual stress exceeds the actual strength for the first time.

There are four basic ways in which the designer can increase reliability (Rothbart, 1986; Ireson and Grant, 1988; O'Connor, 1992)

- Increase mean strength. This can be achieved by increasing size, weight, using stronger material, etc.
- Decrease average stress. This can be done by controlling loads or using higher dimensions.
- Decrease stress variations. This variation is harder to control but can be effectively truncated by putting limitation on use conditions.
- Decrease strength variation. The inherent part - to - part variation can be reduced by improving the basic process controlling the process and utilizing tests to eliminate the less desirable parts.

3.5.2 Measures of reliability

3.5.2.1 Factor of safety.

The assessment of risk and safety of engineering systems are traditionally based on allowable factor of safety; these are estimated from previous experience with the behaviour of particular or from observed responses of similar systems. A conventional measure of the safety factor taken engineers is the ratio of the assumed nominal values of capacity x and demand y (Kottegoda and Rosso, 1997; Duncan, 2000).

$$Z = x/y \qquad (3.23)$$

The demand on a system often results from a number of uncertain components such as wind loadings, pollutant loads etc. Because the nominal values of both the capacity x and the demand y cannot be determined with certainty, the capacity and demand functions must be

considered as probability distribution. Hence, the safety factor given by the ratio z = x/y of the two random variables X and Y is also a random variable.

The inadequacy of the system to meet the demand as measured by the probability of failure, associated with that portion of the distribution of the safety ratio wherein it becomes less than unity, is, the portion in which

$$Z = X/Y \leq 1 \tag{3.24}$$

The probability P_f of system failure is thus given by (Kottegoda and Rosso, 1997)

$$p_f = \Pr[Z \leq 1] = F_z(1) \tag{3.25}$$

The corresponding probability of non - failure is

$$r = 1 - p_f = \Pr[Z > 1] = 1 - F_z(1) \tag{3.26}$$

which can be interpreted as survival probability or simply reliability. When the joint probability distribution of X and Y is known, the reliability of the system can be evaluated by determining the cumulative distribution function of X/Y. There is zero probability of failure ($p_f = 0$) and a reliability of 100 percent (r = 1) only if the maximum demand Y_{max} does not exceed the minimum capacity X_{min}, so that the two distributions do not overlap.

3.5.2.2 Safety margin

If the maximum demand Y_{max}. exceeds the minimum capacity X_{min}., the distributions overlap and there is a non zero probability of failure. To assess this probability, one can take the difference between capacity and demand (Kottegoda and Rosso, 1997).

$$S = X - Y \tag{3.27}$$

Eqn. 3.27 is usually referred to as the safety margin. Because X and Y are random, the safety margin is also a random variable.

The inadequacy of the system to meet the demand, as measured by P_f is associated with that portion of the distribution of the safety margin wherein S takes the negative values, i.e. the portion in which

$$S = X - Y \leq 0 \quad (3.28)$$

Thus

$$P_f = P_r[(X - Y) \leq 0] = P_r[S \leq 0], \quad (3.29)$$

and the corresponding reliability is

$$r = 1 - P_f = P_r[(X - Y) > 0] = P_r[S \geq 0]. \quad (3.30)$$

When the joint probability distribution of X and Y is known, the reliability of the system can be evaluated by determining the cumulative distribution function cdf of X – Y.

3.5.2.3 Reliability index

Another measure of the adequacy of an engineering design is the reliability index, defined as

$$\beta = \mu_s / \sigma_s \quad (3.31)$$

This can be interpreted as the number of sigma units (the number of standard deviation σ_s) between the mean value of the safety margin

$$E[S] = \mu_s \quad (3.32)$$

and its critical value

$$S = 0 \quad (3.33)$$

The reliability index of a system, denoted by β is defined as the ratio between the mean and standard deviation of the safety margin of the system.

By definition, the reliability index is the reciprocal of the coefficient of variation of the safety margin, that is $\beta = 1/V_s$ (Kottegoda and Rosso, 1997).

3.5.3 Concept of First – Order Reliability Method (FORM)

The probabilistic and deterministic approaches to design differ in principle. Deterministic design is based on total 'discounting' of the contingency of failure. Design problems involve element of uncertainty: unpredictability or randomness. Probabilistic design is concerned

with the probability that the structure will realize the functions assigned to it (Afolayan and Abubakar, 2003).

If l is the strength capacity and s the loading effect(s) of a system which are random variables, the main objective of reliability analysis of any system or component is to ensure that r is never exceeded by s. In practice, r and s are usually functions of different basic variables. In order to investigate the effect of the variables on the performance of the system, a limit state equation in terms of the basic design variable is required (Afolayan and Abubakar, 2003). This limit state equation is referred to as the performance or state function and expressed as:

$$g(x_i) = g(x_1, x_2, \ldots..x_n) = l - s \quad (3.34)$$

where x_i, for i = 1,2,….. n, represent the basic design variables.

The limit state of the system can then be expressed as

$$g(x_i) = 0 \quad (3.35)$$

Graphically, the line $g(x_i) = 0$ represents the failure surface while $g(x_i) > 0$ represents the safe region and $g(x_i) < 0$ corresponds to the failure region. This is shown in Fig. 3.3

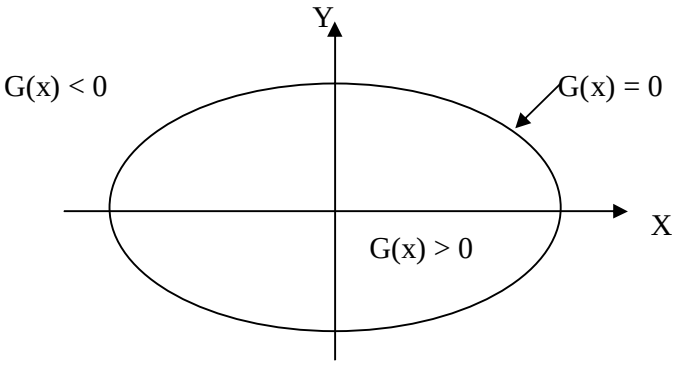


Fig. 3.3 The most likely failure point. (Throft-Christensen and Baker, 1982).

3.5.4 Geotechnical/geoenvironmental reliability analysis

There is currently considerable interest in the application of reliability analysis to geotechnical and geoenvironmental engineering problems (Whiteman, 2001). From a

geotechnical/geoenvironmental perspective, the principal interest lies in the reliability analysis of post construction characterized by hydraulic conductivity.

The reliability is

$$r = P(k < k_o) \tag{3.36}$$

where K_o = the specified hydraulic conductivity limit, such as 10^{-9} m/s.

The objective of the reliability analysis is to establish with reasonable confidence that event $k < k_o$ throughout the useful life or at least some specified design period of the liner.

A reliability analysis for compacted soil liners can be based on post construction performance characterized by either hydraulic conductivity or the travel time of contaminants through the liner. The reliability analysis conducted in this work was based on hydraulic conductivity.

Safety factors are assigned to cater for some level of ignorance. These factors cannot consistently be adequate for all widely varying degrees of uncertainty common in geoenvironmental engineering practice. The reliability analysis yields conditional probabilities of the liner material performing as intended when subjected to given loads.

In the reliability evaluation of compacted soil liners, failure may be defined as the event of a liner hydraulic conductivity equal or greater than the specified regulatory maximum (that is $1x10^{-9}$ m/s) over a given period of time such as the design life. The reliability problem is then formulated in terms of a limit state function G(x), where x is a vector random variables, and $G(x) < 0$ denotes the region in which the threshold value is met or exceeded. Adequate performance of the liner is established when $G(x) > 0$. The limit state surface is denoted by as $G(x) = 0$. Thus, a threshold hydraulic conductivity k_o, the limit state function is of the form (Borgardi *et al*., 1989, 1990):

$$G(x) = \ln k_0 - \ln k_e \tag{3.37}$$

where k_e is the expected hydraulic conductivity and

k_o the regulatory (specified) hydraulic conductivity limt such as 10^{-9} m/s .

Failure is said to occur when k_e exceeds k_o or some other value that is less than the regulatory maximum.

k_e can be represented as (Benson et al., 1994)

$$\ln k_e = Y = x\,\beta^1 + \varepsilon \qquad (3.38)$$

where Y = a random variable describing distribution of $\ln k_e$; x = a vector containing m random variables that describe the spatial distribution of m quality control measurements related to hydraulic conductivity such as the index properties and compaction variables; β^1 = a vector of coefficients and ε = an independent non-zero Gaussian random error term.

CHAPTER FOUR

MATERIALS AND METHODS

4.1 Materials

4.1.1 Soil

The lateritic soil used in this study is a natural reddish brown lateritic soil with inclusions of white mottles collected from a borrow pit in Shika, Zaria, (Latitude $11^{o}15'$ N and Longitude $7^{o}45'$ E), Nigeria at depths of between 1.5m and 2.0m. It belongs to the group of ferruginous tropical soils derived from igneous and metamorphic rocks. In a study of soil samples from this borrow pit; Osinubi (1998) reported that kaolinite is the predominant clay mineral.

4.1.2 Bentonite

The bentonite used in this study is in powdered form and was obtained from a major supplier in Lagos, Nigeria. A sieve analysis of the air dry material showed that the percentage passing BS No. 200 sieve is 92%. It is representative of the typical bentonite available for construction purposes.

4.1.3 Municipal solid waste leachates

Two of the leachate samples designated L1 and L2 for this study were collected from Ahmadu Bello University, Zaria main refuse dumpsite. The dumpsite is located adjacent to a brook and has no form of cover hence the wastes are exposed to direct sun and rainfall. The third sample, L3 was obtained from a dumpsite at samaru, along the Zaria – Sokoto road, Zaria.

4.1.4 Water

Tap and distilled water were collected from Ahmadu Bello University, Zaria.

4.2 Methods

4.2.1 Experimental programme.

Specimens were prepared using 3 variables at the following indicated levels:

(1) Bentonite content: Limited to 10% by dry weight of soil (Gleason *et al.*, 1997; Howell and Shackelford, 1997; Kenny et al., 1992; Daniel, 1987) and stepped at 2.5, 5, 7.5, 10% concentration levels.

(2) Moulding water content: Moulding water content was at 10, 12.5, 15, 17.5, and 20%. The effect of moulding water on the various parameters was determined.

(3) Compactive effort: Four energy levels to simulate compactive effort expected in the field namely Reduced British Standard Light (RBSL), British Standard Light (BSL), West African Standard (WAS), British Standard Heavy (BSH) to represent minimum, medium and high compactive efforts respectively expected in the field were used. The RBSL is equivalent to the reduced Proctor effort described in Daniel and Benson (1990) as well as Benson and Trast (1995) and utilizes 15 blows instead of 27 blows per layer over three layers while the WAS compaction involves using a 4.5kg rammer falling through 45cm onto five layers in a BS compaction mould, each layer receiving ten blows (Osinubi, 1998b). The influence of compactive effort in the mixture was determined.

The particle size distributions were determined for the lateritic soil as well as the soil bentonite mixtures from conventional sieve analysis and sedimentation analysis conducted according to standard procedures in (BS 1377 (1990) and Head (1992).

4.2.2 Compaction tests

The laboratory compaction tests were conducted in accordance with BS 1377 (1990) Test 3 and the West African standard compaction in accordance with the Nigerian General Specification for Road works and Bridges (1970). The soil sample is air-dried and pulverized

sufficiently to run through the No. 4 sieve (4.75 mm). For samples that contained bentonite, the relevant quantities of dry soil and bentonite were mixed to a uniform mixture.

The required percentage of tap water based on the dry weight was added and thoroughly mixed until a uniform consistency is achieved. The specimens were compacted with four compactive efforts at no compaction delays.

4.2.3 Hydraulic conductivity test

The hydraulic conductivity was measured using rigid wall permeameter under falling head condition as recommended by Head (1994). Processed soils were compacted with four energies (RBSL,BSL, WAS and BSH) at moulding water contents ranging from 10 – 20% within the permeameter cells. Each specimen (one hundred in number) was soaked for at least 24 hours in deaired tap water before the commencement of permeation. The permeant liquid was tap water and permeation was terminated when steady condition was reached.

4.2.4 Volumetric shrinkage test

The volumetric shrinkage upon desiccation was measured by extruding compacted cylindrical specimens (one hundred in number) from the compaction moulds and allowing the cylindrical specimens to dry on a laboratory table (as shown in Plate 4.1). The specimens were prepared by compacting processed soils with four energies (Reduced BS light, BS light, WAS and BS Heavy compaction) at moulding water contents ranging from 10 – 20%. Shrinkage was monitored for 28 days by taking average diameter and height to compute volume, hence the volume change.



Plate 4.1 Desiccating samples of compacted lateritic soil bentonite mixtures.

4.2.5 Strength test

The unconfined compression test was carried in accordance with the BS 1377 (1990) Procedure as well as Head, (1994). The test was conducted on specimens (one hundred in number) mixed with tap water and compacted at moulding water contents ranging between 10% and 20% using four compactive efforts namely RBSL, BSL, WAS and BSH compaction. The compacted specimens were stored in cellophane bags and kept in humid environment for at least 48 hours before trimming and testing. The test was performed on cylindrical specimens having a diameter and length of 38mm and 76mm respectively, which were trimmed from the larger compacted cylinders. The samples were tested in triaxial compression test machine without applying cell pressure.

4.2.6 Soil water characteristics curve (Soil suction measurements)

4.2.6.1 Specimen preparation

Specimens of each soil – bentonite mix were prepared at compaction water contents approximately 2% dry of optimum, near optimum water content and approximately 2% wet of optimum water content using RBSL, BSL, WAS, and BSH compactive efforts. Soil specimens for the test were first compacted in the BS compaction mould, extruded and a stainless steel ring with a beveled sharp cutting edge having inside diameter of 50mm and height of 50mm was used to core columns of soil from the specimens. A total of 60 specimens were used to develop soil water characteristic curves. These specimens were soaked in smoking basin and brought to full saturation by capillary action (Within two weeks) and then subjected to pressure plate drying test.

4.2.6.2 Pressure plate drying test

The pressure plate drying test used in this study was conducted to determine the relationships between volumetric water content and matric suction potentials in a soil subjected to pressures ranging between 0 - 1500kPa. A photograph of setup is shown in Plate 4.2. Procedures followed were in accordance with ASTM D3152-72 (ASTM, 1994).

The saturated soil specimens were placed in the pressure plate cell (see Plate 4.3) and pressure was applied to a predetermined value to induce matric suction. Testing was terminated when the outflow stopped indicating that specimens had equilibrated with the applied matric suction. The specimens were removed and their volumetric water content was determined. The procedure was repeated to develop an entire SWCC by subjecting the soil specimens to different pressures (0.1 – 1500kPa). The whole process involving specimen preparation, saturation and pressure application took about 3 months to be completed.



4.2 Pressure plate extractor setup in the laboratory



Plate 4.3 Soil specimens in the pressure plate cell

4.2.7 Compatibility test

To investigate the compatibility of lateritic and lateritic – bentonite mixtures with municipal solid waste leachate, free swell test and hydraulic conductivity tests on specimens containing 0 - 10% bentonite were conducted using MSW leachate as the hydrating/permeant fluid.

Hydraulic conductivity test procedures described previously were followed except that elapsed time was 90 days.

4.2.7.1 Free swell test:

A modified form of free swell test was conducted (Sivapullaiah et al., 1978; NBBRI, 1983; Chen, 1988; ASTM D 5890-02) where 10cc of dry specimens (i.e., lateritic soil, bentonite and lateritic soil – bentonite mixtures) passing BS 425µm was slowly poured into a 100cc graduated cylinder filled with hydrating fluid and noting the swollen volume of the soil after it comes to rest at the bottom.

Test codes call for recording of swell volume at 24 hours. However, test was allowed to continue as swelling continued at the 24 hour termination and at 100 hours, the rate of swell diminished, indicating that equilibrium was nearly achieved. Thus a 100 hour swell time was adopted for all the tests (Narejo and Memon, 1995; Lin and Benson, 2000).

Free swell was computed as follows:

$$\text{Free swell} = \frac{V - V_1}{V_1} \qquad (4.1)$$

where V = soil volume after swelling, cc.

V_1 = soil volume before swelling (10cc)

4.2.8 Batch equilibrium test.

A simple form of the batch equilibrium procedure was adopted. This involved a single batch extraction test where 200ml of MSW leachate solution and 50g of dry lateritic soil or lateritic soil –betonite mixture (1:4 ratio) were mixed and gently agitated in a mechanical shaker for a period of 48 hours. At the conclusion of the 48-hour period, the slurry was decanted and filtered for laboratory analysis namely the measurement of equilibrium/residual concentration

c of the contaminant constituents by Atomic Adsorption Spectrophotometer and the pH. The test was repeated using two different MSW leachate solutions with different initial concentrations (Catwright and Hansel, 1993; Osinubi and Amadi, 2003)

The sorbed concentrations, q, were computed using the expression:

$$q = \frac{(c_i - c_f)V}{M_s} mg/g \qquad (4.2)$$

where c_i = Initial concentration of the specific constituent contaminant mg/l

V = Volume of solution ml

c_f = Equilibrium concentration, mg/l

M_s = The soil mass, g

4.2.9 Diffusion test

The diffusion test followed the procedure outlined by Shackelford and Daniel (1991) for single – reservoir, decreasing source tests. The diffusion experiment was conducted in cells made of PVC material specially designed for this purpose. The cells were sealed on top with transparent plastic sheet and provisions were made for the introduction of leachate solution through a small diameter hose fitted to the plastic cover. In the cells containing compacted specimens, representative leachate solution was placed on top of the saturated soil sample. Diffusion took place under concentration gradient. The soil specimens were extruded and sliced at the end of the test and the concentrations of the pore water of the soil are measured to provide the soil concentration profile.

4.2.9.1 Specimen preparation for diffusion tests

Lateritic soil samples with appropriate bentonite content (0, 2.5%, 5%, 7.5% and 10% by dry weight of soil) were compacted at water content approximately 2% wet of optimum moisture content based on BS Heavy compactive effort. In order to achieve specimen saturation, distilled water was allowed to pond over the compacted soil in the cell for 30 days.

4.2.9.2 Diffusion test procedure

Diffusion test was initiated by first draining the water used in saturating the specimen and then introducing the MSW leachate solution into the cells. All diffusion tests were performed at laboratory temperatures ranging from 19^{o}C to 21^{o}C.

On completion of the diffusion stage, which lasted for 90 days, the cells were disassembled. The soil specimens were carefully extruded from the cell and sectioned at regular interval into slices approximately 2.54mm in thickness to provide data for the concentration profile of specified ions used to determine the effective diffusion coefficient. The water content of each slice of soil was determined by oven drying while the concentrations of ions in the pore fluid of the soil were measured by UNICAM 969 Atomic Absorption Spectrophotometer.

CHAPTER FIVE

DATA ANALYSES AND DISCUSSION OF RESULTS.

5.1 Chemical composition

The chemical composition of the lateritic soil used in this study is summarized in Table 5.1 (After Osinubi, 1998), Aluminum oxide Al_2O_3, Silicon dioxide SiO_2 and Iron oxide Fe_2O_3 were the predominant chemical components of the soil. The soil pH was 6.67 which is an evidence of leaching and the acidic nature of the parent material. The composition of bentonite is presented in Table 5.2. The plasticity Index (PI) indicates that it is a highly plastic material and has a high swelling potential.

The chemical characteristics of leachate solutions used in this research study are presented in Table 5.3.

Table 5.1 Chemical composition of lateritic soil

Oxide	Concentration (%)
CaO	0.28
SiO_2	35.60
Al_2O_3	27.40
Fe_2O_3	24.0
MgO	0.22
SO_3	0.85
Mn_2O_3	2.00
P_2O_5	-
Loss on ignition	146.00

(Adapted from Osinubi, 1998a)

Table 5.2 Oxide composition of bentonite sample

Oxide	Concentration (%)
SiO_2	58.14
Al_2O_3	21.73
Fe_2O_3	2.46
CaO	0.86
MgO	2.42
K_2O	0.52
Na_2O	2.08
TiO_2	1.86
P_2O	-
Loss on ignition	13.28

Table 5.3 Chemical characteristics of leachates used.

Parameters	Concentrations		
	L 1	L2	L3
Electrical Conductivity (µmhos/cm)	1600	3500	5000
Total Dissolved Solids (mg/l)	4200	4020	3800
Hardness (mg/l)	12.76	15.58	10.44
Chemical Oxygen Demand (mg/l)	960	1000	1050
Biological Oxygen Demand (mg/l)	81.0	66.0	42.0
pH	8.4	8.9	8.5
Calcium (mg/l)	32.45	8.32	11.11
Potassium (mg/l)	236.05	254.52	252.84
Sodium (mg/l)	139.86	548.80	723.20
Iron (mg/l)	0.28	0.44	0.75
Lead (mg/l)	0.24	0.25	*
Cr^{4+}, Mg^{2+},Mn, Cu^{2+}, Zn^{2+}, P (mg/l)	*	*	*
Chloride (mg/l)	7.8	24.4	41.0
Sulphate (mg/l)	265.0	348.0	410.0

* - Trace quantity, L1, L2, L3 - Leachate samples 1, 2, and 3 respectively.

Table 5.4 Properties of bentonite used in the study

Property	Value
Liquid Limit (%)	250
Plasticity Index (%)	205
Percent Passing # 200 Sieve	92
Specific Gravity	2.38
Swelling Potential	High

5.2 Particle Size Distribution

The particle size distributions curves of the soil as well as the 4 mixtures are shown in Fig. 5.1. The maximum grain size was limited to particles passing the no. 4 sieve (4.75mm). The particle size distributions are similar and resemble that of the base soil. The sand content (0.063 – 2mm) is between 35 – 40%, fines content (percentage passing no. 200 sieve) ranges from 57-61.86%. Clay content (percentage smaller than 2μm) as determined using sedimentation analysis ranged from 24 – 31% increasing with increase in bentonite content. On the basis of data developed from Atterberg limit test run on portions of samples passing the no. 40 sieve as specified by BS 1377 (1990), the mixtures were classified as CL according to Unified soil classification system and demonstrated measurable increases in plasticity index (PI) and linear shrinkage with the stepped introduction of bentonite into the natural soil. The liquid limit (LL) ranges from 42.2 for the natural soil to 57.0 for sample containing 10% bentonite and the plasticity index (PI) increased from 22.22 to 33.42 (samples fall above A – line on the plasticity chart). The increasing trend in plasticity index (see Fig. 5.2) is expected and is indicative of the increase in clay fraction (bentonite) in the soil samples.

Plasticity index is directly related to the mineralogy of the soil and the clay content. Increase in clay content generally leads to increase in plasticity especially when the primary clay mineral is the more active montmorillonite as found in bentonites. All the mixtures satisfied

the PI design requirement (ie $7 \leq PI \leq 35$) (Daniel, 1991, Daniel, 1993; Benson et al., 1994; Rowe et al., 1995). For plasticity index value greater than 35, excessive shrinkage can be expected (Daniel, 1991). Thus the mixtures have suitable plasticity property to minimize hydraulic conductivity and shrinkage susceptibility as well.

Table 5.5 Properties of soil samples.

Characteristics	**Quantity**				
	Bentonite Content, %				
	0	2.5	5	7.5	10
Natural moisture content (%)	19.60	-	-	-	-
Liquid Limit (%)	42.20	48.0	49.5	53.5	57.0
Plasticity Index (%)	22.22	26.64	28.56	30.72	33.42
Linear shrinkage (%)	5.56	5.98	6.91	8.02	8.73
USCS Classification	CL	CL	CL	CL	CL
Specific gravity	2.60	2.58	2.54	2.51	2.48
pH	6.67	7.2	7.4	7.6	7.9
Colour	Reddish brown				
Dominant Clay Mineral	Kaolinite				

B – Bentonite content.

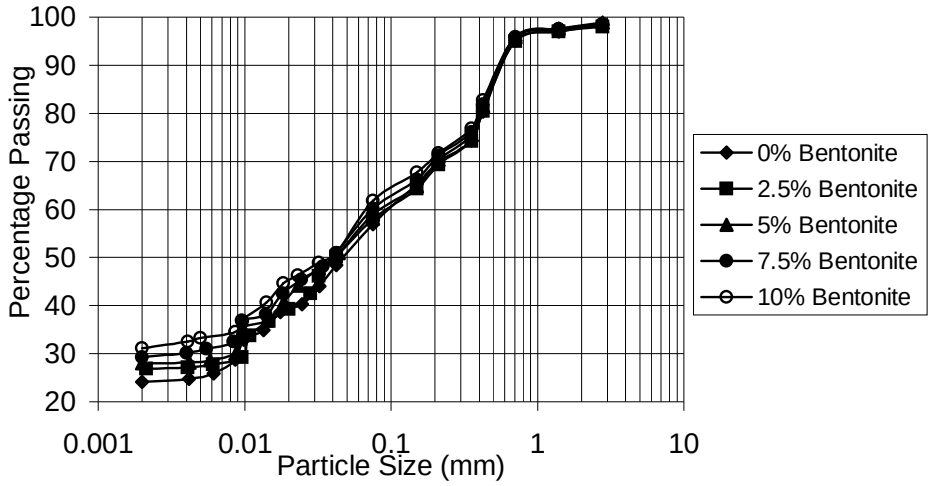


Fig. 5.1 Particle Size Distribution of Soil-Bentonite mixtures and Base soil.

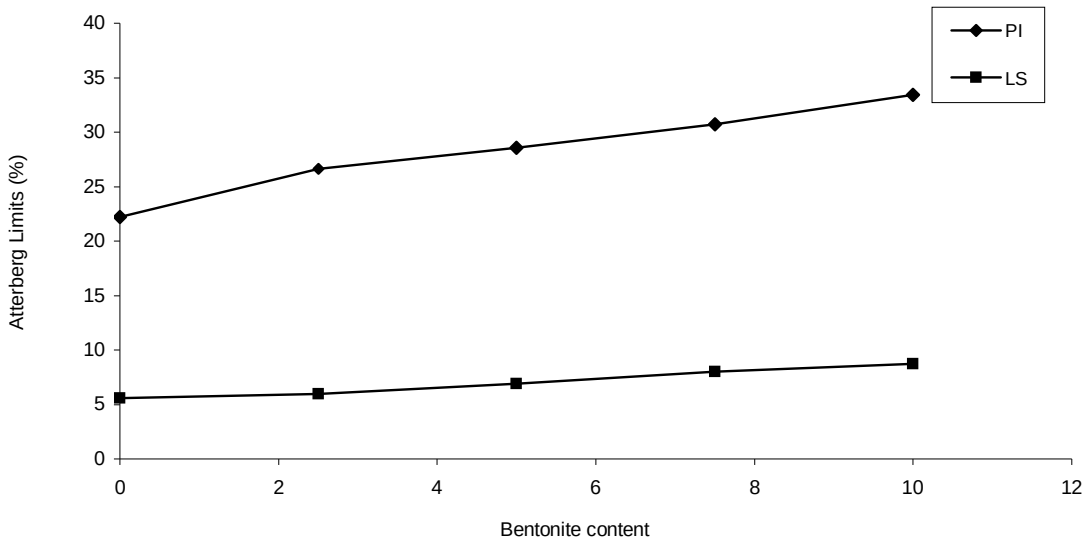


Fig. 5.2 Variation of Plasticity index and Linear Shrinkage with Bentonite content

The activity A (A = PI/Clay fraction) of the samples increased from 0.9 for the natural soil to 1.075 for sample containing 10% bentonite.

The activity A of a soil is an index of the surface activity of the clay fraction. It increased with stepped increase in bentonite content corresponding to larger PI due to greater quantity of highly active montmorillonite mineral present in bentonite. The values of A for the various mixes are all within the range required in containment barriers ie $A \geq 0.3$ (Benson et al., 1994).

The linear shrinkage increased with higher bentonite content from 5.56% for the natural soil to 8.73% for sample containing 10% bentonite as shown in Fig. 5.2.

5.3 Compaction Characteristics

The results of laboratory compaction tests conducted on samples of the natural soil as well as mixtures containing 2.5, 5, 7.5 and 10% bentonite at the four compactive efforts are shown in Figs. 5.3 – 5.7.

Fig. 5.3 Variation of Dry Unit Weight with Moulding water content for 0% Bentonite content

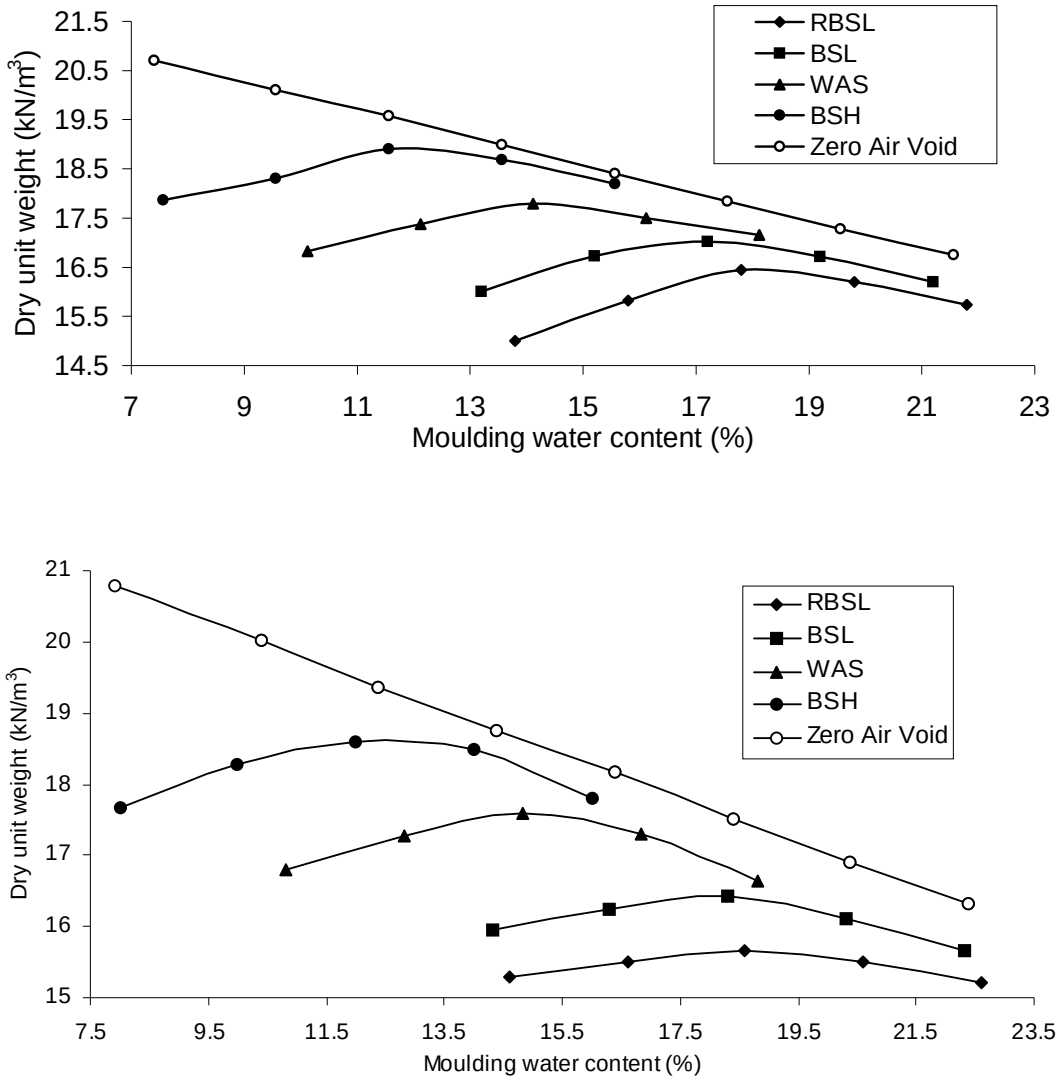


Fig. 5.4 Variation of Dry Unit Weight with Moulding water content for 2.5% Bentonite content

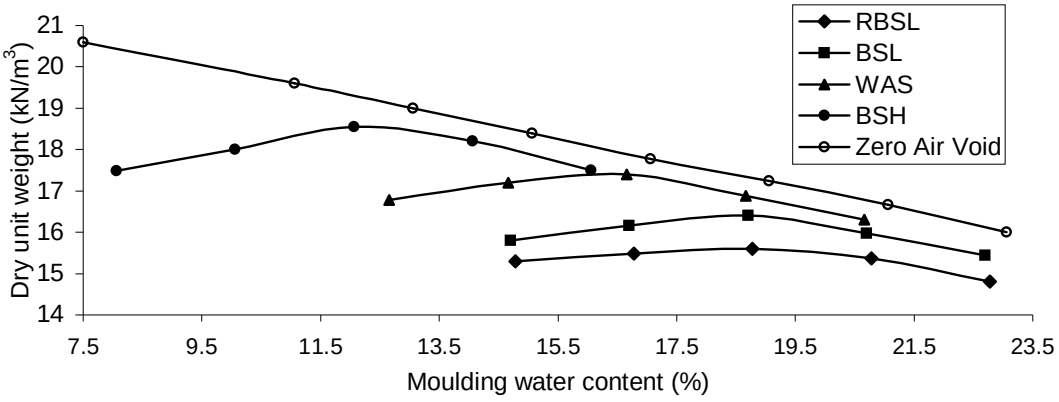


Fig. 5.5 Variation of Dry Unit Weight with Moulding water content for 5% Bentonite content

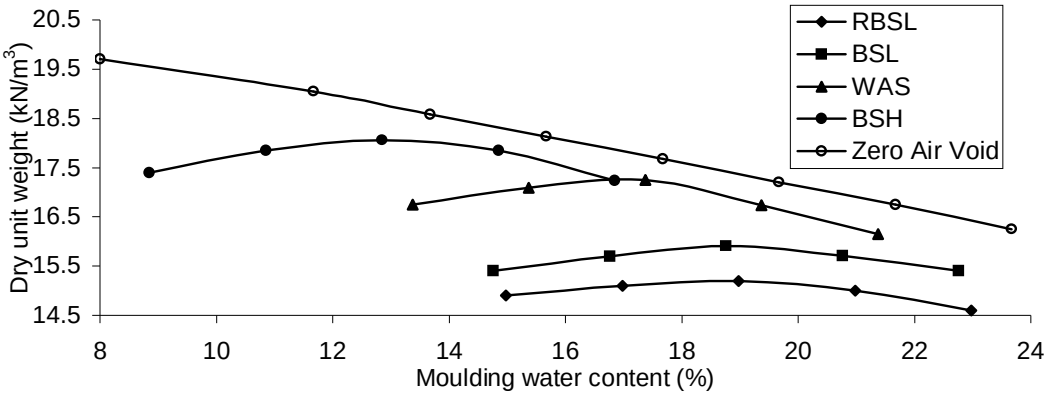


Fig. 5.6 Variation of Dry Unit Weight with Moulding water content for 7.5% Bentonite content

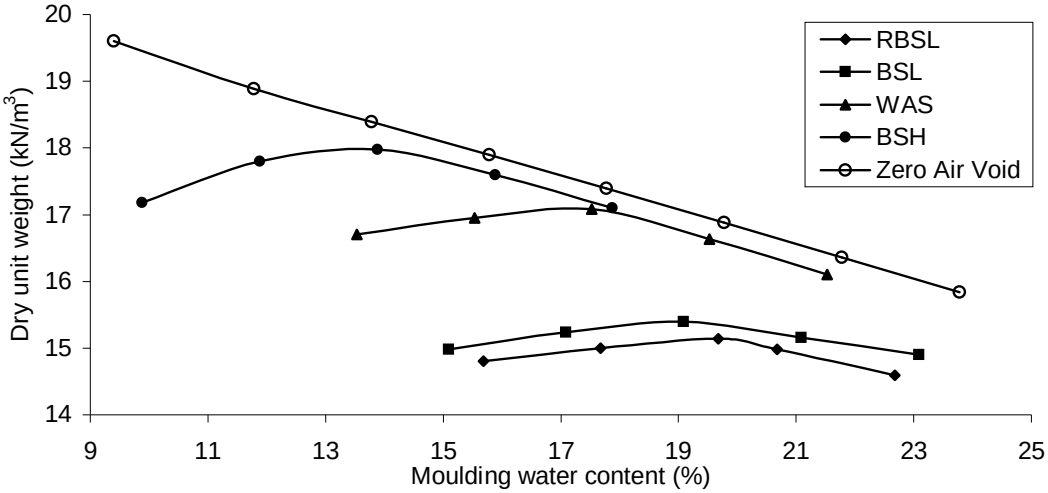


Fig. 5.7 Variation of Dry Unit Weight with Moulding water content for 10% Bentonite content

The compaction curves clearly demonstrate that the dry unit weight is influenced by compaction water, bentonite and compactive efforts. At the dry side of optimum water content of the curves and for each compactive effort, the dry unit weight increased with water content. This is attributed to the development of large water films around the soil particles, which tend to lubricate the particles and make them easier to be moved about and re-oriented into a denser configuration (Holtz and Kovacs, 1981). On the other hand, at the wet side of optimum water content, water starts to replace soil particles in the mass and since the unit weight of water is much less than the unit weight of soil, the dry unit weight decreases with the increasing compaction water.

The addition of bentonite resulted in decreased maximum dry unit weight but the optimum water content increased slightly with increase in bentonite content. The maximum dry unit weight and the corresponding optimum water content for the mixes for all the efforts are summarized in Table 5.6.

Table 5.6 Compaction Characteristics

Compaction Parameters	**Quantity**				
	Bentonite Content, %				
	0	2.5	5	7.5	10
Reduced BS Light Compaction					
Maximum Dry Unit Weight (kN/m^3)	16.45	15.65	15.6	15.20	15.18
Optimum Moisture Content (%)	17.58	18.14	18.46	18.80	18.93
BS Light Compaction					
Maximum Dry Unit Weight (kN/m^3)	17.02	16.43	16.41	15.90	15.40
Optimum Moisture Content (%)	17.50	18.31	18.76	18.84	19.09
West African Standard Compaction					
Maximum Dry Unit Weight (kN/m^3)	17.79	17.60	17.40	17.25	17.08
Optimum Moisture Content (%)	14.12	14.83	16.66	17.37	17.53
BS Heavy Compaction					
Maximum Dry Unit Weight (kN/m^3)	18.91	18.60	18.55	18.05	17.98
Optimum Moisture Content (%)	11.65	12.87	13.76	13.85	13.88

The maximum dry unit weight decreased from 16.45 to 15.18 kN/m^3 for reduced BS Light compaction, 17.02 to 15.4 kN/ m^3 for BS Light compaction, 17.79 to 17.08 kN/ m^3 for West African Standard compaction and 18.91 to 17.98 for BS Heavy compaction. The variation of maximum dry unit weight with bentonite content is shown in Fig. 5.8. As expected, higher dry unit weight was achieved with greater compactive efforts for all bentonite contents. This is because higher compactive efforts yielded a more parallel orientation of the clay particles, which gives a more dispersed structure. The particles become closer and a higher unit weight of compaction results (Das, 1998).

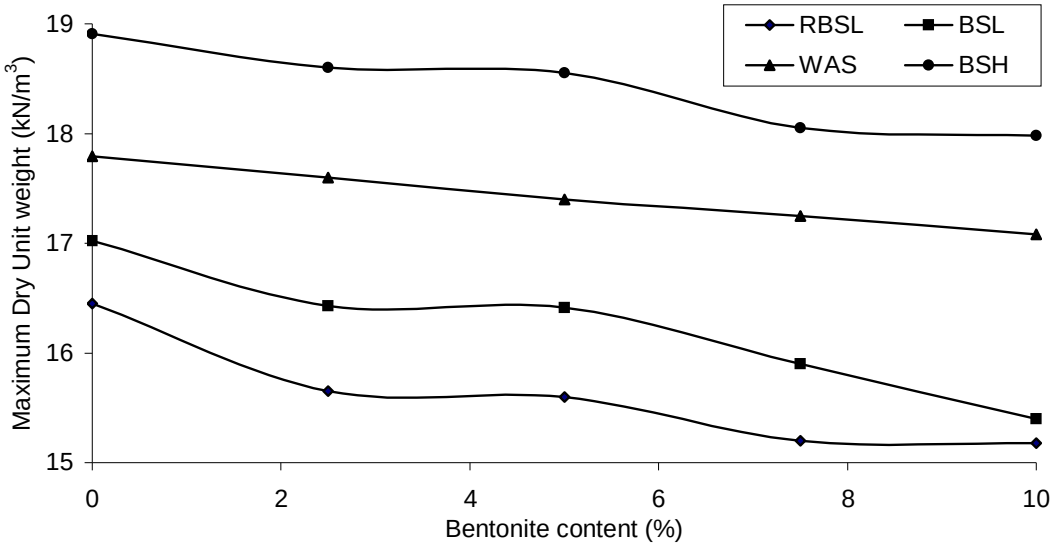


Fig. 5.8 Variation of maximum Dry unit weight with Bentonite content.

