INFLUENCE OF FINES CONTENT ON WATER AND CHEMICAL FLOWS THROUGH COMPACTED LATERITIC SOIL LINERS

BY

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DECLARATION

This thesis is an original work carried out by the author under the supervision of Dr. K. J. Osinubi, and Dr. J. O. Afolayan. To the best of my knowledge, no part therefore has been submitted elsewhere for the award of a degree or diploma. The work of others is duly acknowledged and referenced.

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CERTIFICATION

This project titled: Influence of fines content on water and chemical flows through compacted lateritic soil liners, meets the regulation governing the award of Doctor of Philosophy in Civil Engineering and is an original work undertaken by Ibrahim Bajeh, Ph.D/ ENG/27221/2000-2001.

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ABSTRACT

Compacted clay liners are an integral component of the lining systems for covering or capping new or old landfills or waste impoundments. As the use of laterites that is widespread in the tropics, for waste contaminant has not been quite documented, this study was aimed at assessing the influence of fines content on water and chemical flows through lateritic liners. Seven laterites (samples A-G) from Zaria (Lat. $11^{o}\ 51^{1}$N and Long. $7^{o}\ 45^{1}$E) environs were subjected to various laboratory tests such as index properties, differential thermal analysis, compaction, hydraulic conductivity, volumetric shrinkage, unconfined compressive strength, suction test for soil water characteristics curves (SWCC), batch equilibrium, and diffusion tests using three compactive efforts (i.e. British Standard Heavy (BSH), West African Standard (WAS), and British Standard Light (BSL)).

From their index properties, Laterite A with the highest clay content, activity, and cation exchange capacity of 40%, 0.75, and 19.2meg/100g, respectively, is the most suitable liner material based on the specifications (Das, 1998) for municipal solid wastes liners. Some compositional variables (i.e. initial saturation, plasticity index, and clay content) obtained were further subjected to First Order Reliability analysis using a FORTRAN- based FORM programme as well as analysis of variance, while SWCC results were subjected to multiple regression analysis and analysis of variance.

Results obtained indicate that the hydraulic conductivity is sensitive to compaction conditions, initial saturation, plasticity index, liquid limit and fines content. Furthermore, the shape of the SWCC depends on compaction conditions, and plasticity product of the soil. Soils with higher plasticity index have greater air-entry suction. Changes in SWCC were also reflected in the mineralogical composition of the soil. Based on a minimum hydraulic conductivity $k \leq 1x10^{-7}$ cm/s, minimum shear strength of 200 KN/m^2, and a maximum volumetric shrinkage strain of 4%, acceptable zones for the seven laterites were delineated.

The results of the diffusion tests were analyzed using an analytical solution to Fick's second law. The transport parameters obtained, which included the effective diffusion coefficient, D* (ranges from $0.003x10^{-8}m^2/s$ to $0.880x10^{-8}m^2/s$ for Na^+; from $0.010x10^{-}m^2/s$ to $1.24x10^{-8}m^2/s$ for K^+; from $1.120x10^{-8}m^2/s$ to $2.010x10^{-8}m^2/s$ for Ca^{2+}), retardation factor (ranges from 2.481 to 3.653 for Na^+; from 3.589 to 3.998 for K^+; and from 5.821 to 7.011 for Ca^{2+}), and tortuosity factor for Cl^- (ranges from 0.241 to 0.328) are within the ranges of values found in literature; and are also sensitive to the plasticity product of the soils. The lateritic samples are compatible with the leachate. The breakthrough curves obtained show that the retardation of Na^+ and K^+ is a function of the soil compositional properties.

Generally the fines content has an effect on the water and chemical flows through the compacted lateritic soils in the following way; as the fines content increases from 53% (Laterite G) to 70% (Laterite A), the hydraulic conductivity decreases from 2.42E-06 to 3.14E-08 for BSL specimens, 1.56E-06 to 2.91E-08 for WAS specimens, and 1.10E-06 to 2.76E-08 for BSH specimens. This shows that lower hydraulic conductivities are associated with soils compacted with the highest effort, BSH, followed by WAS specimens, while BSL specimens have the highest conductivities. Also, from the results of the reliability analysis, the fines content as well as the initial saturation, and compactive effort have a great influence in controlling the hydraulic conductivity of the compacted lateritic soils. Generally, as the fines content decreases, the hydraulic conductivity increases. Furthermore, the results show that Laterite G, with the least fines content, has the least retardation factor for Na^+, K^+, and Ca^{2+}, and as the fines content increases to the highest in Laterite A, the retardation factor for the three cations increases, while their effective diffusion coefficient decreases from Laterite G to the lowest values in Laterite A.

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ABBREVIATIONS

AASHTO = American Association of State Highway and Transport Officials

ANOVA = Analysis of variance

ASTM = American Society for Testing and Materials

BC = Brooks- Corey

BSH = British Standard Heavy (compaction)

BSL = British Standard Light (compaction)

COV = Coefficient of variation

DNAPLs = Dense non- agueous phase liquids

DPTs = Direct – push technologies

EPA = Environmental Protection Agency

FORM (5) = First Order- Reliability – Method Version 5

FORTRAN = Formula Translation

GCL = Geosynthetic clay liner

Gs = Specific gravity

HDPE = High density polyethylene

LL = Liquid limit

LNAPL = Light non- aqueous phase liquid

MSW = Municipal solid waste

NAPLs = Non – aqueous phase liquids

Pl = Plasticity index

PP = Plasticity product

PRW = Permeable reactive wall

PVC = Polyvinyl chloride

PVF = Pore volume of flow

Si = Degree of initial saturation

SWCC = Soil water characteristic curve

UCS = Unconfined compressive strength

VG = van Genuchten

VOCs = Volatile organic compounds

WAS = West African Standard (compaction)

XRD = X-ray Diffraction

GLOSSARY OF TERMS

Acceptable zone: A zone or window on the compaction plane within which field compaction can be carried out to ensure acceptable hydraulic conductivity of liners and covers.

Adsorption isotherm: the relationship between the adsorbed concentration and the pore water concentration with respect to a given solute

Bioreactor landfills: a landfill that uses a control to enhance microbiological processes.

Clay – leachate compatibility: the condition of a compacted clay's hydraulic conductivity not been adversely affected when the soil is exposed to leachate or other fluids containing some chemical contaminants.

Compacted clay liners: compacted earthen materials used to line waste landfills. A liner has as its primary purpose the prevention of migration of fluids (mainly leachate) directly into the underlying subsurface during both the active disposal period, typically 10 to 20 years, as well as the inactive, or post-closure, period.

Compact clay covers (cover system): compacted earthen materials placed over solid wastes in the daily operation of a disposal facility. Its primary purpose is to minimize the amount of precipitation percolating through the waste.

Diffusion: the process by which matter is transported from one part of a system to another as a result of random molecular motions or a process in which solutes in a solution flow in response to a gradient in concentration; the solution itself need not flow for diffusive transport to occur.

Environmental Geotechnics: A discipline existing somewhere between geotechnical engineering, environmental engineering, hydrogeology, and soil physics (also known as Geoenvironmental Engineering)

Hazardous waste: waste containing dangerous substance that can cause some harm to human health.

Hydraulic barriers: Earthen or synthetic materials that can be used to impede the flow of water or other (contaiminated) fluids into a waste and to groundwater.

Hydraulic conductivity: also known as permeability, is a soil property which describes or expresses how water flows through soils.

Lateritic soils / laterites: All products of tropical weathering with red, reddish brown or dark brown colour with or without concretions and generally (but not exclusively) found below hardened ferruginous crusts or hard pan

Municipal solid wastes: The aggregated detritus of households with the exception of such bulky waste as old refrigerators, and car tyres.

Plasticity product: the plasticity index multiplied by the percentage passing 75mm.

Reliability index: the distance from the origin to a specific point of a limit- state surface, Lu, which is the number of standard deviations to walk from the mean and to the critical border in the direction of the point

Soil water characteristic curves: these are curves that define the volumetric water content as a function of the soil water pressure head (h) or matric suction (ψ)

Volumetric shrinkage strain: change in the volume of a compacted soil cylinder after drying expressed as a percentage of the original volume.

SYMBOLS

A = activity of clay

C_e= the equilibrium concentration of the adsorbate in the solution after adsorption

C_s = the adsorbed concentration (mass adsorbed/ mass of solid)

D* = effective diffusion coefficient

D_A = apparent diffusion coefficient

dh = change in height or pressure head

D_m = the mechanical dispersion coefficient

D_o = free solution diffusion coefficient

F = fines

G = gravel content

G_s = specific gravity

h = head

H = harmonic mean

H_l = height of leachate ponding over compacted soil during diffusion testing

I = hydraulic gradient

J = total mass flux of the solute or contaminant species

K_{max} = maximum hydraulic conductivity

K_p = partition coefficient or distribution coefficient

K_r = relative hydraulic conductivity

K_{sat} = saturated hydraulic conductivity

K_ψ = unsaturated hydraulic conductivity

L = length of the test specimen

LL = liquid limit

L = the macroscopic, straight line distance between two points defining the flow path

L_e = the actual microscopic or effective distance of transport between the same points

n = soil porosity (or total porosity)

P_f = probability of failure

Pl = plasticity index

PL = plastic limit

R_d = retardation factor

S = degree of saturation

S_i = degree of initial saturation

T = absolute temperature

V = volume of solution

V_s = seepage velocity

w = gravimetric water content

W = compactor weight

α = soil parameter in van Genuchten's model related to the air entry value or bubbling pressure or a constant in diffusion coefficient calculation

α_L = the longitudinal dispersivity of the porous medium in the direction of transport

β = reliability or safety index or empirically determined constant in the equation for hydrodynamic dispersion coefficient

γ = unit weight

Δh = change in height of water level in a standpipe

Δt = change in time with respect to water level in a standpipe head to decrease from h_1 to h_2 $(= t_1 - t_2)$

ε = the dielectric constant or an independent mean – zero Gaussian random – error term

$\varepsilon_{(\%)}$ = strain in percentage

η = concentration of the cation

θ = volumetric water content (cm^3/ cm^3)

Θ = normalized volumetric water content

θ_r = residual volumetric water content (cm^3/cm^3)

θ_s = saturated volumetric water content (cm^3/cm^3)

θ_w = volumetric water content (cm^3/cm^3)

λ = Brooks-Corey pore size distribution index

ρ = total or bulk density

ρ_d = dry density

σ = unconfined compressive strength or standard deviation

τ or τ_a = dimensionless tortuousity factor

ψ = matric suction

ψ_a = matric suction at the air entry

ψ_r = suction value that corresponds to the residual volumetric water content, θ_r.

CHAPTER ONE

INTRODUCTION

1.1 General Introduction

The primary environmental issue facing the whole world, in terms of subsurface environment, is the protection of good quality groundwater and the clean-up or remediation of already contaminated groundwater resources. Before 1976, US citizens had used open dumping of wastes which invariably resulted into soil contamination, groundwater pollution, and adverse health hazards, including cancer in children traceable to contamination by industrial chemicals. This led to public outcry and environmental regulations promulgated by both federal and state governments. These regulations provided the impetus for the development of environmental geotechnics in the US. Similar regulations were soon promulgated in other developed countries, resulting in worldwide growth of environmental geotechnics. Today, highly engineered waste containment facilities, site investigations to characterize the nature and extent of pollution, and remediation technologies to clean up soil and groundwater, are commonly used (Benson, 1999; Shackelford, 1990).

In a developing country like Nigeria where the practise of waste containment is non-existent, deadly health hazards from soil or groundwater pollution by harmful elements like mercury (Hg), potassium (K), sodium (Na), zinc (Zn), lead (Pb), etc are a reality especially when the country steps into proper development of her natural mineral resources. A conceptual model of a contaminated site is shown in Fig.1.1, in which plumes of migrating groundwater can be divided into three zones: (i) the core zone, located closest to the polluted source, is a region where the geochemical attenuation capacity of the natural soil has been so greatly depleted that the chemical composition of the groundwater at the core zone is the

same with that of the source, (ii) the active zone, is a region of intense geochemical interactions between soil materials and migrating polluted groundwater, and (iii) the downgradient neutralized zone containing conservative components which do not partake in geochemical attenuation reactions (Rouse and Pyrih, 1990).

These can include most of the major-ion components of the migrating seepage plume, but may include oxidized species of chromium or selenium. This model is of great advantage in the design, operation, and interpretation of a phased monitoring system using lysimeters (standpipes for collecting water) for groundwater sampling. The onset of the arrival of contaminants is characterized by drastic changes in the major-ion chemistry and detection of other conservative contaminants.

To impede the transport processes of these contaminants, compacted clay liners are normally used as an integral component of the lining systems to cover landfills, municipal and hazardous waste impoundments, and also to cap new or old waste disposal units (Daniel and Benson, 1990; Albrecht and Benson, 2001). Considering advection as a major transport process, the most significant factor affecting the performance of compacted clays is hydraulic conductivity. Most regulatory agencies in the United States require that the hydraulic conductivity of clay liners be less than or equal to $1x10^{-7}$cm/s. Successful design and construction of clay liners and covers involves many facets, such as selection of materials, assessment of chemical compatibility, bearing capacity, evaluation of subsidence, assessment of environmental factors like desiccation, and the development and execution of a construction quality assurance plan (Daniel, 1987).

Apart from hydraulic conductivity, diffusion is a very important transport mechanism especially if the seepage velocity is or approaches zero (Shackelford and Daniel, 1991). Therefore, considering both hydraulic conductivity and diffusion as major transport mechanisms, this study has highlighted the influence of fines content on water and chemical

flow through compacted lateritic clay liners. Laterites have been chosen because of their abundance in the tropics as well as lack of published work assessing its potential as a liner material. Also, soil water characteristics curve (SWCC) study for the evaluation of the unsaturated hydraulic properties, in addition to diffusion, batch equilibrium, and reliability study of the results of hydraulic conductivity tests, was carried out.

The specific objectives include: (1) to define ranges of fines content at which the compacted specimens will have low hydraulic conductivity ($k \leq 1x10^{-7}$cm/s), minimal potential for shrinkage upon drying, adequate shear strength, compatibility with leachates, (2) to measure the effective diffusion coefficients (D*) of ionic species diffusing in compacted specimens, retardation factor, tortuosity factor, and (3) to estimate the unsaturated hydraulic conductivities using the combination of SWCC data, Brooks-Corey's and van Genuchten's models.

1.2 **Role of Environmental Geotechnics in a Modern setting**

Environmental geotechnics can be divided into four groups namely, site characterization, polluted site remediation, waste containment and barrier systems, and recycled industrial by-products (Benson, 1999).

1.2.1 **Site characterization**

Contaminants travel along the pathway of least resistance, and narrow permeable channels usually govern their movement. Defining these transport pathways is of paramount importance, and is difficult using traditional methods of drilling and sampling because the volume explored is small, sampling is slow, and the cost of sampling is often prohibitive. In addition, traditional drilling and sampling normally brings contaminated cuttings to the

surface, which often need to be handled as hazardous wastes in the US. As a result, exploration methods that are more rapid, cheaper, and efficient are being developed.

Typical examples include direct-push technologies (DPT) which evolved from cone penetrometer testing (CPT) normally used for rapid and extensive definition of the stratigraphy of a site without bringing contaminated cuttings to the surface; penetrometers equipped with Wenner arrays for delineating free product; penetrometers fitted with scintillation gamma sensors for detecting radioactive contaminants; penetrometers equipped with laser-induced fluorescence technology capable of causing fluorescence of contaminants in soils and be delineated; and an array of advanced geophysical tools like ground penetrating radar (GPR) and tomographic imaging techniques to develop detailed cross-sections of water content in the vadose zone.

1.2.2 **Polluted site remediation**

The treatment of contaminated soil has many methods including (a) Pump and treat system, (b) Use of electrokinetics, (c) In-situ permeable reactive wall, and (d) Deep mixing of reagents and natural attenuation of soil.

(a) Pump and treat system: It is suitable for coarse-grained soils and requires pumping out contaminated water treatment plant, and then discharging it into a surface body or reinjecting it to the aquifer.

(b) Use of electrokinetics: This uses an applied electrical current passed through the soil via electrodes to remove primarily miscible contaminants from soil (Shackelford, 1994). The potential also exists for surfactant removal of immiscible contaminants from fine-grained soils using electrokinetics. The governing processes are mass fluxes due to diffusion, electromigration, electroosmotic advection, and electrophoresis. In addition, sorption, precipitation/dissolution, complexation, and a host of other chemical/biological reactions

may play a significant role. Therefore, several compositional and chemical factors, including soil mineralogy, initial pore fluid chemistry, and chemical characteristics, affect the success of electrokinetics.

(c) In-situ permeable reactive wall: This involves placing a reactive barrier on the pathway of groundwater flow. A reactive barrier incorporates chemical and/or biological reagents or catalysts into the porous medium used for the barrier to degrade or otherwise reduce the concentrations of harmful chemical species during passage of the contaminated groundwater through the barrier, such as a slurry cutoff wall (Shackelford, 1994). However, a major difference between the more traditional containment barriers and chemically reactive barriers is that the hydraulic conductivity of the reactive barrier must be sufficiently large to allow timely and efficient processing of the contaminated groundwater. The reaction that occurs as the pollutants pass through the reactive wall may be sorption combined with biodegradation or decay, reductive dehalogenation, precipitation, or any other conversion technique (Pankow *et al.*, 1993; Kershaw and Pamucku, 1997; Charlet *et al.*, 1998). The result is treated water emanating from the effluent end of the wall.

(d) Deep mixing of reagents and natural attenuation of soil: In deep mixing of reagent, a reactant is selected that results in rapid and near complete destructing of the contaminant without creating toxic products when injected into a contaminated site. The reactant is normally injected while stirring the contaminated soil with deep mixing augers (Benson, 1999). In natural attenuation method, the technology consists of no more than monitoring the natural breakdown of hydrocarbons. Here, assurance is needed that the contaminated plume remains within the site boundaries while the pollutants degrade naturally.

1.2.3 **Waste containment and barrier systems**

These include mainly (a) Engineered landfills, and (b) Bioreactor landfills.

(a) Engineered landfills: The use of engineered landfills or landfills lined with compacted soils (usually clay) and geosynthetics depend on the type of waste to put inside them. For this purpose, waste materials can be classified into four categories: (i) municipal waste, (ii) industrial waste, (iii) hazardous waste, and (iv) low-level ratioactive waste (Das, 1998). Moisture from meteoric water or snow normally reacts with the waste inside landfills to generate leachate which is responsible for contaminating groundwater, therefore; leachate must be carefully contained or removed constantly.

Typical examples of different liners in use in the US are shown in Fig.1.2 (a and b). In Fig.1.2 (a), the landfill is a single clay liner system with perforated pipes for the removal of leachate, while in Fig.1.2 (b) the system has a double clay liner, geomembrane, geotexile, geonet, as well as perforated pipes for leachate removal. The geosynthetics (including geotextiles, geomembranes, geogrids, geonets, and geocomposites) incorporated are fabriclike material made from polymers such as polyester, polyethylene, polyvinyl chloride (PVC), nylon, etc; and they perform one or more of the following functions: separation, reinforcement, filtration, drainage, and moisture barrier (Das, 1998). One other important component of a landfill is its cover. When a landfill is filled up, it must be covered with a cap as shown in Fig.1.3 (a), which will reduce and ultimately eliminate leachate generation.

A schematic diagram of the layering system for a landfill cap is shown in Fig.1.3 (b). It consists basically of a compacted clay cap over solid waste, a geomembrane liner, a drainage layer and a cover of topsoil. The manhole used for leachate collection penetrates the landfill cover. Leachate removal continues until its generation stops, which could be about 30 years in the case of hazardous waste landfill sites.

(b) Bioreactor landfills: In this case, the landfill is designed to act as a bioreactor where the waste is degraded into innocuous compost (Benson, 1999). Bioreactor landfills employ

leachate circulation and, in some cases, addition of water to facilitate biodegradation of the waste. However, the technology involved is still at its infancy.

1.2.4 **Use of recycled industrial by-products**

Engineering for sustainable development is relatively a new concept. Sustainable development requires that fewer wastes be generated, and the wastes generated be reused in other applications wherever possible (Benson, 1999). In this case, reusable wastes are no longer ordinary wastes but industrial by-products. However, several steps are required for successful beneficial reuse of industrial by products.

1.3 **Some important considerations in the construction of liners**

Though there are many factors to consider, a brief highlight of some fundamental ones can be listed as follows (Das, 1998):

(i) The clay liners are required to have low hydraulic conductivity ($k \leq 1x10^{-7}$cm/s).

(ii) The soil should have at least 20% fines (both silt and clay).

(iii) The plasticity index should be greater than 10. However, soils with plasticity index greater than 30 are difficult to work with in the field.

(iv) The soil should not include more than 10% gravel-size particles.

(v) The soil should not contain particles larger than about 25 to 50mm.

(vi) At construction sites where the local soil is nonplastic, imported clay minerals like sodium bentonite can be added and used as an admixture to achieve the required low conductivity.

(vii) A soil will have a lower hydraulic conductivity when it is compacted on the wet side of the optimum (1-3%).

(viii) During compaction, the clods must be broken down mechanically to small sizes.

(ix) Bonding between lifts can substantially reduce the overall hydraulic conductivity of a compacted clay, hence; should be carefully done to eliminate flow.

In addition, liners are expected to be compatible with leachates, therefore; a careful selection of materials is important. Also, the stability of side slope liner, and leak response action plan are equally necessary in consideration.

1.4 **Aim of the study**

Since the use of compacted lateritic soils for the purpose of constructing liners and covers has not been widely documented by researchers, there is need for such a study to be carried out, especially to evaluate the effect of fines content and laterite–leachate compatibility. Therefore, this study was aimed at evaluating the effect of fines content on the critical design parameters governing the engineering behaviour of laterites with respect to both the hydraulic conductivity and diffusion as contaminants transport mechanisms. And by so doing to define ranges of fines content at which the compacted soil specimens would have the following:

(i) Low hydraulic conductivity ($k \leq 1x10^{-7}$cm/s), (ii) minimal potential for shrinkage upon drying, (iii) adequate shear strength, and (iv) laterite-leachate compatibility. In addition, it was hoped to measure the diffusive transport parameters such as the effective diffusion coefficient (D*), partition coefficients, retardation factors, and tortuosity factors, all of which are important in the transit time design of lateritic soil liners or earthen barriers for waste containment. Furthermore, the soil water characteristics curves (SWCC) to evaluate the unsaturated hydraulic properties of the lateritic specimens were studied.

1.5 **Study approach**

Disturbed naturally occurring laterites were collected from seven sites (one at Basawa village, three at Ahmadu Bello University farm, and the last three at Shika area) around Zaria environs. The index properties of these soils and the physical and chemical properties were determined using laboratory procedures outlined in BS 1377 (1990) and Head (1992). The bulk specimens used in the particle-size distribution analysis were first soaked in water for at least three days prior to sieving. Specimens were compacted using three efforts (British Standard Heavy, West African Standard, and British Standard Light) to establish the moisture-density relationships.

Using the three compactive efforts, hydraulic conductivity tests were carried out on five sets of specimens compacted at 0%, $\pm$2%, and $\pm$4% of the optimum moisture content (OMC). Next, a selection of the molding water content (about 2%OMC) corresponding to the lowest hydraulic conductivity was made, and specimens for diffusion tests were compacted at this molding water content. Other laboratory tests used included volumetric shrinkage, and unconfined compressive strength in order to define ranges of fines content at which the compacted specimens will have low hydraulic conductivity ($k \leq 1x\ 10^{-7}$ cm/s), minimal potential for shrinkage upon drying, and adequate shear strength. Furthermore, batch equilibrium, long-term diffusion tests and soil water characteristics curve (SWCC) study were carried out. Some of the results of the index tests and hydraulic conductivities were subjected to reliability analysis, while the SWCC results were subjected to various statistical analysis such as multiple regression analysis, T-test, and analysis of variance (ANOVA).

CHAPTER TWO

BASIC AND ENGINEERING SITE CONCEPTS

2.1 **Overview of Barrier Systems**

The impact of a waste disposal facility on groundwater quality depends on the nature of the site, the climate, the type of waste, the local hydrogeology and the presence of a dominant flow path, and, perhaps most importantly on the nature of the barrier which is intended to limit and control contaminant migration (Rowe *et al*., 1996). Modern barriers may include one or more of the following components; (1) natural clayey soils such as lacustrine clay or clayey till, (2) recompacted clayey liners, (3) cut-off walls, (4) natural bedrock, and (5) geomembranes in composite liner systems. Some design considerations for each of these barrier systems are as follows:

2.1.1 **Natural clayey deposits**

These can be divided into four sub-groups; (a) diffusion–controlled systems, (b) advective–diffusive transport, (c) fractured clays, and (d) hydraulic traps.

(a) Diffusion–controlled systems: The simplest barrier systems involve thick natural deposits of clayey soil where the water table is near the surface. For the unusual case of a landfill which does not mound above the original ground surface, it may be possible to design the landfill such that, after closure, the leachate levels in the landfill rise to the natural water table as shown in Fig. 2.1, and there is no significant gradient or advective transport. The primary consideration in the evaluation of this type of barrier design is to ensure that the hydrogeologic environment is such that one can reasonably expect low hydraulic gradients over the long term (Rowe *et al.*, 1996).

Under these conditions, the movement of contaminants will be slow and controlled mainly by diffusion. The clay itself can also act as an important medium for the attenuation of some contaminants due to processes such as sorption, precipitation, biodegradation, adsorption, co-precipitation, and volatization (Rouse and Pyrih, 1990; Dragun, 1988). Nevertheless, migration will occur, hence, it is important to check that the impact of the contaminant at the site boundary, owing to molecular diffusion from the landfill, meets the regulatory requirements.

(b) Advective-diffusive transport: This is a more realistic situation concerning a mounded waste than the earlier type (a) Prior to landfilling, there are downward gradients from the water table to the underlying aquifer as shown in Fig. 2.2. The clayey soil provides a natural barrier, however, in order to design a suitable landfill, careful considerations must be given to the selection of the landfill base elevations and to the design of the leachate collection system. In Fig. 2.2, the base elevations are below the groundwater level but above the potentiometric surface (water surface) in the aquifer, hence, there will be a downward advective flow from the landfill to the aquifer. However, in Fig. 2.3 where the base elevations are such that the design leachate mounds is below the potentiometric surface in the aquifer, there is upward advective flow from the aquifer to the landfill. This design relies on hydraulic containment of contaminants and may be referred to as a hydraulic trap (Rowe *et al.*, 1996).

(c) Fractured clays: A major consideration for a landfill design such as that shown in Fig. 2.2 is whether the clayey soil or till is fractured. Herzog *et al*. (1989) and D'Astous *et al*. (1989) reported that clay till beneath obviously weathered and fractured zone may be fractured to depths of 6-15m. It is important to recognize that fractures with very thin openings can have a significant effect in terms of increasing the bulk hydraulic conductivity. The presence of fractures will likely control the hydraulic conductivity of the clay, and hence, the downward

advective velocity, therefore, it may be necessary to construct a clay liner by reworking the existing soil to reduce the outward advective transport from the landfill to acceptable levels.

(d) Hydraulic traps: Since the inward advective flow of groundwater from the aquifer tends to inhibit the outward diffusion of contaminant in this case (see Fig. 2.3), however, there may be untold consequences on the underlying aquifer. Therefore, appropriate evaluation is compulsory in hydraulic traps. In reality, it is difficult to handle since, for example, lowering the landfill base elevations increases the inward gradient and hence inward advective flow (Rowe *et al.*, 1996) which favourably reduces the outward diffusion of contaminants, but with a negative impact as the thickness of the barrier must be reduced.

The lower the base of a barrier the greater the potential for cracking and total blowout of the base since the uplift pressures are greater. However, blowout may be minimized by depressuring the aquifer during construction. In addition, the design of a hydraulic trap implies that water will be removed constantly from the aquifer by the landfill, resulting in the reduction of the water pressure in the aquifer which must be carefully balanced. Also, pumping of aquifer for water supply and climatic changes can have adverse effect on a hydraulic trap, therefore, necessitating a more elaborate design.

2.1.2 **Compacted clay liners**

With good engineering practice and quality control, good quality, low hydraulic conductivity liners can be constructed from compacted inactive clays by considering the clay-leachate compatility though the threat of concentrated organic wastes increasing the hydraulic conductivity of clays is real (Rowe *et al.*, 1996). Inactive clayey liners which are compacted at water content higher than the standard Proctor optimum moisture content and not allowed subsequently to dry out, will be nearly saturated, compressible on loading and should behave in a satisfactory manner similar to natural unfractured clayey barriers.

Provided that the design minimizes outward gradients, the primary transport mechanism through a well-designed compacted clayey barrier will be diffusion.

Two main types of clay liners under this section are: (a) single liner, and (b) liner with secondary leachate collection/hydraulic control system (Rowe *et al.*, 1996).

(a) Single liner: This type of liner, shown in Fig. 2.4, is a recompacted clayey barrier constructed on natural soils for two purposes. One, if the natural soil is a fractured clayey soil, then the liner may be required to retard movement of contaminants along the fractures. For situations where there is outward flow from the landfill (e.g water level (a) in Fig. 2.4), the assessment of potential impact is likely to consider advective-diffusive migration through the liner and the potential for attenuation due to matrix diffusion as the contaminants migrate along the fractures.

For situations where there is a hydraulic trap (e.g water level (b) in Fig. 2.4), the assessment of potential impact will require consideration of outward diffusion. The effect of the inward advective flow needs to be considered carefully depending on whether the fractures are closely-spaced or not. The second reason for constructing a clay liner is that though the subsurface may not be fractured, the natural soil does not have a low enough hydraulic conductivity to provide an adequate barrier (Rowe *et al*., 1996).

(b) Liner with secondary leachate collection/hydraulic control system: In this case, an additional secondary leachate collection system is needed to provide sufficient confidence that there will be a negligible effect on groundwater pollution. A typical example is shown in Fig. 2.5 where the potentiometric surface in the underlying aquifer is at or below the base of the landfill. This design serves two purposes.

Firstly, the drainage layer functions as a secondary leachate collection system capable of removing some of the leachates which pass through the liner, and secondly, it reduces the hydraulic gradient through the underlying soil for aquifer water levels below the landfill base

(case (a) in Fig. 2.5) or to induce a hydraulic trap to the secondary system when the potentiometric surface in the aquifer is above the level of the secondary leachate collection system (water level (b) in Fig. 2.5). A more sophisticated design using a geomembrane layer (secondary composite liner) instead is shown in Fig. 2.6. This design can minimize the contaminant impact, volume of fluid needed to maintain the hydraulic trap, and can also be used as a secondary leachate collection system (Rowe *et al*., 1996). The design of all landfills should involve careful considerations such as the design life of the engineered components, failure of the primary leachate collection system, the effect of the landfill shadow on the long-term performance of the hydraulic trap and the potential for a blowout.

2.1.3 **Cut-off walls and permeable surrounds**

These are commonly used for limiting pollutant migration from existing sites which have not been adequately designed. In addition, they can also be used in controlling pollutant migration from new sites where it may be desirable to isolate the polluted groundwater in a relatively thin and shallow aquifer beneath the landfill. Where they are constructed, water flow is directed around the outside of the pit rather than through the pit, and contaminant migration will be predominantly by diffusion from the waste through a less permeable material, together with advective-dispersive transport within the more permeable outer zone (Rowe *et al*., 1996).

2.1.4 **Natural bedrock**

Unfractured bedrock normally has a very low hydraulic conductivity; therefore, contaminant migration will primarily involve advective-dispersive transport along existing fractures in the rock. In this case, the main mechanism limiting the movement of

contaminants is the process of matrix diffusion, whereby contaminants are removed from the fracture as they diffuse into the matrix of the rock.

2.1.5 **Geomembranes in composite liner systems**

Where local natural recompacted clays fail to provide a safe primary barrier, geomembranes may provide a cost effective means of gaining the environmental protection required when used as part of a primary [or secondary] composite liner. A schematic detail of a 1-2mm thick geomembrane used as a primary liner in a liner system is shown in Fig. 2.7. The installation of a geomembrane above a clay liner would be expected to reduce considerably the outward movement of contaminant by advective processes; however, some migration would still be expected due to leakage through the geomembrane, and diffusion.

The upward flow of groundwater (if present) can have an adverse effect on a geomembrane as the uplift pressure can damage the geomembrane. Therefore, it may be necessary to install a granular groundwater control layer beneath a liner, or to depressurize the underlying aquifer during construction to prevent the problems due to uplift.

A leak detection system, which may be a granular layer and or a geosynthetic drainage layer in some cases, is sometimes incorporated to detect and collect most of the leachate which does escape through the primary membrane (Rowe *et al.*, 1996). A clay liner is often used as a backup in this leakage control system. Furthermore, for composite liner systems, care must be taken to prevent desiccation of the underlying clay liner due to high temperatures (Collins, 1993).

2.2 **Use of Landfill Cover**

The generation of mobile pollutants is often related to the movement of fluid or gas into the waste. For example, the volume of leachate generated in a landfill is directly related

to the movement of water through the cover. The leachate concentrations and the contaminating lifespan of a landfill can be related to the infilteration through the cover which may influence the performance of the barrier.

2.3 **Transport Mechanisms and Governing Equations**

When dealing with contaminant transport through saturated clayey barriers or the matrix of an intact rock, the primary transport mechanisms are usually advection and diffusion. But for aquifers, the transport mechanisms are usually advection and dispersion, while in fractured rocks the transport mechanisms are advection and dispersion along the fractures, and diffusion from the fractures into the matrix for the adjacent porous media (Rowe *et al*., 1996).

2.3.1 **Advective transport mechanism**

The mass flux, f, which is the mass of contaminant transported by advection per unit area per unit time, can be expressed as (Rowe *et al*., 1996):

$$f = nvc = V_a c \quad \text{..... (2.1)}$$

where n is the effective porosity of the soil; v is the groundwater velocity (seepage velocity); V_a is the Darcy velocity; c is the concentration of the contaminant at the point and time of interest. The mass of contaminant transported from the landfill by advection, m_a, will therefore be

$$m_a = A_0 \int_0^t nvc\,dt \quad \text{..... (2.2)}$$

where A_o is the cross-sectional area of the landfill through which contaminants are passing; t is a specific time.

It is important to note that the velocity at which contaminant moves by advection is the groundwater velocity, hence, if there is no flow there will be no movement of contaminant into the barrier by advection.

2.3.2 **Diffusive transport mechanism**

Diffusion involves the transport of contaminants from points of high chemical potential (concentration) to points of low chemical potential (concentration). The mass flux transported by only diffusion can be expressed as (Rowe *et al.*, 1996):

$$f = -nD_e\frac{dc}{dz} \qquad \text{.....(2.3)}$$

where n is the effective porosity; D_e is the effective diffusion coefficient; dc/dz is the concentration gradient, which is always negative because contaminants move from high to low concentrations.

The total mass of contaminant, m_d, transported out of a landfill by diffusion up to some specific time, t, will then be

$$m_d = A_0\int_0^t\left(-nD_e\frac{dc}{dz}\right)dt \qquad \text{.....(2.4)}$$

where all terms are as previously defined.

2.3.3 **Advective – diffusive transport mechanism**

When diffusion occurs in the same direction with advection, it quickens the transport period. However, diffusion can also work in the opposite direction with advection. For the case of advective–diffusive transport, the mass flux, f, can be written as (Rowe *et al.*, 1996):

$$f = nvc - nD_e\frac{dc}{dz} \qquad \text{.....(2.5)}$$

The total mass, m, transported from the landfill up to a specific time, t, is

given by

$$m = A_0 \int_0^t \left[nvc - nD_e \frac{dc}{dz} \right] dt \qquad \text{.....(2.6)}$$

where the velocity, v, is positive if it is out of the landfill and negative if it is in to the landfill while other terms remain the same.

2.3.4 **Dispersion mechanism**

In many aquifers where the velocity of flow is high, mechanical dispersion is a significant factor in contaminant migration. This mechanism involves mixing that occurs due to local variations in the flow velocity of the groundwater or due to nonhomogeneity in the aquifer (Freeze and Cherry, 1979). Though the mechanical dispersion is different from diffusion, both are often lumped together as a composite parameter, D, known as hydrodynamic dispersion:

$$D = D_e + D_{md} \qquad \text{.....(2.7)}$$

where D_e is the effective (molecular) diffusion coefficient for the contaminant species of interest; D_{md} is the coefficient of mechanical dispersion.

The mass flux is then given by

$$f = nvc - nD\frac{dc}{dz} \qquad \text{.....(2.8)}$$

where all the terms are as previously defined.

In transport of contaminant through clay, diffusion dominates the parameter D while dispersion is negligible (Gilham and Cherry, 1982). However, in aquifers, dispersion tends to dominate. It is often convenient to model the dispersive process as a liner function of velocity (Bear, 1979):

$$D_{md} = \alpha V \qquad \text{---------- (2.9a)}$$

where α is dispersivity.

The dispersivity, α, tends to be scale dependent. A plausible estimate of dispersivity is (Rowe *et al*., 1996):

$$\left.\begin{array}{ll} \alpha_L = \dfrac{x^4}{100} m & x \leq 10m \\ \alpha_L = 200m & x > 10m \end{array}\right\} \text{---------- (2.9b)}$$

where α_L is the longitudinal dispersivity in meters; x is the scale of the observation in meters. When considering a fractured medium, the effect of matrix diffusion and mechanical dispersion must be examined separately.

2.3.5 **Sorption mechanism**

This is a mechanism by which contaminants are removed from polluted solution or groundwater. Many processes involved include cation exchange whereby cations such as K^+, N_a^+, Pb^{2+}, Fe^{2+} etc replace other cations (e.g Ca^{2+}, Mg^{2+}) on clay surface, and hence are removed from solution, leading to an increase in the pore fluid concentration of the desorbed cations, e.g. Ca^{2+}, Mg^{2+} (Rowe *et al*., 1996). Other mechanisms include precipitation of heavy metals (Yanful *et al*., 1988) and removed of organic contaminants from solution by interaction with solid organic matter in the soil. Sorption can be modelled being linear and reversible, hence, the mass of contaminant removed from solution, S, is proportional to the concentration in solution, C (Rowe *et al*., 1996):

$$S = K_d C \qquad \text{---------- (2.10)}$$

where S is the mass of solute removed from solution per unit mass of solid; K_d is the partition or distribution coefficient; C is the equilibrium concentration of solute.

A plot of the variation in solid-phase concentration S versus the solution-phase concentration under equilibrium is termed an isopleth or isotherm. Eq. (2.10) represents a case of a linear isopleth, which is a reasonable approximation for low concentrations of contaminants. However, at high concentrations, sorption is nonlinear; curves are either concave or convex as the relationship become more complicated. Two out of the many methods for handling this complicated relationship are the Langmuir, and Freundlich isopleths (Rowe *et al*., 1996).

The Langmuir isopleth can be expressed as (Rowe *et al.*, 1996):

$$S = \frac{S_m bC}{1+bC} \quad \text{---------- (2.11)}$$

where S_m is the solid-phase concentration corresponding to all available sorption sites being occupied; b represents the rate of sorption, while the other parameters have already been defined. The parameters S_m and b are best obtained by performing batch equilibrium tests which entails a known volume of contaminant at a known concentration to be mixed with a known volume of dry soil.

The Freundlich isopleth is an empirically based alternative model which sometimes yields better results. It can be expressed as (Rowe *et al.*, 1996):

$$S = K_f C^{\varepsilon} \quad \text{---------- (2.12a)}$$

where K_f and ε are empirically determined constants.

The Freundlich parameters are best determined by performing batch equilibrium tests. Eq. (2.12a) can be rewritten as

$$\text{In } S = \text{In } K_f + \varepsilon \text{ Ln } C \quad \text{---------- (2.12b)}$$

A plot of In S versus In C readily allows the evaluation of K_f and ε.

2.3.6 **Radiactive and biological decay**

Some contaminants undergo radioactive or biological decay to form lighter elements or simpler compounds. For materials which undergo first order decay, the rate of reduction of concentration is proportional to the current concentration, and can be expressed as (Rowe *et al.*, 1996):

$$\frac{dc}{dt} = -\lambda C \quad \text{---------- (2.13a)}$$

where λ is the first order decay constant which has three components due to radioactive decay, biological decay, and fluid withdrawal respectively:

$$\lambda = \lceil R + \lceil B + \lceil S$$

where $\lceil R = \text{In } 2/(\text{radioactive half-life})$ is the radioactive decay constant; $\lceil B = \text{In } 2/(\text{biological decay half-life})$ is the biological decay constant; $\lceil S$ is the volume of fluid removed per unit volume of soil per unit time from beneath a landfill. The solution to this equation is

$$C(t) = C_o e^{-\lambda t} \quad \text{---------- (2.13b)}$$

where C_o is the initial concentration.

2.3.7 **Governing differential equations**

In the case of advective-diffusive-dispersive transport mechanism earlier treated (see eq. (2.8)), considering conservation of mass within any small region, the change in concentration with time can be expressed as (Rowe *et al.*, 1996):

$$n\frac{dc}{dt} = -\frac{df}{dz} - \rho\frac{ds}{dt} - n\lambda c \quad \text{---------- (2.14a)}$$

This means that the increase in contaminant concentration within a small volume is equal to the increase in mass due to advective-diffusive transport (-df/dz) where the negative sign implies a net increase in mass within the element, minus the decrease in mass due to sorption (-ds/dt) minus the decrease in mass due to first order decay process (-$n\lambda c$).

Substituting eq. (2.8) for the mass flux, f, and eq. (2.10) for the sorbed concentration S into eq. (2.14a) then results to:

$$n\frac{dc}{dz} = \{nD\frac{d^2c}{dz^2} - nv\frac{dc}{dz}\} - \rho K_d\frac{dc}{dz} - n\lambda c \quad \text{---------- (2.14b)}$$

where n is the effective porosity of the soil; c is the concentration at depth z and time t; v is the seepage velocity; ρ is the dry density of the soil; $v_a = nv$ is the Darcy or discharge velocity, and all other terms are as earlier defined.

Eq. (2.14b) can be rearranged as:

$$(n + \rho K_d)\frac{dc}{dt} = nD\frac{d^2c}{dz^2} - nv\frac{dc}{dz} - n\lambda c \quad \text{---------- (2.14c)}$$

Neglecting first order decay and dividing throughout by $(n + \rho K_d)$ this becomes

$$\frac{dc}{dt} = D^*\frac{d^2c}{dz^2} - V^*\frac{dc}{dz} \quad \text{---------- (2.14d)}$$

where $D^* = D/R$; $V^* = V/R$; $R = 1 + \rho K_d/n$ is the retardation coefficient.

Though eq. (2.14d) is mathematically correct, but can result to severe difficulties if the boundary conditions are flux-controlled. Hence, the use of parameters D* and V* is not encouraged. Furthermore, eq. (2.14b) describes advective-diffusive-dispersive transport mechanism under one-dimensional conditions. However, for three-dimensional conditions, it can be generalized as (Rowe *et al.*, 1996):

$$n\frac{dc}{dt} = nD_x\frac{d^2c}{dx^2} + nD_y\frac{d^2c}{dy^2} + nD_z\frac{d^2c}{dz^2} - nV_x\frac{dc}{dx} - nV_y\frac{dc}{dy} - nV_z\frac{dc}{dz} - \rho K_d\frac{dc}{dt} - n\lambda c \quad \text{---------- (2.15)}$$

where D_x, D_y, D_z are the coefficients of hydrodynamic dispersion in the x, y, and z Cartesian directions respectively; V_x, V_y, V_z are the components of groundwater velocity in the three Cartesian directions.

2.3.8 **Importance of different transport mechanisms**

Quite a number of laboratory tests have been performed to validate the advection – dispersion model of eqs. (2.7), (2.8), and (2.14). Results indicate that at relatively low velocities, the dispersion coefficient is equal to the effective diffusion coefficient, while at high velocities, the dispersion coefficient increases as a linear function of velocity (Rowe *et al.*, 1996). However, Perkins and Johnston (1963) published an empirical relationship which expresses the hydrodynamic dispersion, D, as

$$D = D_e + 1.75dv \ (m^2/a) \text{ ---------- (2.16)}$$

where d is the mean grain diameter of the soil (m). The mass of contaminant passing through the barrier and into the underlying aquifer will increase with increasing Darcy velocity V_a. The peak flux loading, f, on the aquifer may be estimated by performing the following two calculations (Rowe *et al.*, 1996):

$$f = - nD \frac{dc}{dz} = \frac{nDC_o}{H} \text{ ---------- (2.17a)}$$

(assuming $V_a << nD$)

$$f = nvC_o = V_a C_o \text{ ---------- (2.17b)}$$

(assuming $V_a >> nD$)

where H is the thickness of the liner; C_o is the constant leachate concentration. However, for cases where neither V_a nor D dominate, this approach can be misleading.

Generally, at velocities likely to be encountered with a well-constructed composite liner, diffusion is likely to dominate. For velocities often encountered with outward gradients and a good compacted clay liner (K about 10^{-10} m/s), diffusion and advection may both be of significance, but not mechanical dispersion. On the other hand, in sand and gravel aquifers, advection will dominate over diffusion, and dispersion may dominate over diffusion.

2.4 **Transport in Complicated Conditions**

2.4.1 **Unsaturated soils**

The advective-diffusive transport of contaminants through unsaturated soils is more complicated that for saturated soils. The partial differential equation governing one-dimensional movement can be written as (Rowe *et al.*, 1996):

$$\frac{d}{dt}(\theta C) = \frac{d}{dz}\left(\theta D \frac{dc}{dz}\right) - \frac{d}{dz}(V_a C) - \theta\lambda c \quad \text{---------- (2.18)}$$

where θ is the volumetric water content (equal to porosity for a saturated soil); λ is a term which accounts for sorption and biological, chemical and radioactive decay; while the other terms have previously been defined. It has been shown by Rowe and Badv (1994) that the effective diffusion coefficient in unsaturated soils may be substantially lower than in similar saturated soils. De Smedt (1981) has shown that a reasonable fit to his experimental data could be obtained using eq. (2.18) and an effective dispersion coefficient, D, given by

$$D = \frac{\theta_m}{\theta} D_m + \frac{\theta^2_{im} V^2}{\theta\theta_m\beta} \quad \text{---------- (2.19)}$$

where D_m is the dispersion coefficient in the mobile water;

θ_{im}, θ_m, θ are volumetric water content in the immobile and mobile phases, and the bulk soil respectively; V is the seepage velocity; β is the coefficient for solute transport between phases.

2.4.2 **Fractured porous media**

The transport of contaminants through fractured rock is principally by advection and dispersion (Rowe *et al.*, 1996). As contaminants move along fractures, there is always a

difference between the concentration of a given species in the fracture at a given point and the concentration in the pore water of the adjacent intact material. This concentration difference or gradient gives rise to diffusion of contaminants between the fluid in the fractured and the pore water in the matrix of the adjacent material (the matrix pore water). For example, if the concentration of the contaminant in the fractures is higher than in the matrix, then contaminant will diffuse into the matrix, thereby reducing the concentration in the fracture. This phenomenon is termed matrix diffusion, and can have a significant effect on the movement of contaminants through fractured media.

The one-dimensional movement of contaminants in a fractured system consisting of a set of parallel fractures can be expressed as (Rowe *et al*., 1996):

$$n_f \frac{dc_f}{dt} = n_f D \frac{d^2 C_f}{dz^2} - V_a \frac{dC_f}{dz} - \Delta K_f \frac{dc_f}{dt} - n_f \lambda C_f - q \quad \ldots\ldots\ldots\ldots\ldots\ldots (2.20)$$

where n_f is the fracture porosity; C_f is the concentration in a fracture at depth z and time t; D is the coefficient of hydrodynamic dispersion in the fractures; V_a is the Darcy velocity; Δ is the surface area of fracture per unit volume within the fractured medium; K_f is the fracture distribution coefficient for sorption onto the fractured surface defined by Freeze and Cherry (1979) as the mass of solute adsorbed per unit area of surface divided by the concentration of solute in solution; λ is the first order decay constant; q is the rate of contaminant transport into the matrix by diffusion from the fractures. The rate of contaminant transport into the matrix, q , depends on the porosity of the intact rock, the diffusion coefficient of the matrix material, and fracture spacing.

