ENVIRONMENTAL GEOTECHNICS

Edited by

International Technical Committee No. 5 (ITC5) on Environmental Geotechnics of the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE)

First Edition: September 2005

Second Edition: June 2006

CONTENTS

PREFACE

This report has been generated as part of the activities of International Technical Committee No. 5 (ITC5) on Environmental Geotechnics of the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) during the period 2001-2005. The primary aim in generating this report is to provide an overview of the state-of-the-art and the state-of-practice pertaining to three main areas of environmental geotechnics, viz., waste disposal by landfills, remediation of contaminated soils and underwater geoenvironmental issues. All three of these topics are considered from both scientific and technical points of view, and are covered for the purpose of both practitioners and researchers working in the field of environmental geotechnics, as well as individuals in other professional areas (e.g., chemical engineering, environmental engineering) who desire a comprehensive, yet relatively concise, overview of the field of environmental geotechnics.

The report describes the roles and functions of the fundamental components required for safe disposal of polluting materials or remediation of polluted lands from the standpoint of design, control and management. Attention also is given to research currently in progress that is devoted to improving our knowledge of design parameters significantly influencing the performance of modern landfill containment systems. The introduction and, in some cases, reintroduction of performance-based design as a more desirable approach relative to prescribed or regulated design and its increasing acceptance by the geoenvironmental community has led to a rethinking of how the design of containment systems should be approached. With the trend towards performance-based design, the design engineer must take into account numerous parameters, including contaminant transport parameters and the service life of clay (mineral) barriers, drainage layers, geosynthetics, and the main features of the waste in order to be able to estimate the leachate quality and production over the landfill activity and post-closure periods. Today, most of these parameters are rather readily assessable, whereas others are presently being investigated through research programs and, therefore, still are not considered fully reliable.

Chapter 1 includes a description of the design basics and performance criteria in the field of environmental geotechnics, including a detailed collection of definitions and proper terminology, as well as descriptions of the primary consideration for design, construction, quality control and risk assessment. Chapter 2 provides a generalized overview of the requirements for managing contaminated sites from the viewpoint of risk and site assessment. Chapter 3 provides a comprehensive overview of the topic of traditional barrier technologies for waste disposal, as well as a description of some of the more novel and innovative barrier technologies currently being evaluated or researched for possible use in waste disposal applications. Chapter 4 provides an overview of the management and utilization of waste sludge and dredging, including recent developments in dredging operations and containment techniques, and beneficial use of dredged materials for reclamation. Chapter 5 describes the performance of solid waste landfills and lining systems during earthquakes including the analysis of solid waste landfill stability. Finally, Chapter 6 covers the important topic of education in environmental geotechnics, including an overview of the current status of undergraduate and graduate education programs of some universities, primarily in Europe (EU) and United States (USA), with the goal of highlighting the primary aspects required for formal education in environmental geotechnics.

The contributors to the report include members of ITC5 who served as the focal points for preparation of each of the chapters, as well as a number of individuals, both ITC5 members and others, who contributed to various parts of the chapter. These individuals are recognized at the beginning of each chapter. In addition to the contributors, each chapter of the report was peer reviewed externally by a number of recognized experts, as follows: Chapter 1: David Daniel (USA) and Jorge Zornberg (USA); Chapter 2: Akram Alshawabkeh (USA) and Ian Hosking (Australia); Chapter 3: John Boweders (USA) and Russell Jones (UK); Chapter 4: Andy Fourie (South Africa) and Mark Van den Broek (Belgium); Chapter 5: George Buckovalas (Greece) and Edward Kavazanjian (USA); and Chapter 6: Pietro Jarre (Italy) and Hywel Thomas (UK). The editors sincerely appreciate the efforts of all contributors and reviewers. Finally, appreciation is extended to Rolf Katzenbach (Germany), who provided much of the impetus for this report during his tenure as the former ITC5 Chair.

September 2005

Mario Manassero (Italy), *ITC5 Co-Chair*
Charles Shackelford (USA), *ITC5 Co-Chair*
William van Impe (Belgium), *ISSMGE President*

A 7th Chapter has been added to the first version of the TC5 report (dated September 2005). Chapter 7 is devoted to nuclear waste disposal and management and describes the roles and functions of the fundamental components required for safe disposal of nuclear waste. The environmental problems and potential solutions are treated distinguishing uranium tailings, low and intermediate level nuclear waste and high level nuclear waste.

As for the other six Chapters, the topics are covered for the purpose of practitioners and researchers working in the field of Environmental Geotechnics, and of individuals in other professional areas who desire an overview of the nuclear waste issues.

June 2006

Mario Manassero (Italy), *TC5 Chairman*

Chapter 1

Design Basics and Performance Criteria

Author:

Roger CLARK
CL Associates, Leamington, United Kingdom

Contributing Authors:

E. KODA, M. LIPINSKI, W. WOLSKI (§ 1.5)
Warsaw Agricultural University, Poland
R.K. ROWE (§ 1.6)
Queen's University, Kingstone, Canada
J. VAN GULCK (§ 1.6)
University of Manitoba, Canada

The authors gratefully acknowledge the review of this chapter and the contribution made to it by:

David E. DANIEL, *University of Illinois, Urbana, USA*
Jorge G. ZORNBERG, *The University of Texas–Austin, USA*

1.1 TERMINOLOGY, DEFINITIONS AND UNITS

Abiotic
Not involving (formation or degradation) biological processes.

Absorption
Incorporation of a chemical compound or species within a solid or a liquid by penetration into the bulk mass of the substance.

Active "Barrier"
In-situ wall or element that absorbs, reacts with, or degrades contaminants in groundwater migrating through it.

Active Protective Measure
Process designed to control or mitigate an identified hazard or risk.

Adsorption
Retention of chemical compounds or species onto the surface of another substance.

Advection
Process of transfer of fluids (gases and/or liquids) and solutes through a medium in response to a pressure gradient.

Aeration
Process of bringing air into contact with a liquid or a solid.

Aerobic
Able to live, grow or take place only when sufficient oxygen is present.

Aerobic Biological Treatment
Biological treatment in the presence of oxygen.

Air Sparging
Introduction of air under pressure into the ground water for treatment and/or removal of contamination.

Air Stripping
Mass transfer process that enhances the volatilisation of compounds from water by passing air through the water to improve the transfer between the air and water phases. Also Steam Stripping where steam is used instead of air.

Aliphatic
Of or pertaining to a broad category of carbon compounds distinguished by a straight, or branched, open chain arrangement of the constituent carbons. The carbon-carbon bonds may be either saturated or unsaturated. Alkanes, alkenes and alkynes are aliphatic hyrocarbons.

Alkanes
Homologous group of linear saturated aliphatic hydrocarbons having the general formula C_nH_{2n+2}. Alkanes can be straight chains, branched chains or ring structures. Also referred to as paraffins.

Alkenes
Group of unsaturated hydrocarbons having the general formula C_nH_{2n} and characterised by being more chemically reactive. Also referred to as olefins.

Alkynes
Group of unsaturated hydrocarbons with a triple carbon-carbon bond having the general formula C_nH_{2n-2}. More reactive than alkenes.

Alloys, Polymeric
A blend of two or more polymers (e.g. a rubber and plastic) to improve a given property (e.g. impact strength).

Alternative Cover System
An engineered cover for waste containment applications that is designed on the basis of water balance principles (e.g. evapotranspiration, surface runoff, water storage, intralayer flow) and typically represents an alternative to regulated or prescribed covers. Examples include capillary barriers and monolithic covers.