The optimum water content falls within the range of 17.58 to 18.93% for RBSL compaction, 17.5 to 19.09% for BSL compaction, 14.12 to 17. 53% for WAS compaction and 11.65 to 13.88% for BSH compaction. The variation of optimum water content with bentonite content for the four efforts used is shown in Fig. 5.9. Typical of fine grained soils, optimum water content is lower for higher compactive efforts. The trends with bentonite content are weaker than those observed for the maximum dry unit weight which suggests that optimum water content is less sensitive to bentonite content.

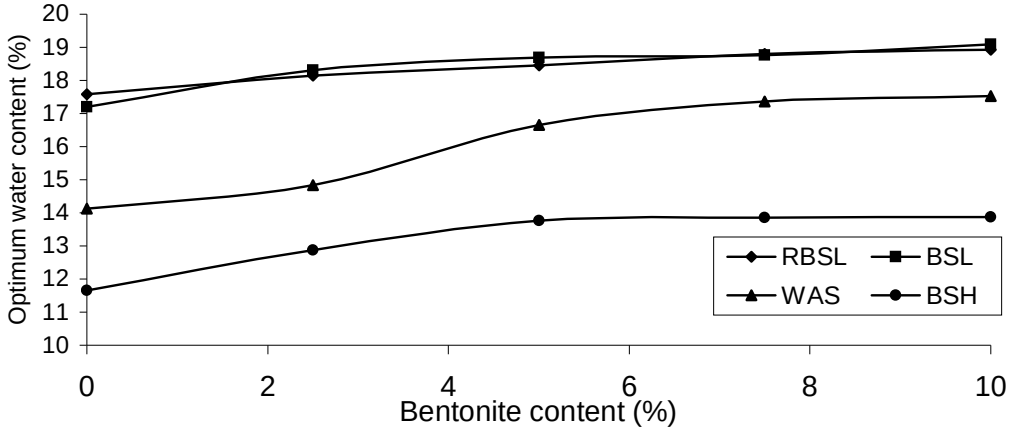


Fig. 5.9 Variation of Optimum moisture content with Bentonite content.

5.4 Geotechnical Design Parameters

5.4.1 Hydraulic conductivity

5.4.1.1 Variation of hydraulic conductivity with moulding water content and compaction energy:

Relationships between hydraulic conductivity and moulding water content for RBSL, BSL, WAS, BSH compaction efforts are illustrated by data shown in Figs. 5.10 – 5.14. In a manner similar to clay soils, increase in compaction water content led to a decrease in hydraulic conductivity, k. The decrease in hydraulic conductivity is attributed to the larger degree of dispersion in soil structure with higher moisture content. For a given compaction energy, the hydraulic conductivity wet of optimum water content, w_{opt}, is significantly lower than the hydraulic conductivity dry of optimum water content (by an order of magnitude in most cases). The lower hydraulic conductivity wet of optimum was explained by Mitchell *et al*., (1965) on the basis of a microscale effect in which a lower soil shear strength results in larger induced shear strain during compaction and reorientation of the clay particles creating smaller interparticle pores. At similar compaction water contents, lower hydraulic conductivities are generally obtained with higher compaction effort. Increase in compaction energy results in lower hydraulic conductivity values for two reasons: (i) a larger compaction energy results in denser soils and smaller void ratios with correspondingly lower k values and (ii) a larger

compaction energy also helps to destroy large inter-aggregate pores or micropores (Holtz and Kovacs, 1981; Mtchell, 1993).

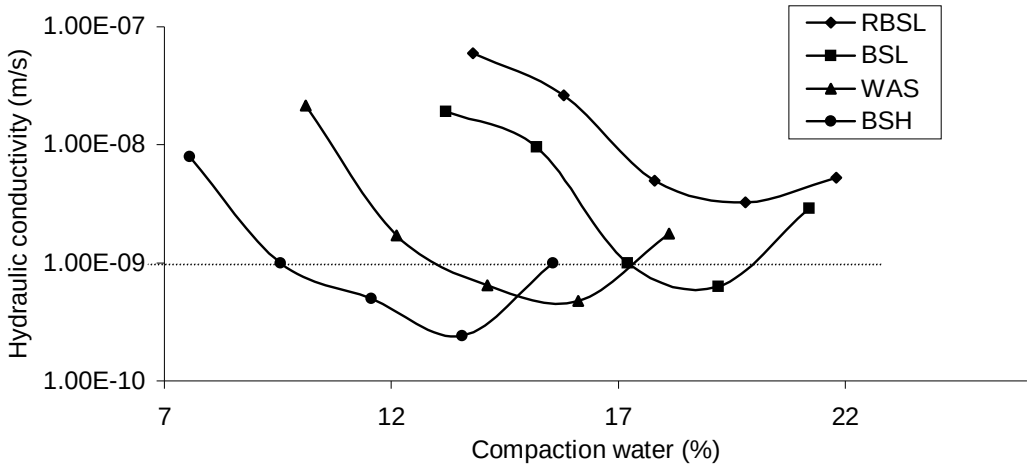


Fig. 5.10 Variation of Hydraulic conductivity with moulding water content for natural soil.

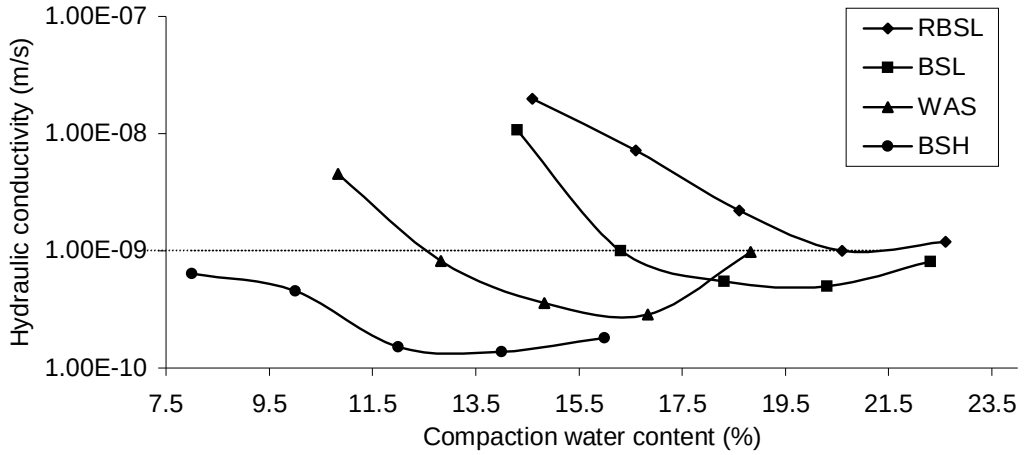


Fig. 5.11 Variation of Hydraulic conductivity with moulding water content soil specimen with 2.5% Bentonite content.

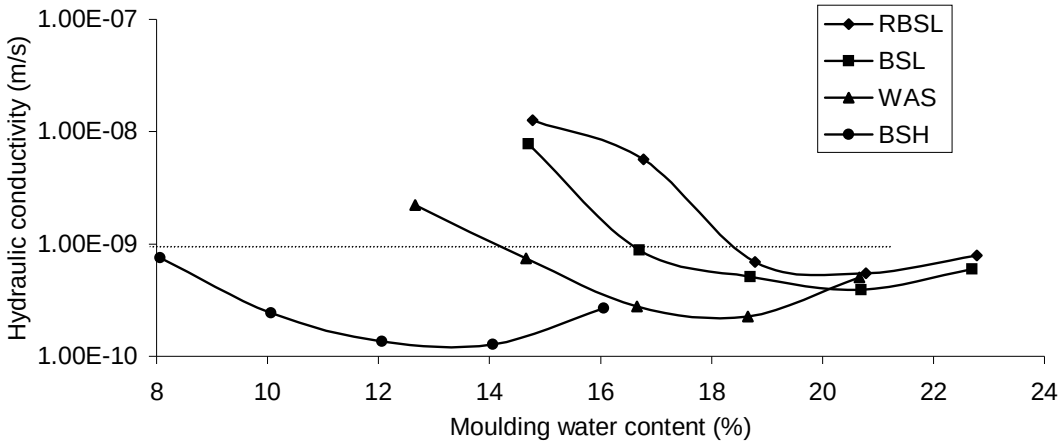


Fig. 5.12 Variation of Hydraulic conductivity with moulding water content soil specimen with 5% Bentonite content.

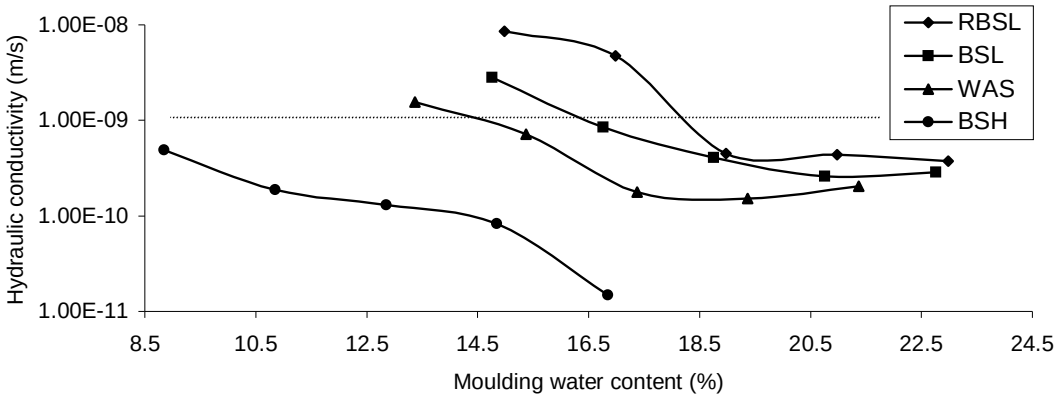


Fig. 5.13 Variation of Hydraulic conductivity with moulding water content soil specimen with 7.5% Bentonite content.

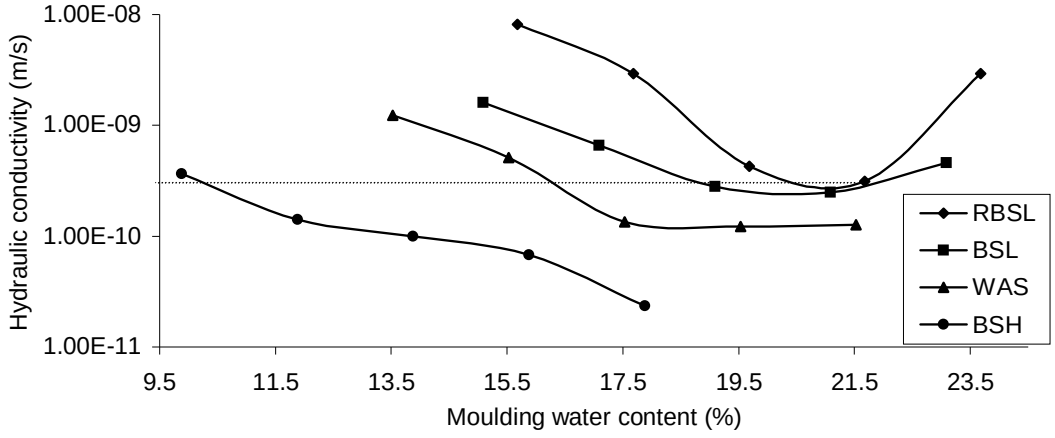


Fig. 5.14 Variation of Hydraulic conductivity with moulding water content soil specimen with 10% Bentonite content.

5.4.1.2 Variation of hydraulic conductivity with bentonite content

Variation of hydraulic conductivity at optimum, dry and wet of optimum water content with bentonite content is shown in Figs. 5.15-5.17 for specimens prepared with reduced BSL, BSL, WAS, BSH compaction efforts. These results indicate that a decrease in hydraulic conductivity was achieved with increase in bentonite content. For samples prepared at bentonite content >2.5% with reduced BSL, BSL, and WAS compaction efforts, little reduction in hydraulic conductivity was observed while samples prepared with BSH decreased progressively with increasing bentonite content.

Samples containing bentonite (except for those prepared with RBSL) have low hydraulic conductivity required for the construction of landfill barriers i.e $k \leq 10^{-9}$m/s. The large specific surface of montmorillonite (the primary clay mineral in bentonite) combined with its large net negative charge resulted in adsorption of a large number of hydrated cations as well as adsorption of water molecules, and interlayer separation during hydration (Grim, 1968; Mitchell, 1993). These adsorbed water molecules and cations can comprise a significant fraction of the pore space, and are essentially immobile (Mesri and Olson, 1971; Mitchell, 1993). As a result, the portion of the pore space occupied by bulk water (ie non-adsorbed water that is free to flow) is relatively small and the pathways formed by the bulk water have irregular shapes (Mesri and Olson, 1971; Anandarajah, 2003). Consequently, significant decrease in hydraulic conductivity values ranging from 5.98×10^{-8} to 3.12×10^{-10} m/s for RBSL, 1.91×10^{-8} to 2.4×10^{-10} m/s for BSL, 2.14×10^{-8} to 1.22×10^{-10} m/s for WAS and 7.91×10^{-9} to 1.49×10^{-11} m/s for BSH compactive effort as the percentage of bentonite increased from 0 to 10% was observed. These results are similar to those obtained by a number of researchers (eg Kenny *et al*., 1992; O'Sadnick *et al*., 1995; Shackelford *et al*., 2001; Osinubi and Nwaiwu, 2005; Osinubi and Eberemu, 2006) for other soil bentonite/slag mixtures.

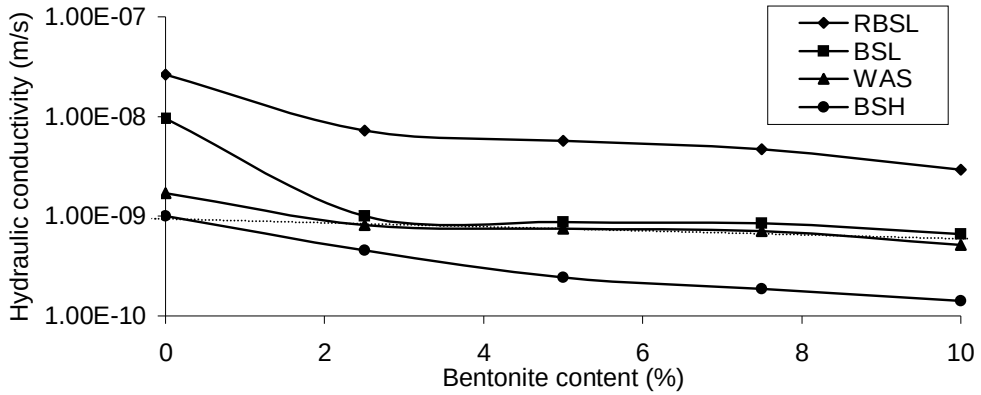


Fig. 5.15 Variation of hydraulic conductivity with bentonite content at 2% dry of optimum water content.

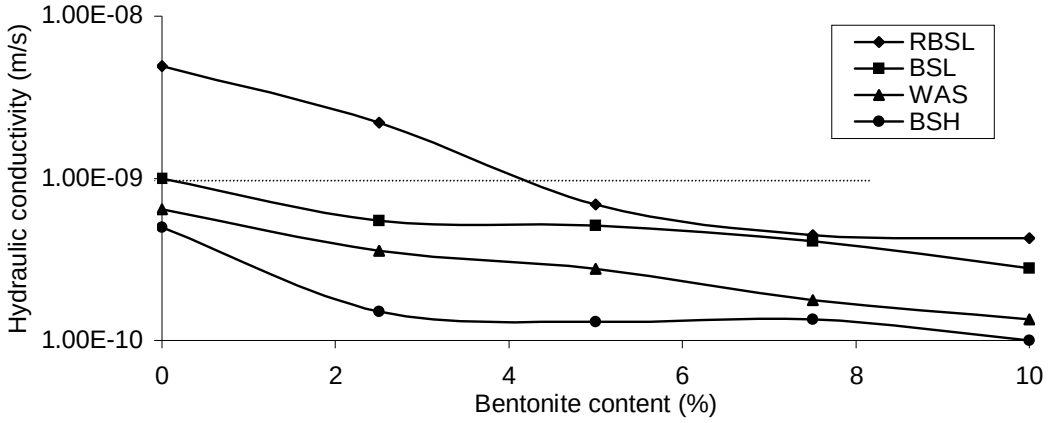


Fig. 5.16 Variation of hydraulic conductivity with bentonite content at optimum water content.

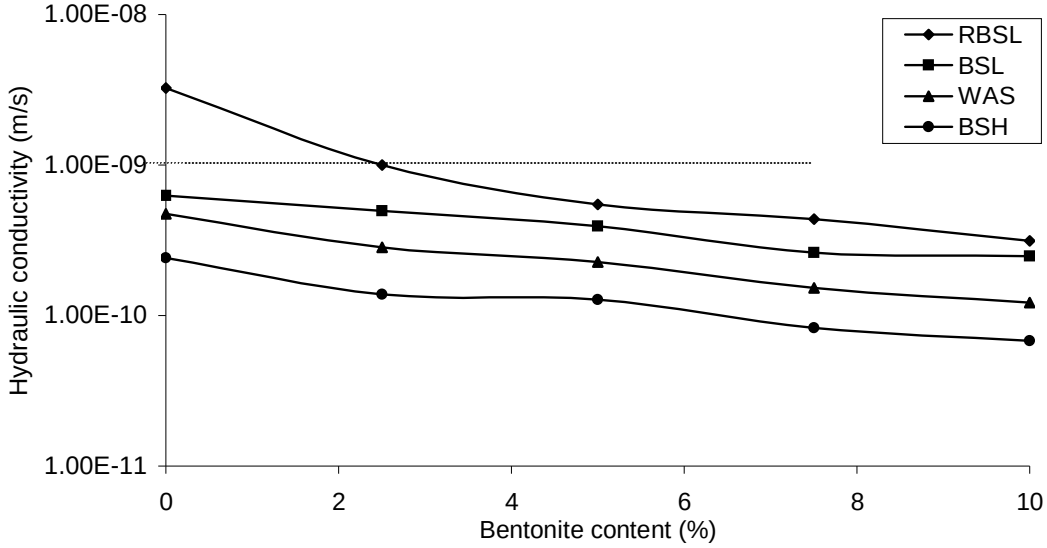


Fig. 5.17 Variation of hydraulic conductivity with bentonite content at 2% wet of optimum water content.

5.4.1.3 Variation of hydraulic conductivity with initial degree of saturation:

The decrease in hydraulic conductivity as initial saturation S_i (i.e saturation at compaction) increased for the various soil bentonite mixtures are shown in Figs. 5.18-5.22. This can be achieved by compacting specimens to higher dry unit weight (using higher compactive effort) or increasing the compaction water content (Benson and Trast, 1995). Generally, specimens compacted to higher initial saturation are mainly those compacted on the wet side of optimum water content and have low hydraulic conductivities which vary by an order of magnitude or more. Thus, the trend suggests that providing compaction states which result in higher initial saturation (i.e., higher compactive effort and moulding water content) will yield lower hydraulic conductivity.

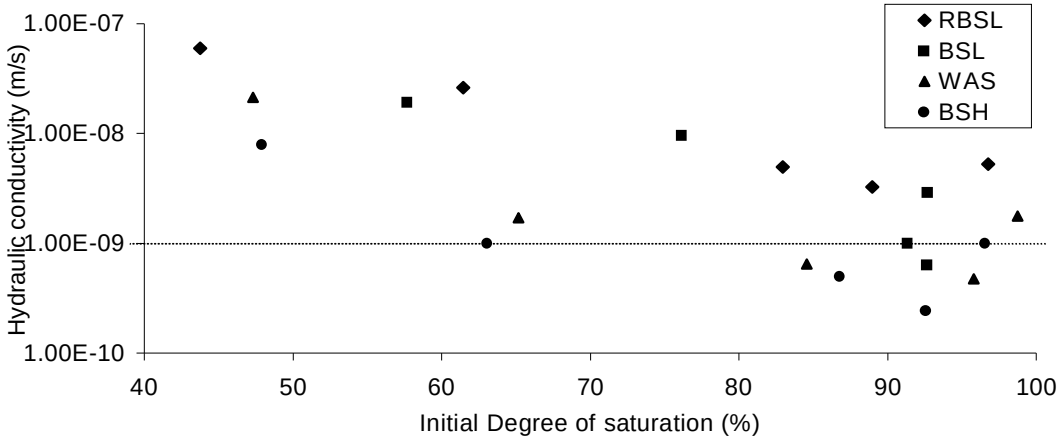


Fig. 5.18 Variation of hydraulic conductivity with initial degree of saturation for 0% bentonite content.

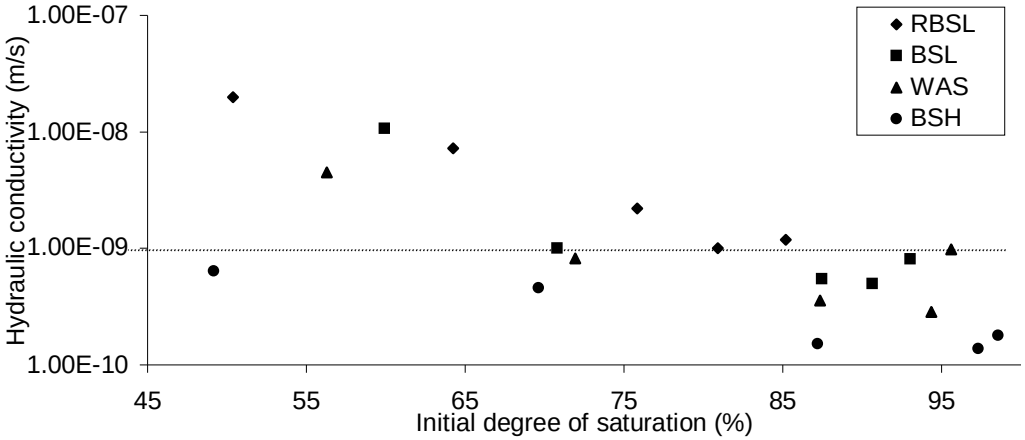


Fig. 5.19 Variation of hydraulic conductivity with initial degree of saturation for 2.5% bentonite content.

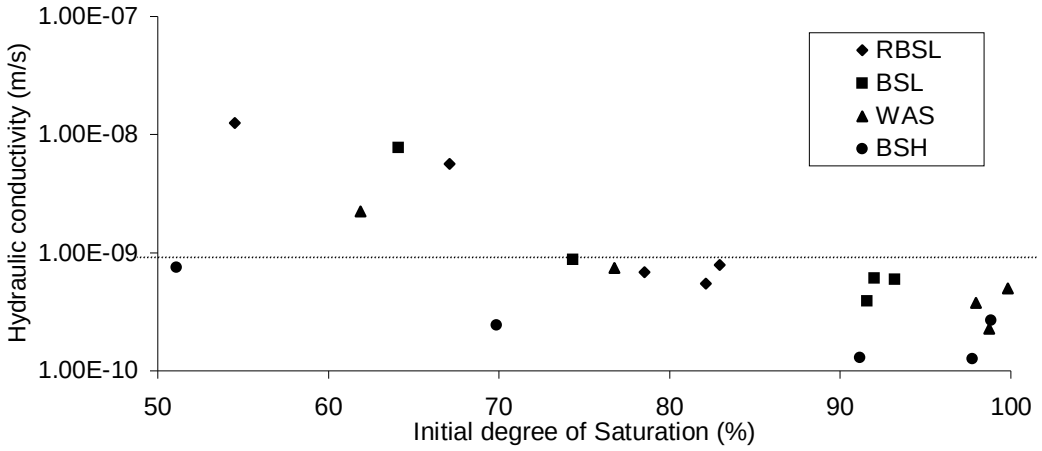


Fig. 5.20 Variation of hydraulic conductivity with initial degree of saturation for 5% bentonite content.

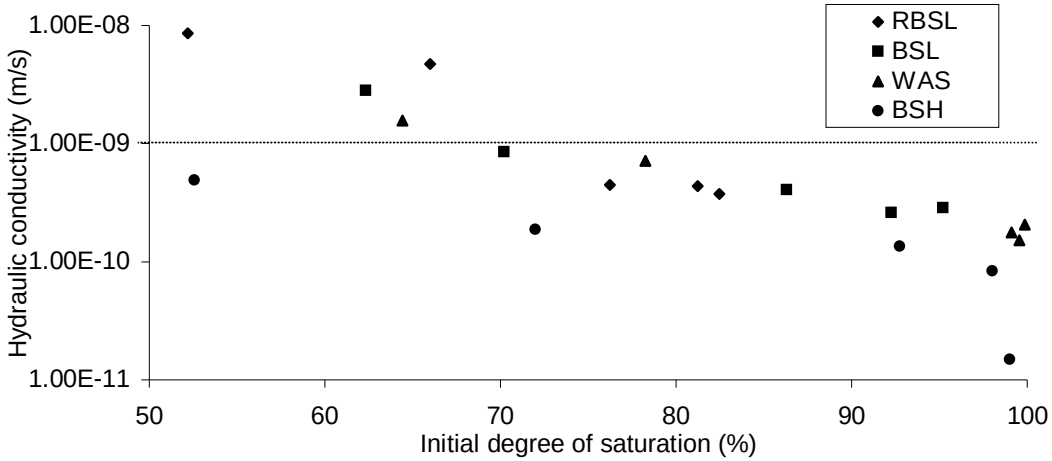


Fig. 5.21 Variation of hydraulic conductivity with initial degree of saturation for 7.5% bentonite content.

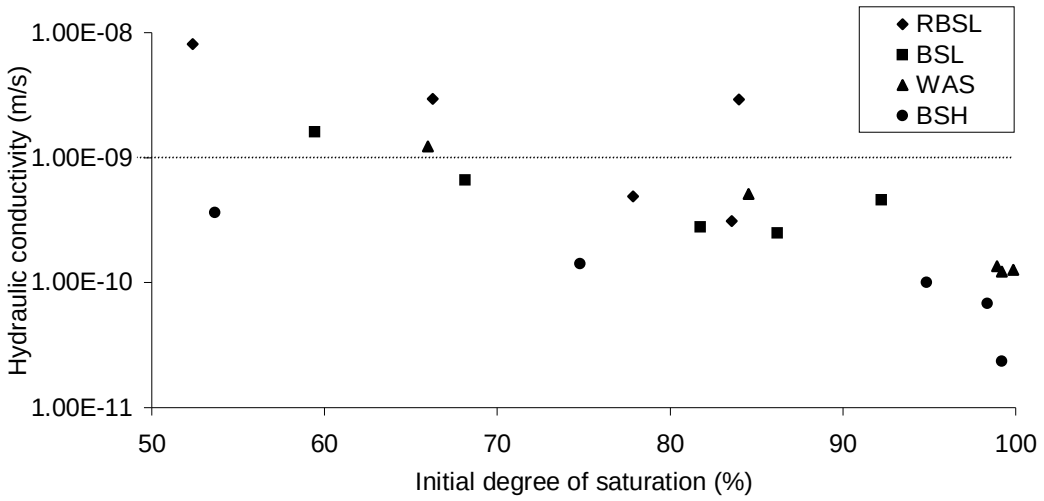


Fig. 5.22 Variation of hydraulic conductivity with initial degree of saturation for 10% bentonite content.

5.4.1.4 Variation of hydraulic conductivity with dry unit weight:

The hydraulic conductivity of test specimens is shown to decrease significantly with higher dry unit weight (see Figs. 5.23 – 5.27), hence lower hydraulic conductivities were recorded with hifher dry unit weight, γ_d. This trend was expected because specimens were prepared at optimum, dry and wet of optimum water content and at varying concentrations of bentonite. The upper points which correspond to higher hydraulic conductivities indicate dry unit weights obtained on the dry side of optimum moisture content while the lower points having lower hydraulic conductivities indicate dry unit weights obtained on the wet side of optimum content. Thus to achieve hydraulic conductivity less than 1×10^{-9} m/s as required by most regulatory agencies, it is necessary to specify dry unit weight value on the wet side of optimum moisture content.

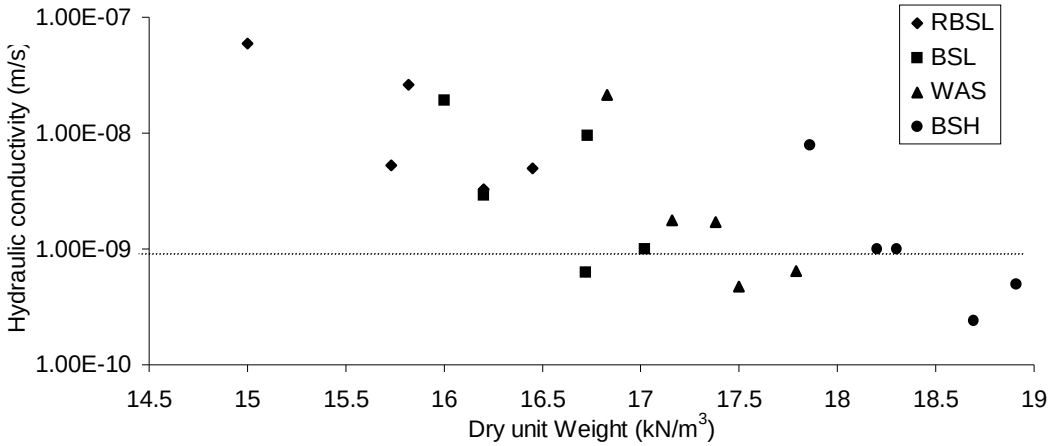


Fig. 5.23 Variation of hydraulic conductivity with dry unit weight for 0% bentonite content.

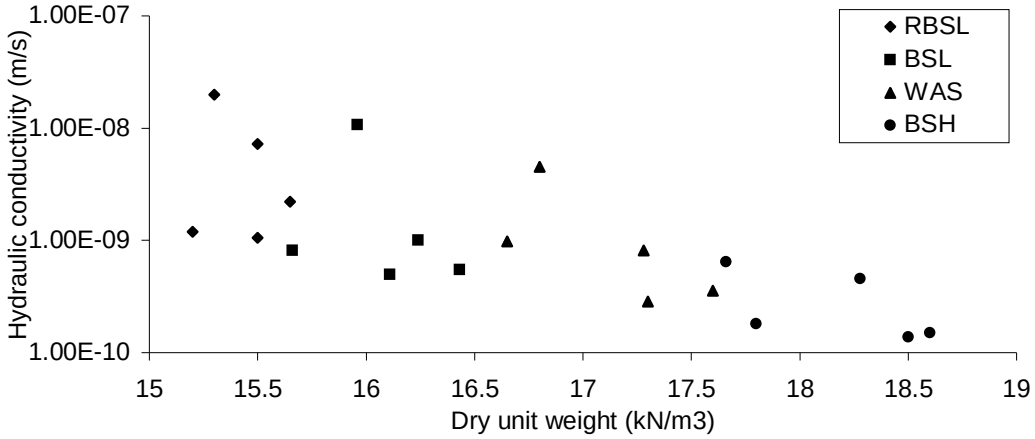


Fig. 5.24 Variation of hydraulic conductivity with dry unit weight for 2.5% bentonite content.

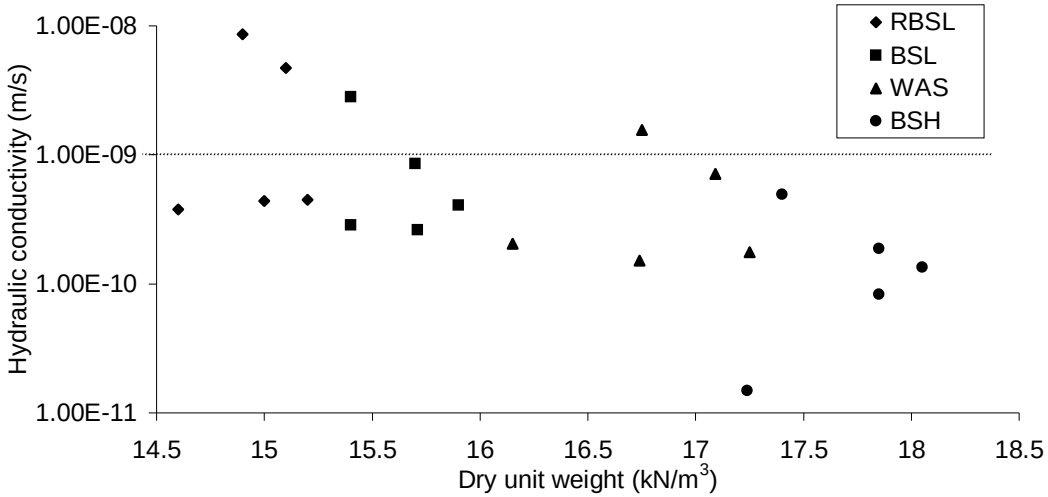


Fig. 5.25 Variation of hydraulic conductivity with dry unit weight for 5% bentonite content.

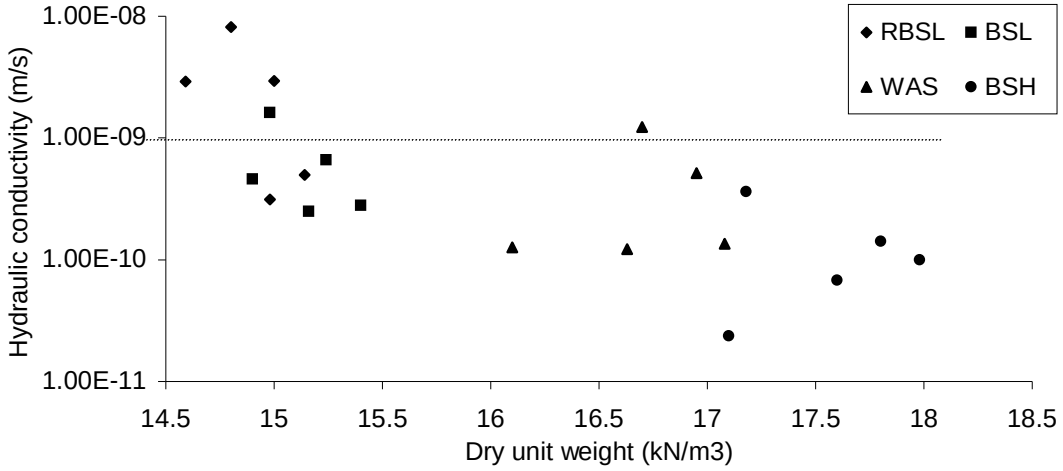


Fig. 5.26 Variation of hydraulic conductivity with dry unit weight for 7.5% bentonite content.

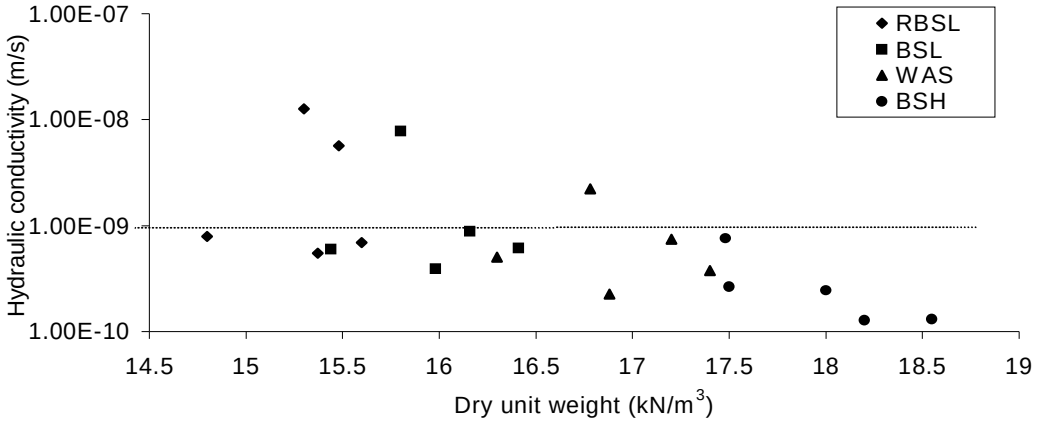


Fig. 5.27 Variation of hydraulic conductivity with dry unit weight for 10% bentonite content.

5.4.2 Volumetric Shrinkage Strain Analyses

5.4.2.1 Variation of volumetric shrinkage with water content and compactive effort

The effect of compaction water and compactive effort on volumetric shrinkage strain (VSS) for the various soil bentonite mixtures are shown in Figs. 5.28 – 5.32. All mixtures exhibited volumetric shrinkage upon air-drying with the amount of shrinkage increasing with higher compaction water content. VSS for samples compacted at water content wet of optimum (2%) are consistently higher than for those compacted dry of optimum or at optimum.

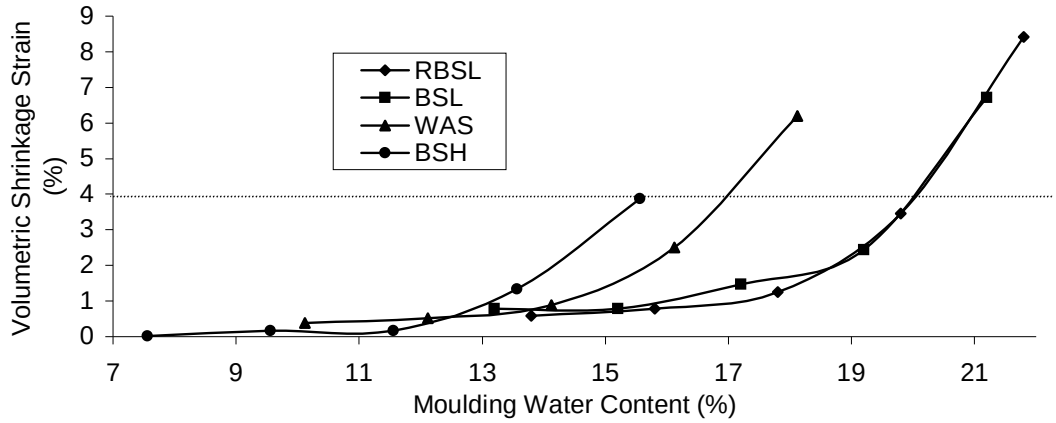


Fig. 5.28 Variation of volumetric shrinkage strain with moulding water content for soil specimen with 0% bentonite content.

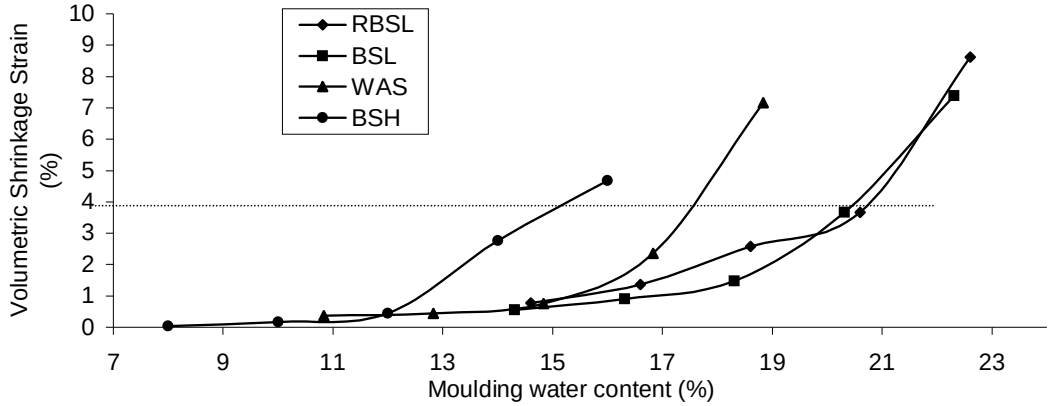


Fig. 5.29 Variation of volumetric shrinkage strain with moulding water content for soil specimen with 2.5% Bentonite content.