2.5 **Modelling a Finite Mass of Contaminant**

The concentration of potential contaminants generally increases during operation of the disposal facility, reaches a peak and then declines. The increase in concentration may be

related to (Rowe *et al.*, 1996); (a) the physical processes of leaching of contaminant from solid waste as water infiltrates through the waste and or (b) chemical and biological processes which generate the chemical species of interest from the synthesis, or breakdown, of existing chemical species in the waste (e.g. due to biological action such as oxidation-reduction reactions as a result of organic matter being oxidized in the presence of bacteria and oxygen).

Similarly, the decrease in concentration with time may be related to the processes that form leachate from the landfill and or processes which result in precipitation and or the synthesis or breakdown of the chemical species of interest into other chemical forms. In the design of barrier systems, reasonable engineering approximations can be made which will allow the designer to obtain some insight into the potential impact of the finite mass of contaminant. Hence, two basic assumptions are necessary; firstly, that the concentration of a contaminant of interest reaches the peak concentration, C_o, instantaneously, and secondly, all of the mass of this contaminant species, M_{TC}, is in solution at the time that peak concentration occurs.

The mass of contaminant available for transport into the soil can be represented in terms of the peak concentration, C_o, and the reference height of leachate, H_r, or the equivalent height of leachate, H_f. In laboratory diffusion tests, H_r and H_f are identical and correspond to the actual height of leachate directly above the soil. But for a landfill, H_r, may be defined for each contaminant species of interest and corresponds to the volume of fluid that at a concentration, C_o, would contain the total mass, M_o, of that contaminant species which could be released either for transport or collection. Considering conservation of mass within the source solution (Rowe *et al.*, 1996), the mass of contaminant can be expressed as:

[Mass of contaminant within source at time t] = [Initial mass of source contaminant] - [Mass of contaminant transported into the soil] – [Mass of contaminant lost due to first order decay processes] or

$$M_t = M_{TC} - M - M_{DC} \quad \text{---------- (2.21)}$$

Substituting eq. (2.6) for the mass of contaminant transported into the soil gives

$$M_t = M_{TC} - A_o \int_0^t (nvc\,(t) - nD\frac{dc(t)}{dz})dt - M_{DC} \quad \text{----- (2.22)}$$

where $M_t = A_o H_r C(t)$ is the mass of contaminant in source at time t; $M_{TC} =$ $A_o H_r C_o$ is the initial mass of contaminant in the landfill; $M_{DC} = A_o H_r \int_0^t \lambda_T C(t)\,dt$ is the mass lost due to first order decay processes; C (t) is the concentration of the contaminant in the landfill at time t.

Hence, if f_T is the flux entering the surface of the deposit and C_T is the concentration at the surface, then

$$C_T(t) = C_o(-1/H_r)\int_0^t f_T(t)\,dt - \int_0^t \lambda_T C_T(t)\,dt \quad \text{----- (2.23a)}$$

If λ_T = o and $H_r \rightarrow \infty$, eq. (2.23a) leads to the boundary condition $C_T(t) = C_o$. Eq. (2.23a) explicitly models both the full mass of a given contaminant in the landfill ($M_{TC} = A_o H_r C_o$) and removal of contaminant from the landfill (e.g. by a leachate collection system).

An alternative method is to reduce the total mass to the amount available for transport into the groundwater ($M_o = A_o H_f C_o$) i.e. excluding that portion of the mass which will be collected by a primary leachate collection system. For this case, the finite mass boundary condition can be written in terms of the equivalent height of leachate, H_f, giving rise to boundary condition (Rowe *et al.*, 1996)

$$C_T(t) = C_o (1/H_f) \int_0^t f_T(t)\, dt \text{ ---------- (2.23b)}$$

where H_f is the equivalent height of leachate.

If $\lambda_T = O$ (i.e no mass removed from landfill except by migration into the underlying barrier system or groundwater) then $H_r = H_f$ and eqs. (2.23a) and (2.23b) are identical.

2.6 **Modelling a Thin Permeable Layer as a Boundary Condition**

It is desirable to be able to model advective-diffusive transport through a relatively thin clayey barrier and along an aquifer of thickness h and porosity n_b. Assuming that the mass flux, f_x, into the aquifer is governed by eq. (2.8), then the concentration of the contaminant in the aquifer at position x at time t (i.e C_b (x, t)) can be expressed as (Rowe *et al.*, 1996):

$$C_b(x, t) = \int_0^t \left[\frac{f_b(x,c,t)}{n_b h} - \frac{V_b}{n_b}\frac{dc_b(x,t)}{dx} + D_H \frac{d2c_b(x,t)}{dx2} \right] dt - \int_0^t \lambda^*_b C_b(x,t) dt \text{.. (2.24)}$$

where V_b is the Darcy velocity in the aquifer (in the x direction); D_H is the coefficient of hydrodynamic dispersion in the aquifer; C_b (x, t) is the concentration in the aquifer at position x at time t; f_b (x,t) is the flux into the aquifer at point x at time t; λ^*_b is the first order decay constant for the base.

For two-dimensional problems, an aquifer can be modelled as a boundary condition by simply applying eq. (2.24).There are many practical situations where additional simplification is possible by considering one-dimensional (1-D) advective-diffusive transport down to the aquifer (e.g vertically) combined with 1-D advective (e.g horizontally) transport out from beneath the landfill.

Hence, for a landfill of width W and length L (in the direction of groundwater flow), the equation for concentration in the aquifer (eq. (2.24)) reduces to (Rowe *et al.*, 1996):

$$C_b(t) = \int_0^t [\, f_b(C_b,t)/n_b h - \lambda_b C_b(t)]\, dt \quad \text{-------- ---------- (2.25)}$$

2.7 **Some Analytical Solutions in Use**

There are many analytical solutions to eq. (2.24) using simplified cases typically involving a single homogeneous layer (barrier) subject to simplified boundary conditions. These are considered in the subsequent paragraphs.

2.7.1 **Ogata-Banks equation**

A well-known analytical solution is that for the concentration, C, at time, t, and depth, z, beneath the surface of a barrier which is assumed to be infinitely deep and subject to a constant surface concentration, C_o (Ogata, 1970):

$C(z,o) = 0 \qquad z > o \qquad$ ---------- (2.26a)

$C(o,t) = C_o \qquad t \geq o \qquad$ ---------- (2.26b)

$C(\alpha,t) = 0 \qquad t \geq o \qquad$ ---------- (2.26c)

This solution can be written as:

$$\frac{C}{C_o} = \frac{1}{2}\left[\operatorname{erfc}\left(\frac{(z - vt)}{2(Dt)^{1/2}}\right) + \exp(vz/D)\operatorname{erfc}\left(\frac{(z + vt)}{2(Dt)^{1/2}}\right)\right] \ldots\ldots (2.27)$$

where erfc (x) = 1 – erf (x).

2.7.2 **Booker-Rowe equation**

An analytical solution for one-dimensional migration in an infinitely deep deposit where the source concentration varies with time (as mass is transported into the barrier) has been proposed by Booker and Rowe (1987). In this case, the landfill can be represented in

terms of an equivalent height of leachate, H_f, and the concentration at any depth, z, and time, t, can be written as:

$$C(z,t) = C_o \exp(ab-b^2t)\,(bf(b,t) - df\,\frac{(d,t)}{(b-d)} \quad \text{---------- (2.28a)}$$

where

$$f(b,t) = \exp(ab + b^2t)\,\text{erfc}\,(a + \frac{bt}{2t}) \quad \text{---------- (2.28b)}$$

$$f(d,t) = \exp(ad + d^2t)\,\text{erfc}\,(a + \frac{dt}{2t}) \quad \text{---------- (2.28c)}$$

$$a = Z[(n+ \rho K_d)/nD]^{½}$$

$$b = V\,[n/(4D(n+ \rho K_d))]½$$

$$d = \frac{nD}{H_f}\,[(n + \rho K_d)/nD]½ - b$$

All the terms are as previously defined.

2.8 **Integration of Hydrogeology in Landfill Engineering Designs**

In order to design waste disposal sites [engineered barrier systems] it is essential that the interaction between the natural and engineered systems be considered (Rowe *et al*., 1996). Some of the most important considerations are discussed as follows:

2.8.1 **Hydraulic conductivity of aquitards**

In the design of systems where there will be gradients, and hence flow out of the waste disposal facility, the bulk hydraulic conductivity of the barrier or aquitard will be a key factor affecting potential impact of the landfill on an underlying aquifer. The installation of a liner over a fractured rock may result in the performance of the combined system being substantially better than that of either the liner alone or fractured aquitard alone. When

dealing with outward gradients, it is generally conservative to adopt the highest reasonable hydraulic conductivity for the purpose of impact assessment (Rowe *et al*., 1996).

The process of matrix diffusion (i.e the migration of contaminants between the fractures and the pore of the adjacent intact material due to diffusion from high concentration to low concentration) in retarding contaminant migration through fractured media is important. However, when dealing with inward gradient designs (i.e hydraulic traps), the conventional approach is no longer valid. Rather, the greatest impact is likely to occur for the lowest reasonable hydraulic conductivity as contaminants will migrate outwards from the landfill due to the process of molecular diffusion which will be opposed by the inward flow of water into the landfill.

Furthermore, a well-constructed geomembrane liner installed over the compacted clay liner will likely control the transport of contaminants by the process of diffusion to minimize the demand for water to maintain the engineered hydraulic trap. In this case, the potential for impact on the underlying aquifer may be minimized by active removal of contaminated water from the hydraulic control layer (HCL) when significant levels of contaminant eventually break through the primary clay liner.

2.8.2 **Some considerations in the design of highly engineered barrier systems**

There are many ways in which an engineered permeable layer beneath the primary liner can be designed. This section highlights how the hydrogeology and engineered designs interrelate.

(1) **Potentiometric surface in aquifer below secondary collection system:** For situations where the water table and potentiometric surface in the underlying aquifer are well below the base of the landfill, the construction of a permeable drainage system, which located beneath the compacted clay liner, serves two purposes (Rowe *et al*., 1996). Firstly, the drainage layer

functions as a secondary leachate collection system, which can remove a portion of the leachate which escapes through the liner (some escape is to be expected through any liner system where there are downward gradients). Secondly, this layer serves to reduce the hydraulic gradient through the underlying soil.

The impact of the contaminant on the underlying aquifer depends on the properties of the soil, the drainage system and the liner, the geometry of the landfill, the design of the primary and secondary leachate collection system, and the properties of any underlying aquifer. In addition, contaminant migration through the natural soil must be anticipated in the design of the barrier system. Even in the limiting case where no leachate mounding occurred in the secondary leachate collection system and all the leachate that escaped through the liner was collected, there would still be diffusion of contaminants into the natural soil from the secondary leachate collection layer. In most cases, there will also be downward advective contaminant transport, since not all the leachate escaping through the liner is likely to be collected. Indeed, situations can readily occur where the majority of the leachate migrating through the primary liner also migrates down through the natural soil.

Ideally, the separation between the aquifer and the landfill would be as large as possible for a design involving a secondary leachate collection system. However, in many practical situations, the actual thickness may be quite small. Under these circumstances, it cannot be assumed that even a perfectly operating liner, and primary and secondary leachate collection system will necessarily prevent contamination of the underlying aquifer.

For a thin secondary liner with a perfect secondary leachate collection system where there is no advective escape through the secondary liner, the process of molecular diffusion through the secondary liner results in significant impact in the aquifer (Rowe *et al.*, 1996). With a thick secondary liner-aquintard system, the impact is much smaller. For cases where contaminant migration is only by diffusion, dilution in the aquifer is an important attenuation

mechanism. If the Darcy velocity were ten times higher, the impacts would be about ten times less. Thus, for these systems, the hydrogeologic characterization of the head and flow in the aquifer can be quite important when assessing the potential impact due to a proposed design.

(2) **Secondary leachate collection (composite primary liner)**: The advective transport through the primary compacted clay liner can sometimes amount to a great percentage of leahate generated, thereby creating a significant impact on the underlying aquifer. One method of reducing this negative impact is to place a geomembrane above the compacted clay liner to create a composite primary liner (Rowe *et al*., 1996).

Diffusion transport mechanism is a major consideration in the design of systems that use a geomembrane liner to minimize advective transport through the liner system. Though considerable research has been conducted into the diffusion of contaminants in clayey barriers, however, much more research is required into the diffusion of contaminants through geomembranes and also through unsaturated granular or geosynthetic secondary leachate collection systems since this is likely to control the impact on these systems.

(3) **Hydraulic control layer:** In this case (Rowe *et al*., 1996), there is both a natural hydraulic trap (i.e. water flows from the natural soil into the hydraulic control layer) and an engineered hydraulic trap (i.e. water flows from the hydraulic control layer into the landfill). There are three factors that must be considered in the design of this engineered hydraulic trap. Firstly, the head in the hydraulic control layer must be controlled such that blowout of either liner does not occur during or after construction. Secondly, the volumes of water collected by the hydraulic trap must be manageable, and the hydrogeologic system must have the capacity to provide the water required to maintain the hydraulic trap; if this is not the case, then the head in the aquifer will drop, and the effectiveness of the trap may deteriorate

with time. Thirdly, although there is a hydraulic trap, some outward diffusion of contaminants is to be expected in most cases.

Contaminant migration analyses are required to assess what impact may occur under these conditions. If the impact at the site boundary is not acceptable, then it can be reduced by pumping water through the hydraulic control layer (i.e. by injecting fresh water at one end and extracting contaminated water at the other end). This design has the following advantages (Rowe *et al*., 1996). Firstly, since there is inward flow to the hydraulic control layer and a relatively impermeable clay liner, it may be possible to design the system such that the engineered hydraulic trap is entirely passive. That is, all water required to maintain an inward gradient is provided by the natural hydrogeologic system, and no injection of water to the hydraulic control layer is required. Secondly, because of the two-level hydraulic trap, there will be substantially greater attenuation of any contaminants that do migrate through the primary liner.

Thirdly, since fluid can be injected and withdrawn from the hydraulic control layer, it is possible to control the concentration of contaminant in the layer, and hence minimize the impact at the site boundary in the event of a major failure of the leachate collection system. In the design of hydraulic control layers, it is important that the transmissivity of the layer is high enough to ensure a relatively uniform head on the base of the primary and top of the sendary liners over the area of the landfill. Generally, the larger the landfill, the greater the transmissivity required. Impact on a potential aquifer will depend on both the hydraulic conductivity of the various natural and engineered components of the system and the head distribution between the aquifer and the top of the primary liner system. Basically, the most important considerations in the design of barrier systems include the following (Rowe *et al.*, 1996).

(i) Advective transport out from the landfill needs to be minimized in order

to minimize impact on an underlying aquifer.

(ii) The design of engineered systems requires careful consideration of the likely range of hydraulic conductivities of both the artificial and natural components of the barrier system.

(iii) For some engineered systems, it is not necessarily conservative to consider either the highest or lowest hydraulic conductivity values when assessing potential contaminant impact. In many cases, a sensitivity study over the expected range may be required in order to identify the peak potential impact.

(iv) Based on the limited available evidence, it would appear that there can be significant diffusion of at least some contaminants through even relatively thick (2mm) HDPE geomembranes. Under these circumstances, there can be significant impact on an aquifer due to diffusion through the geomembrane, even if the volume of leachate physically leaking through defects in the geomembrane is negligible. Thus, the use of a geomembrane or composite geomembrane-clay liner to minimize physical leakage is not, in itself, enough to ensure negligible impact on an aquifer. However, the geomembrane may be a very useful component of a barrier system.

(v) Even a perfect secondary leachate collection system which collects 100% of the volume of fluid escaping through the primary liner may not be sufficient to prevent impact on an underlying aquifer.

(vi) The effectiveness of the secondary leachate collection system as a barrier to contaminant transport is considerably enhanced when the advection through the primary liner is minimized. This is most effectively achieved by using a composite liner composed of a geomembrane and clay layer.

(vii) The performance of a secondary leachate collection system as a barrier to contaminant transport can be enhanced by designing the system (where practical) to encourage inward

flow from an underlying aquifer to the secondary layer. This may reduce the effectiveness of the secondary layer for the detection of leaks through the primary liner, but it can considerably enhance the resistance to outward contaminant movement from the secondary collection system to an underlying aquifer. Thus, in many cases, the disadvantages of this approach are outweighed by the advantages in terms of long-term environmental protection.

(viii)If there is an outward gradient from a secondary system to an underlying aquifer, there is an advantage to constructing a secondary composite liner below the secondary collection system. However, due to potential diffusive transport through the secondary composite liner, its presence does not guarantee negligible impact in all cases; each case must be examined individually from the perspective of potential contaminant impact.

(ix) The presence of an unsaturated zone (e.g the secondary leachate collection system) has the potential to reduce the diffusion of contaminant for systems where there is a well-constructed composite liner which minimizes leakage to negligible levels. More research is required to quantify fully the effectiveness of these unsaturated systems as diffusion barriers.

(x) In situations where the potentiometric surface in an aquifer is high relative to the base contours, a natural hydraulic trap can be designed to minimize the impact on an underlying aquifer. In situations where the natural trap alone is not sufficient, the introduction of a hydraulic control layer can permit long-term control of potential impact on an underlying aquifer.

2.8.3 Effect of an engineered facility on groundwater (shadow effect)

A key consideration in hydrogeologic investigations is the understanding of the groundwater flow direction, and of the natural variability in the water table and in the potentiometric surface in any underlying aquifers, and flow in these aquifers (Rowe *et al*., 1996). Landfills have been designed on the basis of this information; however, it is important

to recognize that the construction of a landfill may change these conditions. This is particularly true when the landfill is of a large extent.

If a landfill is located in a recharge area, the construction of an engineered landfill involving a leachate collection system and, frequently, one or more liners, will often reduce the recharge to the underlying aquifer. Thus, the landfill casts a hydraulic shadow over the aquifer (and hence the term 'shadow effect') which may lead to a drop in water levels (i.e. lowering of the potentiometric surface) within the aquifer. The extent to which this will occur will depend on the existing gradients and transmissivity of the aquifer, the hydraulic conductivity of the engineered barriers and the elevation of the base contours relative to the existing potentiometric surface in the aquifer.

The greatest potential effect of the landfill on groundwater conditions occurs when a landfill of large areal extent is to be designed as a natural hydraulic trap above an aquifer of low transmissivity. In this case, not only is the charge cut off over the area of the landfill, but also water is being extracted from the aquifer to provide the inward flow necessary to have a natural hydraulic trap. This can lead to a significant drop in water levels in the aquifer and, if not considered in the design, could lead to failure of the hydraulic trap. The magnitude of the change in water levels may be estimated using numerical models or analytical models (Rowe and Nadarajah, 1994).

The potential significance of the shadow effect can only be assessed by an integrated examination of both the engineering design and existing hydrogeology. This will frequently involve flow modelling. In instances where the water levels in the aquifer are critical to the design, consideration should be given to the potential implications of long-term variation in water levels due to factors such as long-term changes in climate and potential development in the recharge area for the aquifer. These effects are quite intangible, and in these cases a contingency may be required to deal with changes in the aquifer water levels.

Under operating conditions, no water is intended to be added or removed from this layer (i.e. the heads in the hydraulic control layers will be maintained by flow from the underlying aquifer), however, in the event of some unexpected drop in the potentiometric surface in the aquifer, water may be added to the hydraulic control layer to create an engineered hydraulic trap. Alternatively, the layer may be pumped and operated as a secondary leachate collection system.

2.8.4 **Service life of collection systems, leachate monitoring and trigger levels**

It is expected that the primary leachate collection system will maintain the leachate head on the primary liner to 0.3m or less. There are many mechanisms that can cause degradation of even an excellent landfill. Based on available evidence, it would seem reasonable to project a service life for a well–designed multicomponent leachate system for municipal solid waste to be 40-100 years (Rowe *et al*., 1996).

The service life of systems with small stone-sand blanket and or inappropriately used geonets or geotextiles may be substantially less. Thus, if the contaminating lifespan of the landfill exceeds the service life of the primary collection system, consideration must be given to the implications of increased leachate mounding as the collection system fails. It is noted that a collection blanket may be regarded to have failed once its hydraulic conductivity drops to about 10^{-6}m/s (or lower); at this hydraulic conductivity it may still be collecting leachate, however, mounding of leachate will occur, increasing the head on the base of the landfill, and hence increasing the potential for contaminant impact on underlying aquifer.

Contingency measures for landfills, such as the installation of purge wells in an aquifer, have typically been intended to remove contaminants if they unpredictably reach the aquifer. However, if the contaminating lifespan of the landfill exceeds the service life of the primary leachate collection system, then it will be prudent to design an alternative means of

controlling leachate levels once such a failure occurs, rather than relying on purge wells in an underlying aquifer to collect contaminants after they reach the aquifer. This can be, for example, by means of leachate extraction wells in the waste (Rowe and Nadarajah, 1994), or by collecting the increased exfiltration through the primary liner by means of an appropriately designed secondary leachate collection system or active operation of a hydraulic control layer.

Accepting that, at some time, failure of a leachate underdrain system is likely to occur, it is necessary to monitor leachate levels to detect this failure, and to have a trigger in terms of leachate level and concentration at which alternative leachate control will be initiated. The trigger levels of leachate mounding will vary from one landfill to another and will depend on the level of attenuation which can occur between the base of the landfill and the underlying aquifer (Rowe *et al.*, 1996). This in turn will depend on the engineering, and the hydrogeologic characteristics of the underlying strata. Based on the available field data and engineered design, calculations can be performed to consider these factors to establish triggers in terms of leachate levels and concentrations at which leachate control measures will be initiated.

2.8.5 **Contaminating lifespan and finite mass of contaminant**

The contaminating lifespan of a landfill is the period of time during which the landfill will produce contaminants at levels that could have unacceptable impact if they were discharged into the surrounding environment (Rowe *et al.*, 1996). When dealing with groundwater contamination, it is necessary to consider the transport pathway (and consequent attenuation) when assessing the contaminating lifespan. This varies from one landfill to another. The contaminating lifespan of a landfill depends on the mass of contaminant per unit area, the infiltration through the cover, leachate characteristics, and the pathway for

contaminant release. The thicker the waste, the greater the mass of any given contaminant and, all other things being equal, the longer the contaminating lifespan. The greater the infiltration, the shorter will be the contaminating lifespan, since there is greater opportunity for contaminant to be leached out and removed. The greater the potential for attenuation along the escape pathway, the shorter the contaminating lifespan. For waste disposal sites such as municipal landfills, the mass of any potential contaminant within the landfill is finite. The process of collecting and treating leachate involves the removal of mass from the landfill, and hence a decrease in the amount of contaminant that is available for transport into the general groundwater system.

Similarly, the migration of contaminant through the underlying deposit also results in a decrease in the mass available within the landfill. For a situation where leachate is continually being generated, the removal of mass by either leachate collection and or contaminant migration will result in a decrease in leachate strength with time. The peak concentration, C_o, of given contaminant species can usually be estimated from past experience with similar landfills. The total mass of contaminant is more difficult to determine, however, estimates can be made by considering the observed variation in concentration with time at landfills where leachate concentration has been monitored or by considering the composition of the waste.

For the purposes of modelling the decrease in concentration in the leachate due to movement of contaminant into the collection system and through the barrier, it is convenient to represent the mass of a particular contaminant species in terms of the reference height of leachate, H_r (Rowe *et al.*, 1996), defined below. If the initial mass, M_{TC}, of a contaminant species can be estimated, then the reference volume of leachate which would contain this mass at an initial concentration, C_o, is;

$$V_{TC} = \frac{M_{TC}}{_____} \qquad (2.29)$$

C_o

In general, this volume will not correspond to the actual volume of leachate because it is based on the assumption that all this available mass can be quickly leached from the solid waste. It is convenient for both mathematical and physical reasons to express the volume, V_{TC}, in terms of a reference height of leachate, H_r, which is defined as the reference volume of leachate divided by the area, A_o, through which contaminant passes into the primary barrier:

$$H_r = \frac{V_{TC}}{A_o} \qquad \text{---------- (2.30)}$$

or

$$H_r = \frac{M_{TC}}{C_o A_o} \qquad \text{---------- (2.31)}$$

where M_{TC} is the total mass of a contaminant species of interest; C_o is the peak concentration of that species in the landfill; A_o is the area through which contaminant can migrate into the underlying layer; H_r is the mass of contaminant available for transport into the soil and or collection by the leachate collection system and or loss to surface wasters through landfill seeps.

For the case where there is an infiltration, q_o, through the landfill cover, and where all the leachate is collected (i.e no migration into the underlying soils), it can be mathematically shown (Rowe, 1991) that the process of dilution within the landfill gives rise to a decrease in the average annual leachate strength with time (recognizing that leachate concentrations may vary seasonally throughout a year) such that the concentration in the landfill at time t, C_{OL} (t), is related to the peak concentration C_o by the relationship:

$$C_{OL}(t) = C_o \exp(-q_o t)/ H_r \qquad \text{---------- (2.32a)}$$

or, on rearranging terms, the time required for the leachate strength to reduce to some specified value, C_{OL}, is given by,

$$t = -\frac{H_r}{q_o} \ln (C_{OL})/C_o \qquad \text{---------- (2.32b)}$$

where H_r is the reference height of leachate, and q_o the infiltration.

For contaminant species that experience first order decay (e.g. due to biological and or chemical processes), with a decay constant, Γ_{BT}, eqs. (2.32a) and (2.32b) can be rewritten as (Rowe *et al*., 1996),

$$C_{OL}(t) = C_o \exp\left[-\left(\frac{q_o}{H_r} + \Gamma_{BT} \right) t \right] \qquad \text{……..(2.32c)}$$

and on rearranging,

$$t = -\left(\frac{q_o}{H_r} + \Gamma_{BT} \right)^{-1} \ln\left(\frac{C_{OL}}{C_o} \right) \qquad \text{……..(2.32d)}$$

The decline of contaminant concentration as implied by eqs. (2.32a) and (2.32c) has been observed for both conservative and non-conservative species. It may be argued that in some cases, not all the contaminant in the source is immediately available for transport or collection. For example, some components of a contaminant may be released due to biological breakdown of the waste over a period of years. In these cases, the source concentration may increase to a peak value and then remain relatively constant (allowing for the usual seasonal variations) for a period of time, because the removal of contaminant is being balanced by new contaminants becoming available (e.g. due to biodegradation). However, eventually all the contaminants that can be released are released, and at this point the concentration in the landfill must drop (due to conservation of mass). If a reasonable estimate can be made of the time period over which the source concentration is likely to remain relatively constant, then the behaviour can be readily modelled.

When considering the contaminating lifespan of a landfill, it is necessary to define what is meant by unacceptable impact. In the Province of Ontario, the Ministry of the Environment and Energy has a policy such that if a reasonable use for groundwater was as drinking water, then an unacceptable impact could be interpreted as an increase in contaminant which exceeds half the difference between the drinking water objective and background levels for aesthetic parameters (e.g. chloride), and a quarter of the difference between the drinking water objective and background for health-related parameters, e.g. dichloromethane (Rowe *et al.*, 1996). For chloride, this would mean a maximum increase at the site boundaries of 125mg/l (or less if there are background levels of chloride in the groundwater). Based on leachate strength decay, one can estimate the time required to reach a given concentration using eqs. (2.32b) or (2.32d).

2.8.6 **Modelling the service life of primary and secondary systems**

The contaminating lifespan of a landfill can also be determined by estimating the service life of the key engineering features and then modelling the operation and, at the appropriate time, breakdown of these engineered features. This can be done using a programme, POLLUTE, (Rowe *et al.*, 1996) by considering the following three steps.

(1) The initial operation of the system is modelled as a natural hydraulic trap with water flowing upward from the aquifer through the secondary liner, the hydraulic control layer, primary liner and into the landfill. While the primary leachate collection system functions as designed, the water level in the hydraulic control layer will be controlled by the hydraulics of the system which can be readily calculated. The hydraulic control is completely passive during this period. The system requires no human intervention except to pump the primary leachate collection system continuously.

(2) After some time, t_1 (e.g. 50years) the service life of the primary leachate collection system is reached, and it fails to control the leachate mound at the design value. A leachate head on the liner will then build up. Once this is detected (either by monitoring the leachate mound in the landfill or by monitoring the hydraulic control layer), the hydraulic control layer may be pumped and operated as a secondary leachate collection system. The gradual build-up in the leachate mound can be modelled over a period of time. Pumping of the secondary leachate collection system induces a natural hydraulic trap from the aquifer to the secondary collection system. There will be a strong downward gradient from the landfill through the primary liner and into the secondary system, but only diffusion from the secondary system down to the aquifer.

(3) After some time, the service life of the secondary leachate collection system is reached and it fails to remove all the leachate. As the hydraulic conductivity of the secondary system falls, there will be a gradual build-up in pressure in the layer over a period of time, and contaminant will begin to move down to the aquifer by advection as well as diffusion. Deterministic analyses can be readily performed based on assumed service lives of the engineering systems. The greatest problem is estimating the service lives. Since there is inevitably some uncertainty regarding the service lives, one approach is to develop some distribution of service life for each system and then perform Monte Carlo simulations to establish the probability that the concentration in the aquifer will exceed some limiting allowable value.

Due to the impending environmental hazards, a country like Nigeria that depends a lot on groundwater resources should develop strong requirements for groundwater protection. In the USA, for example, the Congress has mandated regulations relating to the minimum design for waste disposal sites which involve a single composite liner and a landfill cover no more permeable than the liner as the standard design. Barriers should not be designed in

isolation. Careful considerations must be given to the site hydrogeology, geotechnical conditions and climate as well as the size of the footprint and capacity of the landfill. Depending on these conditions, landfills which will not significantly affect groundwater quality may range from very simple designs involving natural clayey barriers to highly engineered systems involving multiple liners and leachate collection systems.

CHAPTER THREE

LITERATURE REVIEW

3.1 **Major Transport Mechanisms**

The three major transport mechanisms of miscible contaminants (one-dimensional) in saturated porous media are: (1) Advection, (2) diffusion, and (3) mechanical dispersion (Shackelford, 1993). Their impact on aquifers is discussed as follows.

3.1.1 **Advection**

This is the process by which solutes are transported along with the flowing fluid or solvent, typically water, in response to a gradient in total hydraulic head. Due to advection, non-reactive 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 fluid, and can be represented as (Shackelford, 1993):

$$V_S = \frac{V}{n} \qquad \text{……..(3.1)}$$

where V_s is the seepage or average liner velocity of the water or solvent; n is the total porosity of the porous material; V is the flux of water, which is the quantity of flow per unit area per unit time.

This eq. (3.1) presupposes that all the voids participate in conducting flow. However, for one-dimensional flow, the flux can be expressed by Darcy's law as

$$V = \frac{Q}{A} = -\frac{Kdh}{dx} = Ki \qquad \text{……..(3.2)}$$

where Q is the volumetric flow rate of the water; A is the total cross-sectional area (solids plus voids) perpendicular to the direction of flow; K is the hydraulic conductivity; h is the total hydraulic head; x is the direction of flow; i is the dimensionless hydraulic gradient. The seepage velocity, V_s, reflects the fact that the fluid can actually flow only through the void space of the porous material whereas the flux, V, represents the volumetric flow of fluid through the total cross-sectional area of the material.

The time required for a nonreactive solute to migrate through a saturated soil of thickness L, known as the solute transit time, due to advection can be calculated using the seepage velocity as follows:

$$t = \frac{L}{V_S} = \frac{nL}{Ki} \qquad \text{..... (3.3)}$$

where t is the transit time.

Using eq. (3.3) to estimate the transit time of the solute, implies that all solute constituents are carried along with the fluid at the velocity V_s. Under these conditions, the advective mass flux of a particular chemical species can be calculated as,

$$J_A = VC = Kic = nV_SC \qquad \text{.....(3.4)}$$

where J_A is the advective mass flux (mass flowing through a unit cross-sectional area in a unit time); C is the concentration of the solute in the liquid phase of the porous materials based on the volume of solution in the porous material (i.e. mass of solute per unit volume of mixture).

With some porous materials (such as fractured rocks), most of the advective flow occurs through only part of the total void space of the material. For these materials, an effective porosity, n_e, is defined as the volume of fluid conducting pores divided by the total volume (pores and solids) of the material. For materials with n_e less than n, n_e should be substituted for n in eqs. (3.1) – (3.4).

3.1.2 **Diffusion**

This is a transport mechanism whereby a chemical or chemical species migrate in response to a gradient in its concentration, although the actual driving force for diffusion is the gradient in chemical potential of the solute (Robinson and Stokes, 1959). A hydraulic gradient is not required for transport of contaminants by diffusion.

The fundamental equation for diffusion is Fick's first law, which, for one-dimensional transport, can be written as (Shackelford, 1993),

$$J_D = - D_o \frac{dc}{dx} \qquad \text{---------- (3.5)}$$

where J_D is the diffusive mass flux, x is the direction of transport, and D_o is the free-solution diffusion coefficient. Eq.(3.5) represents the one-dimensional form of Fick's first law describing diffusion in aqueous or free solution (i.e. no porous material). For diffusive transport in saturated porous material, a modified form of Fick's first law is used as

$$J_D = -\tau D_o n \frac{dc}{dx} \qquad \text{---------- (3.6)}$$

or

$$J_D = - D^* n \frac{dc}{dx} \qquad \text{---------- (3.7)}$$

where τ is a dimensionless tortuosity factor; D^* is the effective diffusion coefficient. The porosity term is required in eq. (3.7) since the diffusive flux, J_D, is defined with respect to the total cross-sectional area of the porous medium. The tortuosity factor accounts for the increased distance of transport and the more tortuous pathways experienced by solutes diffusing through porous media. Tortuosity can be expressed as

$$\tau = \left(\frac{L}{L_e} \right)^2 \qquad \text{---------- (3.8)}$$

where L is the macroscopic, straight-line distance between two points defining the flow path; L_e is the actual, microscopic or effective distance of transport between the same two points. Since $L_e > L$, $\tau < 1.0$ and $D^* < D_o$. Therefore, mass transport due to diffusion in porous media is slower than mass transport due to diffusion in free or aqueous solution.

3.1.3 **Mechanical dispersion**

In traditional contaminant transport theory, a mechanical dispersive flux, J_M, is added to the total mass flux of the solute to account for the spreading of the solute due to variations in the seepage velocity, V_s, which occur during transport in and through porous materials. On a microscopic level, these variations are thought to be related to three different effects [Bear, 1979; Freeze and Cherry, 1979]. First, the velocity of flow across any given pore channel within the material will be greater in the middle of the pore channel than it is near the walls of the pore channel. This is the same effect known to occur in fluid flow through pipes and rivers, streams, and channels. Second, the equation of continuity predicts that the flow velocity across a smaller pore opening will be greater than that across a larger pore opening. Third, velocity variations will result due to the tortuous nature of the flow paths existing in nearly all porous materials. On a larger scale, mechanical dispersion is thought to be caused by the different flow rates resulting from heterogeneities that typically are encountered whenever transport occurs over relatively larger areas or regions.

This type of dispersion can be found where flow through the bulk sand medium is interrupted by the existence of the low-hydraulic-conductivity clay lenses dispersed throughout the sand layer. Due to the existence of the clay lenses, the transport proceeds around rather than through the clay resulting in a situation which is analogous to that of microscopic dispersion. Therefore, this dispersion at a large scale is thought to be a

macroscopic tortuosity effect. The mechanical dispersive flux typically is assumed to be a Fickian process, which, for one dimension, can be expressed as (Shackelford, 1993):

$$J_m = - D_m n \frac{dc}{dx} \quad \text{---------- (3.9)}$$

where J_m is the mechanical dispersive flux; D_m is the mechanical dispersion coefficient. Since mechanical dispersion results from variations in the magnitude of the seepage velocity, the mechanical dispersion coefficient often is assumed to be a function of the seepage velocity, or

$$D_m = \alpha_L V_s^{\beta} \quad \text{---------- (3.10)}$$

where α_L is the longitudinal dispersivity of the porous medium in the direction of transport, and β is an empirically determined constant between one and two (Freeze and Cherry, 1979). In most applications, the exponent, β, is assumed to be unity, i.e., D_m is assumed to be a linear function of V_s in eq. 3.10. However, β may be greater than unity in many situations. Also, the dispersivity, α_L, is probably scale dependent with larger values for α_L being associated with greater transport distances (Pickens and Grisak, 1981). For example, values of α_L reported from the results of field studies may be as much as four to six orders of magnitude greater than the corresponding laboratory measured values which commonly are found to range between 0.1 and 10mm (Freeze and Cherry, 1979). Regardless of the appropriate values for α_L and β, the relationship for D_m in eq. (3.10) indicates that the effect of mechanical dispersion increases as the seepage velocity increases and vice versa. As a result, for typical problems involving slow-hydraulic-conductivity barrier materials, mechanical dispersion does not appear to be significant due to the relatively low flow rates (Shackelford, 1993).

3.1.4 **Combined transport**

In the absence of coupled flow processes, the total mass flux, J, of the solute or contaminant species is the sum of the advective, diffusive, and dispersive fluxes, or

$$J = J_A + J_D + J_M \text{ ---------- (3.11)}$$

Based on eqs. (2.4), (3.7), and (3.9), the total mass flux for one-dimensional transport in saturated porous material is (Shakelford, 1993):

$$J = nV_s c - D^* n \frac{dc}{dx} - D_m n \frac{dc}{dx} \text{ ---------- (3.12)}$$

or $$J = nV_s c - D_h n \frac{dc}{dx} \text{ ---------- (3.13)}$$

where D_h is the hydrodynamic dispersion coefficient given by

$$D_h = D^* + D_m = D^* + \alpha_L V_s \text{ ---------- (3.14)}$$

The exponent β in eq. (3.10) has been assumed to be unity in the formulation of eq. (3.14). The hydrodynamic dispersion coefficient accounts for dispersion of the solute due to both diffusion and mechanical dispersion.

3.2 **Coupled Flow Processes**

In general, water can flow through porous media not only in response to a gradient in total hydraulic head but also in response to gradients in chemical composition (chemico-osmosis), electricity (electro-osmosis), and temperature or thermal osmosis (Shackelford, 1993). Also, in the absence of advective flow, solutes can migrate through porous materials not only in response to a concentration gradient but also in response to an electrical gradient (electro-phoresis) and a thermal gradient (thermal diffusion). Solutes also may be filtered (ultrafiltration) by some porous materials.

These additional flow processes typically are referred to as coupled flow processes in order to distinguish them from the direct flow processes represented by advection and

diffusion. The description of coupled flow processes is based on the theory of the thermodynamics of irreversible processes.

In the formulation of the contaminant transport equations, coupled flow processes often are neglected or are considered to be insignificant for at least two reasons. First, coupled flow processes are significant only at relatively low flow rates which typically are associated with flow through fine-grained soils (e.g clays and silts). Since most of the applications of contaminant transport theory traditionally have been associated with problems pertaining to the transport of contaminants through highly permeable, granular materials (e.g aquifers), the effects of coupled flow processes typically have been assumed to be negligible.

Nevertheless, coupled flow processes may be significant in materials with high activity and or low void ratio (Olsen, 1969). Second, the determination of the phenomenological coefficients associated with the equations describing coupled flow processes is difficult, at best. Hence, such determinations usually can be performed only under highly controlled conditions in the laboratory and extrapolation of the results to actual, field situations is uncertain.

3.3 **Contaminant Transport by Advective Mechanism:-**

Soil barriers containing enough clay minerals to provide low permeability (hydraulic conductivity) are used extensively to prevent the rapid advective migration of various leachates form waste disposal sites. The clayey barriers vary from thin bentonite liners (1-3cm thick), to compacted clay liners (0.9-3m thick), and to natural undisturbed clayey barriers up to 30 or 40 m thick (Rowe *et al.*, 1996).

The hydraulic conductivity of both undisturbed natural clays and compacted clayey liners depend on the mineralogy, the placement and the stress history of the liner. The performance of a clay liner depends on the choice of material (i.e the geologic origin,

grainsize distribution, and mineralogy), and the manner in which this material is broken up and recompacted.

3.3.1 **Liner specifications**

Generally, the following requirements are needed (Rowe *et al.*, 1996):

(1) The liner shall have a hydraulic conductivity, k, of 10^{-9}m/s or less, and be free of natural or compaction-induced fractures. From a chemical flux point of view, 10^{-10} m/s is preferable, since diffusion often becomes the dominant migration mechanism.

(2) Because low hydraulic conductivity is normally associated with the presence of clay minerals, a minimum of 15-20% of the soil with a particle size smaller than 2 microns (<2mm), a plasticity index greater than 7% and an activity of 0.3 or greater may be specified. Alternatively, a minimum cation exchange capacity of about 10 milliequivalents (meq) per 100g of soil may be specified.

(3) The clayey barrier shall be compatible with the leachates to be retained. In order words, the hydraulic conductivity, k, shall not increase significantly on exposure to leachate similar to that to be contained by the barrier.

(4) The minimum thickness of a compacted clayey liner for a domestic waste facility will generally be about 0.9-1m; however, this is highly variable from jurisdiction to jurisdiction and in some cases is reduced to 0.6m if the clayey liner is used together with a geomembrane to form a composite liner.

(5) The minimum thickness of a clayey liner for an industrial/toxic facility shall generally be 3-4m, although some jurisdictions require up to 15m or multiple composite liner systems. It should be noted that while the foregoing represent common characteristics of clayey liners, the design of a liner is site-specific, and its characteristics must be selected such that the barrier system (of which the liner is usually only part) will provide adequate control on

contaminant migration. This can only be assessed by appropriate site-specific contaminant impact modelling of the landfill-barrier system. Furthermore, experience has shown that the following criteria normally enhance the success of compacted clay liner (Rowe *et al*., 1996):

(1) Field compaction should normally be wet of standard Proctor optimum, and probably close to the plastic limit. This should produce a clayey barrier soft enough to self-heal on stress application.

(2) Compaction should normally be by excessive passes of pad-foot or club-foot rollers so that the interclod voids are destroyed.

(3) Most borrow pits contain stratified materials, so careful control may be required to obtained a uniform field distribution of minimum hydraulic conductivity (k) values.

(4) Even homogeneous clays are often variably weathered so that successive lifts have different optimum water content, dry density (max), and k-values.

(5) Liners should be immediately covered , and kept moist enough to prevent shrinkage until control k-testing permits waste disposal.

Similarly, smectitic liner components must not be allowed to free-swell.

(6) Desiccated liner sections (dry weather compaction) should be removed, pulverized, rewetted and recompacted.

(7) Liner sections must be protected from adverse climatic changes such as frost action and ice lensing, and if damaged should be replaced.

(8) Wet-of-optimum compaction that produces minimum k-values, produces clays wet enough to experience chemical shrinkage on exposure to certain leachates. Clay-leachate compatibility testing is therefore necessary for soils for which no data exist or whose clay mineralogy indicates reactive mineral constituents.

(9) A major benefit of wet-of-optimum compaction is that the pre-consolidation pressure of inactive clayey soils is 100-150kPa (Rowe *et al.*, 1996). Thus, if the amount of effective

stress applied by the mounded waste exceeds about 200kPa, the wet clay liner will consolidate. The resultant decrease in void ratio should enable the wet clay to self-heal even in the presence of deleterious chemicals.

(10) The above points apply to inactive clayey barriers and possibly to calcium smectites. The problems associated with high swell Na^+ bentonites require special treatment.

3.3.2 **Effect of compaction and scale dependent on hydraulic conductivity**

Generally, a criterion used to control construction should ensure compaction wet of the line of optimums. This condition results in remolding of clods, elimination of air-filled voids, low hydraulic conductivity, and comparable hydraulic conductivity at field and laboratory scales (Benson and Boutwell, 1992). Specimens used for tests in the laboratory, however, can be too small to adequately capture features controlling flow at field scale, especially if construction defects exist. As a result, laboratory-scale hydraulic conductivity, and field-scale hydraulic conductivity can differ significantly.

Discrepancies between laboratory and field-scale hydraulic conductivity are largest when hydraulic defects are present. Conditions resulting in defects include inadequate compactive effort, water content that is too low, poor remolding of clods, deterioration caused by environmental stresses such as desiccation and freeze-thaw, and inadequate compaction criteria or control (Benson and Boutwell, 1992). By carefully controlling compaction during construction, it is possible to get liners with low field-scale hydraulic conductivities, which are comparable to results of small-scale laboratory tests. The common compaction control criteria can permit compaction dry of the line of optimums which can result in macroscopic hydraulic defects and scale-dependent hydraulic conductivities. However modern compaction criteria based on degree of saturation or the line of optimums generally exclude compaction conditions under which macroscopic defects exist. When

modern criteria are applied, the field and laboratory-measured hydraulic conductivities typically agree within the normal scarter associated with hydraulic conductivity testing.

The sensitivity of hydraulic conductivity to water content and compactive effort has long been reported by many researchers (Lambe, 1954; Mitchell *et al.*, 1965; Benson and Daniel, 1990). For water contents in excess of the line of optimums, low hydraulic conductivities can be achieved. For similar water content, lower hydraulic conductivities can be achieved by increasing the compactive effort. The degree of saturation combines the effects of water content and compactive effort in a single parameter. Hence, as the water content or compactive effort is increased, the degree of saturation increases, which accounts for lower hydraulic conductivities at higher degrees of saturation.

Furthermore, the change in hydraulic conductivity occurring near the line of optimums is caused by a change in the distribution of pore sizes. Clay compacted dry of optimum generally has large macroscopic pores, and hence, higher hydraulic conductivity (Benson and Daniel, 1990). As the water content is increased, the clods of clay become softer and are more easily remolded into a dense mass devoid of large pores (Benson and Daniel, 1990; Shackelford and Javed, 1991; Haug and Wong, 1992). Wet of the line of optimums, most if not all of the large macroscopic pores are eliminated. The remaining micro-scale pores provide a tortuous pathway for flow and hence the hydraulic conductivity is low.

Differences between laboratory and field-scale hydraulic conductivities exist when the pores controlling flow at field-scale are inadequately represented in laboratory specimens (Benson and Hardianto, 1992). When a clay liner is compacted in a manner that does not eliminate macroscopic interclod voids (e.g. at low degree of saturation), the pores in specimens are more representative of small pores in the matrix of a clod instead of the larger interclod voids conducting flow at field-scale. Because the interclod voids are not

represented in the specimen, its hydraulic conductivity is lower than the hydraulic conductivity at field-scale. Conversely, when clay is compacted in a manner that eliminates interclod voids (e.g at high degree of saturation), the pores conducting flow at field-scale are adequately represented.

As a result, the hydraulic conductivity measured in the laboratory is essentially the same as the hydraulic conductivity at field scale. Furthermore, investigations have shown that the quantity and size of air-filled voids in compacted clay decreases as the molding water content, degree of saturation, and dry unit weight increase (Benson and Daniel, 1990). Because these conditions result in smaller more tortuous pores, the hydraulic conductivity also decreases. An index of the quantity of air-filled voids is the air porosity defined as the volume of voids filled with air divided by the total volume of voids.

For compaction wet of the line of optimums, the air porosity is low (Benson and Boutwell, 1992); the remaining pores are generally small, and few are with air. In contrast, compaction dry of the line of optimums results in a greater quantity of large air-filled pores and consequently higher air porosity. The clay index, which is the product of the plasticity index and percentage of fines, is a good measure of the quantity and type of hydrated fine-grained particles in a soil, and hence, should be related to hydraulic conductivity. Normally the clay index increases as the percentage of fines increases or as the clay components become more active (Benson and Boutwell, 1992).