Ambient
Surrounding.

Anaerobic
Able to live, grow or take place in the absence of gaseous or dissolved oxygen.

Anaerobic Biological Treatment
Biological treatment in the absence of gaseous or dissolved oxygen.

Analog
In chemistry, a structural derivative of a parent compound.

Anchor Trench
Trench at the top of a slope to secure upper edges of geosynthetics on slopes. Geosynthetics are laid in the trench and backfilled.

Anisotropic
Condition in which a material property (e.g., hydraulic conductivity) is not equal in all directions at a point in the material.

Anoxic
Total deprivation of oxygen.

Apparent Opening Size (AOS), O95
See **Opening Size**.

Aqueous Solubility
Extent to which a compound will dissolve in water.

Aquifer
Geological formation capable of transmitting significant quantities of water under practically relevant hydraulic gradients.

Aquitard
Geological formation that may contain ground water but is not capable of transmitting significant quantities of ground water under normal hydraulic gradients. In some situations, aquitards may function as confining beds.

Aromatic
Of or relating to organic compounds that resemble benzene in chemical behaviour. These compounds are unsaturated and are characterised by containing at least one 6-carbon benzene ring.

Attenuation
The process of immobilizing, retarding, or otherwise degrading chemical constituents that exist in the soil or ground water. (e.g., transfer to another medium such as by volatilisation or oxidation-reduction reactions).

Autoignition Temperature
Temperature at which a substance will spontaneously ignite. Autoignition temperature is an indicator of thermal stability for petroleum hydrocarbons.

Autotrophic
Designating or typical of organisms that derive carbon for the manufacture of cell mass from inorganic carbon (carbon dioxide).

Bacteria
Unicellular microorganisms that exist either as free-living organisms or as parasites and have a broad range of biochemical and sometimes pathogenic properties.

Barrier System
Combination of sealing elements for optimized contaminant retention.

Bioassay
Method for the determination of the toxicity of specific chemical contaminants.

Bioaugmentation
Introduction of selected cultured microorganisms into the subsurface environment for the purpose of enhancing bioremediation of organic contaminants. Nutrients can also be added.

Bioavailability
Availability of a compound for biodegradation. In exposure assessments it is the degree to which chemicals present in a soil matrix or a waste is available to be absorbed or metabolised by a human or ecological receptor.

Biocide
Substance capable of destroying living organisms.

Biodegradation
Transformation (through metabolic or enzymatic action) of organic substances to smaller molecules via oxidation and reduction mechanisms induced by the metabolic activity of microorganisms.

Biomass
Amount of living matter.

Bioremediation or Biotreatment
The process where a biological agent (e.g., bacteria, fungi, plants, enzymes) is used to reduce contaminant mass and toxicity in soil, groundwater, and air. See **Biodegradation**.

Bioreactor
Equipment in which biotreatment is carried out.

Bioslurping
Placing an extraction well at the interface between the liquid phase, typically an LNAPL, and the gas-phase of the vadose zone.

Biosparging
The process whereby air, nutrients, and other essential components for biodegradation are injected into the saturated zone.

Biotransformation
A general term representing any biologically catalyzed conversion of a metal (which can only change to a speciation of a metal) or organic chemical.

Bioventing
In situ process in which vapour extraction/air infiltration rates are adjusted to induce, activate, or optimise biodegradation reactions

Bottom Barrier System
In-ground in-situ horizontal barrier used to isolate/contain contamination.

Breakthrough
Contaminant arrival at an outflow or receptor.

Breakthrough Curve
The temporal distribution of concentrations emanating from the effluent end of a porous medium (e.g., column of soil or soil barrier).

Cap
Another name for a cover; see **Cover System**.

Capillary Barrier
A cover/cap comprised of a relatively fine layer overlying a relatively coarse layer to minimize infiltration of precipitation.

Capillary Break Layer
Layer of high permeability granular material used to stop upward capillary movement of soluble contaminants.

Capillary Fringe
Zone of a porous medium above the water table within which the porous medium is saturated with water under pressure less than atmospheric pressure.

Capillary Pressure (or Matric Suction)
The difference in pressure between a non-wetting and a wetting fluid in contact at an interface.

Capillary Rise
Process whereby water rises above the water table into the void spaces of a soil due to tension between the water and soil particles.

Carbon Adsorption
Process in which a soluble contaminant is removed from water by contact with carbon that has been processed to significantly increase the internal surface area (activated carbon)

Carcinogenic
Factors that cause cancer in humans and animals.

Cation Exchange Capacity
The capacity of a material with respect to cations that can undergo exchange with surrounding cations.

CCL
Compacted Clay Liner

Chain-of-Custody
Recording procedure to provide information on sample integrity, e.g. when transferred between the field and laboratory and within a laboratory.

Characteristic
Property or attribute of a material that is measured, compared or noted.

Chemical Stability
In construction materials; ability to resist degradation from chemicals, such as acids, bases, solvents, oils and oxidation agents; and chemical reactions, including those catalyzed by light.

Characteristic Size
A measurable size to define the characteristics, the composition or the behaviour of a material.

Chemical Treatment
Treatment of contaminated soil, sediment, water or other material in which the principal mechanism for degradation or conversion to a less environmentally harmful form is a chemical reaction or combination of reactions.

Chemicophysical Treatment
Process-based treatment relying on a combination of physical and chemical processes.

Chlorosulfonated Polyethylene (CSPE)
Family of polymers that is produced by polyethylene reacting with chlorine and sulfur dioxide. CSPEs contain 25 to 43% chlorine and 1.0 to 1.4% sulfur.

Clogging
Chemical precipitation and/or biofilm growth within a filter (fabric or soil filter) and/or the movement by mechanical action or hydraulic flow of soil particles into the voids of a filter and retention therein, thereby reducing the hydraulic conductivity of the filter.

Cometabolism
Simultaneous metabolism of two compounds in which the degradation of the second compound depends on the presence of the first compound, e. g., whilst degrading methane, some bacteria can degrade chlorinated solvents that they would otherwise be unable to attack or would not attack at a useful rate because they are in insufficient quantity.

Complexation (metal)
Reaction in which a metal ion and one or more ligands chemically bond. Complexes often prevent the precipitation of metals.

Compliance Point (or Point of Compliance)
The point along the contaminant migration pathway where the **target concentration** should not be exceeded. This may be the receptor, such as a surface abstraction, a groundwater supply well, the aquifer or water within the near surface soils.

Composite Barrier
Combination of geomembrane and clay barrier in intimate contact in base sealing and capping applications for optimized contaminant retention. Could also involve a gel.

Composting
Biological treatment, often in a treatment bed, where organic substances are submitted to aerobic transformation.

Concentration
The concentration of a substance such as a contaminant is the amount of that substance in a host material such as a soil. The substance and the host can be in the same or different phase (solid, liquid or gas).

Condensate
Liquid that separates from a vapour during condensation. Landfill gas condensate has to be expected in landfill gas collection systems because of water saturation of the gas.

Conservative
In the case of a contaminant, one that does not degrade and the movement of which is not retarded. Is nonreactive with the environment. {Actually, reactive and non reactive are not the same as conservative and unconservative, although these terms typically are used interchangeably}.

Constituent
Essential part or component of a system or group, e.g., an ingredient of a chemical mixture. Benzene is one constituent of gasoline.