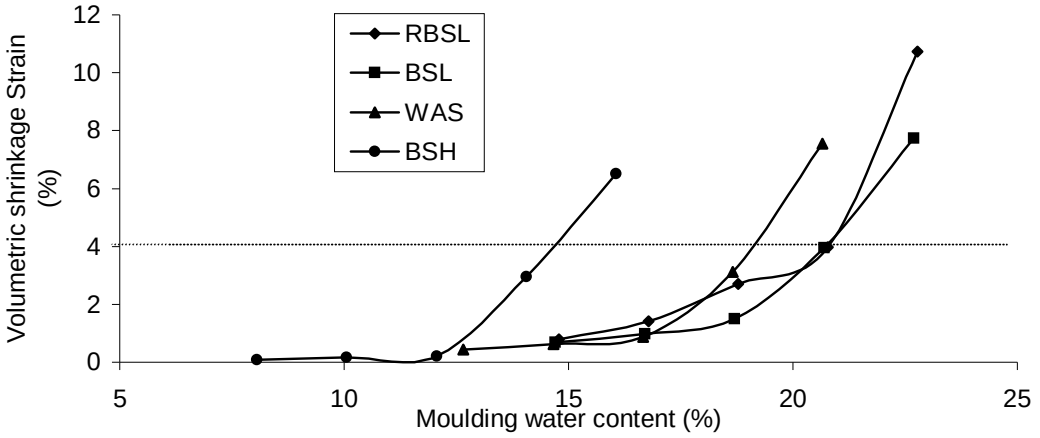


Fig. 5.30 Variation of volumetric shrinkage strain with moulding water content for soil specimen with 5% bentonite content.

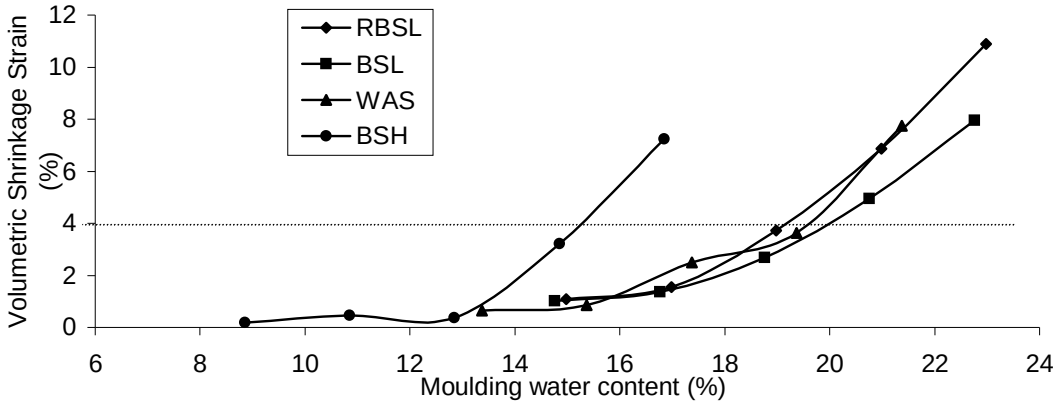


Fig. 5.31 Variation of volumetric shrinkage strain with moulding water content for soil specimen with 7.5% bentonite content.

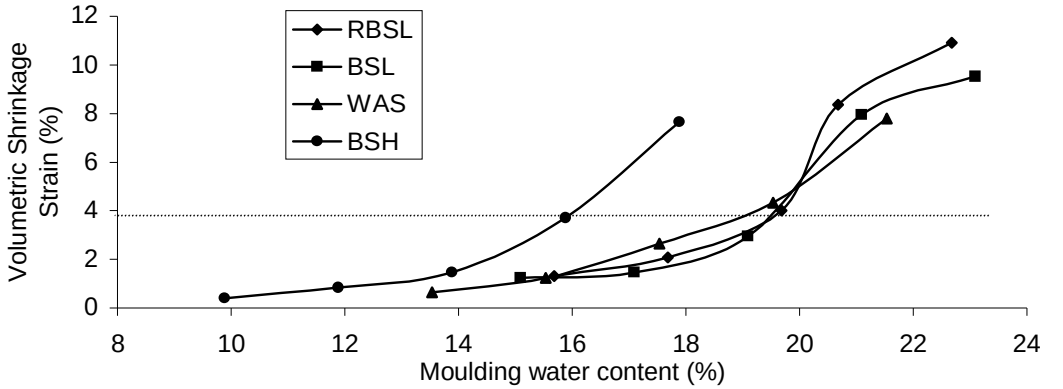


Fig. 5.32 Variation of volumetric shrinkage strain with moulding water content for soil specimen with 10% bentonite content.

The specimens compacted wet of optimum contain more water and therefore will undergo larger volume changes during drying relative to specimens compacted dry of optimum or at optimum water content. These results are in agreement with the results of Daniel and Wu (1993), Albrecht and Benson (2001), Osinubi and Nwaiwu (2006, 2008) as well as Osinubi *et al*. (2006a). At any moulding water content, mixtures containing 10% bentonite shrink more than those with lesser bentonite content, but the shrinkage is insensitive to the compaction effort.

5.4.2.2 Variation of volumetric shrinkage with bentonite content

The tendency of a soil to form desiccation cracks is directly related to the shrinkage strains and associated stresses developed within the soil mass during air drying. The data pertaining to the effect of bentonite content on volumetric shrinkage is shown in Figs. 5.33 – 5.35. Compacted samples showed different volumetric shrinkage properties according to bentonite content. Figs. 5.33 – 5.35 demonstrates trends in the increased in shrinkage strain as bentonite content increased for samples compacted dry of optimum, wet of optimum and at optimum. This implies that desiccation induced shrinkage effects such as cracking will present problems at higher bentonite contents and should therefore be strictly monitored. Volumetric shrinkage in samples increased non linearly with bentonite content as well as plasticity index.

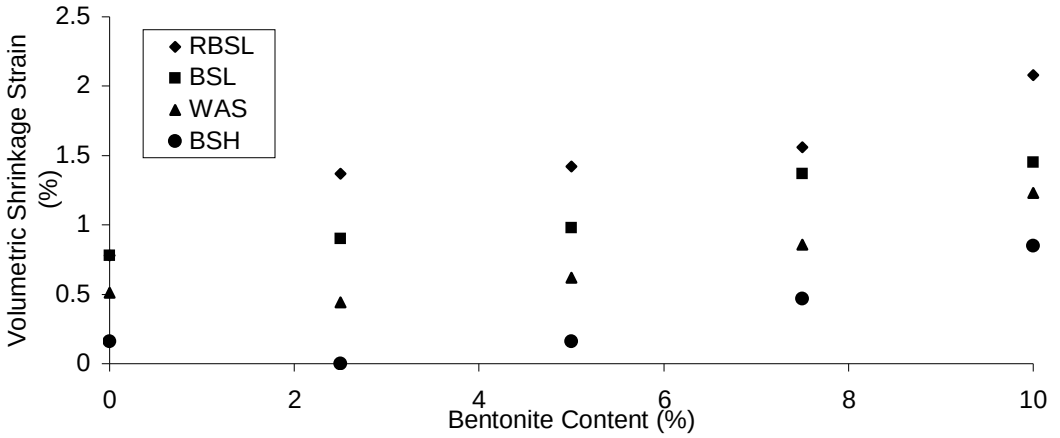


Fig. 5.33 Variation of volumetric shrinkage strain with bentonite content at 2% dry of optimum water content.

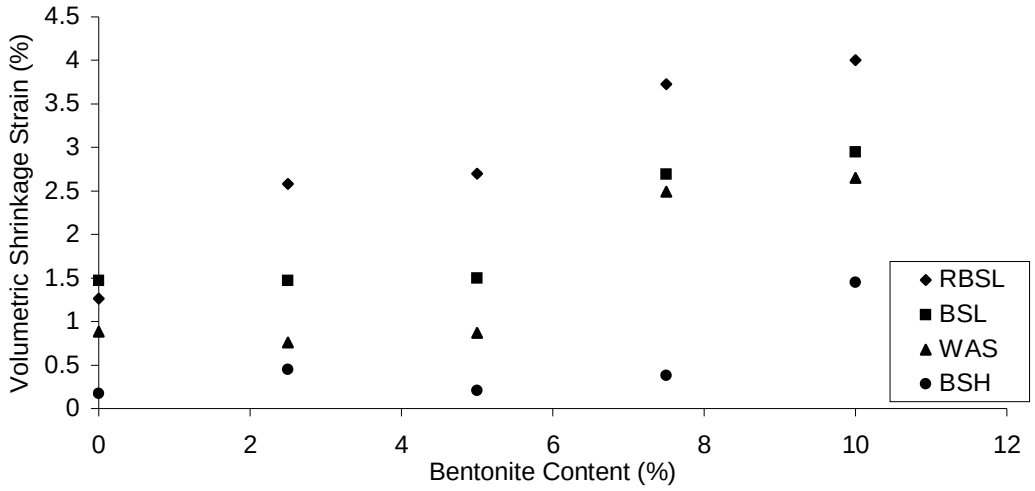


Fig. 5.34 Variation of volumetric shrinkage strain with bentonite content at optimum water content.

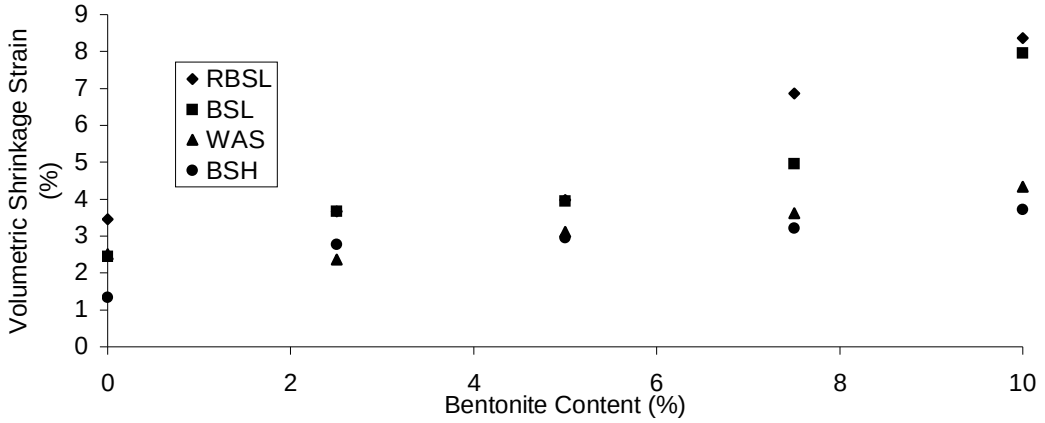


Fig. 5.35 Variation of volumetric shrinkage strain with bentonite content at 2.5% wet of optimum water content.

These increases are associated with changes in kaolinite/montmorillonite ratio, which are the dominant clay minerals in lateritic soil and bentonite respectively. Relatively high values up to 9% were observed in samples containing 10% bentonite. The increase in volumetric shrinkage resulting from higher bentonite content was due to increase in plasticity index, hence a greater affinity for water. Consequently, samples with higher bentonite content or plasticity index invariably have greater quantity of water to be removed during drying

especially at the primary stage of the process. Compacted samples for all levels of bentonite content exhibited no visible desiccation cracking.

5.4.2.3 Changes in Mass of desiccating samples of lateritic soil –bentonite mxtures with time

The change in mass of desiccation samples of lateritic soil treated with various levels of bentonite content compacted to reduced BSL, BSL, WAS and BSH at dry, wet and at optimum moisture content i s shown in Figs 5.36 – 5.40. The pattern of variation in mass are essentially identical in all the cases showing no significant change in mass after the first seven days of drying. Subsequent measurements showed little loss in mass because at this stage, water surrounding the soil particles has been removed, allowing the particles to move closer and eventual entry of air in the soil.

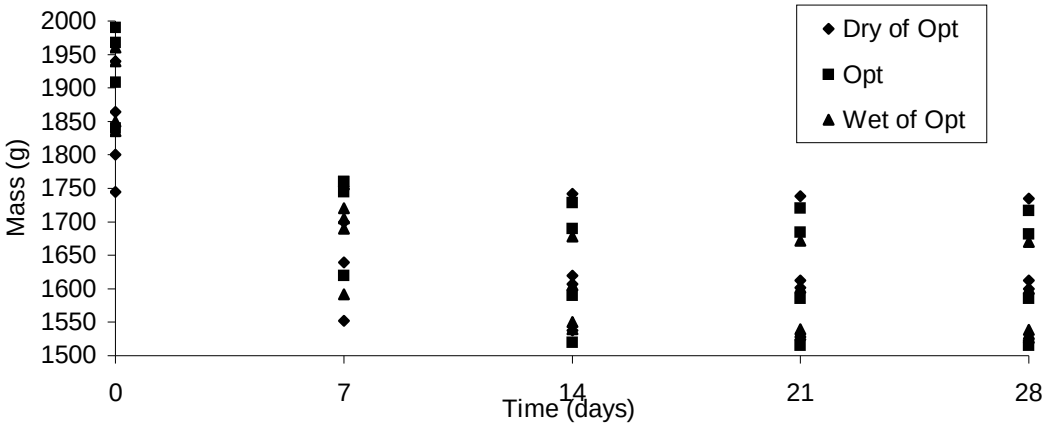


Fig. 5.36 Variation in mass of desiccating samples containing 0% bentonite with time

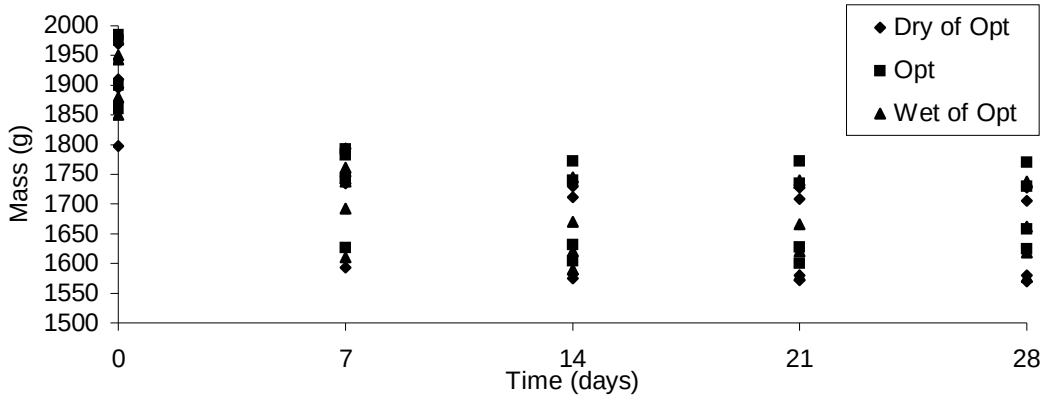


Fig. 5.37 Variation in mass of desiccating samples containing 2.5% bentonite with time

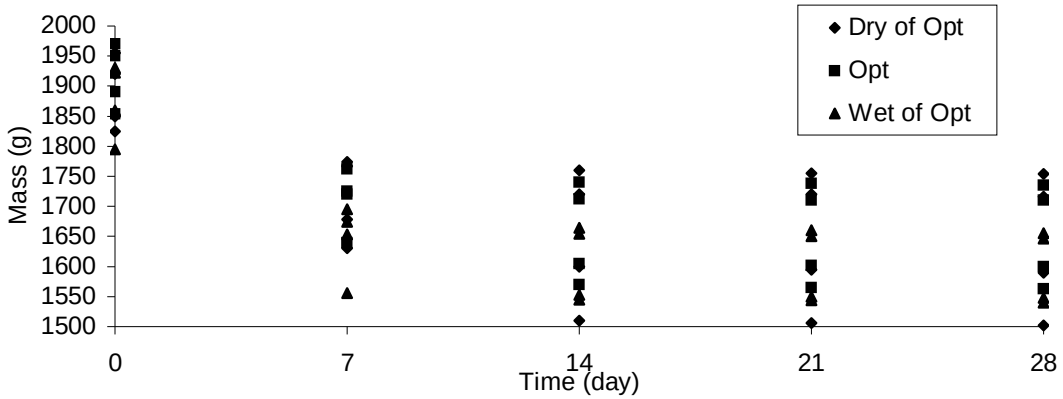


Fig. 5.38 Variation in mass of desiccating samples containing 5% bentonite with time

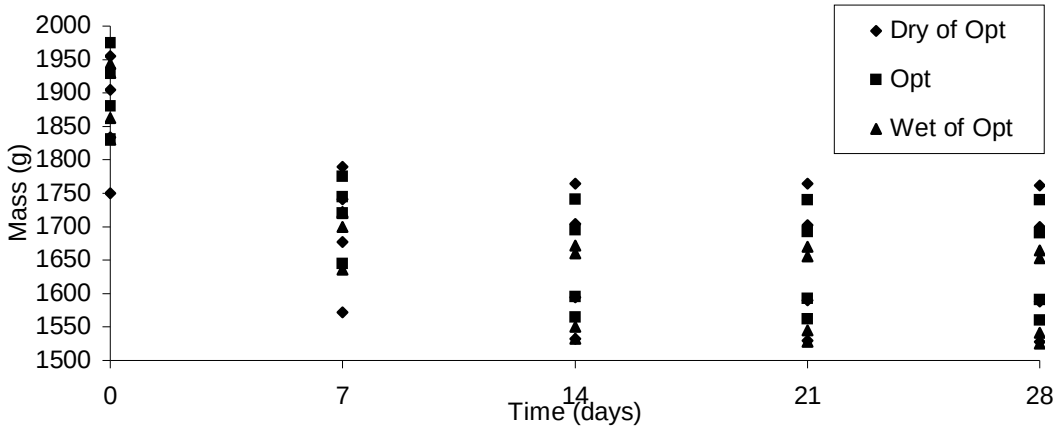


Fig. 5.39 Variation in mass of desiccating samples containing 7.5% bentonite with time

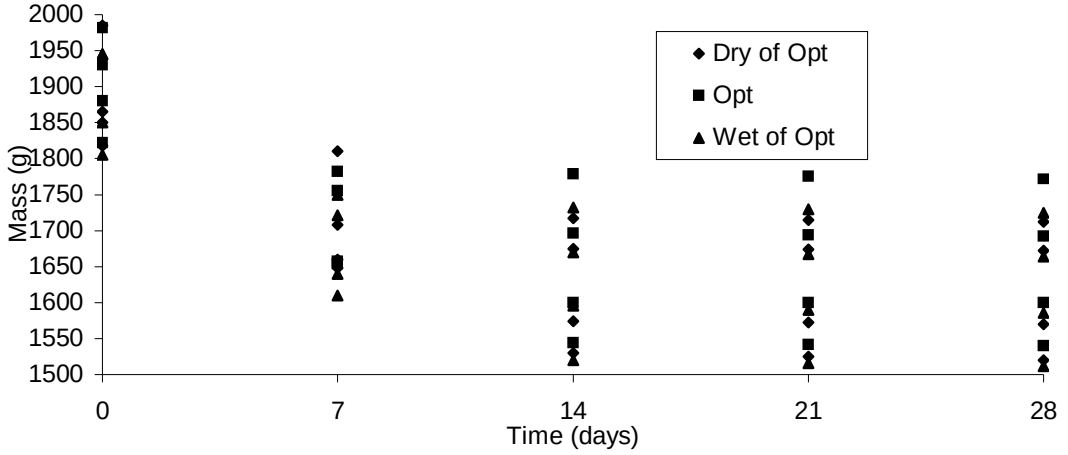


Fig. 5.40 Variation in mass of desiccating samples containing 10% bentonite with time

5.4.2.4 Variation of volumetric shrinkage strain with dry unit weight

The variation of volumetric shrinkage strain (VSS) with dry unit weight for the various mixtures is shown in Figs. 5.41 – 5.45. The VSS of specimens of varying bentonite content showed little or no relationship or trend with the dry unit weight which suggests that volumetric shrinkage is not sensitive to dry unit weight while dry unit weight is a poor indicator of shrinkage. Samples compacted dry or wet of optimum water content at the same dry unit weight have different VSS (Albrecht and Benson, 2001).

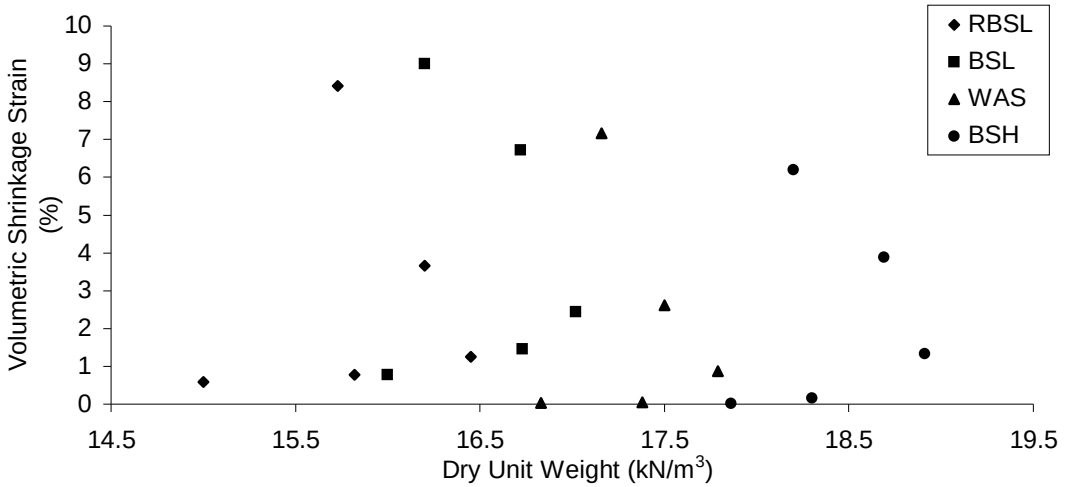


Fig. 5.41 Variation of volumetric shrinkage strain with dry unit weight for 0% bentonite content.

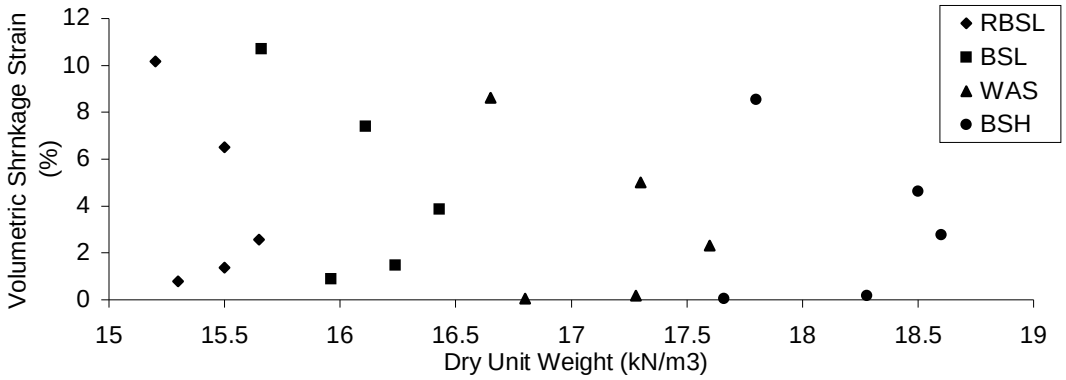


Fig. 5.42 Variation of volumetric shrinkage strain with dry unit weight for 2.5% bentonite content.

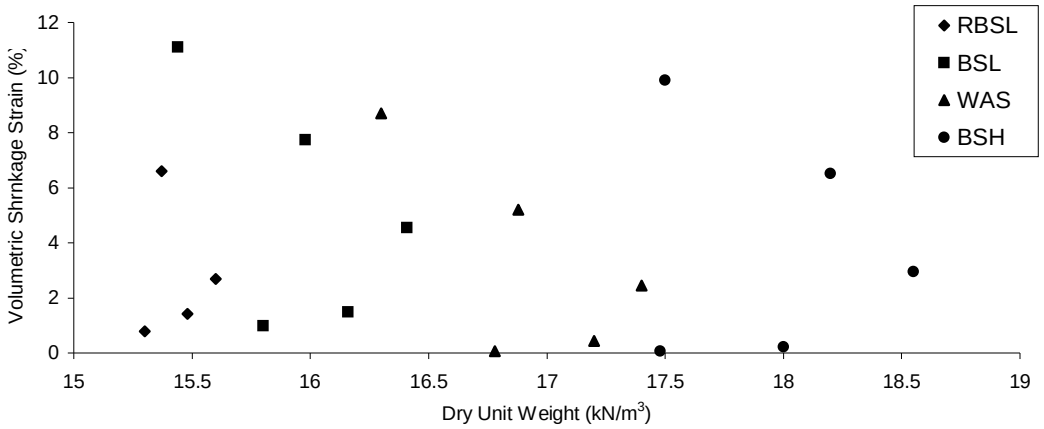


Fig. 5.43 Variation of volumetric shrinkage strain with dry unit weight for 5% bentonite content.

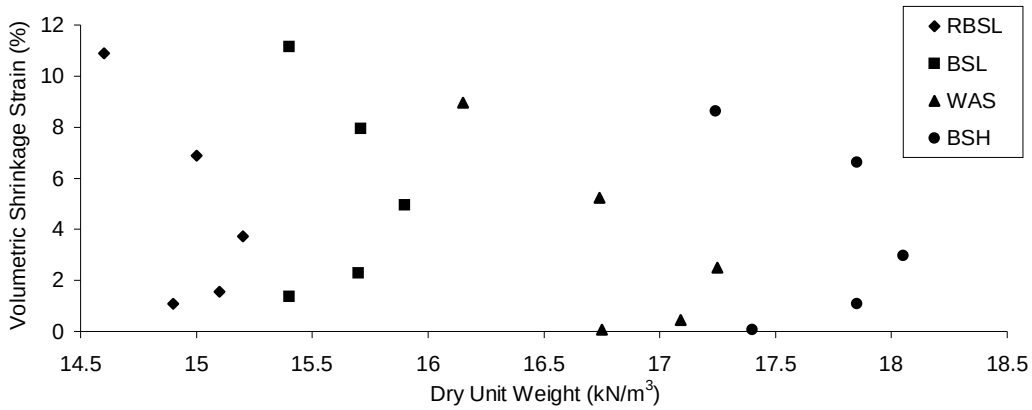


Fig. 5.44 Variation of volumetric shrinkage strain with dry unit weight for 7.5% bentonite content.

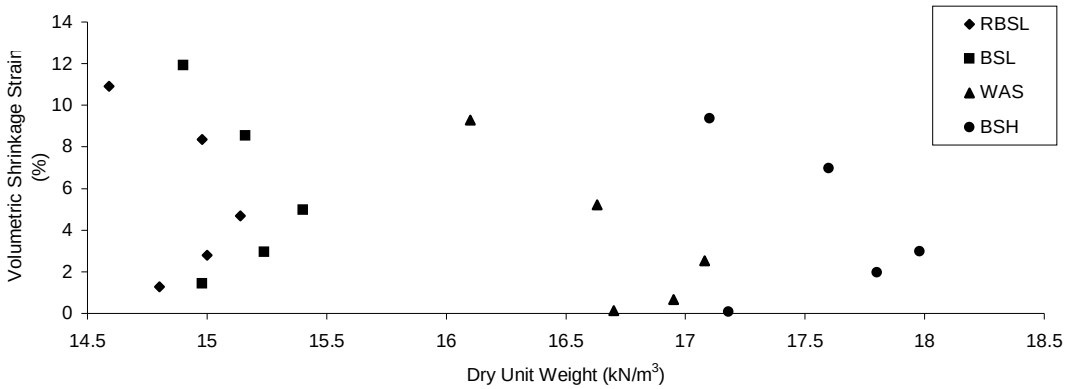


Fig. 5.45 Variation of volumetric shrinkage strain with dry unit weight for 10% bentonite content.

5.4.3 Unconfined compression strength

5.4.3.1 Variation of unconfined compression strength with water content and compactive effort

Graphical representations of the relationship between unconfined compression strength UCS, water content and compactive effort for the various soil bentonite mixtures are shown in Figs. 5.46 – 5.50. The strength of compacted specimens decreased with higher compaction water content. At compaction water content dry of optimum, UCS increased with higher compactive effort. But at water content wet of optimum, no clear trend is observed. Increase in strength resulting from increase in compaction effort is due to changes in pore space and pore size distribution that affect the proximity of the particles (closer packing of soil fabrics). Daniel and Wu (1993), Kabir and Taha (2005) as well as Osinubi and Nwaiwu (2006) reported similar trend.

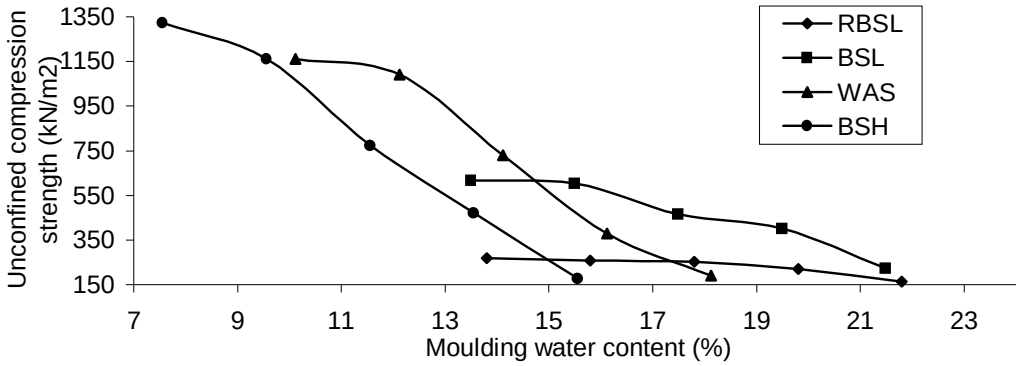


Fig. 5.46 Variation of unconfined compressive strength with moulding water content for soil specimen with 0% bentonite content.

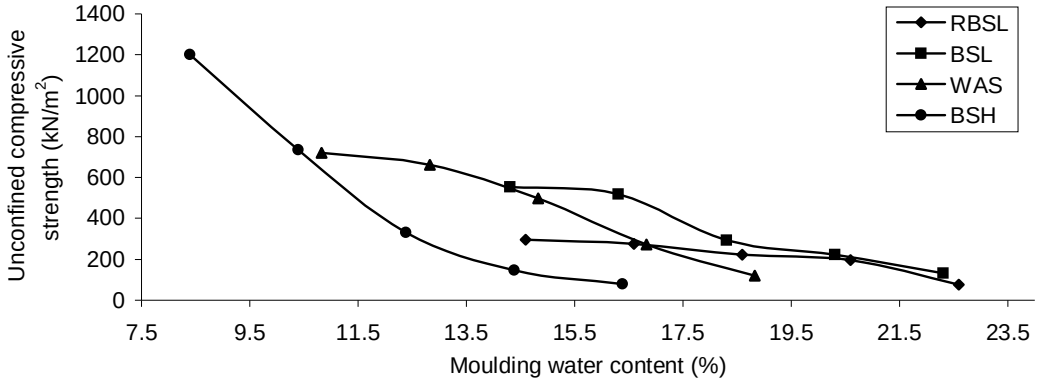


Fig. 5.47 Variation of unconfined compressive strength with moulding water content for soil specimen with 2.5% bentonite content.

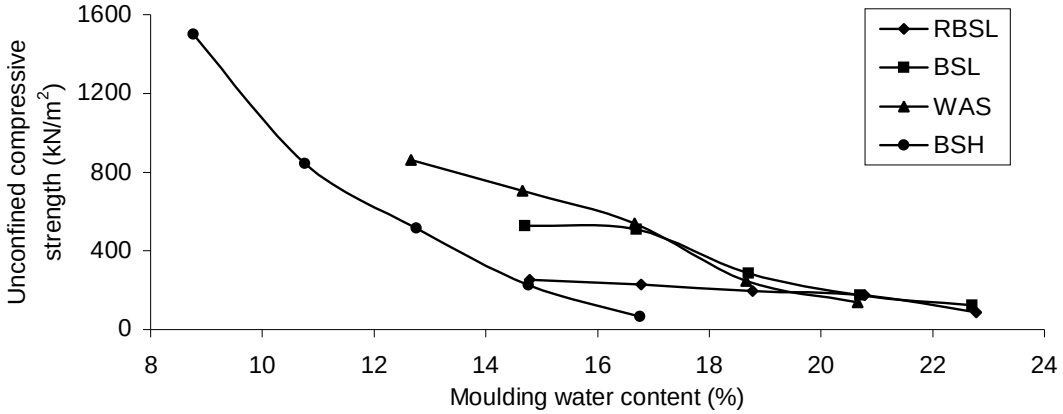


Fig. 5.48 Variation of unconfined compressive strength with moulding water content for soil specimen with 5% bentonite content.

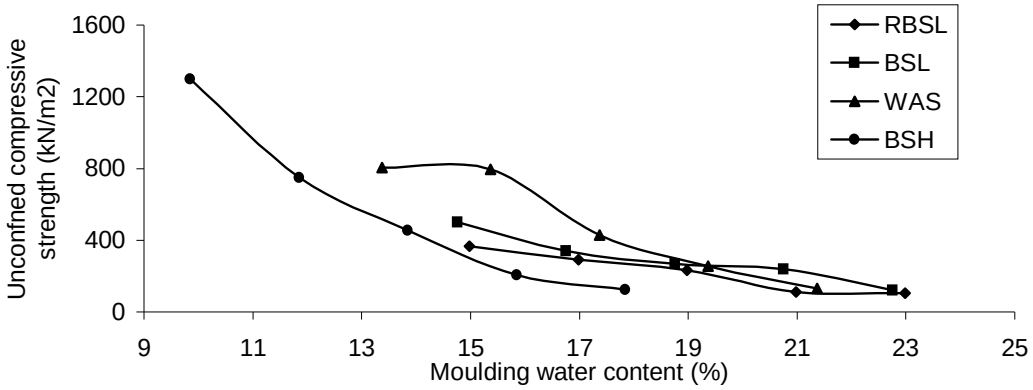


Fig. 5.49 Variation of unconfined compressive strength with moulding water content for soil specimen with 7.5% bentonite content.

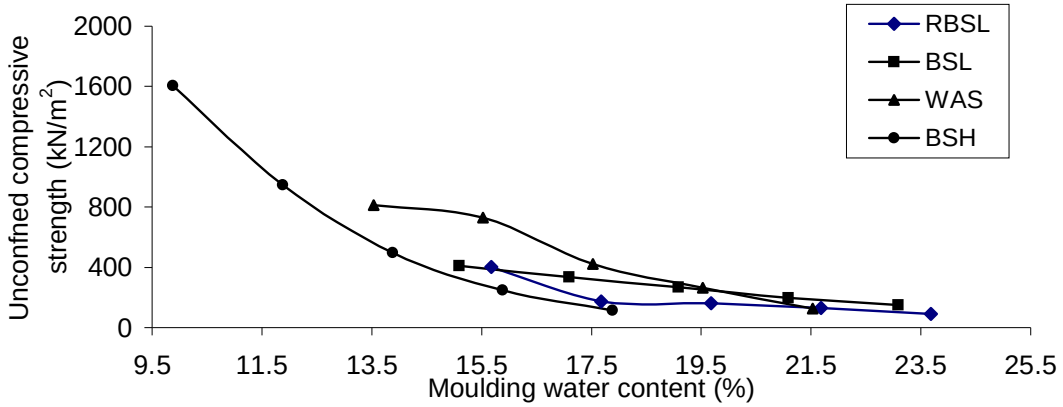


Fig. 5.50 Variation of unconfined compressive strength with moulding water content for soil specimen with 10% bentonite content.

5.4.3.2 Variation of unconfined compression strength with bentonite content

The variation of unconfined compressive strength (UCS) with bentonite content at various compaction water contents is shown in Figs. 5.51 – 5.53. UCS of compacted samples are shown to decreased with higher bentonite content.

From the figures, all samples prepared with WAS and BSH compactive effort at water content wet of optimum met the specified strength of 200kN/m^2. For BSL compaction, only samples containing 2.5% bentonite were able to attain the specified strength while samples compacted with RBSL effort did not meet this requirement except for the natural soil samples (i.e 0% bentonite).

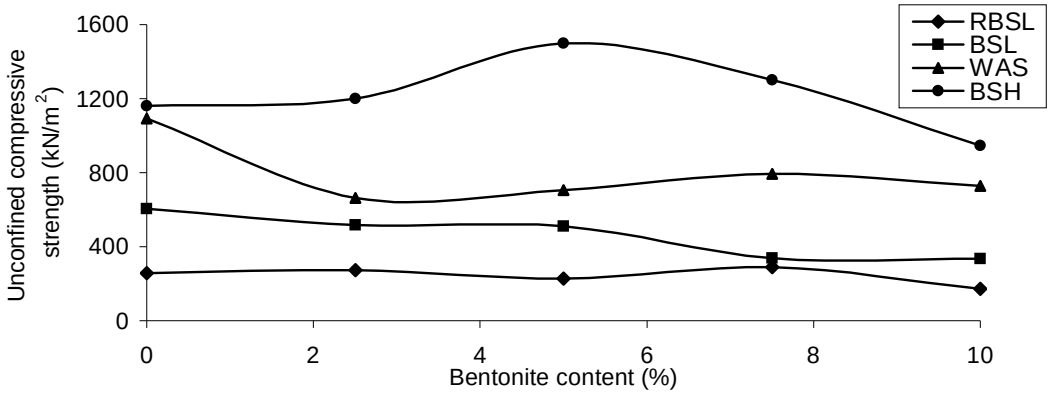


Fig. 5.51 Variation of unconfined compressive strength with bentonite Content at 2.5% dry of optimum water content.

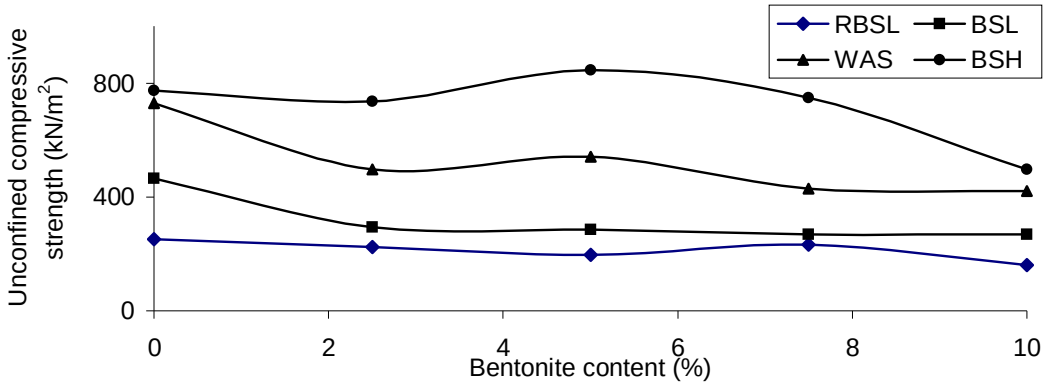


Fig. 5.52 Variation of unconfined compressive strength with bentonite content at optimum water content.

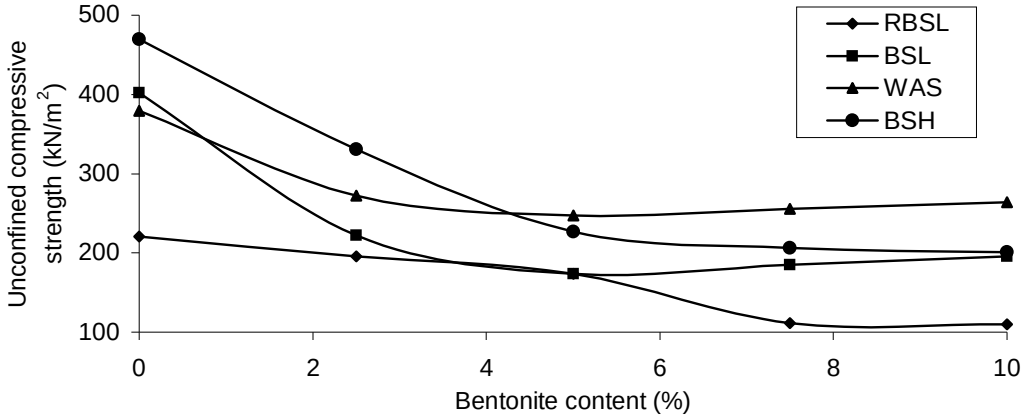


Fig. 5.53 Variation of unconfined compressive strength with bentonite content at 2.5% wet of optimum water content.

The reduction in shear strength is partly attributed to the increase in clay size particles (bentonite) that filled voids between sand particles, which lowered the frictional resistance between the sand particles at their contact points.