3.3.3 **Waste-soil interactions that affect hydraulic conductivity**

Significant increases in hydraulic conductivity may result from flocculation of clay particles due to interactions with electrolyte solutions, shrinkage of the soil matrix in the presence of concentrated organic solvent, and acid-base dissolution of the soil (Shackelford, 1994). Observed effects are greater in rigid-wall permeameters than flexible-wall

permeameters. Since low hydraulic conductivity values are associated only with clay soils, only soils with significant clay content are considered for use as waste containment barriers.

However, the same properties which make clay soils desirable from the standpoint of the ability to achieve relatively low hydraulic conductivity values also typically make the same clay soils more susceptible to adverse interactions with waste liquids. Therefore, evaluations of the effects of waste liquids on soil hydraulic conductivity usually are made with respect to clay soils and, in particular, the clay content of clay soils. The chemical properties of waste liquid which are normally measured include; the concentration and types of chemicals present in the liquid, the density and viscosity of the liquid, the pH and specific (or electrical) conductance (EC) of aqueous liquids, and the polarity, solubility, and dielectric constant of organic compounds (Shackelford, 1994). Other parameters such as the electron potential (Eh), total dissolved solids (TDS), total organic carbon (TOC), chemical oxygen demand (COD), and biological oxygen demand (BOD), also may be important. The concentration and type of solutes in the pore fluid play an important role in determining the soil fabric.

Researchers like Bowders (1985), Bowders and Daniel (1986), and Bowders *et al*. (1986) performed Atterberg limits tests and hydraulic conductivity tests with various aqueous concentrations of methanol (0, 20,40,60,80 and 100% methanol by volume) and a kaolinite soil, and found that increases in both the liquid limit, plastic limit, and hydraulic conductivity all occurred only after the methanol concentration was greater than 80%. Daniel *et al*. (1988) also found no significant differences in the Atterberg limits or hydraulic conductivity of three natural soils when dilute organic liquids were used in the tests.

Acar *et al*. (1985), and Acar and Oliveri (1990) performed liquid limit and plastic limit tests with three soils; a kaolinite, Na-montmorillonite, and Ca-montmorillonite, and eleven pure organic compounds with various dielectric constants and water. The response in

the liquid limit of the kaolinite soil as a function of dielectric constant was variable. However, the response of the montmorillonite soils indicated an increase in liquid limit with an increase in dielectric constant, a trend opposite to that observed by Bowders (1985), Bowders and Daniel (1986), and Bowders *et al*. (1986) for their kaolinite soil, but consistent with the trends observed by Foreman and Daniel (1984) for the liquid limits of smectitic and illitic soils with methanol relative to water. Acar *et al*. (1985) concluded that changes in the liquid limit of kaolinite were consistent with the small changes in the intrinsic permeability of the kaolinite.

However, Acar and Oliveri (1990) concluded that significant decreases in the liquid limit if a high activity soil (e.g. montmorillonite) resulting from the use of an organic compound with a low dielectric constant (i.e relative to water) provides an indirect and conservative indication that the soil would be expected to shrink with a concomitant increase in hydraulic conductivity upon permeation with the organic liquid. Also, Lo *et al*. (1994) reported that the hydraulic conductivity of a manufactured bentonite liner material permeated first with pure methanol is significantly greater that the same material permeated first with water followed by permeation with methanol.

Mitchell and Madsen (1987) indicated three levels of soil fabric that are important for describing the flows of waste liquids through clay soils. The microfabric, or smallest level of fabric, refers to the arrangement of individual particles into small aggregations. Lambe (1958) further defined three types of microfabric. Flocculated microfabrics are characterized by large micropores relative to dispersed microfabrics and, therefore, relatively larger hydraulic conductivities are associated with flocculated microfabrics. The minifabric is an intermediate scale of fabric representing the arrangements of several microfabric assemblages.

The pores between microfabric assemblages, or inter-assemblage pores, are larger than micropores and, therefore, allow for much greater flow rates. The largest scale of fabric is the macrofabric consisting of large assemblages of both microfabric and minifabric assemblages, sometimes referred to as clods of soil. The transassemblages pores between clods, often referred to as macropores, are significantly greater than interassemblage pores in minifabric. Since liquids follow paths of least resistance, the flow rate through clay soils is dominated by the existence of macropores. Therefore, the objective in construction of clay soil barriers for waste contaminant is to destroy the clods and the associated macropores, thereby minimizing the hydraulic conductivity of the clay soil (Benson and Daniel 1990; Shackelford and Javed, 1991).

However, even if the clods are destroyed during construction, post construction effects like desiccation and freeze-thaw can result in the formation of large macropores, such as cracks and fissures, and failure of the barrier material (Shackelford, 1994). Based on these considerations, two aspects of soil fabric with respect to the influence of wastes-soil interactions are of prime importance: one, the influence of the waste liquid on the microfabric of clay soils; and the other is the influence of the waste liquid on determining the volume changes of the soil matrix. Acids and bases are known to dissolve clay minerals. The solubility of a clay mineral depends on the nature and concentration of the acid (or base), the ratio of the amount of acid (or base) to clay, and the temperature and duration of the treatment (Grim, 1953). The relative dissolution of clay minerals to acids and bases typically is based on measurement of aluminum and silica, respectively, before and after treatment with the acid or base.

The solubility relationship between clay minerals tends to increase, in part, with a decrease in particle size (increase in exposed surface area). Grim (1953) reports results of a study in which 3, 11, and from 33 to 89% of the aluminum present in kaolinite, illite, and

montmorillonite, respectively, was released when the clays were immersed in sulphuric acid. For bases, Grim (1953) reports results in which various amounts of silica (SiO_2) were released from montmorillonite and attapulgite clays after exposure to sodium carbonate (Na_2CO_3) concentrations ranging from 0 to 1% (by weight). In addition to dissolution of aluminosilicate minerals, the dissolution of carbonates ($CaCO_3$, $CaMg(CO_3)_2$), and sulphates (e.g. $CaSO_4.2H_2O$) in the presence of acid [low pH] environments is well recognized (Freeze and Cherry, 1979). The dissolution of these minerals may be represented by the following reactions (Shackelford, 1994):

$$\left.\begin{array}{l} CaCO_3 \rightarrow Ca^{2+} + CO_3^{2-} \\ CaMg(CO_3)_2 \rightarrow Ca^{2+} + Mg^{2+} + 2CO_3^{2-} \\ CaSO_4.2H_2O \rightarrow Ca^{2+} + SO_4^{2-} + 2H_2O \end{array}\right\} \text{------ (3.15)}$$

Several studies have evaluated the influence of acidic and or basic permeants on the hydraulic conductivity of clay soils (D'Appolonia, 1980; Gordon and Forrest, 19981; Peterson and Gee, 1985). Interactions with inorganic acidic permeants typically are explained in three ways. First, a low pH environment promotes flocculation and, therefore, an increase in hydraulic conductivity. For example, Gibson (1985) reports that the hydraulic conductivity of mixtures of silty sand with 7.5, 10, and 15% bentonite increased by seven times, thirteen times, and forty one times respectively after permeation with an acid liquor.

D'Appolonia (1980) notes that in sand-bentonite mixtures with low bentonite contents (< 3%), a low permeant pH may lead to smaller clay particles which are then flushed or piped from the soil resulting in larger pore channels and an increase in the hydraulic conductivity. However, a dispersion of clay particles would occur only in the case where tendency towards flocculation due to a low pH is offset by a greater tendency towards dispersion due to high concentrations of monovalent cations (e.g. Na^+) in solution. Also, Relyea and Martin (1982) describe this same process in terms of a decrease in hydraulic conductivity resulting from swelling of the clay particles in a soil-bentonite mixture.

However, the soil used by Relyea and Martin (1982) was a natural clay soil, and the bentonite content was 10% much greater than the 3% limit proposed by D'Appolonia (1980).

A significant change in hydraulic conductivity also may result when a strong acid or base disintegrates the soil into small fragments or dissolves the clay mineral into solution resulting in larger pore channels (D'Appolonia, 1980). If the fragments are flushed or piped from the system (e.g. permeameter) the hydraulic conductivity of the soil increases. Furthermore, D'Applolonia (1980) notes that extremely basic solutions usually produce a greater increase in hydraulic conductivity relative to extremely acid solutions since amorphous silica is quite soluble in highly alkaline solutions.

Another possible mechanism for acid-soil interactions affecting soil hydraulic conductivity results from initial dissolution of carbonates and oxides in the soil followed by neutralization (buffering) of the pH of the liquid. As the pH increases, the dissolved constituents in the liquid re-precipitate and the clay particles disperse (higher pH) resulting in an initial decrease in the hydraulic conductivity (Gordon and Forrest, 1981; Gibson, 1985; Peterson and Gee, 1985). For example, considering the neutralization process for calcite ($CaCO_3$), initial dissolution of calcite in accordance with the equilibrium reaction in eq. (3.15) results in aqueous phase distribution of Ca^{2+}, CO_3^{2-}, and $CaCO_3$ in accordance with the following equilibrium reaction (Shackelford, 1994):

$$Ca^{2+} + CO_3^{2-} \leftrightarrow CaCO_3 \quad \text{---------- (3.16)}$$

For a low pH environment, a reaction between the $CaCO_3$ and cations (H+) results as follows:

$$2CaCO_3 + 3H^+ \rightarrow 2Ca^{2+} + HCO_3 + H_2CO_3 \quad \text{---------- (3.17)}$$

The reaction in eq. (3.17) represents a neutralization reaction (Peterson and Gee, 1985). Furthermore, Relyea and Martin (1982) describe the neutralization reactions occurring

during permeation of natural clay soils with acidic permeants, in which the protons in the reaction in eq. (3.17) are derived from sulphuric acid (H_2SO_4), or:

$$H_2SO_4 \leftrightarrow H^+ + SO_4^{2-} \text{ ---------- (3.18)}$$

As the pH rises due to the neutralization reaction (eq. (3.17)) the free sulphate (SO_4^{2-}) reacts with the free calcium (Ca^{2+}) and sulphuric acid reacts with calcium carbonate to precipitate as calcium sulphate ($CaSO_4$), or:

$$\left.\begin{array}{l} Ca^{2+} + SO_4^{2-} \rightarrow CaSO_4 \\ Ca\,CO_3 + H_2SO_4 \rightarrow CaSO_4 + CO_2 + H_2O \end{array}\right\} \text{ ----- (3.19)}$$

Gordon and Forrest (1981) believed the precipitation of $CaSO_4$ was indicated by (1) a slight weight gain in the soil, (2) decrease in effluent electrical conductance (EC), and (3) an increase in effluent pH. In addition, Gordon and Forrest (1981) attributed an increase in volumetric outflow relative to volumetric inflow in the hydraulic conductivity tests to the production of water related to the second reaction in eq. (3.19). Gibson (1985) describes similar waste-soil interactions in which phosphoric acid (H_3PO_4) is the source of the cations for the buffering of acid permeants in the presence of carbonates in clay soils.

There have been only a few studies on the effects of bases on the hydraulic conductivity of clay soils. Gordon and Forrest (1981) permeated sedimented mud slimes with sodium carbonate (Na_2CO_3; pH = 8 to 9) solutions and noted that the stronger solution (14% Na_2CO_3) resulted in a smaller hydraulic conductivity than the weaker solution (1% Na_2CO_3). Therefore, they attributed the difference to chemical reactions in the pore liquid instead of soil structure effects. Lentz *et al.* (1985) attributed a thirteen times reduction in the hydraulic conductivity of a magnesium montmorillonite upon permeation with a sodium hydroxide in

much the same way as described earlier for the acid permeants. Oxidation-reduction (redox) reactions also play a significant role in determining the pH of a pore fluid and, therefore, potentially can affect the hydraulic conductivity of the soil in much the same way as acid-base reactions. For example, in groundwater systems, organic matter (CH_2O) is oxidized in the presence of bacteria and oxygen as follows (Freeze and Cherry, 1979):

$$O_2 + CH_2O \leftrightarrow CO_2 + H_2O \text{ ---------- (3.20)}$$

Further reactions between the carbon dioxide (CO_2) formed and water (H_2O) can result in the formation of carbonic acid (H_2CO_3), or

$$CO_2 + H_2O \leftrightarrow H_2CO_3 \text{ ---------- (3.21)}$$

which is an acid of considerable strength in soils.

Furthermore, the results of several studies in which modified Atterberg limit tests were performed (Shackelford, 1994) indicate that significant changes in the Atterberg limits of clay soils are to be expected only in the case where pure organic solvents are used in the tests. In addition, test results indicate that only pure organic solvents tend to decrease significantly the liquid limit and the plasticity of highly active clays, such as montmorillonite clays, but do not affect the Atterberg limits to the same extent for less active clays, such as kaolins. Shackelford (1994) further suggested that three mechanisms might have contributed to the observed increase in hydraulic conductivity of the clay soils upon permeation with acid permeants: (1) flocculation of the clay, (2) dissolution of the clay minerals (aluminosilicates), and (3) dissolution of other minerals (e.g. $CaCO_3$) in the clay soil. The dissolution and piping of clay minerals lead to increases in hydraulic conductivity. Shackelford (1994) suggested that the dissolution of carbonates initially led to buffering, re-precipitation, pore clogging, and a decrease in hydraulic conductivity. Depletion of the buffering capacity led to a decrease in pH, dissolution of constituents, and a possible increase in hydraulic conductivity.

3.4 **Contaminant Transport by Diffusive Mechanism**

There are essentially four different types of diffusion coefficients (Shackelford, 1989): (1) self-diffusion, (2) tracer diffusion, (3) salt diffusion, and (4) counter or interdiffusion. The type of diffusion coefficient that is measured depends on the system used to measure it. The four systems that describe the four types of diffusion coefficients are schematically represented in Fig. 3.1 in which sodium chloride (NaCl) or potassium chloride (KCl) or both is assumed to be the diffusing compound.

In self-diffusion (see Fig. 3.1 (a)), each half-cell of the system initially contains an equal concentration of NaCl. However, in one half-cell, a small amount of the sodium (Na^+) ions are not tied to that of the ions of the opposite sign (Cl^-), and the tracer ions may be considered to be moving relative to a stationary background of nondiffusing ions. This movement of the tracer ions is termed self-diffusion, and the diffusion coefficient describing it is called the self-diffusion coefficient.

Tracer diffusion is similar to self-diffusion in all respects except that the isotopic species is of a different element. For example, consider a system of two half-cells, each containing equal concentrations of NaCl. If a small amount of the Na^+ in one of the half-cells is replaced by an equal amount of a radioisotope of a different element (e.g potassium isotope-42), and the two half-cells are connected, the diffusion of the potassium isotope-42 may be traced (see Fig. 3.1(b)). In this case, the diffusion of the potassium isotope-42 is termed tracer diffusion. At infinite dilution, both tracer and self-diffusion coefficients are the same.

Salt diffusion is shown in Fig. 3.1 (c). In this case, one half-cell contains only the solvent. When diffusion is allowed, both the Na^+ and Cl^- diffuse in the same direction. Counter-diffusion or interdiffusion describes the process whereby different ions are diffusing against, or in opposite directions to each other as shown in Fig. 3.1 (d). This same process

applies to any system in which concentration gradients are established in opposite directions. In reality, both self-and tracer diffusion are limiting cases of counterdiffusion, while salt diffusion and counterdiffusion usually occur simultaneously in most systems.

3.4.1 **Fick's first law (free solution)**

In free solutions (i.e. no porous matrix), diffusion is described mathematically by Fick's first law (Robinson and Stokes, 1959):

$$J_D = - D^o \text{ grad } C \quad \text{---------- (3.22)}$$

where J_D is the diffusive mass flux or the rate of transport of chemical species per unit cross-sectional area; D^o is the free-solution diffusion coefficient of the chemical species; C is the concentration of the chemical species; grad is the gradient or vector differential operation. The following points can be made about eq. (3.22): (a) in general, the mass flux is a vector quantity, (b) the equation applies to steady-state transport, and (c) the free solution diffusion coefficient is assumed to be constant, which is a valid assumption only for relatively dilute solutions. If diffusion is assumed to occur in one direction, say the x-direction, then eq. (3.22) can be reduced to (Shackelford, 1989):

$$J_D = - D^o \frac{dc}{dx} \quad \text{---------- (3.23)}$$

The diffusive mass flux has been changed from a vector to a scalar because the direction of transport has been fixed.

3.4.2 **Fundamental basis for diffusive transport**

Under ideal conditions (i.e. microscopic scale, infinite dilution), the absolute mobility of a particle may be defined as the limiting velocity attained under a unit force (Robinson and

Stokes, 1959). On this basis, and considering the postulates of irreversible thermodynamics, another expression for diffusion can be written as (Shackelford, 1989):

$$J_D = \frac{URT}{N}\frac{dc}{dx} \qquad \text{---------- (3.24)}$$

where

$$D_o = \frac{URT}{N} \qquad \text{---------- (3.25)}$$

$$= uKT \qquad \text{---------- (3.26)}$$

R is the universal gas constant (8.134 J $mole^{-1}K^{-1}$); T is absolute temperature (in Kelvins); N is the Avogadro's number (6.022 x 10^{23} $mole^{-1}$); u is the absolute mobility of the particle; K is Bolzman's constant (R/N).

D^o can still be expressed in other forms like (Shackelford, 1989),

$$D^o = \frac{RT\lambda^o}{F^2 |Z|} \qquad \text{---------- (3.27)}$$

and

$$D^o = \frac{RT}{6\pi N\eta r} \qquad \text{---------- (3.28)}$$

where F is the Faraday (96, 490 coulombs); $|Z|$ is the equivalents, or charge, per mole of chemical species; λ^o is the limiting ionic equivalent conductivity; η is the absolute viscosity of the medium; r is the hydrated radius of the ion or molecule.

Eqs. (3.27) and (3.28) are referred to as the Nernst and the Einstein-Stokes equations, respectively. The D^o values obtained using eqs. (3.27) or (3.28) should be considered to be maximum values attainable under ideal conditions. However, under nonideal conditions (macroscopic scale, concentrated solutions), several effects which are negligible for ideal conditions become important. For convenience, the effects associated with nonideal diffusion

can be separated into three groups: (1) those due to the requirements for electroneutrality, (2) those due to concentrated, nondilute solutions or electrolytes, and (3) those due to solution speciation or electrolyte strength (Shackelford, 1989).

(1) **Effect of electroneutrality:**

When two oppositely charged ions are diffusing in the same direction in solution (salt diffusion), a microscopic charge separation or electrical potential gradient is established between the ions because of their different mobilities. The effect of this charge separation is to speed up the slower-moving ion and to slow down faster-moving ion. Because, on a macroscopic scale, electroneutrality must be satisfied, the resultant speeds of both ions must be equal (Robinson and Stokes, 1959).

As a result, each ion of an electrolytic solution is moving under the influence of not only the force due to the gradient in the chemical potential for that ionic species but also the force due to the gradient in the electrical potential between the motion of the oppositely charged particles. Therefore, D^o must be re-defined to account for the additional force. This modification has been made for the specialized case of a single electrolyte, one molecule of which gives v_1 cations of algebraic valency Z_1 and V_2 anions of algebraic valency z_2 (Robinson and Stokes, 1959). The theoretical development for this specialized case results in the following forms for the Nernst-Einstein expression (eqs. (3.29 a) and (3.29 b)) and for the Nernst expression (eqs. (3.30) and (3.31)):

$$D^o_{12} = \frac{u_1 u_2 (v_1 + v_2) RT}{(v_1 u_2 + v_2 u_1) N} \qquad \text{---------- (3.29 a)}$$

$$= \frac{D^o_1 D^o_2 (v_1 + v_2)}{v_1 u_2 + v_2 u_1} \qquad \text{---------- (3.29 b)}$$

$$D^o_{12} = \frac{\lambda^\circ_1 \lambda^\circ_2 (v_1 + v_2) RT}{}$$

$$(\lambda^{\circ}_1 \lambda^{\circ}_2)v_1 \,|Z_1|\, F^2$$

$$= \frac{\lambda^{\circ}_1 \lambda^{\circ}_2 (v_1 + v_2)}{(\lambda^{\circ}_1 \lambda^{\circ}_2)v_2 \,|Z_2|\, F^2} RT \qquad \text{---------- (3.30)}$$

$$D^{o}_{12} = \frac{\lambda^{\circ}_1 \lambda^{\circ}_2 (|Z_1| + |Z_2|)}{(\lambda^{\circ}_{1+}\lambda^{\circ}_2)\,|Z_1 Z_2|\, F^2} RT \qquad \text{---------- (3.31)}$$

where D^{o}_{12} is the free-solution diffusion coefficient of the single electrolyte diffusing in one direction at infinite dilution; D^{o}_1 and D^{o}_2 are respective self-diffusion or tracer diffusion coefficients of the cation and the anion at infinite dilution; λ^{o}_1, λ^{o}_1 are respective limiting ionic conductances.

Another similar equation but for the case where two different ions with the same valences and concentration gradient diffusing in opposite directions, i.e. counterdiffusion, is (Shackelford, 1989):

$$D^{o}_{12} = \frac{(C_1 + C_2)\, D^{o}_1 D^{o}_2}{D^{o}_1 C_1 + D^{o}_2 C_2} \qquad \text{---------- (3.32)}$$

If the two ions have different valences, eq. (3.32) is modified as follows (Shackelford, 1989):

$$D^{o}_{12} = \frac{(|Z_1| n_1 + |Z_2| n_2)\, D^{o}_1 D^{o}_2}{|Z_1| n_1 D^{o}_1 + |Z_2| n_2 D^{o}_2} \qquad \text{---------- (3.33)}$$

where n_1 and n_2 are the concentrations of the respective ions expressed as normalities (i.e equivalents per litre of solution).

General equations can be derived but not necessarily solved for cases where more than two ions are present because there is an infinite number of ways to satisfy the electroneutality condition in such cases (Robinson and Stokes, 1959).

(2) **Effects of concentration:** As the concentration of the chemical species increases, solute-solute and solute-solvent interactions become more significant. The overall effect is that the free-solution diffusion coefficient is a function of concentration though there is no general trend (Shackelford, 1989).

(3) **Effects of speciation or electrolyte strength:**

Electrolytes can be classified into two broad groups **(**Robinson and Stokes, 1959): one, nonassociated, completely dissociated, or strong electrolytes; and two, associated, incompletely dissociated, or weak electrolytes. Generally, the strength of the electrolyte theoretically can affect the value of the free-solution diffusion coefficient.

(4) **Effect of pressure:**

Changes in pressure also can affect the value of the free-solution diffusion coefficient. However, if water is the solvent, the pressure dependence is small (Shackelford, 1989). In most cases, the pressure dependence of the diffusion coefficient can be neglected.

3.4.3 Fick's second law (free solutions)

When Fick's first law (eq. (3.22)) is combined with the conservation of mass, the result is Fick's second law (Robinson and Stokes, 1959):

$$\frac{dc}{dt} = \text{div}\,(D^o \,\text{grad}\, c) \quad \text{---------- (3.34)}$$

where div is the divergence operator; t is time.

Eq. (3.34) applies to transient diffusion of an ionic species at infinite dilution in three-dimensional space. For diffusion in one direction, i.e. the x- direction, eq. (3.34) becomes

$$\frac{dc}{dt} = \frac{d}{dx}\left(D^o \frac{dc}{dx}\right) \quad \text{---------- (3.35)}$$

If the free-solution diffusion coefficient is assumed to be independent of the distance of transport, eq.(3.35) can be reduced further to

$$\frac{dc}{dt} = (D^o \frac{d^2c}{dx^2}) \quad \text{---------- (3.36)}$$

Generally, eqs. (3.35) and (3.36) are the forms of Fick's second law that are often used in the study of the transient diffusion of solute species in free solutions.

3.4.4 **Diffusion in soil**

Diffusion in soil necessarily is slower than diffusion in free solution for three reasons (Shackelford, 1989):

(1) There is a reduced cross-sectional area of flow,

(2) There is a more tortuous diffusion pathway because of the particulate nature of the porous medium, and

(3) There may be reduced mobility of the diffusing chemical species in the soil pores because of interactions with the pore walls.

For the transport of contaminants in soil, Fick's first and second laws have to be modified as follows:

(1) **Effect of reduction in cross-sectional area of flow:**

Under steady-state conditions when the concentration distribution is a linear function of distance, the differential operators in eq. (3.23) may be replaced by difference operators, or (Shackelford, 1989),

$$J_D = - D^o \square \frac{\Delta c}{\Delta x} \quad \text{---------- (3.37)}$$

Because of the reduced cross-sectional area in the soil, the concentration of the diffusing species, c, is the concentration in the liquid phase of the pore space. For diffusion in soil, eq. (3.37) must be modified as follows:

$$J_D = - D^o \theta \frac{\Delta c}{\Delta x} \qquad \text{----------} \quad (3.38)$$

where θ is the volumetric water content defined as

$$\theta = nS_r \qquad \text{----------} \quad (3.39)$$

where n is the porosity; S_r is the degree of saturation.

Substituting eq. (3.39) into eq. (3.38), Fick's first law for diffusion in soil becomes,

$$J_D = - D^o nS_r \frac{\Delta c}{\Delta x} \qquad \text{----------} \quad (3.40)$$

Therefore, the most conservative case (i.e. maximum flux for liquid phase diffusion) will occur when the soil is saturated ($S_r = 1.0$), all other conditions being equal.

(2) **Effect of tortuous pathway:**

There are two effects due to the more tortuous pathway associated with diffusion through soils (Shackelford, 1989); (a) the cross-sectional area perpendicular to the actual microscopic flow direction in the soil is less than that perpendicular to the macroscopic flow direction defined by the x- coordinate system, and (b) the driving force (i.e. concentration gradient) is also less because of the longer, or actual, length of flow, L_e, versus the macroscopically measured length of flow, L. For these effects, eq. (3.40) must be modified as follows (Shackelford, 1989):

$$J_D = - D^o n^1 S_r \frac{\Delta c}{\Delta x^1} \qquad \text{----------} \quad (3.41)$$

where n^1 and Δx^1 are the actual (microscopic) porosity and distance of flow respectively. Because macroscopic qualities are desired for the measurement of diffusion coefficients, the macroscopic equivalents for n^1 and Δx^1 may be substituted into eq. (3.41).

$$J_D = - D^o \left(\frac{L}{Le}\right)^n S_r \left(\frac{Le}{L}\right) \frac{\Delta c}{\Delta x} \quad \text{---------- (3.42 a)}$$

$$= - D^o \left(\frac{L}{Le}\right)^2 n S_r \frac{\Delta c}{\Delta x} \quad \text{---------- (3.42 b)}$$

The tortuosity factor is usually represented by τ:

$$\tau = \left(\frac{L}{Le}\right)^2 \quad \text{---------- (3.43)}$$

When eqs. (3.39), (3.42), and (3.43) are combined, Fick's first law for diffusion in soil becomes (Shackelford, 1989),

$$J_D = - D^o \tau \theta \frac{\Delta c}{\Delta x} \quad \text{---------- (3.44)}$$

(3) **Effect of effective diffusion coefficient:**

At present, tortuosity factors cannot be measured independently. This obstacle is surmounted by defining an effective diffusion coefficient, D^*, where

$$D^* = D^o \tau \quad \text{---------- (3.45)}$$

When eq. (3.45) is substituted into eq. (3.44), Fick's first law for diffusion in soil becomes,

$$J_D = - D^* \theta \frac{\Delta c}{\Delta x} \quad \text{---------- (3.46)}$$

Eq. (3.46) can be used to measure the effective diffusion coefficients of chemical species, D^*, diffusing in soil.

3.4.4 Fick's second law (diffusion in soil)

The transient form of the transport equation for diffusion in soil depends on whether the chemical species, or tracer, is reactive or nonreactive. A reactive chemical species is subject to chemical and biochemical reactions or radioactive decay, or both. When Fick's

first law (eq. (3.46)) is combined with the equation of continuity (i.e. conservation of mass), the result is Fick's second law for the transient diffusion of a nonreactive chemical species in soil (Robinson and Stokes, 1959):

$$\frac{dc}{dt} = \frac{d}{dx}\left(D^* \frac{dc}{dx}\right) \quad \text{---------- (3.47)}$$

or, if D^* is not a function of the transport distance,

$$\frac{dc}{dt} = D^* \frac{d^2c}{dx^2} \quad \text{---------- (3.48)}$$

Of the numerous types of chemical or biochemical reactions that can affect contaminant concentrations during transport in soil, only adsorption-desorption (sorption) reactions are routinely modelled. The other reactions typically are accounted for by indirect, qualitative analyses. For reactive solutes subject to reversible sorption reactions during diffusive transport in soil, eq. (3.48) should be modified as follows (Shackelford, 1989):

$$\frac{dc}{dt} = \frac{D^*}{R_d} \frac{d^2c}{dx^2} \quad \text{---------- (3.49)}$$

where the retardation factor, R_d, is given by the following equation:

$$R_d = 1 + \frac{\rho_b K_p}{\theta} \quad \text{---------- (3.50)}$$

where ρ_b is the dry (bulk) density of the soil; K_p is the partition coefficient (also known as distribution coefficient). The partition coefficient relates the mass of solute sorbed per mass of soil, q, to the concentration of the solute in solution, c, at equilibrium. The relationship between q and c, known as an adsorption isotherm, is often determined for an appropriate range of concentrations form the results of batch equilibrium tests (Shackelford, 1989).

Three general shapes for adsorption isotherms resulting from batch-equilibrium tests are normally used in the determination of the partition coefficient. When the q versus c relationship is constant, the adsorption isotherm is linear, and K_p is termed the distribution

coefficient, K_p. Otherwise, K_p is a function of the equilibrium concentration in soil. Since the effective diffusion coefficient in eq. (3.49) is divided by the retardation factor, the rate of diffusive transport of a chemical species undergoing reversible sorption reactions is inversely proportional to the value of the partition coefficient.

When the adsorption isotherm is nonlinear, the resulting differential equation describing solute transport in soil with simultaneous adsorption (eq. (3.49)) is nonlinear, for which no general analytical solutions exist. However, analytical solutions do exist when the adsorption behaviour is linear. The existence of analytical solutions makes the assumption of linear adsorption desirable. For this reason, linear adsorption isotherms are often assumed to apply when, in fact, experimental data suggest nonlinear adsorption behaviour.

3.4.5 **Apparent diffusion coefficients**

Some researchers have found it convenient to rewrite eq. (2.49) as follows (Shackelford, 1989):

$$\frac{dc}{dt} = D_A^* \frac{d^2c}{dx^2} \quad \text{---------- (3.51)}$$

where D_A^* is an apparent diffusion coefficient, also known as the effective diffusion coefficient of the relative solute, D, or

$$D_A^* = D_S = \frac{D^*}{R_d} \quad \text{---------- (3.52)}$$

The result of this substitution is that only one unknown (D_A^* or D_S) must be solved for, instead of two unknowns (i.e. D^*, and R_d or K_p). Then, based on some information regarding either D^* or K_p and known value of D_A^*, the other unknown parameter may be solved for by eq. (3.51). However, Rowe *et al*. (1985) and Rowe (1987) caution against the use of a single coefficient, D_A^* or D_S, in eq. (3.49) when the boundary conditions are flux-controlled, since

the resulting analyses will be both incorrect and unconservative. In addition, the coefficient D_A^* or D_S is a function of the sorption characteristics of the soil, whereas D^* is not. Therefore, it is meaningless to report a value for D_A^*or D_S without reporting the associated R_d value, since different soils have different sorption characteristics.

3.4.6 **Diffusion through soil barriers**

The diffusion of contaminants through barriers can be generally categorized into four; (1) Transport mechanisms, (2) Diffusion without advection, (3) Diffusion with positive advection, and (4) Diffusion with negative advection (Shackelford, 1989).

(1) **Transport Mechanisms:**

Three transport mechanisms (advection, mechanical dispersion, and molecular diffusion) are generally recognized as governing the flow of contaminants in porous materials. Advection is the transport of contaminants in response to a gradient in total hydraulic head. Due to advection, nonreactive contaminants are transported at an average rate equal to the seepage velocity, V_S, which, for a saturated system, has already been defined in eq. (3.1).

Both mechanical dispersion and molecular diffusion are mixing processes that cause the contaminant front to spread out or disperse during transport through the porous material. However, mechanical dispersion is a function of the seepage velocity, whereas molecular diffusion is not (i.e. the greater the seepage velocity, the greater the effect of mechanical dispersion, and vice versa). Therefore, even though all three mechanisms are active during transport, the effect of mechanical dispersion becomes less significant as the seepage velocity decreases. For the relatively low seepage velocities typically associated with fine-grained soil barriers, the effect of mechanical dispersion is negligible (Shackelford, 1989). As a result, the transport of contaminants through waste contaminant barriers generally can be described using an appropriate advective-diffusive model. The appropriate model to be used

depends on the susceptibility of the contaminant to geochemical reactions (i.e nonreactive versus reactive solutes) and the overall flow conditions. In general, three possible cases should be considered (see Fig. 3.2): (a) Diffusion without advection (pure diffusion), (b) diffusion with positive advection, and (c) diffusion with negative advection.

(2) **Diffusion without advection:**

For the case shown in Fig. 3.2 (a), the hydraulic conductivity gradient across the clay barrier is zero, so there is no advective flow (Shackelford, 1989). However, because the concentration of the contaminants in the leachate is greater than that in the natural soil, a concentration gradient is established across the barrier. As a result of this concentration gradient, diffusion of contaminants will occur through the barrier. The appropriate equations for modelling this case have been presented previously. Under steady-state conditions, the diffusive flux of contaminants is described by eq. (3.46). The transient diffusive transport of contaminants can be modelled by using eq. (3.48) for the nonreactive solutes, and eq. (3.49) for the reactive solutes.

The significance of pure diffusive transport of contaminants across clay barriers can be illustrated with the aid of an example. If the concentration of contaminants in he leachate is assumed to be constant with time, then, the concentration of the contaminant reaching the bottom of a clay barrier of thickness L can be described as a function of time, using the following analytical solution to eq. (3.48) (Crank, 1975):

$$\frac{C}{C_o} = \text{erfc}\left(\frac{L}{2(D^* t)^{1/2}} \right) \qquad \text{---------- (3.53)}$$

where C_o is the concentration of the nonreactive solute in the leachate; C is the corresponding concentration at the bottom of the barrier; erfc (x) is a complementary error function; t is the transit time (i.e. the time required for the solute to travel from the top to the bottom of the

barrier). Tables of values for the erfc (x) are given by Abramowitz and Stegun (1972), Carslaw and Jaeger (1959), Crank (1975), and Freeze and Cherry (1979), among others.

(3) **Diffusion with positive advection:**

For the case shown in Fig. 3.2 (b), a hydraulic gradient has been established across the clay barrier such that advective flow of contaminants occurs in the same direction as the diffusive flow in the pure diffusive case. The advective flow is termed positive because it results in an increase in the contaminant concentration in the natural soil. This case is commonly described in texts on contaminant transport (Bear, 1972; Freeze and Cherry, 1979). For this case, the total, steady-state flux, J, of the contaminant is given by the sum of the advective flux, J_A, and the diffusive flux, J_D, or (Shackelford, 1989):

$$J = J_A + J_D \quad \text{---------- (3.54)}$$

where $$J_A = nV_S c \quad \text{---------- (3.55)}$$

and V_S has already been defined. Since J_D is given by eq. (3.46), the total flux (assuming saturated conditions) is described by

$$J = nV_S c - D^* n \frac{dc}{dx} \quad \text{---------- (3.56)}$$

The transient model for this case, assuming a homogeneous soil, is described by the well-known advective-dispersive equation for nonreactive solutes (Bear, 1972; Freeze and Cherry, 1979):

$$\frac{dc}{dt} = D^* \frac{d^2c}{dx^2} - V_s \frac{dc}{dx} \quad \text{…… (3.57)}$$

where mechanical dispersion has been considered to be negligible. There are several analytical solutions to eq. (3.57), depending on the boundary and initial conditions (Bear,

1972). One of the more popular solutions is as follows (Freeze and Cherry, 1979; Ogata and Banks, 1961):

$$\frac{C}{Co} = \frac{1}{2}\left\{ erfc\left[\frac{x - Vst}{2(D*t)^{1/2}} \right] + \exp\left[\frac{xVs}{D*} \right] erfc\left[\frac{x + Vst}{2(D*t)^{1/2}} \right] \right\} \quad \text{.....(3.58)}$$

Eq. (3.58) applies to a semi-infinite medium in which the concentration of the contaminant in the leachate, C_o, is assumed to be constant with time. The transport of contaminants through clay barriers is a function of the magnitude of the seepage velocity. Hence, the best containment barrier that can be built for the case of positive advection is one in which diffusion is the controlling mechanism of contaminant transport. However, relatively short transit times may still result even though diffusion is the controlling mechanism of transport, especially for relatively thin barriers and or low values of C/C_o (Shackelford, 1989). For reactive solutes subject to reversible sorption reactions, eq. (3.57) is not an appropriate model but should be modified as follows (Shackelford, 1989):

$$\frac{dc}{dt} = \frac{D^*}{R_d} \frac{d^2c}{dx^2} - \frac{V_S}{R_d} \frac{dc}{dx} \qquad \text{--------- (3.59)}$$

Eq. (3.58) also represents an analytical solution to eq. (3.59) if D^* in eq. (3.58) is replaced by D^*/R_d.

(4) **Diffusion with negative advection:** In some cases, the direction of advective transport may be opposite that of diffusive transport. This situation may occur in practice, for example, when the barrier (either natural of man-made) is located over a confined aquifer under artesian pressure, or when vertical barriers, such as slurry walls, are used to isolate a contaminated area (Shackelford, 1989). In this case,

the advective flow is termed negative because it works to prevent the escape of pollutants from the contaminated area. The steady-state and transient equations describing this case can be formulated by simply substituting $-V_S$ for V_S in the equations for the previous case (Shackelford, 1989). However, the boundary and flow conditions for this case can be quite complex. For example, if the concentration, C_2, in the slurry wall scenario is initially zero, then there is no initial advective flux of contaminants into the barrier.

The initial flow of water into the contaminant area may dilute the concentration C_1 with time and, therefore, reduce the concentration gradient for outward (positive) diffusive transport. The contaminant migration front, if any, will depend on the offsetting effects of the advective and diffusive fluxes. The situation for reactive solutes is more complex. Regardless, significant diffusive transport of contaminants may still result if a relatively thin barrier is built to contain high concentrations of priority pollutions over extended periods.

3.5 **The Use of Column Testing**

Column tests are essentially compatibility tests in which the concentrations of the contaminant constituents (e.g. solutes) in the effluent solution of the column or permeameter are monitored continuously. Extensive knowledge regarding the determination of transport parameters, such as the dispersion/diffusion coefficient (D), and retardation factor (R_d), for miscible contaminant transport through porous media has been gained through laboratory column testing (Shackelford, 2000). However, the measured transport parameters from laboratory column tests generally are not considered to be representative in terms of the evaluation and or design of waste contaminant barriers, especially for the case where the column lengths are on the same order as the thickness of the containment barrier.

Though there are many models for analyzing column test results, only four analytical models are presented in this section, which are; (1) traditional column testing approach, (2)

models based on resident concentrations, (3) models based on flux concentrations, and (4) cumulative mass models. The cumulative mass approach differs from the more traditional approach in that the transport parameters are based on the measurement of the total solute mass which exits the soil column instead of the effluent concentrations. The potential advantages of this test methodology according to Shackelford (2000) are; the effect of the effluent sample volume is removed from consideration since the effluent solute mass is a more fundamental parameter than is solute concentration; and less control on the effluent sampling procedure is required since the solute is allowed to accumulate over time in the effluent reservoir. Typical examples of cells used for column tests are shown in Fig. 3.3.

The cumulative mass models, though relatively new, are derived in a sequential manner. First, concentration-based analytical models to the advection-dispersion equation which commonly are used in traditional column studies are highlighted. Second, the solute flux expressions corresponding to each respective concentration-based analytical model are derived. Third, each solute flux expression is integrated in closed form to arrive at the corresponding cumulative mass model (Shackelford, 2000).

3.5.1 **Theoretical background**

Advection-dispersion theory: For many practical applications involving transport of miscible contaminants through relatively thin low permeability soil barriers (e.g clay liners and slurry walls), the soil is assumed to be homogeneous, isotropic, and non deformable, and the contaminant migration is assumed to be governed by steady flow of an incompressible fluid. Also, coupled flow processes (e.g chemical osmosis) generally are neglected and only linearly instantaneous and reversible sorption reactions are included in the analysis. When these assumptions are acceptable, one-dimensional transient transport of chemical species

through saturated soil is described by the well-unknown advection-dispersion equation (Freeze and Cherry, 1979), or:

$$R_d \frac{dc_r}{dt} = \frac{Dd^2c_r}{dx^2} - \frac{Vdc_r}{dx} \qquad \text{---------- (3.60)}$$

where c_r is the solute concentration in the pore water of the soil, also known as the resident concentration (van Genuchten and Parker, 1984), t is time, x is the macroscopic distance in the direction of transport, v is average linear or seepage velocity, D is the dispersion/diffusion coefficient, R_d is the retardation factor. The dispersion/diffusion coefficient, D, accounts for the spreading of the contaminant during migration, with D values decreasing as the seepage velocity, V, decreases.

In the limit (i.e. when $V \rightarrow 0$), D equals the effective diffusion coefficient, D^*, and transport is governed solely by diffusion (Shackelford, 2000). The dimensionless retardation factor, R_d, represents a measure of the attenuation capacity of a given soil for a given chemical species due to linearly instantaneous and reversible equilibrium sorption of reactive solutes; i.e. the larger R_d, the greater the sorption of the chemical species. For a non- reactive (non- adsorbing) solute, R_d is one.

(1) Traditional column testing approach**:** Column tests traditionally are performed in three steps (Shackelford, 2000). First, the steady-state solvent flow is established through a column of soil of length, L. After steady state has been established, a solution containing one or more miscible contaminant species each with a known concentration, C_o, is introduced continuously into the influent side of the column. The third step involves the measurement of the effluent concentrations, C_e, of the same contaminant species as a function of time.

The measured effluent concentrations typically are normalized with respect to the influent concentration (C_e/C_o), which is maintained constant, and are plotted versus pore volumes of flow, T (vt/L), a dimensionless measure of time. The continuous distribution of

measured C_e/C_o versus T is known as a solute or effluent breakthrough curve which is then analyzed for the transport parameters D and R_d using an appropriate solution to eq. (3.60). The values for D and R_d determined from laboratory column tests can be used subsequently in analytical and or numerical models to describe or predict the migration of miscible contaminants in practical problems, such as in the design of soil barriers for waste disposal or remediation (Shackelford, 2000).

(2) Models based on resident concentrations: Four analytical solutions to eq. (3.60), as summarized in Table 3.1, are highlighted under this part. The analytical models for resident concentrations listed in Table 3.1 can be used to evaluate column effluent data by substituting x=L (i.e assuming C_r (L,t)/C_o = C_e/C_o) in the appropriate model, and regressing the theoretical solution in the measured effluent concentration data as a function of D and R_d (Shackelford, 2000). The values of D and R_d which provide the best fit to the data are considered to be the measured transport parameters. However, different values of D and R_d might be expected depending on which analytical model is utilized in the analysis due to the differences in the boundary conditions (van Genuchten and Parker, 1984).

(3) Models based on flux concentrations**:** The evaluation of effluent concentrations utilizing solutions based on resident concentrations is conceptually incorrect when the effluent is collected in a reservoir from which liquid samples are recovered on a periodic basis and analyzed for chemical concentrations of specified chemical species (van Genuchten and Parker, 1984). In addition, the earlier assumption that C_r (L,t)/C_o = C_e/C_o for semi-infinite porous media may not be appropriate. In such cases, flux-averaged or flowing effluent concentrations may be required for correct evaluation of column effluent data (van Genuchten and Parker, 1984). Flux-averaged effluent concentrations are defined as the mass flux of solute passing through the effluent end of the soil column per unit fluid flux during an elementary time interval, or (Shackelford, 2000):

$$C_e = \frac{J\,(L,t)}{q} \quad \text{---------- (3.61)}$$

where J (L,t) is the solute mass flux evaluated just inside the effluent end of the column of soil of length, L; and q is the fluid flux given by Darcy's law, or

$$q = \frac{Q}{A} = -Ki \quad \text{---------- (3.62)}$$

where Q is the volumetric flow rate; A is the total cross sectional area of the soil perpendicular to the direction of flow; k is the hydraulic conductivity of the soil; i is the hydraulic gradient (< 0).

For a saturated soil in which all of the pore space is available for transport, the transient solute flux at the effluent end of the column of soil, J (L,t), is the sum of the transient advective solute flux, J_a (L,t), and the transient dispersive/diffusive solute flux, J_d (L,t), or (Shackelford, 2000):

$$J\,(L,t) = J_a\,(L,t) + J_d\,(L,t) = qc_r\,(L,t) - nD\frac{dc_r\,(L,t)}{dx} \quad \text{---------- (3.63)}$$

where n is the total porosity of the soil. Thus, the evaluation of flux-averaged effluent concentrations in accordance with eq. (3.61) requires an evaluation of the solute flux, J (L, t), in accordance with eq. (3.63). Normalization of the effluent concentrations in eq. (3.61) with respect to the influent concentration, C_o, results in the following expression:

$$\frac{C_e}{C_o} = \frac{J\,(L,\,t)}{qC_o} = RF \quad \text{---------- (3.64)}$$

where RF is the relative solute flux defined as follows (Manassero and Shackelford, 1994):

$$RF = \frac{J\,(L,t)}{qc_o} = \frac{J_a\,(L,t) + J_d\,(L,t)}{qc_o} \quad \text{---------- (3.65)}$$

where qc_o is the steady-state solute flux (i.e. at steady-state transport, J (L,t) = qc_o and RF = 1).

Thus, the relative flux is the same as the relative flux-averaged concentration ratio utilized in column testing. The relative flux models resulting from the concentration–based models in Table 3.1 are listed in Table 3.2 in terms of the dimesionless parameters P_L and T_r, where P_L is referred to as the column Peclet number defined as (Shackelford, 2000):

$$P_L = \frac{VL}{D} \quad \text{---------- (3.66)}$$

and T_r is the pore volumes of flow divided by the retardation factor, or,

$$T_r = \frac{T}{R_d} \quad \text{---------- (3.67)}$$

A comparison of the resident concentration models in Table 3.1 with the relative flux models given in Table 3.2 reveals that model A.1 has exactly the same form as RF.2 in Table 3.2 when A.1 is written in terms of P_L and T_r. Based on this similarity and the appropriateness of the boundary conditions for A.2 upon which RF.2 is based, van Genuchten and Parker (1984) recommend the analytical form of model A.1 for use in evaluating column effluent data from finite soil columns when v > o even though model A.1 is written ostensibly for semi-infinite soil columns (Shackelford, 2000).

(4) Cumulative mass models**:** Breakthrough curves of either c_r (L, t)/c_o or C_e/C_o represent continuous distributions of an infinite number of instantaneous concentrations due to the contiuum assumption inherent in the derivation of the advection-dispersion equation (eq. (3.60)). However, measurement of instantaneous concentrations is not possible (Shackelford, 2000). In most practical situations, a sufficient number of effluent (liquid) samples of finite volume are collected to define the solution breakthrough curves. In these cases, the measured concentrations represent average concentrations over the sample collection interval since the concentration represents the accumulated solute mass divided by the amount of the effluent

sample volume. Thus, the measured concentrations are biased with respect to the effluent sample volume.

This bias can be minimized by collecting relatively small volumes of effluent at sufficient discrete and separate intervals in time to provide an adequate description of the solute breakthrough curve. As an alternative, relatively large incremental volumes of liquid can be collected, and the measured average concentrations can be plotted at the middle of the sample interval with respect to elapsed time or pore volumes of flow to provide reasonable estimates of the transport parameters (Shackelford, 2000). Nevertheless, the measured concentration in either case is affected by the volume of effluent collected resulting in the measurement of average concentrations over the collection interval of the sample.