Containment
Control of migration of gaseous, liquid or solid contaminated media from a site by use of measure(s) such as caps and horizontal and/or vertical in-situ barriers.

Contaminant
Substance that can cause harm to human health and the environment. Substances not normally present in the environment.

Contaminant Pathways
For Composite Barrier; Advection transport of inorganic and organic solutes only through geomembrane defects; diffusion of only organic solutes through geomembrane and soil liner.

Contaminant Transport
The movement of contaminants from one location to another.

Contaminated Land
Land that contains substances that are present in sufficient quantities or concentrations to be likely to cause harm directly or indirectly to humans, the environment or on occasions to other receptors. Some countries may have a specific legal definition of contaminated land.

Contaminating Lifespan
In landfill waste management, the period of time during which a landfill will produce contaminants at levels that could have unacceptable impact if they were discharged into the surrounding environment.

Contamination
Occurs when a contaminant is introduced into the ground either in liquid or solid form.

Convenience Sample
Sample chosen on the basis of accessibility, expediency, cost, efficiency, or other reason not directly concerned with sampling parameters.

Cover System
One or more layers of material, such as soils, suitable mineral wastes and geosynthetics, superimposed on the surface of a site and designed to control egress of contaminants from and infiltration of rainfall, etc., into a source of contamination.

Creep
In landfill engineering or similar applications, the elongation of a geosynthetic under constant load. Relevant design parameter for geosynthetics in soil reinforcing applications

Cross-contamination
The undesired movement of contamination from one location or item to another.

Cross-machine Direction
In geosynthetics; the axis within the plane of a fabric perpendicular to the predominant axis of the direction of production.

Cross-plane
The direction of a geosynthetic which is perpendicular to the plane of its manufactured direction. Referred to in hydraulic situations.

Dechlorination
Chemical process designed to remove chlorine from chlorinated organic compounds such as polychlorinated biphenyls (PCBs).

Decontamination
Reduction of contamination to an environmentally acceptable level.

Decontamination Methods
Technical methods for removal, transformation or reduction of contaminants in soil, water or air.

Degradation Potential
Degree to which a substance is likely to be reduced to a simpler form by bacterial activity.

Degraded Land
Land which, due to natural processes or human activity, is no longer able to properly sustain an economic function and/or its original natural, or near-natural, ecological function.

Denitrification
Bacterial reduction of nitrate to gaseous nitrogen under anaerobic conditions.

Derelict Site
Site so damaged by human activity as to be incapable of beneficial use without treatment. The damage may be aesthetic, physical, engineering, environmental or contamination.

Diffusion
Movement of solutes in response to a difference in concentration, from an area of high solute concentration to an area of low solute concentration.

Dilution
Reduction in contaminant concentration by increasing the volume of solvent (can be water) in the mixture.

Direct Thermal Desorption
Desorption induced by direct application of heat to the medium to be treated.

Dispersion
Irregular spreading of solutes during migration due to aquifer heterogeneities at pore-grain scale (mechanical dispersion) or field scale (macroscopic dispersion). Thus, a chemical spreads and dilutes in flowing groundwater or soil gas.

Displacement Barrier
In-situ barrier installed without excavation.

Dissolution
Dissolving of chemical substances from free product or solid waste to solutes in the ground water.

Distribution Coefficient (k_d)
The ratio of the mass of a chemical on the solid phase (e.g. soil particle) to that in the liquid phase (e.g., water) for the case of linear, reversible, and instantaneous partitioning (e.g., sorption).

DNAPL (Dense non-aqueous phase liquid)
A non-aqueous phase liquid (i.e. immiscible with water) that has a density greater than water.

Double Fusion Weld
Seaming of geomembranes with two parallel hot wedges with an air pressure channel for seam testing.

Downgradient
Direction of decreasing static head.

Dredging Waste
Materials that have been dredged or excavated from navigable waters or reservoirs.

Effects
Chemical changes in air, water or ground (abiotic effects) and / or chemical/biological changes in humans, animals, plants or microorganism (biological effects).

Effective Limit
The concentration at which an effect starts. Not all substances have an effective limit. Carcinogenic, teratogenic and mutagenic substances either do not have an effective limit and if they do, it can not be determined. With acute and chronic toxic substances the effective limit can be seen.

Electrokinetic Remediation
Application of electrokinetic processes such as electro-osmosis to remove contaminants from soil or other solids.

Eluate
A solution resulting from the mixing of soil and water in order to remove sorbed substances.

Encapsulation
To provide a barrier around all sides of a zone of contamination or a body of waste or to mix the contaminated ground/waste with a cementitious material.

Engineering-based Method
Civil engineering technique, such as excavation or containment, used to remove the contaminant source or soil material, or to modify pathways without necessarily removing, destroying or modifying the contaminant source.

Entrained
Particulates or vapour transported along with flowing gas or liquid.

Environmental Assessment
Process of drawing together, in a systematic way, an assessment of likely significant environmental effects.

Equilibrium
No net transfer between two phases.

Evapotranspiration
The release of water to the atmosphere via the combined processes of evaporation and plant transpiration.

Excavation
Removal of material from the ground for treatment or disposal.

Excavated Barrier
Barrier formed in an excavation, such as a slurry wall.

Exposure
Opportunity to receive a dose of a contaminant.

Exposure Assessment
Process of establishing whether, and how much, exposure will occur between a receptor and a contamination source.

Exposure Pathway
Path a contaminant takes from a source to a receptor. There may be more than one exposure pathway between a source and a receptor.

Ex-situ Treatment
Treatment applied to medium to be treated after extraction from the ground.

Extraction Method
Method by which contaminated soil is cleaned.

Extraction Well
Well employed to extract water, gas, free product or a combination of these from the ground.

Extrusion Welding
Seaming of geomembranes by welding a strand of extrudate over the edges of two overlapping geomembranes with the same polymer resin.

Fibre
Basic element of fabrics and other textile structures, characterized by having a length at least 100 times its diameter or width which can be spun into a yarn or otherwise made into a fabric.

FML (Flexible Membrane Liner)
See **Geomembrane**.

Field Capacity
The upper limit of a porous material's ability to absorb water.

Field Spike
Sample collected in the field and spiked with compounds of interest or related compounds to check on the potential for loss of analyte on sampling, transportation, storage, preparation and testing.

Filament Yarn
The yarn made from continuous filament fibers.

Filtration
In geotextiles or grain filters; the process of retaining soil in place while allowing water to pass from soil at low gradients. Removal of particle from a fluid stream.

Flux
Rate of movement of mass or heat or chemical species through a unit cross-sectional area per unit time in response to a driving force.

Free Product
Product (e.g., gasoline, diesel) that is present in its original state and at a high saturation (**Free Phase**).

Fungal Treatment
Biological treatment based on use of fungi such as dry rot fungus.

Gas Control System
System designed to control the migration and release of landfill gases and other gases from a site. Landfill gas condensate should be considered as typically being more aggressive than leachate.

Gas Protection Measure
Measure to protect buildings from landfill gases and other gases.

GCL (Geosynthetic Clay Liner)
See **Geosynthetic Clay Liner**.

Genotoxic
A substance which has a harmful effect on the genes of humans, animals and microorganisms.

Geocell
A three-dimensional structure filled with soil, thereby forming a mattress for increased stability when used with loose or compressible subsoils.

Geochemical Attenuation
Attenuation that results from geochemical interactions between natural geological material and chemical constituents in ground water.

Geocomposite
A manufactured material using geotextiles, geogrids, and/or geomembranes in laminated or composite form. May or may not include natural materials.