5.4.3.3 Variation of unconfined compressive strength with dry unit weight

Plots of UCS as a function of the dry unit weight for the varying bentonite content are shown in Figs. 5.54 – 5.58. At any given bentonite content, an increase in dry unit weight generally results in an increase in unconfined compressive strength. The trend is however relatively weak and exhibits significant scatter. In addition dry unit weight had little or no effect on unconfined compressive strength of specimens until the dry unit weight exceeded a certain limit depending on bentonite content.

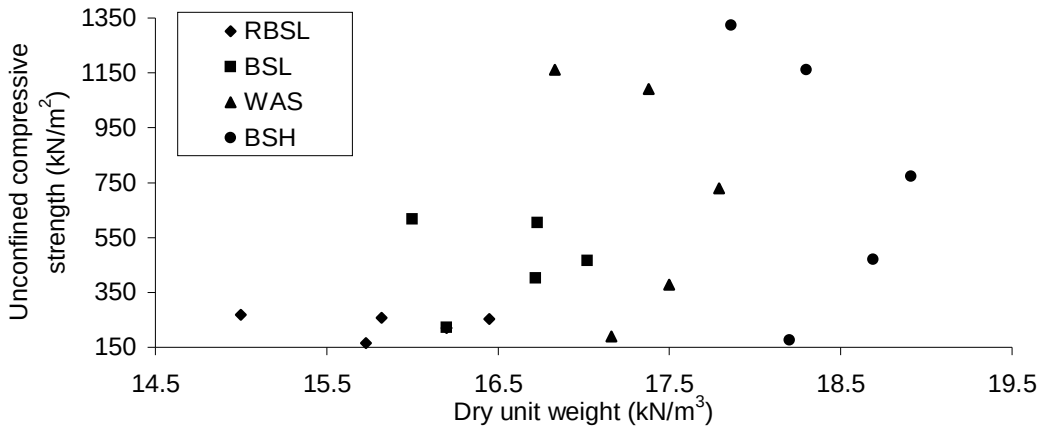


Fig. 5.54 Variation of unconfined compressive strength with dry unit weight for 0% bentonite content

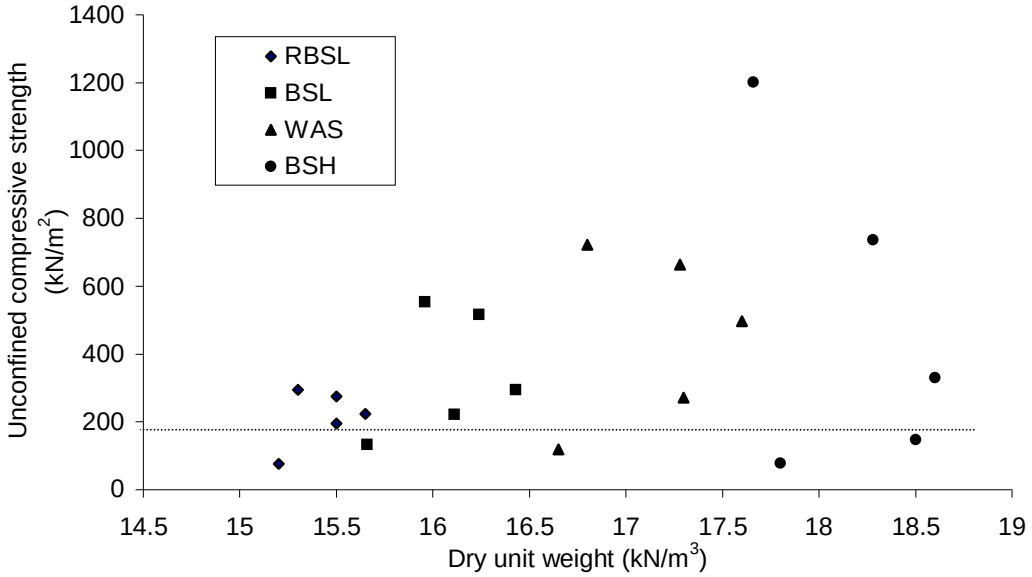


Fig. 5.55 Variation of unconfined compressive strength with dry unit weight for 2.5% bentonite content

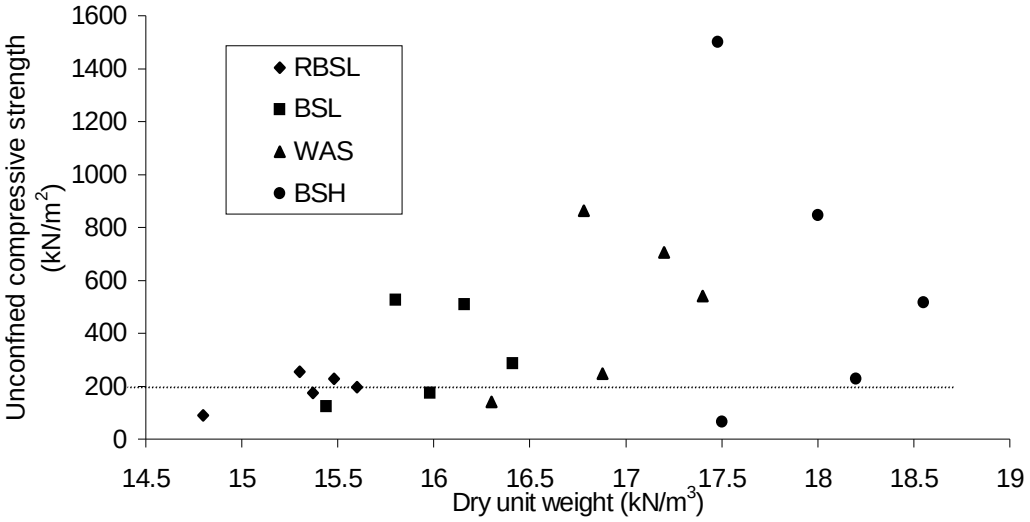


Fig. 5.56 Variation of unconfined compressive strength with dry unit weight for 5% bentonite content

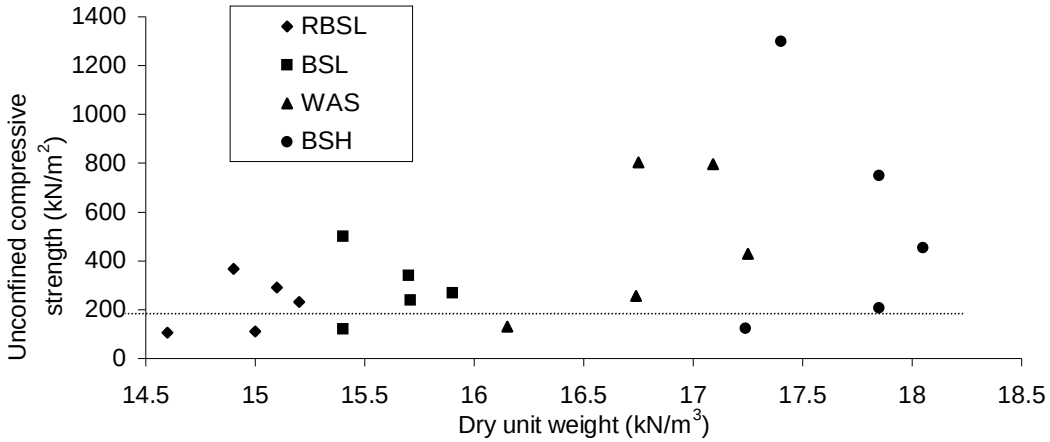


Fig. 5.57 Variation of unconfined compressive strength with dry unit weight for 7.5% bentonite content

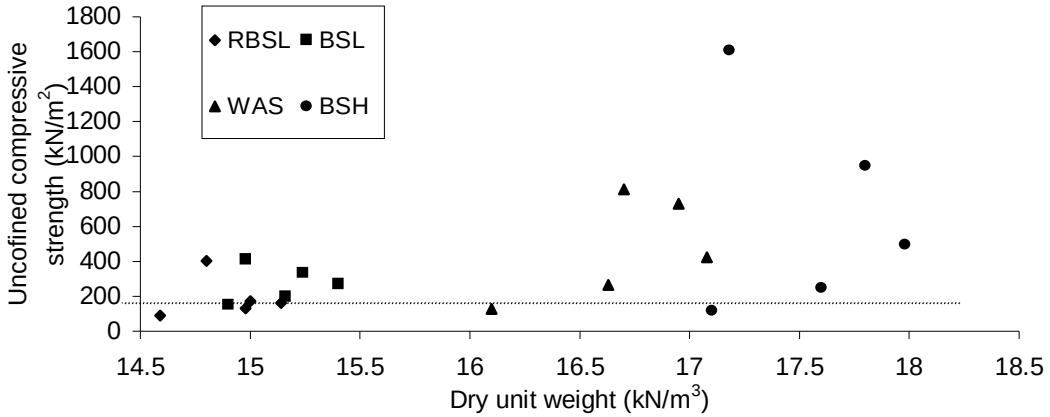


Fig. 5.58 Variation of unconfined compressive strength with dry unit weight for 10% bentonite content.

5.4.4 Delineation of acceptable ranges of water content and dry unit weight

To ensure that compacted clay liners are well constructed and perform as they are designed, it is important to implement effective quality control during design emphasizing on material specifications and construction practices. For quality control during construction, some geotechnical design parameters (such as hydraulic conductivity, strength, and shrinkage potential) greatly influenced by water content were considered. It is therefore important to delineate the range of water content within which the compacted mix will achieve the specifications of hydraulic conductivity, $k \leq 10^{-7}$ cm/s, volumetric shrinkage $\leq 4\%$ and unconfined compressive strength ≥ 200 kN/m^2 normally required by regulatory agencies.

The procedure employed in this study is similar to those adopted by Daniel and Wu, 1993 and follows the procedure described by Daniel and Benson, (1990).

This involves delineation of suitable ranges of compaction water content and corresponding dry unit weights by an envelop drawn to define the zone where moisture – density combinations produce acceptable/established values of hydraulic conductivity, unconfined compression strength and volumetric shrinkage strain on water content – dry unit weight curve. The acceptable zones of hydraulic conductivity, unconfined compression strength and volumetric shrinkage strain for all the soil - bentonite mixtures are shown in Figs. 5.59–5.73. Overall acceptable ranges of compaction water content and dry unit weights were defined by superimposing the various acceptable zones of the respective design parameters considered. The acceptable ranges of water content for the various mixes is shown in Table 5.7, while the overall acceptable zones for the five lateritic soil bentonite mixes are shown in Figs. 5.74 – 5.78. The range of acceptable water content form the left and right boundary of the zone, while the upper bound of the acceptable is the zero – air void line.

Thus, Figs. 5.74 – 5.78 show that for the various compacted laterite soil bentonite mixtures, a range of water content and dry unit weight exist that satisfy all the three established criteria namely:- hydraulic conductivity, strength, and volumetric shrinkage. Generally, it can be observed from Table 5.7 that the acceptable range of water content was broadened by the introduction of bentonite. The implication of this is that construction operations at project sites will significantly be accelerated, as compaction will be carried out within a wide range of water content to achieve the established standards for the design parameters (Daniel and Benson, 1990; Osinubi *et al.*, 2006b).

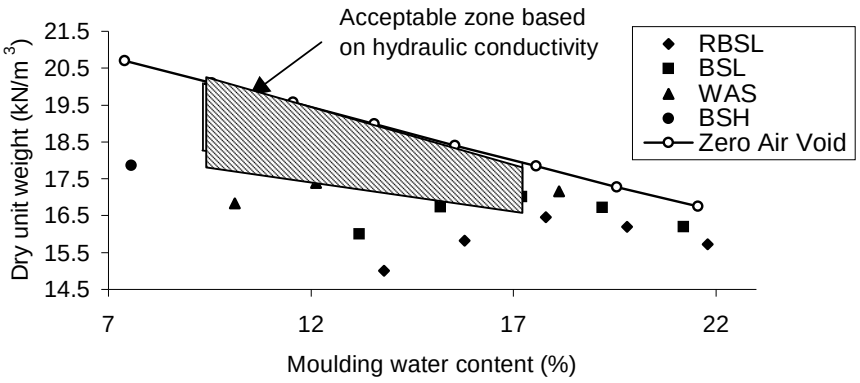


Fig. 5.59 Acceptable zone based on hydraulic conductivity for 0% bentonite content.

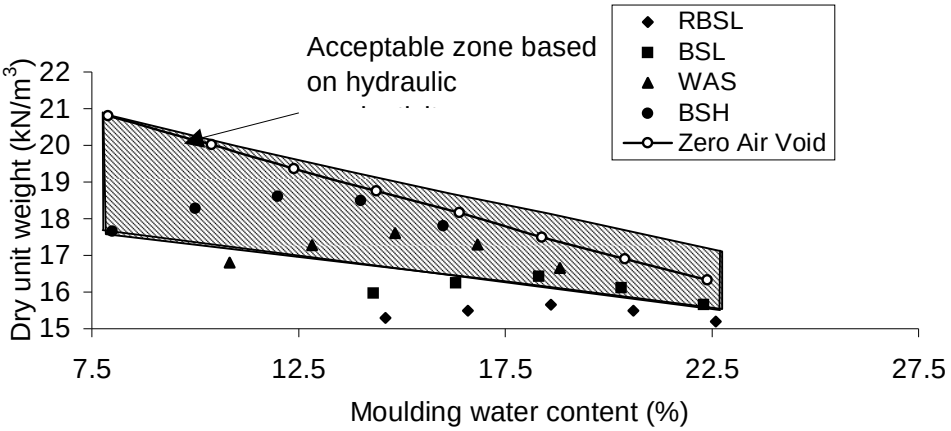


Fig. 5.60 Acceptable zone based on hydraulic conductivity for 2.5% bentonite content.

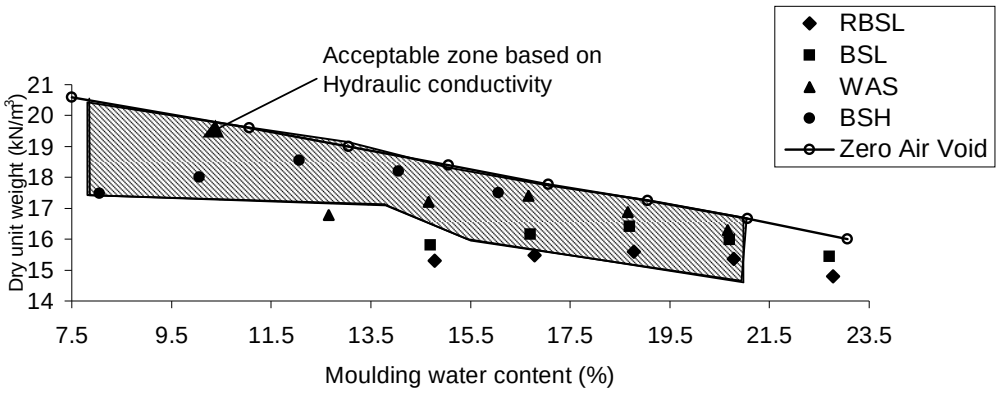


Fig. 5.61 Acceptable zone based on hydraulic conductivity for 5% bentonite content.

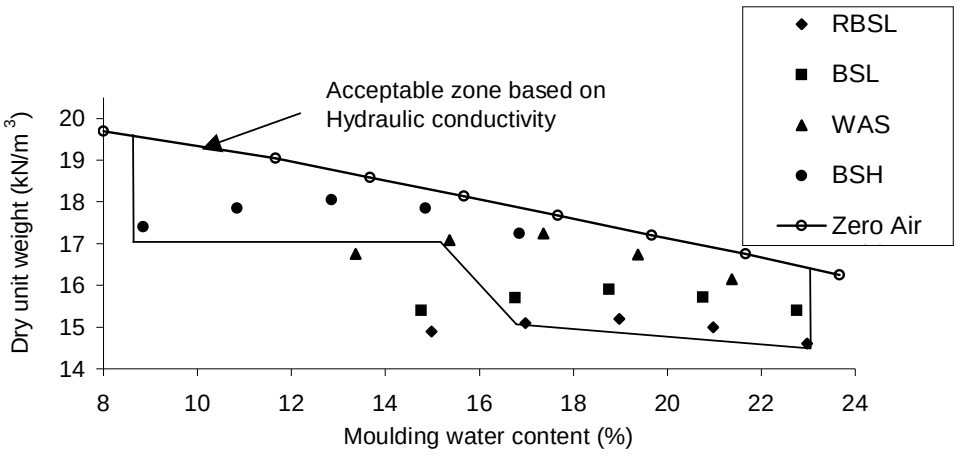


Fig. 5.62 Acceptable zone based on hydraulic conductivity for 7.5% bentonite content.

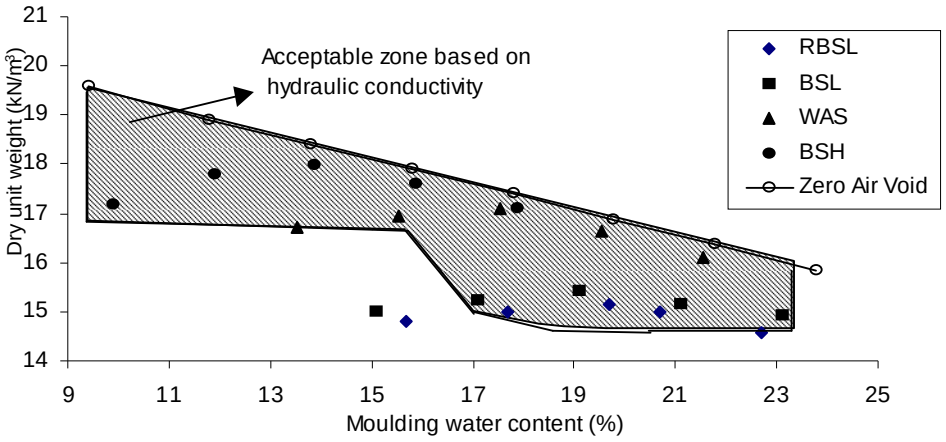


Fig. 5.63 Acceptable zone based on hydraulic conductivity for 10% bentonite content.

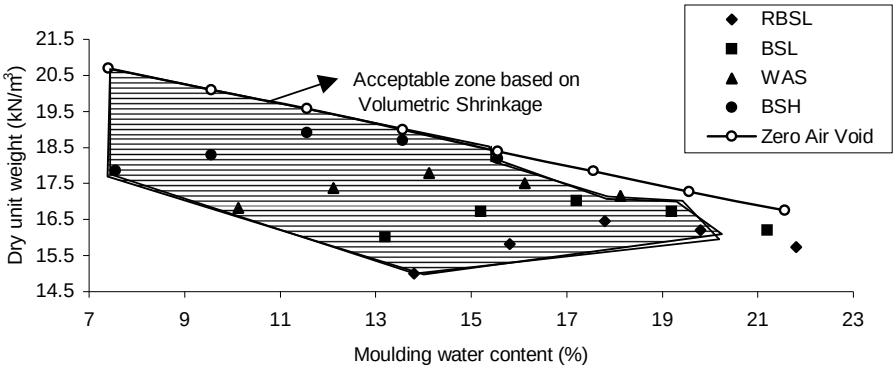


Fig. 5.64 Acceptable zone based on volumetric shrinkage strain for 0% bentonite content.

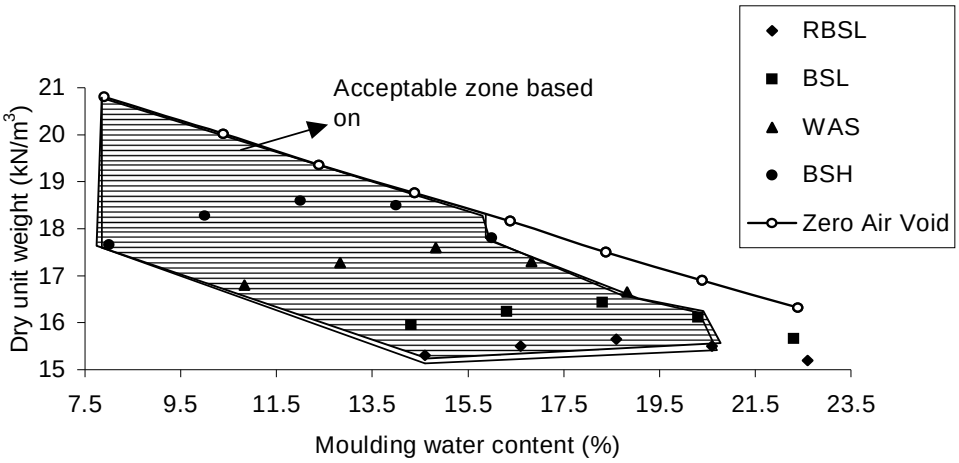


Fig. 5.65 Acceptable zone based on volumetric shrinkage strain for 2.5% bentonite content.

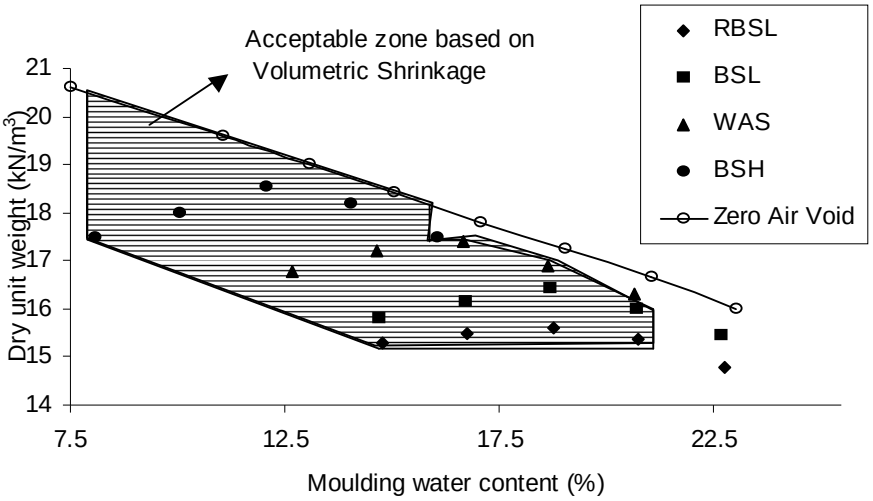


Fig. 5.66 Acceptable zone based on volumetric shrinkage strain for 5% bentonite content.

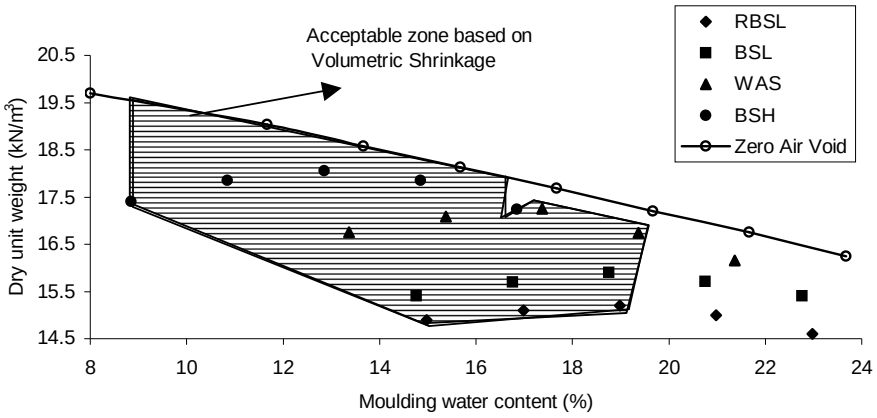


Fig. 5.67 Acceptable zone based on volumetric shrinkage strain for 7.5% bentonite content.

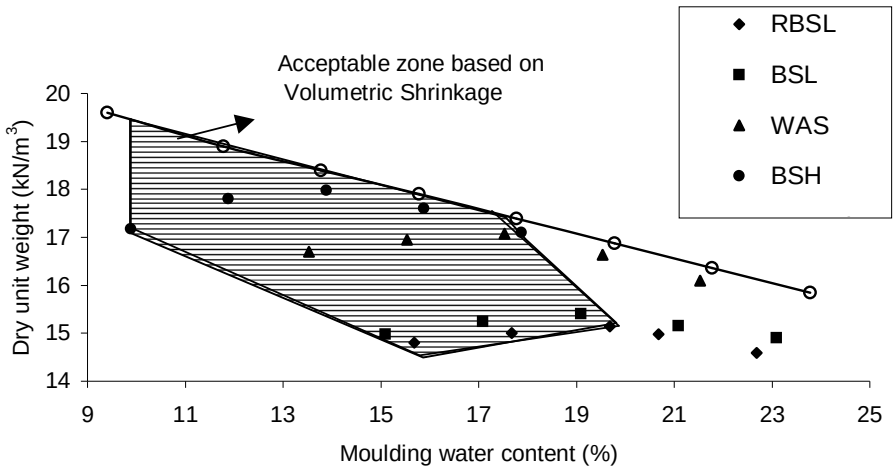


Fig. 5.68 Acceptable zone based on volumetric shrinkage strain for 10% bentonite content.

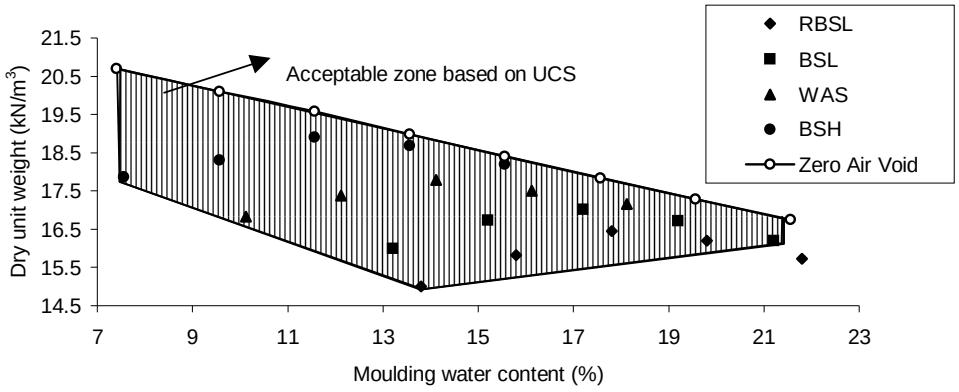


Fig. 5.69 Acceptable zone based on unconfined compressive strength for 0% bentonite content.

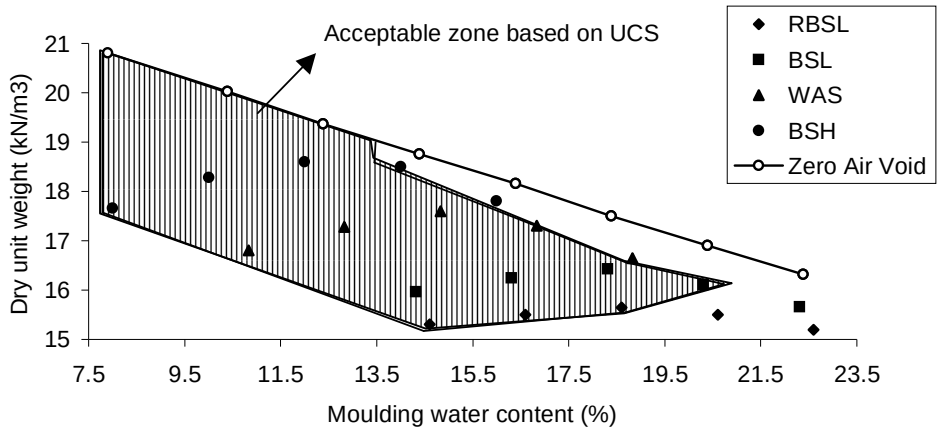


Fig. 5.70 Acceptable zone based on unconfined compressive strength for 2.5% bentonite content.

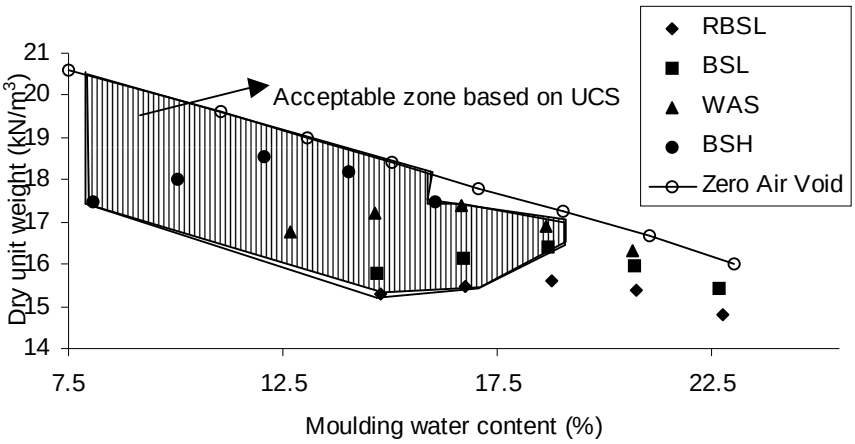


Fig. 5.71 Acceptable zone based on unconfined compressive strength for 5% bentonite content.

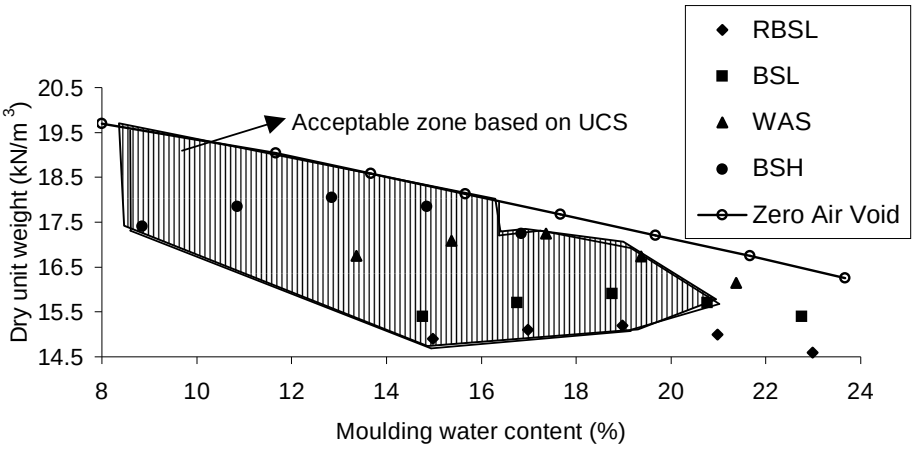


Fig. 5.72 Acceptable zone based on unconfined compressive strength for 7.5% bentonite content.

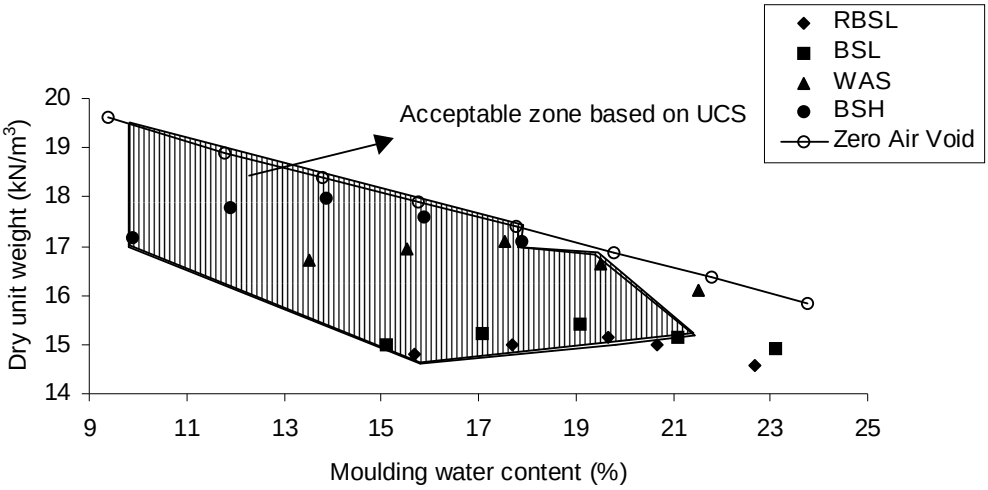


Fig. 5.73 Acceptable zone based on unconfined compressive strength for 10% bentonite content.

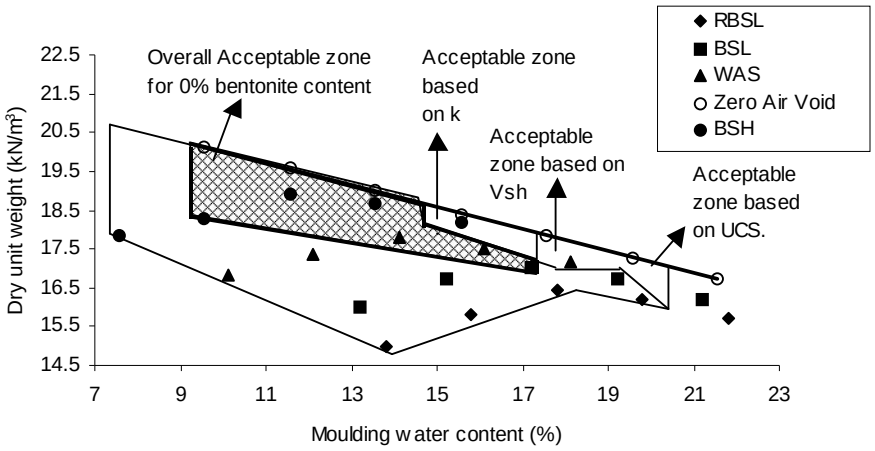


Fig. 5.74 Overall acceptable zone for 0% bentonite content.

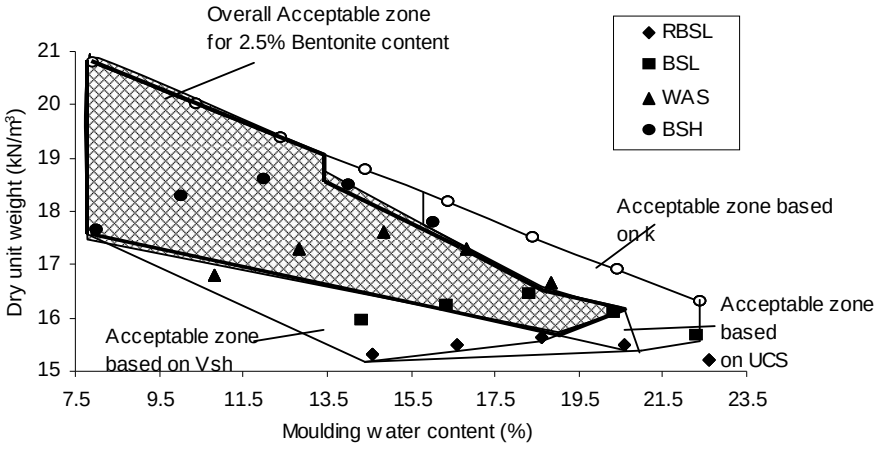


Fig. 5.75 Overall acceptable zone for 2.5% bentonite content.

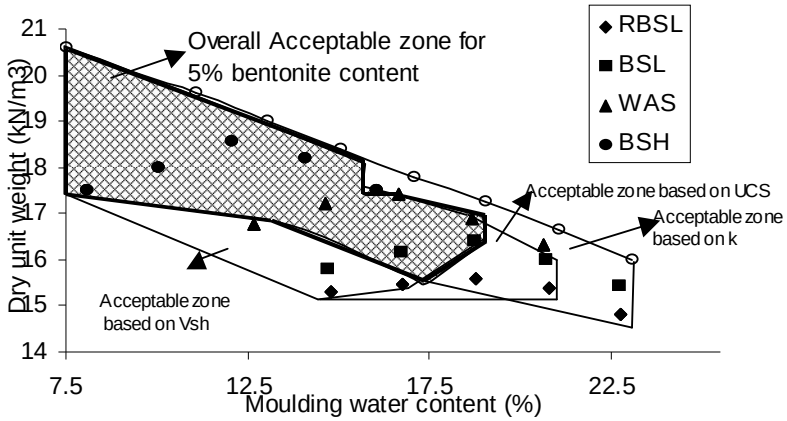


Fig. 5.76 Overall acceptable zone for 5% bentonite content

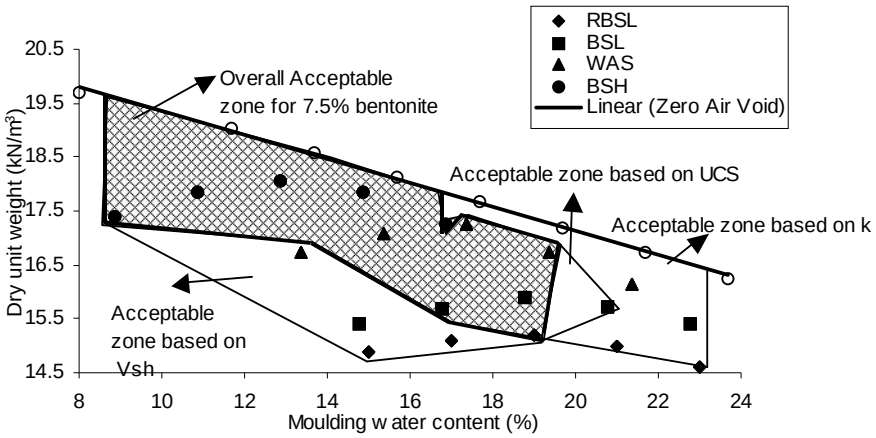


Fig. 5.77 Overall acceptable zone for 7.5% bentonite content

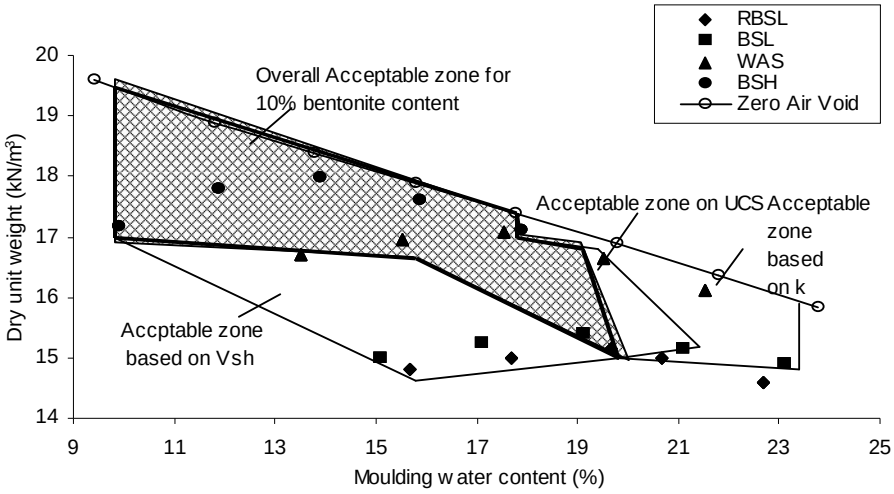


Fig. 5.78 Overall acceptable zone for 10% bentonite content

Table 5.7 Acceptable range of water content for the various mixes

Parameter	Bentonite content (%)				
	0	2.5	5.0	7.5	10
	Acceptable range of moisture content (%)				
k	9.56 to 17.2	8.0 to 22.31	8.06 to 22.78	8.85 to 22.98	9.88 to 23.09
UCS	7.56 to 21.2	8.17 to 20.31	8.06 to 18.7	8.85 to 20.76	9.88 to 21.09
V_{sh}	7.56 to 19.8	8.0 to 20.6	8.06 to 20.78	8.85 to 19.37	9.88 to 19.68
Overall	9.56 to 17.20	8.0 to 20.31	8.06 to 18.70	8.85 to 19.37	9.88 to 19.09

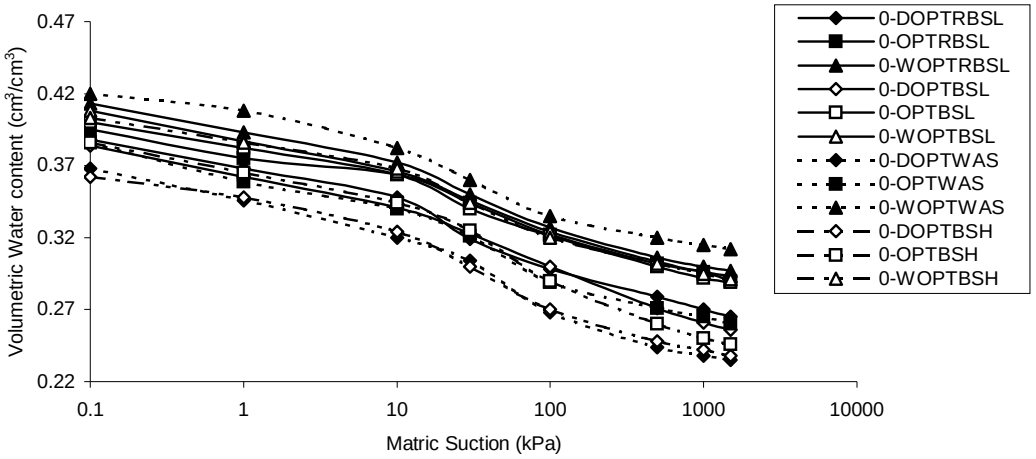
5.5 Unsaturated Hydraulic Properties

5.5.1 Soil water characteristic curves (SWCC)

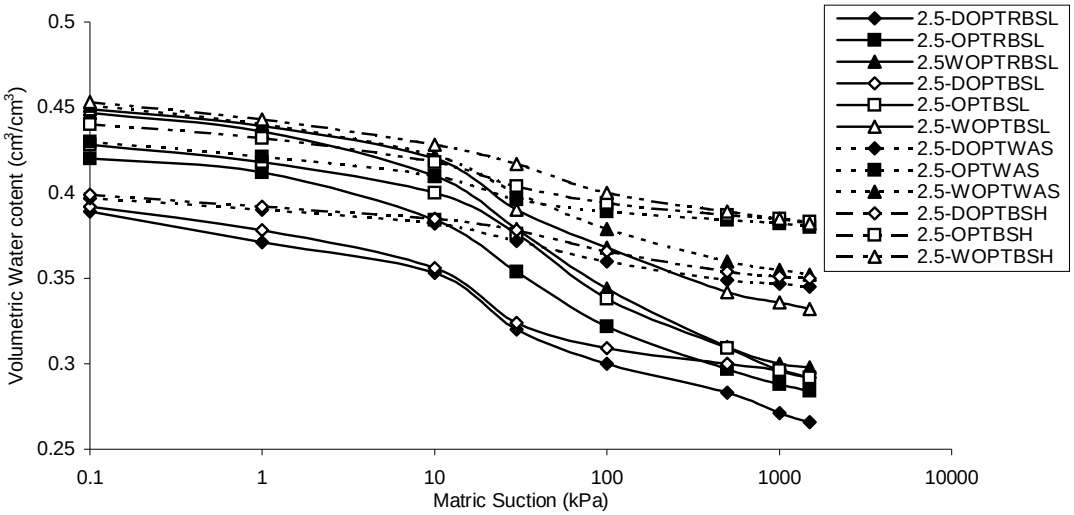
The relationship between the matric suction versus volumetric water content was developed for individually compacted specimens using data from pressure plate tests. Graphical procedures suggested by van Genuchten (1980) were employed to obtain the fitting parameters namely; van Genuchten (α and n) and Brooks-Corey (ψ and λ), while van Genuchten and Brooks – Corey equations (see eqns. 3.2 and 3.3) were used to determine the best fit for the laboratory data of Soil Water Characteristic Curve.