An alternative approach based on solute mass instead of solute concentration recently has been proposed by Shackelford (2000). The steps for the effluent monitoring procedure in this approach are as follows; (1) allow the effluent to collect in a reservoir over an increment in time, Δt, or pore volume increment, ΔT; (2) determine the effluent volume, Δv_e, which has entered the reservoir during this time increment ($\Delta v_e = \Delta T.V_p$, where v_p is the pore or void volume of the soil); (3) shake the collected volume of the effluent vigorously before removing an aliquot of effluent for measurement of the average effluent concentration, C_e; and (4) determine the total solute mass in the collected volume of effluent, $m = \bar{C}_e \Delta V_e$, (Shackelford, 2000).

This procedure is repeated for each subsequent pore volume increment, and the cumulative mass values ($\sum m = \sum \bar{C}_e \Delta V_e$) are normalized with respect to the equilibrium mass in the pore fluid of the soil ($V_p C_e$) and plotted versus the corresponding elapsed time, t, or pore volumes, T, for analysis of the transport parameters. Thus, the analysis of the effluent data in the cumulative mass approach is based on the solute mass which has collected in the

effluent reservoir. As a result, the influence of the effluent sample volume is removed from consideration in the cumulative mass approach since mass is a fundamental parameter (Shackelford, 2000). Also, frequent removal of effluent is not required since the mass is allowed to accumulate in the effluent reservoir. Finally, Shackelford (2000) shows that essentially the same values of the parameters D and R_d can be determined based on a reduced number of data points using the cumulative mass approach relative to the traditional concentration approach providing a convenient and economical procedure for evaluating the transport parameters.

The cumulative mass approach is based on evaluation of the cumulative mass ratio, CMR, defined as (Shackelford, 2000):

$$CMR = \frac{\Sigma m}{v_P c_O} = \frac{A\int_0^t J(L,t)dt}{v_P c_O} = \frac{\int_0^T J(L,T)dT}{qc_O} = \frac{\int_0^T c_e(T)dT}{c_O} = \int_0^T (RF)dT \quad ....(3.68)$$

where CMR is a function of time (i.e. CMR = CMR (L,T). Thus, the CMR models are determined by integration of relative flux models. The CMR models resulting from closed-form integration of the relative flux models given in Table 3.2 are summarized in Table 3.3.

The retardation factor, $R_{d,}$ in this case is defined as (Shackelford, 2000):

$$R_d = \int_0^{T_{SS}} \left[1 - \frac{C_e(T)}{c_O}\right] dT = \int_0^{T_{SS}} dT - \int_0^{T_{SS}} \frac{C_e(t)dT}{c_O} = T_{SS} - \int_0^{T_{SS}} \frac{C_e(T)dT}{c_O} \quad \ldots(3.69)$$

where T_{ss} is the pore volumes of flow required to achieve steady-state transport. Hence, the evaluation of the retardation factor requires the determination of the area above the effluent breakthrough curve after steady-state transport conditions have been achieved (Shackelford, 2000).

3.6 **Clay Mineralogy**

Most clay minerals are sheet silicates (phyllosilicates). The single most important characteristics of the clay minerals is that they carry a net negative charge which is expressed as the number of deficient electrons/unit cell of ten oxygens with repect to charge neutrality (Rowe *et al*., 1996). If there are no fixed cations between the sheet, the negative charge deficiency correlates directly with the cation exchange capacity (CEC) which is the measure of the number of positive cations in milliequivalents to neutralize 100g of clay. The layer combinations and both the size and the location of the charge deficiency influence all aspects of engineering behaviour, including the cation exchange capacity.

The two most important elementary structural elements of the clay minerals are the silicon tetrahedron and the aluminum or magnesium octahedron. The charges present on a given clay mineral are located in either or both of these units, due to isomorphous substitution of cations for one another. Based on structural components, clay minerals can be categorized into: (a) the silicon (and aluminum) tetrahedron, (b) aluminum and magnesium octahedron, (c) tetrahedral sheets, (d) octahedral sheets, and (e) two-layer and three-layer units. When clay minerals form during weathering, the silica sheets grow simultaneously with the octahedral sheets to form two-layer and three-layer units. Under slightly acidic tropical conditions (pH = 5–6), aluminum is retained and silica is gradually depleted from the soil so that two-layer units or 1:1 clays tend to form (Rowe *et al.*, 1996). These clays consist of one silica sheet merged with one octahedral sheet. The kaolins are typical of these clays.

Under conditions of incomplete leaching (arid, flooded, etc.) three-layer units or 2:1 layer clays tend to form. These clays consist of two silica sheets sandwiching one octahedral sheet. The smectites and illites are typical of these clays. The clay minerals form very small crystals rarely exceeding 2-3 micrometer in diameter. The reason for this is the distortion of the layer structure because of the poor match between the unit cell dimensions of the tetrahedral and octahedral sheets. Halloysite, a form of tubular kaolin, is the most spectacular

example of this distortion, since it forms in curved sheets, with the gibbsite sheet inside the tetrahedral sheet. Montmorillonite forms very small curved particles, since there is not inter layer bonding between the 2:1 layer units, whereas muscovite may form large, table-sized slabs, due to a very strong, inter layer K^+ bond which resists the distortion.

3.6.1 **Important clay minerals found in soil**

The clay mineral species normally encountered in engineering practice are the kaolins, illites, smectites, vermiculites and chlorites (Rowe *et al*., 1996). Emphasis is placed on the importance of the cation exchange capacity, CEC, which is the measured abundance of exchangeable cations required to be adsorbed onto the negative clay platelets to render them neutral (measured in milli-equivalents of cations/100g of clay). These five mineral types are described in some detail as follows.

(1) **Kaolinite (and halloysite):**

This clay mineral consists of stacked 1:1 layer units which have a c-axis lattice spacing, d, of 0.7nm. There is very little charge on this mineral. Its formula consists of neutral octahedral layers and very slightly charged tetrahedral layers, probably formed by isomorphous substitution of Si^{4+} by Al^{3+} (Rowe et al., 1996). As a result, very few cations are adsorbed onto this clay, and the cation exchange capacity (CEC) is very low at about 5meq/100g.

The bonding between the sheets is a fairly strong hydrogen ion bond. The H^+ is supplied by the (OH^-) forming the gibbsite sheet, and tends to co-binds with O^{2-} on the surface of the tetrahedral sheet. The strength of this bond enables the kaolinites to form fairly large crystals so that kaolinite has a fairly low surface area (specific surface) of only $15m^2/g$. The specific surface of a mineral is its measured surface area per unit mass or volume as desired. Although the double layers around kaolinite may be fairly thick, this mineral is

generally very inactive, has little adsorptive capacity and a hydraulic conductivity, k, that is rarely less than 10^{-8}m/s, unless very pure and fine-grained. Fairly rapid advective transport and little retardating capacity are characteristics. Kaolinite, therefore, is not an ideal clay mineral for a liner, even though it is relatively immune to damage when exposed to many chemicals (Rowe *et al.*, 1996).

The clay mineral halloysite is actually a tubular form of kaolinite. This tropical mineral crystallizes with water between the sheets. Since the unit cell of the Al sheet is slightly smaller than that of the Si sheet, the sheets curl with the gibbsite sheet on the inside. When dried, the interlayer water is irreversibly lost, and its thickness shrinks to a permanent size similar to kaolinite. In addition, this mineral is difficult to compact because of its tubular structure, so that k-values tend to be high and erratic (Lambe and Martin, 1955).

(2) **Illite:** Illite is really a form of mica, and in many soils illite appears to be fine-grained muscovite, and occasionally biotite, derived by mechanical weathering from igneous and metamorphic rocks (Rowe *et al*., 1996). This mineral consists of stacks of 2:1 layer units held together by a very strong potassium bond to form a c-axis repeat unit, d = 1.0nm. The high strength of this bond is related to two main factors:

(a) a very high negative charge within the tetrahedral sheets (-0.6 to -1.0e/unit cell of ten oxygens); and

(b) the holes in the tetrahedral sheets into which the K^+ is partially countersunk.

The cause of this negative charge is isomorphous substitution of one in four Si^{4+} by Al^{3+}. Most of the large negative charge on illite is satisfied by fixed potassium. A small unbalanced charge corresponding to a CEC of 25meq/100g remains and these cations are what make illite a clay mineral of normal activity. Illite efficiently compacted or consolidated to form clayey soils having a hydraulic conductivity, K, of 10^{-9} to 10^{-11}m/s depending on the

void ratio. Also, the CEC of 25 meq/100g is adequate to permit abundant adsorption of undesirable species such as heavy metals.

Finally, there is no interlayer, c-axis expansion or contraction possible, so illite is often considered to be one of the most desirable clay minerals for use in engineered clay lienrs for municipal solid waste (Rowe *et al*., 1996). In temperate areas of acid weathering, the interlayer K^+ is sometimes leached out of illite, enabling x-axis expansion and formation of degraded illite or soil vermiculite. Such degradation would also occur if illite was leached by acid mine or tailing waters with a pH of about 2.0.

(3) **Chlorite:** Chlorite is 2:1 layer clay bonded together by positively charged octahedral sheets having Mg, Al, or Fe in the central position (Rowe *et al*., 1996). The resulting mineral has a d-value of 1.4nm, and the bond is normally strong enough to prevent c-axis swelling. In many respects, chlorite has properties similar to illite and is often favoured as effective, nonreactive barrier clay for municipal solid waste (MSW). Some iron chlorides, however, are highly susceptible to oxidation weathering. Fe^{2+} in the octahedral position oxidizes to Fe^{3+}, reducing the charge deficiency enough to permit swell and formation of vermiculites or smectites. These latter two minerals can be very troublesome.

(4) **Smectites:** Smectites is a group name for the 2:1 layer swelling clays which have a range in charge deficiency from –0.2e to –0.6e per unit cell (Rowe *et al*., 1996). Normally, this charge is present in the octahedral layer, and is too low to fix cations and binds the layer together. This mineral, therefore, experiences interlayer, c-axis swelling from 1.7nm to over 10nm depending on the chemical state of the clay. It is important to realize that a smectite with Ca^{2+} cations surrounding and neutralizing its negative charge will have a d-value of about 1.8nm, and actually consists volumetrically of 0.9nm of water for each 0.9nm of solid (i.e. a void ratio of unity for the solid phase). When the double-layer water and free pore water are added in, it is not hard to understand why smectites have such high void ratios.

Smectites consist of very tiny particles which produce a specific surface of up to $800m^2/g$. Thus, smectites may have low hydraulic conductivities (k between 10^{-11} and 10^{-15}m/s) even at high void ratios. For this reason, they are marked (as bentonites) for use as additives to sand to make clayey liners (or in combination with geotextiles to form geosynthetic clay liners). Great care must be taken, however, to ensure that they are compatible with the leachate to be retained, since they may be prone to chemically-induced c-axis contraction, double-layer shrinkage and craking. High-charge smectites (-0.6e/unit cell) require particular attention since they fix K^+, making them even more prone to shrinkage and reduction in CEC.

(5) **Soil vermiculites:** Vermiculite is a common soil additive used by gardeners as a modifier. A fine-grained equivalent occurs in many soils where it can be troublesome if abundant enough. It is a high charge, 2:1 layer clay (Rowe *et al.*, 1996). The charge of –0.6e to –0.9e per unit cell is satisfied by hydrated interlayer Mg^{2+}. Vermiculite rarely expands beyond 1.4nm, but if it comes into contact with K^+, it promptly fixed it and collapses to 1.0nm illite. This c-axis contraction causes shrinkage of the soil phase, and an abrupt decrease in CEC. This converts an active, adsorbent clay into a much less active illite. If sufficient quantities of vermiculite are present, shrinkage, and possible cracking, seems probable in the absence of high effective stresses. Smectites and vermiculites overlap at a charge deficiency of –0.6e/unit cell, and it is very difficult to decide which mineral is present and how it will behave in a barrier contacted by leachate. The amount of c-axis contraction to be expected can be assessed by x-ray diffraction analysis.

Generally speaking, inert clay minerals of modest cation exchange capacity (CEC), such as chloride and illite, probably produce the most reliable clayey liners. Vermiculites tend to fix K^+ and NH_4^+ causing reductions in CEC and, thus, require compatibility testing if present in significant amounts. The kaolins are the most inert, but suffer from very low CEC

values and problems in obtaining low values of hydraulic conductivity. Smectites may, in the end, be a very difficult clay mineral to control (especially in the Na^+ state), due to possible c-axis contraction caused by cation exchange and changes in electrolyte concentration. Another factor to be considered is that smectite barriers tend to be very thin (in some cases, as little as a few centimeters), and consequently a fairly large diffusion flux may be expected to pass through them, unless they are used in conjunction with another soil which will act as a diffusion barrier (Rowe *et al.*, 1996).

Most natural soils contain two or three different clay minerals, often mixed with quartz, feldspar and carbonates. To make matters even more complicated, some clay minerals are interstratified with one another (e.g illite-smectite, illite-chlorite, illite-vermiculite soils). Quantitative mineralogical analyses, complete with CEC measurements and an assessment of the adsorbed cation regime are now normally run as part of each site appraisal. Assessment of diffusion coefficients, and occasionally clay-leachate compatibility assessment by hydraulic conductivity testing may also be required.

3.6.2 **Clay colloid chemistry**

In order to understand clay colloid chemistry, this section is subdivided into two: (I) the nature of the clay-water micelle and (II) adsorbed cation regimes and cation exchange.

(I) **The nature of the clay-water micelle:** A single, tubular-shaped clay particle is shown in Fig. 3.4. The negatively charged particle is surrounded in nature by a swarm of hydrated cations which balance the charge. The particle and its surrounding double layer of water together compose a micelle (Rowe *et al.*, 1996). Even though most clay particles are far larger than the typical colloidal size of 1micrometer to 1nm, they behave like colloids because they carry a negative charge. Their electrical properties, therefore, have a far greater

effect on other physical properties than do gravitational factors, and under the right chemical conditions, clays (especially smectites) will form a stable suspension.

The concentration of hydraulic cations is highest next to the charged particle, decreasing with distance to the same values as the free pore water. Similarly, the concentration of anions is virtually zero next to the clay particle (except the edges) and increases to that of the free pore water with distance. Cation concentrations of 2-5m (molar:gram molecular weight per litre) are typically of the double layer. The electric potential illustrated in Fig. 3.4a incorporates the Stern layer concept in which the proximity of the first layer of firmly attached cations results in an abrupt drop in potential through the Stern layer, as shown. The distance δ corresponds to the center-line of the first layer of cations, and ψ_δ corresponds to the reduced potential which actually controls the size of the double layer. ψ_δ is somewhat variable, since an increase in concentration will add more cations to the Stern layer, reducing ψ_δ.

Also, changes in the cation species can cause great alterations. Perhaps the most important factor in clay-leachate interaction is expansion and contraction of the double layers. As illustrated in Fig. 3.4b a large double-layer contraction at constant void ratio, sometimes referred to as flocculation, creates a large increase in hydraulic conductivity, and possibly in the diffusion co-efficient as well. Conversely, a chemical change that peptizes, disperses or expands the double layers may eliminate most of the free spore space and reduce hydraulic conductivity.

For most conceptual needs in engineering practice, the Gouy-Chapman theory of potential adequately explains double-layer behaviour. The Gouy-Chapman equation can be expressed as (Rowe *et al.*, 1996):

$$\psi = \psi_0 e^{-kx} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.70)$$

where $k = (4\pi e^2 \sum n_{10} z_i^2)^{1/2}/(\varepsilon kT)^{1/2}$, n_{10} = ions concentration in bulk suspension, z_i = valence of ions, k = Boltzmann's constant, T = Absolute temperature, e = elementary charge, and ε = dielectric constant of pore fluid.

The Gouy-Chapman theory can be expressed graphically as presented in Fig. 3.5 with a sketch of the double-layer potential (see Fig. 3.6). The centre of gravity of the area under the potential curve occurs at a value of x = 1/x, and many discussions now refer to the 1/x as the double-layer thickness (Mitchell, 1993). The main messages to be derived from the equation for potential are as follows (Rowe *et al*., 1996):

1. If the pore water concentration is increased, the double layer contract.
2. If the cations are changed from monovalent to divalent or trivalent the double layers contract (e.g. from Na^+ to Ca^{2+}).
3. If the dielectric constant, ε, is reduced, the double layers contract (e.g. from 80 for water to 35 for ethanol).
4. If the temperature is increased, the dielectric constant ε decreases and the double layers contract (this is not indicated by the equation).
5. All four of the above factors will cause the hydraulic conductivity to increase (sometimes dramatically) if the void ratio remains constant.

Two plots of potential versus distance from the clay particle are presented in Fig. 3.6 to illustrate concentration, valency and dielectric effects. The upper plot (Fig. 3.6 (b)) demonstrates how monovalent to divalent cation exchange can reduce the double-layer thickness from 100nm to 50nm at dilute concentrations (0.01_M). At very high concentrations, double-layer thickness of only 0.5-1.0nm is indicated (Van Olphen, 1977). The second plot for changes in dielectric constant (Fig. 3.6 (c)) is equally dramatic.

(II) **Adsorbed cation regimes and cation exchange:** Cations normally found in nature are summarized as follows (Rowe *et al.*, 1996):

1. Ca^{2+} and Mg^{2+} dominate most systems.
2. Na^{+} and K^{+} are common in marine soils and certain volcanic ash deposits.
3. Al and Fe hydroxide cations having 'partial valences' of ~0.5^{+} exist in low pH podsolic environments (pH < 4).
4. H^{+} and H_3O^{+} are common in acid soils.
5. Metallics are rare, and are rapidly adsorbed or precipitated, except in very low pH environments.

The replacing power of the normally occurring cations can be arranged in the following sequence (Yong and Warkentin, 1975):

$$A\iota^{3+} >> Ca^{2+} > Mg^{2+} >> NH_4^{+} > K^{+} > H^{+} > Na^{+} > L_1^{+}$$

The Gapon exchange equation can be used to calculate roughly the relative abundance of the various cations on clay, but it is normally more useful to do an NH_4Cl exchange or a combined Ag-thiourea/KCl exchange and actually measure the adsorbed cations. Since Na^{+} is the most common and troublesome monovalent cation found on natural soils, measurement of the sodium adsorption ratio (SAR) is useful and often required during geotechnical assessment, and can be expressed as,

$$SAR = \frac{Na}{[0.5\,(Ca^{2+} + Mg^{2+})]^{1/2}}\ (meq/l)^{1/2} \qquad \text{---------- (3.71)}$$

Analyses are usually run on soil wetted to their saturation water content (i.e. the water content close to the liquid limit at which a shine appears on the wet soil) and all concentrations are in meq/l. The SAR (which is actually a form of Gapon's equation) is

particularly useful because it often correlates with the amount of exchangeable sodium (Mitchell, 1993). Dispersed, low-salt soils containing abundant adsorbed Na^+ may initially have low hydraulic conductivities but they may also be susceptible to chemical shrinkage and cracking if chemically altered, especially at low confining stresses.

3.6.3 **Cation exchange**

The levels of Na^+, K^+ and NH_4^+ in MSW leachate are sufficiently high that they effectively exchange some of the Ca^{2+} and Mg^{2+} present on natural clays during advection and diffusion. Since it takes two Na^+ to exchange one Ca^{2+}, this reaction should expand the double layers and decrease k (Rowe *et al*., 1996). The role of the similarly sized cations, K^+ and NH_4^+, is quite different. These two species tend to fix onto vermiculites and high charge smectites and are highly retarded.

Worse than this, however, they may contract the c-axis spacing, causing a reduction in the mineral or solids volume which should increase hydraulic conductivity, k. If they do fix into the holes in the tetrahedral sheets they also may reduce the CEC. This may contract the double layers, owing to charge reduction, causing large increases in free pore space at constant void ratio. In turn, this may cause increases in k, and possibly even liner cracking.

CHAPTER FOUR

ENGINEERING DESIGN CONSIDERATIONS, FLOW AND CONTAMINANT TRANSPORT MODELLING

4.1 Engineering Design Considerations

Attitudes regarding the design of municipal waste landfills and other disposal facilities for industrial and mining waste have changed significantly over the last couple of decades. It is not so long ago that waste was simply dumped in old rock quarries or gravel pits, and it was assumed that the impact would be reduced to acceptable levels, due to dilution of the leachate or chemical waste by the groundwater. In response to the environmental problems that this approach has caused, many governments are now regulating the design of waste disposal facilities. In the USA, for example, the Congress has mandated regulations relating to the minimum design for waste disposal sites which involve a single composite liner and a landfill cover no more permeable than the liner as the standard design.

Alternative designs may be permitted in approved states. In general, interest is only focused on a 30-year period after closure of the landfill. While this approach is much better than uncontrolled disposal of waste, it does not necessarily ensure that 'good' designs are implemented and that a site which meets the Environmental Protection Agency (EPA) requirements will not be an environmental hazard for the future (Rowe, 1991). In the province of Ontario, Canada, the proponent of a disposal site is required to demonstrate that the proposed facility will not impact on the present or future 'reasonable use' of groundwater at the site boundary (MoEE, 1993). In essence, this recognizes that over the long term it is

almost impossible to guarantee that there will be no impact on groundwater due to a waste disposal facility. While it is impossible to prove that any facility will never cause an environmental problem, it is possible to design a facility which, based on what we know today, should not impact on the reasonable use of groundwater, and hence on the environment.

Similarly, it is also possible to identify designs which, based on what we know today, have a reasonable likelihood of containing groundwater and the environment beyond the site. It may be that it will be tens or even hundreds of years before there are a serious impact. However, a long time to impact does not justify a design in the context of modern attitudes towards protecting the environment, not only for ourselves, but also for future generations.

Barriers should not be designed in isolation. Careful considerations must be given to the site hydrogeology, geotechnical conditions and climate as well as the size of the footprint and capacity of the landfill. Depending on these conditions, landfills which will not significantly affect groundwater quality may range from very simple designs involving natural clayey barriers to highly engineered systems involving multiple liners and leachate collection systems. In general, the larger the landfill, the longer the contaminant lifespan and the more elaborate the barrier system required to provide adequate environmental protection (Rowe *et al.*, 1996).

A barrier that is perfectly adequate for a given landfill at one site may be totally inadequate for a larger landfill at the same site or for the same landfill at a second site. In particular, it should be emphasized that recommendations such as those by the US EPA represent minimum technological requirements for US conditions. While these documents provide a very useful resource, their recommendations should not be adopted without careful evaluation on a site-specific basis (Rowe *et al.*, 1996). Both natural and synthetic materials have advantages and disadvantages. A good design will recognize this and make appropriate

use of both types of material. For example, if designed appropriately, giving due consideration to the function and service life of the various components in the context of the contaminating lifespan of the landfill, geosythetics can provide a means of considerably enhancing the performance of these facilities.

However, appropriate design is only the first step. It remains to construct the facility. It should be emphasized that construction quality control and assurance (CQC/CQA) are critical for both the natural and synthetic components of the engineered system. The CQC/COA procedure should be such as to ensure that the designer's intentions are met (Rowe and Giroud, 1994). It is relatively straightforward to design landfills to meet environmental requirements for a 30-year post-closure period. The longer the period of environmental protection required, the more difficult is the design. The design of landfills which, given what we know today, can be reasonably expected not to cause unacceptable environmental impact at any time in the future, often requires the balancing of a number of conflicting criteria.

The challenge is to use effectively the engineered components of the barrier system such that eventual failure of key components, like the leachate collection system or a geomembrane liner, combined with the relatively long contaminant travel times through suitable clayey liners does not create environmental problems for future generations (Rowe *et al.*, 1996). This section provides a background discussion concerning approaches to contaminant impact assessment, the contaminating lifespan of a landfill, and some factors to be considered when assessing the service life of the components of an engineered system. The objective of the design will be to design a system such that even when allowance is made for the uncertainty concerning the service life of some components of the system, the system itself can be designed such that the service life of the system exceeds the contaminating lifespan of the landfill.

4.1.1 Impact assessment

As noted in paragraph 4, the design of a barrier system is often intimately related to the environmental impact assessment of the proposed landfill. Environmental impact assessments are generally driven by regulatory requirements and, as a consequence, typical 'acceptable' barrier systems can be expected to vary regionally as a result of both variations in hydrogeologic conditions and variations in regulatory requirements. The fundamental question underlying most impact assessment is whether the proposed landfill will have no more than a negligible effect on groundwater quality at the site boundary. However, this perfectly reasonable question raises two subsidiary questions-what is a 'negligible effect' and over what period of time must the effect be negligible? The answer to the latter question has very important implications since it is intimately related to the design life of the engineering features of the facility.

The design of a barrier system for a landfill that is only required to have 'negligible' impact for a 30-year post-closure period is likely to be different from the design for a 100-year period which, in turn, may be quite different from that required to have negligible effect on groundwater quality in perpetuity. Typically, environmental regulations fall into one of the following categories (Rowe *et al.*, 1996):

1. Essentially no regulations;
2. Prescriptive regulations which specify minimum requirements such as 'two liners of which at least one is a synthetic liner';
3. Regulations requiring 'no impact' or 'negligible impact' for a prescribed period of time (e.g 30-years or 100-years post-closure);
4. Regulations requiring negligible impact in perpetuity.

1) **Non-existent regulations:** The situation where there is no regulation provides considerable latitude to the landfill proponent and designer in terms of barrier system adopted. It also provides little assurance that the environment will be protected unless the design is subjected to a rigorous pre-contsruction review.

2) **Prescriptive regulations:** Prescriptive regulations are simple. They are typically based on the perception of the regulators as to what constitutes a safe design, and will implicity have negligible effects on groundwater quality. Unfortunately, prescriptive regulations may not recognize that potential impact is not only related to the details of the barrier, but may also be highly dependent on many other factors including (but not limited to) the local hydrogeologic conditions, the size of the landfill (both in areal extent and thickness of waste), the infiltration into the landfill and the detailed design of the leachate collection system. Prescriptive regulations may be easy to administer, but they create a situation where for one landfill the design may be overly conservative, while for a second landfill the prescriptive design may provide no assurance that the long-term impact will be negligible.

3) **No-impact versus negligible impact:** Regulations which require no impact are emotionally desirable but involve practical difficulties. Typically, some form of model is required to demonstrate that there will be no impact, and a monitoring programme is required to assess the performance of the facility. In terms of modelling, the no-impact requirement is often impossible to achieve, since for most designs the process of molecular diffusion will result in some contaminant migration from the landfill, even for a 30m thick clay deposit, or a composite liner system composed of a geomembrane and compacted clay liner.

The requirement of no impact is also impossible to enforce since, at the very least, the impact that can actually occur is controlled by the detection limits used in the chemical analysis of the groundwater. Thus no-impact in practice becomes a negligible impact condition where negligible is related to sampling procedure and the analytical detection

limits, which vary depending on the chemicals being considered and the analytical technique used. In short, no-impact regulations may place unrealistic restrictions on the design of facilities since, if properly modelled, the vast majority of facilities would not meet this requirement.

Furthermore, this approach creates an arbitrary *de facto* limit on allowable potential impact, which is directly related to field sampling and laboratory techniques, and not to considerations of health and safety. A more meaningful approach is to require that the facility has negligible effect on groundwater, where negligible is quantified on the basis of considerations of background chemistry, the chemical species and the potential aesthetic and health-related implications of an increase in concentration in the groundwater. An example of this type of approach is contained in the Ontario Ministry of the Environment and Energy (MoEE) guidelines associated with its 'Reasonable Use Policy' (MoEE, 1993) which provides a means of qualifying a negligible effect in a rational and consistent manner.

4) **Regulatory period:** The regulated time period over which a landfill is required to have no, or negligible, effect on groundwater quality, or on a neighbour's reasonable use of groundwater drawn from near the property boundary, has a significant effect on the design. For example, one can readily design a barrier (and buffers) such that there is negligible impact at the site boundary for a 30-year period, simply by designing the system such that the travel time for contaminants to reach the boundary (at detectable levels) is greater than 30 years (or 100 years or whatever arbitary limit is set).

However, this does not mean that there will not be a significant environmental impact. It may simply mean that the impact of that facility is being passed on to future generations. One can readily encounter situations where the proposed design is not expected to have more than a negligible impact for 100 years but can be predicted to have a significant and unacceptable impact after 150-years. In the design of barriers, it is important to recognize

that the clay and geosynthetic-clay barrier being commonly used today will, if properly constructed, greatly retard the migration of contaminants from a waste disposal facility but, unless carefully designed, may still untimately result in significant long-term contamination of groundwater. The likehood of this occurring increases with the trend to larger landfills.

5) **Service life and contaminating lifespan:** A second factor intimately related to the regulated period of no or negligible impact is the service life of the facility. If property designed, specified and constructed, the service life of many of the key components of a landfill barrier system, such as leachate collection systems and geomembrane liners, is likely to exceed 30 years. Under these circumstances, it is a relatively straightforward matter to design a landfill to meet requirements for no or negligible impact for a 30-year post-closure period. Monitoring and contingency measures should, of course, be established to detect and rectify any unpredictable failure that might occur during this period.

However, when one is required to ensure negligible impact for periods exceeding 30 years, careful considerations must be given to the effect of degradation of the engineering components of the system. The most stringent regulations with respect to environmental assessment require that the landfill have negligible effects on groundwater quality (typically at the site bondary) in perpetuity. For example, the Ontario Ministry of the Environment and Energy's (MoEE) 'Reasonable Use Policy' places no time constraint on the period during which the landfill is to have a negligible effect on a neighbour's reasonable use of groundwater. The MoEE's Engineered Facilities Policy (MoEE, 1993) addresses the service life of the engineered facility, and requires that the service life exceed the contaminating lifespan, where the contaminating lifespan may be defined as the period of time during which the landfill will produce contaminants at levels that could have unacceptable impacts if they were discharge to the environment.

The contaminating lifespan will depend on the contaminant transport pathway, the leachate strength, the mass of waste (and, in particular, the thickness of waste) and the infiltration through the cover. The contaminating lifespan generally increases with increasing thickness of waste and decreasing infiltration into the landfill. The more hydrogeologically suitable the site (i.e. the greater the potential for natural attenuation of contaminants), the shorter the contaminating lifespan. For the limiting case of a site that has sufficient natural attenuation that no engineered barrier system or leachate collection system is required, the contaminating lifespan is zero.

4.1.2 Waste and leachate composition

This paragraph focuses on two types of leachate:

1. Domestic or municipal solid waste leachate (MSWL);

2. Liquid hydrocarbons, soluble and insoluble, in simple aqueous systems.

1. **Domestic waste leachate**

a) Domestic solid refuse

The composition of average municipal refuse (picked up at the curbside) may vary from country to country, and will depend on the local cultural background of the generating community. In North America, the composition of refuse should normally be similar to that presented in Table 4.1 (Ham *et al.*, 1979). While the composition may vary from one region or country to another, information such as that presented in Table 4.1 can be compiled for each such region/country.

b) Leachate generation

Percolation of rainwater through solid waste leaches out soluble salts and biodegraded organic products to form a foul-smelling, gray leachate. Fine-grained soils used as daily cover may also be incorporated as suspended solids in the leachate. Bacterial growth

starts immediately under oxidizing conditions generating temperatures which have been recorded to be in excess of 60^{o}C in some wastes (Collins, 1993). Since optimum growth takes place, it is probably food wastes which fuel early biological reactions. Low average nitrogen levels in bulk waste (only 0.5%) and rapid depletion of oxygen soon render the system anaerobic, cooler and far less reactive. Farquhar (1987) reports representative temperatures in the waste in landfills in Ontario as typically between 10 and 20^{o}C, which is well below the 35^{o}C considered to be optimal for methanogenesis. In other parts of the world (e.g UK, Germany) internal temperatures in landfills of 30-40^{o}C are common (Rowe *et al*., 1996).

MSW leachate is a complex liquid which changes in characteristics as one passes from the early acetic phase of young leachate. Because of the time period over which landfills may be constructed, the leachate that is collected may be a mixture of young and older leachate from different parts of the landfill. Typically, the acetic phase is characterized by high organics (Ehrig, 1989). Table 4.2 summarizes some typical characteristics of young and older leachates. These acids are very important since they render leachate weakly acidic and mobilize heavy metals. The remaining organics appear to be predominantly humic-carbohydrates, carboxyl compounds, carbony1 compounds, etc. (Chian, 1977).

Experience would indicate that the concentration of key contaminants in leachate reaches a peak value some time after landfilling begins and then decreases with time. Even for conservative species there is a sound theoretical basis for a decrease in concentration with time due to dilution as new leachate is generated and other leachate is removed by the leachate collection system (Rowe, 1991). For nonconservative species, this decay can be accelerated by both species, and biological breakdown.

Based on available data, a chloride concentration of 1500 to 2500mg/l would appear to be a reasonable source concentration for impact calculations for the MSW leachate in

Onatario. An average concentration for chloride of 2100mg/1 has been reported by Ehrig and Scheelhaase (1993) based on European data. In areas where there may be less leachate dilution, higher values may be more appropriate. Similarly, if the landfill is used to dispose of road salt, sewage sludge of industrial salty wastes then higher values may be appropriate.

Much of the organic phase is extremely biodegradable. Chian (1977) reports original total organic carbon values of ~1.7g/1 (~1.7%) in a two-month-old Illinois leachate with the following fatty acid composition (g/1).

1. Acetic acid	1.75
2. Propionic acid	0.51
3. Isobutyric acid	0.31
4. Butyric acid	3.08
5. Isovaleric acid	0.72
6. Valeric acid	0.56
7. Hexanoic acid	1.49

There is some question concerning the potential effect of recycling on leachate quality. Preliminary German experience (Collins, 1991) suggests that the removal of paper, glass metals, plastics and textiles (about 36% of the refuse stream) had no significant effect on chloride concentrations, but significantly reduced the concentration and mass of iron and calcium collected in leachate. The removal of an additional 32% of the waste as organic material to be composted lead to additional reductions in concentrations of iron and calcium and did not significantly affect chloride. This is beneficial with respect to extending the longevity of leachate collection systems. Additional research is required to identify whether these German findings are applicable in other countries/regions. It should be noted that the concentrations of organic compounds are low. The concentrations of various potential

contaminants existing in waste may be compared with typical drinking water standards as listed in Table 4.3.

c) MSW leachate stability

The dissolved organics in raw municipal solid waste leachate are highly biodegradable so that biochemical alteration continues in the sample bottles brought back to the laboratory for testing (Rowe *et al.*, 1996). Bacteria growth results in a rapid rise in pH, reduction in E_h (oxygen depletion as in reduction of SO_4^{2-} to S^{2-}) and apparently production of CO^{2-}. Since most leachates seem to have a high concentration of C_a^{2+}, flocs of calcite ($CaCO_3$) appear within a couple of days of laboratory storage if stored at room temperature (20^oC) and Ca^{2+} levels in the leachate experience a rapid reduction. Similarly, Fe^{2+} which may be abundant in acidic leachate declines rapidly as it forms an amorphous black slime of FeS_2.

Although these reactions are greatly inhibited by storage at 4^oC, this temperature is not suitable for long duration hydraulic conductivity testing. This lack of stability may cause chemical control and interpretation problems in laboratory tests. For example, the Ca^{2-}, Fe and pH values in the influent permeant must be continiously monitored along with the effluent chemistry during hydraulic conductivity testing for clay-leachate compatibility assessment (Rowe *et al.*, 1996). Also, the potential for similar decreases should be considered during diffusion testing.

2) **Liquid hydrocarbons**

The very complex subject of hydrocarbon liquids defies succinct summarization. Nevertheless, a useful summary of the classes of organic compounds and their properties has been prepared by Michell and Madsen (1987). Two classes of liquid hydrocarbons are considered:

1. Organics which are insoluble in water, this includes dense nonaqueous phase liquids (DNAPLs) and light monaqueous phase liquids (LNAPLs);

2. Organics which are soluble in water.

Since the solubility decreases rapidly with increasing size of a soluble organic molecule, consideration will be focused on small molecules consisting of up to about eight carbon atoms. The solubility of organic liquids is generally controlled by their polarity. Nonpolar or weakly polar substances dissolve in nonpolar or weakly polar solvents. Highly polar compounds dissolve in highly polar solvents including water.

Fortunately, the polarity or dipole momemt is usually closely proportional to the dielectric constant, ε, which figures so significantly in the Gouy-Chapman theory of potential (see paragraph 3.6). Methanol, CH_3OH, is soluble in water because the hydrogen on a methanol molecule interchanges easily with hydrogen on water molecules producing nearly infinite miscibility. The solubility decreases rapidly for alcohols consisting of chains longer than three carbons (Rowe *et al.*, 1996).

In water-wet clay barriers, solubility plays a critical role in the interrelationships between the barrier and a retained organic liquid. Several terms are useful to describe solubility.

i) Hydrophilic is used to describe water-loving or soluble organics;

ii) Hydrophobic is used to describe water-hating or soluble organics;

iii) Lyophilic is a general term describing mutual attraction (solubility) between two liquids;

iv) Lyophobic is a general term describing mutual repulsion between two liquids.

When large volumes of contaminant are available, the source concentration may be controlled by the solubility limit and the equivalent height of leachate may be quite large. Finally, a few words about the dielectric constant, ε. Liquids with a high dielectric

constant (water at 80^{o}C, methanol at 34^{o}C, etc) dissolve ionic compounds, not only because they are polar and efficiently hydrate the dissociated species, but also because they have insulating properties. In the case of NaC1 in water, the force of attraction between the hydrated Na^{+} and $C1^{-}$ atoms is greatly reduced by the high dielectric solvent. This applies equally in the double layer where the presence of water reduces the force of attraction between the negative clay particle and positive cations. If organic liquids of lower dielectric constant enter the double layer they effect an increase in the force of attraction and contract the double layer. Organic liquids are subdivided into two groups (Rowe *et al.*, 1996).

i) Insoluble (Iyophobic) DNAPLs and LNAPLs which normally have values for dielectric constant of 2 to 4;

ii) Soluble (Iyophilic) compounds which normally have dielectric constant values of about 10 or greater.

These use of the dielectric constant is very convenient since it plays a

prominent role in double layer theory.

4.1.3. Leachate mounding and collections

1) **Function of the collection system**

The movement of contaminant through a barrier will, as discussed in paragraph 2, depend on both the advective (Darcy) velocity and diffusion. The Darcy velocity will, in turn, depend on the hydraulic conductivity of the barrier and the hydraulic gradient will often depend on the engineering of the landfill, since landfill construction will frequently change the hydraulic conditions at a site. For example, modern landfills are commonly designed to have a leachate underdrain system.

These collection systems may serve several functions. Firstly, by lowering the height of leachate mounding, leachate seeps (and consequent contamination of surface waters) can

be minimized. Secondly, by reducing 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 in many cases. Thirdly, by removing contaminant from the landfill, the mass of contaminant available for transport into the hydrogeologic system will be reduced.

Of the functions listed above, the second and third are of greatest significance to the impact of the landfill on groundwater quality (Rowe *et al.*, 1996). Various methods of estimating the height of the leachate mound (and hence the head within the landfill) have been proposed. Once the height of leachate mounding has been calculated, it is a relatively straightforward calculation to estimate the average hydraulic gradient and Darcy velocity through the barrier. As demonstrated in the following section, it is then possible to estimate the impact of the leachate collection system on the mass of contaminant available for transport into the barrier. The following section will discuss a number of methods available for estimating the height of leachate mounding.

2) **Leachate mounding**

If the barrier beneath the waste is flat and of low permeability compared to the waste, then the height, h, of the mound between two drains separated by a distance (see Fig. 4.1(a)) may be estimated from an equation given by Harr (1962) which can be written as:

$$h = \Omega^{1/2}\{(l-x)x\}^{1/2} \quad \text{..... (4.1a)}$$

$$\Omega = \frac{q}{k_w} \quad \text{..... (4.1b)}$$

where h is measured relative to the head at the leachate collection drains; l is the spacing between drains; x is the distance from one of these points; q is the portion of the steady-state infiltration that is being collected by the drains; and k_w the hydraulic conductivity of the

waste (or other material) between the drains. At the midpoint between drawdown points, this equation reduces to

$$h_{max} = \frac{0.5}{\Omega^{1/2}} \qquad \text{…….. ……..} \quad (4.2)$$

where h_{max} is the maximum height of mounding above the barrier. Based on the variation in h with position x, it can be shown that the average value of h is given by

$$h = 0.785 h_{max} = \frac{0.393}{\Omega^{1/2}} \qquad \text{…….. ……} \quad (4.3)$$

Using the same basic assumptions as adopted in the development of eq. (4.2), Moore (1983) developed a solution for the case of a sloping geometry, as indicated in Fig. 4.1 (b), and derived an equation for the maximum height of mounding above the barrier, h_{max}.

$$h_{max} = \frac{0.5}{[(\Omega + s^2)^{1/2} - s]} \qquad \text{…….……} \quad (4.4)$$

where $s = \tan \alpha$ is the slope of the barrier.

McBean *et al.* (1982) considered a different sloping collection system, as shown in Fig. 4.1(c). This case is somewhat more complicated that the previous two cases, and one cannot write a simple explicit equation for the height h of leachate above the barrier. However, assuming zero pressure head within the drains, the height h and the distance at which it occurs can be related by the equation (Rowe *et al.*, 1996)

$$x = \lambda(1 - A \exp B) \qquad \text{……} \quad (4.5a)$$

where

$$A = \frac{\Omega^{1/2}}{\left[\frac{h^2}{(\lambda - x)^2} - \frac{sh}{(\lambda - x)} + \Omega\right]^{1/2}} \qquad \text{……...} \quad (4.5b)$$

$$B = \frac{s}{\left(4\Omega + s^2\right)^{1/2}} \left[\tan^{-1}\left(\frac{-s}{\left(4\Omega + s^2\right)^{1/2}} \right) - \tan^{-1} \frac{\left(\frac{2h}{(\lambda - x)} - s \right)}{\left(4\Omega + s^2\right)^{1/2}} \right] \quad \text{.....(4.5c)}$$

The leachate mounding between drains can be readily calculated by determining the location x for an assumed height of mounding h using a successive substitutions algorithm. Eqs. (4.1)-(4.5) were developed assuming that the infiltration q is equal to the flow passing into the collection points. When a portion of the infiltration water moves down through the barrier, the actual flow to the collection points q is equal to the difference between the infiltration q_o, and the flow into the barrier. These equations provide an approximate estimate of the head above the barrier (Rowe *et al.*, 1996). More elaborate equations for estimating the height of leachate mounding have been proposed (Wong, 1977).

The adoption of these more elaborate equations is only likely to improve the estimate of leachate mounding if the assumptions are reasonably applicable for a given situation. In many cases, they are not. If the hydraulic conductivity of the barrier is significantly lower than that of the overlying waste and drainage layers, then it may be appropriate to use the simple equations given above, recognizing the simplifications involved. If the hydraulic conductivity of the 'barrier' is of a similar order to that of the waste (e.g. the landfill is constructed directly on a silty sand or silty soil or fractured/weathered till), then the simplified equations are not valid, and it may be necessary to use the finite element technique to model the entire flow system (i.e. waste, barrier and underlying hydrostratigraphy).

Irrespective of the method of analysis or equation used in calculating the leachate head, probably the greatest uncertainty is associated with the hydraulic conductivity of the waste. To date, relatively little research has been conducted into the determination of appropriate values for use in design. A hydraulic conductivity commonly used in design is

10^{-6} m/s; however, there is growing evidence to suggest that the hydraulic conductivity could be one or two orders of magnitude lower near the bottom of the waste or when waste has been disposed with sewage sludge. Recognizing that waste is likely to be heterogeneous, it is not surprising that a limited number of field measurements (Hughes *et al.*, 1971) indicate a wide range (9×10^{-s} to 8.5×10^{-s} m/s) of hydraulic conductivities. Furthermore, if the waste is anisotropic, then these values may not be representative of the parameters controlling the leachtae mounding. Consequently, the design of leachate collection systems and the calculation of potential impact on groundwater quality should involve some consideration of implications of uncertainty regarding the hydraulic conductivity of the waste.

3) **Clogging of granular blanket drains**

While the foregoing works well in theory, experience has indicted that sand blankets placed directly beneath waste have a tendency to clog (Rowe *et al.*, 1996). This clogging may be a result of a combination of particulate clogging, clogging due to chemical precipitation and clogging due to biofilm growth. It must be recognized that 'clogging' of a drainage layer is not synonymous with it becoming impermeable. On the contrary, a clogged sand blanket may still be substantially more permeable than, say, an underlying clay liner. 'Clogging' of a drainage layer becomes significant when the hydraulic conductivity of the blanket drops to, or below, the hydraulic conductivity of the overlying waste.

Moderate clogging: The sand/granular blanket may be deemed to have clogged once its horizontal hydraulic conductivity drops to a value similar to that of the waste. Once this occurs, leachate migration to the drains will no longer be preferential to the drainage blanket, although a significant proportion of the leachate may still be being transmitted horizontally to the drains through the 'clogged' sand blanket.

Severe clogging: Severe clogging of a sand/granular blanket may be regarded to have occurred if the horizontal hydraulic conductivity drops to more than an order of magnitude

less than in the waste. Once this occurs, the majority of lateral flow towards the drains will occur in the waste.

4) **Minimizing clogging of blanket drains**

Clogging of granular blanket drains may be the result of a combination of particulate clogging, clogging due to chemical precipitation and biofilm growth. Thus, the likehood of clogging occurring can be minimized by (Rowe *et al.*, 1996):

i) Maximizing the flow velocity in the drain;

ii) Maximizing the void size;

iii) Minimizing the surface area available for biofilm growth.

By maximizing the flow velocity, one reduces the residence time for leachate in the collection system, thereby reducing the amount of sedimentation of particulates and chemical precipitates which can occur. Clearly, the flow velocity can be increased either by increasing the gradient (e.g. by increasing the slope of the landfill base) and/or by increasing the hydraulic conductivity of the granular layer. Ideally both the slope and hydraulic conductivity will be as large as practicable. Maximizing the void size within the drainage blanket tends to increase the initial hydraulic conductivity and reduces the likehood of the voids becoming blocked. Biofilm growth is related to the surface area available on the particles forming the drainage layer. Since the surface area is proportional to the square of a particle's diameter, and the volume is proportional to the cube of a particle's diameter, it is evident that the surface area can be minimized by increasing the diameter of particles used to construct the leachate collection system.

Recent research has demonstrated a clear relationship between the grain size distribution and clogging. For example, Brune *et al.* (1991) obtained samples from clogged leachate collection systems and from laboratory studies. They demonstrated that a graded sandy gravel with grain size in the range 1-32mm and fine gravel (2-4mm size) experience

severe clogging in laboratory column tests conducted using high strength leachate under anaerobic conditions. Coarser material (medium gravel 8-16mm size) experienced clogging of smaller pores, but the larger pores remained open at the end of the experiment (16 months). Coarse gravel (16-32mm) in direct contact with waste was covered with a thick film and locally clumped together; however, the large pores remained unclogged throughout the (16 months) test and maintained its hydraulic conductivity.

Similar tests conduced using a nonwoven geotextile filter above the drainage material indicated that the geotextile played a sacrificial role, substantially reducing clogging of the underlying drainage material with the formation of a cake of clogged waste in the upper portion of the geotextile and above the geotextile. Analyses of the material which clogged the drainage systems (both in laboratory and field studies) have shown (Brune *et al.,* 1991) that it consists of a network of aggregates of bacteria with deposits of inorganic material on their surface. This process is initiated on the bacterial cells and the slime fibrils which they excrete. Precipitation then centres on these seeds. Chemical analyses indicate that the precipitate consists of calcium, iron, magnesium and manganese combined with carbonate and sulfur. As indicated by Brune *et al*. (1991), the precipitation and clogging of drainage material in landfills is caused by the following two processes.

i) Iron-reducing bacteria solubilize Fe (III) by reducing it to Fe (II), while sulface-reducing bacteria reduce sulfate to sulfide. This bioreduction causes the region immediately around the bacteria to become more alkaline, and leads to the precipitation of sulfur as an insoluble metal sulfide.

ii) Calcium is mobilized in the waste as a result of fermentative organisms producing organic acids which lower the pH of the leachate, and hence mobilize calcium (and other metals). As in the case of sulfide, it would appear that calcium carbonate then precipitates in the alkaline region immediately around the bacteria, as discussed above.