Geogrid
Open grid structure of orthogonal filaments and strands of polymeric material used primarily for soil reinforcement.

Geomembrane
Synthetic membrane liners used with any geotechnical engineering related material, properly welded and installed to form an advective barrier to control fluid (liquid or gas) migration in a man-made project, structure, or system. Often used in composite liner systems, e.g. in combination with CCLs or GCLs.

Geonet
A geosynthetic consisting of integrally connected parallel sets of ribs overlying similar sets at various angles for planar drainage of liquids or gases.

Geopipe
Any plastic pipe used with foundation, soil, rock, earth, or any other subsurface related material as an integral part of a man-made project, structure, or system.

Geosynthetics
A planar product manufactured from polymeric material used with soil, rock, earth, or other geotechnical engineering related material as an integral part of a man-made project, structure, or system.

Geosynthetic Clay Liner (GCL)
Factory-manufactured hydraulic barriers consisting of a layer of bentonite clay or other very low permeability material supported by geotextiles and/or geomembranes, and mechanically held together by needling, stitching, or chemical adhesives.

Geotextile
Any permeable textile used with foundation, soil, rock, earth, or any other geotechnical engineering-related material as an integral part of a human-made project, structure, or system.

Geotextile Separation
Placement of a flexible porous textile between dissimilar materials so that the integrity and functioning of both materials can remain intact or be improved.

GML
Geomembrane liner (same as FML).

Grab Test
In geosynthetic testing; a tension test in which only a part of the width of the specimen is gripped in the clamps.

Gradient Ratio
In geosynthetic testing; the ratio of the average hydraulic gradient across the fabric and the 25 mm of soil immediately next to the fabric to the average hydraulic gradient across the 50 mm of soil between 25 and 75 mm above the fabric, as measured in a constant head permeability test.

Hazard
Inherently dangerous quality of a substance, procedure or event.

Hazardous Site
Site which, by reason of the substances or agents present, is judged to be hazardous to human health or safety, or to the environment.

Heat Bonded
In geosynthetics; thermally bonded by melting the fibers to form weld points.

Hot Wedge
Common method of heat seaming of thermoplastic geomembranes by a fusing process wherein heat is delivered by a hot wedge passing between the opposing surfaces to be bonded.

HDPE High Density Polyethylene
Synthetic material with highest overall chemical stability and environmental durability. Usually 98 % resin, 2 % carbon black and stabilizer. Stress crack resistance has to be considered for resin type selection.

HDPE Membrane
High Density Polyethylene geomembrane manufactured in thin sheets from synthetic polymers. Contaminant barrier in landfill barrier systems. Thickness requirements in relation to installation conditions and/or limiting diffusion of contaminants. HDPE also used for pipes, shafts, gas collecting systems, leachate collecting systems.

Hydraulic Conductivity
A measure of the capability of a material to transmit water or gases.

Hydraulic Gradient
The change in total head with a change in distance in a given direction. The direction is that which yields a maximum rate of decrease in head.

Hydraulic Permeability, Permittivity
See **Permeability** and **Permittivity**.

Hydraulic Transmissivity
In geosynthetics; the volumetric flow rate of water per unit width of specimen per unit gradient in a direction parallel to the plane of the specimen.

Hydrocarbon
Chemical compounds composed of carbon and hydrogen.

Hydrophilic
Having an affinity for water, or capable of dissolving in water. Soluble or miscible in water.

Hydrophobic
Tending not to combine with water or incapable of dissolving in water. Insoluble or immiscible in water.

Immiscible
Not soluble in a solvent, usually assumed to be water.

Immobilise
Suppression of mobility of pollution.

Index Test
In geosynthetics, laboratory testing of material properties to compare different products or for quality control testing.

Infiltration
Downward movement of water through a soil in response to gravity and capillary suction.

Injection Well
Well used to inject liquid, vapour or gas under pressure into the ground.

Inlet Well
Well through which a liquid or gas is allowed to enter the ground under natural pressure.

Inclusion Methods
Interruption of the operational path between the pollution source and objects.

Indigenous
Describing organisms that have developed naturally in a given area and not as a consequence of artificial introduction

Industrial Waste
Waste generated from industries.

Influence
To effect receptors through pollution.

Injected Barrier
Barrier formed by injecting material such as cementitious grouts under pressure into the ground to seal natural migration pathways.

Inorganic Solutes
Contaminants that are completely retained by an advective barrier such as a HDPE geomembrane. In a composite barrier inorganic contaminant transport only through geomembrane defects caused by installation damage or seam failures.

In-plane
In geosynthetics; the direction of a geosynthetic that is parallel to its longitudinal, manufactured, or machine direction. Referred to in shear strength and hydraulic situations.

In-situ Remediation
Remediation carried out on contaminants that remain in the ground.

Installation Tests
Field tests which characterize performance of construction materials under installation conditions.

Interface Shear Strength
Important parameter for multi-layered sealing systems. Shear strength/friction must be investigated between all geosynthetic/soil or geosynthetic/geosynthetic interfaces for proper design of landfill slopes.

Intimate Contact
Direct and uniform contact between geomembrane and clay barrier in a composite liner.

Intrinsic Permeability
Measure of the relative ease with which a permeable medium can transmit a liquid or gas. Intrinsic permeability is a property only of the medium and is independent of the properties of the liquid or gas.

Intrinsic Bioremediation
Reduction in contaminant concentrations in soil, groundwater or other media, caused by natural biological processes in the absence of human intervention.

Ion Exchange
Competitive exchange of ions between two different phases. Sorption can be a sub-set of ion exchange.

Ionisation
Ability of substances to donate protons when in aqueous solutions. Acids loosing a proton become anions and their solubility in water increases significantly.

Isotropic
Condition in which the material property (e.g. hydraulic conductivity) is equal in all directions at a point in the material.

Judgemental Sampling
Sampling in which locations are chosen according to the judgement of an expert.

Landfarming
Biotreatment applied directly in situ to surface contaminated soils using largely agricultural techniques.

Landfill
Permanent emplacement of wastes in or on the ground.

Leachate
Fluid resulting from the leaching of contaminants from waste, contaminated ground in the field or samples of these in the laboratory.

Leachate Collection Layer
Drainage system above primary liner at landfill base to collect and drain the leachate. Components are drainage composites, filter layer, drainage pipes, service and monitoring shafts.

Leak Detection Layer
Drainage system between primary and secondary liners at landfill base e.g. for hazardous waste landfills.

Limit
Uppermost value permitted for a substance.

LNAPL (Light Non Aqueous Phase Liquid)
A non-aqueous phase liquid that has a density lower than water.

Liner
A layer of emplaced materials beneath a surface impoundment or landfill which serves to restrict the escape of waste or its constituents from the impoundment or landfill.

Lower Explosive Limit (LEL)
Concentration of a gas in air below which it is insufficient to support an explosion. LELs for most organics are generally in the range 1 to 5 % by volume.

Machine Direction
In geosynthetics, the direction in the plane of the product, parallel to the direction of manufacture.

Macroencapsulation
Mechanism whereby contaminants are held in discontinuous pores within a stabilising material.

Mass Per Unit Area
In geosynthetics, the proper term to represent and compare to amount of material per unit area (units are oz/yd^2 or g/m^2) of a geosynthetic.

Medium
Air, water, soil or biota (plant or animal life)

Methanogenic
Referring to the formation of methane by anaerobic bacteria during the process of anaerobic fermentation.