The SWCCs (i.e, the relationship between pore water suction and volumetric water content) for the various levels of bentonite treatment are shown in Fig. 5.79. Typical of clayey soils, linear and less steeply sloping SWCCs were obtained due to nonuniform grain size and pore size distributions (Marinho, 2005). Between suctions of 100 and 1500 kPa, the SWCCs for all the mixes were approximately straight lines, which is characteristic of soils with liquid limit of more than 25% (Marinho and Chandler, 1993).

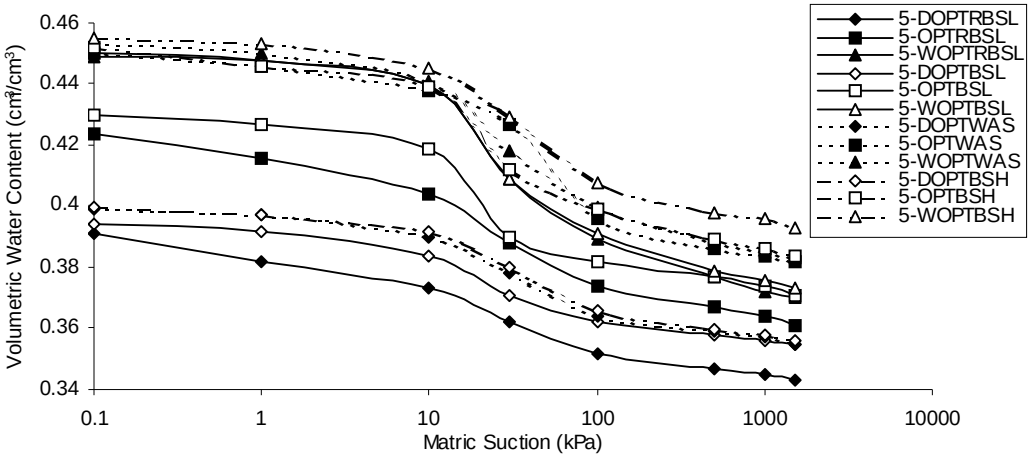
Although each equation yielded different sets of empirical parameters, the resulting curves did not show marked deviations from the curves of the measured data. Figs. 5.80 – 5.84 show predicted curves for the various soil – bentonite mixes compacted with reduced BSL, BSL, WAS and BSH effort at optimum and wet of optimum water content. Generally, van Genuchten equation tends to underestimate the volumetric water content θ, while Brooks – Corey's equation oversetimate θ values for corresponding matric suction levels. Tinjum et al., (1997); Chiu and Shackelford (1998) as well as Miller *et al*. (2002) reported similar observations.



A

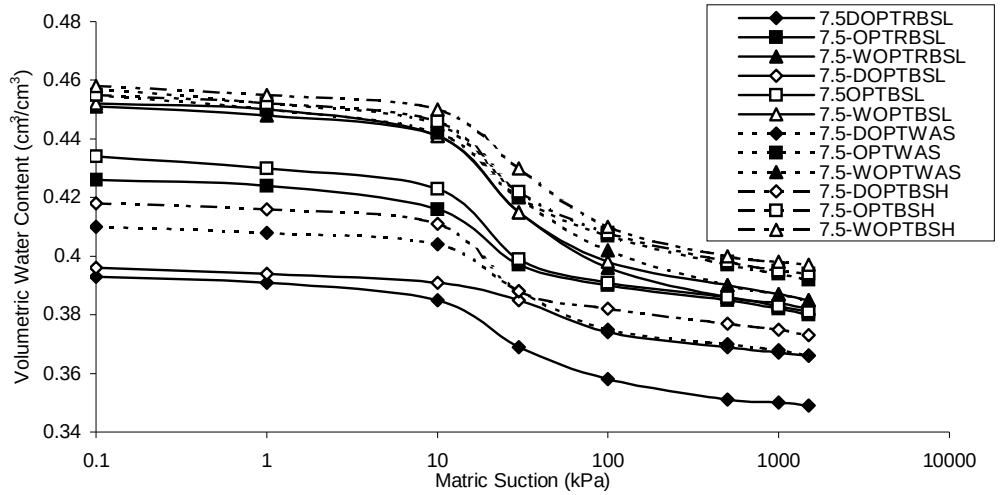


B

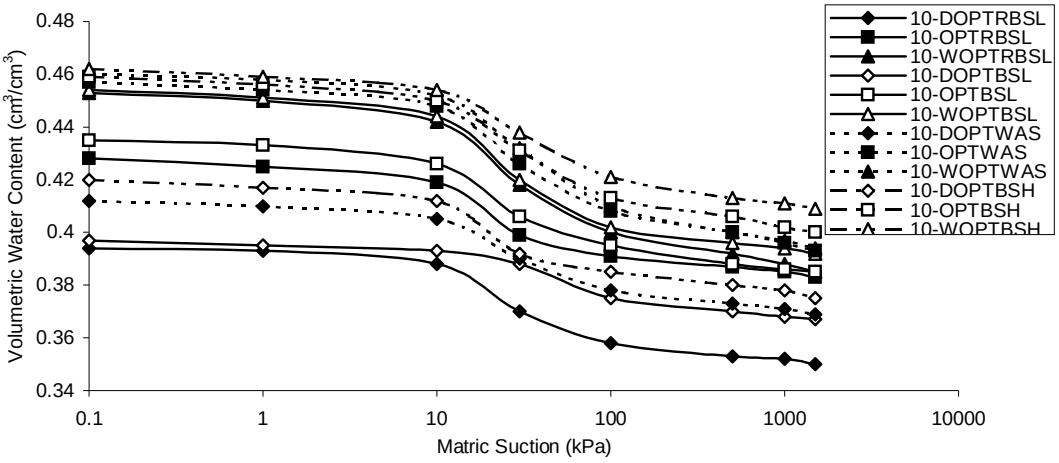


C

Fig. 5.79A,B and C. Soil Water characteristic curves for (A) 0, (B) 2.5 and (C) 5% bentonite content.

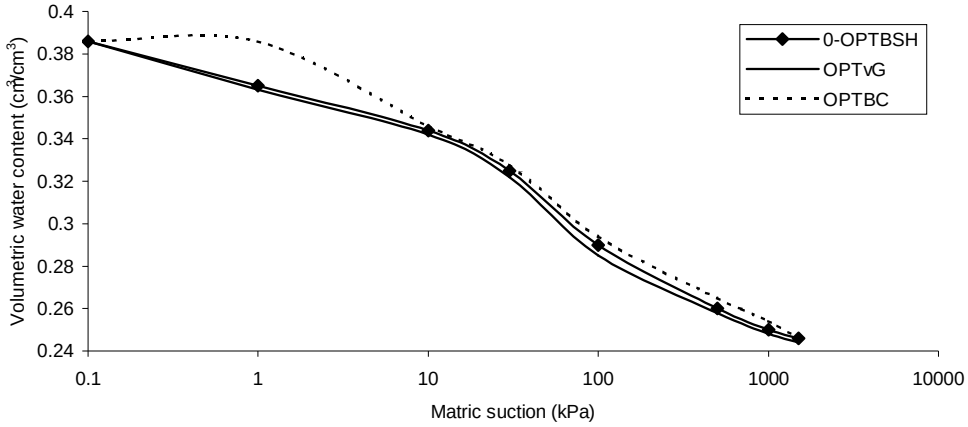


D

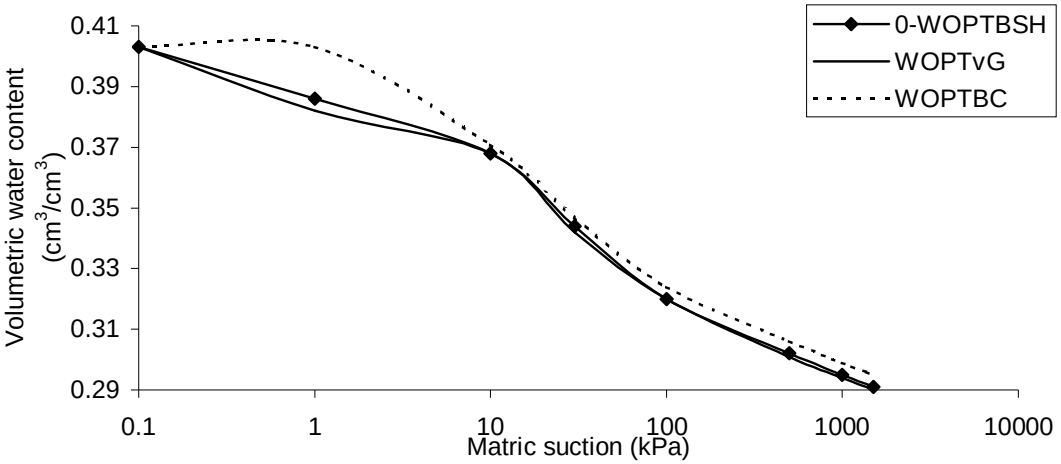


E

Fig. 5.79D,E. Soil water characteristic curves for (D) 7.5 and (E) 10% bentonite content.

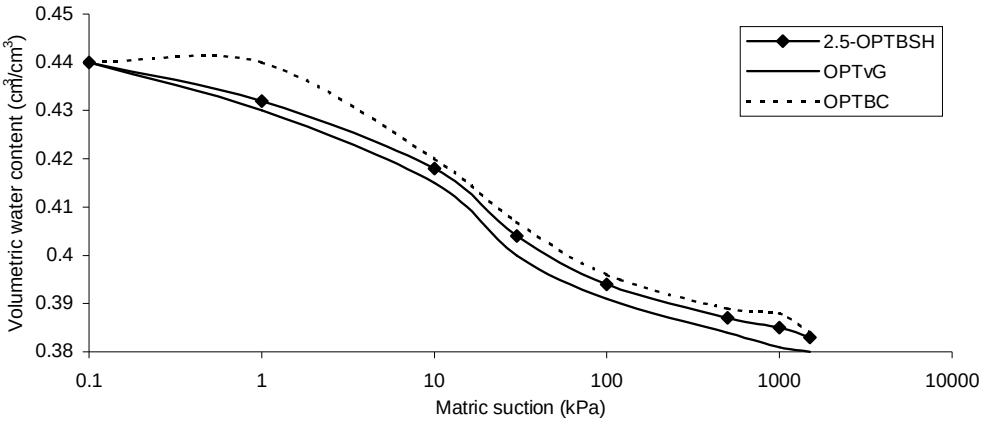


A

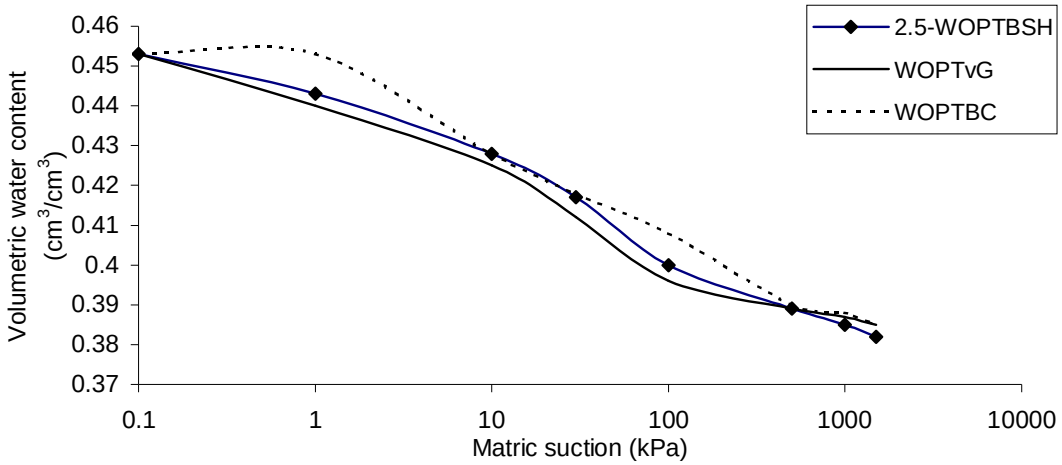


B

Fig. 5.80 Measured and predicted SWCCs based on Brooks-Corey and van Genuchten functions for 0% content at (A) optimum and (B) wet of optimum with BSH compactive effort.

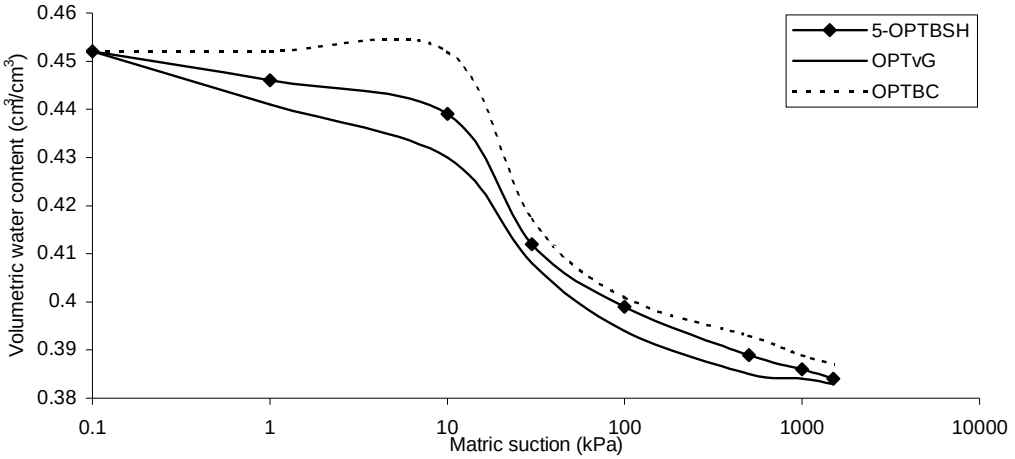


A

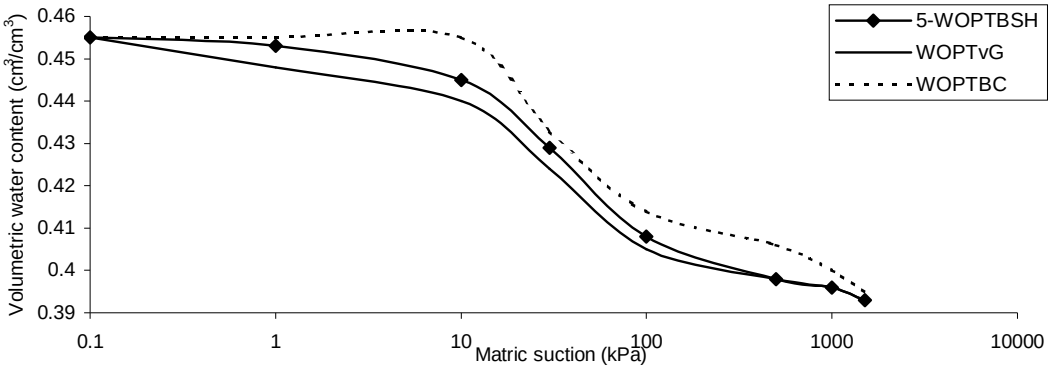


B

Fig. 5.81 Measured and predicted SWCCs based on Brooks-Corey and van Genuchten functions for 2.5% content at (A) optimum and (B) wet of optimum with BSH compactive effort.



A



B

Fig. 5.82**.** Measured and predicted SWCCs based on Brooks-Corey and van Genuchten functions for 5% content at (A) optimum and (B) wet of optimum with BSH compactive effort.

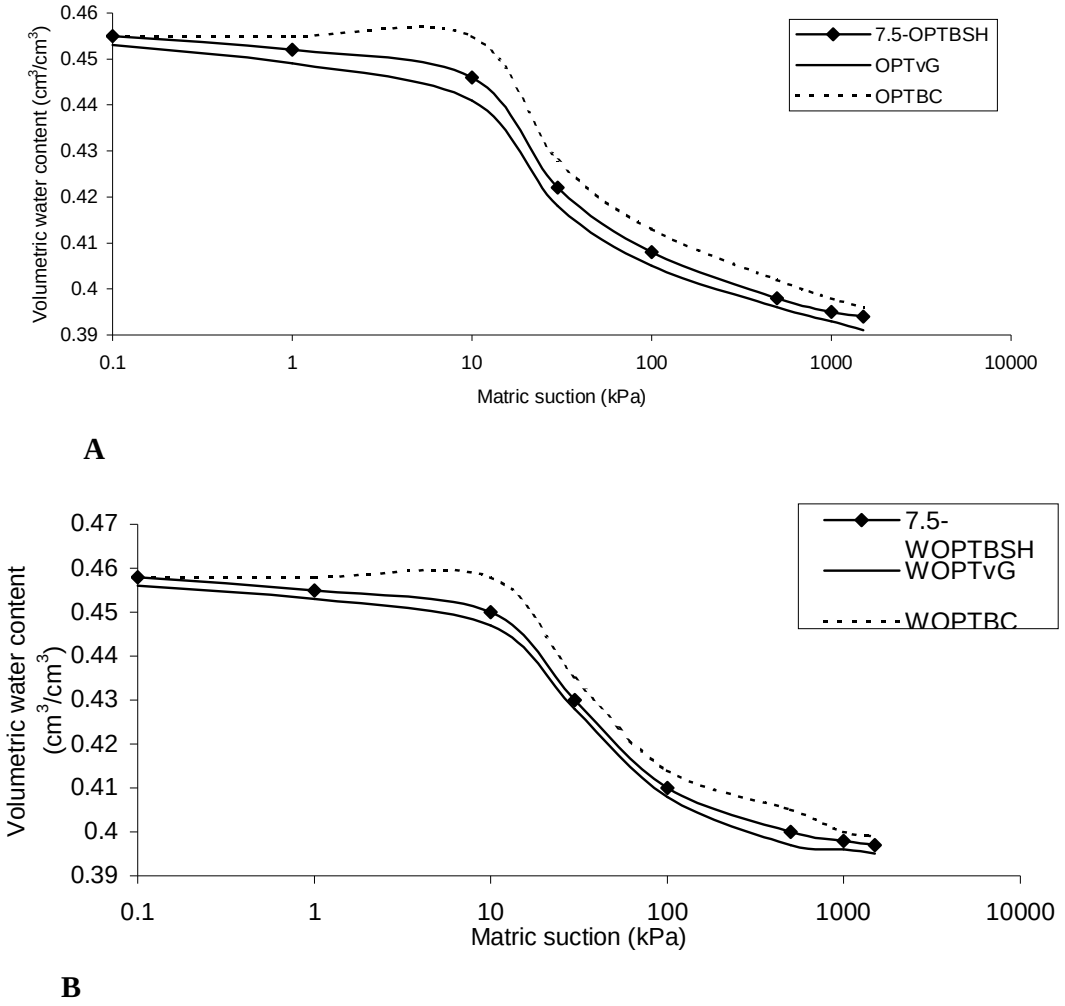
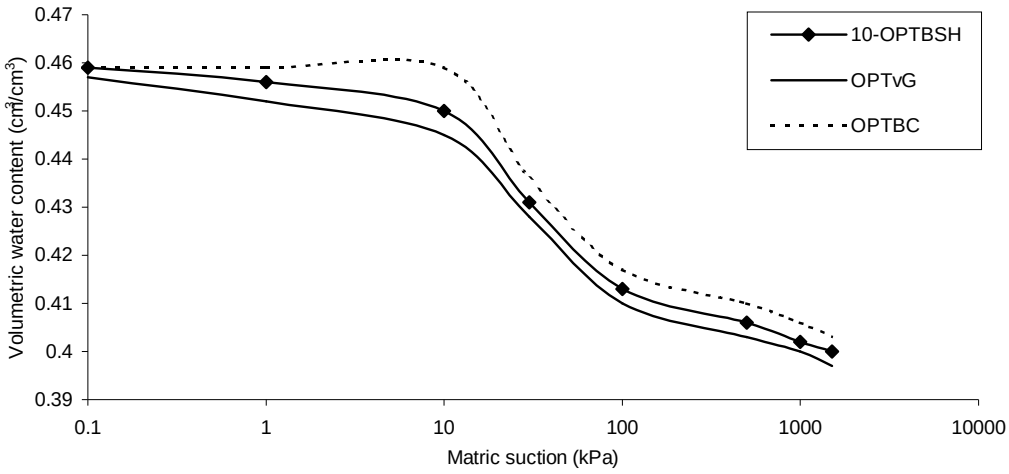
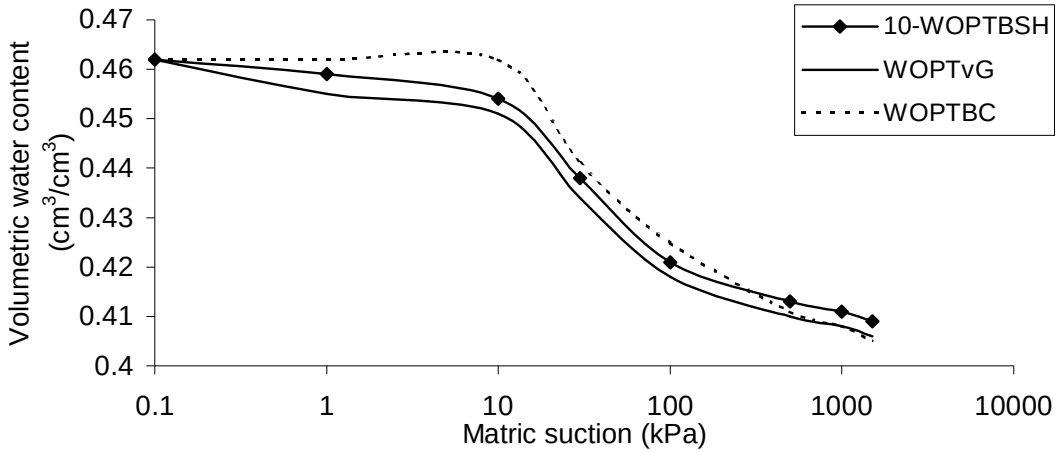


Fig. 5.83. Measured and predicted SWCCs based on Brooks-Corey and van Genuchten functions for 7. 5% content at (A) optimum and (B) wet of optimum with BSH compactive effort.



A



B

Fig. 5.84. Measured and predicted SWCCs based on Brooks-Corey and van Genuchten functions for 10% content at (A) optimum and (B) wet of optimum with BSH compactive effort.

5.5.1.1 Effect of bentonite content on the soil water characteristic curves

The plasticity of the mixtures increased with higher bentonite content and therefore, volumetric water content, θ increased as more water was available for removal. The variation of van Genuchten (α and n) and Brooks-Corey (ψ and λ) fitting parameters with bentonite contents at dry, wet and optimum moisture contents are shown in Figs. 5.85 - 5.96.

A pronounced effect on the magnitude of α resulting from an increase in bentonite content was observed. The values of α decreased with higher bentonite content as shown in Fig. 5.85-5.87 which corresponds to higher air entry suctions.

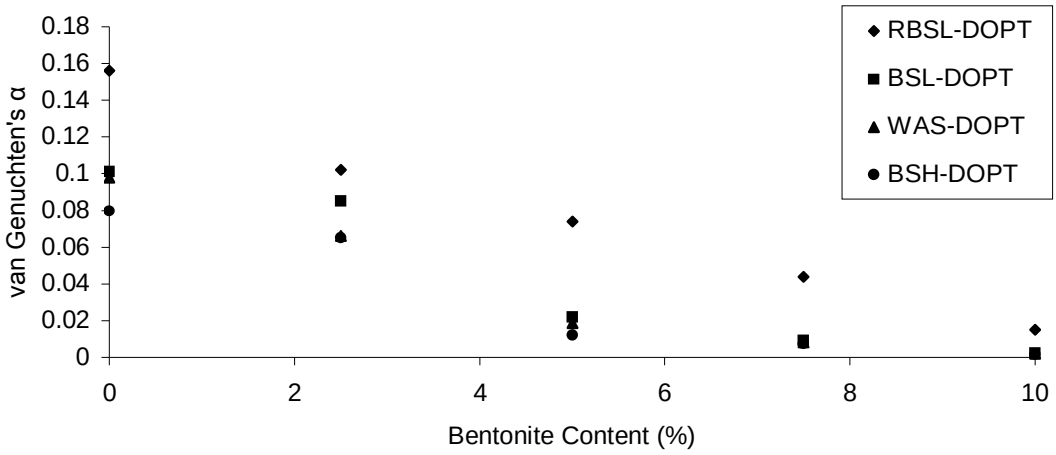


Fig. 5.85 van Genuchten's α versus bentonite content at dry of optimum water content.

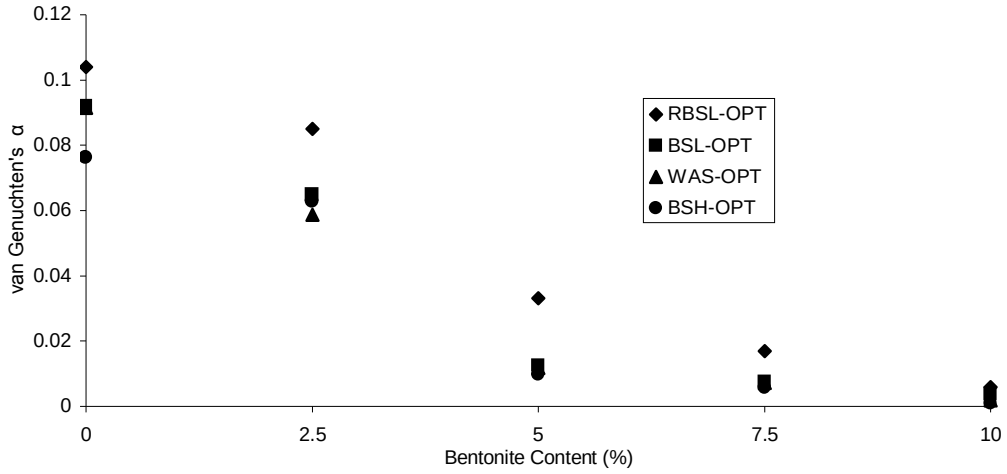


Fig. 5.86 van Genuchten's α versus Bentonite content at optimum water content.

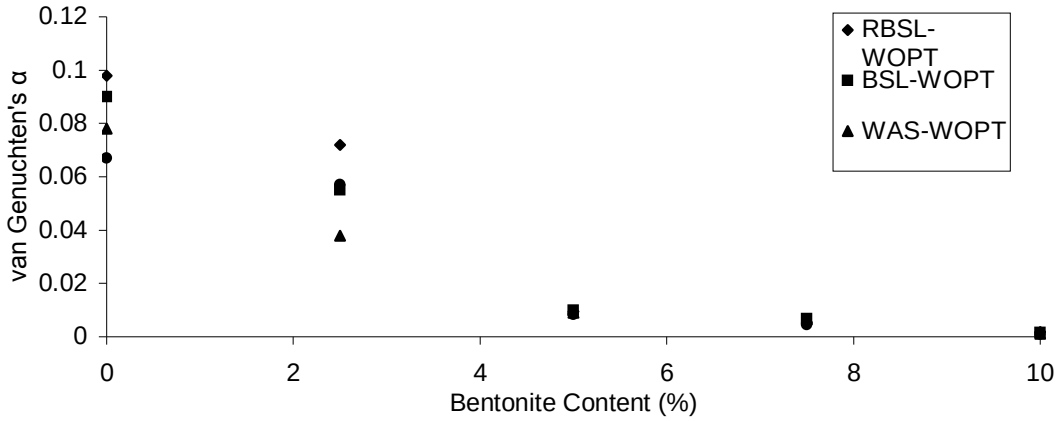


Fig. 5.87 van Genuchten's α versus bentonite content at wet of optimum moisture content.

As bentonite that is known to have thick double layers increased in content, the effective pore size decreased (Mitchell, 1993) which resulted in greater air-entry suction. Therefore, greater matric suctions will be required to remove water from the compacted mixtures with higher bentonite content (Tinjum, *et al*., 1997; Simms and Yanful, 2000). On the other hand, n increased with increase in bentonite content varying from 1.0 to 1.78 (see Fig. 5.88– 5.90). The larger the values of n, the more uniform the pore sizes in the soil. This indicates that the treated specimens exhibit a more uniform pore size distribution than the control specimen. All these are consistent with the expected trend for these fitting parameters.

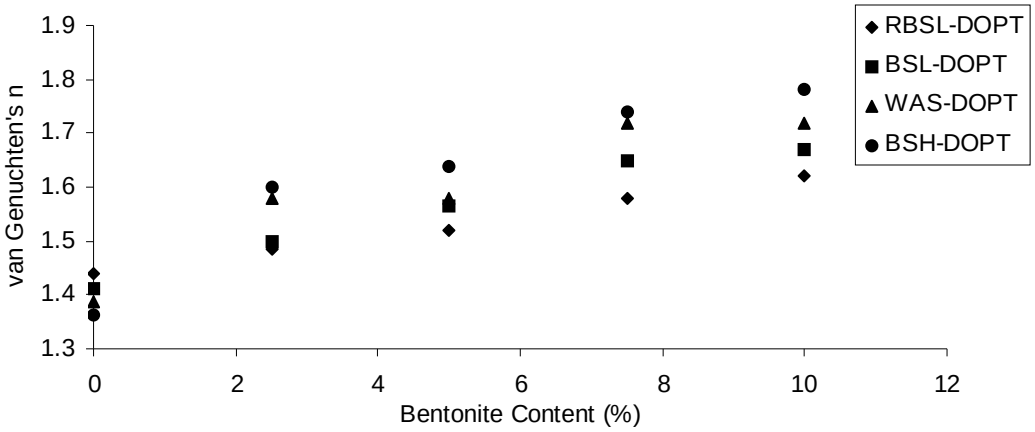


Fig. 5.88 van Genuchten parameter n versus bentonite content at dry of optimum water content

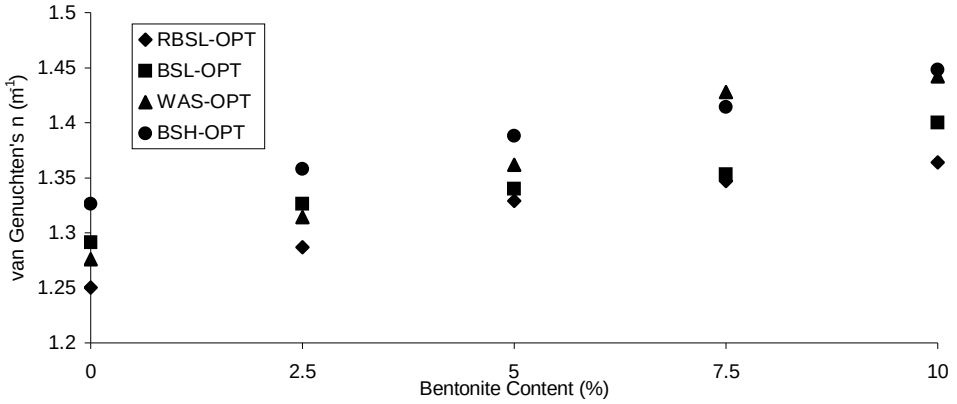


Fig. 5.89 van Genuchten parameter n versus bentonite content at optimum water content

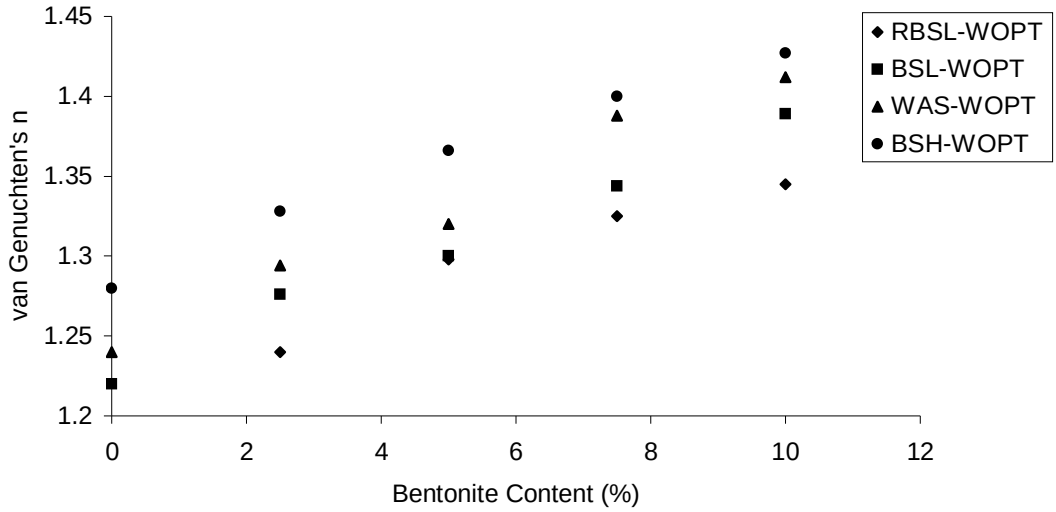


Fig. 5.90 van Genuchten parameter n versus bentonite content at wet of optimum water content

In the case of the BC fitting parameters, ψ_a and λ, the air entry value, ψ_a is related to the particle size distribution of specimens (Brooks and Corey, 1966). When there is large proportion of fine particles as in bentonite treated specimens, the intra-particle pore spaces between soil particles become smaller and as a result, the air entry value, ψ_a is high.

The λ is expected to decrease as the soil becomes more well graded and the amount of finer particles in the soil increased. The increase in ψ_a with higher bentonite content is presented in Figs. 5.91-5.93 while λ decreased with higher bentonite content as shown in Figs. 5.94-5.96. Thus BC model provided expected predictions for ψ_a and λ (Chiu and Shackelford, 1998).

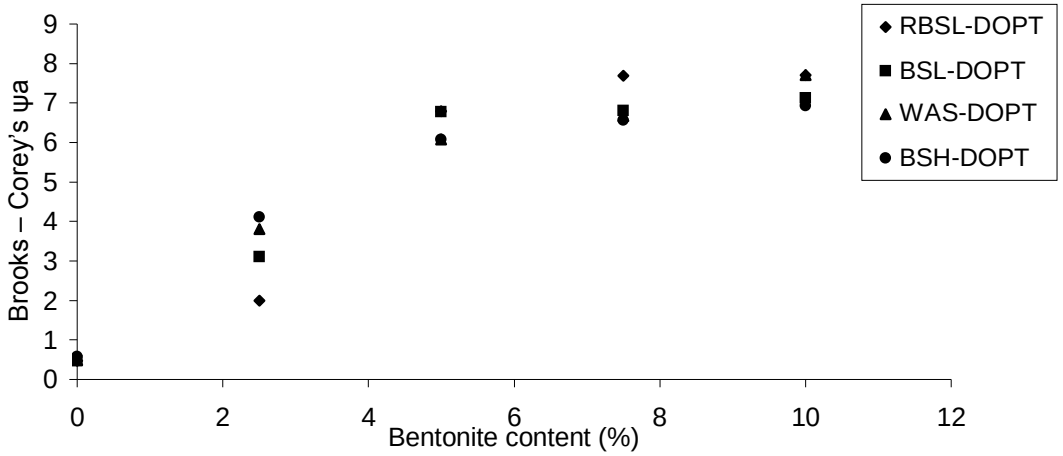


Fig. 5.91 Brooks – Corey's ψ_a versus Bentonite content at dry of optimum water content.

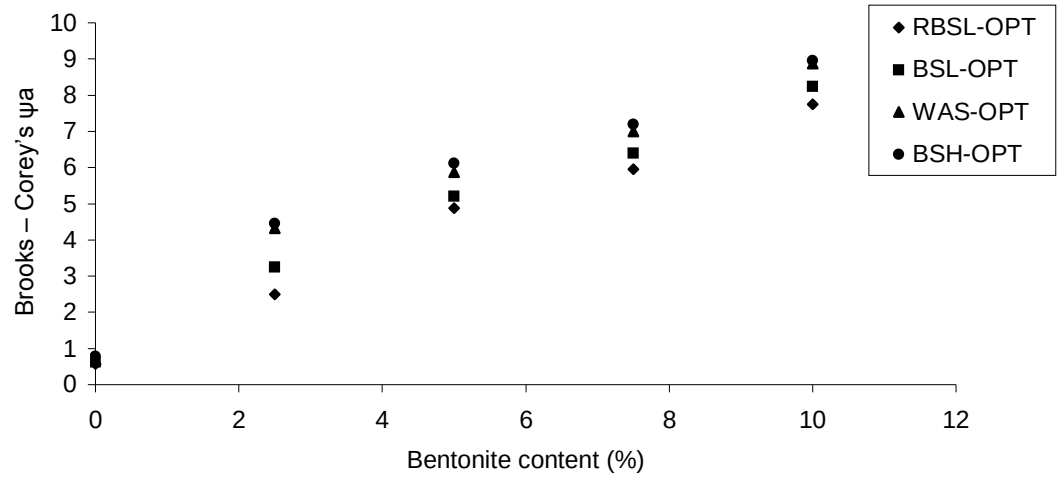


Fig. 5.92 Brooks – Corey's ψ_a versus Bentonite content at optimum water content.

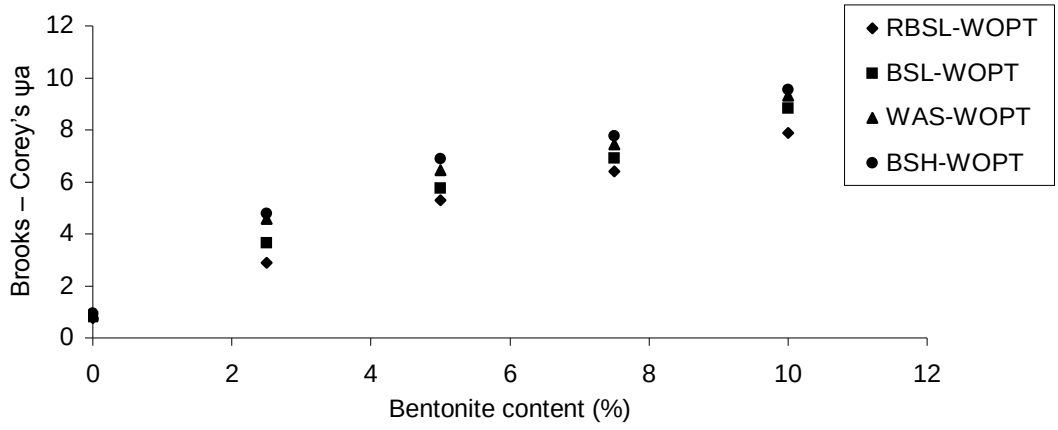


Fig. 5.93 Brooks – Corey's ψ_a versus Bentonite content wet of optimum water content.