Thus, the clogging of drainage systems is the result of a mobilization process involving fermentative bacteria together with iron and manganese-reducing bacteria, followed by precipitation processes involving primarily methane- and sulfate-reducing bacteria. These latter processes are identical to the precipitation processes (Rowe *et al.*, 1996) on leachate stability. Another example of clogging of the drainage material around leachate collection pipes in the drain around a landfill has been reported by McBean *et al*. (1993), who noted extensive clogging which resulted in excessive leachate mounding and leachate seeps.

Typical specifications require that the hydraulic conductivity of a granular drainage blanket be greater than about 1×10^{-5}m/s. There are many granular materials that would meet this specification, and some are far more susceptible to clogging than others. Based on the foregoing argument, it is evident that the potential for clogging is greatest for well-graded silty sands (which will have a hydraulic conductivity close to 10^{-5} m/s, small voids and a large available surface area), will be less for uniform coarse sand and will be a minimum for uniformly graded coarse stone (e.g. 50mm clear stone).

This would lead one to specify clear stone as the granular, blanket; however, if clear stone is placed in direct contact with the waste, one can expect migration of particles (e.g. sand and silt size particles) of the waste into the clear stone. This in turn would reduce the effective void space and the hydraulic conductivity, and would increase the surface area available for biofilm growth. This leaves either or both of the following options (Rowe *et al*., 1996):

i) Placing a select waste above the coarse granular blanket;

ii) Having a multicomponent collection system involving a filter which will minimize the migration of particles into the granular blanket.

If the stone is to be placed on top of a compacted clay liner, it may also be desirable to install a geotextile separator between the stone and the clay to minimize spoiling of stone and liner. The use of geotextiles as both filter and separator above and below a granular drainage blanket is shown in Figs. 4.2 and 4.3 and discussed in the next section.

5) **Geotextiles as filters and separators in leachate collection systems**

Geotextiles have been used to minimize contamination/clogging of granular drainage layers (see Fig.4.2). The upper filter geotextile is intended to minimize the migration of solids from the waste (including a portion of suspended solids in the leachate) while permitting liquid to pass; however, there is a significant risk that this will clog due to biological action and chemical precipitation, as discussed below. The lower separator geotextile shown in Fig. 4.2 is intended to minimize intermixing of the fine-grained liner material and the coarse-grained drainage stone.

An alternative design, shown in Fig. 4.3 makes use of a geotextile filter to separate two components of a granular blanket. This type of detail might be adopted in the design of a landfill as a hydraulic trap where a primary consideration is to avoid the build-up of leachate head directly on the clay liner, since any such build-up in excess of the design value will reduce (and possibly reverse) the inward gradient. With the detail shown in Fig. 4.3, there may be clogging of the geotextile and the development of a perched leachate mound in the overlying stone; however, this will not affect the performance of the hydraulic trap. With a system such as that shown in Fig. 4.3 it is desirable to provide a direct (pipe) connection between the stone above the geotextile and the manholes, in addition to the normal connection for the pipe in the drainage layer below the geotextile (Rowe *et al.*, 1996).

It should be noted that the design of leachate collection systems involves consideration of both hydraulic and structural aspects. The structural capacity of the pipe

should be considered in the context of the loading conditions and bedding for the pipe (Moore, 1993).

a) Geotextile filter design

In filter design, candidate geotextiles must satisfy three requirements: they must have adequate soil retention capability, be sufficiently permeable and have sufficient resistance to clogging. With regard to retention capability, the objective of the geotextile is to filter out particle sizes which would settle out if permitted to migrate into the drain. Thus the design of the geotextile is related to the hydraulic design of the drainage layer - the slower the flow in the granular layer (i.e. the lower the hydraulic conductivity and or/ slope of the drainage layer) the slower will be flow, and hence the smaller the particle size which can be permitted to pass through the geotextile.

The candidate geotextile must be sufficiently permeable to allow leachate to pass through the waste into the drainage layer: this criterion will be readily satisfied provided that the hydraulic conductivity of the geotextile exceeds that of the waste. The filtration characteristics of the geotextile may be selected on the basis of a number of techniques. For example, Faure *et al.* (1986) proposed the following guidelines:

1. For uniform soils, the filteration opening size (FOS) < $1.5d_{85;}$

2. For well-graded soils, $1.5 < FOS/d_{85} < 3.0$.

d_{85} is based on the grainsize distribution of the soil (or waste) to be filtered, and 85% (by dry weight) of the soil has a particle size smaller than d_{85}; FOS the filtration opening size (FOS) of the geotextile as determined by the hydrodynamic sieving technique. The terms 'uniform' and 'well-graded' relate to the shape of the grainsize curve of the soil to be filtered.

This is often quantified in terms of the uniformity coefficient, C_u, where (Rowe *et al.*, 1996) C_u is $d_{60}/d_{10.}$ d_{10} and d_{60} correspond to the grainsizes below which 110% and 60% of the soil lies on a grainsize curve. Commonly, soils with C_u < 3 to 4 are regarded as being

poorly graded; soils with $C_u > 6$ are well-graded provided that the grainsize curve is smooth and reasonably symmetrical (Rowe *et al.* 1996)

Giroud (1982, 1994) has proposed an alternative guideline:

	$(1<C_u < 3$	$C_u > 3)$
$1_D<35\%$	$AOS<10C_u d_{50}$	$AOS<9d_{50}/C_u$
$35\%<1_D<65\%$	$AOS<1.5C_u d_{50}$	$AOS<13.5d_{50}/C_u$
$1_D>65\%$	$AOS<2.0C_u d_{50}$	$AOS<18.0d_{50}/C_u$

where d_{50} corresponds to the grainsize below which 50% of the soil lies on a grainsize curve; AOS is the apparent opening size of the geotextile as determined from a dry sieving technique; $I_D$is the density index for the soil which is an indicator of whether the soil is in a loose, compact or dense state of compaction. It should be noted that the foregoing design criteria were developed for cohesionless soils; compact cohesive soils rarely pose a problem with respect to filtration. It follows from the foregoing that the most difficult soils to filter are fine uniform silts. As an example of a filter design in a landfill application, Rollin and Denis (1987) have recommended the use of nonwoven geotextiles with FOS < 140 μm for the separator/filter between compacted clay liners and granular (sand) leachate collection systems, and FOS < 75 μm for geotextiles between in-situ clay soils and granular (sand) secondary leachate collection/pressure relief systems.

b) Geotextile clogging

The high potential for clogging of leachate collection systems is evident from observed clogging of sand layers discussed previously. An indication of the degree of clogging of geotextile filters that can occur due to accumulations of leachate fines and microbial growth may be obtained from recent studies (Cancelli and Cazzuffi, 1987; Koerner and Koerner, 1989, 1990; Cazzuffi *et al.*, 1991; Brune *et al.*, 1991).These studies suggest that

decreases in geotextile permittivity (cross-plane permeability) of many orders of magnitude may occur after exposure to leachate flow. One study (Koerner and Koerner, 1989) indicates a rough correlation between the organic constituent levels of the leachate and the degree of geotextile clogging. However, the study by Brune *et al*. (1991) indicated only modest clogging of a geotextile. Even under extreme conditions, the geotextile filter did provide protection of the underlying gravel drainage layer, but did itself experince a substantial decrease in hydraulic conductivity.

While some filter clogging must be expected, there are several strategies which may be employed to decrease the degree of clogging and the rate of fines accumulation and bioslime growth. Two such strategies are suggested by test results reported by Koerner and Koerner (1990). These texts indicate that inclusion of a granular layer over the geotextile reduces the rate of permeability decrease; however, if sand is used then there is the increased risk that the sand itself will clog. This problem could be addressed by using a clear gravel with particle size in the range of 16-32mm as shown in Fig.4.3.Decreases in the degree and rate of filter clogging may be achieved by the addition of antimicrobial agents to the geotextile material and by selection of a high percent open area geotextile (Koerner and Koerner, 1989), although this latter criterion must be balanced by the need to provide adequate retention.

The use of one or more of the above strategies may result in a decreased degree and rate of clogging and a more efficient collection of leachate, but in any case, clogging of a filter does not mean that the filter becomes impermeable (it simply means that its permeability has decreased).Typically the geotextile will have had an initial permeability (hydraulic conductivity) several orders of magnitude larger than typical values for compacted waste. The clogging of the geotextile would not be expected to have any significant effect so long as the permeability of the clogged geotextile is above that of the waste.

Once the permeability drops to below that of the waste, then the effects of the geotextile clogging will be highly dependent on where the geotextile is used and on what measures are taken to allow leachate drainage once clogging occurs. For example, Rowe (1993) indicated that the use of French drains with a geotextile around the stone would be particularly undesirable, because clogging of the geotextile would result in an increase in the height of the leachate mound which is acting directly on the barrier. This increased leachate mound have immediate implications with respect to contaminant migration from the landfill. On the other hand, clogging that decreases the permeability below that of the waste may not be a major concern if the filter is continuous across the site and is underlain by a continuous drainage blanket. A badly clogged filter will result in some perching of leachate above it.

This is not problematic with respect to contaminant impact on underlying groundwater/aquifers since the leachate is separated from the barrier by the drainage layer and hence the perched mounding does not represent mounding increases, so too does the gradient across the clogged geotextile, until the gradient is sufficient to provide transmission of infltration to the drainage layer. For example, if the clogging of a 3mm thick geotextile causes a decrease in permeability from say 1 x 10^{-8} m/s (i.e. by five orders of magnitude), the permitivity of the geotextile would still be high enough to transmit the arriving infiltration of up to about 0.3m/a without any significant long-term perching of leachate (short-term perching may occur during periods of high infiltration)(Rowe *et al*, 1996).

(c) Construction damage

Defects arising from construction damage, poor seaming techique, and rupture from excessive subgrade settlment may cause an intact geotextile to become discontinuous. The survivability of geotextiles during installation has been the subject of some study (Koerner and Koerner, 1990; Bonaparte *et al.*, 1988). Damage arising from construction is possible from the puncture of coarse granular materials above and below the geotextile and from

tearing due to the action of heavy machinery. The geotextile should have sufficient strength and puncture resistance to survive normal construction. The construction specification should cearly emphasize the importance of maintaining the integrity of the geotextile. Rupture from subgrade settlements can be minimized by the use of low modulus geotextiles or by appropriate subgrade preparation. In a recent study (Rowe *et al*., 1993), samples of geotextiles meeting medium and high classifications were removed from the interface between:

i. approximately 200mm of angular 50mm diameter crushed stone and a compacted clay liner;

ii. a compacted clay liner and a rounded 10-30mm diameter gravel layer.

No significant damage of the high survivability geotextile was observed either in terms of observed holes, or a change in tensile strength as obtained from the wide width tension test. The medium survivability geotextile was indented but generally did not contain holes when placed between the liner and rounded 10-30mm gravel; when placed between the 50mm crushed stone and a compacted clay (having been subjected to many passes of a loaded water truck) some holes were observed. The reduction in wide width tensile strength was not significantly different for the two exposure conditions. The average reduction in tensile strength of the medium geotextile, due to construction damage, was about 15% (Rowe *et al*, 1993). It is notable that the strain to failure decreased significantly (and the modulus increased) for both geotextiles. This finding is similar to that by Bonaparte *et al.*, (1988).

d) Separator design

The design of the separator layer between the drainage layer and the underlying fine-grained natural or compacted soil is more straight-forward than the design of the filter layer discussed above, and follows conventional practice (Koerner, 1990). Selection of the

geotextile for this layer also requires consideration of ways of minimizing the potential defects as discussed above.

6) **Geosynthetic drainage layers**

Geosynthetic drainage layers are typically net materials (geonets) which are structured to include open channels for the in-plane conveyance of relatively large quantities of fluids. The use of these geosynthetic drainage layers for leachate collection and detection at waste disposal facilities is becoming increasingly common (Koerner and Hwu, 1989). For this application, the geosynthetic drainage layer is often used in conjunction with an upstream geotextile filter and downstream geomembrane barrier, to form a composite system (Fig.4.4).

The geotextile and the geomembrane restrict the movement of soil particles into the open channels. Fig. 4.4 shows a typical leachate collection and leak detection system using geonets, with geotextiles to minimize soil intrusion into the geonets. Fig. 4.5 shows a conventional primary leachate collection system and a geosynthetic leak detection system. A primary design consideration for these drainage layers is their insitu flow capacity and the leachate residence time within the drainage layer.

a) Flow capacity and compressibility

The flow regime within the geosynthetic drains is usually turbulent, and so flow calculations based on the assumption of an intrinstic material permeability or transmissivity are not possible, since flow is not linearly propotional to the hydraulic gradient. Thus the design must be based on direct comparisons of the required and actual material flow rates. In addition to the hydraulic gradient, several other factors have been identified which will influence the flow behaviour and capacity of the geosynthetic drainage (Koerner *et al.*, 1986). These include compression due to over-burden loads, compression-included creep, thickness (or number of layers), boundary conditions, the temperature and viscosity of the

flowing fluids, and the directional drainage characteristics (Rowe *et al.*, 1996). In order to select suitable geosynthetic drainage material to meet a design flow, it is necessary to have results from laboratory simulation tests on candidate materials, using the actual loads and boundary conditions of the field situation.

The test program should be conducted to provide relations between the flow rate, hydraulic gradient and applied pressure. The compressive strength of the drainage layer must be determined to be adequate under expected overburden loads. Consideration must also be given to compressive creep behaviour. Koerner *et al.* (1986) have used a three-element model to estimate creep behaviour. This approach shows promise; however, caution is required in extrapolating long-term behaviour. Three time dependent material behaviours are involved: the waster, the geosynthetic and the soil. It is difficult enough to predict the longer-term deformation to any one of these components; it is even more difficult to predict the long-term interaction of all three (Rowe *et al.*, 1996).

As with granular systems, geosynthetic drainage layers have the potential to clog significantly. This should be carefully considered in the design. The level of confidence in the long-term behaviour of these systems decreases with the length of time that the system is required to be operative. In this respect, careful consideration should be given to the contaminating lifespan of the landfill. Generally, (Rowe *et al*., 1996) the larger the landfill thickness and the lower the infiltration, the greater will be the concern regarding long-term performance. The longer the contaminating lifespan the greater is the need for backup systems in the event that the leachate collection system fails.

b) Leak detection

When geosynthetic drains are being designed to detect leakage through a primary liner, the leak detection rates are governed by the transmissivity of the geonet at the design

slope and the length of the flow path. Geonet flow capacity may be evaluated for a range of hydraulic gradients and compressive loads, as detailed by Williams *et al*. (1984).

c) Reinforcement function of adjacent geosynthetics

The use of geonets for detection and leachate collection imposes a reinforcement function on the adjacent materials (usually geotextiles or geomembranes), which must span openings on the surface of the mat. Tests by Hwu *et al.*(1990) indicate that a flow rate reduction of one order of magnitude is possible due to geotextile intrusion into the apertures of geonets. Modest decreases in intrusion (and increase in flow) were observed for geotextiles stiffened by resin treating, burnishing and scrim reinforcing, and a large decrease in intrusion results with the use of a composite fabric (a needle-punched nonwoven over a woven slit-film geotextile).

Some mats have been designed with limited opening size adjacent to the geotextile, or with a three-layer system, to minimize the effect of geotextile/soil intrusion. In some cases there may be advantages to using a geocomposite drainage layer with the geotextile heat bonded to the geonet, although care is required to ensure that the permittivity (and hence permeability) of the geotextile is not significantly decreased by this process. A further consideration in the design of geotextiles and geomembranes over geonet openings is long-term tensile creep. Limited studies of this design aspect have been performed (Hwu *et al*., 1990). The earlier comments regarding long-term behaviour also apply to this aspect of drain performance.

d) Selection and acceptability

Published data (e.g. Koerner *et al.,* 1986) show wide variations in flow rate of various geosynthetic drainage products; some have flow capacities of the same order as a 0.3m thickness of clear stone while others are similar to the flow capacity of 0.3m of sand. The wide variations in flow rate illustrate the importance of laboratory simulation of the actual

field situation for selection purposes. It should be noted that some products may be placed in multiple layers to increase the flow capacity.

Test results (Koerner and Hwu, 1989) show that the flow rate increases from the use of additional layers cannot be reliably estimated from tests with a single layer, but must also be determined from simulation testing. An alternative to the use of multiple layers is to use thicker geonets; however, the greater the three- dimensionality of the grid the greater the need to examine carefully the time-dependent response of the system (Rowe *et al.*,1996). With respect to the seaming of synthetic drainage mats, only limited research, notably by Zargorski and Wayne (1990) has been carried out, and it was concerned with geonet seams only. Of particular interest is the considerable reduction in geonet transmissivity when overlapping is used, due to intrusion of one geonet into the other.

The acceptability of drainage mats for leachate collection will depend on the regulatory environment, the nature of the leachate (i.e. the potential for clogging) and the length of time that the drainage mat must operate to protect the environment (i.e. the contaminating lifespan). It is a relatively simple matter to design a mat that is only required to function for a 30-year post-closure period for a landfill that will produce low strength leachate. However, many modern landfills have contaminating lifespan of several hundred years (Ehrig and Scheelhaase, 1993). The long-term performance of any leachate collection system is a major consideration in these cases, particularly when regulations require negligible impact on groundwater in perpetuity. One means of increasing the probability of the long-term operation is to design a redundant system (Rowe *et al*., 1996). A redundant system is an unused landfill which is a standby incase an operating landfill fails. This idea is better than waiting for a landfill in operation to fail before designing and construction of another landfdill.

4.1.4. **Leakage through liners**

Although frequently used, the term ‘leakage’ is both pejorative and misleading. For example, both a high quality geomembrane liner and a very low permeability clay liner will allow some passage of fluid under an outward hydraulic gradient, and thus some would say that they leak, even though the volume of fluid moving through the liner may be substantially less than that considered in the design, and may be so low that the net impact is negligible. On the other hand, even if there is no actual fluid flow across the liner (i.e. zero leakage), there could be significant outward diffusive mass transport through the liner which could, in some circumstances, have a significant net impact on the environment. Thus the actual magnitude of the flow (leakage) through a liner is not itself of primary significance.

What is of significance is the magnitude of contaminant transport through the liner and the manner in which the barrier system has been designed to accommodate this, and hence provide protection to the environment. However, it is still necessary to be able to estimate the advective transport through a liner to provide input to the design of the barrier system and, subsequently, the contaminant impact assessment (Rowe *et al.*, 1996).

1) **Soil liners**

Advective transport through the soil is typically represented in terms of the Darcy or discharge velocity. This, in turn, depends on the hydraulic conductivity (permeability) of the soil and the hydraulic gradient (Rowe *et al.*, 1996).

a) Gradients

The gradient may depend upon the initial hydrogeologic conditions, or may be totally controlled by the engineering design. The gradient will often depend on the design of the leachate collection system and the height of leachate mounding. When calculating gradients based on natural hydrogeologic conditions, it is essential to carry out the following (Rowe *et al.*, 1996):

i) Ensure that there is an adequate data base conserning natural water levels over both short- and long-term periods.

ii) Consider the effects construction of the facility itself may have on water levels beneath the site. This is particularly critical for landfills designed as hydraulic traps where the landfill extracts water from the underlying groundwater system. It may also be significance when the landfill simply reduces the natural re-charge over a large area. Where necessary, flow modeling may be useful for assessing the interaction between the landfill and the hydrogeologic system.

iii) Consider the implications of potential long-term changes in climate and/or industrial or municipal development on groundwater re-charge groundwater levels. The significance of this factor increases with increasing contaminating lifespan. Systems should be designed such that they are not sensitive to these changes.

b) Hydraulic conductivity of compacted clay liners.

A major factor affecting the long-term durability of clayey liners is clay-leachate compatibility. Having noted that low hydraulic conductivity clayey liners have been successfully constructed, it should be recognized that some variability in hydraulic conductivity is to be expected and should be considered in both the design and specifications. One of the greatest potential dangers to a well-constructed liner is cracking of the liner if it is not properly maintained following compaction. For example, the liner must be protected from drying (with consequent shrinkage cracks) and freezing (with consequent cracks from ice lensing). If cracking of the liner occurs then the primary path for contaminant transport will obviously be through the cracks/fissures.

It should also be noted that it is a much simpler matter to construct a compacted clay liner on an existing natural soil (e.g. stiff clay, till or dense sand) than to construct it on top of other engineered components of a system such as a granular hydraulic control layer,

geosynthetic leak detection system and/or geomembrane liner. Invariably, the need to minimize damage to underlying engineering systems has some effect on the hydraulic conductivity of at least the first lift of an overlying compacted liner. When designing these systems, a test pad which emulates the proposed cross-section is highly desirable in order to identify potential problems and refine the construction specifications (Rowe *et al.*, 1993).

c) Hydraulic conductivity of natural barriers

The hydraulic conductivity of undisturbed or natural barriers (e.g. undisturbed clayey deposits) may be assessed by a combination of laboratory and field hydraulic conductivity tests. In performing these studies, it must be recognized that natural deposits can be variable. This variability should be considered in the design. One issue requiring particular attention when dealing with natural clayey deposits is the potential existence of fractures. It is well-recognized that the weathered crust of a clay or clay till will be fractured. What is not well-recognized is the possibility that the unweathered till or clay could also be fractured (Herzog *et al.*, 1989) to suggest that many tills in North America may be fractured to depths well below the weathered crust.

Where fractures are present, the bulk hydraulic conductivity may be considerably higher than values obtained from laboratory tests on intact samples. Unfortunately, in many cases (Rowe *et al.*, 1996) the presence of fractures was not evident either from borehole samples or hydraulic conductivity tests conducted in boreholes. In most cases the fractures were only readily observed by construction of deep test pits or from the results of a pumping test conducted on an aquifer below the fractured unit. If present, the effect of hydraulically significant fractures can be minimized by (Rowe *et al*, 1996):

i) Operating the landfill as a hydraulic trap;

ii) Reworking some of the soil as a compacted clayey liner; and /or

iii) Constructing a composite liner system.

d) Diffusion through soil liners

As previously noted, the phenomenon of molecular diffusion may be primary contaminant transport mechanism through a well-designed and constructed liner.

2) **Geomembrane liners**

Under certain hydrogeologic and geotechnical conditions, natural soil or compacted clay barriers may be unable to provide the required level of environmental protection from landfill contaminants. Often, the use of geomembranes as a complementary barrier material can provide an economical means of obtaining the additional protection necessary. As with clay liners, there are two primary mechanisms for contaminant transport through geomembranes: advection and diffusion (Rowe *et al.*, 1996).

a) Advective transport through geomembranes

Intact geomembrane have a very low hydraulic conductivity to water. Values of the order of 10^{-13}-10^{-15} m/s have been reported (Giroud and Bonaparte, 1989) although, as noted by these authors, the actual mechanism of water migration across the membrane is different from pure advection. The majority of advective flow through a geomembrane will be localized to defects (e.g. tears, inadequate seams) in the geomembrane. An assessment of leakage through holes in the geomembrane requires an estimate of the size and frequency of such holes. Such an assessment based on a combination of theory and an analysis of geomembrane case histories has been provided by Giroud and Bonaparte (1989) who indicate that even with intensive quality assurances, it is reasonable to expect three or five geomembrane defects per hectare.

Equations relating the leakage rate through these holes to the hydraulic head acting for the case where the geomembrane is underlain by a clay layer (composite liner) and for the case where the liner is a geomembrane underlain by a pervious material (e.g. a drainage net) have been presented by Giroud and Bonaparte (1989). This work has been refined and

extended by Giroud *et al*. (1992) who considered long cracks in the geomembrane, and presented a number of equations that can be used to estimate the rate of leakage through a composite liner. These equations can be combined with a contaminant transport model as described by Rowe *et al*. (1994). The occurrence of defects in the geomembrane liner can be reduced by the use of a geotextile 'cushion' between the geomembrane and adjacent soil layers. When the geotextile is used between the geomembrane and a clay barrier, the geotextile may increase leakage rates by providing a lateral flow plane, and increasing the area over which the hydraulic head acts.

b) Diffusive transport through geomembranes

The process of molecular diffusion under a chemical potential gradient is well understood (Park, 1986) and applies to geomembranes just as it applies to clay soils. Despite its potential significance, relatively little research has been undertaken with respect to this mode of contaminant transport. The limited available data suggest, that diffusion through geomembranes does occur, and that it depends on both the chemical species being examined and the geomembrane.

Considerable additional work is required to evaluate the significance of diffusion through geomembranes. Until such time as more definitive studies are completed, it would be prudent to consider diffusion when examining the potential impacts of landfills (Rowe *et al*., 1996). This can be readily done using theoretical models. For example, a geomembrane overlying a compacted clay liner can be modeled as separate layers. The finite layer technique readily allows modeling of a 1-2mm thick geomembrane in contact with a clay layer which may range in thickness from a few centimeters to tens of meters.

4.1.5 Leakage detection systems

Just as the concept of leakage through a liner is subject to a number of interpretations, so too is the role (and consequent design) of leak detection systems. Considerable work has been conducted in the USA with respect to leak detection (e.g. Geoservices, 1987) and the performance of a primary liner is judged in terms of the leakage (i.e. flow) monitored in leak detection systems. The effectiveness of these leak detection systems will depend on the detection sensitivity and the time between when leakage occurs and its detection.

The implementation of contingency plans in case of failure of all or part of the waste containment system will occur when unacceptable levels of leachate are collected from the leak detection layer. However, there are other sources of flow from leak detection layers, as identified by Gross *et al.* (1990). If leakage through the primary liner into the leak detection layer occurs simultaneously with flow from these other sources, the presence of the leachate may be masked.

Chemical analysis of the flows may identify the presence of leachate, but there may be considerable dilution. The leak detection system should be designed such that in case of a failure (i.e. a large rate of leakage) the time between detection and implementation of contingency measures will not be so long that environmental damage will occur (Rowe *et al.*, 1996). A secondary leachate collection system provides an additional means of monitoring for failure of the primary leachate collection system. A significant increase in leachate collected in this system (i.e. over and above that which would be reasonably predicted for a properly operating system) would provide a good indication of the failure of the liner and/or primary leachate collection system. However, the converse is not necessarily true (Rowe *et al.*, 1996).

It may be possible to have a failure of part of the primary leachate collection system which is not clearly evident in the flows in the secondary leachate collection system. This can

arise because the change is masked by the variability of infiltration and flows to the secondary system with time, combined with the gradual nature of the failure of the leachate collection system. An additional factor is the escape of leachate which passes through the first barrier into the secondary barrier. The sensitivity of the volume of leachate collected in the secondary leachate collection system to migration of leachate through the primary liner is least when the area of the landfill is large, when the slopes on the lower leachate collection system are small, and when the material beneath the secondary leachate collection system has a permeability of the order of 10^{-9}m/s or greater. The effectiveness can be increased by one or more of the following (Rowe *et al*., 1996):

i) Reducing the distance contaminant may have to travel before it reaches a collection point;

ii) Increasing the slopes to provide for faster transmission of leachate;

iii) Ensuring that the underlying material has as low a hydraulic conductivity as is practical (e.g. 10^{-10}m/s liner and/or the use of a geomembrane over the soil/clay liner).

4.1.6. Landfill capping and the control of infiltration

1) Geosynthetic and clay covers

A schematic of a design which uses geosynthetics as part of the leachate collection, leak detection and leachate containment systems is shown in Fig. 4.6. This schematic also shows the geosynthetic components of the landfill capping system used to control the amounts of leachate generated by rainfall infiltration. The geosynthetic capping system will typically comprise a geomembrane barrier, a geosynthetic drainage layer for rainfall collection, geotextile filter between the drainage layer and the final soil cover, and a geotextile drainage layer below the geomembrane for the transmission of landfill gases to a higher vent or collection location. These systems can be quite effective in constructing a relatively tight cap which will place a control on the infiltration into the landfill facility.

However, it is important to recognize the interrelationship between the cover designs (which controls infiltration), the contaminating lifespan of the landfill and the service life of the components of the leachate collection, leak detection and leachate containment systems (Rowe *et al*., 1996). Two key factors affecting the contaminating lifespan are the mass of waste per unit area of landfill (the height of waste) and the infiltration through the landfill cover (i.e. long-term leachate generation per unit area). Increasing the thickness of a landfill or reducing the infiltration both tend to increase contaminating lifespan. It is also important to recognize the significance of these two variables when attempting to extrapolate data from existing landfills to infer contaminant decay curves for future landfills.

2) Controlling infiltration

There are various philosophies to controlling the generation of leachate. One is to construct as impermeable a cover as possible, as soon as possible, so as to minimize the generation of leachate. This approach has the benefits of minimizing both the amount of leachate that must be collected and treated, and the mounding of leachate within the landfill; it also has the disadvantage of extending the contaminating lifespan (Rowe *et al*., 1996). Because of the heterogeneous nature of waste, some leachate will be generated almost immediately, even for landfills designed with low infiltration covers. With low infiltration, it may take from decades to centuries before the field capacity of the waste is reached and full leachate generation occurs.

This means that the full capacity of the leachate collection system may not be required for many decades after construction. However, during this period of time, degradation and biological clogging of the leachate collection system can be occurring unless the waste has been pre-treated to convert it to an essentially inert form. Furthermore, due to the variable nature of leachate generation, it may be difficult to assess whether there has been a failure of the leachate collection system by monitoring the volumes of leachate extracted.

An alternative philosophy is to allow as much infiltration as would practically occur. This would bring the landfill to field capacity quickly and allow the removal of a large proportion of contaminants (by the leachate collection system) during the period when the leachate collection system is most effective and is being carefully monitored (e.g. during landfill construction and, say, 30 years after closure). The disadvantages of this approach are two-fold. Firstly, larger volumes of leachate must be treated; this has economic consequences for the proponent. Secondly, if the leachate collection system fails, a high infiltration will result in significant leachate mounding (Rowe *et al.*, 1996).

The low infiltration philosophy is readily suited to meet environmental regulations that only require a limited period during which the landfill must not cause an unacceptable impact (be it 30 years or 100 years). However, engineers have a moral responsibility also to consider the longer-term consequences. In some areas (e.g. Ontario, Canada), there is also a regulatory responsibility to consider environmental protection in perpetuity. When long-term protection is considered, it may be desirable to find a balance between the low/high infiltration philosophies. Although it is not always practical (e.g. because of after-use requirements), one option is to allow high infiltration during construction of the entire landfill (not just the cell) and for a period after landfilling ceases. Once the landfill has been largely stabilized or the leachate collection system starts to degrade noticeably, then a low infiltration cover would be constructed.

This approach rapidly brings the landfill to field capacity and removes a substantial portion of the potential contaminants early in the life of the landfill when the engineered components are at their most reliable, thereby reducing the contaminating lifespan. This approach also reduces the infiltration and potential mounding of leachate after the low infiltration cover has been constructed, and hence minimizes potential problems once the performance of key components of the engineered system (like the primary leachate

collection system) have degraded. These factors should be considered in selecting the barrier system, in developing the monitoring and contingency plans, and in assessing the service life and contaminating lifespan of the facility. It should also be recognized that while it may be possible to construct relatively tight landfill covers that permit very little infiltration into the landfill, these covers will degrade with time due to factors such as (Rowe *et al.*, 1996):

i) Differential settlement of the waste (subsidence);

ii) Freezing and drying;

iii) Damage by insects and rodents;

iv) Degradation of synthetic material with time (due to physical, chemical and biological causes);

v) Vegetation;

vi) End –use of the finished landfill;

vii) Erosion.

Thus, in order to maintain low infiltration it may be necessary to repair or indeed replace landfill covers on a regular basis over the contaminating lifespan of the landfill. For a tight cover, this may be a very long time.

4.1.7. Choice of barrier system and service life consideration

The choice of barrier system should always be site specific and should take account of the following (Rowe *et al.*, 1996):

i) Local hydrogeology;

ii) Nature of the waste;

iii) Size of the landfill (in both vertical and areal extent);

iv) Climatic conditions;

v) Availability of suitable materials;

vi) Potential impact of failure of the system on health and safety;

vii) Local social and regulatory systems.

It must be acknowledged that there is always more to learn, and it is not possible to guarantee that a facility thought to be safe today will last, say, 500 years. On the other hand, professionals responsible for environmental management according to Rowe *et al.* (1996) should design facilities to ensure that, based on what we know today, a waste disposal facility will not cause an unacceptable environmental impact or risk to human health and safety, irrespective of when this might occur. This design philosophy recognizes that disposal sites will have a number of phases.

Firstly, during the active life of the landfill, careful quality control and monitoring will be required to ensure that the engineered features are looking as intended, if not, problems should be rectified. This will involve careful inspection of existing clayey deposits, and leachate collection systems. Double liner systems have an engineered backup which can contain much of the leachate in the event of a failure of the primary system. In other situations; the hydrogeologic setting must be selected such that it provides a backup in the event of a failure of the primary engineering. The facility should be designed such that it will not impact on the reasonable use of the groundwater or the environment during its operational life.

Secondly, it must be recognized that eventually some engineering components of the landfill may fail. The effective life of drainage and leachate collection systems and, to a lesser extent, geosynthetic liners, is the subject of much debate. Reviews of the potential for closing of leachate collection systems (Bass *et al.*, 1984) have indicated that clogging may in fact occur due to physical, chemical, biochemical and biological mechanisms. Based on their study, Bass *et al.* (1984) concluded that it is reasonable to expect clogging to occur in a probabilistic manner during the active and post-closure operational lifetime.

Notwithstanding this, the potential for clogging of the leachate collection system, and in particular the clogging of the granular component and pipes which cannot be cleaned should be considered when performing environmental impact assessments. One cannot justifiably assume that, because the pipes are clean, the leachate system is necessarily functioning correctly. Similarly, it is difficult to justify arguments that failure of the leachate collection system can be detected by monitoring the volume of leachate extracted.

Monitoring of head in the granular component of the leachate collection system is one means of monitoring its performance. However, consideration should then be given to the potential impact of mounding, which is not detected by these monitors, and the potential impact should be quantified. Depending on the results of these analyses, additional levels of redundancy may be required as part of the engineered system (e.g. in some cases it may be necessary to use a double liner with either secondary leachate collection capabilities or hydraulic control) (Rowe *et al.*, 1996).

In addition to consideration of clogging of the leachate collection system by a combination of biological and particulate clogging, reduction in transmissivity of geonet drainage systems due to time-dependent intrusion of soil should be considered. Similarly, when dealing with geomembranes, time-dependent degradation of the geomembrane should also be considered. As noted by Koerner (1990), the degradation of any geosynthetic material must come from molecular chain scission of the polymeric resin itself or bond breaking and reactions with the compounding materials. The process is too slow to follow in the laboratory and experience in the field is still too short to make long-term degradation predictions.

Degradation mechanisms include ultraviolet (UV) exposure (during construction), chemical reaction with leachate, and swelling due to chemical absorption, extraction, oxidation and biological attack (Koerner *et al.*, 1990). The severity of the degradation mechanisms listed above will govern the service life of the various geosynthetic components

of the facility. Installation damage, the magnitude of the imposed mechanical stresses, and synergism between the various mechanisms may increase the rate of degradation (Koerner *et al*., 1990).

Accelerated tests have been developed which have been used to predict service lifetimes; however, these tests are not particularly realistic. With respect to geomembranes, Koerner (1990) has noted that the service lifetimes of the various geomembranes are rarely in excess of 30 years. This should not be interpreted as meaning that geomembranes will only last 30-40 years; they are likely to last much longer with current estimates being about 200 years. However, the limited experience should be carefully considered in design.

4.1.8 Geotechnical considerations

Many aspects of geotechnical and geosynthetic engineering, beyond that directly associated with leachate collection, detection and contaminant migration; need to be considered in the design of barrier systems. Most of these issues are dealt with in standard textbooks on geotechnical engineering. Landva and Clark (1990) provide a good general discussion of the geotechnics of waste, which Landva and Knowles (1990) provide numerous articles dealing with different aspects of waste geotechnics. The objective of this section is to highlight some important considerations.

1) Settlement and bearing capacity

Differential settlement of the soil beneath a landfill, and of compacted clay liners, may reduce the effectiveness of leachate collection systems. Similarly, settlement of the cover (due to settlement of underlying waste) will reduce the effectiveness of the cover. Leachate (or water) will pond in areas where settlement occurs, increasing the hydraulic head and consequent flow through the underlying soil. With the use of geomembranes, an additional consideration is that the straining of the geomembrane over the settlement may

initiate holes or ruptures. The formation of tension cracks in the underlying compacted or natural clay barrier is also a possibility under some extreme circumstances. An expected increase in the use of vertical landfill expansions to satisfy demand for landfill space may be expected to increase the occurrence of problems related to differential settlements. The use of geogrid reinforcement over areas of potential differential settlements may reduce such impacts (Rowe *et al*., 1996).

Bearing capacity and settlement is a concern when designing manholes to be constructed within the waste areas. Bearing failure or excessive settlement of the manhole may reduce the efficiency of the system and/or damage the barrier. Factors to be considered in assessing the settlement of manholes include not only the weight of the manhole itself, but also the downdrag forces that may develop due to settlement/consolidation of adjacent waste (Biener and Sasse, 1993). There is a direct analogy with the design of piles for downdrag. One technique of avoiding large shafts in landfills is the use of tunnels beneath the waste (Geusebroek and Luning, 1993).

2) General stability

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 the following (Rowe *et al*., 1996):

i. Stability of the excavated trenches for the landfill (where appropriate);

ii. Stability during development of the landfill;

iii. Stability of the completed landfill.

Particular care is required not to underestimate the load that can be applied by partially saturated waste, and a conservative estimate of the unit weight of waste should be considered when evaluating landfill stability. This is particularly critical for landfills being constructed in soft soils, sensitive soil, near escarpments or on slopes. Landfill failures have occurred due to inadequate assessment of overall stability. A number of potential failure mechanisms are shown in Fig. 4.7.

3) Blowout of basal heave

In addition to considering general stability due to bearing capacity or slope failure, consideration should also be given to the potential for blowout (basal heave) of the bottom of the excavation. This is particularly critical for landfills being designed as hydraulic traps where the water pressure in an underlying aquifer is to be used to minimize contaminant transport from the landfill. This same water pressure (if not controlled) can cause blowout of the base of the landfill. Blowout consideration may necessitate pumping of underlying aquifers to reduce water pressure (and hence to ensure an adequate factor of safety against blowout) during excavation of a cell, placement of the barrier systems and the waste (Rowe *et al.*, 1996).

4) Stability of trenches

A landfill may be designed on a crusted soil such that the base of the landfill lies with the crust and overall stability is satisfied. However, problems may still occur if careful consideration is not given to stability of pipe trenches cut into any underlying soft, or very soft, soil. It is important that consideration be given not only to the global stability of the landfill, but also to the stability of individual components (Rowe *et al.*, 1996).

5) Stability of engineered systems on side slopes

Fig. 4.6 shows an engineered system involving geomembranes, geonet drains, geotextiles and compacted clay all extending up a side slope. Tensile forces will be mobilized in the

geosynthetic components lining the side slopes of the waste containment facility due to the waste overburdern loads, waste settlement and consolidation (downdrag forces), and from the self-weight of the geosynthetic components themselves.

The geosynthetics in the landfill capping system will also develop tensile forces due to the sliding potential of the final soil cover. The current way to evaluate these tensile forces is by static equilibrium methods (Richardson and Koerner, 1988). This evaluation requires the knowledge of the friction characteristics between the various components of the lining system. These characteristics may be estimated on the basis of direct shear friction testing. An example of a failure attributed to sliding along the interfaces within a composite, multilayered geosynthetic-clay liner system has been reported by Mitchell et al. (1990).

When interface friction alone is not sufficient to prevent sliding, either slopes must be reduced or tensile elements must be introduced to carry loads that would otherwise be expressed as shear along a potential failure surface. The geosynthetics in tension must be anchored at the top of the slope, and descriptions of anchoring methods and determination of the anchorage capacity are available (Richardson and Koerner, 1988). Fig. 4.6 shows the inclusion of geogrids to steepen the side slopes, which will increase the available landfill airpspace. Jewell (1991) presented design charts for geogrid reinforced slopes in which an increase in slope angle will be accompanied by an increase in the tensile forces mobilized in the geosynthetics lining the slope and an increased risk of sliding, unless appropriate measures are taken.

The geosynthetics in tension must have sufficient strength to withstand the tensile forces to which they are subjected, and high safety factors should be used since strength losses due to installation damage, long-term creep, and degradation mechanisms must be expected. Possible degradation mechanisms include UV exposure during construction and unprotected slopes, chemical reactions with leachate, swelling due to chemical absorption,

extraction, oxidation and biological attack (Koerner *et al*., 1990). The possibility of chemical and swelling degradation can be minimized by the selection of geosythetics chemically resistant to the leachate.

The US EPA has developed a method in which geomembrane specimens are exposed to leachate in controlled environment, followed by physical and mechanical testing. Changes in mechanical behaviour after exposure are evaluated, and the compatability of the geomembrane with the leachate is assessed. This method may be readily extended to include all geosynthetic components which may be exposed to the landfill leachate (Koerner, 1990). Similarly, compatibility of the geosynthetics above the waste with landfill gases should be assured (Rowe *et al.*, 1996).

6) Post-construction desiccation of liners

When compacted clay liners are isolated from the groundwater, consideration should be given to the potential for the liner cracking due to desiccation. The risk of cracking is lowest when the liner is constructed of clays of low activity with a high applied vertical stress (e.g. due to the weight of the waste). Conversely, the risk of cracking is greatest for highly active clays under conditions of low confirming stress (Rowe *et al.*, 1996).

4.2 Flow Modelling

The design of barrier systems typically requires some associated contaminant impact assessment. This should involve consideration of both advective and diffusive transport. The calculation of the advective components requires that an assessment be made of the flow in the barrier system. In some cases, this may be isolated from the general groundwater system (e.g. in an arid region where there is a thick unsaturated zone above the aquifer and negligible on-site recharge to the aquifer either before or after landfill construction).

However, in most cases, the construction of a landfill will involve some interaction between the engineered system and the local hydrogeology. Thus the objective of this section is to summarize a number of approaches for calculating flows in barrier systems. Initially, consideration is given to one-dimensional steady state flow systems where simple hand calculations can be used to estimate flows. In some cases, the flow system may be too complicated to use one-dimensional hand calculations. For these situations, it is usual to use an analytical model as reported by Rowe and Nadarajah (1994) or perform numerical (finite element) analyses.

4.2.1. One-dimensional flow models

Flow modelling is based on Darcy's law i.e

$v_a = ki$ ……..(4.6a)

$q = v_aA = kiA$ ……(4.6b)

$q = ki$ for $A = 1$ …….. (4.6c)

where v_a is the Darcy velocity (flux); k is the hydraulic conductivity at the location where flow is being evaluated; i is the hydraulic gradient at the same location; q is the flow at the location of interest; and when A = 1, q is flow per unit area; A is the cross-sectional area through which flow is occurring at the location of interest; A = 1 in this case. Hydraulic gradient may be defined as a change in (total) head $\Delta\Phi$ over a given distance, H (Rowe *et al*., 1996):

$$i = \frac{\Delta\Phi}{H} \qquad \text{……(4.6d)}$$

Total head is the sum of two significant components-the pressure head, *hp,* and the position head, z. Total head is always measured relative to some datum. Since flow only depends on the change in total head (over a given distance) the choice of datum is not

important. If the position head is defined as the vertical distance, z, above the datum, then the total head, Φ, at any point of interest, is given by

$\Phi = h_p + z$…… (4.7)

where Φ is the total head; h_p is the pressure head (= u/γ_w, where u is the pore water pressure and γ_w is the unit weight of water); z is the position head. In a multilayered system, it can be shown that the equivalent hydraulic conductivity of the system is given by the harmonic mean k where (Rowe *et al.*, 1996)

$$\frac{\sum H_i}{k} = \sum \frac{H_i}{k_i} \quad \text{……..…….. (4.8)}$$

where the summation is performed over all layers. In this case, the flow can then be calculated as:

$q = k_i$……. (4.9)

where i is the hydraulic gradient across the entire system. For a two-layered liner system,

$q = q_1 = k_1 (\Phi_b - \Phi_c)/ H_1$ …….. (4.10)

This approach (eqs. (4.8) to (4.10)) often represents the most convenient method of calculating a head distribution. Furthermore, i can be expressed as,

$$\frac{i1}{i2} = \frac{k2}{k1} \quad \text{.....(4.11)}$$

Thus, the change of slope in the total head distribution is directly related to the change in hydraulic conductivity between layers.

4.2.2. Analysis of two and three-dimensional flow

While the simple one-dimensional calculations are adequate for some important practical applications, there are situations where the flow regime is more complex, and it is

necessary to use more elaborate analysis techniques to model it adequately, although one can often do useful checks using simple one-dimesional hand calculations (Rowe *et al.*, 1996).

1) Governing equations

Suppose that the occurrence of some event, such as the construction of a landfill, the installation of cut-off wall or the introduction of a well or drain, disturbs the equilibrium of the pore water in the soil. At first there will be a period during which the pore water pressure and the stress distribution will change, but ultimately the pore water pressure and the stress state will reach a new equilibrium, and steady-state flow condition will be established. It is usually convenient to analyze the steady-state flow in terms of the total head, Φ, defined by eq. (4.7). For two and three-dimensional cases, the form of Darcy's law become (Rowe *et al.*, 1996):

$$\left.\begin{aligned} v_x &= k_{xx} i_x \\ v_y &= k_{yy} i_y \\ v_z &= k_{zz} i_z \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.12)$$

where $v = (v_x\ v_y\ v_z)^T$ is the Darcy velocity vector, and $i = [i_x\ i_y\ I_z]^T$

is the hydraulic gradient vector, with

$$\left.\begin{aligned} i_x &= -\frac{\delta\Phi}{\delta x} \\ i_x &= -\frac{\delta\Phi}{\delta y} \\ i_z &= -\frac{\delta\Phi}{\delta z} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.13)$$

The negative sign has been introduced in recognition that water flows from high head to low head, and so $\delta\Phi/\delta x$ will be negative for flow in the positive x-direction, and similarly for $\delta\Phi/\delta y$ and $\delta\Phi/\delta z$ in the y- and z- directions. Since a steady state has been reached, there will

be no change in the effective stress state, and thus an element of soil will undergo no deformation, and in particular will undergo no change in volume. If the pore fluid can be considered as incompressible, then continuity of flow within an element will be satisfied, provided that

$$\frac{\delta v_x}{\delta x} + \frac{\delta v_y}{\delta y} + \frac{\delta v_z}{\delta z} = 0 \qquad \text{(4.14)}$$

This is called the continuity equation. It follows from Darcy's law that the total head satisfies

$$\frac{\delta}{\delta x}\left(k_{xx}\frac{\delta \Phi}{\delta x}\right) + \frac{\delta}{\delta y}\left(k_{yy}\frac{\delta \Phi}{\delta y}\right) + \frac{\delta}{\delta z}\left(k_{zz}\frac{\delta \Phi}{\delta z}\right) = 0 \qquad \text{(4.15a)}$$

If the soil is homogeneous and isotropic, this reduces to Laplace's equation,

$$f\nabla^2\Phi = \frac{\delta^2\Phi}{\delta x^2} + \frac{\delta^2\Phi}{\delta y^2} + \frac{\delta^2\Phi}{\delta z^2} = 0 \qquad \text{(4.15b)}$$

For two-dimesional plane flow (in the x-z-plane), this becomes

$$\frac{\delta^2\Phi}{\delta x^2} + \frac{\delta^2\Phi}{\delta z^2} = 0 \qquad \text{(4.15c)}$$

2) Boundary conditions

Fig. 4.8 illustrates the flow from a leachate pond through a clay cut-off wall overlying an impermeable stratum, and into a drain. This example serves to illustrate four common types of boundary conditions (Rowe *et al.*, 1996):

i) On AB the water pressure is given by $u = \gamma_w h$ and thus for the datum shown in Fig. 4.8, $\Phi = h + z = H$ at any arbitrary point along AB. Thus the line AB is a line of constant head, or equipotential. This is a special case of what is often called a Dirichlet or first type boundary condition, where the head is specified along a boundary.

ii) The line AD is an imperemeable surface. There can be no flow across such a surface and thus the direction of flow must be parallel to AD. The line AD is called an impermeable

surface or a flow line. This is a special case of the Neumann or second type boundary condition where the flux (flow) normal to the boundary is specified; in this case, the flow across the boundary is zero.

iii) On CD the pore water pressure is equal to atmospheric pressure (usually taken as zero) and since $\Phi = +z$, the head varies linearly with distance above the datum. Such a line is called a line of constant pressure.

iv) When seepage occurs in the clay cut-off wall illustrated in Fig. 4.8, the entire soil mass does not remain saturated. The material above BC becomes unsaturated and ultimately the water drains from there to a boundary called the phreatic surface which represents the boundary between unsaturated and saturated materials. The phreatic line BC is both a line of constant pore water pressure, which is equal to atmospheric pressure and taken as zero, and a flow line. The position of this phreatic line is not known initially, and is determined by an iterative algorithm.