Microencapsulation
Mechanism whereby contaminants are entrapped within the structure of a solidified matrix.

Microorganisms
Microscopic organisms including bacteria, protozoans, yeast, fungi, mould viruses and algae.

Mineralization
Complete biodegradation, refers to conversion of the target compound to carbon dioxide and water under aerobic conditions, or methane (or ethane or ethene) and other simple inorganic species (such as SO_4, NO_3, etc) under anaerobic conditions.

Mobile Treatment System
Readily movable, eg lorry or barge mounted, process-based treatment method.

Mobility Enhancement
Processes that increase the mobility of chemical substances.

Mobilisation
Conversion of a matter from an immobilised form into a shifting or available form (eg. solution, dispersion, and volatilisation).

Molecular Diffusion
Movement of molecules due to a difference in their concentration, from higher concentration to lower concentration.

Monitoring Plan
Programme of inspection and/or testing.

Monoaromatic
Aromatic hydrocarbons containing a single benzene ring.

Monofilament
A single filament of a fiber (normally synthetic).

Monolithic Cover
A type of alternative cover for waste disposal applications consisting of a slab of soil with vegetation on top that is designed to store and release water via soil-water storage and evapotranspiration, respectively.

Mullen Burst
Hydraulic bursting strength of textiles.

Multi-Barrier System
A number of barriers designed to work together such as geological barrier, base sealing system and capping system of a landfill.

Multifilament
A yarn consisting of many continuous filaments or strands.

Municipal Waste
Heterogeneous mixtures of wastes that are primarily of residential and commercial origin.

Mutagenic
Factors that can cause changes in the genes of humans, animals and other life forms.

Natural Attenuation
Natural processes, including chemical, physical and biological processes, that result in reduction in contaminant concentrations in the soil or groundwater. In the case of ground water, natural attenuation may occur at the source and during migration of contaminants.

Needle Punched
In geotextiles, mechanically bonding of staple or filament fibers with barbed needles to form a compact fabric.

Non-aqueous Phase Liquid (NAPL)
Liquids that are immiscible with water.

Non-Destructive Seam Testing
Non-destructive testing of geomembrane seams by e.g. pressurized double fusion weld, vacuum chambers or electric sparking.

Nonwoven fabric
A textile structure produced by bonding or interlocking of fibers, or both, accomplished by mechanical, thermal, or chemical means.

Nutrients
Major elements (e.g. carbon, nitrogen and phosphorus) and trace elements (e.g. sulphur, potassium, calcium and magnesium) that are essential for the growth of organisms.

Octanol/Water Partition Coefficient (k_{ow})
Coefficient representing the ratio of the solubility of a compound in octanol (a non-polar solvent) to its solubility in water (a polar solvent). The higher the k_{ow} the more non-polar the compound. Log k_{ow} values are generally inversely related to aqueous solubility and directly proportional to molecular weight. Thus, empirical constant that describes how a chemical distributes itself between two media such as organic soils and water.

Off-site Treatment
Treatment applied away from the site to be remediated. The treatment is usually performed in a stationary plant used specifically for this purpose.

Olefins
See **Alkenes**.

On-site Treatment
Treatment applied on the site being remediated. The treatment is usually carried out in a mobile or semi-mobile plant, which is transferred from site to site.

Opening Size
For a geotextile; a property that indicates the diameter of the approximate largest soil particle that would effectively pass through the geotextile. Testing after ASTM with a dry sieving method to estimate AOS (Apparent Opening Size), O95; Testing after CEN / ISO with a wet sieving test to estimate O 90,W.

Organic Carbon Partition Coefficient (k_{oc})
A measure of the proportion of adsorption of organic chemicals by a soil that is due to the organic carbon content of a soil. Most of the adsorption in a soil of organic compounds in water is attributed to the organic carbon content.

Organic Solutes
Organic compounds in the aqueous phase; the water soluble component of an organic compound

Oxidation and Reduction (Redox)
Chemical reaction consisting of an oxidation reaction in which a substance loses or donates electrons and a reduction reaction in which a substance gains or accepts electrons. Redox reactions are always coupled because free electrons cannot exist in solution and electrons must be conserved.

Paraffins
See **Alkanes**.

Parent Population
Totality of items under consideration.

Partition Coefficient
See **Octanol/Water Partition Coefficient (k_{ow})**.

Partitioning
The process by which a contaminant, released originally in one phase (e.g. adsorbed onto soil grains), becomes distributed between other phases (i.e. vapour and dissolved phases)

Pathway
The route whereby a contaminant migrates from a source to a receptor.

Passive Protective Measure
Protective measure that provides protection solely through its presence.

Permeability
Quantitative description of the relative ease with which strata in the ground will transmit a liquid or gas. Often used as a synonym for hydraulic conductivity or coefficient of permeability.

Permittivity
In geosynthetics, the volumetric flow rate of water per unit cross sectional area per unit head under laminar flow conditions in the normal direction through a geotextile or geocomposite. For barrier materials, generally the hydraulic conductivity divided by the barrier thickness.

Plasticizer
A material, frequently solvent-like, incorporated in a plastic (e.g. PVC) or a rubber to increase its ease of workability, its flexibility, or distensibility.

Phase 1 Investigation (Preliminary Investigation)
Desk study and site reconnaissance.

Phase 2 Investigation (Exploratory Investigation)
Collection of samples for analysis to test the hypothesis concerning soil quality from Phase 1 investigation and to provide information to enable the design of the main investigation (Phase 3).

Phase 3 Investigation
Evaluation of the extent and degree of contamination and identification and assessment of risks.

Phase 4 Investigation (Supplementary Investigation)
Further collection of information for the selection and design of remedial work (if necessary).

Phreatic Zone
The zone located below the water table where the fluid pressure is positive

Physical Treatment
Process-based treatment such as vapour extraction based primarily on physical processes

Phytoremediation
Use of plants in remediation. These plants can promote degradation of toxic substances or accumulate them in their tissues and/or pump water and contaminants from the subsurface.

Pollutant
Substances which, due to their properties, amount or concentration, cause impact on (harm to) water and soil function or use. (c.f. contaminant).

Pollution
The effect of mans activity on the environment in terms of introducing **pollutants**.

Polyaromatic Hydrocarbons (PAH)
Aromatic hydrocarbons containing more than one fused benzene ring. More correctly **Polynuclear Aromatic Hydrocarbons**.

Polyester Fibre
Generic name for a manufactured fibre in which the fibre-forming substance is any long-chain synthetic polymer composed of an ester of a dihydric alcohol and terepthalic acid.

Polyethylene
A polyolefin formed by bulk polymerization (for low density) or solution polymerization (for high density) where the ethylene monomer is placed in a reactor under high pressure and temperature.

Polymer
A macromolecular material formed by the chemical combination of monomers having either the same or different chemical composition. Plastics, rubbers, and textile fibers are all high-molecular-weight polymers.

Polynuclear Aromatic Hyrocarbons
Synonymous with and correct name for **Polyaromatic Hydrocarbons**.

Polyolefin
A family of polymeric materials that includes polypropylene and polyethylene, the former being very common in geotextiles, the latter in geomembranes.

Polyvinyl Chloride (PVC)
A synthetic thermoplastic polymer prepared from vinyl chloride. The majority of PVCs contain 50 % vinyl chloride and 50 % plasticizer, filler and stabilizer.

Post Treatment Measure
Any activity such as drying applied to the product of a process based treatment to prepare it for disposal or re-use.