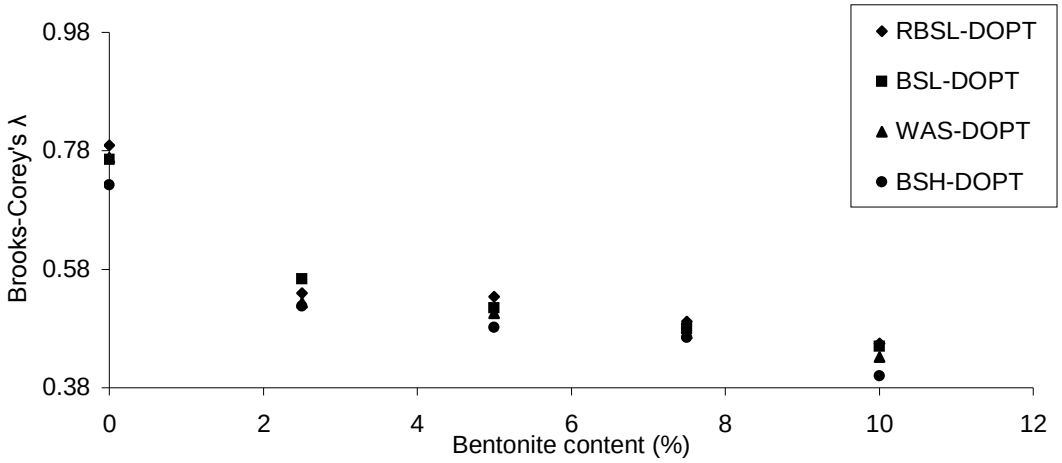


Fig. 5.94 Brooks – Corey's λ versus Bentonite content dry of optimum water content.

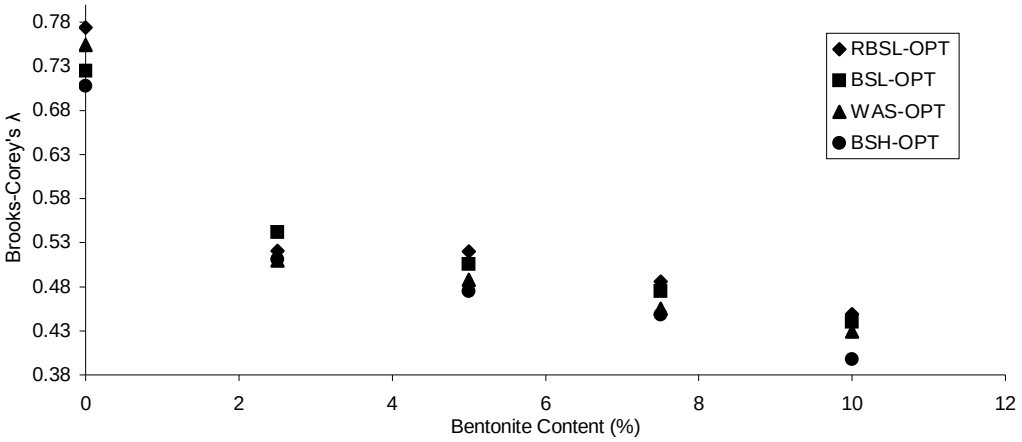


Fig. 5.95 Brooks – Corey's λ versus Bentonite content at optimum water content.

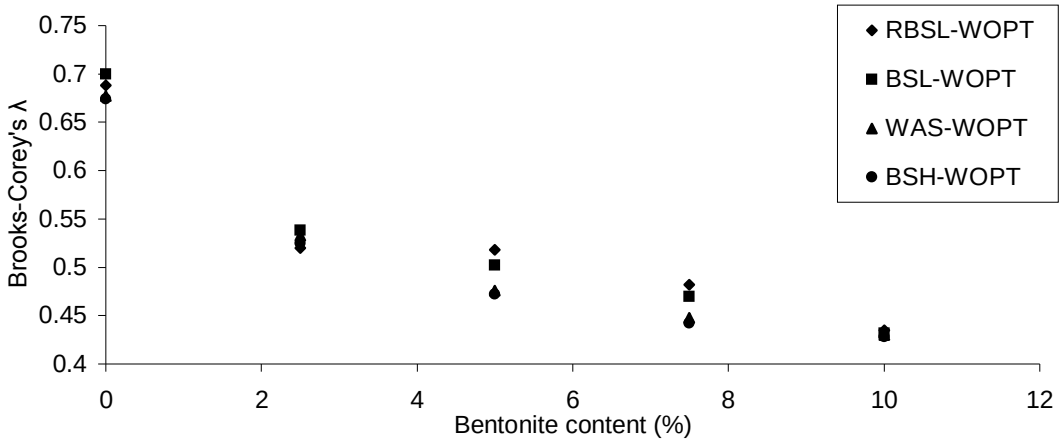


Fig. 5.96 Brooks – Corey's λ versus Bentonite content wet of optimum water content.

The residual volumetric water contents θ_r varied between the models. The effect of bentonite content on the residual volumetric water content for the various mixtures can be evaluated from the information provided in Figs. 5. 97-5.99. As expected θ_r increased with higher bentonite content. Also, for a given bentonite content, BC values of θ_r are marginally smaller than the VG values and ranged between 0 and 0.38. These values are within the range of values provided in the literature (Leong and Rahardjo, 1997; Tinjum, *et al.*, 1997).

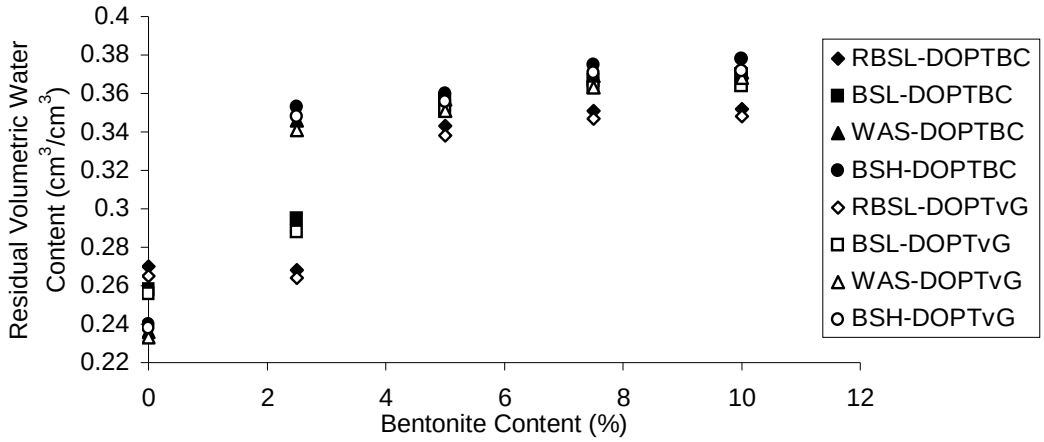


Fig. 5.97 Residual volumetric water content θ_r versus bentonite content dry of optimum water content.

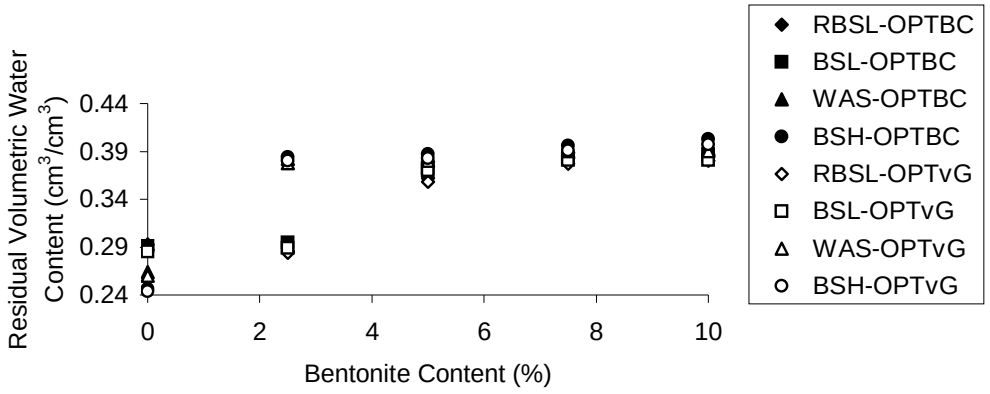


Fig. 5.98 Residual volumetric water content θ_r versus bentonite content at optimum water content.

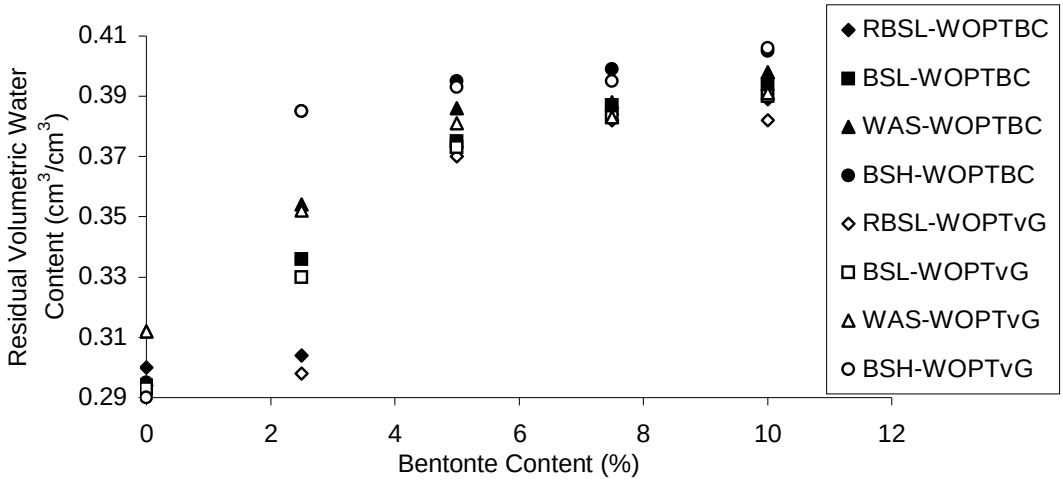
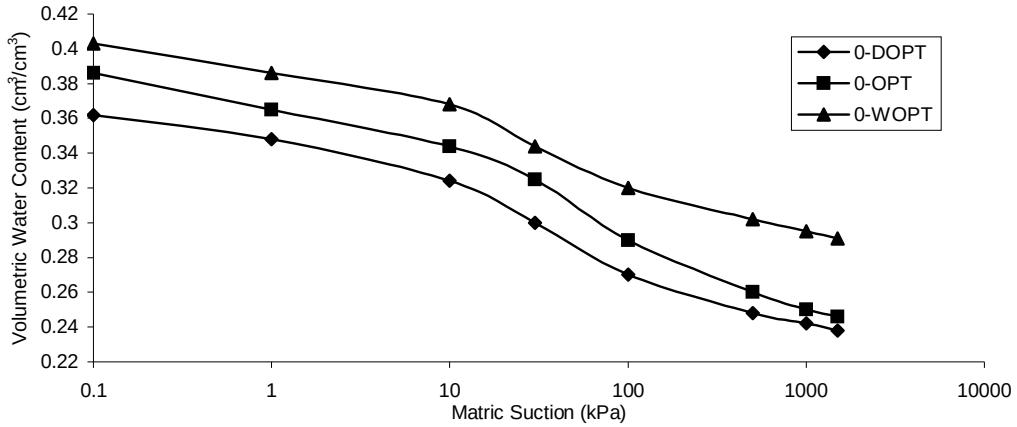


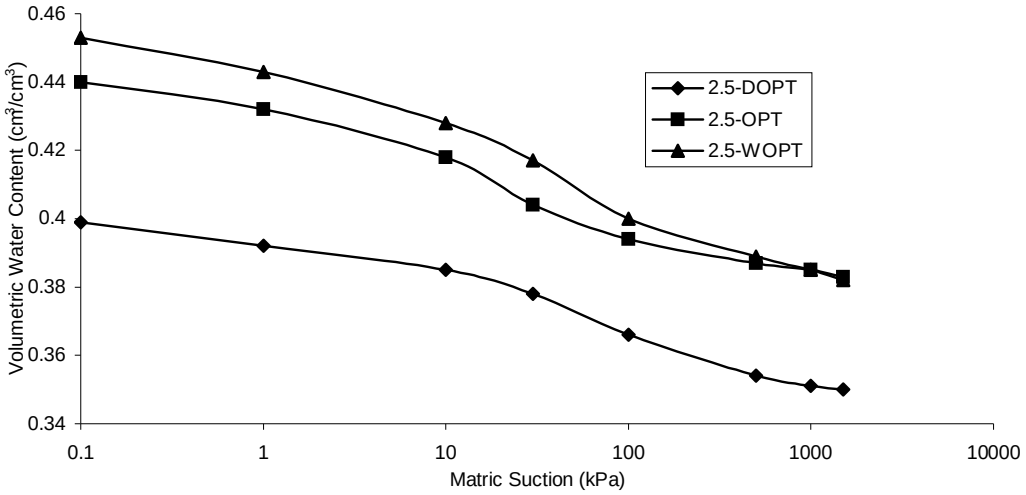
Fig. 5.99 Residual volumetric water content θ_r versus bentonite content at wet of optimum water content.

5.5.1.2 Effect of compaction conditions

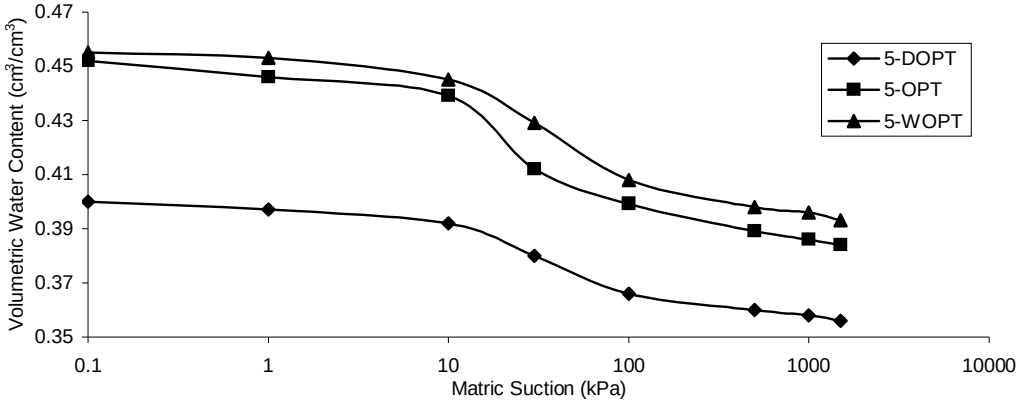
The impact of compaction water content on the resulting SWCCs is illustrated in Fig. 5.100 for specimens prepared dry, wet and at optimum water content using the BSH compactive efforts. The moulding water content has considerable influence on the resulting structure and aggregation of fine-grained soils (Vanapalli *et al.*, 1999). The resistance to water flow (desaturation) is relatively low in the dry of optimum specimens in comparison to optimum and wet of optimum specimens. The dry of optimum water content specimens contain relatively large pore spaces located between the clods of soil as compared to the pore spaces within the clods. Therefore, macro structure controls the SWCC for specimen compacted dry of optimum water content. Thus dry of optimum specimens act like coarse grained soil with a highly aggregated macro structure. The pore spaces in clayey soils compacted at water content wet of optimum are more homogeneous, have higher storage capacity due to their different structure and offer more resistance to desaturation under an applied suction in comparison to specimens compacted dry of optimum water content.



A

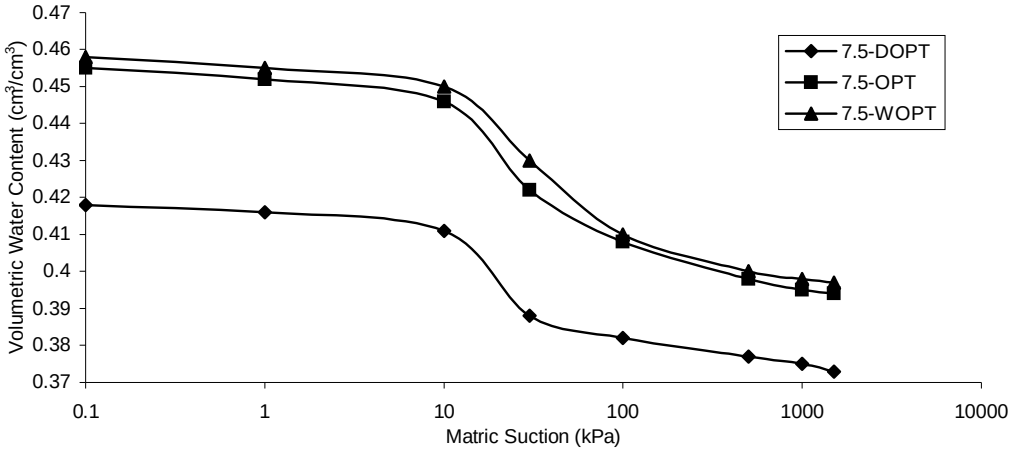


B

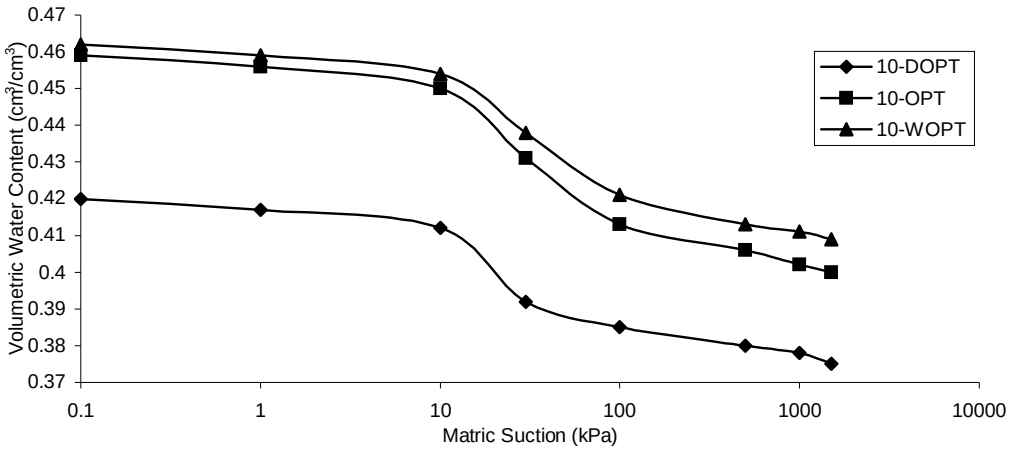


C

Fig. 5.100 Variation in SWCCs behaviour due to moulding water content for (A) 0, (B) 2.5 and (C) 5 % bentonite content at BSH effort



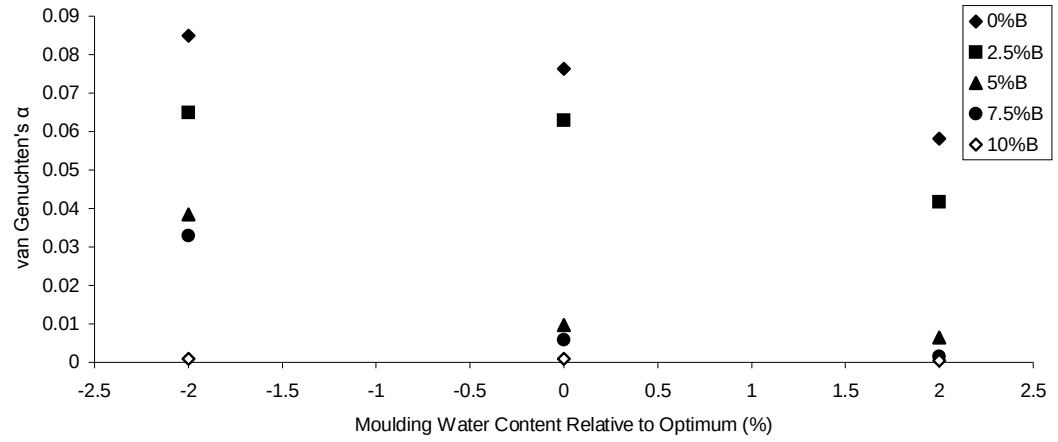
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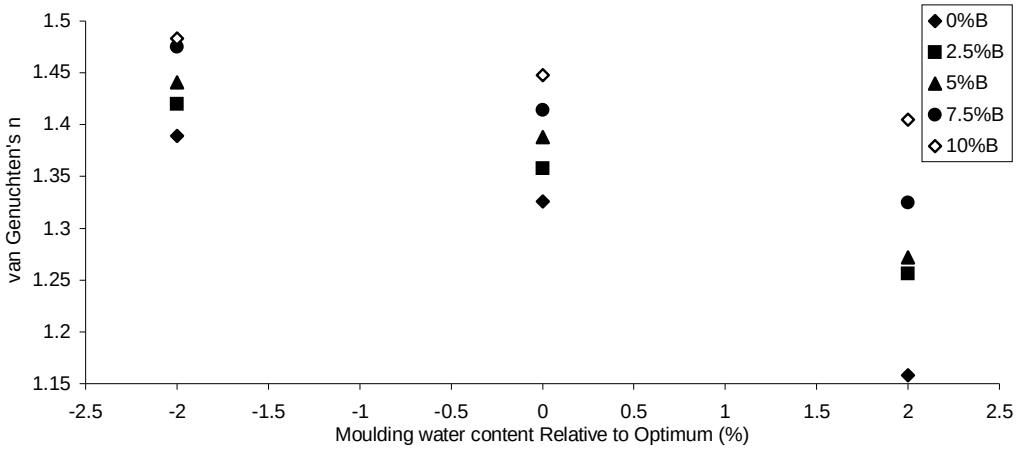
E

Fig. 5.100 Variation in SWCCs behaviour due to moulding water content for (D) 7.5 and (E) 10% bentonite content at BSH effort

Fig. 5.101 presents the relationship between fitting parameters, α and n of van Genuchten equation and compaction water content relative to optimum. α decreased with higher compaction water content, while n decreased with higher compaction water content. Similar trends exist for BC's ψ_a and λ. While ψ_a values increased with higher compaction water content, λ decreased with increasing compaction water content. These results are in agreement with data presented by Tinjum *et al.* (1997) for compacted clays as well as Nwaiwu (2004) for compacted lateritic soils proposed for use as landfill barrier.



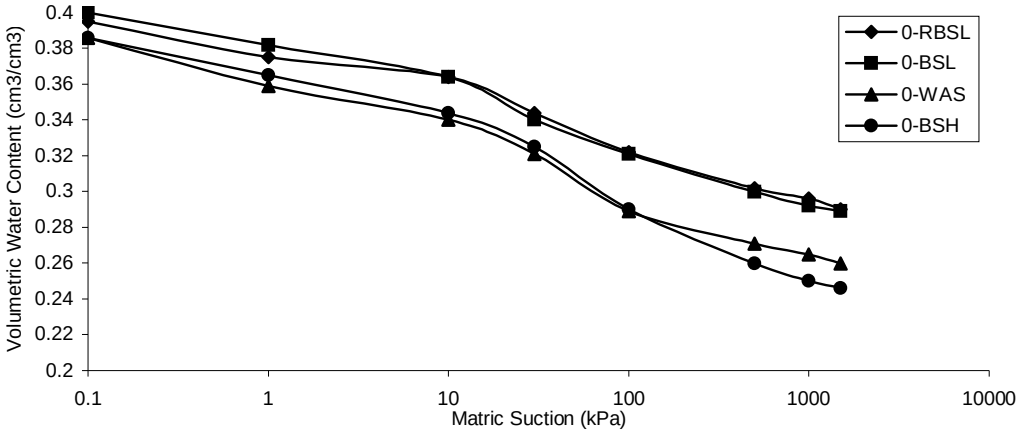
A



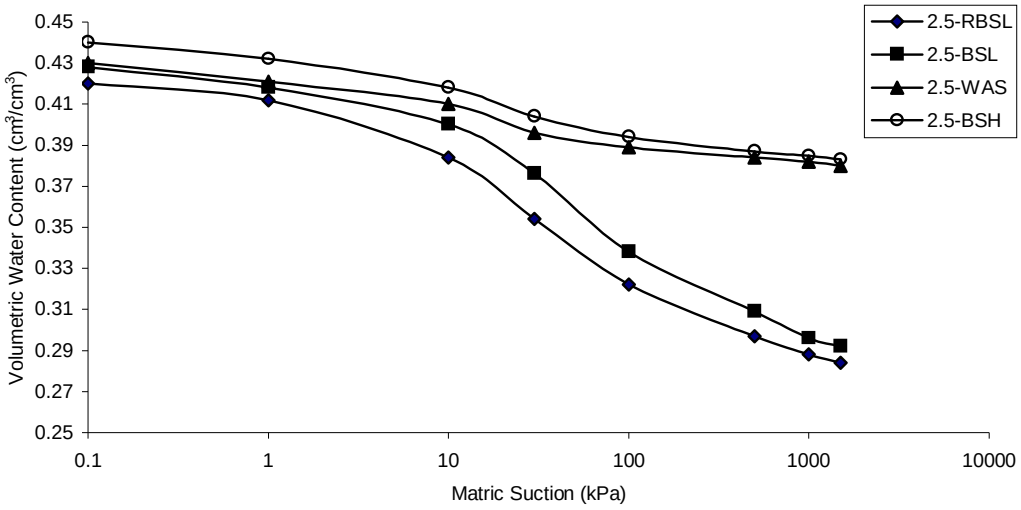
B

Fig. 5.101 van Genuchten's (A) α and (B) n versus compaction water content Relative to Optimum Water Content

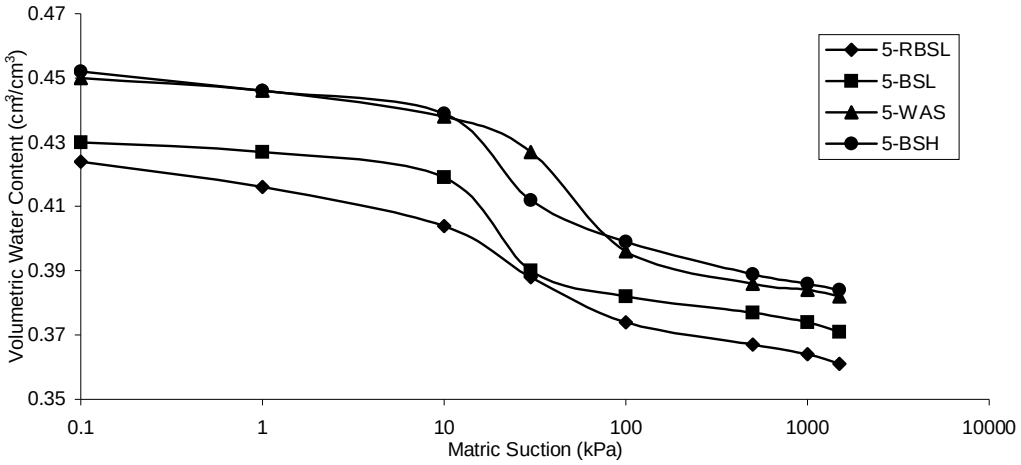
The influence of compactive effort on SWCCs for specimens prepared at optimum water content and compacted at the four energy levels is illustrated in Fig. 5.102. SWCCs of specimens compacted with higher efforts generally plot above the SWCCs of specimens compacted with lower compaction efforts (see Fig. 5.102). This is because higher compactive effort results in smaller pores that lead to increase in suction. Similar behaviour was observed by Tinjum *et al.* (1997) and Miller *et al*. (2002).



A

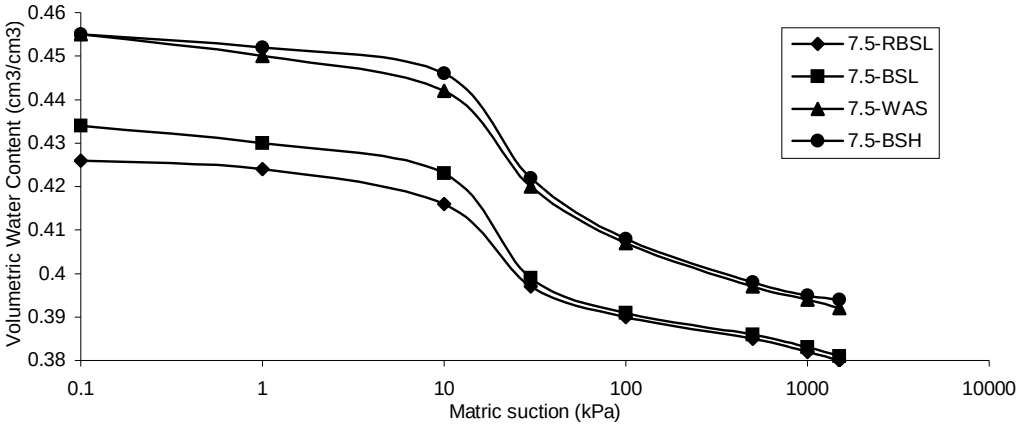


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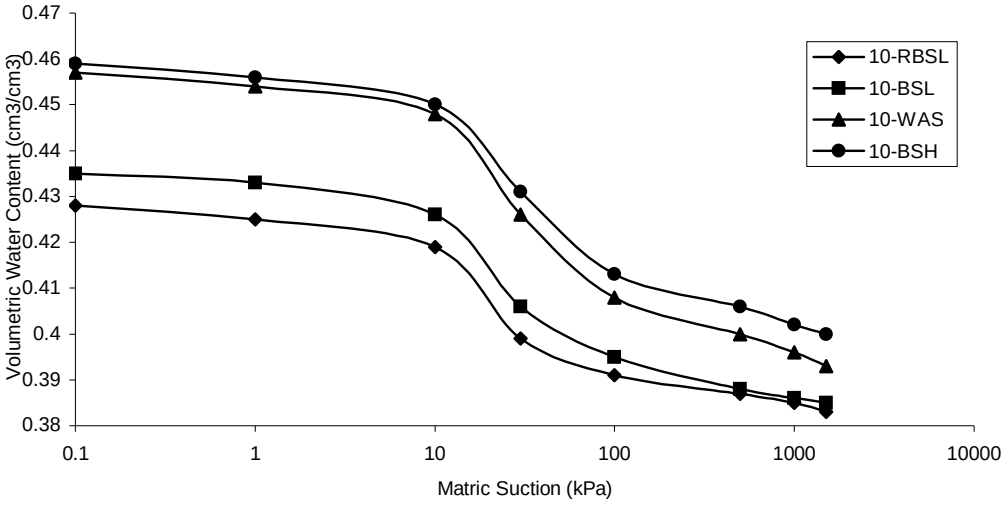


C

Fig. 5.102A,B,C Variation in SWCC due to compaction effort for (A) 0%, (B) 2.5% and (C) 5% Bentonite content.



D



E

Fig. 5.102D,E Variation in SWCC due to compaction effort for (D) 7.5 and (E) 10% bentonite

5.5.2 Unsaturated hydraulic conductivity

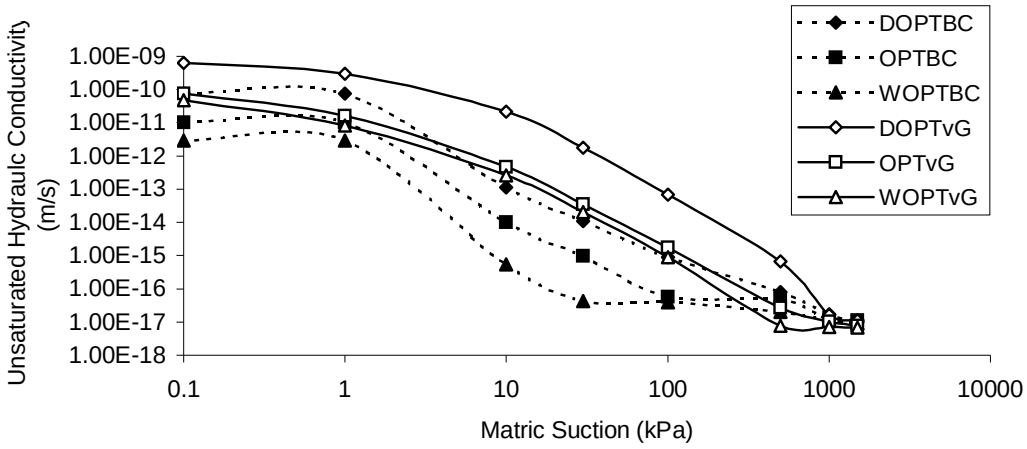
The relative hydraulic conductivity k_r for specimens were computed using equations 3.6, 3.7 and 3.8 in conjunction with Brooks - Corey and van Genuchten empirical parameters from SWCCs. The unsaturated hydraulic conductivity values were obtained by multiplying the k_r values with the saturated hydraulic conductivity k_{sat} values (see Eqns. 3.7) measured in the laboratory.

Two predicted k – ψ relationships are shown in each plot for the two functions used (i.e. Brooks – Corey and van Genuchten functions).

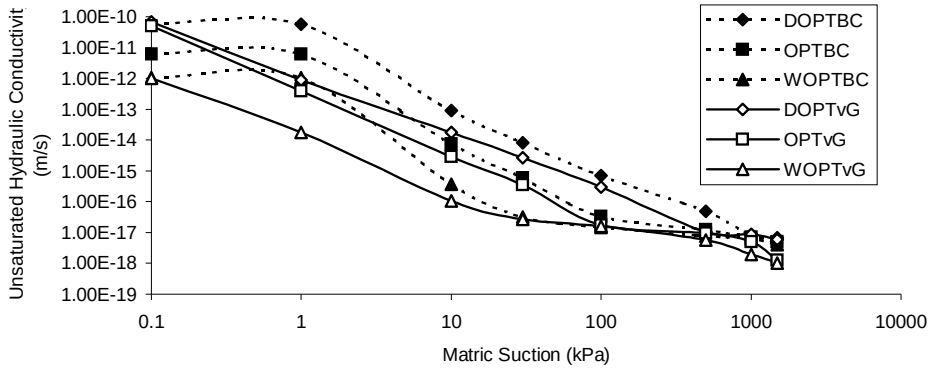
5.5.2.1 Effect of moulding water content

The unsaturated hydraulic conductivity of specimens prepared with BSH effort at moulding water content 2% dry of optimum, at optimum and at 2% wet of optimum water content for the various soil bentonite mixtures have been shown in Fig. 5.103A,B,C,D and E to illustrate the effect of moulding water content on unsaturated hydraulic conductivity. At lower matric suctions specimens compacted wet of optimum water content generally had lower hydraulic conductivity, but as the suction increased, the disparity reduced irrespective of the relative hydraulic conductivity function (either BCB or VGM) i.e the structural changes that occur varying the moulding water content Garcia-Bengochea *et al.*, 1979; Benson and Daniel, 1990) resulted in insignificant variation in the unsaturated hydraulic conductivity at matric suctions greater than 500kPa.

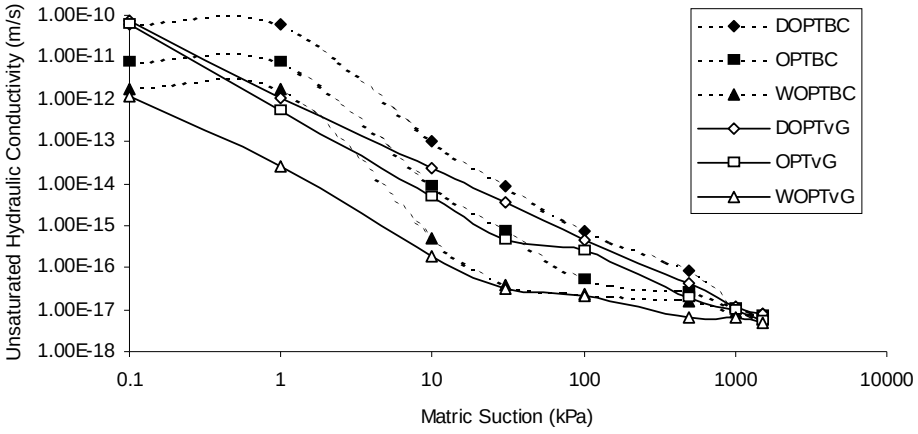
Under unsaturated conditions, the largest pores empty first and fill last (Hillel, 1982). The larger pores responsible for high k dry of optimum gradually become hydraulically inactive as the soil desaturates (Benson and Khire, 1995; Meerdink *et al.*, 1996).



A

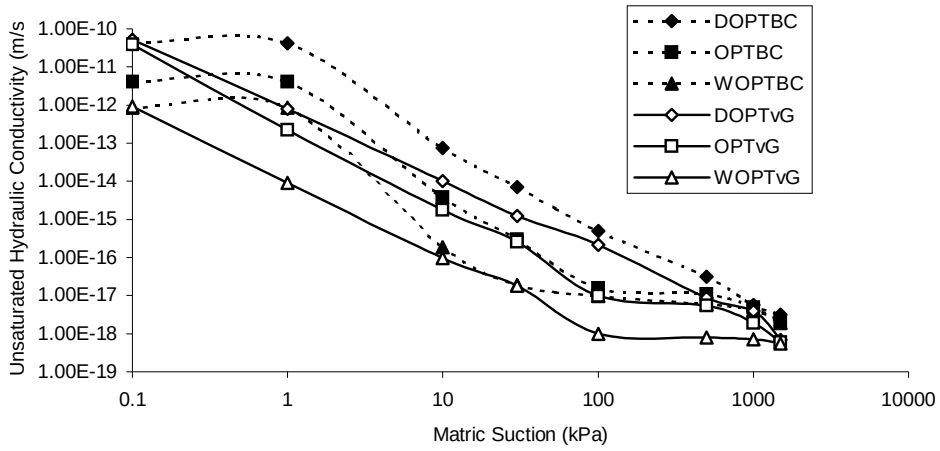


B

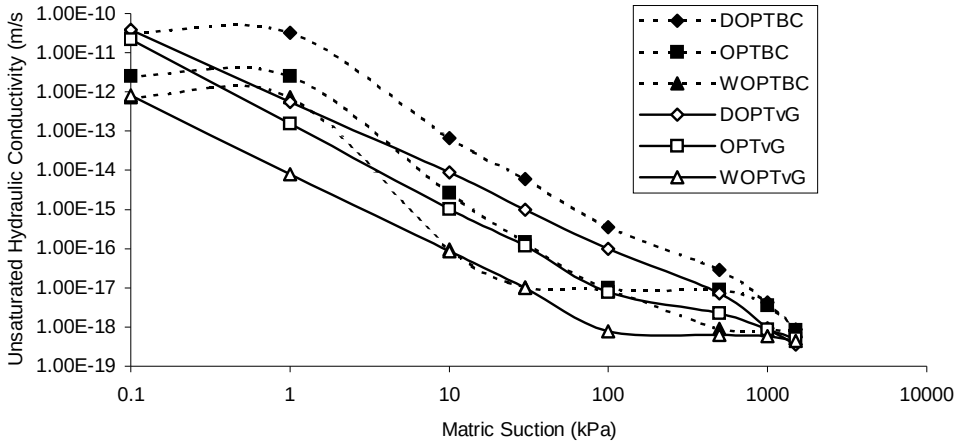


C

Fig. 5.103A,B,C Unsaturated Hydraulic Conductivity of (A) 0 and (B) 2.5, (C) 5% lateritic soil – Bentonite mixtures as a function of Matric suction at Dry, Optimum and Wet of optimum water content for British Standard Heavy Compactive effort.



D



E

Fig. 5.103D,E. Unsaturated Hydraulic Conductivity of (D) 7.5 and (E) 10% lateritic soil – Bentonite mixtures as a function of Matric suction at Dry, Optimum and Wet of optimum water content for British Standard Heavy compactive effort based on BCB and VGM equations.