3) Continuity conditions

Natural soil deposits and artificial barriers (e.g.below a waste facility), often consist of a number of distinct layers, as illustrated in Fig. 4.9. At the interface of two such layers, the pore water pressure just above the interface must equal the pore water pressure just below the interface, and thus (Rowe *et al.*, 1996)

$\Phi_A = \Phi_B$ …….(4.16a)

Also, there can be no net flow into the interface, and thus

$v_{nA} = v_{nB}$ ……. (4.16b)

Any solution of eq.(4.15) must satisfy these continuity conditions.

4.3 Contaminant transport modelling

A contaminant transport model consists of the governing equations together with the boundary and initial conditions. Once the model has been formulated and the appropriate parameters have been determined, it remains to find a solution to the governing equations subject to the approapriate boundary and initial conditions. The most frequently used solution techniques can be subdivided into five broad categories; namely, analytical, finite layer, boundary element, finite difference and finite element techniques (Rowe *et al*., 1996).

4.3.1 Finite layer techniques

The finite layer technique is applicable to situations where the hydrostratigraphy can be idealized as being horizontally layered, with the soil properties being the same at any horizontal location within the layer. For these conditions, the governing equations can be considerably simplified by introducing a Laplace transform and a Fourier transform (the latter only being required for two-or three-dimensional problems). The transformed equations can then be readily solved. This procedure paralles that adopted in the development of many analytical solutions.

The difference between finite layer solutions and analytical solutions arises from the fact that in the finite layer approach, the solution is inverted numerically rather than analytically. As a consequence, it is possible to examine more complicated and realistic situations. Finite layer techniques have been described by Rowe and Booker (1985, 1987) and are available as computer programmes for $1^1/_2$-D (POLLUTE: Rowe and Booker, 1994) and 2-D (MIGRATE: Rowe and Booker, 1988) conditions.

For situations involving a clayey liner overlying a drainage layer which can be pumped, or overlying a thin natural aquifer, a reasonable initial estimate of contaminant impact can be obtained in seconds using a microcomputer and $1^1/_2$-D finite layer programmes

such as POLLUTE (e.g. Rowe and Booker, 1994). The designation of these programs as $1^1/_2$-D is intended to indicate that they consider one-dimensional transport down through aquitard layers and into the aquifer(s) or drainage layer(s), while approximately taking account of contaminant removal due to lateral flow or pumping of these permeable layers. These techniques will often provide a reasonable estimate of both the magnitude of the peak impact beneath the landfill and the time at which this occurs (Rowe *et al.*, 1996).

A more rigorous solution of this problem can be obtained using a full 2-D analysis (e.g. program MIGRATE). The 2-D approach allows one to consider impact at points outside the landfill. The full 2-D analysis can be readily performed on a micro-computer, but it does involve substantially more computation than the $1^1/_2$-D analysis. Before performing 2-D analyses, a $1^1/_2$-D analysis should always be performed to estimate the likely magnitude of impact and its time of occurrence beneath the landfill; the 2-D programme can then run for appropriate times, using the $1^1/_2$-D results as a reference. One of the advantages of finite layer techniques over conventional finite element methods is that it is unnecessary to determine solutions at times prior to the time period of interest (Rowe *et al.*, 1996).

Finite layer techniques have many of the advantages of analytical techniques. They are easy to use, and the user need not be an expert in numerical analysis. They also require minimal input, and only give results at the times and locations of interest. This technique is particularly well-suited for performing sensitivity studies to identify the potential impact of uncertainty regarding the value of key design parameters.

The technique is also well-suited for performing checks on the results of numerical analyses using finite element or finite difference techniques (Rowe *et al,* 1996). Finite layer techniques are not appropriate for modelling situations where there is a complex geometry or flow pattern which cannot reasonably be idealized in terms of horizontal layers with uniform properties.

4.3.2 Boundary element techniques

The boundary element technique is suitable for solving the advection-dispersion equation (e.g. Brebbia and Skerget, 1984). Its primary advantage over finite layer methods is the ability to model more complicated geometries. To date, boundary elements have not found wide applications in contaminant migration studies.

4.3.3. Finite difference and finite element techniques

There has been extensive research in the use of finite difference and finite element techniques for the analysis of contaminant migration through soils. These numerical techniques are likely to be used for (Rowe *et al*, 1996):

i) Calculating the steady-state flow pattern within a hydrogeologic system, thereby defining the velocity field;

ii) Calculating the rate of migration of contaminants by solving the advection-dispersion equation (after using velocities determined from (i)).

The techniques for modelling steady-state flow are well-established (e.g. Frind, 1987) and numerous commericial software packages are available. Many of the packages will run on micro-computers; however, there is a potential danger that inappropriate finite element mesh arrangements may be used simply to fit the problem onto a micro-computer. It is essential that the suitability of any finite element mesh be checked, either against a known solution, or by comparison with results obtained by substantially, refining the mesh (Rowe *et al.*, 1996). Finite element techniques provide the opportunity of modelling problems with complex geometries, complicated flow patterns, heterogeneity and nonlinearity. There is a wealth of literature dealing with different algorithms which have been proposed for solving the advection-dispersion equation. The

sheer volume of literature is itself a warning that the use of these techniques is not as simple as might first appear.

This is particularly so when dealing with problems where there are high advective velocities, low dispersivities and/or high contrast in dispersivity (e.g. Allan, 1984). While good results can be obtained, particular care is required to ensure the selection of a suitable computational algorithm, finite element mesh and time step. Of all the techniques of contaminant modelling, the two most usefull are finite layer and finite element methods. Finite layer techniques require some idealization of the problem; however, they also provide a means of quickly assessing the implications regarding different design scenarios and different assumed parameters.

The cost of performing these analyses is relatively small. The finite element method provides the most general technique for solving the governing equations and including complex geometry, velocity fields and nonlinearity. However, use of the approach requires an experienced user, as well as relatiavely large data preparation and computational cost.

4.3.4. Analytical solutions

In the preliminary investigation of contaminant migration there is often insufficient information to warrant more than a simple idealization of the physical situation. In some cases one may be able to formulate the problem in such a way that an analytical solution is possible. Analytical solutions are ideal for making quick premilinary calculations, valuable in performing sensitivity studies and useful in checking the results of more sophisticated analyses. Numerous solutions have been reported in the literature (Ogata and Banks, 1961; Ogata, 1970, Rowe and Booker, 1985; Booker and Rowe, 1987). Although these analytical solutions have sometimes been expressed in graphical form suitable for hand calculations as

reported by Ogata (1970) as well as Booker and Rowe (1987), it is often more convenient to use a computer.

The computation involved in such calculations is usually not particularly sophisticated, and is made easier because of the existence of efficient approximations for many mathematical functions which can often be evaluated on programmable calculators (Abramovitz and Stegun, 1972), and because of robust readily portable subroutines, which can be incorporated into relatively simple programmes which can be run on a modest personal computer.

1) Solutions for deep deposits (pure diffusion)

In the absence of advection and first order decay, the equation governing contaminant migration reduces to a simple diffusion or heat flow equation. Thus the extensive literature on heat flow by Carslaw and Jaegar (1959) provides a rich source of analytical solutions. Fig.4.10 shows a transmissive layer ($-H/2 < z < H/2$) which is bounded by non-transmissive strata. A portion of the layer ($-a < x < +a$) has become contaminated. If the initial concentration in the contaminated zone is c_0, then c can be expressed as (Rowe *et al.*, 1996):

$$c = \frac{c_o}{2}\left[\operatorname{erfc}\left(\frac{|x| + a}{2(D^* t)^{1/2}}\right) - \operatorname{erfc}\left(\frac{|x| - a}{2(D^* t)^{1/2}}\right)\right] \qquad \ldots\ldots(4.17)$$

where $D^* = nD/(n + \rho K_d)$.

The concentration may be evaluated either by using tabulated values of the complementary error function, erfc (x), or by employing a polynomial approximation (Abramowitz and Stegun, 1972) or by using the graphical values of the function

$$\Phi(X) = e^{x^2} \operatorname{erfc}(X) \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4.18)$$

The variation of the concentration profile with time is shown in Fig. 4.11, and, as would be expected, this figure shows a reduction of concentration within the initially

contaminated zone as contaminant is transported by diffusion to the initially uncontaminanted portion of the transmissive layer. Fig. 4.12 shows a deep deposit which contains a contaminated spherical zone (R<a). Initially, the contaminant concentration in the spherical zone is c_0; analysis of the diffusion equation shows that subsequently the concentration is given by:

$$c = \frac{C_O}{2}\left\{ erfc\left(\frac{R-a}{2(D*t)^{1/2}}\right) - erfc\left(\frac{R+a}{2(D*t)^{1/2}}\right) - 2\left(\frac{D*t}{\pi R^2}\right)\exp\left(\frac{-(R-a)^2}{4D*t}\right) - \exp\left(\frac{-(R+a)^2}{4D*t}\right)\right\} \text{.....(4.19)}$$

2) Solutions for deep deposits (advection and diffusion)

The solutions derived in the previous subsection can be modified to incorporate the effect of advection. The modification depends on the following observation. Suppose that

$c = F(x, y, z, t,)$…… ……(4.20a)

is a solution of the transport equation with no advection ($v_x = 0$, $v_y = 0$, $v_z = 0$) and no first order (e.g. radioactive or biological) decay ($\lambda = 0$); then a solution to the equation of contaminant transport with advection and first order decay is found to be (Rowe *et al.*, 1996):

$c = e^{-\wedge t} F(x - v_x*t, y - v_y*t, z - v_z*t, t)$…… (4.20b)

where

$v_x^* = nv_x/(n = {}_\ell K_d)$

$v_y^* = nv_y/(n = {}_\ell K_d)$

$v_z^* = nv_z/(n = {}_\ell K_d)$

$\wedge = n\lambda/(n = {}_\ell K_d)$

The solution given by eq. (4.20b) obeys identical initial conditions ($t = 0$) to the solution given by eq. (4.20a). There is no guarantee that the boundary conditions of the problem will be satisfied; however, in problems involving deep deposits where the boundary conditions are especially simple, the exact solution is found to be given by eq. (4.20b).

To illustrate the application of this method, return to the situation shown in Fig. 4.10, and suppose that there is a groundwater velocity, v, in the transmissive layer. The distribution of contaminant is then found to be (Rowe *et al.*, 1996):

$$c = \tfrac{1}{2}c_0\, e^{-\wedge t}[\mathrm{erfc}(X_m)-\mathrm{erfc}(X_p)] \quad \ldots\ldots (4.21)$$

where

$$X_m = \frac{|x-v*t|-a}{2(D*t)^{1/2}}$$

$$X_p = \frac{|x-v*t|+a}{2(D*t)^{1/2}}$$

Similarly, it is possible to determine the solution for the initially contaminated spherical zone shown in Fig. (4.12) when there is a uniform advective velocity v in the x-direction. It then follows from eq. (4.19) and eq. (4.20) that

$$c = c_{0e}^{-\wedge t}\left[(\mathrm{erfc}(R_m) - \mathrm{erfc}(R_p)) -2(D*t/\pi R^2)^{1/2}(\exp(-R_m^2)-\exp(-R_p^2))\right] \ldots.(4.22)$$

$$R = [(x-v*t)^2 + y^2 + z^2]^{1/2}$$

$$R_m = \frac{R - a}{2(D*t)^{1/2}}$$

$$R_p = \frac{R + a}{2(D*t)^{1/2}}$$

4.4. Typical Examples of Field Diffusion

This section presents a typical example of each of long-term (about 10,000 years) and short- term (3 to 22 years) field diffusion landfill.

4.4.1. Freshwater clay (example of long-termed field diffusion)

A typical example of a natural diffusion profile is shown for chloride in Fig. 4.13 (Rowe and Sawicki, 1992). At this site, 34 m to 39 m of fine-grained glaciolacustrine deposit consisting primarily of silts and clays overlie bedrock of the Salina Formation which is composed of soft, erodible shales with evaporites (gypsum) and harder dolostone layers. Based on chemical analyses from four wells in the bedrock, the average bedrock chloride concentration is about 1400 mg/1. The observed concentration extends upward from the bedrock with decreasing concentration towards the surface.

Radiocarbon dates of 12000 to 13000 years have been reported (Lewis, 1969) for sediments deposited in glacial Lake Warren (the source of this clay unit). Based on the observed gradients and measured hydraulic conductivity values, the downward Darcy velocity is estimated to be between 0.0004 m/a and 0.0016m/a. Laboratory diffusion tests give a diffusion coefficient of 3.8×10^{-10} m^2/s (0.012 m^2/a) (Rowe *et al.*, 1996). Fig. 4.13 shows four calculated concentration profiles for three combinations of parameters and two times. A prediction based on the velocity of 0.0004m/a (curve 1) gives a reasonable fit to the observed concentration profile for 12000 years of diffusion. The potential importance of downward advection is shown by comparing curve 1 (v_a = 0.0004m/a) and curve 2(v_a = 0.0016m/a) for a diffusion coefficient of $3.8 \times 10^{-10} m^2/s$ (0.012 m^2/a). Curve 2 does not provide a good fit to the data, and this implies that the downward advective velocity must have been very low for most of the past 12000 years.

To illustrate the sensitivity of results to diffusion coefficients and time, curves 3 and 4 show the predicted profile for a diffusion coefficient of $5.7 \times 10^{-10} m^2/s$ ($0.018 m^2/a$) and times of 12000 and 10000 years. Based on these analyses, it appears that diffusion is generally consistent with a diffusion coefficient of between $3.8 \times 10^{-10} m^2/s$ (0.012 m^2/a) and $5.7 \times 10^{-10} m^2/s$ (0.018 m^2/a) (Rowe *et al.*, 1996).

4.4.2. Keele Valley (example of short-term field diffusion)

A chloride ion diffusion profile and a series of cation diffusion profiles have been obtained at Metropolitan Toronto's and Keele Valley landfill site (King *et al.*, 1993). The chloride data obtained from liner samples exposed by an exhumation are presented in Fig. 4.14. The water samples for analysis were centrifuged from the sand layer and squeezed from the clay liner. The results for chloride show a smooth curve starting at the top of the sand layer and extending a total distance of 70-75cm over a period of 4.25 years. A diffusion profile fitted to the data, assuming negligible advection, yielded a field value for $D_{Cl} = 6.5 \times 10^{-10} m^2/s$ ($0.02 m^2/a$) (Rowe *et al.*, 1996).

The most important aspect of the chloride curve is that diffusion starts at the top of the partly clogged sand, which appears to be acting as part of the barrier, adding an extra 30cm of thickness with respect to diffusion. This is good from the perspective of diffusive transport, but not so good from the perspective of leachate mounding. Fortunately, the hydraulic conductivity of the clay liner is so low (5×10^{-11} m/s) that advection is small, even with the leachate mounding that has occurred to date.

CHAPTER FIVE

MATERIALS AND METHODS

5.1 Geology of the Sampling Sites

Zaria and its environs are underlain by the Basement rocks of Precambrian age. They are mainly granites and gneisses and a few schists (see Fig. 5.1). Granitic intrusions form a suite of batholith (the Zaria batholith) part of which outcrops as the Kufena hill (Kogbe, 1973). The gneisses are found as small belts within the granitic intrusions, and also found east and west of the batholith. The gneiss extends westwards to form a gradational boundary with the schist belt. The gneiss also extends eastwards to some extent and is occasionally broken up by the Older Granite whose intrusion was supposed to have formed at the bottom of a fold mountain belt.

Furthermore, the thrusting imposed on the Basement Complex during the Pan African Orogeny is believed to have brought together rocks of different ages with different structural and metamorphic styles. The metasediment belongs to the sedimentary and granite facies that were probably formed in a geosynclinal trough, which had earlier developed at the end of the Precambrian Orogeny.The geological map of Zaria environs, is presented in Fig. 5.2. There are extensive formations of different types of laterites formed in-situ over the basement rocks in many places of the tropics. Laterites are generally formed from limestones when the rainwater leaches out the soluble rock materials leaving behind the insoluble hydroxides of iron and aluminum.

The various rock units, especially the gneisses and schists have been affected by many periods of orogenic movements resulting in extensive deformation and migmatisation (McCurry, 1973). McCurry (1973) identified two folds; one trending East-West, while the

other North-South directions, and having similar orientation and interpretative history. Chukwuike (1978), in the study of the regional tectonic setting of North Western Nigeria, considered the East-West trend as the earliest basement folds which was later refolded severely along North-South axis. The East-West folds are now rarely preserved as low granitic quartzite ridges. The major rock units identified by Oluyide (1995) are; (i) Migmatite Gneiss Complex, (ii) Meta-Sedimentary Blocks (iii) Pan African Granites, and (iv) Basalts.

5.1.1 Migmatite Gneiss Complex

The migmatite gneiss complex constitutes the highest group of rocks occurring Kaduna State as a whole. The occurrence spreads from east of Birnin Gwari Local Government Authority to Jema'a Ikara Local Government Authority. The complex consists of migmatites, biotite and granitic gneisses. The migmatite gneiss complex represents reactivated older meta-sediments, the oldest being the Kaduna migmatite (Oluyide, 1995). Hence, they are characterized by a variety of structures and textures. Numerous pegmatites and quartzo-feldspathic veins cross-cut the migmatites. The migmatites are poor sources of economic minerals while the invading pegmatite veins are usually barren. However, gneisses are the most abundant rock type of the complex and are of various kinds. Lineated and banded gneisses occupy a large area. The lineated biotite gneiss is very uniform in character and strike while foliations are remarkably constant in direction (Oluyide, 1995).

5.1.2 Meta-Sedimentary Blocks

These are metamorphosed sedimentary and meta-volcanic rocks, which occur predominantly in the southeastern part and to a small extent in Jema'a area of Kaduna State. The rock group consists of ferruginous quartzite (banded iron formation) amphibolites, pelitic, slightly migmatite schists (mixed flaggy quartzites, gneisses and pegmatoids) all of which have been grouped as Birnin Gwari formation (Chukwuike, 1978). Undifferentiated schists occur as isolated narrow bodies within the migmatite gneiss complex especially in

Zaria area stretching to Malumfashi, south of Kaduna and Jema'a areas. The bodies have been grouped together as Malumfashi formation and they are more migmatised and feldspathic than the western equivalents (Oluyide, 1995). The meta-sedimentary blocks host about 90% of the mineral occurrences (i.e. gold, iron ore, talc, and kyanite) found in Kaduna State. They also include fine-grained dense greenish rocks not necessarily schistose. The schistose are associated with schist of sedimentary origin and certain peculiar conglomerates.

5.1.3 Pan African Granites

The granitoids constitute the second largest group of rocks in Kaduna State. They are characterized by lofty topography and inselbergs as exemplified by Kusheriki granites in the southwestern part, the Zaria granite inselbergs in the central part, and Kafanchan-Kagoro range of hills in the eastern parts (Ike, 1988).They were emplaced during the Late Paleozoic Era, and include coarse-grained porphyritic ganites, biotite hornblende granites and fayalite-quartz monzonites.

5.1.4 Basalts

Basalts are extrusive igneous rocks, which are flat lying and hence form an extensive plane land. The basaltic rock is confined to the eastern borders of Kaduna State, especially in Jema,a Gwandu area (Ike, 1988).Weathered basalts provide very rich agricultural land. The majority of the gemstone occurrences in Kafanchan areas are won in the vicinity of the basaltic formations. The overburden in the Basement Complex is fine-medium grained and fairly undulated sandy soils.

The surface soils are normally greyish-brown and fine-grained. They contain about 50% of sand, and are capped at high elevations by patches of laterised terraces, which compose of iron-oxides concentration and are broken up by concretionary iron stones and some quartz (Oluyide, 1995). The stress of the folding episode that occurred here gave rise to two sets of faults, one associated with shattered belt fault trending southeast and the other

east-south-east directions (Ike, 1988). After the folding of sediments and formation of schist, a period of granitic intrusion followed. The granites vary considerably in their petrology and chemical composition and belong to intrusive cycles (Oluyide, 1995).

5.2 Soils

Disturbed reddish brown lateritic samples were collected from seven different sites around Basawa and Shika areas, Zaria (Latitude 11^0 15'N and Longitude 7^0 45'E). These soils, designated as Laterites A-G, are ferruginous, and were formed as a result of weathering and leaching of in-situ basement rocks which are predominantly igneous and metamorphic rocks (Osinubi, 1998a). To achieve the objectives of this study, laboratory tests were carried out, that included natural moisture content, specific gravity, particle size distribution, Atterberg limits, hydrometer analysis, and cation exchange capacity (CEC), mineralogical composition using differential thermal analysis (DTA), compaction tests, hydraulic conductivity tests, suction tests for soil water characteristics curves (SWCC), batch equilibrium tests, and diffusion tests. The methodology used in each case is outlined in the subsequent sections.

5.2.1 Index Properties and Mineralogical Composition Tests

The index properties were determined by following standard procedures as outlined in BS 1377 (1990) and Head (1992). For the particle-size distribution tests, gravel was defined as particles larger than 4.8mm (No 4sieve) and fines were particles smaller than 0.075mm (No 200 sieve), which are definitions of Unfied Soil Classification System (USCS). Clay content was defined as the percentage of particles smaller than 2microns (as determined using sedimentation analysis) which is the common definition used in geo-environmental engineering practice.

5.2.2 Natural moisture content and specific gravity of the soils

The standard method for determining the moisture content of soils is the oven-drying method (Head, 1992). Hence, tests were conducted using the standard moisture content apparatus and procedure in accordance with BS 1377:Part2: 1990. A drying temperature of 105-110^{0}c was utilized. To calculate the moisture content of any soil sample, a simple procedure as outlined in Head (1992) was used.

$$\text{Moisture content (\%)} = w = \frac{m_2 - m_3}{m_3 - m_1} \times 100\% \quad \ldots\ldots\ldots\ldots\ldots\ldots (5.1)$$

where m_1, m_2, and m_3 are weight of container (g), weight of soil and container (g), and weight of dry soil and container (g) respectively.

For the specific gravity or particle density tests, the density bottle (small pyknometer) method is the traditional method for the accurate measurement of the density of particles heavier than water, or of the density of liquids (Head, 1992). This method is particularly suitable for fine-grained soils like many laterites. Tests were also conducted in accordance with BS 1377:Part2: 1990. Water was used as the density bottle fluid throughout. To calculate the specific gravity of any soil sample, a simple equation was used as follows (Head, 1992):

$$G_s = \frac{\rho_L (m_2 - m_1)}{(m_4 - m_1) - (m_3 - m_2)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.2)$$

where G_s is the specific gravity of the soil sample; ρ_L is the density of the liquid used at a constant temperature (for distilled water, it may be assumed to be equal to 1.000g/ml for most purposed); m_1 is mass of density bottle; m_2 is mass of bottle plus dry soil (g); m_3 is mass of bottle plus soil plus liquid (g); m_4 is mass of bottle plus liquid only (g).

For both the natural moisture content and also the specific gravity tests, three samples of the soil were used, and their averages taken as shown in Table 5.1. Test results for Laterites A-G are as shown in Appendices A and B.

Table 5.1. Natural moisture content and specific gravity of the soils.

S.No	Soil Type	Average natural moisture content [%]	Average specific gravity
1	Laterite A	7.45	2.74
2	Laterite B	6.60	2.93
3	Laterite C	6.70	2.64
4	Laterite D	6.80	2.87
5	Laterite E	6.21	2.80
6	Laterite F	7.12	2.80
7	Laterite G	6.84	2.69

5.2.3 Particle-size distribution of the soils

If a soil contains silt or clay, or both, even in small quantities, it is necessary to carry out a wet sieving procedure in order to measure the proportion of fine mineral present (Head, 1992). Even when dry, fine particles of silt and clay can adhere to sand-size particles and cannot be separated by dry sieving. Washing the soil through sieves is the only practicable means of ensuring complete separation of fines for a reliable assessment of their percentage. If clay is present, or if there is evidence of particles sticking together, the soil must be well soaked or immersed in a dispersant solution before washing. After three days of soaking, the

seven lateritic samples were washed using 2mm sieve nested in 63micron sieve before oven drying, cooling, and weighing. Generally, the apparatus and procedure used were in accordance with BS 1377:Part2: 1990. The results of sieve analysis were presented for the seven laterites as shown in Appendix C.

Hydrometer Analysis:

In this method, a density hydrometer was used to measure the density of a soil in a suspension in water at various intervals of time. From these measurements, the distribution of particle sizes in the silt range (60-2microns) can be assessed. The test is not usually performed if less than 10% of the material passes the 63microns sieve (Head, 1992). The apparatus and procedure used were in accordance with BS 1377:Part2: 1990. The sedimentation test data for all the laterites is presented in Appendix D, while the particle-size curve from hydrometer test and related sieving for all the laterites are presented in Figs. 6.1(a) and (b).

In order to accomplish the sedimentation analysis, the following terms were defined as (Head, 1992):

i) Effective depth, $H_R = 214\text{-}3.9R_h$,…….…… (5.3)

where $R_h = R^1_h + C_m$

C_m is the meniscus correction

ii) Modified reading, $R_d = R^1_h - R_o$…….. (5.4)

where R_o is the datum correction

iii) Particle diameter, $D = 0.005531\ [\eta H_R/(-1)t]^{½}$..... .....(5.5)

where η = viscosity of water; t is the elapsed time.

iv) Percentage finer than D, $K = \frac{\rho_s \times R_d \times 100}{m(\rho_s\text{-}1)}$…… …….. (5.6)

where ρ_s is the particle density; m is the initial sample dry mass.

5.2.4 Atterberg limits

The liquid limit is the water content at which the soil has such a small shear strength that it flows to close a groove of standard width when jarred in a specified manner. The plastic limit is the water content at which the soil begins to crumble when rolled into threads of specified size (Smith, 1978). The plasticity of a soil is measured by the plasticity index, which is equal to the liquid limit minus the plastic limit. The procedures used for these tests) were in accordance with B.S 1377:Part2: 1990, and Head (1992). The liquid limit (LL) can be expressed as (Head, 1992):

$$LL = W_N \left(\frac{N}{25}\right)^{0.121} \quad \ldots\ldots (5.7)$$

where W_N is the water content of the soil which closes in N blows in the standard liquid limit device. From the laboratory data, the liquid limit is the moisture content corresponding to 25 blows.

The plasticity index, I_P, = LL-PL …… …….. (5.8)

where PL is the plastic limit.

Group Index = (X-35) [0.2+0.005(LL-40)] + 0.01(X-15)(Ip-10)…….(5.9)

where X = percentage passing No. 200 sieve, expressed as a whole number.

Another important index property of natural clay soils is the liquidity index, I_L, defined as:

$$I_L = \frac{w\text{-}PL}{LL\text{-}PL} = \frac{w\text{-}PL}{Ip} \quad \ldots\ldots\ldots\ldots (5.10)$$

The results of the Atterberg limit tests for all the seven laterites were presented in Appendix E.

5.2.5 Cation Exchange Capacity (CEC)

The Atterberg limits vary with the amount of clay present in a soil, the type of clay mineral, and the nature of the ions adsorbed on the clay surface. It has been noted by Peck *et*

al. (1974) that differences in the atomic structure of clay minerals lead to differences in the electric charges on the clay surfaces. The existence of such charges is indicated by the ability of clays to adsorb ions from solutions. Cations are more readily adsorbed than anions; hence, negative charges must be predominant on the clay surfaces.

A cation, such as Na^+ is readily attracted from a salt solution and attached to the clay surface. However, the adsorbed Na^+ ion is not permanently attached; it can be replaced by K^+ ions if the clay is placed in a solution of potassium chloride (KCL). The process of replacement by the excess cation is called cation exchange. The cation exchange capacity of the different types of clay minerals may be measured by washing a sample of each with a solution of a salt not commonly found in nature, such as ammonium chloride (NH_4Cl), and determining the amount of adsorbed NH_4^+ by measuring the difference between the original and final concentration of the washing solution (Peck *et al*., 1974).

It is convenient to express the cation exchange capacity in terms of the number of equivalent weights of an ion adsorbed per 100g of clay mineral, since this factor is independent of the weight of each adsorbed ion and the number of charges associated with it.This test was carried out at Department of Soil Science, Ahmadu Bello University, Zaria. The results of the cation exchange capacity test for the seven laterites are summarized in Table 5.2(a).

5.2.6 Differential thermal analysis (DTA)

On heating, reactions involving crystallization, oxidation and some other chemical reactions are exothermic. An exothermic peak means that the temperature of the specimen being analyzed has become higher than that of the inert reference material (usually alumina) because the reaction produces heat. Phase changes, dehydration, decomposition, and crystalline inversions are generally endothermic (Hutchison, 1974).

An endothermic peak indicates that the specimen has absorbed more heat than the reference material during the reaction. The difference in temperature between the specimen and inert reference material will cause a current to flow in the differential thermocouple. The direction of current flow determines whether the deflection is upward (exothermic) or downward (endothermic) on the chart. The more energetic the reaction, the greater the current flow, hence, the greater the peak.

The DTA curve, differential thermogram, (differential temperature, DT, versus furnace temperature, t) will be fairly reproducible for any mineral on a particular machine under identical operating conditions. The mass of the reacting material is approximately proportional to the area under the curve. The recognition of departure from and return to base line is most important when quantitative estimations are required (Hutchison, 1974).

The DTA method, unlike the X-ray powder diffractometry, ordinarily cannot be used on a completely unknown material to identify its mineralogy from the record because the curves of different components overlap and there is also a chance of reaction between components. Any reaction in the sample will give a peak. Using the apparatus and procedure stated in Hutchison (1974), the seven lateritic specimens were subjected to DTA tests. The results for all the seven laterites were presented in Appendix F1. The thermograms (see Appendix F2) for the seven laterites were interpreted as described by Hutchison (1974), and their results were summarized in Table 5.2(b).

5.2.7 Compaction tests

Compaction tests were carried out in accordance with BS 1377:1990:Part3. The relationships between the moisture content, bulk density and dry density are as follows (Smith, 1978):

$$\gamma_b = \frac{W_W + W_S}{V} \quad \text{.....(5.11)}$$

$$\gamma_d = \frac{W_S}{V} \quad \text{.....(5.12)}$$

$$w = \frac{W_W}{W_S} \quad \text{.....(5.13)}$$

where γ_b is the bulk density of the soil, γ_d is the dry density of the soil; w is the moisture content of the soil; W_W is the weight of water in soil; W_S is the weight of dry soil; V is the volume of mould (or soil in the mould). From eq. (5.13),

$$W_W = WW_S$$

Substituting the above expression in eq. (5.11),

$$\gamma_b = \frac{W_S}{V}(1 + w) \quad \text{.....(5.14)}$$

$$\gamma_d = \frac{\gamma_b}{1+w} \quad \text{.....(5.15)}$$

A compaction curve is not complete without the addition of air voids lines. An air voids line is a (curved) line showing the dry density-moisture content relation for soil having a constant percentage of air voids. A set of air voids lines can be drawn from calculated data if the particle density of the soil grains is known. A relationship between the air voids, V_a, and the dry density, γ_d, is (Head, 1992):

$$\gamma_d = \frac{(1-\frac{V_a}{100})}{\frac{1}{\rho_s} + \frac{w}{100}} \quad \text{.....(5.16)}$$

where ρ_s is the density of soil solids.

For a case of fully saturated soil (no air voids), $V_a = 0$. Hence,

$$\gamma_{d(sat)} = \frac{1}{\frac{1}{\rho_s} + \frac{w}{100}} \rho_w \quad \text{.....(5.17)}$$

This equation defines the zero air avoids line, or the saturation line. It is impossible for a point on a compaction curve (in terms of dry density) to lie to the right of this line, whatever the degree of compactive effort applied. Prior to compaction, lateritic soil specimens were crushed to pass sieve 4.8mm openings as outlined in Head (1992). Gravel and other particles too large to pass through this sieve 4.8mm openings were discarded. Generally, the percentage of gravel-size particles was small, being less than 8%, and more commonly 0-2%.

Specimens of each soil were compacted at various molding water contents using three compactive efforts namely, BS Light (BSL), West African Standard (WAS), and BS Heavy (BSH). The WAS compaction involves using a 4.5kg rammer falling through 45.72cm onto five layers in a BS compaction mould with each layer receiving ten blows (Osinubi, 1998b). The compaction plots showing the relationship between the dry unit weight and moisture content for all the laterites are presented in Appendix G.

5.2.8 Hydraulic conductivity tests

The permeability of a soil is a measure of its capacity to allow the flow of a fluid through it. The fluid may be either a liquid or a gas, but soils engineers are concerned only with liquid permeability, and the liquid is usually water or water-related solution. Generally, some of the voids in soils are interconnected, which enables water to pass through them; that is, soils are permeable to water. The degree of permeability is determined by applying a hydraulic pressure difference across a sample of soil, which is fully saturated, and measuring the consequent rate of flow of water (Head, 1992). The coefficient of permeability is expressed in terms of a velocity. The flow of water through soils of all types, from free-draining gravels and sands to impervious clays, are governed by the same physical laws.

The method used for measuring permeability depends upon the characteristics of the material. The two types of laboratory test for the direct measurement of the permeability of

soils are constant head test (for soils of high permeability such as sands), and falling head test (for soils of intermediate and low permeability such as silts and clays). In a laboratory constant head permeability test, the hydraulic conductivity, k, can be expressed as (Smith, 1978):

$$k = \frac{q}{Ai} = \frac{Ql}{tAh} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.18)$$

where q is the quantity of unit flow; A is the area of a sample; i is the hydraulic gradient; Q is the quantity of flow passing through an area A in time t; h is the head loss between two points along the length of the sample distance, l, apart. For the falling head permeability test, the hydraulic conductivity, k, can be expressed as (Smith, 1978):

$$k = 2.3 \frac{al}{At} \log_{10} \frac{h_1}{h_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.19)$$

where a is the cross-sectional area of the stand pipe; l is the length of a sample; h_1 is the initial height in the standpipe; h_2 is the height after time t. Throughout the hydraulic conductivity tests in this research work, eq. (5.19) was used to derive the hydraulic conductivity, k.

Using the three compactive efforts, samples for hydraulic conductivity tests were compacted into five sets (i.e specimens compacted at the optimum moisture content, specimens compacted at $\pm$ 2% of OMC, and specimens compacted at $\pm$ 4% of OMC). The specimens were placed in rigid-wall permeameters for hydraulic conductivity testing. All tests were conducted using the falling-head method in accordance with the procedures described in Head (1992). No backpressure was applied. Testing was continued until no trend in the hydraulic conductivity was apparent, the last four measurements of the hydraulic conductivity varied by no more than 25%, and inflow was equal to outflow. Tap water was used as the permeant.The results of the hydraulic conductivities as they vary with the molding water contents are presented in Table 5.3 for all the seven laterites.

Compaction control criteria

In the field, the two criteria normally used for compaction with a view of achieving quite low hydraulic conductivities are: (i) Common criterion (traditional approach), and (ii) Modern criterion (Benson and Boutwell, 1992).

(i) Common Criterion**:** The compaction criterion commonly used for the construction of clay liners generally includes two parts: a minimum dry unit weight and a range of acceptable water contents. Common requirements are:

For dry unit weight**:** Requirements for minimum dry unit weight are typically stated as a percentage of the maximum dry unit weight varying between 90% of the maximum dry unit weight based on standard Proctor to 95% of the maximum dry unit weight based on modified Proctor. The most common requirements are 95% of the maximum dry unit weight based on standard Proctor or 90% based on modified Proctor.

For water content**:** The water content is generally required to be wet of optimum, with optimum being defined by standard or modified Proctor effort. Allowable ranges vary from 0 to 2% wet of optimum to 2 to 6% wet of optimum, where the optimum water content depends on the compaction method. The most common requirement is 0 to 4% wet of the optimum based on standard or modified Proctor effort (Benson and Boutwell, 1992).

The common criterion has two main deficiencies (Daniel and Benson, 1990). First, it is often too restrictive; liners with low hydraulic conductivity have been built with combinations of water content and dry unit weight that fall outside the common criterion. This tends to occur when high dry unit weight is achieved at water content less than a specified optimum or when high water contents are used. Second, the common criterion permits combinations of water content and dry unit weight that fall dry of the line of optimums (see Fig. 5.3.), a condition known to yield higher, scale-dependent hydraulic conductivities.

(ii) Modern Criterion: A more sophisticated approach to compaction control is to develop an acceptable diagram, a technique proposed by Mundell and Bailey (1985). A window of acceptable combinations of water content and dry unit weight is identified in the compaction plane that results in sufficiently low hydraulic conductivity while maintaining other desirable properties such as shear strength (see Fig. 5.4.). The modern criterion is more reliable in yielding the required low field hydraulic conductivities, and, therefore, recommended by many researchers (Mundell and Bailey, 1985; Boutwell and Hedges, 1989; Daniel and Benson, 1990). By following the recommended procedure, it is quite possible to delineate an overall acceptable zone for any of the seven laterites considered in this study.

5.2.9 Volumetric shrinkage tests

Before the compaction of the various specimens of all the seven laterites, the lateritic soils were crushed to pass the sieve 4.8mm openings. Gravel and other particles too large to pass through this sieve size 4.8mm were rejected. However, only a very small percentage (much less than 3%) fell into this category that was rejected. Using tap water for mixing, desiccation test specimens were compacted the same way those for hydraulic conductivity were prepared using the BSL, WAS, and BSH compactive efforts.

Specimens were subsequently extruded carefully from the compaction moulds, weighed, and left to dry at room temperature (25^{o}C) on laboratory tables. An average of three measurements of the height, diameter, and weight of each specimen was made daily for the computation of the volumetric shrinkage strain until there was no longer decrease in the weight of the specimen. After full desiccation, specimens were oven-dried and weighed so as to obtain values of water contents and corresponding dry unit weights. The volumetric shrinkage strain can be expressed as (Smith, 1978):

Volumetric strain = $\frac{\text{change in volume}}{\text{original volume}} = \frac{\Delta V}{V}$ ……..(5.20)

Volumetric shrinkage, % = $\frac{\Delta V}{V}$ x 100…… …….(5.21)

where $\Delta V = \frac{\pi D^2}{4} - \frac{\pi d^2}{4}$

D is the initial diameter of a specimen; d is the diameter after desssication. The results of the volumetric shrinkage strain for all the seven laterites are summarized in Table 5.4.

5.2.10 Unconfined compressive strength tests

Unconfined compressive strength (UCS) tests were conducted on the seven lateritic samples previously mixed with tap water and compacted at various molding water contents using the three compactive efforts (BS Light, WAS, and BS Heavy) in ways similar to specimens compacted for hydraulic conductivity tests. Compacted samples were next extruded carefully, sealed with plastic bags and allowed to equilibrate for at least 24 hours before trimming for the UCS tests. At least, three trimmed specimens (38mm by 76mm) per molding water content for each compactive effort were used. Generally, the tests were carried out in accordance with BS 1377: Part 7:1990 and Head (1992). The change in specimen's length, L, can be expressed as (Bishop and Henkel,1982):

L= a-b ---------- (5.22)

where a is movement of the top of the proving ring = K(in mm per revolution) x number of revolutions. K is a property of the gear box, and is found by caliberation. b is the compression of the providing ring = proving ring dial (mm)- initial proving ring dial reading. The axial strain, E, can be expressed as:

$E = \frac{L}{L_o}$ ……(5.23)

where L_o is the initial specimen length.

The average cross-sectional area, A, at any point in time is:

$$A = \frac{A_o}{1-E} \quad \text{.................. ....(5.24)}$$

where A_o is the initial cross-sectional area of specimen with a diameter of 38.1mm.

$$A_o = \frac{\pi(38.1)^2}{4(10)^6} = 1.14x10^{-3}m^2$$

$$\text{The applied stress} = \frac{P}{A} \quad \text{.......................... ....(5.25)}$$

where P is the applied force

The compressive strength, q_u, can be expressed as (Head, 1992):

$$q_u = \frac{R \text{ x } C_r (100\text{-}E\%) \text{ x } 1000 \text{ x } 6.895}{100 \text{ x } A_o} \quad \text{........................(5.26)}$$

where R is the reading obtained; C_r is the ring constant;

$$E\% = f\,\frac{\Delta L}{Lo}$$

The laboratory data sheets for the unconfined compressive strength (UCS) test for the seven laterites are presented in Appendix H, while a summary of all the results for the seven laterites and using the three compactive efforts were presented in Table 5.4.

5.2.11 Soil water characteristics curves (SWCC)

Compacted clays are often used in landfills and other waste-containment facilities as barriers to water and contaminant flow. Because compacted soil liners are frequently unsaturated, modelling of flow and transport requires knowledge of their unsaturated hydraulic properties (Chiu and Shackelford, 1994; Wang and Benson, 1995). This is particularly true in earthen landfill covers, which are normally unsaturated (Khire *et al.*, 1997). One necessary property is the soil-water characteristic curve (SWCC), that is the

relationship between matric suction (ψ) and water content [gravimetric (ω) or volumetric (θ)] or degree of saturation (S).

An SWCC is J- or S-shaped, and is hysteretic; that is, for a given water content, higher matric suctions exist for desorption (drying) than sorption (wetting). Hysteresis is caused by size differences between the primary pores and the interconnecting pore throats, changes in the contact angle during wetting and drying, and trapped air (Tinjum *et al.*, 1997).

Because of the experimental difficulties inherent in measuring the sorption curve, only the desorption portion of the curve is usually measured (Hillel, 1980).

The matric suction corresponding to the break in the curve (near the saturated water content θ_s) generally is referred to as the air-entry suction (ψ_a). The air-entry suction corresponds to the matric suction required to remove water from the largest pores (Brooks and Corey, 1966). The water content corresponding to the asymptote of the SWCC at low degrees of saturation is called the residual water content (θ_r). The shape of the curve is a function of soil type. Soils with smaller pores have higher ψ_a. Soils with a wider range of pore sizes exhibit greater changes in matric suction with water content (i.e. the slope of the SWCC is steeper) (Hillel, 1980; Fredlund and Rahardjo, 1993).

Desorption (drying) curves were measured for the seven laterites compacted with British Standard Light (BSL), West African Standard compaction (WAS), and also two others (Laterites A and G) compacted with British Standard Heavy (BSH) compaction. As the lowest hydraulic conductivities of clays are normally found not more than 2% wet of the optimum water content, therefore, the specimens were prepared at 2% wet of the optimum water content. Functions normally used to describe SWCCs were fit to the data. These SWCCs are useful for illustrating how compaction conditions, index properties, and mineralogy affect the unsaturated hydraulic properties of clay barrier soils and can be

employed when modelling flow and transport. There are many mathematical models in use which can approximately describe the SWCC for a particular soil type by using a few data points. The two most commonly used models are the Brooks-Corey equation (Brooks and Corey, 1966) and the van Genuchten equation (van Genuchten, 1980). The Brooks-Corey's equation is (Brooks and Corey, 1966):

$$\Theta = \frac{\theta - \theta_r}{\theta_s - \theta_r} = \left[\frac{\psi_a}{\psi}\right]^{\lambda}, \quad \psi \geq \psi_a \ldots\ldots\ldots\ldots\ldots\ldots (5.27(a))$$

$$\Theta = 1 \text{ and } \theta = \theta_s, \ \psi < \psi_a \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.27(b))$$

where Θ = effective saturation; λ = a fitting parameter called the pore-size distribution index (Corey, 1994), which is a function of the distribution of pores in the soil; ψ_a = the air-entry suction; θ_s = saturated water content; θ_r = the residual water content. If $\theta_r = \theta$, then $\Theta = S$, the degree of saturation. The van Genuchten equation is (van Genuchten, 1980):

$$\Theta = \frac{\theta - \theta_r}{\theta_s - \theta_r} = \left\{\frac{1}{[1 + (\alpha\psi)^n]}\right\}^m \quad \ldots\ldots\ldots\ldots (5.28)$$

where α, n and m = fitting parameters. The parameter m is frequently set to be $(1 - n^{-1})$. This constraint reduces the flexibility of eq. (5.28) but results in greater stability during fitting (van Genuchten *et al.*, 1991) and also permits analytical solutions to relative hydraulic conductivity functions based on the parameters α and n (van Genuchten, 1981).

The two commonly used relative hydraulic conductivity functions in unsaturated flow modelling are the Brooks-Corey (BC), and van Genuchten (VG) relative hydraulic conductivity functions. The BC relative hydraulic conductivity function can be expressed as (Chiu and Shackelford, 1998):

$$\left\{\left[\frac{\psi_a}{\psi}\right]^{2+\lambda} \qquad \psi > \psi_a\right.$$

$$K_r = 1.0 \quad \psi \leq \psi_a \; \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.29)$$

where K_r = relative hydraulic conductivity defined as the hydraulic conductivity of the soil, K, relative to the saturated hydraulic conductivity, K_{sat}, of the soil, or

$$K_r = \frac{K}{K_{sat}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.30)$$

for VG function, K_r can be expressed as (van Genuchten, 1980):

$$K_r = \left\{ \frac{1-(\alpha\psi)^{n-1}\left(1+(\alpha\psi)^{n}\right)^{-m}}{[1+(\alpha\psi)^{n}]^{m/2}} \right\}^{2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.31)$$

where α, n, and m have been defined earlier.

The compaction characteristics of the seven laterites used for the SWCC studies are presented in Table 5.5. Furthermore, the initial parameters of the lateritic soils also used for these SWCC studies are given in Table 5.6. Volumetric pressure plate extractors were used to measure the SWCCs. Pressure plate extractors work on the principle of axis translation. Axis translation employs the fundamental definition of matric suction; i.e, matric suction is the pressure difference across the air-water interface, namely, $\psi = U_a - U_w$, where U_a is the pore air pressure, and U_w is the pore water pressure. In a pressure plate extractor, U_w is maintained constant (usually $U_w = 0$) and U_a is increased to obtain the desired matric suction (Fredlund and Rahardjo, 1993).

Prior to the commencement of the SWCC test, the compacted specimens in BS moulds were carefully extruded, cored into aluminium rings (50mm diameter by 50mm height). The ring was placed on a ceramic disk and bolted to the bottom plate of the extractor. The ring was designed such that the ceramic disk is solely in contact with the soil

and is not directly exposed to pressurized gas within the pressure vessel. This reduces the diffusion of gas through the ceramic disk (Tinjum *et al.*, 1997). After the specimen was bolted to the extractor, it was submerged in de-aired water for saturation.