Prescribed Cover System
Cover systems used at waste disposal facilities that are designed on the basis of prescribed guidelines, e.g., as set forth in environmental regulations.

Pressure Gradient
The gradient of a pressure differential in a given medium (e.g. water or air), i.e. pressure differential per unit length, which tends to induce movement from areas of high pressure to areas of lower pressure.

Pretreatment Measure
Any activity, such as drying, grinding, grading applied to a material before it enters the main treatment process or train.

Primary Treatment
Application of a process, such as filtration, or combination of processes, forming the first stage of a treatment train.

Process-based Treatment
Application of physical, chemical or biological processes either to remove or destroy contaminants or to reduce their availability to the environment.

Protection Layer
Thick non-woven geotextile layer above or below a geomembrane to avoid damage or puncturing of geomembrane placed on or backfilled with soils with coarse components like gravel or stones. Main parameter: mass per unit area.

Protective Measure
Measure designed to protect a specified receptor against an identified hazard or risk. Control of contamination rather than removal.

Pump-and-treat System
System in which water is extracted for treatment above ground.

Quality Assurance (QA)
A planned system of activities whose purpose is to provide a continuing evaluation of the quality control program, initiating corrective action where necessary. It is applicable to both the manufactured product and its field installation.

Quality Control (QC)
Actions that provide a means of controlling and measuring the characteristics of (both) the manufactured and the field installed product.

Quality Control Sample
Sample taken for the purpose of quality assurance of field sampling.

Radius of Influence
Maximum distance away from an injection or extraction source that is significantly affected by a change in pressure.

Random sample
Sample taken without any systematic procedure or taken at a time or place selected on a random basis.

Reagent
Substance or solution used in a chemical reaction.

Receptor
Person, animal, ecosystem, structure, utility, surface water, groundwater or water supply well that may be adversely affected by a release

Redox Potential
Abbreviated term for oxidation-reduction potential.

Refractory Index
Measure of the ability of a substance to be biodegraded by bacterial activity. The lower the refractory index, the greater the biodegradability.

Regulated Cover System
See **Prescribed Cover System**.

Reinforcement
In geosynthetics; the function of a geotextile or geogrid to increase the strength of a soil.

Rehabilitation, Restoration, Reclamation
The return of contaminated, damaged, degraded or derelict land to beneficial use.

Remedial Target Concentration
The contaminant concentration above which remediation is required.

Remediation
Process of dealing with contaminated soil or ground to eliminate or control risks to human health or the environment.

Remedial Investigation
Investigation to collect all information necessary to design and execute remediation strategy.

Remediation Strategy
Combination of remediation methods and associated works that will meet specified contamination-related objectives and overcome site-specific constraints.

Remediation Target
The performance to be achieved by remediation. Usually expressed in terms of residual concentrations.

Replicate (Duplicate) Sample
One of the two or more samples or subsamples obtained separately at the same time by the same sampling procedure or sub-sampling procedure.

Representative Sample
Sample resulting from a sampling plan that can be expected to reflect adequately the properties of the whole of the soil under consideration.

Residual Contamination
Amount or concentration of contaminants remaining in specific media following remediation.

Retardation
An attenuation process resulting in a reduction in the rate of migration of a contaminant..

Retention Time
The period of time that a contaminant remains in a system.

Re-use
Useful and harmless utilisation of soil or other materials.

Risk
Combination of the probability of occurrence of harm and the severity of that harm.

Risk Analysis
Use of available information to identify hazard and to estimate the risk.

Risk Assessment
The process of risk analysis and risk characterisation.

Risk Characterisation
Evaluation and conclusion based on hazard identification and exposure and effect assessment.

Sample
Portion of material selected from a larger quantity of material.

Sample Preservation
Any procedure used to stabilise a sample in such a way that the properties under examination are maintained stable from the time of collection to the time of preparation for analysis.

Sample Size
The quantity of material constituting a sample.

Sampling Error
The part of the total error which is due to using only a fraction of the population and extrapolating to the whole, as distinct from analytical error.

Sampling Pattern
System of predetermined sampling points.

Sampling Procedure
Operational requirements and/or instructions relating to the use of a particular sampling plan.

Sampling Strategy
Strategy to ensure that samples are reliable and representative for the respective pollution and that the best locations are chosen for taking samples.

Saturated Zone
Zone in which all the voids in the rock or soil are filled with water at greater than atmospheric pressure. The water table is the top of the saturated zone in an unconfined aquifer. See **Phreatic Zone**.

Sealing Compounds
In mineral barrier; components to improve the properties of e.g. clay barriers like colloids (such as bentonite), hydraulic binding agents, mineral fillers or additives.

Secondary Treatment
Application of a process or combination of processes forming the second stage (usually the main stage) of a treatment train.

Selective Sample
Sample that is deliberately chosen by using a sampling plan that selects material with only certain characteristics or that selects a particular location .

Sentinel Well
Groundwater monitoring well situated between a sensitive receptor downgradient and the source of a contaminant plume upgradient. The sentinel well should be far enough upgradient from the receptor so that once contamination is detected in the well there is enough time to take preventative measures.

Separation
The function of geosynthetics as a partition between two adjacent materials (usually dissimilar) to prevent mixing of the two materials.

Service Life
The period of time that an engineered component continues to perform its design function as anticipated in the design (eg. for a leachate collection system this may be the period of time the collection system maintains the head on an underlying liner below some specified design value).

Slurry Bioreactor
Equipment in which bio-treatment is applied to a slurry of contaminated soil or sediment with water, biological agents, nutrients etc.

Soil Rehabilitation
Measures taken to improve the ability of a degraded soil to perform specified functions

Soil Washing
Ex situ water-based system usually for the separation of contaminants and fine fraction from the coarser material in a contaminated soil.

Soil-Water Partition Coefficient (k_p)
Measure of the tendency of a chemical to be adsorbed by soil or sediment.

Soil Flushing
In situ process in which contaminants are removed from soil by the movement of water through the ground, sometimes with surfactants or other chemicals added to the water.

Soil Vapour Extraction
In situ process involving extraction of volatile vapours from the ground under suction. An air stream passes through the soil thereby transferring the contaminants from the soil or soil/water matrix to the air stream.

Solidification
Addition of reagents to contaminated soil to solidify it and prevent access by external mobilising agents, such as water, to contaminants contained in the solid product.

Solubility
Mass of a compound that will dissolve in a unit volume of solution.

Solutes
Contaminants dissolved in water or other liquids.

Solvent Extraction
Use of non-aqueous liquids, such as organic solvents, to separate contaminants from soil.

Sorption
The process by which a component (the sorbate or contaminant) moves from one phase to another across some boundary. A general term used to encompass the processes of absorption, adsorption and desorption.

Source
Soil or soil component from which a chemical or hazardous agent is released for potential exposure of a receptor.

Sparge
Injection of air below the water table to strip dissolved volatile organic compounds and / or oxygenate the groundwater to facilitate aerobic biodegradation of organic compounds. See **Air Sparging**.

Specification
A precise statement of a set of requirements to be satisfied by a material, product, system or service.

Specimen
Specifically selected unit/portion of a material.

Spot Sample
Sample of soil from a specified place and considered representative only of its own immediate or local environment.

Spun-bonded fabrics
Fabric formed by continuous filaments which have been spun (extruded), drawn, laid into a web and bonded (chemical, mechanical, or thermal bonding) together in one continuous process.