5.5.2.2 Effect of compactive effort

In order to evaluate the effect of compaction effort on unsaturated hydraulic conductivity, specimens of the various mixes prepared at optimum water content with the four compaction energies namely: RBSL, BSL, WAS and BSH efforts were used for illustration. For all the mixtures and at all pressures measured, specimens compacted with BSH compaction effort had unsaturated hydraulic conductivities lower than those of specimens compacted at lower energy levels regardless of the relative hydraulic conductivity function used (see Fig. 5.104).

The saturated hydraulic conductivity test results followed this trend. This result is consistent with the result of previous studies based on compacted naturally occurring clay soils by Meerdink *et al*. (1996). It is believed that specimens compacted with the higher efforts had lower hydraulic conductivities because such compaction energies reduce the frequency of large pores, reduce the void ratio and decrease the average pore size (Garcia-Bengochea *et al*., 1979; Acar and Oliveri, 1989; Benson and Daniel, 1990). The reductions in large pore frequency and average pore size should result in lower unsaturated hydraulic conductivity.

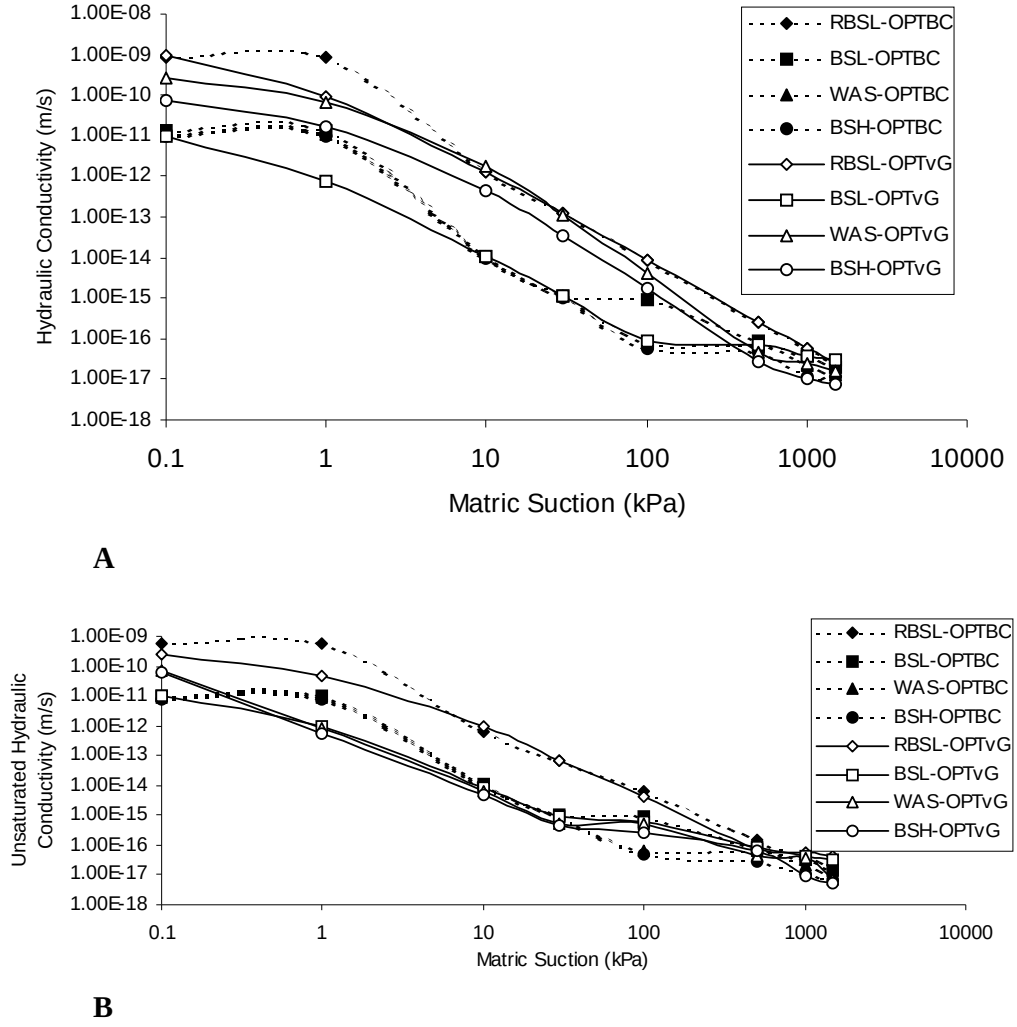
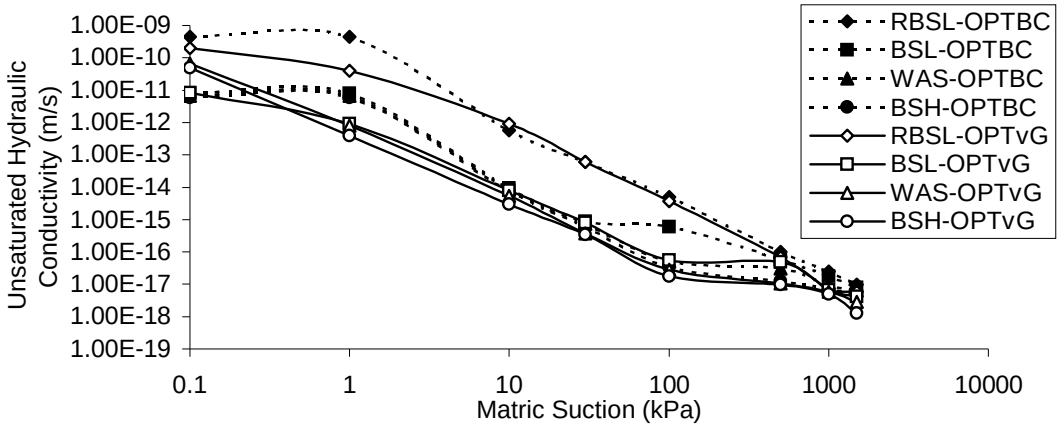
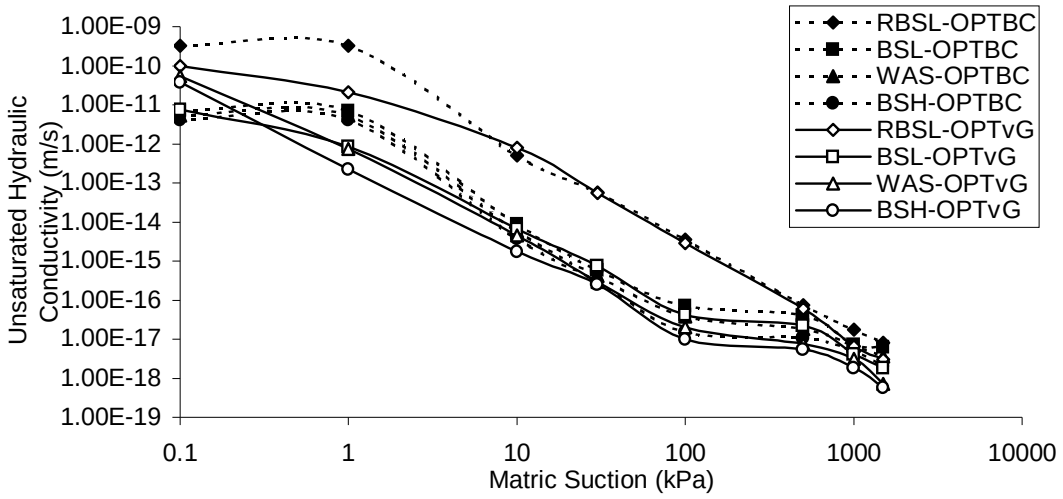


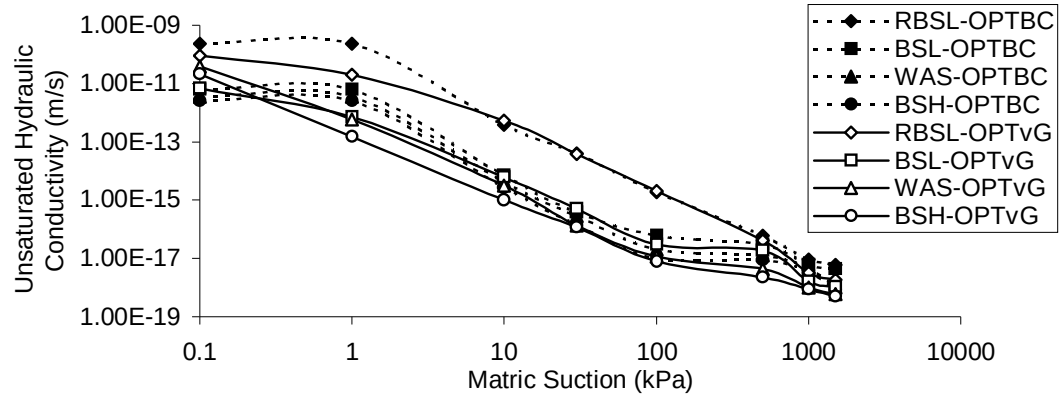
Fig. 5.104A,B Unsaturated Hydraulic Conductivity of (A) 0%, and (B) 2.5% Bentonite content as a function of Matric suction compacted with RBSL, BSL, WAS and BSH efforts at Optimum water content based on BCB and VGM equations.



C



D

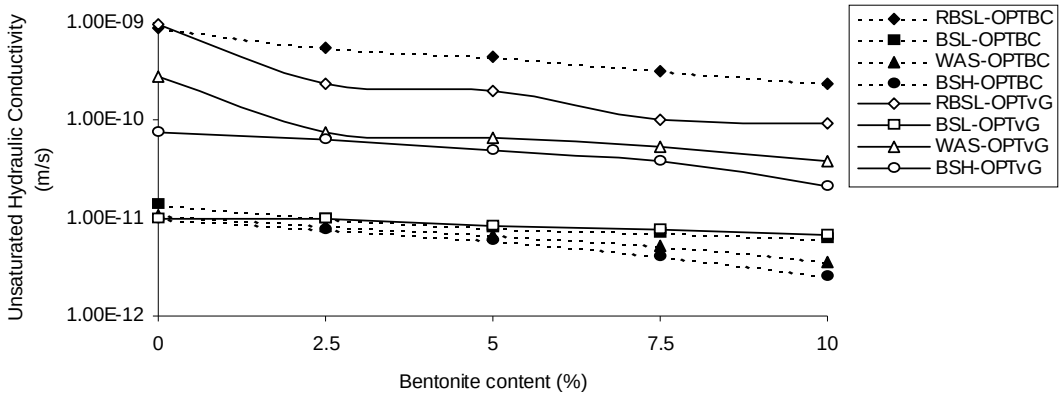


E

Fig. 5.104C,D,E Unsaturated Hydraulic Conductivity of (C) 5, (D) 7.5, (E) 10% Bentonite content as a function of Matric suction compacted with RBSL, BSL, WAS and BSH efforts at Optimum water content based on BCB and VGM equations.

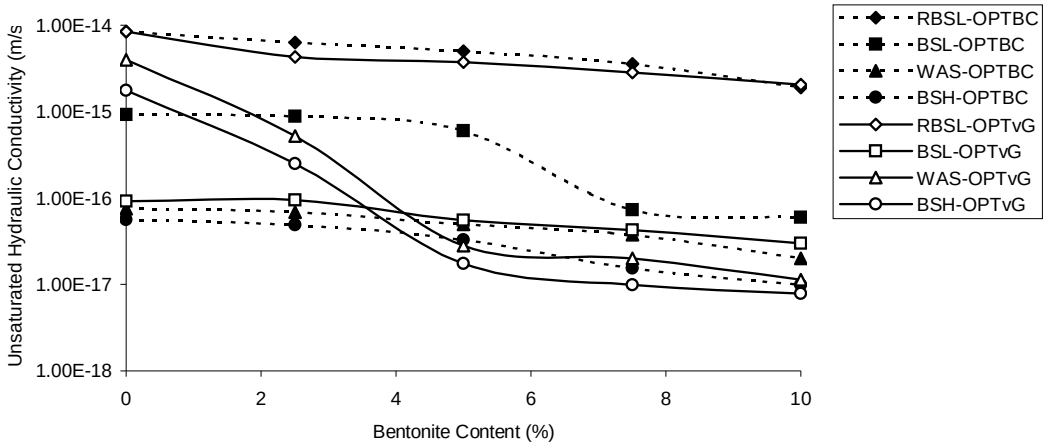
5.5.2.3 Effect of bentonite content

The effect of bentonite content on compacted specimens for suctions, ψ 0.1, 100, and 1500kPa were used to illustrate the influence of bentonite (see Fig. 105A,B and C). The value of unsaturated hydraulic conductivity decreased slightly with increasing bentonite content irrespective of the matric suction and the relative hydraulic conductivity function used. However, the predicted unsaturated hydraulic conductivity based on VGM function decreased more at higher suctions (i.e.100 and 1500kPa). For example, for specimens compacted at optimum water content with BSH compaction effort, when k is based on VGM, k decreased from 1.75E-15 for 0% bentonite content to 7.88E-18 for 10% bentonite at ψ = 100kPa, while k decreased from 5.64E-17 to 9.88E-18 at the same suction range and bentonite content for BCB function.

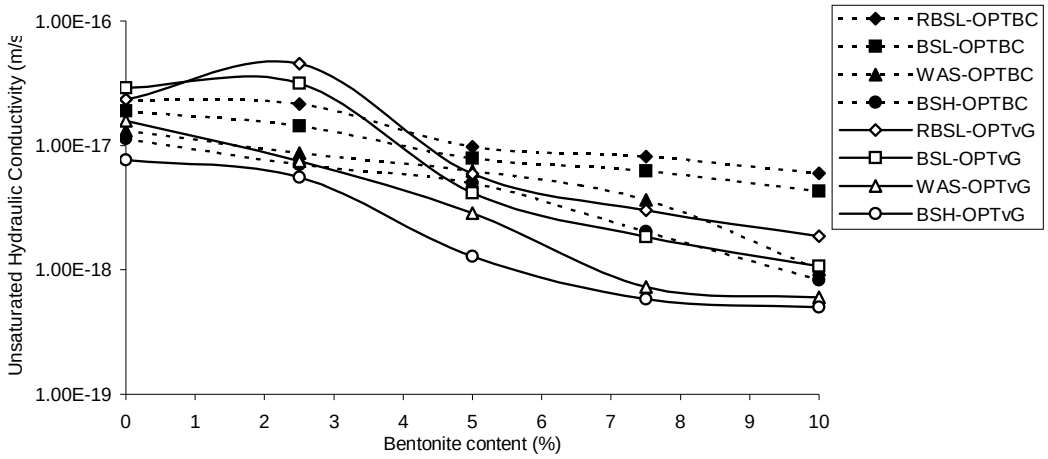


A

Fig. 5.105A Predicted Unsaturated Hydraulic Conductivity as a function of Bentonite content at (a) 0.1kPa Matric suctions for VGM and BCB equations.



B



C

Fig. 5.105B,C Predicted Unsaturated Hydraulic Conductivity as a function of Bentonite content at (B) 100 and (C) 1500kPa Matric suctions for VGM and BCB equations.

5.6 Contaminant Transport in Compacted Lateritic Soil-Bentonite Mixtures

5.6.1 Hydraulic conductivity tests for evaluating chemical compatibility

The hydraulic conductivity test data for the various compacted mixtures permeated with multispecies MSW leachate solution for 90 days is shown in Fig. 5.106. During prehydration, i,e, when hydrating specimens with tap water prior to permeation with actual solution (Shackelford and Stern, 1998), only a small change in hydraulic conductivity occurred. For example, the hydraulic conductivity for 0% bentonite content initially 3.2 x 10^{-9} m/s, decreased by a factor of 1.3, Similar decreases were observed in the other mixtures: 1.35 for

specimen containing 2.5% bentonite, 1.4 for 5% and 7.5% bentonite and 1.5 for 10% bentonite at the end of prehydration testing.

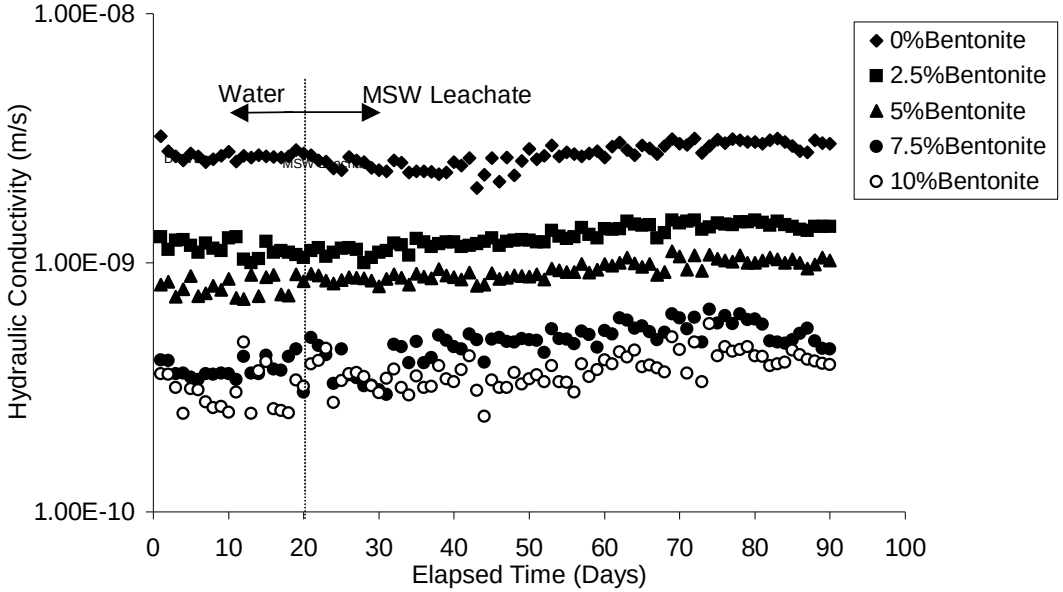


Fig. 5.106 Variation of Hydraulic conductivity with Elapsed time for soil-Bentonite mixtures using MSW Leachate solution as permeant liquid

On introduction of the MSW leachate solution the hydraulic conductivity for the various compacted mixtures was essentially the same as obtained with tap water. However, slight increases in the hydraulic conductivity of all the specimens was observed towards the end of test duration. Overall the hydraulic conductivity of specimens increased by a factor of 1.2 to 1.7. These gradual changes in hydraulic conductivity are similar to those reported by Petrov and Rowe 1997; Shackelford *et al*., 2000; Egloffestein, 2001and Jo *et al.*, 2001. This implies that long term exposure of lateritic soil – bentonite mixtures to MSW leachate will no have serious adverse effects.

5.6.2 Free swell test method for evaluating chemical compatibility of specimens

The results of free swell of specimens at 100 hours in deionised water as well as in MSW leachate solutions are presented in Fig. 5.107.

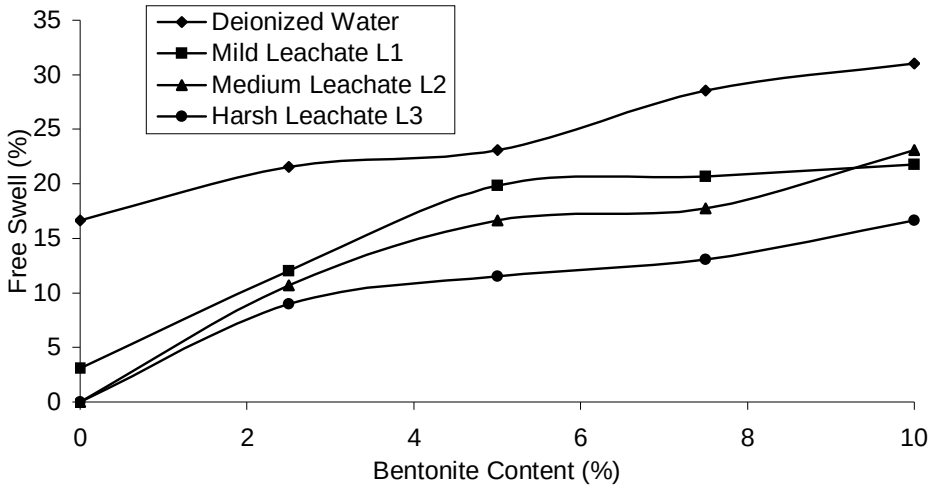


Fig. 5.107 Variation of free swell of soil-bentonite mixtures in MSW leachate solutions/deionised water with bentonite content.

As shown in Fig. 5.107, free swell increased almost linearly as the bentonite content increased from 0 to 10%. It is also observed that free swell values with deionised water for all the specimens are higher than values obtained with leachate solutions recording the highest free swell of 31% associated with specimen containing 10% bentonite followed by other specimen in order of decreasing bentonite content. The higher values with water are attributed to the relatively high swelling potential of bentonite in the presence of water. The low values recorded upon hydration with leachates indicate that the mixtures are less reactive and this is consistent with flocculation of the bentonite in the multispecies leachate solution (Shackelford, 1994).

In general, leachate solutions influenced the free swell of the mixtures. In a soil - solution interaction, the thickness of double diffuse layer is influenced by pore fluid chemistry, an increase in concentration in the pore fluid causes a decrease in adsorbed layer thickness

(Mitchell, 1993), hence the greatest swell occurred when the hydrating liquid is deionised water because the adsorbed layer is at maximum thickness. A significant decrease in free swell relative to that obtained with deionised water is indicative of shrinkage of the adsorbed layer and a potential incompatibility between the specimen and the liquid (Mitchell, 1993).

5.6.3 Relationship between free swell and hydraulic conductivity of specimens

When the free swell of specimens at 100 hours was correlated with long term hydraulic conductivity test results on identically prepared specimens compacted at optimum water content with British Standard Light effort (BS 1377, 1990) and permeated with tap water and the leachates solutions in a falling head apparatus, a strong relationship between hydraulic conductivity and free swell was apparent.

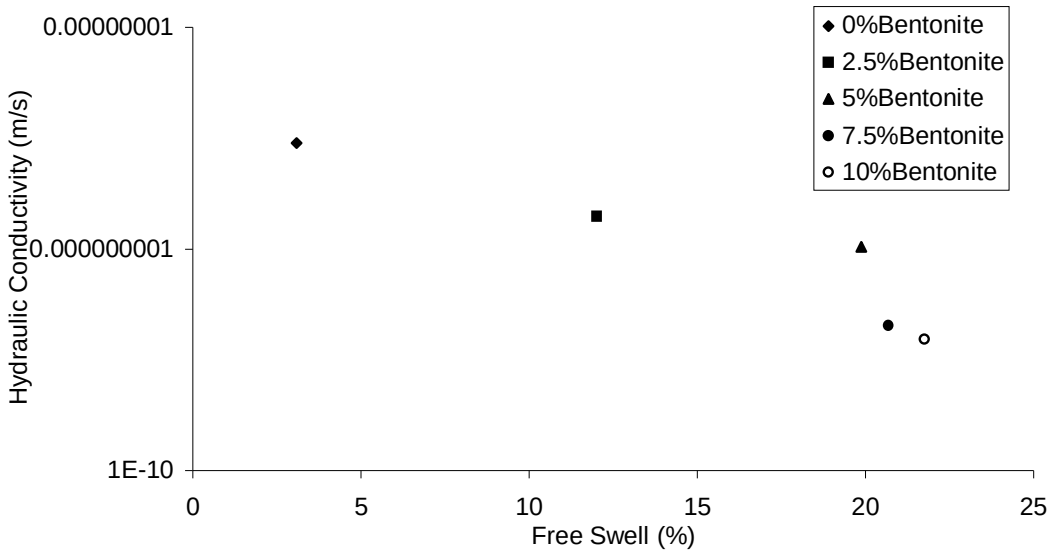


Fig. 5.108 Relationship between hydraulic conductivity at 90-day permeation with free swell at 100-hour hydration for the various mixtures.

The hydraulic conductivity of the compacted mixtures showed an inverse relationship with the swell potential, i.e., a decrease in free swell will result in an increase in hydraulic conductivity, k. Similar result was reported by Lin and Benson, 2000 as well as Shackelford *et al.*, 2000 for bentonite from geosynthetic clay liners (GCL). In the field, increases in

hydraulic conductivity of liners are often traced to compatibility problems with the liquid being contained (eg. Goldman *et al*., 1986; Daniel and Brown, 1988).

As with the results of the free swell tests, the changes in hydraulic conductivities are consistent with changes in the thickness of the adsorbed layer (Narejo and Memon, 1995).

According to Gouy-Chapman theory, as the ionic concentration in the leachates increased from mild to harsh, the double diffuse layer (DDL) decreased, consequently, the swell volume decreased while the hydraulic conductivity increased (Mitchell, 1993; Gleason *et al*., 1997; Rowe *et al*., 2001).

Therefore, drastic reduction in free swell of specimens as compared to the values obtained when hydrated with water may signify chemical compatibility problems.

5.6.4 Sorption performance assessment of a lateritic soil – bentonite mixtures

Sorption parameters are important components of mass transport models used to predict the migration of contaminants through compacted earthen materials. In practice, these parameters are generally evaluated using either batch or column techniques.

Data obtained from batch adsorption tests conducted for lateritic soil - bentonite mixtures with MSW leachate solutions to quantify the potential adsorption of some specified ions namely K^+, Na^+ and Pb^+ are presented using adsorption isotherms shown in Figs. 5.109 – 5.111 for these ions. Generally, the equilibrium isotherms for Na and K had a strong Fredluinch character and are fitted using Freundlich adsorption equation to obtain the maximum capacity of the material while Pb exhibited linear characteristics, an indication that the mass of the contaminant, q, removed from the solution is proportional to the equilibrium concentration c_f.

The solute adsorbed per mass of soil solids, q, was determined from the following expression:

$$q = \frac{(c_i - c_e)V}{Ms} mg/g \qquad (5.1)$$

where c_i = Initial concentration of the specific constituent contaminant mg/l, V = Volume of solution ml, c_e = Equilibrium concentration, mg/l and M_s = The soil mass, g

The slope of the adsorption isotherm is called the distribution coefficient K_d for linear adsorption isotherms and partitioning coefficient K_p for non linear adsorption isotherms.

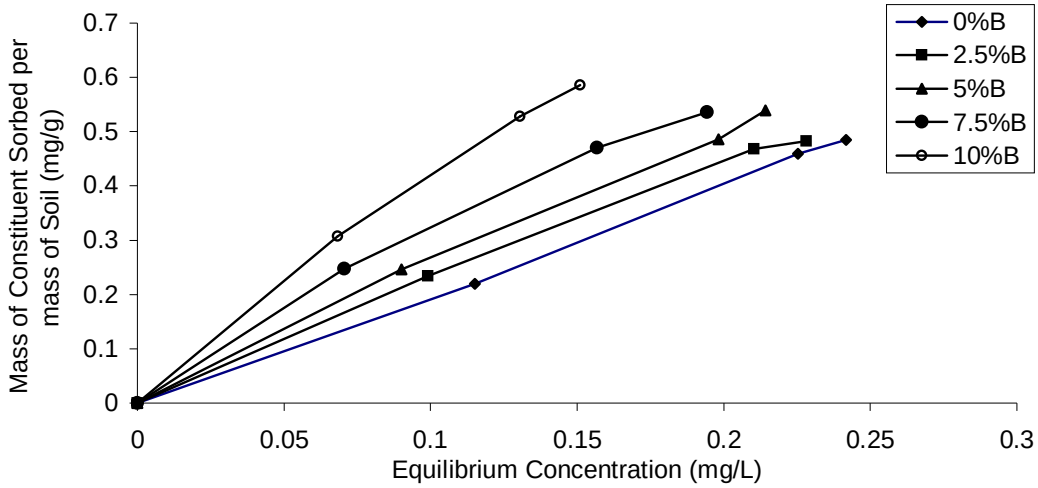


Fig. 5.109 Adsorption isotherm for Pb

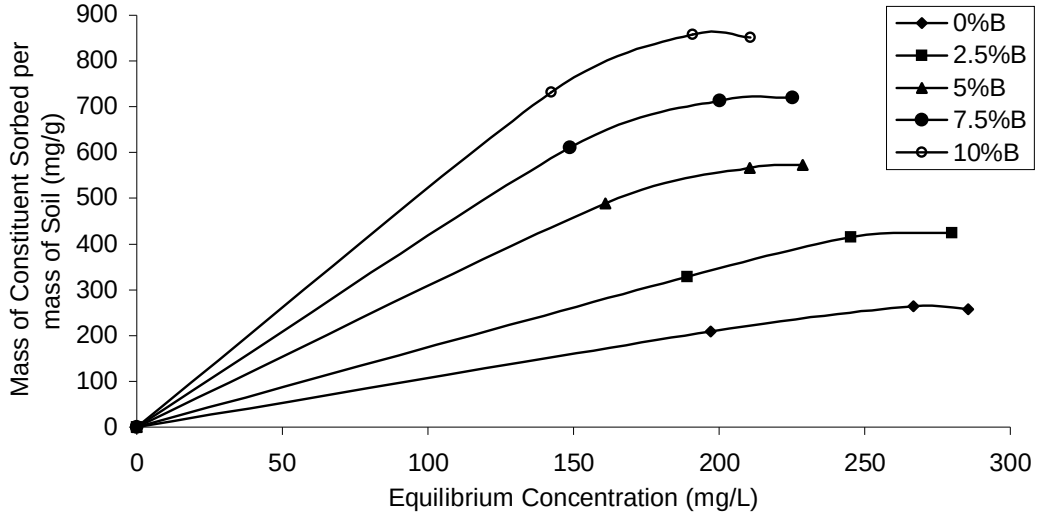


Fig. 5.110 Adsorption isotherm for Potassium

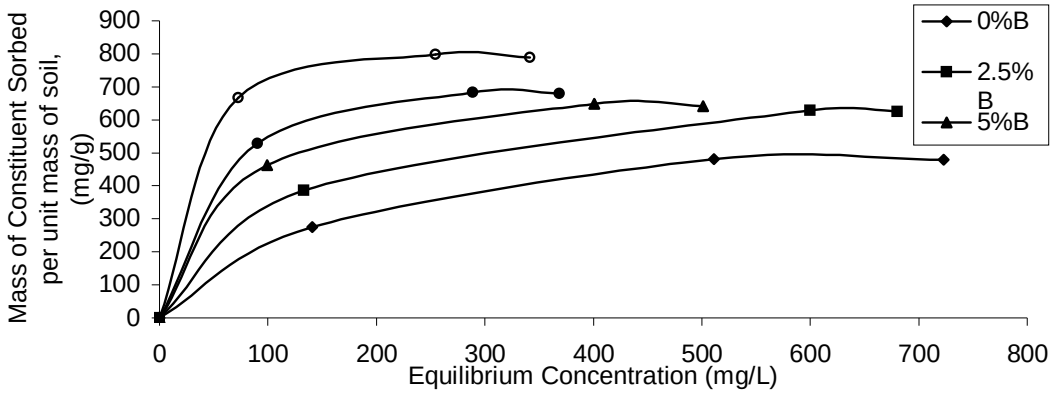


Fig. 5.111 Adsorption isotherm for Sodium

5.6.5 **Partition/distribution coefficient**

The partition/distribution coefficient K_p/ K_d is defined as the ratio of the adsorbed concentration on the clay surfaces to concentration of solution in equilibrium.

The distribution coefficient, K_d, is given by a linear equation (Rowe *et al*., 1995):

$$K_d = \frac{dq}{dc_e} \quad \text{ml/g} \tag{5.2}$$

$$q = K_d c_e \tag{5.3}$$

While nonlinear adsorption behaviour is described by a Freundlich isotherm equation, as

$$q = K_f c_e^{\,n} \tag{5.4}$$

where q is the adsorbed concentration, K_f and n are the Freundlich isotherm equation parameters. The partitioning coefficient is given as

$$K_p = K_f c_e^{\,n-1} \tag{5.5}$$

The values of partitioning/distribution coefficients of the cations used in this study obtained from eqs. 5.2 and 5.4 are summarized in Table 5.8

Adsorption of metal ions followed the expected trend of increased metal adsorption characteristics with increased bentonite content. This is demonstrated by the distribution /partitioning coefficients of the ions presented in Table 5.8 For 0% bentonite content, K_d values were 2.006 (Pb), 0.2 (Na) and 1.25 (K) while for samples containing 10% bentonite,

K_d values were 3.973 (Pb), 0.457 (Na) and 3.03 (K). This shows that the removal capacity of laterite soil - bentonite mixture at 10% bentonite is four fold greater than pure lateritic soil for K, three times for Na and two for Pb.

As highlighted in the Table, the minimum K_p was found in Na which represents the minimum adsorption on the clay platelet surfaces. The maximum K_p (= 3.975) was found in specimens containing 10% bentonite indicating the maximum adsorption on the clay platelet surfaces. It can be observed that the higher concentrated diffusants results in a lower K_p due to the collapse of the clay platelets as a result of shrinkage in the double diffuse layer and the formation of more aggregated particles where the total adsorption capacity per unit surface area decreases compared with the increasing concentration of pore fluid in equilibrium.

Table 5.8 The partitioning/distribution coefficients of the cations

	K_d	K_p	
Bentonite content (%)	Pb	Na	K
0	2.006	.20	.769
2.5	2.215	.293	1.25
5	2.519	.356	1.852
7.5	2.756	.412	2.381
10	3.973	.457	3.03

The study established that sorption capacities are influenced by the chemical composition of the soils and due to the sorption process, transported heavy metals are significantly attenuated in the soil material. The mobility/migration rates of ions were also reduced as bentonite content in the compacted mixtures increased since greater distribution/partitioning coefficients results in slower migration rate.

5.6.6 Retardation factor

The Retardation factor, R_d, a correction factor that takes into account the bulk density of the media, the porosity and the distribution/partition coefficient K_d was determined by (Shackelford, 1994; Shackelford and Daniel, 1991):

$$R_d = 1 + \frac{\rho_d}{n_e} K_d \tag{5.6}$$

or

$$R_d = 1 + \frac{\rho_d}{n_e} K_p \tag{5.7}$$

where ρ_d and n_e are the dry or bulk density and effective porosity, respectively of the soil. The retardation factors R_d for the cations determined with eqn. 5.7 using data in Table 5.8 are presented in Table 5.9.

Table 5.9 The retardation factors R_d for the ions

Bentonite content (%)	Retardation Factor		
	Pb	Na	K
0	12.268	2.123	5.321
2.5	12.932	2.559	7.734
5	14.249	2.871	10.74
7.5	14.911	3.077	13.018
10	20.301	3.219	15.721

The retardation factor for all the cations is found to be more than unity which indicate that adsorption happens on the surface of clay platelets due to diffusion flow of cations (Muhammed, 2004). The smallest retardation factor was obtained for all cations with the more porous specimen containing 0% bentonite. This is probably because of their minimum resistance to diffusion flow. The 10% bentonite specimen produced the maximum retardation

factor indicating a maximum resistance to diffusion on bentonite platelets. The thickness of double diffuse layer is higher in mixtures with higher bentonite content which result in a zone of immobility within the pore spaces. By comparing tests with 0, 2.5, 5, 7.5 and 10% bentonite content, it can be concluded that mixtures with lower bentonite content generate lower R_d because of the lower double diffuse layer thickness which eventually creates more available pore spaces for solute mobility.

5.6.7 Diffusion coefficient

Two diffusion parameters namely: Effective diffusion coefficient and tortuosity factor are calculated in this study.

5.6.7.1 Diffusion data evaluation

The suitability of a clay liner has been conventionally evaluated on the basis of hydraulic conductivity to control advective contaminant transport. Previous works however indicate that chemical diffusion is an important transport process in liner systems with low hydraulic conductivities and must be taken into consideration (Quigley and Rowe, 1986; Rowe et al., 1988 etc). Therefore, for a well engineered compacted clay landfill liner, diffusive transport through the liner is the main mass transport mechanism from the landfill. Hence, accurate estimates of diffusion coefficients is critical for predicting migration rates and fluxes of contaminants that are input parameters used in the engineering design of clay – based barrier materials in waste containment systems. Diffusion characteristics of compacted lateritic soil bentonite mixtures using a representative municipal solid waste leachate were evaluated by a laboratory diffusion column test. Test data were evaluated using the analytical solution to Fick's second law for one dimensional diffusion through saturated soil for the case of a decreasing source concentration and finite cell length (Crank, 1975; Shackelford, 1991).

Thus, the effective diffusion coefficients of cations considered in this study were calculated using eqns. 5.8, 5.9 and 5.10.

$$\frac{c(x \geq 0, t)}{c_0} = \frac{\alpha}{1+\alpha} + \sum_{m=1}^{\infty} \frac{2\alpha}{1+\alpha+\alpha^2 q_m^2} \exp\left(-\frac{D^* q_m^2 t}{R_d L^2}\right) \frac{\cos\left[q_m\left(1-\frac{x}{L}\right)\right]}{\cos(q_m)} \quad (5.8)$$

where all q_m = nonzero positive roots of the following functions:

$$\tan(q_m) = -\alpha q_m \quad (5.9)$$

and α = a constant defined as follows:

$$\alpha = \frac{H_L}{nR_d L} \quad (5.10)$$

where H_L and L = height of the liquid n the source reservoir and length of the soil specimen, respectively and n = total porosity of the soil specimen.

5.6.7.2 Variation of post diffusion water content with depth

The water content profile determined for each slice of soil samples represent the water contents after diffusion testing. The pattern of variations of water content with depth is shown in Fig. 5.112 for all the compacted mixtures of lateritic soil and bentonite under study. The water content profile gives an indication of the degree of saturation of soil samples which is a major factor affecting value of effective diffusion coefficient. In practice, diffuson testings are performed only on saturated soil samples because of the greater difficulty in performing diffusion tests on unsaturated soils.

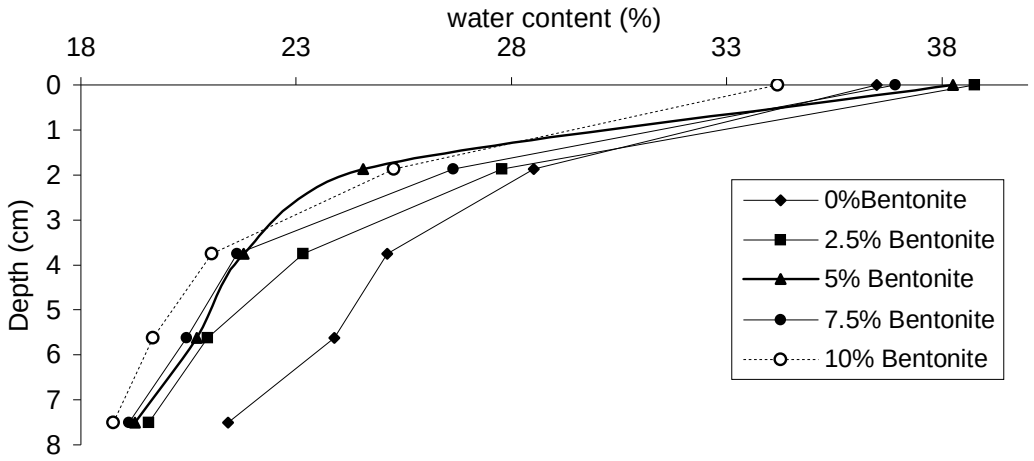


Fig. 5.112 Variation of Water with depth after diffusion testing.

5.6.7.3 Variation of pore water concentration with depth

The concentration profiles in compacted lateritic soil – bentonite column for the cations of interest namely Lead, Sodium and Potassium are presented in Figs. 5.113 – 5.115.

Due to sorption processes the distribution of heavy metals such as Pb will be limited to short distances in clay liners such as lateritic soil treated with bentonite.

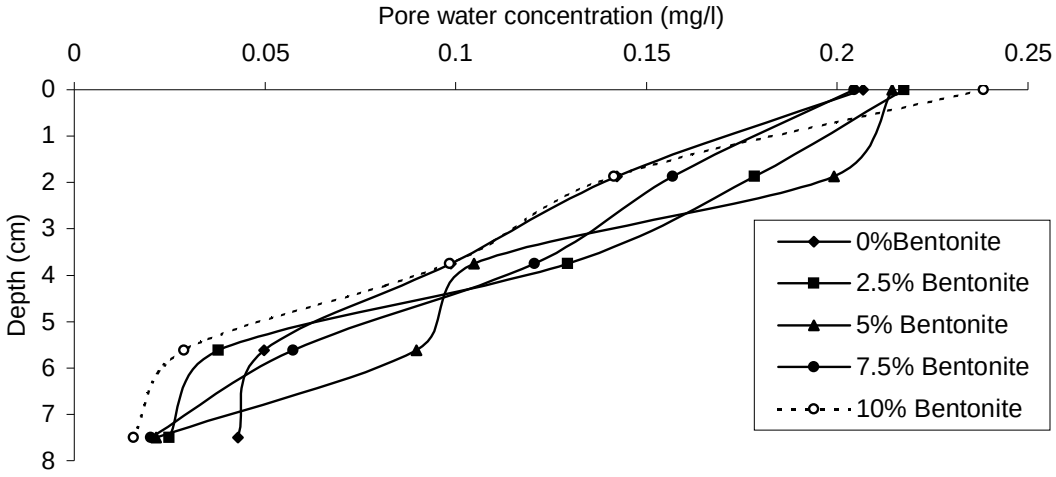


Fig. 5.113 Variation of Pore water concentration with depth for Pb after diffusion testing.