SWCC specimens were brought to full saturation by capillary action within at least six weeks. At the end of the saturation period, the pressure vessel was assembled and the outflow collection system was flushed to remove any air bubbles. The initial pressure was then applied and outflow readings were recorded until outflow stopped which meant that the soil-specimen was in equilibrium with the applied pressure. Once equilibrium was established, a higher pressure was applied and outflow readings made.

This process was repeated until a high pressure of about 2000kPa was achieved, which is near the air-entry pressure of the ceramic disk. On completion of the test, the extractor was immediately disassembled. This test was carried out at the Department of Soil Science, Ahmadu Bello University, Zaria. The interpreted parameters from the SWCCs studies are presented in Tables 5.7a, b, and c. Furthermore, Brooks-Corey's method was used in addition to deduce important parameters such as the air-entry suction (Ψa), residual water content (θ_r), saturated water content (θ_s), and corresponding lambda (λ) values as shown in Tables 5.8a and b. Similarly van Genuchten's parameters deduced, including fitting parameters (see eq. 5.28) α, n, and m (m was set to equal $[1-n^{-1}]$) are shown in Tables 5.9a and b.

UNSATURATED HYDRAULIC CONDUCTIVITY

The relationship between hydraulic conductivity and matric suctions is known as hydraulic conductivity function. Owing to the fact that compacted soil barriers are frequently unsaturated, modelling of flow and transport oftens requires the knowledge of their

unsaturated hydraulic properties which is vital in predicting water flow and solute transport in the vadose zone (Tinjum *et al*., 1997). Though there are many closed form analytical expressions for predicting the unsaturated hydraulic conductivity, the most popular ones, Brooks and Corey (1966) and van Genuchten (1980) were used as expressed in eqs. (5.29) and (5.31) to calculate the relative hydraulic conductivity and unsaturated hydraulic conductivity shown in Table 5.10a for BSL effort, and Table 5.10b for WAS effort. Table 5.10c shows a similar compilation but for only Laterites A and G compacted with a heavier effort (BSH).

5.2.12 Batch equilibrium tests

Batch equilibrium tests were performed with the seven lateritic soils to determine the adsorption characteristics of the soils with respect to K^+, Na^+, and Ca^{2+}. Competition between the ions for the exchange sites on the soil was accounted for directly by using leachate from the municipal solid waste at a waste disposal site at Samaru. All the chemical and soil analyses were carried out at Department of Soil Science, Ahmadu Bello University, Zaria. The chemical composition of the leachate was analysed using UNICAM 969 Atomic Absorption Spectrometer (AAS), and the results are summarized in Table 5.11.

A 1:4 (soil: solution ratio) by weight, in accordance with Shackelford *et al*. (1989), was used to provide an approximate environment similar to the conditions inside the diffusion cell. By starting with 50g of soil and 200g of leachate in previously washed plastic containers, the mixture were agitated intermittently for about 48 hours before obtaining the supernatant by filtering into clean plastic sample bottles using filter papers. The concentrations of the leachate in the containers were varied by serial dilution using water. Generally, the procedures used for the batch equilibrium tests were in accordance with those

used by Shackelford *et al*. (1989). Cation concentrations were measured using UNICAM 969 Atomic Absorption Spectrometer (AAS).

The results of the chemical analyses are presented in Table 5.12 for all the seven laterites. Furthermore, these results were plotted in the form of adsorption isotherms, or sorbed concentration, S, versus dissolved equilibrium concentrations, C, of solute for each ion. The sorbed concentrations were determined by mass balance using the following equation:

$$S = \frac{(C_o - C)\,V}{M_S} \text{ ---------- (5.32)}$$

where C_o is the initial concentration of the specified ion in the plastic container, V is the volume of the solution; M_s is the soil mass.

5.2.13 Diffusion tests

Transport of non-reactive solutes: In a saturated soil under a transient condition, a non-dimensional nonreactive solute transport may be expressed as (Shackelford *et al*., 1989):

$$\frac{dc}{dt} = \frac{Dd^2c}{dx^2} - V_s \frac{dc}{dx} \text{ …….. (5.33)}$$

where c is the concentration of the solute in the liquid; t is time; D is the coeffiienct of hydrodynamic dispersion in the direction of transport; V_s is the average linear groundwater or seepage velocity; x is the space coordinate.The hydrodynamic dispersion coefficient accounts for the spreading of the solute front during transport, and consists of mechanical dispersion and diffusion which can be expressed as

$$D = D_m + D^* \text{ …… (5.34)}$$

where D_m is the mechanical dispersion coefficient, and D^* is the effective diffusion coefficient. The effective diffusion coefficient is assumed to be directly proportional to the

free-solution diffusion coefficient, D_o, of a solute in an aqueous solution. In contaminant transport, the effective diffusion coefficient can be defined as (Shackelford *et al*., 1989):

$$D^* = D_o \iota \qquad \text{..... (5.35)}$$

where ι , is a tortuosity factor. The tortuosity factor accounts for the tortuous pathways experienced by solutes diffusion through soil. Bear (1972) recommends a tortuosity factor of 0.67 for unconsolidated media, however, earlier Perkins and Johnston (1963) reported that it ranged from 0.5 to 0.8 for granular materials.

Transport of reactive solutes subject to sorption reaction:

For reactive solutes subject to reversible sorption, eq. (5.33) can be modified as (Shackelford *et al*., 1989):

$$\frac{dc}{dt} = \frac{D}{R}\frac{d^2c}{dx^2} - \frac{V_s}{R}\frac{dc}{dx} \qquad \text{.....(5.36)}$$

where R is the retardation factor, which can be expressed as

$$R = 1 + \frac{\rho_b}{n} K_p \qquad \text{..... (5.37(a))}$$

or $$R = 1 + \frac{\rho_b}{\theta} K_p \qquad \text{..... (5.37(b))}$$

where ρ_b is the bulk (dry) density of the soil, n is the total porosity; K_P is the partition coefficient; θ is the volumetric water content.

Advective versus diffusive transport

Both eqs. (5.33) and (5.36) account for advective and diffusive transport of solutes. In coarse-grained soils like gravels and sands, the velocity of flow is Darcian; therefore, advection dominates the mass transport while diffusion is negligible. However, as the advective velocity becomes very low, diffusion becomes dominant while advection tends to zero. In the limit (i.e., $V_s \rightarrow 0$), eqs. (5.33) and (5.36) reduce to Fick's second law, or, (Shackelford *et al*., 1989):

$$\frac{dc}{dt} = D^* \frac{d^2c}{dx^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.38)$$

and

$$\frac{dc}{dt} = D^* \frac{d^2c}{Rdx^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. (5.39)$$

which describe pure diffusion transport of nonreactive and reactive solutes respectively.

Diffusion methodology

The same leachate used for the batch equilibrium tests was utilized here. By using British Standard Light (BSL) compaction, specimens were compacted at 2% wet of the optimum water content in a similar way to those used for hydraulic conductivity tests. Compacted specimens, about 3cm thick inside BS moulds were soaked in water for about 40 days to minimize mass transport due to advection or suction in the soil, drained, and fitted on top with a burette to provide volume change readings during diffusion period (see Fig. 5.5).

Also, a funnel was fitted at the perforated base of the mould to aid easy collection of effluent leachate from the cell. After the introduction of the influent leachate through the top of the burette, the diffusion process was monitored for about 76 days during which the hydraulic conductivity was constantly monitored, and also the effluent periodically collected for chemical analysis. The effluent leachate samples were analyzed for the following ions: Na^+, K^+, Ca^{2+}, Cl^-, and SO_4^{2-}, to determine the concentration of the effluent with time. The diffusion tests were performed at ambient laboratory temperatures ranging from 25^oC to 26^oC.

At the end of the diffusion test, the cells were disassembled, the soil specimens extruded and sectioned into slices of equal dimensions (0.5cm thick) to provide: (1) a distribution of water contents in the soil specimens by oven-drying, and (2) a concentration profile of the specified cations for use in determining the effective diffusion coefficients. The cations were analysed using UNICAM 969 Atomic Absorption Spectrometer (AAS). The

results of the variation of the water content with depth are presented in Table 5.13, while the ion concentration depth profiles are given in Table 5.14.

Data Analysis

The determination of the effective diffusion coefficient, D*, can be done by using the ion concentration in the sliced soil after an elapsed time, t, and fitting theoretical curves to the measured data by assuming different D* values in a solution to eq. (5.39). The mass of contaminant in the soil at the end of diffusion test, M_t, can be measured and used to determine D* according to Carslaw and Jaeger (1959), and (Crank, 1975):

$$\frac{M_t}{M_\infty} = 1 - \sum_{m=0}^{\infty} \frac{8}{(2m+1)^2\pi^2} \exp\left(\frac{-D^* (2m+1)^2 \pi^2 t}{4R_d L^2} \right) \quad \ldots\ldots\ldots\ldots (5.40)$$

where M_∞ is the corresponding total mass of the contaminant in the soil at infinite time (i.e. at equilibrium), given by

$$M_\infty = ALn\, R_d\, C_o \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.41)$$

where A = diameter of the soil specimen; n = total porosity; L = the length or thickness of the soil specimen. This method, eq. (5.40), was used to determine D* values which are presented in Table 5.15.

The results of the hydraulic conductivity monitored constantly throughout the duration of the diffusion, are summarized in Table 5.16. From Table 5.16, as the pore volume increases from 0 mg/l to 4 mg/l, the corresponding changes in the hydraulic conductivities for all the seven laterites are not high. Also, for all the other pore volume changes (up to 20.0 mg/l), the corresponding changes in the hydraulic conductivities are not substantial, pointing

to the compatibility of the leachate and these laterites. Furthermore, the results of the solute breakthrough curves for these laterites are presented in Table 5.17.

CHAPTER SIX

RESULTS, DATA ANALYSES AND DISCUSSIONS

6.1 Index Properties and Mineralogy

From the results of the index properties of the seven laterites (see Table 5.2 (a) and also the physical and chemical properties of these soils (see Table 5.2 (b)), some deductions can generally be made concerning these particular types of soil. For example, Laterite A has the higest fines content (70%), and also the highest clay content (40%), while Laterite G has the least fines content (53%) and least clay content (17.5%). As for Laterites B-F, their fines content is 68%, 67%, 62%, 61%, and 56% while their clay content is 37%, 31.5%, 26%, 22.2%, and 19.5% respectively.

The particle size distribution curve is the result of gradation analysis using sieve and hydrometer analysis for all the seven laterites presented in Figs. 6.1(a) and (b). Furthermore, the seven lateritic soils vary in plasticity ($20 \leq LL \leq 54$; $10 \leq PI \leq 30$), particle size distribution ($53\% \leq$ fines $\leq 70\%$; $17.5\% \leq$ clay content $\leq 40\%$), cation exchange capacity (9.6 to19.2 meq/100g of soil) and soil classification. The mineralogy, using differential thermal anlysis (DTA), shows that smectite is predominant in Laterites A to E while kaolinite dominates Laterites F and G (see the differential thermograms of Appendix F2).

6.2 Compaction

The results of the compactions tests for all the seven laterites are summarized in Table 5.3. Generally, from Table 5.3, the compactive effort has a direct influence on the results. For example, for all the seven laterites, the British Standard Light (BSL) compaction gave the lowest values of dry unit weight followed by the West African Standard (WAS)

compaction, while the British Standard Heavy (BSH) compaction was the highest. And more importantly the BSL compaction resulted into the highest hydraulic conductivity values followed by the WAS compaction, while the BSH compaction gave the lowest values of hydraulic conductivity for each type of laterite. In clay compaction, a high compactive effort normally gives low hydraulic conductivities (Benson and Daniel, 1990; Benson and Boutwell, 1992; Daniel, 1987).

The relationship between the moisture content and dry unit weight for all the seven laterites are presented in Figs. 6.2 (a)-(g). Generally from Figs. 6.2 (a)-(g), the dry unit weight of a soil (and hence the dry density) varies with the molding water content at which it is compacted. By definition, the maximum dry density is obtained at what is referred to as the optimum water content, ω_{opt}. Figs. 6.3 (a)-(g) show the corresponding relationship between the hydraulic conductivity (permeability) k and molding water content for the seven laterites.

From Figs. 6.3 (a)-(g), the hydraulic conductivity normally decreases from high values when compacted at water contents less than the optimum water content (this is referred to as being dry of optimum water content) to minimum values at water contents 2% to 4% above the optimum water content (this is referred to as being wet of optimum). These decreases are normally attributed to the presence of a more dispersed soil structure when the soil is compacted wet of optimum (Mitchell *et al*. 1965, Dunn and Mitchell, 1984). When a clay soil is compacted at water contents appreciably lower than the optimum moisture content, a randomly oriented structure is obtained. Increasing water content reduces the randomness. If water content is above optimum moisture content, the structure is more oriented. The type of clay also influences the soil structure. A less active clay would produce a more oriented structure.

Shear strains play a significant part in the arrangement of particles during compaction. High shear strains imposed during compaction tend to orient the particles into a parallel orientation. Samples compacted with dynamic compactor show a higher shear strength than those compacted statically if samples are sheared normal to the direction of compaction of the specimens. However, if sheared in the direction of compaction, statically compacted samples would have greater strengths.

Dynamically compacted samples possess random structure while statically compacted ones have a more oriented structure. Shear strength is highest if the shear plane is normal to the plane of orientation of the soil particles. The shear strain imposed during compaction tends to force the particles to align themselves in parallel arrangement to the applied load. Hence, higher compaction shear strains produce a more oriented structure.

Benson and Daniel (1990) as well as Elsbury *et al*. (1990) have reported a reduction of clod size from 19 mm down to 4.8 mm before standard Proctor compaction produces a reduction of four to five orders of magnitude in laboratory measured k-values. Such extreme variations in k imply open voids between hard clods that must be destroyed by field compaction procedures (pre-crushing, extended wetting, heavier equipment, extra passes, etc). Even if one decides to compact wet of standard Proctor using extra passes to achieve extra kneading and destruction of soil clods and the contacts between them, a few risks may still remain (Rowe *et al*., 1996). All such soils shrink on drying. The kneaded soil demonstrates the greatest axial (vertical) shrinkage, probably because of the horizontal preferred orientation of the clay platelets. Since horizontal shrinkage would cause immediate cracking rather than shrinkage, it is important that the clay liner not be allowed to dry out, so as not to rapidly induce vertical desiccation cracking.

6.3 Volumetric shrinkage

The plots of dessication induced volumetric shrinkage strain versus the molding water control for the seven laterites are presented in Figs. 6.4 (a)-(g). As the molding water content increases, the volumetric shrinkage strain increases. The trend was generally similar to all the laterites considered. From Figs. 6.4 (a)-(g), the greater the compactive effort, the lower the shrinkage strain for a particular molding water content. The BSL compaction has the greatest strain values, followed by the WAS compaction, and lastly BSH compaction.

These results are in accordance with those reported by other researchers such as Albrecht and Benson (2001) who, from the study of the effect of dessication on compacted natural clays, noted that the lowest volumetric shrinkage strain generally occurred in specimens compacted near optimum water content, and that strain increased with an increase in compaction water content relative to optimum. Furthermore, Albrecht and Benson (2001) reported that the volumetric shrinkage strains as large as 30% were observed at water contents greater than 5% wet of optimum. Likewise, the volumetric shrinkage strain increased as the water content became dry of optimum with volumetric shrinkage strains above 15%.

Haines (1923) described the drying process of saturated soils as having two important stages which could be termed primary and residual drying. Primary drying is the first stage of drying, and occurs as water leaves the soil without entry of air. Since air is not entering the soil, the volume change is equal to the volume of water leaving the soil. The majority of the total volume change occurs during the primary stage of drying. Water surrounding the individual soil particles is removed allowing the soil particles to move closer together as the water retreats. At some point the soil particles contact each other, and the drying process slows as the structure of the soil begins to resist additional volume change. In this phase of drying, termed “residual drying,” air enters the soil and replaces the water being removed because the particles are in contact.

Little change in the soil structure or total volume occurs during residual drying because the particles are in contact. Therefore, the total amount of volume change is closely related to relative volume of water and solids present in the soil as drying begin. Albrecht and Benson (2001) had results which are consistent with Haines's theory, as they noted that specimens compacted at optimum water have the largest volume of soil particles and the least volume of water/unit volume of soil at any given compactive effort, and thus undergo the smallest volume change during drying.

Furthermore, Albrecht and Benson (2001) noted that specimens compacted with the lowest compactive effort shrank the most, and for all compactive efforts, specimens with the lowest volumetric shrinkage strain typically were compacted near optimum water content. Also, the volumetric shrinkage strain increases with increasing clay content. The volumetric shrinkage strain increases because soils with higher clay content or PI have a greater affinity for water. Their greater affinity for water is reflected in the increase in optimum water content and decrease in maximum dry unit weight as the PI increases (Johnson *et al*. 1990; Blotz *et al*. 1998). As a result, soils with larger clay content or PI have a greater quantity of water that can be removed during primary drying. Substantial volumetric shrinkage strains (5-10%) were obtained for clay contents less than 30% (Albrecht and Benson, 2001).

A similar observation was made by Croney and Coleman (1954) who described changes in the soil water characteristic curve (SWCC) for a natural clay soil during drying. Permanent changes in the SWCC occurred after an initial drying from near saturation. However, repeated wetting and drying after the initial drying produced no further changes in the SWCC, suggesting that any permanent change in the soil structure took place during the initial drying. The large volumetric shrinkage strains that occurred during the first drying cycle and the similar volumetric shrinkage strain at the end of subsequent cycles suggests that the interparticle stress incurred during drying is much higher than the stress applied

during compaction, and a similar interparticle stress is applied during subsequent drying cycles (Albrecht and Benson, 2001).

Another important effect on the drying of clays is cracking, though not observed with any of the seven laterites for the first five months. However, after six months of drying, minute cracks appeared on Laterites A and B compacted with BSL effort. Eight months after drying, minute cracks also appeared on the remaining specimens of Laterites A and B compacted with WAS and BSH efforts as well as Laterites C-G compacted with BSL effort. Ten months after drying, minute cracks appeared on the remaining specimens of Laterites C-G compacted with WAS and BSH efforts, with the least effect on Laterites F and G. Generally, specimens compacted with high molding water content had higher volumetric strain than those with low molding water content. The highest volumetric shrinkage strain for Laterites A-G is 14.4, 13.0, 12.2, 12.1, 11.9, 11.8, and 11.7 respectively; all specimens compacted with BSL effort, and had no cracks for the first five months. Hence, a maximum strain of 11% for all the laterites is recommended. Based on the volumetric shrinkage, WAS and BSH efforts are recommended as well as lower molding water content for all the laterites. The most severe cracking normally occurred in specimens with very high volumetric shrinkage strain (Albrecht and Benson, 2001). For example, several specimens of the highly plastic soil that underwent volumetric shrinkage strains of up to 30% contained continuous cracks throughout their depth after three drying cycles.

In contrast, specimens of the low plasticity soil contained no visible cracks regardless of their compaction water content, and these specimens had the lowest volumetric shrinkage strains (0-10.6%). Similar behaviour should be expected in the field, although in the field some degree of cracking should be expected for nearly all clayey soils because even small volumetric strains are likely to result in the formation of some cracks. Because of the

possibility of cracking, Albrecht and Benson (2001) recommended less than 5% volumetric strain in the design standard for all clay liners.

6.4 Unconfined Compressive Strength

The results of the unconfined compressive strength (UCS) are presented in Table 5.4 for all the seven laterites, while their graphical plots are as shown in Figs. 6.5 (a)-(g). Generally, the unconfined compressive strength decreased with higher molding water content for all the laterites, and BSH specimens have greater UCS values than either WAS or BSL samples, while BSL have the least UCS values for particular molding water content. These results are in agreement with the results of Daniel and Wu (1993) that used a minimum of $200kN/m^2$ in their design standard for compacted clay liners.

6.5 Hydraulic Conductivity

Compaction conditions

The hydraulic conductivity versus the initial saturation was plotted as shown in Fig. 6.6. The initial saturation, S_i, was computed using (Benson and Trast, 1995)

$$S_i = \left(\frac{w}{\frac{\gamma_w}{\gamma_d} - \frac{1}{G_s}} \right) \qquad \text{……(6.1)}$$

where w is the molding water; γ_d is the dry unit weight; γ_w is the unit weight of water; and G_s is the specific gravity of solids. A trend of decreasing hydraulic conductivity with increasing initial saturation exists. Those specimens compacted with higher compactive effort also generally have lower hydraulic conductivity.

Based on the relationship shown in Fig. 6.6, it is apparent that hydraulic conductivity less than 10^{-7} cm/s, the common regulatory maximum hydraulic conductivity in the United States, can be achieved for most of the soils in this study by compacting to an initial

saturation in excess of 85%. These results are in accordance with those reported by Boutwell and Hedges (1989); Acar and Oliveri (1990); Benson and Daniel (1990); Benson and Boutwell (1992); Benson *et al*. (1994); Benson and Trast (1995), who noted a strong relationship between the contours of initial saturation and zones of similar hydraulic conductivity.

Furthermore, Laterite A, having the highest fine content of 70% and highest activity of 0.75, has the lowest hydraulic conductivities for BSL, WAS, and BSH efforts which range from 5.8E-09 cm/s to 8.1E-08 cm/s, followed by Laterite B with a lower fine content of 68% and activity of 0.70 with higher k-values that range from 9.6E-09 cm/s to 3.0E-07 cm/s for the three compactive efforts. Similarly for Laterites C-G, as the fine content decreased to 67%, 62%, 61%, 56%, and 53% respectively, the activity decreased to 0.67, 0.62, 0.63, 0.56, and 0.57 respectively, while k-values increased with a range of 1.6E-08 cm/s to 3.4E-06 cm/s, 1.9E-08 cm/s to 6.8E-07 cm/s, 1.0E-08 cm/s to 2.4E-06 cm/s, 7.6E-07 cm/s to 7.1E-06 cm/s, and 8.3E-07 cm/s to 9.9E-06 cm/s respectively for the three compactive efforts.

These results are similar to those reported by many researchers in the last four decades (Lambe, 1954; Bjerum and Huder, 1957; Mitchell et al., 1965; Garcia-Bengochea et al., 1979; Acar and Oliveri, 1990; Benson and Daniel, 1990; Daniel and Benson, 1990). Hydraulic conductivity is sensitive to molding water content and compactive effort. The lowest hydraulic conductivities generally occur at molding water contents 1-2% wet of the line of optimums for a given compactive effort.

Effect of Index Properties

The composition of the soil can also significantly affect hydraulic conductivity, particularly for compaction wet of the line of optimums where flow is controlled by the size, shape, and connectivity of microscale pores (Acar and Oliveri, 1990; Benson *et al*., 1994). In

particular, soils having a greater quantity of fines and clay, and more active clay minerals, generally have lower hydraulic conductivity because they contain clay particles that are smaller and have thicker double layers (Lambe, 1954; Mesri and Olson, 1971; D'Appolonia, 1980; Daniel, 1987; Kenney *et al*., 1992; Benson *et al*., 1994).

Effect of Atterberg Limits on hydraulic conductivity

Hydraulic conductivity generally decreased with increasing liquid limit and plasticity index (see Figs. 6.7.and 6.8.). These trends are expected because the liquid limit and plasticity index are directly related to the mineralogy of the soil and the clay content. An increase in clay content or the presence of more active clay minerals generally corresponds to a decrease in the size of microscale pores, which control flow in soils, compacted wet of the line of optimums. Consequently, lower hydraulic conductivity occurs. That is, soils having higher liquid limit and plasticity index generally contain more active clay minerals and/or greater clay content and typically have lower hydraulic conductivity.

The trends shown in Figs. 6.7 and 6.8 are also similar to those reported by Benson *et al*. (1994) for specimens collected from actual compacted soil liners. That is, a rapid decrease in hydraulic conductivity occurs as the liquid limit is increased from 20 to 40% and the plasticity index is increased from 10 to 30%. Thereafter, hydraulic conductivity is less sensitive to liquid limit or plasticity index. However, there is some discrepancy between the field results and laboratory results.

Although the cause for this discrepancy in hydraulic conductivity is not obvious, two possible reasons are offered. First, the field specimens described by Benson *et al*. (1994) may have contained defects that are not likely to exist in the specimens which were prepared under controlled laboratory conditions. Second, the compactive effort applied in the field may have been less than was applied in the laboratory. It has been observed that in many

field applications, the compactive effort actually delivered is slightly less than that corresponding to standard Proctor compaction (Benson and Trast, 1995). Either of these factors, or both, could be responsible for the differences in hydraulic conductivity.

Particle size distribution

The influence that percentage fines and clay content have on hydraulic conductivity wet of the line of optimums is shown in Fig. 6.9. An increase in percentage fines generally results in a decrease in hydraulic conductivity, but the trend is not very strong. As the fines content for Laterite A decreased progressively from 70% to 68%, 67%, 62%, 61%, 56%, and 53% for Laterites B-G respectively, the clay content decreased as well from 40% for Laterite A to 37%, 31.5%, 26%, 22.2%, 19.5%, and 17.5% for Laterites B-G. The hydraulic conductivity in this case increased from Laterite A to Laterite G with the following range; 5.8E-09 cm/s to 8.1E-08 cm/s, 9.6E-09 cm/s to 3.0E-07 cm/s, 1.6E-08 cm/s to 3.4E-06 cm/s, 1.9E-08 cm/s to 6.8E-07 cm/s, 1.0E-08 cm/s to 2.4E-06 cm/s, 7.6E-07 cm/s to 7.1E-06 cm/s, and 8.3E-07 cm/s to 9.9E-06 cm/s for Laterites A-G respectively.

Similar behaviour was reported by Benson *et al*. (1994) who noted a weak relationship between hydraulic conductivity and fines/ clay content. Hydraulic conductivity should not be expected to be strongly related to percentage fines or clay content for the soils that were tested (Benson and Trast, 1995). All of the soils contain more than 50% fines, and therefore classify in the USCS as fine-grained soils.

Furthermore, five of the soils classify as clays in the USCS (CL or CH, see Table 5.2a.). Because these five soils contain a large quantity of fines and are sufficiently plastic to classify as clays, it is expected that the pore structure controlling flow should be affected more by mineralogical composition and less by particle size distribution (Benson and Trast,

1995). In fact, those soils categorized as smectitic on average tend to have lower hydraulic conductivity.

6.6 Reliability Analysis

The First Order Reliability Method (FORM) was used to assess the potential of lateritic soils as liners and covers. The seven laterites (Laterites A-G) were used. Based on the specifications for the municipal solid wastes liners, at least 15-20% clay-size particles, minimum activity of 0.3, and minimum CEC of 10 meq/100g of soil (Das 1998), Laterite A is the most suitable. This work presents a procedure developed for assessing lateritic clay liner materials by studying the effects of compactive effort, coefficient of variation, and compositional variables like initial saturation, plasticity index, and clay content on reliability indices for lateritic soils. Three compactive efforts were used: British Standard Heavy (BSH), West African Standard (WAS) effort, and British Standard Light (BSL).

The practice of structural condition assessment using probabilistic based methods has been generally successful in many structural engineering problems as well as geotechnical engineering designs (Halim and Tang, 1991; Ahner *et al*., 1998; Gui *et al*., 2000). The structural system here is normally analyzed using a reliability model based on different possible failure modes. As the risk of structural failure can never be totally eliminated, the target is to define the safety margin so that the risk of failure is highly minimized. In conventional structural optimization problems, the design objective is to achieve maximum economy in the resulting design subject to certain constraints that limit the design. Presently, the objective of achieving this maximum has been related to minimum weight, minimum cost, and many other deterministic parameters. However, for a complete assessment of different failure modes, uncertainties in the input parameters must be considered. A

reliability analysis can be utilized in the study of the failure process; it can show which failure path is eminent.

A probabilistic approach in structural evaluations can incorporate both the on-site inspection results and nondestructive evaluation of data to provide a better reliability analysis (Yang and DeWolf, 2002). While the reliability analysis of a structure can be examined on the basis of individual components, a better approach is to model the entire structural system. A system approach considers the reliability of individual components as well as their relationship and importance to the entire system. Since lateritic soils are quite abundant in the tropics, their uses as liners and covers in landfill waste containment can be stochastically modelled in terms of their compositional variables and deterministic parameters.

6.6.1 Background

Before the advent of reliability optimization, structural inspection and repair programmes were entirely based on past engineering and experiences. However, in modern times, the technique of reliability assessment has been increasingly applied successfully in diverse engineering applications (Yang and DeWolf, 2002) including modern design codes such as the American Association of State Highway and Transportation Officials (AASHTO). Furthermore, probability theory in which some statistical knowledge of random variables such as mean values and standard deviations is utilized, has proved increasingly successful in many engineering designs. FORM (Gollwitzer *et al*., 1988) was used in this study. The general problem to which FORM provides an approximate solution is as follows. The state of a system is a function of many variables some of which may be uncertain. These uncertain variables are random with joint distribution function, defining the stochastic model, which can be expressed as (Gollwitzer *et al*., 1988):

$$F_{\underline{X}}(\underline{\mathbf{X}}) = P\left(\bigcap_{i=1}^{n} \{X_i \le X_i\}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6.2)$$

For FORM (Gollwitzer *et al*., 1988), it is required that $F_{\underline{X}}$ ($\underline{\mathbf{X}}$) is at least locally continuously differentiable, i.e., that probability densities exist. The random variables $\underline{\mathbf{X}}$ = ($X_{1\ldots}$ X_n) are called basic variables. The locally sufficiently smooth state function is denoted by g ($\underline{\mathbf{X}}$). It is defined such that g ($\underline{\mathbf{X}}$) > 0 corresponds to favourable (safe); g ($\underline{\mathbf{X}}$) = 0 denotes the so-called limit state or the failure boundary. Hence, g ($\underline{\mathbf{X}}$) < 0 (sometimes also g ($\underline{\mathbf{X}}$) ≤ 0) defines the failure [unacceptable or diverse] domain F.

g ($\underline{\mathbf{X}}$) can be defined as an analytic function or an algorithm (e.g. a finite element code). In the context of FORM, it is convenient but necessary only locally that g($\underline{\mathbf{X}}$) is a monotonic function. FORM works by transforming non-gaussian (non-normal) variables into independent standard normal variables by locating the so-called β-point (checking point or most likely failure point) through an optimization procedure by linearising the limit state standard normal integral (Gollwitzer *et al*., 1988). Among other uses, FORM produces an approximation to the following.

$$P_f = P(\underline{\mathbf{X}} \in F) = P\,(g\,(\underline{\mathbf{X}}) \le 0) = \int_{g\,(\underline{\mathbf{X}}) \le 0} dF_{\underline{X}}\,(\underline{\mathbf{X}}) \ldots\ldots\ldots\ldots\ldots\ldots \ldots\ldots\ldots\ldots (6.3)$$

A First -Order approximation to eq. (6.3) is

$$P_f \approx \phi\,(-\beta) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6.4)$$

where ϕ ( ) is the standard normal integral, and β is the so-called (geometric) safety index defined as

$\beta = \min \{ \| \underline{\mathbf{x}} \| \}$ for $\{\underline{\mathbf{x}}: g(\underline{\mathbf{x}}) \leq 0\}$ ……..(6.5)

Applying reliability analysis to hydraulic conductivity, the probability of failure, P_f, can be defined as

$P_f = P (K_o - K_e (S_i, PI, C, E) \leq 0)$……...…….(6.6)

where K_e = expected hydraulic conductivity

K_o = the specified hydraulic conductivity which is equal to 10^{-9} m/s.

The main and subroutine program used in this study is presented in Appendix I. In eq. (6.6), the probability of failure depends entirely on the variables S_i (initial saturation), PI (plasticity index), C (clay content) and E (compactive effort. The appropriate limit state function is of the form:

$g(X) = \text{Ln } K_o - \text{Ln } K_e$ ……. (6.7)

Failure occurs when K_e exceeds K_o (the regulatory maximum hydraulic conductivity). K_e can be defined as (Benson *et al.*, 1994):

$\text{Ln } K_e = Y = \mathbf{x}\, \beta^1 + \varepsilon$ ……..…… (6.8)

where Y = a random variable describing the distribution of $\text{Ln}K_e$; $\underline{\mathbf{x}}$ = 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. A better estimate of K_e can be made as follows (Benson and Trast, 1995):

$\text{Ln } K_e = -15.0 - 0.087\, S_i - 0.054\, PI + 0.022\, C + 0.91E + \varepsilon$ ……..(6.9)

The compactive effort index is an integer categorical variable describing the compactive effort; E was assigned the value –1, 0, and 1 for the three compactive efforts; BSH, WAS, and BSL respectively. The partial F's for the independent variables in eq. (6.9) are 262 for S_i, 65 for E, 24 for PI, and 5 for clay content. This means that the most significant

independent variables that affect hydraulic conductivity, in decreasing order, are; (i) S_i, initial saturation, (ii) E, compactive effort, (iii) PI, plasticity index, and (iv) C, clay content. The improved version of K_e (eq. 6.9), was employed in the limit state function of eq. (6.7) which was used in the stochastic modelling.

6.6.2 Reliability Analysis Procedure

The index properties of the seven laterites used are as shown in Table 5.2(a). Fundamentally, hydraulic conductivity is normally assumed to have a lognormal distribution (Benson, 1993; Gui *et al.*, 2000). Also, initial saturation, and plasticity index are equally lognormal. However, clay content has a normal distribution while the compactive effort is a deterministic parameter (-1, 0, +1). The limit state function utilized was incorporated into a FORTRAN-based programme, FORM 5 (Gollwitzer *et al.*, 1988), used for the modelling.

A typical input data for the reliability analysis using Laterite A is presented in Table 6.1 while Table 6.2 shows a summary of the results as applicable to all the seven soils. Furthermore, these results were plotted as shown in Fig. 6.10 from which Laterite A was chosen for the study of the effect of initial saturation (S_i), plasticity index (PI), and clay content (C) on the reliability of the hydraulic conductivity (K). The determination of the reliability indices for one variable at a time was accomplished over a range of coefficient of variation from 10% to 100% while the original values for other variables were kept constant. The results of these variations are shown in Fig. 6.11 through to Fig. 6.14. Also, using only a compactive effort, E=-1, the effects of the variables on the reliability indices are as shown in Fig. 6.15.

6.6.2.1 Reliability indices of the lateritic soils

a. Effect of compactive effort

Fig. 6.10 shows clearly how the compactive effort affects the seven laterites. As the compactive effort changes from –1 to +1, the reliability index decreases appreciably for all the soils. Both Laterites A and B give higher beta values than the remaining five laterites. Laterite A has the highest beta values ranging from 3.85 to 1.88 while Laterite G has the least beta values ranging from 2.99 to 0.99; therefore, Laterite A only stands out to be selected for further analysis.

b. Effect of hydraulic conductivity

There is a slight decrease in the reliability indices as the coefficient of variation increases from 10% to 100% for the three compactive efforts. From Fig.6.11, the BSH effort has the highest values which fall from 3.89 to 3.49 followed by the intermediate compactive effort (WAS) with a range of 2.85 to 2.49, and lastly the BSL effort with a range of 1.91 to 1.58. These values are in close agreement with those of other researchers like Benson *et al*. (1994) who noted values ranging from 2.77 to 0.27. Hence, the hydraulic conductivity is not the most important factor of concern as a slight decrease in the reliability indices is observed when the coefficient of variation is increased from 10% to 100%. Furthermore, soil samples compacted with higher compactive effort generally have lower values of hydraulic conductivity.

c. Effect of initial saturation

For the three efforts, there is a decrease in reliability indices as the coefficient of variation increases from 10% to 100%. From Fig. 6.12, the heaviest compactive effort, BSH, leads to the highest beta values which fall drastically from 4.82 to 0.39 followed by WAS (3.57 to 0.15), while the least effort, BSL, leads to the lowest (2.41 to 0.01). The results show that unlike the hydraulic conductivity, the initial saturation is a significant factor that must be carefully controlled during compaction, as a rapid change in the coefficient of variation has a

significant effect on the reliability of the hydraulic conductivity. Also, heavy compactive efforts generally lead to high degrees of initial saturation, while lower efforts have lower values. In the field, the initial saturation can be increased by either increasing the compactive effort or molding water content or both (Benson and Trast, 1995).

d. Effect of plasticity index

Generally, for the three compactive efforts, the reliability indices decrease as the coefficient of variation increases from 10% to 100% though not as significantly as the case of the initial saturation (see Fig. 6.13). The reliability indices for BSH are highest (4.19 decreases to 3.04), followed by those of WAS (3.07 decreases to 2.1), while BSL is the least (2.05 decreases to 1.28). This means that to decrease the hydraulic conductivity, an increase in the plasticity index is one of the possible means. However, soils with very high plasticity index are difficult to work with in the field; hence, there are restrictions concerning compactibility for clay liners. A range of plasticity index, 10% to 30%, is generally recommended (Das, 1998).

e. Effect of clay content

As the coefficient of variation increases from 10% to 100%, there is generally a linear decrease, though not as drastic as the case of initial saturation, in the reliability indices for all the three compactive efforts as shown in Fig. 6.14.The beta values for BSH are highest (4.23 decreases to 2.88) followed again by those of WAS (3.03 decreases to 2.17), while the least is BSL (1.99 decreases to 1.49). This means that increasing purely clay content simply leads to an increase in the hydraulic conductivity (as seen in eq. 6.9) provided all other variables in eq. (6.9) are held constant. In reality, soils having a great quantity of fines and clay with a lot of active clay minerals normally have lower hydraulic conductivity because they contain clay particles that are quite small and have thicker double layers (Benson and Trast, 1995).

6.6.3 Comparison of the soil properties

A comparison of the soil properties (i.e. hydraulic conductivity, plasticity index, initial saturation, and clay content) for Laterite A at a compactive effort of BSH is as shown in Fig. 6.15. Generally, for all these soil properties, the beta value decreases as the coefficient of variation increases from 10% to 100%. However, the most significantly affected is the initial saturation which stands out clearly as the most important factor to handle carefully in achieving low hydraulic conductivity for clay liners.

6.6.4 Statistical Analysis

Using analysis of variance, 2-way without replication and at 5% level of significance, the results used for plotting Figs. 6.10-6.14 were further studied. Table 6.3 shows a summary of the statistical analysis of the variance of reliability indices (which include F-values (calculated), F-values (critical) and (p-values). From the table, a comparison of all the F-values (critical) to F-values (calculated) for hydraulic conductivity, initial saturation, plasticity index and clay content shows generally that all F-values (critical) are lower. Hence, the effects of all these variables are significant.

Also, a close looks at all the p-values as well shows that they are quite small, which attest to the significance of these variables. Furthermore, with respect to coefficient of variation, the F-values for plasticity index (F-74.39) is highest followed by initial saturation (F-34.89), while the lowest is clay content (F-13.77). Similarly, with respect to compactive effort, the F-value for hydraulic conductivity is highest (F-65525.55) followed by the plasticity index (F-2003.57), clay content (F-373.65), and lastly initial saturation (F-21.64).

6.6.5 Stochastic Model Assessment

To address uncertainty in structural reliability (and for ultimate limit state), beta values ranging from 2.5 to 4.0 (corresponding to low probability of failure, $P_{f,}$ less than 6.25 x 10^{-3}) are normally considered safe. Similarly, a minimum beta value of 2.5 can be used to assess the model utilized in this work (Benson *et al*., 1994). On this basis, the summary deduced is as shown in Table 6.4. Generally, compaction with BSH effort gave beta values greater than the minimum 2.5 while BSL effort failed to meet this minimum standard in all circumstances. However, compaction with WAS effort fairly met the requirement under some coefficient of variations.

For example, considering hydraulic conductivity, all the beta values for BSL (1.58-1.91) are below the minimum 2.5, while for WAS 10% - 90% coefficient of variation (2.54-2.85) meet the requirement; as for BSH, all beta values (3.49-3.89) are quite greater than the minimum standard. Similarly, for initial saturation, BSL does not meet the minimum 2.5; WAS satisfies it (3.566) only when the coefficient of variation is 10%; while BSH meets the requirement only when the coefficient of variation ranges from 10% to 20% (2.88-4.82). Furthermore, for plasticity index, all beta values for BSL are below 2.5; a coefficient of variation of 10% to 50% (2.508-3.068) for WAS to achieve the minimum 2.5; while BSH meets the requirement at all the coefficient of variation considered (10% to 100%). From these results, some important deductions that can be made include the following.

(i) To achieve the minimum hydraulic conductivity ($K \leq 1x10^{-9}$m/s) easily in the field, an equivalent of laboratory BSH effort is highly recommended to avoid under compaction.

(ii) WAS effort can also be utilized but with a careful control. An equivalent of BSL effort must never be used for liners.

(iii) The initial saturation is a very sensitive parameter that must be carefully controlled in conjunction with the compactive effort

(iv) Wide variations in clay content and plasticity index are permissible without jeopardizing the minimum target.

(v) These results are in good agreement with those of Benson and Trast (1995) that used partial F-statistics for their analysis. Hence, the hydraulic conductivity model used in the research work is suitable for stochastic modelling of lateritic soils as potential liner materials.

6.6.6 Practical Application

A practical application of these results is in the area of screening borrow materials quickly with respect to achieving low hydraulic conductivity easily without rigorous time-consuming permeability tests, which are equally expensive. However, adequate conductivity tests should be used for the selected soils. Another application is in the area of defining an acceptable zone. If the clay content and plasticity index are known an acceptable zone can be constructed by specifying the maximum permissible hydraulic conductivity (Benson and Trast, 1995). Furthermore, the two most important variables having the greatest impact on the reliability of lateritic soils as liners are the compactive effort and initial saturation, which must be carefully controlled during compaction, while the plasticity index and clay content can be treated without strict restrictions.

6.7 Soil Water Characteristic Curves (SWCC)

The initial parameters of the lateritic soils used for the SWCC studies are shown in Table 5.6. A graphical plot of the volumetric water content versus matric suction (see Figs. 6.16a and b) was used as reported by van Genuchten (1980) for the specimens compacted

with BSL, WAS effort (Laterites A-G), and other specimens compacted with BSH effort (only Laterites A and G as shown in Fig. 6.17). From the results of BSL effort presented in Fig. 6.16a, as the matric suction increases from 0.1 to 2000kPa, the volumetric water content decreases steadily for all the laterites. Also, the variations of the degree of saturation versus the matric suction for the seven lateritic soils (BSL and WAS efforts) are shown in Figs.6.18a and b. As the matric suction increases, the degree of saturation decreases for all the laterites in a trend similar to that of the volumetric water content.

Comparing the results of the volumetric water content, specimens compacted with the lowest effort (BSL) generally have higher values than those compacted with a higher effort (WAS or BSH) of the same soil type as evident in Figs. 6.16a, b, and 6.17. The other six laterites exhibit a similar trend though with lower values of volumetric water content as the suction increases. Higher compactive effort generally leads to the elimination of pores which can cause higher air-entry suction (Ψa) and curves with steep slopes in a particular type of soil (Tinjum *et al.*, 1997). Soils with steeper SWCCs are characterized by smaller λ or n. Higher air-entry suction is characterized by greater ψ_a in eq. (5.27) or smaller α in eq. (5.28). That is, λ and n are directly related, whereas ψ_a and α are inversely related. This result is in good agreement with the report of Corey (1994) who concluded that soils with smaller pore sizes, like Laterite A, typically have greater ψ_a or smaller α than soils with larger pore sizes, like Laterite G; and soils with a broader range of pore sizes have smaller λ or n.

Curve fitting: Eqs.(5.27) and (5.28) were fit to each SWCC result to examine their fitness. Two conditions for the fitness were used i.e $\theta_r = 0$ and θ_r is greater than zero. Typical fits (θ_r is greater than zero) are shown in Figs.6.19a and b for Laterite A compacted with BSL and WAS efforts, while another fit for the same Laterite A but compacted with a heavier effort (BSH) is shown in Fig. (6.20). From Figs. 6.19a and b, as the matric suction increases from

0.1kPa to 2,000kPa for Laterite A compacted with BSL and WAS efforts, the degree of saturation decreases from 105% to 67%. At lower suctions (less than 60kPa) van Genuchten's model fits the data better, while at higher suctions Brooks-Corey's model has a better fit. However, from Fig. 6.20, at lower suctions, both Brooks-Corey and van Genuchten's models fit the data better than at higher suctions.

Furthermore, the relationships between van Genuchten's and Brooks-Corey's parameters obtained from the fits are shown in Figs.6.21a, b and 6.22a and b. Generally, as van Genuchten's alpha (α) increases, Brooks-Corey's air entry pressure (ψ_a) decreases for the three efforts (see Figs.6.21a and b).In addition, as van Genuchten's n increases, Brooks-Corey's lambda also increases for the three efforts (see Figs.6.22a and b). The relationship between van Genuchten's alpha (α) and Brooks-Corey's air entry suction according to Leij *et al*. (1998) is shown in eq. (6.10).

$$\alpha = 0.1281\psi_a^{-2.7785} \text{---------- (6.10)}$$

or

$$\text{Log}\alpha = 0.6317 (\psi_a)^2 - 2.9373 (\psi_a) + 1.3136 \text{ ----------(6.11)}$$

However, eqs. (6.10) and (6.11) failed to explain the relationship between van Genuchten's alpha (α) and Brooks-Corey's air entry suction (ψ_a) when applied to the seven laterites. Hence, using multiple regression analysis, eqs. (6.12a), (6.12b), and (6.12c) were derived for BSL, WAS and BSH specimens respectively.

$$\alpha = 0.06465\psi_a - 0.0161(\psi_a)^2 + 0.832 \quad \text{……(6.12a)}$$

$$\alpha = 0.3959\psi_a - 0.027364(\psi_a)^2 - 0.723 \quad \text{…….(6.12b)}$$

$$\alpha = 82.6388\psi_a - 3.26464(\psi_a)^2 + 1130.792 \text{ ……(6.12c)}$$

Compactive effort: Increased compactive effort led to smaller pores which affected the shape of the SWCC. This is evident in a comparison of Figs.6.16a, b, and 6.17. For a given water

content, the air-entry suction is generally higher and the slope is slightly steeper for specimens compacted with greater compactive effort. In addition, because the air-entry suction is higher, the matric suction corresponding to given water content is higher for soils compacted with higher compactive effort (Tinjum e*t al*., 1997).

Plasticity index: More plastic soils generally have smaller effective pore sizes which are reflected by their lower saturated hydraulic conductivity. Hence, more plastic soils like Laterites A and B tend to exhibit higher air-entry suction than less plastic soils like Laterites F and G. A plot of van Genuchten's n versus plasticity index (see Figs.6.23a and b) shows that the parameter n increases as the plasticity index increases for the three compactive efforts used. Also, as the plasticity index increases, van Genuchten's alpha decreases as shown in Figs.6.24a and b. Specimens compacted with a higher effort, for example, Laterite A (BSH) and Laterite G (BSH), have lower alpha values than the same soil type compacted with a lower effort e.g. Laterite A (BSL or WAS) and Laterite G (BSL or WAS). However, alpha varies in orders of magnitude depending on the plasticity index, whereas n varies in a small range. Hence, alpha is more sensitive to plasticity index than n.

These results are consistent with those of Tinjum *et al.* (1997) which highlighted the fact that increasing the plasticity index results in smaller alpha or greater air entry suction (Ψa) and larger n (shallower slope). Also, higher compactive effort results in smaller alpha but has no statistically significant effect on n. These conditions can lower the saturated hydraulic conductivity considerably.