Stabilisation
Addition of chemicals or other substances to a waste or a contaminated soil to produce more chemically stable constituents and/or improved handling and physical characteristics. Stabilisation is a preferred term to **fixation.**

Staple Fibres
Fibre of short lengths; frequently used to make needle-punched non-woven fabrics.

Static Piles
Term used in bioremediation to indicate piles of waste or contaminated land that, unlike **windrows**, are not turned. The piles are normally built on top of a grid of perforated pipes so that air can be forced or drawn through the piles using a vacuum or forced air system.

Stress Relaxation
Reduction of stress in a geosynthetic while under constant deformation. See **Creep**.

Sub-population
Defined part of a population.

Sum Parameters
Sum parameters are collective parameters such as TOC, COD and BOD.

Supercritical Fluids
Materials at elevated temperatures and pressure that have properties between those of a gas and a liquid.

Survivability
The ability of a geosynthetic to be placed and to perform its intended function without undergoing degradation.

System Boundary
An imaginary boundary around a system.

Systematic Pattern
Sampling pattern indicating sampling locations based on the results of geometrical or statistical procedures.

Systematic Sampling
Sampling to some systematic pattern.

Target Concentration
The contaminant concentration which should not be exceeded at the **Compliance Point.**

Targeted Sample
See **Selective Sample**.

Tensile strength
In geosynthetics; the maximum resistance to deformation developed for a specific material when subjected to tension by an external force. Specimen dimensions may be defined in test procedures.

Tensile Testing
In geosynthetics; a tension test in which the total width of specimen is gripped in the clamps. Testing strength and strain of products.

Teratogen
Teratogens are factors that can cause birth defects in humans or animals. Known teratogenic factors are chemicals, physical influences and viruses.

Tertiary Treatment
Application of a process or combination of processes forming the third (and usually final) stage of a treatment train.

Textured Surface
High-friction surface of geomembranes by special production procedures.

Thermal Desorber
Treatment unit for carrying out thermal desorption.

Thermal Desorption
Thermal treatment involving volatilisation of contaminants from soil.

Thermal Destruction
Treatment in which a contaminated soil is raised to a temperature at which thermal destruction of contaminants occurs.

Thermal Fusion
Seaming of geomembranes by melting and pressing the geomembrane surfaces together with the main seaming parameters temperature, pressure and seaming rate. Single and double fusion weld, double fusion with air canal for seam testing.

Thermal Treatment
Application of heat in the presence of oxygen to destroy, remove or immobilise contaminants.

Total Petroleum Hydrocarbons (TPH)
Measure of the concentration or mass of petroleum hydrocarbon constituents present in a given amount of air, soil or water. The term total is a misnomer in that few, if any, of the procedures for quantifying hydrocarbons are capable of measuring all fractions of petroleum hydrocarbons present in a sample. Volatile hydrocarbons are usually lost in the process and not quantified. Some non-petroleum hydrocarbons may be included in the analysis.

Toxic
A substance that is poisonous for humans, animals and other living organisms.

Traditional Cover System
A loose term that generally refers to a prescribed or regulated cover system for waste disposal applications that includes one or more barriers with low saturated hydraulic conductivity (e.g., compacted clay, geosynthetic clay liner, and/or geomembrane).

Trapezoid Tear Test
Test method used to measure the tearing strength of geotextiles.

Transmissivity
See **Hydraulic Transmissivity**.

Travel (Transit) Time
Time required for a contaminant to travel from the source to a specified point down gradient.

Treatment Bed
Above ground bed of soil designed to enhance biodegradation processes.

Treatment Train
Sequence of treatment processes.

Ultraviolet degradation
For polymers, the breakdown of polymeric structure when exposed to natural sun light.

Unsaturated Zone
Zone between the ground surface and the capillary fringe within which the moisture content is less than saturation and pressure is less than atmospheric. The zone excludes the capillary fringe.

Upgradient
Direction of increasing piezometric head.

Vadose Zone
The zone between the ground surface and the water table within which the moisture content is less than saturation (except in the capillary fringe) and where the fluid pressure is negative. The zone is divided into a saturated portion (capillary fringe), due to capillary rise, and an unsaturated or partially saturated zone. Thus, the vadose zone includes the capillary fringe.

Validation Plan
Programme of inspection and/or testing for the purpose of assessing whether the completed works, or a component of the works, complies with predetermined quality of performance criteria.

Vapour Density
Amount of mass of a vapour per unit volume of the vapour.

Vapour-liquid Partition Coefficient
Ratio of the concentration of a compound in the vapour to the concentration in the liquid at equilibrium. The coefficient is a function of temperature, vapour pressure, atmospheric pressure, the composition of the liquid and vapour, and the specific compound.

Vapour Pressure
Force per unit area exerted by a vapour in an equilibrium state with its pure solid, liquid or solution at a given temperature. Vapour pressure is a measure of a substance's propensity to evaporate. Vapour pressure increases exponentially with increase in temperature.

Vertical Barrier
In-situ vertical structure designed to contain contamination.

Vitrification
Treatment in which temperature is sufficient to melt the bulk of the treated soil to produce a glass or partially glassy material.

VOC
Volatile Organic Compound. Organic compound with a high vapour pressure that allows it to evaporate quickly. In general, physical-chemical properties include high vapour pressure, high Henry's constant, low organic carbon partition coefficient (k_{oc}) and high octanol water partition coefficient (k_{ow}).

Volatilisation
Transfer of a chemical substance from a liquid phase to a gaseous phase.

Waste
Anything which the holder discards, intends to discard, or is required to discard. Material which has passed from the chain of utility.

Water Table
A surface on which the groundwater pressure (pressure head) is exactly atmospheric.

1.2 MULTIDISCIPLINARY INTERACTIONS

1.2.1 General

In many projects there is a need for a number of professionals of different disciplines to work together to solve a problem, produce a design or carry out a study. One of the challenges for all of these professionals is to apply creative, innovative and cost effective solutions. This requires the various disciplines to work together in a constructive manner with each individual having a satisfactory appreciation of the principles that underlie disciplines other than their own.

Environmental Geotechnics continues to become increasingly global. There is therefore a need for specialists in Environmental Geotechnics to not only interact closely with professionals of other disciplines but also of different nationalities, cultures, language and possibly engineering approach. The implementation of international standards will assist in this process including standards that are intended to harmonise.

Multidisciplinary interaction can be considered in two ways. First the input of other disciplines into Environmental Geotechnics or vice versa (eg chemists, microbiologists, hydrogeologists, ecologists, botanists, etc) and second the ability of certain specialists in Environmental Geotechnics to absorb, assimilate and apply skills normally attributed to other disciplines.

1.2.2 Terminology

When disciplines are interacting there is a fundamental need to ensure that there is either consistency in the use of terminology or alternatively there is an understanding by each discipline of how their terminology differs from the terminology used by other disciplines. An example of this is the way in which the water content of a material is calculated. Some disciplines will calculate this as weight of water in relation to the weight of solids and others will calculate it as the weight of water in relation to the total weight, but both may use the term water content or moisture content.

Section 1.1 of this chapter presents a listing of terms used in Environmental Geotechnics and their generally accepted meaning. This should assist in overcoming the problem referred to above.