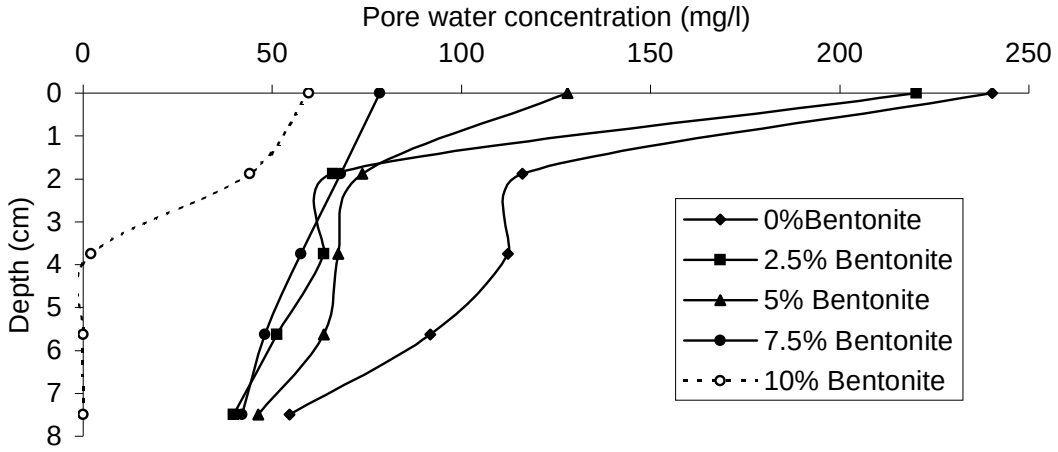


Fig. 5.114 Variation of Pore water concentration with depth for Na after diffusion testing.

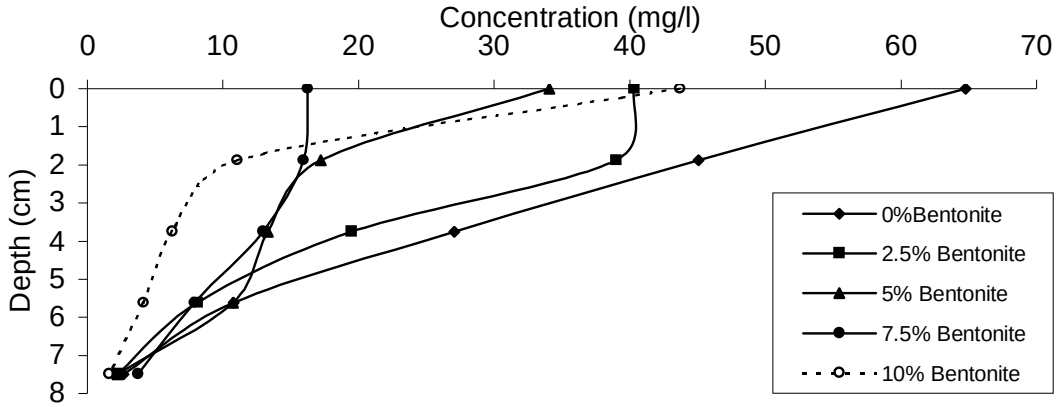


Fig. 5.115 Variation of Pore water concentration with depth for K after diffusion testing.

5.6.7.4 Effective diffusion coefficients for some MSW contaminants

Comparing the tests of same soil thickness using some diffusant concentration and for the range of conditions examined, effective diffusion coefficient D* determined from soil concentration - depth profile varied from 3.84 x 10^{-10} m^2/s to 2.0 x10^{-10} for Pb, 3.97 x 10^{-10} m^2/s to 3.06 x 10^{-10} m^2/s for Na and 5.01 x 10^{-10} m^2/s to 4.12 x10^{-10} m^2/s for K. The calculated effective diffusion coefficients are tabulated in Table 5.10. K cations were found to diffuse faster than the other two cations. The effective diffusion coefficients of the cations were found to be in the order of K > Na > Pb. Diffusion also decreased as the bentonite content increased for all the cations

Table 5.10 Effective Diffusion Coefficients D* for compacted Lateritic soil Bentonite based on soil concentration profile

Bentonite content (%)	D* x 10^{-10} m^2/s		
	Pb	Na	K
0	3.84	3.97	5.01
2.5	2.97	3.56	4.95
5	2.59	3.25	4.69
7.5	2.18	3.11	4.37
10	2.01	3.06	4.12

5.6.7.5 Tortuosity factor τ_a

Mass transfer of different ions in soils is affected by ionic radius, ionic charge, presence of other ions etc. All these factors were incorporated into the determined coefficient of diffusion and should be manifested in the values of tortuosity factor (ratio of effective diffusion coefficient to the free solution (aqueous) diffusion coefficient of the solute D^*/D_o) as presented in Table 5.11. Tortuosity factor accounts for the increased distance of transport and the more tortous pathways experienced by solutes diffusing through soil.

Table 5.11 Tortuosity Factor τ_a

Bentonite content (%)	τ_a		
	Pb	Na	K
0	.415	.298	.255
2.5	.320	.267	.25
5	.280	.244	.24
7.5	.230	.234	.223
10	.217	.23	.211

5.7 Reliability Evaluation

Lateritic soil bentonite mixtures containing up to 10% bentonite proposed as barriers in waste containment structures are being evaluated for their suitability using a reliability approach. The reliability estimates for these mixtures were generated from a regression model developed from laboratory based hydraulic conductivity data. The first-order reliability method (FORM) was employed for the reliability evaluation. Results of the estimates were used to identify and screen variables that give acceptable reliability levels for barrier materials and to identify the critical variables influencing hydraulic conductivity which may need strict monitoring and control during the construction stage.

5.7.1 Reliability analysis procedure

The statistical characteristics (i.e. mean, standard deviation and coefficient of variation) of the soil composition and compaction variables for the various lateritic soil bentonite mixtures are presented in Table 5.12. Hydraulic conductivity is traditionally assumed to be lognormally distributed (Harrop – Williams, 1985; Bogardi *et al.*, 1989; Benson, 1993; Benson and Daniel, 1994a; Benson and Daniel, 1994b; Gui *et al*., 2000). The probability distribution for the

independent variables in the regression equation were established as follows: initial saturation and plasticity index are lognormally distributed, clay content is normally distributed while the compactive effort is a deterministic parameter.

The limit state function adopted was incorporated into a FORTRAN – based programme called FORM5 and was used for the estimation of the reliability indices.

5.7.2 Regression-based model

Regression analysis was conducted to develop a screening model from laboratory measured hydraulic conductivity. Stepwise regression was adopted in order to identify and determine the differential strength of each of the variables to be included in the model. The explanatory variables were for (1) soil composition – plasticity index (PI), Clay content (C) - defined as % < 2μm (2) for compaction conditions – initial saturation (S_i) which defines the average wetness at compaction, {or moulding water content (w), and dry unit weight (γ_d)}, compactive effort E represented by an index (an integer categorical variable describing compactive effort with values assigned as follows: -1, 0 and 1 for BSH, WAS and BSL compactive efforts). Those variables having statistically significant effect on the hydraulic conductivity as indicated in the strength of relationship using F statistic were incorporated in the model.

Table 5.12 Statistical Characteristics of lateritic soil – bentonite mixtures used for the study.

No.	Soil Property (1)	Mean (2)	S.D. (3)	COV (4)
1.	Liquid limit (%)	50.06	5.07	10.13
2.	Plastic limit (%)	21.75	2.86	13.15
3.	Plasticity index (%)	28.31	3.82	13.48
4.	Clay content (%)	27.87	2.33	8.36
5.	Fines content (%)	59.16	1.72	2.91
6.	Sand content (%)	37.48	13.66	36.45
7.	Silt content (%)	21.37	24.92	116.6
8.	Gravel content	2.46	3.62	147.2
9.	Activity	0.98	0.36	36.46
10.	Specific gravity	2.54	0.38	14.96
11.	Optimum water content (%)	16.55	5.4	32.63
12.	Maximum dry unit wt. (kN/m^2)	17.2	3.56	20.69
13.	Initial saturation (%)	80.99	17.46	21.56

S.D: Standard deviation; COV: Coeffiient of variation

A hypothetical statistical model to represent the relationship between the variables takes the general form:

$$\ln k = b_o + b_1 X_1 + b_2 X_2 + ... + b_n X_n. \qquad (5.11)$$

In eq. (5.11), b_i, i = 0, 1, …..,n = coefficient determined by regression analysis, X_i, i = 1, 2, …,n value of the descriptor variable. For the model, α was specified as 0.05. At this significance level, the option of entering those variables with a minimum F- statistic of 4 was adopted (Benson *et al*., 1994)

The resulting regression model that best relates the hydraulic conductivity k to the variables on the basis of above information is:

$$\ln k = -37.55 - 0.54 S_i - 3.9 PI + 3.45 C + 0.55 E + \varepsilon \qquad (5.12)$$

For this model,

Coefficient of determination, R^2 = 0.8545 ; Adjusted R^2 = 0.844 ;

Standard error of estimate = 0.759531 and ε is a Gaussian random error term.

The model (see eq. (5.12)) describes the relationship between ln k and the variables namely the compositional factors and the compaction conditions. In the deletion process, only four variables (Initial saturation (S_i), Plasticity Index, Clay content (% < 2μm) and compactive effort) turned out to be statistically significant at the 0.05 level. Together, these four variables explained 85.14% (R^2 = 0.8514) of the total variance in hydraulic conductivity, although initial saturation alone was a better predictor. From the model (see eq. (5.12)) initial degree of saturation, plasticity index, clay content and compactive effort are the critical variables that need to be carefully monitored and controlled if desired values of k are to be achieved. This model is used limit state function in the reliability evaluation of the proposed barrier system.

5.7.3 Reliability estimates and discussion of results

The input data for the reliability evaluation using FORM5 (Gollwitzer *et al.*, 1988) are presented in Table 5.13. The numbers in bracket in column three of the Table are the various code numbers for the statistical distributions of the variables as used in FORM5 (Gollwitzer *et al.*, 1988). The maximum hydraulic conductivity value of 1.0 x 10^{-9} m/s allowed by regulatory agencies for soil liners was set as the upper limit. The corresponding standard deviation is 8.19 x 10^{-9} m/s.

In order to rate the sensitivity of the variables to load uncertainties, reliability estimates were made for hydraulic conductivity and the input variables were made to assume values of coefficient of variation between 10% and 100% (i.e., 0.1 and 1.0). The COV of hydraulic conductivity were first varied while the values for input variables were kept constant for normally and lognormally distributed hydraulic conductivity. Reliability estimates for BSL, WAS and BSH were obtained using the model generated from laboratory data.

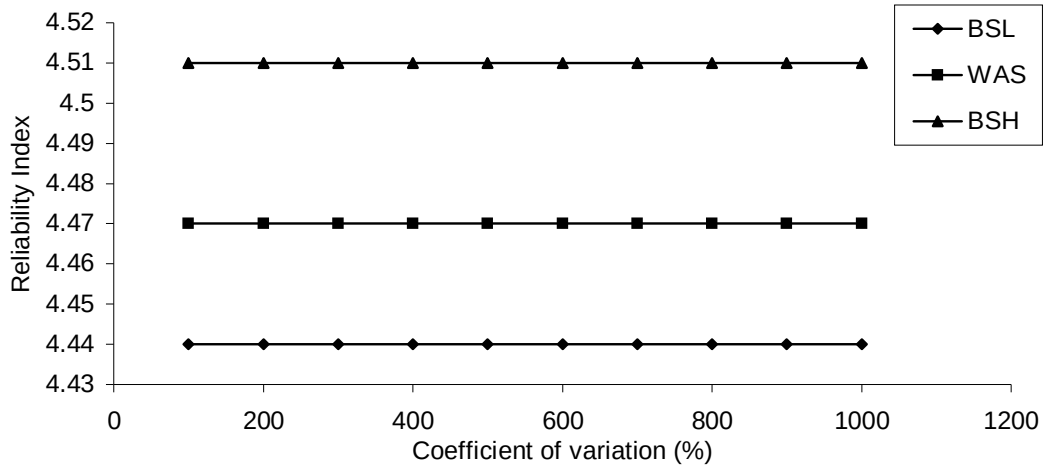
Table 5.13 Initial input parameters for reliability analysis of lateritic soil – bentonite mixtures for barrier applications

No.	Variables	Distribution type	Mean E(X)	Standard deviation S(X)	Coefficient of variation (%)
(1)	(2)	(3)	(4)	(5)	(6)
1.	Max. hydraulic conductivity, K_o	Lognormal (=3)	1×10^{-9} (m/s)	8.19×10^{-9} (m/s)	819
2.	Initial saturation, Si	Lognormal (=3)	80.99%	17.46	21.6
3.	Plasticity index, PI	Lognormal (=3)	28.3%	3.82	13.48
4.	Clay content, C	Normal (=2)	27.87%	2.33	8.3
5.	Compactive effort index, E	Deterministic parameter	(-1, 0, +1)	-	-

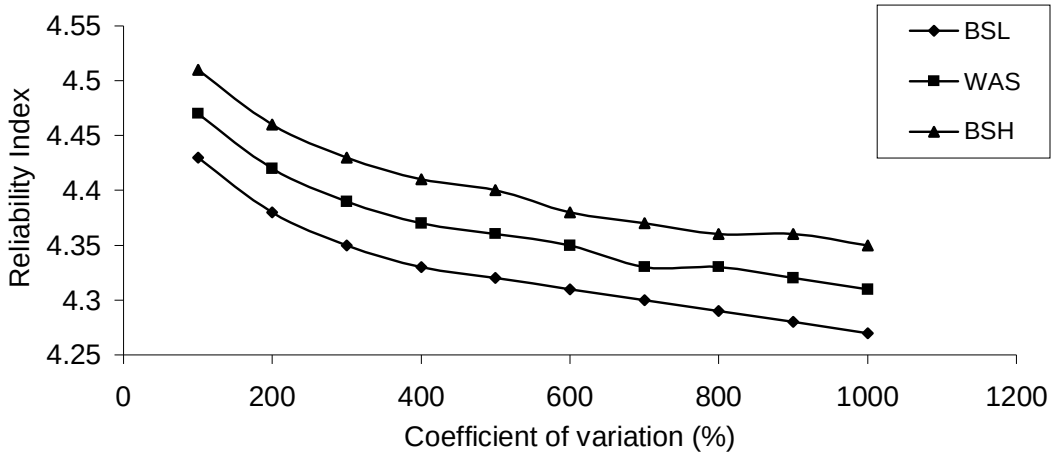
5.7.4 Reliability levels for variations in hydraulic conductivity

Reliability indices obtained using the model generated from laboratory data for predicting soil liner hydraulic conductivity are presented in Fig. 5.116(a) for normally distributed hydraulic conductivity, k and Fig. 5.116(b) for lognormally distributed hydraulic conductivity, k. For normally distributed function, no distinctive relationship was found to exist between the reliability estimates and the coefficient of variation, hence, variation in the hydraulic conductivity is not critical in the model. For lognormally distributed hydraulic conductivity, k, a non-linear decreasing trend was observed. In this case, though it perturbs the model, the decrease in the reliability indices is slight ranging from 4.51 to 4.35 for British Standard Heavy (BSH), 4.47 to 4.31 for West African Standard (WAS) and 4.43 to 4.27 for British Standard Light (BSL). Thus, variaton in the hydraulic conductivity is significant but not the most critical factor influencing the model. The effect of different compactive efforts is seen in Fig. 5.116(a) and (b) with higher reliability index being obtained at higher compactive efforts. This

corresponds to low k of soil specimens associated with higher compactive efforts in laboratory tests. Reliability levels obtained using this model are generally greater than 4.0.



(a)



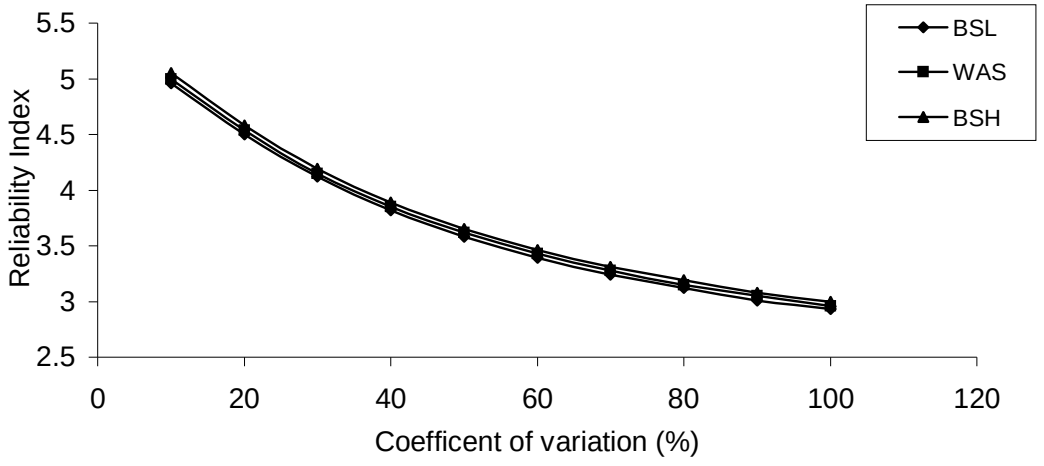
(b)

Fig. 5.116 Reliability Index versus coefficient of variation for hydraulic conductivity, k
(a) Normally Distributed k (b) lognormally distributed k

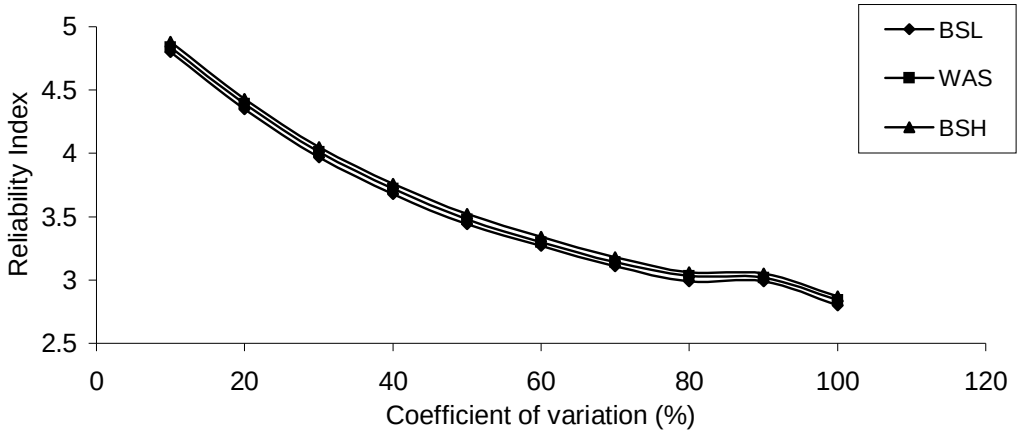
5.7.5 Reliability levels for variation in initial saturation

Variation in reliability levels of the predicted hydraulic conductivity as the coefficient of variation increased from 10% to 100% for the initial saturation is shown in Fig. 5.117(a) and (b). Estimates of reliability index decreased significantly both when k assumes normal and lognormal distribution functions. The pattern of trends in the variations of reliability levels

shown in Fig. 5.117(a) for normally distributed hydraulic conductivity are similar to those shown in Fig. 5.117(b) when k is lognormally distributed. The significant change in reliability levels for variability in initial saturation, Si, indicates that it is a major predictor of hydraulic conductivity, k in the model (Osinubi and Amadi, 2006) and therefore the reliability index is expected to be more sensitive to variations in the initial saturation. The reliability estimates ranged from 4.88 to 2.87 for BSH, 4.84 to 2.84 for WAS and 4.8 to 2.8 for BSL when lognormally distributed k was considered and 5.05 to 3 for BSH, 5 to 2.96 for WAS and 4.96 to 2.93 for BSL when k assumes normal distribution. However, there are no significant differences between the reliability levels for all the compactive efforts, i.e., the differences between the reliability indices are very marginal.



a



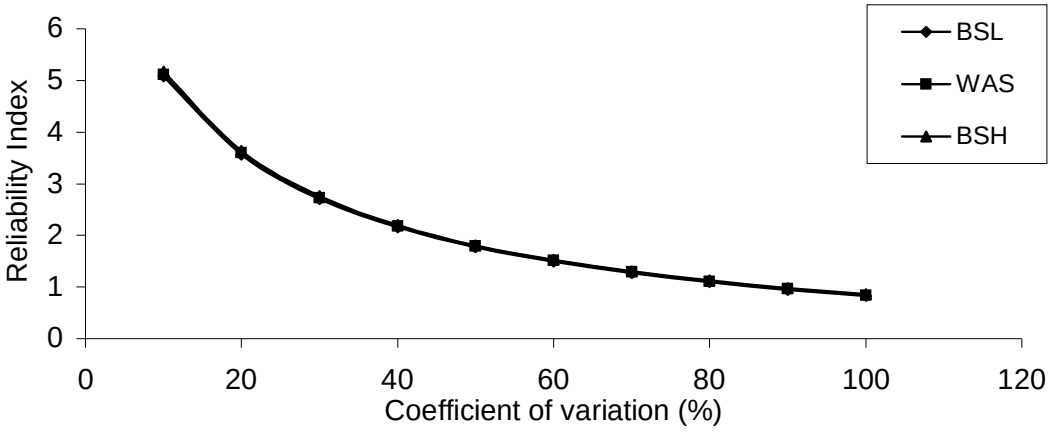
b

Fig. 5.117(a) and (b) Reliability Index versus coefficient of variation for initial saturation (a) Normally Distributed k (b) lognormally distributed k

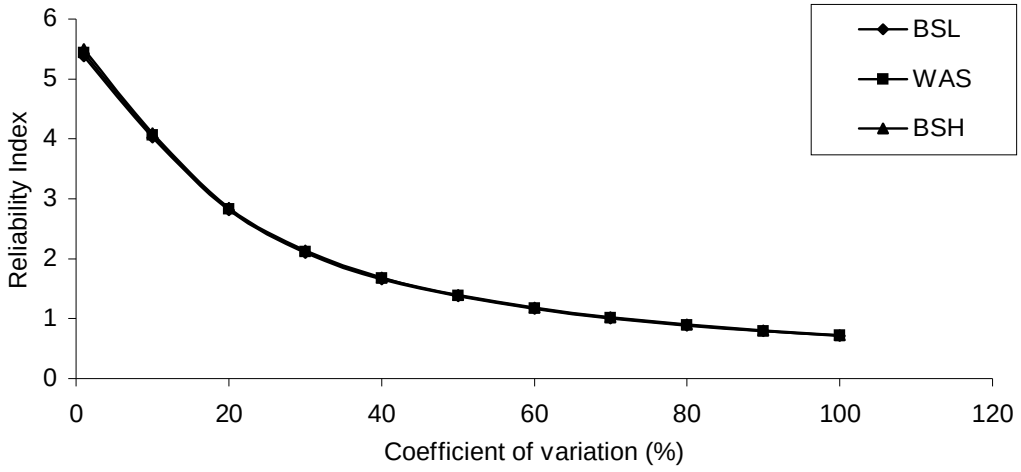
5.7.6 Reliability levels for variation in plasticity index

The decrease in reliability levels of the predicted hydraulic conductivity as the coefficient of variation of the plasticity index increased is shown in Figs. 5.118(a) and (b). It can be seen from the graphical plots that the reliability indices decreased in an exponential manner as the coefficient of variation for plasticity index increased. Reliability estimates when k assumes normal distribution ranged from 5.17 to 0.86 for BSH, 5.12 to 0.84 for WAS and 5.08 to 0.83 for BSL while reliability indices obtained when k is lognormally distributed decreased from 5.05 to 3.0 for BSH, 5.0 to 2.96 for WAS and 4.96 to 2.93 for BSL as the coefficient of variation increased from 10 to 100%. This shows the sensitivity of hydraulic conductivity to plasticity index and this is expected because the plasticity index generally exerts considerable influence on the hydraulic conductivity of clay soils used as hydraulic barriers and therefore has significant influence on the model (Mesri and Olson, 1971). It is also observed that the reliability estimates when the distribution type for k is taken as normally distributed are marginally higher than when it is lognormally distributed. From 10 to 80% coefficient of variation for the plasticity index, the reliability indices have values greater than 1.0. Above 80% coefficient of variation, reliability values less than 1.0 were obtained and decreased to 0.829 and 0.822 for normally and lognormal distributions of k respectively as the coefficient of variation approached 100%. Such reliability levels suggests that the value of plasticity index fall below the range normally prescribed for clay liner construction by most regulatory agencies (< 7 - 10%). An upper limit of 30% is usually prescribed for workability of the material in the field (EPA, 1989; Rowe *et al*., 1995; Das, 1998).

For the various compactive efforts, reliability indices are essentially the same and coincident which probably is an indication that compactive effort has no influence on plasticity index of soil materials.



(a)



(b)

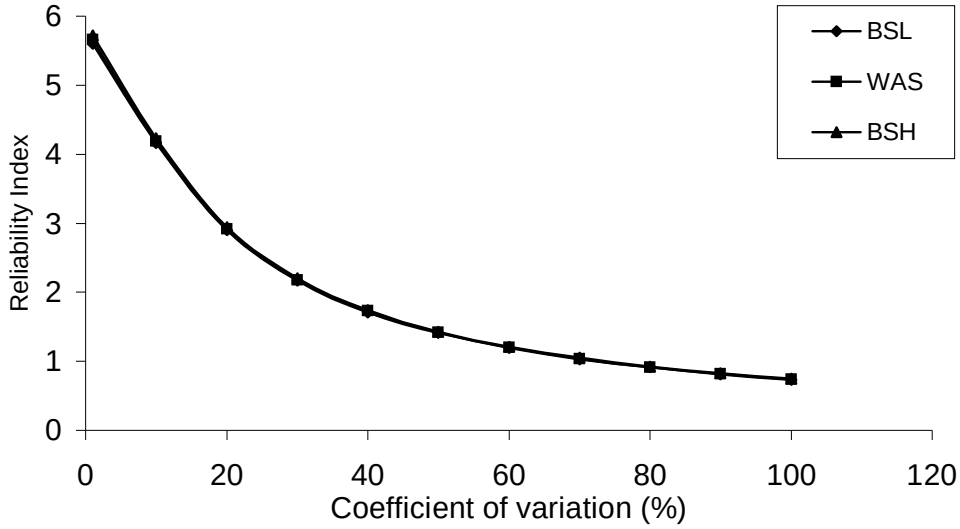
Fig. 5.118 Fig. Reliability Index versus coefficient of variation for plasticity index
(a) Normally Distributed k (b) lognormally distributed k

5.7.7 Reliability levels for variation in clay content

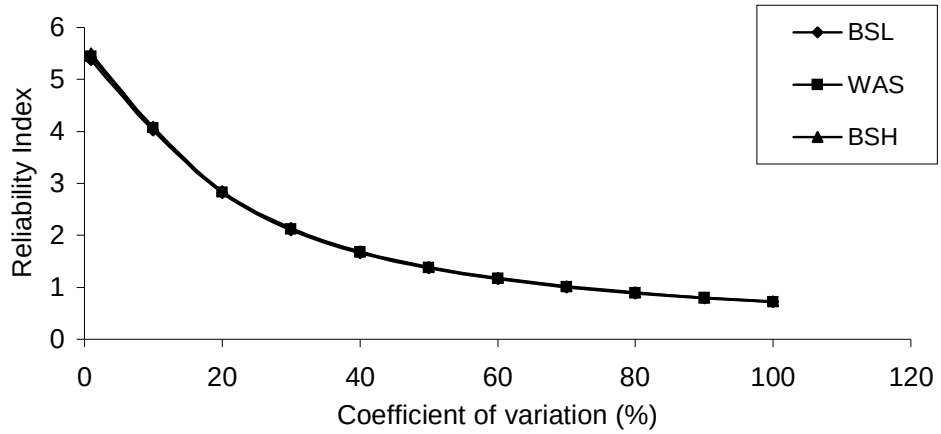
Reliability estimates for the hydraulic conductivity at varying coefficient of variation of clay content presented in Fig. 5.119(a) and (b) when k is normally and lognormally distributed respectively. As expected, the pattern of variation in reliability indices demonstrates a significant decrease as the coefficient of variation increased from 10 to 100%. This is because hydraulic conductivity of clayey materials is sensitive to clay (bentonite) content, achieving low hydraulic conductivity as clay-size particles filled voids between soil particles. Reliability indices obtained when the distribution of k is considered to be normally distributed decreased

from 5. 27 to 0.75 for BSH, 5.66 to .74 for WAS and 5.6 to 0.73 for BSL while in the case of lognormal distribution, reliability indices decreased from 5. 5 to .73 for BSH, 5.44 to .72 for WAS and 5.38 to .71 for BSL as the coefficient of variation increased from 10 to 100%. Visual examination of the graphical plots prove that there is no remarkable difference in the variations of reliability levels between Fig. 5.119(a) for normally distributed k and Fig. 5.119(b) when k taken to be lognormally distributed, however, reliability estimates for normally distributed k are generally higher than the corresponding reliability estimates of lognormally distributed k.

As in the variation of PI, reliability indices for the various compactive efforts are coincident which confirms that no correlation exist between clay content and compactive efforts.



(a)



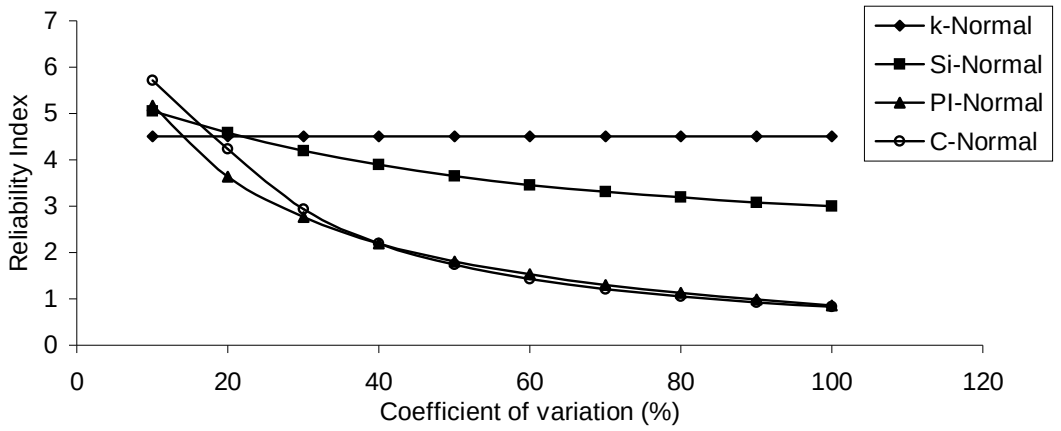
(b)

Fig. 5.119 Reliability Index versus coefficient of variation for clay content
(a) Normally distributed k (b) lognormally distributed k

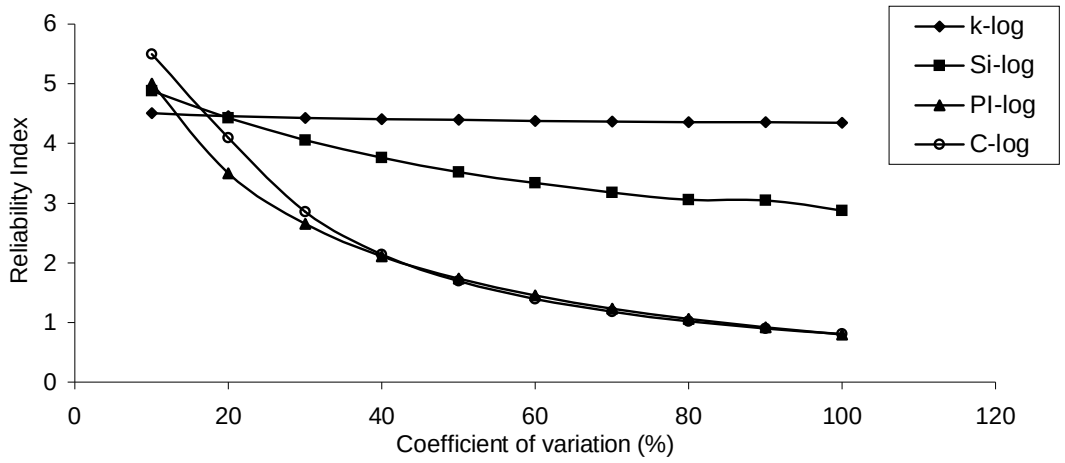
5.7.8 Comparing the sensitivity of reliability indices to coefficient of variation for the system and variables

The influence of the independent variables namely initial saturation, plasticity index and clay content on the hydraulic conductivity model was examined using reliability indices obtained for BSH compactive effort at coefficient of variation of the hydraulic conductivity in the range of 10 to 100%. The variation of the reliability indices as the coefficient of variation of the variables increased is shown in Fig. 5.120(a) for normally distributed hydraulic conductivity function and Fig. 5.120(b) for lognormally distributed hydraulic conductivity function. In the two cases, the reliability levels for the three variables i.e. initial saturation, plasticity index, clay content decreased as the coefficient of variation increased from 10 to 100%. The decrease in the value of reliability indices is observed to be more pronounced for plasticity index and clay content.

This can be attributed to the sensitivity of hydraulic conductivity to plasticity index and clay content as well as the strong relationship that exists between these two variables and hydraulic conductivity, k. It is therefore expected that these two variables will exert a measure of control in the output of the model for hydraulic conductivity when the source of uncertainty is basically from the two variables. Variations in these variables are mainly due to the active clay mineral, bentonite. Although initial saturation is strongly correlated with hydraulic conductivity, the sensitivity of the reliability level of the model to initial saturation is not as high as plasticity index and clay content. A practical conclusion from the foregoing is that the soil compositional factors represented by the plasticity index and clay content influence the reliability of the system more than the compaction conditions represented by initial saturation which is a combined measure of the moulding water content and dry unit weight.



(a)



(b)

Fig. 5.120 Reliability Index versus coefficient of variation for hydraulic conductivity, initial saturation, plasticity index and clay content (a) normal distribution type for k (b) lognormal distribution type for k

5.7.9 Statistical analysis

To further evaluate the reliability indices obtained when hydraulic conductivity was normally and lognormally distributed for variability in hydraulic conductivity, initial saturation, plasticity index and clay content, the F-test at 5% level of significance was employed.

The calculated F-value for the variables from the laboratory-based model are 3.9545 for hydraulic conductivity, 0.9365 for initial saturation, 0.95045 for plasticity index and 0.93049 for clay content. The corresponding critical F – value are 1.8608 for hydraulic conductivity and

0.537399 for the independent variables (initial saturation, plasticity index and clay content). Statistical significance was established when calculated F-value was greater than the critical F-value. It therefore implies that the differences in the reliability indices for the hydraulic conductivity in the range of variability considered for initial saturation, plasticity index, clay content when the hydraulic conductivity was assumed normally and lognormally distributed are statistically significant since the calculated F – value is greater than the corresponding critical F-value for 5% level of significance. From the results, it can be deduced that variability in the hydraulic conductivity as well as the independent variables have considerable influence on the reliability of lateritic soil – bentonite mixtures as containment liner.

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

Lateritic soils occur extensively in Nigeria and have potential use as barrier material for landfills because of their adequate geotechnical properties and the capacity to prevent breakthrough of hazardous contaminants including toxic metals when compacted. However, lateritic soil whose predominant clay mineral, kaolinite suffers from the problem of obtaining low hydraulic conductivity values. Incorporating up to 10% bentonite (by dry weight of soil), which is known to contain highly active clay minerals and widely used in containment application, will improve the hydraulic conductivity thereby optimizing the geotechnical properties of the two materials.

From the analysis of laboratory data obtained in the study, the following conclusion can be drawn.

- The geotechnical properties of the soil were significantly influenced by the introduction of bentonite. The effects of bentonite addition are reflected in an increase in plasticity index, optimum moisture content and a reduction in dry unit weight. Expectedly, the hydraulic conductivity and unconfined compression strength of compacted mixtures decreased non-linearly while the volumetric shrinkage increased with increase in bentonite content.
- For the various mixes investigated, the range of compaction water content and corresponding dry unit weight that will yield the best geotechnical performance with respect to hydraulic conductivity, unconfined compressive strength and volumetric shrinkage required for barriers in waste repositories were delineated in what is referred to as acceptable zone. Generally, the allowable range of water content was

broadened with the introduction of bentonite. This will significantly accelerate construction operations at construction sites, as compaction will not be constrained to narrow range of water content to achieve the established standards for the design parameters.

- The plasticity of the mixtures increased with higher bentonite content and therefore, volumetric water content, θ increased, as more water was available for removal.
- Higher air entry pressures (ψ) were obtained for mixtures with higher bentonite content when compacted wet of optimum water content or with higher compactive efforts.
- Variation in bentonite content resulted in changes in unsaturated hydraulic conductivity.
- Unsaturated hydraulic conductivity decreased slightly with increasing bentonite content irrespective of the matric suction as well as the relative hydraulic conductivity function used. However, the predicted unsaturated hydraulic conductivity based on VGM function decreased more at higher suctions (i.e., 100 and 1500kPa).
- The hydraulic conductivity of specimens permeated with MSW leachate solution increased by a factor of 1.2 to 1.7. Notwithstanding such increases, the hydraulic conductivity of mixtures did not exceed the regulatory value of 1.0×10^{-9} m/s for specimens that initially met this specification.
- The results of free swell test of specimens at 100 hours in deionised water as well as in MSW leachate solutions increased almost linearly as the bentonite content increased from 0 to 10%, while the free swell was found to have an inverse relationship with hydraulic conductivity of identically prepared specimens compacted at optimum water content.
- Various diffusion parameters namely, distribution/partition coefficients, retardation factors, effective diffusion coefficients and tortousity factors of three inorganic

contaminants in municipal solid waste leachates were investigated in this study. Higher retardation factors for Pb indicate its strong affinity for the negatively charged soil surface. Maxmium lead cation adsorption on the soil surface was also confirmed by its higher partition coefficient compared to other cations considered in this study.

- K cations were found to diffuse faster than the other two cations. The effective diffusion coefficients of the cations were found to be in the order of K > Na > Pb Diffusion also decreased as the bentonite content increased for all the cations.

- For the system, reliability indices from the laboratory based hydraulic conductivity model were consistently high. In the case of normally distributed hydraulic conductivity function, no distinctive relationship was found to exist between the reliability estimates and the variability in the hydraulic conductivity, while for lognormally distributed hydraulic conductivity function, a non-linear decreasing trend was observed.
- Variability in the initial saturation, plasticity index and clay content decreased the reliability indices with increasing coefficient of variation of the hydraulic conductivity when it was assumed either normally or lognormally distributed.

In conclusion, best conditions for optimal geotechnical, hydraulic and contaminant transport performance of the various lateritic soil bentonite mixtures as barrier material in waste repositories have been outlined. Clearly the set objectives in this research study including compliance with prescribed regulatory standards for the various parameters involved in the construction of barriers in waste containment facilities were accomplished.

6.2 Recommendations

The following recommendations are made on the basis of the findings of this study.

- Lateritic soil - bentonite mixtures can be used as barrier material in engineered landfills because it provides excellent geotechnical/geoenvironmental performance.
- Designers should specify ranges of compaction water content and dry unit weight within the overall acceptable zone.
- To ensure that mixtures are well constructed and perform as designed, it is important to implement effective quality control measures with emphasis on construction practices that will achieve placement variables as delineated in the acceptable zones.
- The research data, analysis and interpretations presented in this study can be extended to understanding and/or possibly adopting other soil blends with similar geotechnical characteristics.
- Further studies involving field tests such as the construction of test pads or trial sections are recommended. This will provide a more realistic assessment of the hydraulic and contaminant transport performance of the mixtures.

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