UNSATURATED HYDRAULIC CONDUCTIVITY

The relative hydraulic conductivity is plotted against the matric suction as shown in Fig.6.25. In case of Brooks-Corey's model, as the matric suction varies from 0.1kPa to about 2000kPa, the relative hydraulic conductivity generally decreases for all the laterites. These

results highlight two important facts: (1) that Laterite A with a much higher percentage of fines and clay content has a much lower relative hydraulic conductivity than Laterite G with least percentage of fines and clay content of the same compactive effort, and (2) increasing the compactive effort leads to a decrease in the relative hydraulic conductivity.

Similar results were noted in the case of van Genuchten's model (see Figs. 6.26a and b) for the BSL effort. Furthermore, the unsaturated hydraulic conductivity values obtained for both models were plotted against the matric suction for Laterite A as shown in Figs.6.27a and b. Both models show a similar trend. For example, as the matric suction increases, the unsaturated hydraulic conductivity decreases for both Brooks-Corey's and van Genuchten's models. The trend in both the results of the relative hydraulic conductivity and the unsaturated hydraulic conductivity of this work reflects the apparent influence of the plasticity product (see Table 5.2b) highest for Laterite A and decreases to Laterite G with the following values; 2100, 1768, 1206, 992, 854, 616, and 530 respectively on the hydraulic behaviour of these compacted soils.

Statistical Analysis

(a) Multiple regression analysis: Consider a case of one independent variable with a linear regression equation, where for any given set of values $\chi 1$, $\chi 2$, $\chi 3$,……..and χr the mean is:

$$\bar{y} = \beta_0 + \beta_1 \chi_1 + \beta_2 \chi_2 + \ldots\ldots + \beta_r \chi_r \quad \text{----------} \quad (6.13a)$$

Considering a case of only two independent variables, and applying the least squares method to obtain the coefficients of β_0, β_1 and β_2 (Miller and Freund, 1987), the normal solution equation for multiple regression with r = 2 is:

$$\sum y = nb_0 + b_1 \sum x_1 + b_2 \sum x_2$$

$\sum x_1 y = b_0 \sum x_1 + b_1 \sum x_1^2 + b_2 \sum x_1 x_2$ ---------- (6.13b)

$\sum x_2 y = b_0 \sum x_2 + b_1 \sum x_1 x_2 + b_2 \sum x^2_2$

Applying the above method to the results of the SWCC, the following relationships were obtained:

$\theta_s = 0.01\ (LL) - 7.08\ (PI) + 126.31$ …… ……. (6.14a)

$\theta_r = 1.95\ (LL) - 3.01\ (PI) - 14.6$ …… (6.14b)

$\theta_s = 0.00326\ (Si) + 0.00039\ (PI) + 0.0848$ …… (6.14c)

$\theta_r = 0.00188\ (Si) + 0.00217\ (PI) + 0.00679$ …… (6.14d)

$\psi_a = 9.45\ (LL) - 14.34\ (PI) - 68.67$ …… (6.14e)

$\alpha = 0.0057\ (PI) - 0.0156\ (LL) + 1.15$ …… (6.14f)

$\lambda = 5.88\ (PI) - 3.78\ (LL) + 28.09$ …… (6.14g)

From eq. (6.14a), the saturated volumetric water content (θ_s) is weakly related to the liquid limit (LL) , and negatively related to the plasticity index (PI). The residual water content (θ_r) in eq. (6.14b) has a much stronger relationship with the liquid limit and also a negative relationship with the plasticity index. Furthermore, the coefficients of the degree of initial saturation (Si), and the plasticity index (PI) of eqs. (6.14c and d) are very small, indicating poor relationships.

From eq. (6.14e), the air entry suction (ψ_a) is strongly related to the liquid limit (with a coefficient of 9.45), and negatively related to the plasticity index (with a coefficient of – 14.34). Also from eq. (6.14f), van Genuchten's alpha ($\propto$) is poorly related to the plasticity index (coefficient is 0.0057), and poorly negatively related to the liquid limit (coefficient is – 0.0156). Lastly, from eq. (6.14g), Brooks-Corey lambda (λ) is strongly related to the plasticity index (coefficient of 5.88) and quite negatively related to the liquid limit (coefficient of – 3.78). Furthermore, multiple regression analysis was used to study the direct

relationship between θ_s versus PI, θ_r versus LL, ψ_a versus LL, and λ versus PI; and the results presented in Table 6.5a. The F-values of these four cases are quite greater than the F-critical values, which make their relationships significant, however; their t-test values are all lower than their t-critical values, suggesting the absence of other influential soil parameters from Table 6.5a.

(b) Two-way analysis of variance (ANOVA) and t-test for Brooks-Corey's parameter lambda (λ): The pore-size distribution index, λ, which is a function of the distribution of pores in the soil, was subjected to ANOVA test and t-test based on two conditions; (1) θ_r is equal to zero, and (2) θ_r is greater than zero. The results are summarized in Table 6.5b. The F-values obtained are less than the F critical values which means two things; (1) the results are significant, and (2) the first F value (3.4693) is almost equal to the F-critical value (3.4381) and hence a borderline case, that is, the assumption of θ_r is equal to zero may not always be correct. In addition, the t-test value (3.1281) is less than the t-critical value (2.2281) which further supports the significance of the results.

(c) Two-way analysis of variance for the volumetric water content: The results of this test are shown in Tables 6.5c and d. The effect of the variation of the matric suction on the volumetric water content, in all the cases tested, is statistically significant as all the F-values (ranging from 19 to 2007.196) are greater than F-critical values (ranging from 1.0 to 18.513).

6.8 Batch equilibrium

The results of the batch-equilibrium tests for all the seven laterites (see Table 5.12) are presented as adsorption isotherms for the cations (Ca^{2+}, K^+, and Na^+) in Figs. 6.28a - g. All the isotherms exhibit a linear, though negative, trend. For Ca^{2+} and K^+, chemical sorption is more effective at lower concentrations while the sorption of Na^+ into these laterites tends

to occur more at higher concentrations.Generally, for low concentrations of the three cations, the resulting slope is equal to the distribution coefficient, K_d, since the isotherms are linear. This distribution coefficient, K_d, also known as partition coefficient, K_p, can be expressed in terms of the retardation factor, R_d, as (Shackelford *et al.*, 1989):

$$R_d = 1 + \left(\frac{\rho_d}{\theta}\right) K_d \quad \text{---------- (6.15)}$$

where θ is the volumetric water content.

Based on their slopes, the K_d or K_p values (for Na^+)obtained range from 0.393 cm^3/g for Laterite A to 0.463 cm^3/g for Laterite G, while the values for Laterites B-F are; 0.403 cm^3/g, 0.413 cm^3/g, 0.423 cm^3/g, 0.423 cm^3/g, and 0.440 cm^3/g respectively. Similarly the K_d values for K^+ range from 0.390 cm^3/g for Laterite A to 0.460 cm^3/g for Laterite G, while the values for Laterites B-F are; 0.400 cm^3/g, 0.407 cm^3/g, 0.417 cm^3/g, 0.430 cm^3/g, and 0.447 cm^3/g respectively. As for Ca^{2+}, the K_d values range from 0.400 cm^3/g for Laterite A to 0.446 cm^3/g for Laterite G, while the values for Laterites B-F are; 0.409 cm^3/g, 0.417 cm^3/g, 0.430 cm^3/g, 0.438 cm^3/g, and 0.441 cm^3/g respectively.

Their corresponding retardation factors, shown in Table 5.15, were calculated using eq. (6.15), and range from 3.653 to 2.481 for Na^+, 3.998 to 3.589 for K^+, and 7.011 to 5.821 for Ca^{2+}. All the values obtained are within the ranges got by other researchers (Shackelford *et al.*, 1989). As the values of the plasticity product decrease from 2100 for Laterite A to 530 for Laterite G, K_p values increase while R_d values decrease correspondingly. Since all the values of the retardation factor are greater than 1.0, therefore, a process of ionic retardation or attenuation must have taken place within the laterites.

6.9 Diffusion

The water content of the soil slices obtained after diffusion test versus the depth profile were subjected to statistical correlation analysis. A minimum correlation of 0.795 was obtained between Laterites E and A while a maximum correlation of 0.995 between Laterites F and D. The high correlation shows that the variation in moisture content with depth is similar for all the seven laterites. In addition, a two-way analysis of variance without replication was carried out, and the results show F value (45.6224) to be much greater than F-critical value (2.5336), pointing to the significance of these results.

The effective diffusion coefficients, D*, (see Table 5.15) were obtained by using eq. (5.42). They fall within the range reported by Shackelford and Redmond (1995). Generally, the D* value for Na^+ (0.003 x $10^{-8}m^2/s$) in Laterite A increased to the highest value in Laterite G (0.880 x $10^{-8}m^2/s$), while the values for Laterites B-F are; 0.010 x $10^{-8}m^2/s$, 0.019 x $10^{-8}m^2/s$, 0.100 x $10^{-8}m^2/s$, 0.230 x $10^{-8}m^2/s$, and 0.490 x $10^{-8}m^2/s$ respectively. Similarly, the D* value for K^+ (0.010 x $10^{-8}m^2/s$) in Laterite A increased to the highest value in Laterite G (1.24 x $10^{-8}m^2/s$), while the values for Laterites B-F are; 0.090 x $10^{-8}m^2/s$, 0.150 x $10^{-8}m^2/s$, 0.230 x $10^{-8}m^2/s$, 0.450 x $10^{-8}m^2/s$, and 0.980 x $10^{-8}m^2/s$ respectively.

Furthermore, the D* value for Ca^{2+} increased from 1.120 x $10^{-8}m^2/s$ for Laterite A to 2.010 x $10^{-8}m^2/s$ for Laterite G, while the values for Laterites B-F are; 1.330 x $10^{-8}m^2/s$, 1.510 x $10^{-8}m^2/s$, 1.600 x $10^{-8}m^2/s$, 1.760 x $10^{-8}m^2/s$, and 1.920 x $10^{-8}m^2/s$ respectively. Lastly for chloride diffusion, the D* value increased from 4.895 x $10^{-8}m^2/s$ for Laterite A to 6.662 x $10^{-8}m^2/s$ for Laterite G, while the values for Laterites B-F are; 1.330 x $10^{-8}m^2/s$, 1.510 x $10^{-8}m^2/s$, 1.600 x $10^{-8}m^2/s$, 1.760 x $10^{-8}m^2/s$, and 1.920 x $10^{-8}m^2/s$, respectively.

The gradual increase in the cations (Na^+, K^+, and Ca^{2+}) D* values from Laterite A to Laterite G seems to reflect the diffenrence in the plasticity product of the soils. The greater the plasticity product (highest for Laterite A) of the lateritic soil, the lower the D* values for

the three cations. Similarly, the same can be said for the anion (Cl^-). The presence of Na^+ in a high percentage in any soil normally leads to the dispersion of the soil particles, which is a very detrimental problem in compacted soil liners and other related geotechnical engineering matters. The percentage of Na^+ gotten from diffusion tests and other analyses (e.g. batch equilibrium tests) were not high, neither was there any record of alarming level of Na^+ presence. Therefore, the seven laterites used in this research work can be considered for liner materials and other geotechnical engineering designs without the fear of soil dispersion.

The tortuosity factor, τ, obtained was based on Cl^- effective diffusion coefficient using eq. (5.35). The tortuosity factor, τ, ranges from 0.241 for Laterite A to 0.328 for Laterite G, while the values for Laterites B-F are; 0.259, 0.266, 0.298, 0.300, and 0.310, respectively, all within the range of 0.1 to 0.58 for natural soil reported by Daniel and Shackelford (1988). As the plasticity product decreases from 2100 for Laterite A to 530 for Laterite G, the values for tortuosity factor and effective diffusion coeffiecient increases, highlighting the importance of soil compositional factors in lateritic soil attenuation.

During the 76 days of diffusion testing, the hydraulic conductivity was constantly monitored and found to decrease slightly in the pattern shown in Fig. 6.29. This means that the laterites used were quite compatible with the leachate. Furthermore, the ion concentration-depth profiles obtained for Laterites A to G are presented in Figs. 6.30a - g. Again, as the plasticity product increased, the attenuative capacity of the laterite also increased. Lastly, the solute breakthrough curves for the seven laterites are shown in Figs. 6.31a - g. They are a typical desorption curve, often reflecting calcium (Ca^{2+}) and magnesium (Mg^{2+}) displaced by sodium (Na^+) and potassium (K^+) in a permeating leachate (Rowe *et al*., 1996). Occasionally, the Ca^{2+} curve does not return to unity, in which case, long-term dissolution of carbonate or gypsum might be inferred. It is often very important

not to run such tests too long, in case loss of solids causes a large increase in laboratory hydraulic conductivity (breakthrough), something that could never happen in a field situation (Rowe *et al*., 1996). Some deductions that can be made from these breakthrough curves include:

(1) the curves show Na^+ and K^+ retardation due to adsorption onto lateritic clays, and the resultant displacement of Ca^{2+} and Mg^{2+} to form a halo effect which arrives early enough,

(2) in the absence of macropores, the soils with high plasticity product (e.g Laterite A) retard more K^+ and Na^+ than others,

(3) the abrupt flattening of the Na^+ curve and the long slope towards $c/c_0 = 1$ tailing indicates fracture or macropore flow with gradual diffusion of Na^+ into the pore fluid (Rowe *et al*., 1996) of the adjacent soil peds, and ultimately cation exchange onto the clay and desorption of Ca^{2+} and Mg^{2+},

(4) for sodium, C/C_0 reaches 0.5 at about 2.8PV of leachate as shown in Fig. 6.31 (a) for Laterite A, indicating the expected retardation (by adsorption due to the presence of smectitic minerals in the laterite), while C/C_0 reaches 0.5 at about 3.1PV of leachate in Laterite G (see Fig. 6.31(g)). As for Laterites B-F (see Figs 6.31(b-f)), C/C_0 for sodium reaches 0.5 at about 2.85 PV, 2.90 PV, 2.95 PV, 3.00 PV, and 3.05 PV respectively. All these reflect the influence of the plasticity product and soil mineralogy.

(5) for potassium, C/C_0 reaches 0.5 at about 2.8PV of leachate (see Fig. 6.31 (a)) for Laterite A, also indicating the expected retardation by adsorption onto the laterite, while C/C_0 reaches 0.5 at about 3.7PV of leachate in Laterite G as shown Fig. 6.31(g). As for Laterites B-F (see Figs 6.31(b-f)), C/C_0 for potassium reaches 0.5 at about 2.90 PV, 3.10 PV, 3.30 PV, 3.40 PV, and 3.60 PV respectively. All this also reflects the influence of the plasticity product and soil mineralogy.

6.10 Soil-Leachate Compatibility

From the results of the hydraulic conductivity tests conducted for all the seven laterites during the diffusion period (see Table 5.16 and Fig. 6.29); the following observations can be made:

1. Domestic waste leachate caused a slight reduction in hydraulic conductivity for all the seven laterites. As the pore volume varies from 0 to 20.0 mg/l, the k-values for Laterites A-G have the following ranges; 2.6E-8 cm/s to 9.6E-9 cm/s, 2.8E-8 cm/s to 1.0E-8 cm/s, 8.2E-8 cm/s to 4.8E-8 cm/s, 8.6E-8 cm/s to 5.7E-8 cm/s, 1.1E-8 cm/s to 7.3E-8 cm/s, 1.8E-8 cm/s to 7.8E-8 cm/s, and 1.8E-8 cm/s to 7.2E-8 cm/s, respectively.

2. The greater smectite content of Laterite A soil is partly responsible for its low k-value.

6.11 Acceptable zones based on modern design objectives

Using the procedure for modern design criterion (see Section 5.2.7), acceptable zones for all the seven laterites were delineated as shown in Figs. 6.32 (a-g). Three important factors considered in the development of the acceptable zones were as follows;

(1) The compacted soil should have a low hydraulic conductivity equal to or less than $1x10^{-7}$ cm/s (Daniel and Benson, 1990).

(2) Compacted soils used for liners and covers must have adequate shear strength with a minimum of $200KN/m^2$ (Daniel and Wu, 1993).

(3) A maximum allowable volumetric shrinkage strain of 4% (Albrecht and Benson, 2001).

CHAPTER SEVEN

CONCLUSIONS AND RECOMMENDATIONS

7.1 **Conclusions**

In order to study the influence of fines content on water and chemical flows through compacted lateritic soil liners, various laboratory tests, that included index properties, differential thermal analysis, compaction, volumetric shrinkage, unconfined compressive strength, hydraulic conductivity, soil water characteristics curve (SWCC) study for unsaturated hydraulic properties, batch equilibrium, and diffusion tests, were carried out using seven different laterites (Laterites A-G) and three compactive efforts (i.e. British Standard Heavy, West African Standard, and British Standard Light). The seven laterites vary in their index properties, and also physical and chemical properties. Laterite A has the highest fines of 70% and also the highest clay contents of 40%, while Laterite G has the least fine of 53% and least clay content of 17.5%. As for Laterites B-F, their fines content are 68%, 67%, 62%, 61%, and 56% respectively, while their clay contents are 37%, 31.5%, 26%, 22.2%, and 19.5%, respectively. In addition, the laterites vary in plasticity ($20 \leq LL \leq 54$; $10 \leq PI \leq 30$), particle size distribution ($53\% \leq$ fines $\leq 70\%$; $17.5\% \leq$ clay content $\leq 40\%$), cation exchange capacity (9.6 to 19.2 meq/100g of soil), and soil classification. From DTA analysis, smectite dominates Laterites A to E, while kaolinite dominates Laterites F and G.

From the compaction characteristics of the laterites, both the molding water content and compactive effort have a great influence on lowering the hydraulic conductivities of the soils. The hydraulic conductivities generally decreased from high values when compacted dry of optimum water content to minimum values at about the optimum water content, and then increased with an increase in the molding water content. The molding water content and

compactive effort had a direct influence on the volumetric shrinkage strain. The greater the effort the lower the strain. However, because of the serious effect of dessication-induced strain, less than 11% volumetric strain in the design standard for all lateritic liners is recommended. From the results of the unconfined compressive strength, generally, the UCS decreased as the molding water content increased for all the laterites. BSH effort had the greatest UCS values compared to the other two efforts (BSL and WAS), while WAS effort had the least. These results are in accordance with the reports of Daniel and Wu (1993) who recommended a minimum UCS value of 200 KN/m^2 for clay liners when defining an acceptable zone.

Based on the specifications for the municipal solid wastes liners according to Das (1998), (at least 15-20% clay-size particles, minimum activity of 0.3, and minimum CEC of 10 meq/100g of soil), Laterite A is the most suitable liner material. This was further confirmed from the stochastic model assessment used. The hydraulic conductivity results and index properties such as the clay content and plasticity index as well as the initial saturation were subjected to First Order Reliability Analysis (using FORM 5). The effect of compactive effort on reliability index for the seven laterites indicated that Laterite A had the highest beta values (1.88 to 3.85) and as such the most suitable liner material; Laterite G was the least suitable as it had the lowest beta values (0.993 to 2.992), while the beta values for the rest five laterites fell in between both extremes. Also, the BSH effort has the highest reliability indices, followed by WAS effort, while BSL effort is the least. Apart from the influence of the compactive effort, the initial saturation is another critical variable that must be controlled carefully in achieving low hydraulic conductivity ($k \leq 1x10^{-7}$ cm/s), while the plasticity index and clay content do not call for strict design restrictions. These results are in conformity with the reports of Benson and Trast (1995). A practical application is in the area of screening

borrow materials quickly. Based on a minimum hydraulic conductivity $k \leq 1x10^{-7}$cm/s, a minimum UCS value of 200 KN/m^2, and a maximum allowable volumetric shrinkage strain of 4%, acceptable zones were defined for the seven laterites.

Furthermore, the results of SWCC study for desorption show that higher air-entry suctions are obtained for more plastic soils, and the shape of SWCC depends greatly on compaction conditions and soil compositional variables. The SWCC data were used in conjunction with Brooks-Corey's and van Genuchten's model to estimate the unsaturated hydraulic conductivities. Parameters describing the SWCC for laterites can be estimated using the empirical equations developed in this work. Thus, in modelling flow and transport analysis for lateritic liners, though the empirical equations from regression analysis can be useful in the selection of soil samples, appropriate confirmatory tests must be used to enhance success.

The results of the batch equilibrium tests (linear adsorption isotherms for all the seven laterites) show that the distribution coefficient, K_d, or partition coefficient, K_p, has values ranging from 0.393 cm^3/g for Laterite A to 0.463 cm^3/g for Laterite G, while the values for Laterites B-F are; 0.403 cm^3/g, 0.413 cm^3/g, 0.423 cm^3/g, 0.423 cm^3/g, and 0.440 cm^3/g, respectively. Similarly the K_d values for K^+ range from 0.390 cm^3/g for Laterite A to 0.460 cm^3/g for Laterite G, while the values for Laterites B-F are; 0.400 cm^3/g, 0.407 cm^3/g, 0.417 cm^3/g, 0.430 cm^3/g, and 0.447 cm^3/g respectively. As for Ca^{2+}, the K_d values range from 0.400 cm^3/g for Laterite A to 0.446 cm^3/g for Laterite G, while the values for Laterites B-F are; 0.409 cm^3/g, 0.417 cm^3/g, 0.430 cm^3/g, 0.438 cm^3/g, and 0.441 cm^3/g, respectively. Their corresponding retardation factors range from 3.653 to 2.481 for Na^+, 3.998 to 3.589 for K^+, and 7.011 to 5.821 for Ca^{2+}. This shows that as the plasticity product decreases, the K_p values increase while the R_d values decrease correspondingly. These results are in conformity with those of Shackelford *et al*. (1989).

Using an analytical solution to Fick's second law, the results of the effective diffusion coefficient (D*) show that the greater the plasticity product of the laterite, the lower the D* values for Na^+, K^+, and Ca^{2+}. Generally, the D* value for Na^+ (0.003 x $10^{-8}m^2/s$) in Laterite A increases to the highest value in Laterite G (0.880 x $10^{-8}m^2/s$), while the values for Laterites B-F are; 0.010 x $10^{-8}m^2/s$, 0.019 x $10^{-8}m^2/s$, 0.100 x $10^{-8}m^2/s$, 0.230 x $10^{-8}m^2/s$, and 0.490 x $10^{-8}m^2/s$ respectively. Similarly, the D* value for K^+ (0.010 x $10^{-8}m^2/s$) in Laterite A increases to the highest value in Laterite G (1.24 x $10^{-8}m^2/s$), while the values for Laterites B-F are; 0.090 x $10^{-8}m^2/s$, 0.150 x $10^{-8}m^2/s$, 0.230 x $10^{-8}m^2/s$, 0.450 x $10^{-8}m^2/s$, and 0.980 x $10^{-8}m^2/s$ respectively.

Furthermore, the D* value for Ca^{2+} increased from 1.120 x $10^{-8}m^2/s$ for Laterite A to 2.010 x $10^{-8}m^2/s$ for Laterite G, while the values for Laterites B-F are; 1.330 x $10^{-8}m^2/s$, 1.510 x $10^{-8}m^2/s$, 1.600 x $10^{-8}m^2/s$, 1.760 x $10^{-8}m^2/s$, and 1.920 x $10^{-8}m^2/s$ respectively. Lastly for chloride diffusion, the D* value increases from 4.895 x $10^{-8}m^2/s$ for Laterite A to 6.662 x $10^{-8}m^2/s$ for Laterite G, while the values for Laterites B-F are; 1.330 x $10^{-8}m^2/s$, 1.510 x $10^{-8}m^2/s$, 1.600 x $10^{-8}m^2/s$, 1.760 x $10^{-8}m^2/s$, and 1.920 x $10^{-8}m^2/s$ respectively. All the D* values fall within the range reported by Shackelford and Redmon (1995). The tortuosity factor, τ, obtained ranges from 0.241 for Laterite A to 0.328 for Laterite G, while the values for Laterites B-F are; 0.259, 0.266, 0.298, 0.300, and 0.310 respectively, all within the range of 0.1 to 0.58 for natural soil reported by Daniel and Shackelford (1988). Generally, as the plasticity product decreased from 2100 for Laterite A to 530 for Laterite G, the values of tortuosity factor and D* increased, highlighting the importance of soil compositional factors in lateritic soil attenuation.

The solute breakthrough curves for the seven laterites are a typical desorption curve often reflecting calcium (Ca^{2+}) and magnesium (Mg^{2+}) displaced by sodium (Na^+) and

potassium (K^+) in a permeating leachate to form a halo effect. Also, in the absence of macropores, the soils with high plasticity product retarded more K^+ and also Na^+ than the others. These results are in conformity with those of Rowe *et al.* (1996).

The results of the hydraulic conductivity tests conducted for all the seven laterites during the 76 days of diffusion testing show very little changes in the hydraulic conductivities throughout the period. Therefore, the leachate used can be said to be compatible with all the laterites. Also, since the predominant smectite minerals in Laterites A to E are more active in sorption reactions which ultimately results to the removal of cations such as Na^+, K^+, and Ca^{2+} from leachates (or municipal solid wastes) than the less active kaolinite minerals which predominate Laterites F and G, therefore; Laterites A to E should make better liner materials for clay attenuation.

7.2 Recommendations

Based on the above observations and conclusions, the following recommendations are being made.

(1) Field compaction should normally be wet of standard Proctor optimum to achieve easily the regulatory hydraulic conductivity ($k \leq 1x10^{-7}$cm/s).

(2) A BSL effort easily resulted into k-values higher than the regulatory hydraulic conductivity ($k \leq 1x10^{-7}$cm/s), therefore, should not be used. Compaction should normally be by a minimum of WAS or BSL effort so that the interclod voids are destroyed.

(3) The acceptable zones defined for the seven laterites should be used to enhance success.

(4) Most borrow pits contain stratified materials, so careful control may be required to obtain a uniform field distribution of minimum hydraulic conductivity (k) values.

(5) Even homogeneous clays are often variably weathered so that successive lifts have different optimum water content, and k-values.

(6) Liners should be immediately covered and kept moist enough to prevent shrinkage until control k-testing permits waste disposal. Similarly, smectitic liner components must not be allowed to free-swell (Rowe *et al*., 1996).

(7) Desiccated liner sections (dry weather compaction) should be removed, pulverized, rewetted and recompacted.

(8) Liner sections must be protected from severe weather, and if damaged should be replaced.

(9) Wet-of-optimum compaction that produces a minimum k-value, produces a clay wet enough to experience chemical shrinkage on exposure to certain leachates. Clay-leachate compatibility testing is therefore necessary for soils for which no data exist or whose clay mineralogy indicates reactive mineral constituents.

(10) Out of the seven laterites considered, Laterite A is the most suitable liner material on account of all the various laboratory tests carried out, while Laterite G is the least. Hence, Laterite G (and also Laterite F) should not be used alone for liners, while the other five laterites (Laterites A-E) are a better choice.

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APPENDIX

Table 5.14: Concentration – depth profiles

Ion concentration (mg/l)

Depth (cm)	$Cl^{-}\tau$	Na^{+}	K^{+}	Ca^{2+}	Soil Type
0.5	310	21	26	9	Laterite A
1.0	280	11	18	5.5	
1.5	249	8	12	3.5	
2.0	230	6	9	2.5	
2.5	215	5.5	7	1.8	
3.0	210	5	6	1.3	
0.5	290	23	29	10	Laterite B
1.0	251	13	20	7	
1.5	225	9	13	5	
2.0	200	8	10	3.5	
2.5	190	6.5	8	2.9	
3.0	185	6	7	2.0	
0.5	265	24	32	11	Laterite C
1.0	230	15	22	8.5	
1.5	200	10	15	6	
2.0	185	9	12	4	
2.5	170	8	9	3.5	
3.0	164	7	8	3	
0.5	240	26	33	12	Laterite D
1.0	200	17	24	9.5	
1.5	184	11	17	7	
2.0	165	10.5	13	5.5	
2.5	152	10	11	4	
3.0	145	8.5	9	3.5	

0.5	220	29	35	12.5	Laterite E
1.0	190	19	26	10	
1.5	160	13	19	8.5	
2.0	140	12	14	7.0	
2.5	130	11	12	6.5	
3.0	110	10	11	5.5	
0.5	185	32	37	13.5	Laterite F
1.0	140	22	28	11.4	
1.5	100	17	22	9.5	
2.0	90	14.5	16	8	
2.5	80	13	14	7.5	
3.0	70	12.5	13	6.5	
0.5	160	33	40	15	Laterite G
1.0	100	25	30	12.5	
1.5	70	19	23	10.5	
2.0	60	16	18	10	
2.5	55	15	16	9	
3.0	50	14	15	8	

Table 5.16: Hydraulic conductivities for compacted soil-leachate compatibility tests

Soil Type: k(cm/s), Hydraulic Conductivity	Pore volume (mg/l) 0	4	4.1	4.2	8	8.1	8.2	12	12.1	12.2	16	16.1	16.2	
Laterite A: k=	2.6E-8	2.0E-8	2.1E-8	2.0E-8	1.1E-8	1.2E-8	1.1E-8	1.0E-8	1.1E-8	1.0E-8	9.8E-9	9.9E-9	9.8E-9	
Laterite B:	2.8E-8	2.4E-8	2.5E-8	2.4E-8	2.0E-8	2.1E-8	2.0E-8	1.8E-8	1.9E-8	1.8E-8	1.4E-8	1.5E-8	1.4E-8	
Laterite C:	8.2E-8	6.8E-8	6.9E-8	6.8E-8	6.4E-8	6.5E-8	6.4E-8	6.0E-8	6.1E-8	6.0E-8	5.7E-8	5.8E-8	5.7E-8	
Laterite D:	8.6E-8	8.2E-8	8.3E-8	8.2E-8	7.8E-8	7.9E-8	7.8E-8	7.5E-8	7.6E-8	7.5E-8	6.3E-8	6.4E-8	6.3E-8	
Laterite E:	1.1E-8	1.0E-8	1.1E-8	1.0E-8	9.5E-8	9.6E-8	9.5E-8	8.9E-8	9.0E-8	8.9E-8	8.0E-8	8.1E-8	8.0E-8	
Laterite F:	1.8E-8	1.2E-8	1.3E-8	1.2E-8	9.7E-8	9.8E-8	9.7E-8	9.2E-8	9.3E-8	9.2E-8	8.3E-8	8.4E-8	8.3E-8	
Laterite G:	1.8E-8	1.0E-8	1.1E-8	1.0E-8	9.4E-8	9.5E-8	9.4E-8	8.9E-8	9.0E-8	8.9E-8	8.0E-8	8.1E-8	8.0E-8	

Table 5.17: Results of solute breakthrough curves for the Laterites

Soil Type	Pore volume (mg/l) 1.7	4	8	12	16	20	
Laterite G: K^+	0	0.45	0.87	0.96	0.98	0.99	
Na^+	0	0.65	0.8	0.9	0.97	0.99	C/C_O
Ca^{2+}	0	1.06	1.05	1.03	1.02	1.01	
Laterite F: K^+	0	0.69	0.77	0.87	0.96	0.99	
Na^+	0	0.68	0.85	0.94	0.98	0.99	C/C_O
Ca^{2+}	0	1.08	1.006	1.03	1.02	1.01	
Laterite E: K^+	0	0.69	0.91	0.97	0.98	0.99	
Na^+	0	0.7	0.82	0.9	0.94	0.95	C/C_O
Ca^{2+}	0	1.14	1.10	1.06	1.03	1.01	
Laterite D: K^+	0	0.6	0.75	0.88	0.96	0.99	
Na^+	0	0.65	0.78	0.91	0.98	0.99	C/C_O
Ca^{2+}	0	1.15	1.12	1.08	1.05	1.01	
Laterite C: K^+	0	0.86	0.96	0.97	0.98	0.99	
Na^+	0	0.81	0.90	0.93	0,95	0.99	C/Co
Ca^{2+}	0	1.16	1.13	1.09	1.06	1.01	
Laterite B: K^+	0	0.60	0.94	0.96	0.97	0.99	
Na^+	0	0.65	0.77	0.92	0.98	0.99	C/C_O
Ca^{2+}	0	1.17	1.14	1.10	1.07	1.01	
Laterite A: K^+	0	0.85	0.95	0.97	0.98	0.99	
Na^+	0	0.75	0.84	0.90	0.97	0.99	C/C_O
Ca^{2+}	0	1.18	1.15	1.11	1.08	1.01	

Appendix B

Determination of the specific gravity of soil particles (standard laboratory method)

Operator: Ibrahim Bajeh **Job:- Ph.D research**

Soil number/type:- Basawa pit (Laterite A)

Temp. of test: About 28^{o}C

Bottle number	1	2	3
Wt. of bottle + water (full) W_4	42.45	42.65	42.45
Wt. of bottle + soil + water W_3	45.90	46.75	45.40
Wt. of bottle + soil W_2	23.80	24.00	22.75
Wt. of bottle W_1	18.10	17.10	18.10
Wt. addit. of water $(W_4 - W_1)$	24.35	24.85	24.35
Wt. of water added to soil $(W_3 - W_2)$	22.10	22.75	22.65
Wt. of soil $(W_2 - W_1)$	5.70	6.20	4.65
Wt. of water displaced by soil $(W_4 - W_1) - (W_3 - W_2) = W$	2.25	2.10	1.70
Specific gravity of soil particles $(W_2 - W_1)/W$	2.53	2.95	2.74
Average specific gravity, G_s		2.74	

Appendix C

SIEVE ANALYSIS OF SOIL (WET/DRY SIEVING)

Job: Ph.D research Operator: Ibrahim Bajeh
Site: Basawa
Sample No: Basawa pit (Laterite A) Description: Reddish brown Laterite
Total weight of dry sample 200 gm

B.S.Sieve size	Mass Retained	Mass Passing	Summation percentage	Remarks	Max Sieve load
	g	g	%		g
$^1/_8$ in.	0	200.00	100		300
No. 7	13.30	186.70	93		200
No. 14	4.85	181.85	91		100
No. 25	4.75	177.10	89		75
No. 36	19.90	157.20	79		75
No. 52	6.10	151.10	76		50
No. 72	5.75	145.35	73		50
No. 100	3.00	142.35	71		40
No. 200	1.54	140.81	70		25
Passing 200	0.78				

Appendix D1

Hydrometer sedimentation test data and results for Laterite A

Loc. No: Basawa pit **Original wt of dry soil: 100gm**

Hydrom. No: 773122

Soil type: Laterite A **Final wt of dry soil: 40gm** **S.G: 2.74**

Date Time	Elasped time t minutes	Hydrometer reading R^1_h	Effective depth (mm) H_R	Modified reading R_d	Particle Diameter D micron	Percentage finer than D, K(%)
22-11-01 10.07am	0	-	-	-	-	-
	0.5	48.6	22.5	40.9	27.2	64.5
	1	43.8	41.2	40.6	26.0	64
	2	40.2	55.2	40.5	21.3	63.7
	4	40	56.0	40.3	15.2	63.4
	8	37.5	65.8	37.8	11.6	59.4
	15	34	79.4	35.3	9.3	55.5
	30	28	102.8	34.9	7.5	54.9
11.17am	60	22	126.2	32.2	5.8	50.7
12.17pm	120	5	192.5	30.6	5.1	48.2
2.17pm	240	2	204.2	29.2	3.7	46
6.17pm	480	1.5	206.2	28	2.6	44
10.17pm	720	1	208.1	25.4	2.0	40
10.17am	1440	0.5				

Table 5.3. Compaction and hydraulic conductivity data

Soil Type	Molding water content (%)	Dry unit wt kN/m^3	Compactive Effort	k (cm/s)
Laterite A	17.00	15.03	BS Light	8.1E – 08
	19.00	15.73	BS Light	6.9E – 08
	21.40	16.00	BS Light	5.7E – 08

	23.00	15.74	BS Light	2.1E – 08
	25.00	15.19	BS Light	8.4E – 08
	16.00	15.62	WAS	3.6E – 08
	18.00	16.09	WAS	2.3E – 08
	20.00	16.25	WAS	2.0E – 08
	22.00	15.74	WAS	1.4E – 08
	24.00	15.36	WAS	5.1E – 08
	15.00	17.82	BS Heavy	8.4E – 09
	17.00	18.45	BS Heavy	7.8E – 09
	19.00	17.50	BS Heavy	5.8E – 09
	21.00	16.30	BS Heavy	7.1E – 09
	23.00	15.82	BS Heavy	8.0E – 09
Laterite B	10.10	15.14	BS Light	3.0E – 07
	12.10	15.97	BS Light	2.2E – 07
	14.10	16.18	BS Light	5.4E – 08
	16.10	15.82	BS Light	4.1E – 08
	18.10	15.00	BS Light	7.2E – 08
	9.00	16.44	WAS	8.3E – 08
	11.00	16.57	WAS	5.2E – 08
	13.00	17.20	WAS	4.4E – 08
	15.00	18.77	WAS	2.9E – 08
	17.00	16.71	WAS	5.5E – 08
	8.10	18.24	BS Heavy	2.6E – 08
	10.10	18.83	BS Heavy	1.9E – 08
	12.10	18.00	BS Heavy	9.6E – 09
	14.10	17.13	BS Heavy	9.9E – 09
	16.10	16.07	BS Heavy	1.2E – 08
Laterite C	7.90	16.43	BS Light	3.4E – 06
	9.90	17.14	BS Light	8.7E – 07
	11.90	17.28	BS Light	1.2E – 07
	13.90	16.80	BS Light	0.9E – 07
	15.90	15.94	BS Light	4.2E – 07
	6.80	17.45	WAS	1.9E – 06
	8.80	18.24	WAS	7.9E – 07
	10.80	17.24	WAS	5.1E – 08
	12.80	17.66	WAS	6.2E – 08
	14.80	17.03	WAS	6.9E – 08
	3.90	18.89	BS Heavy	2.8E – 08

	5.90	21.38	BS Heavy	2.0E – 08
	7.90	18.77	BS Heavy	1.6E – 08
	9.90	17.77	BS Heavy	2.1E – 08
	11.90	17.77	BS Heavy	3.4E – 08
Laterite D	11.00	15.14	BS Light	6.8E – 07
	12.00	15.97	BS Light	5.2E – 07
	15.00	16.18	BS Light	4.8E - 07
	17.00	15.82	BS Light	2.1E – 07
	19.00	15.00	BS Light	6.3E –07
	11.00	16.44	WAS	3.3E – 07
	13.00	16.57	WAS	3.2E – 08
	15.00	19.40	WAS	0.8E – 08
	17.00	16.77	WAS	6.9E – 09
	19.00	16.71	WAS	9.2E – 08
	7.80	18.24	BS Heavy	3.7E – 08
	9.80	19.83	BS Heavy	3.2E – 08
	11.80	18.00	BS Heavy	1.9E – 08
	13.80	17.13	BS Heavy	2.6E – 08
	15.80	16.07	BS Heavy	3.3E – 08
Laterite E	12.60	15.92	BS Light	2.4E – 06
	14.60	16.53	BS Light	1.9E – 06
	16.60	17.34	BS Light	5.2E – 07
	18.60	16.42	BS Light	1.1E – 07
	20.00	16.40	BS Light	7.2E – 07
	14.00	15.93	WAS	4.4E – 07
	16.00	19.40	WAS	1.4E– 07
	18.00	18.60	WAS	1.2E – 07
	20.00	16.53	WAS	2.4E – 07
	22.00	15.53	WAS	4.1E – 07
	7.80	16.49	BS Heavy	5.7E – 08
	9.80	19.51	BS Heavy	4.9E – 08
	11.80	20.85	BS Heavy	2.8E – 08
	13.80	19.20	BS Heavy	1.0E – 08
	15.80	16.16	BS Heavy	4.6E – 08
Laterite F	21.10	15.67	BS Light	7.1E – 06
	23.10	16.16	BS Light	5.4E – 06
	25.10	15.28	BS Light	2.3E – 06

	27.10	15.14	BS Light	3.4E – 06
	29.10	14.89	BS Light	4.1E – 06
	20.00	16.29	WAS	2.9E – 06
	22.00	17.10	WAS	2.2E – 06
	24.00	16.40	WAS	1.9E – 06
	26.00	15.23	WAS	2.3E – 06
	28.00	14.23	WAS	4.2E – 06
	14.30	1669	BS Heavy	9.6E – 07
	16.30	19.95	BS Heavy	8.3E – 07
	18.30	16.89	BS Heavy	7.6E – 07
	20.30	16.41	BS Heavy	8.4E – 07
	22.30	15.23	BS Heavy	9.2E – 07
Laterite G	16.00	15.76	BS Light	9.9E - 06
	18.00	15.90	BS Light	6.4E – 06
	20.00	16.84	BS Light	4.4E – 06
	22.00	16.13	BS Light	1.1E – 06
	24.00	16.09	BS Light	7.9E – 06
	15.00	16.60	WAS	4.9E – 06
	17.00	17.02	WAS	3.4E – 06
	19.00	18.20	WAS	2.8E – 06
	21.00	17.59	WAS	0.6E – 06
	23.00	15.92	WAS	5.4E – 06
	10.70	18.94	BS Heavy	9.9E – 07
	12.70	20.06	BS Heavy	9.6E – 07
	14.70	18.32	BS Heavy	8.3E – 07
	16.70	17.59	BS Heavy	8.8E – 07
	18.70	16.47	BS Heavy	9.5E – 07

Table 5.12: Batch equilibrium test results

Ions	Equilibrium concent. mg/l			Mass of solute adsorbed micro g/g			Soil Type
	1	2	3	1	2	3	
Ca^{2+}	25.0	12.5	25.0	34.0	84.0	34.0	Laterite F

K^+	138	138	192	468	468	252	
Na^+	161	146	140	75.8	90.8	96.8	
Ca^{2+}	22.0	25	33	46	34	2	Laterite G
K^+	154	146	154	404	436	404	
Na^+	146	138	175	363	395	247	
Ca^{2+}	19	12.5	12.5	58	84	84	Laterite B
K^+	154	138	185	404	468	280	
Na^+	185	154	169	207	331	271	
Ca^{2+}	33	19	25	2	58	34	Laterite C
K^+	146	185	200	436	280	220	
Na^+	146	177	154	363	239	331	
Ca^{2+}	29	19	19	18	58	58	Laterite D
K^+	131	162	138	496	372	468	
Na^+	138	154	146	395	331	363	
Ca^{2+}	25	12.5	12.5	34	84	84	Laterite E
K^+	138	138	209	468	468	184	
Na^+	169	162	138	271	299	395	
Ca^{2+}	25	12.5	16	34	84	70	Laterite A
K^+	108	146	123	588	436	528	
Na^+	131	131	138	423	423	395	

Table 5.4. Volumetric Shrinkage and Unconfined compressive strength (UCS) tests

Soil Type	Molding water content (%)	Dry unit wt kN/m^3	Compactive Effort	Vol. Shrinkage (%)	UCS (kN/m^2)
Laterite A	17.00	15.03	BS Light	3.8	216
	19.00	15.73	BS Light	5.5	170
	21.40	16.00	BS Light	11.0	80
	23.00	15.74	BS Light	11.5	50
	25.00	15.19	BS Light	14.4	23
	16.00	15.62	WAS	2.0	325
	18.00	16.09	WAS	2.3	252
	20.00	16.25	WAS	4.9	230
	22.00	15.74	WAS	11.0	78
	24.00	15.36	WAS	12.5	48
	15.00	17.82	BS Heavy	0.5	880
	17.00	18.45	BS Heavy	2.4	775
	19.00	17.50	BS Heavy	4.6	722
	21.00	16.30	BS Heavy	8.0	424
	23.00	15.82	BS Heavy	10.5	180
Laterite B	10.10	15.14	BS Light	3.7	162
	12.10	15.97	BS Light	5.3	240
	14.10	16.18	BS Light	10.5	85
	16.10	15.82	BS Light	10.9	51
	18.10	15.00	BS Light	13.0	22
	9.00	16.44	WAS	1.8	225
	11.00	16.57	WAS	2.0	250
	13.00	17.20	WAS	7.5	326
	15.00	18.77	WAS	10.2	77
	17.00	16.71	WAS	11.0	50
	8.10	18.24	BS Heavy	0.4	719
	10.10	18.83	BS Heavy	2.1	776
	12.10	18.00	BS Heavy	4.1	878
	14.10	17.13	BS Heavy	7.5	422
	16.10	16.07	BS Heavy	9.9	177
Laterite C	7.90	16.43	BS Light	3.6	159
	9.90	17.14	BS Light	5.1	222

	11.90	17.28	BS Light	10.1	96
	13.90	16.80	BS Light	10.5	54
	15.90	15.94	BS Light	12.2	23
	6.80	17.45	WAS	1.7	230
	8.80	18.24	WAS	1.9	256
	10.80	17.24	WAS	7.3	318
	12.80	17.66	WAS	10.0	71
	14.80	17.03	WAS	10.8	48
	3.90	18.89	BS Heavy	0.4	729
	5.90	21.38	BS Heavy	2.0	780
	7.90	18.77	BS Heavy	4.0	875
	9.90	17.77	BS Heavy	7.3	413
	11.90	17.77	BS Heavy	9.8	190
Laterite D	11.00	15.14	BS Light	3.5	166
	12.00	15.97	BS Light	5.0	245
	15.00	16.18	BS Light	10.0	87
	17.00	15.82	BS Light	10.5	58
	19.00	15.00	BS Light	12.1	31
	11.00	16.44	WAS	1.6	230
	13.00	16.57	WAS	1.8	248
	15.00	19.40	WAS	6.9	330
	17.00	16.77	WAS	10.0	82
	19.00	16.71	WAS	10.7	45
	7.80	18.24	BS Heavy	0.4	714
	9.80	19.83	BS Heavy	1.9	780
	11.80	18.00	BS Heavy	2.0	891
	13.80	17.13	BS Heavy	7.2	431
	15.80	16.07	BS Heavy	96	181
Laterite E	12.60	15.92	BS Light	3.5	158
	14.60	16.53	BS Light	4.9	220
	16.60	17.34	BS Light	9.8	90
	18.00	16.42	BS Light	10.4	49
	20.60	16.40	BS Light	11.9	22
	14.00	15.93	WAS	1.5	228
	16.00	19.40	WAS	1.7	225
	18.00	18.60	WAS	6.6	321
	20.00	16.53	WAS	9.8	77
	22.00	15.53	WAS	10.5	46

	7.80	16.49	BS Heavy	0.4	720
	9.80	19.51	BS Heavy	1.8	771
	11.80	20.85	BS Heavy	2.0	872
	13.80	19.20	BS Heavy	6.6	414
	15.80	16.16	BS Heavy	9.1	178
Laterite F	21.10	15.67	BS Light	3.4	161
	23.10	16.16	BS Light	4.7	242
	25.10	15.28	BS Light	9.8	84
	27.10	15.14	BS Light	10.2	50
	29.10	14.89	BS Light	11.8	20
	20.00	16.29	WAS	1.5	228
	22.00	17.10	WAS	1.7	249
	24.00	16.40	WAS	6.0	325
	26.00	15.23	WAS	9.6	78
	28.00	14.23	WAS	10.2	47
	14.30	16.69	BS Heavy	0.3	710
	16.30	19.95	BS Heavy	1.7	768
	18.30	16.89	BS Heavy	2.0	870
	20.30	16.41	BS Heavy	6.4	420
	22.30	15.23	BS Heavy	9.0	172
Laterite G	16.00	15.76	BS Light	3.3	159
	18.00	15.90	BS Light	4.5	333
	20.00	16.84	BS Light	9.4	84
	22.00	16.13	BS Light	10.1	50
	24.00	16.09	BS Light	11.7	23
	15.00	16.60	WAS	1.3	228
	17.00	17.02	WAS	1.6	249
	19.00	18.20	WAS	5.8	325
	21.00	17.59	WAS	9.5	79
	23.00	15.92	WAS	9.9	45
	10.70	18.94	BS Heavy	0.3	711
	12.70	20.06	BS Heavy	1.6	765
	14.70	18.32	BS Heavy	1.9	870
	16.70	17.59	BS Heavy	6.2	413
	18.70	16.47	BS Heavy	8.8	171