1.2.3 Input of Other Disciplines into Environmental Geotechnics

Before discussing the nature and process of other disciplines providing input to projects in Environmental Geotechnics it is first appropriate to consider the elements normally under consideration as follows:

- Soil system (solid, liquid and gaseous phases)
- Groundwater system (perched water and aquifers)
- Surface water system (water courses, lakes, lagoons, drainage, etc)
- Biological system (micro-organisms)
- Contamination (inorganic and organic)
- Other (land use, climate, temperature, regulatory controls, etc)

Many of the processes dealt with in Environmental Geotechnics are complex. An example of this is contaminant transport in a soil medium. Many of the chemicals that have seeped into the ground resulted from decades of research and have been manufactured by highly complex processes. It would be naive to assume that these complex processes cease entirely once the chemicals enter the ground. Some can continue with the potential for the chemicals to react with the ground and with each other. Daughter chemicals can be produced with different properties, such as mobility, to those of the parent.

In order to be able to carry out much of their work, specialists in Environmental Geotechnics need the knowledge from other disciples either by interacting with them on the project or by acquiring the knowledge themselves as a result of learning from others. Yong (1997) produced the diagram in Figure 1 to illustrate the interaction of various disciplines in the evaluation of the persistence and fate of contaminants. He classified the boxes as;

- Bottom left (Soil Science)
- Bottom centre (Chemistry)
- Bottom right (Earth Science)

For consideration of a problem of soil contamination, Yong points out that in consideration of the soil, the Soil Science box would be consulted. In considering the contaminants the chemistry box would be consulted and in respect of the interactions between the soil and the contaminants, all three boxes would be consulted.

An example of the above would be the determination of partitioning coefficients for use in contaminant transport modelling which could involve many of these disciplines, such as soil chemistry, geochemistry, organic chemistry, soil microbiology, etc.

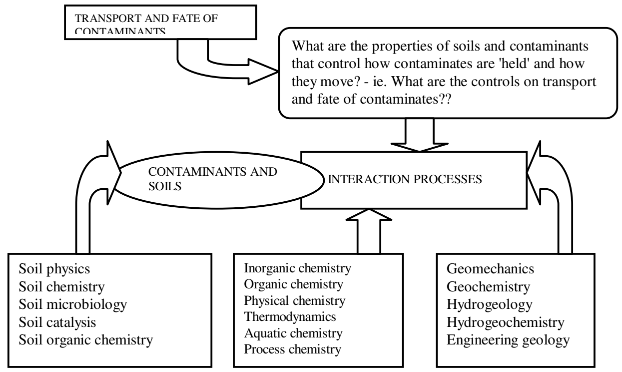


Figure 1. Illustration of interaction of various disciplines in evaluation of fate of contaminants (Yong, 1997)

1.2.4 Interaction with Regulators

One of the greatest interactions outside the discipline of Environmental Geotechnics is likely to be the interaction with regulators. These may be of many different disciplines such as planners, environmental scientists, ecologists, etc as well as the earth sciences disciplines. In addition to being of a different discipline, these regulators may also have a different approach. For agreement to be reached in respect of the Environmental Geotechnics aspects of a project, these differences must be resolved such that constructive interaction can take place.

1.2.5 Interaction with Non-technical People

In addition to specialists in various disciplines being able to interact with each other they must be able to relate to non-technical individuals (in respect of engineering and science) such as the client (who could be a business person or a property developer, etc), the legal profession, planning authorities, health and safety authorities, project managers, quantity surveyors, contractors and the general public. This is likely to mean being able to present concepts, design principles and recommendations in a manner that is easily understood by these non-technical people.

The above is particularly significant where acceptance by the public is an important issue in the overall acceptability of the project. In such circumstances it is all too easy for the public to base their opinions on misconceptions and erroneous interpretations. They must be won over and this can only be achieved by the specialist having the skill to present information in a way which the public can easily understand.

Less than fully effective communication can lead to inappropriate investment decisions, refusal of planning applications, contractor's claims, disagreements on payment, unacceptable safety risks and, if severe, failure of engineering construction.

1.2.6 Education

As indicated above, Environmental Geotechnics requires the interaction of knowledge which comes from a range of disciplines. This knowledge evolves with time and therefore it is necessary to continually 'keep up to date'. In any case, an individual can always improve their performance by learning more. Therefore, specialists in Environmental Geotechnics need to continually improve their understanding of other disciplines. Continuing education in professional practice is therefore essential in order to keep abreast of new developments or just to improve overall understanding.

Education has an important role to play in improving multidisciplinary interaction. There was a time when each discipline tended to be taught in isolation from other disciplines. However, with the increase in understanding, more and more courses are acknowledging the need for interaction with disciplines nominally outside the core subject being taught. Invited lecturers from other departments or other research institutes together with talks by practitioners from industry and public bodies help to further this process.

1.2.7 Concluding Remarks

If Environmental Geotechnics is to continue to progress we must effectively interact with individuals and knowledge from other disciplines.

1.3 CLASSIFICATION AND CHARACTERISATION

1.3.1 General

This section gives an indication of some of the classification and characterisation systems that can be used in Environmental Geotechnics. They are by no means the only systems that are available. In general, classification and characterisation systems assist in communication, assessment and decision making.

1.3.2 Soil and Soil Properties

Classification systems for soil and soil properties are to be found in many other publications on soil mechanics, geotechnical engineering and engineering geology and are not reproduced here. For consistency it is appropriate for these same classification systems to be used in environmental geotechnics. Thereby, the same or similar behaviour can be assigned to particular substances, techniques and/or properties, etc. For example, waste materials do not necessarily behave in the same way as natural soils, even those which are derived from natural soils, but the basic principals applied to natural soils are a good starting point in the understanding of the behaviour of wastes.

Further comment on soil classification in respect of contaminated land is given in Sub-section 1.3.4.2 below.

1.3.3 Chemicals and Chemical Properties

Perhaps one of the simplest ways of classifying chemicals, soils or waters is by pH value (see Table 1) which influences chemical reactions (e.g. precipitation) and the hydraulic conductivity of the soil. Chemical solutions that are highly reactive with soils include strong acids (pH $\leq$ 2) and strong bases (pH $\geq$ 12)

Alternatively, chemicals can be classified according to their chemical formation. A simplified example is given in Figure 2 where the chemicals are classified as inorganic or organic with the organic further being classified as chlorinated or non- chlorinated. This figure also shows a division between aromatic and aliphatic compounds. Aromatics are ring or multiple ring chemical structures containing alternating single and double bonds. Aliphatics are compounds that contain open carbon chains or rings. Aliphatics include alkanes, alkenes, alkynes and their derivatives. In the figure pesticides are divided into chlorinated pesticides and organophosphorus pesticides.

There are of course a number of other ways in which chemicals can be classified in respect of their properties (e.g. mobility, reactivity, stability, etc.) or toxicity (e.g. carcinogeneity, ability to bio-accumulate, etc.) and so on. Such a classification based on properties can be far more relevant to determining how the chemicals will behave in the ground and the appropriate forms of remediation for removing them from the ground.

Shackelford (1994) provides a classification for fluids within the pore space of soils as shown in Figure 3. Shackelford (1999) comments on this classification in respect of pore fluids being categorised as aqueous liquids or solutions containing contaminants that are miscible in water (also known as hydrophilic or 'water-loving'), or non-aqueous liquids consisting of organic compounds immiscible in water (also known as hydrophobic or 'water-hating').

Aqueous liquids can contain inorganic chemicals (acids, bases, salts) and/or hydrophilic organic compounds. Hydrophilic organic compounds are distinguished from hydrophobic organic compounds based on the concept of 'like dissolves like', i.e. polar organic compounds usually will readily dissolve in water a polar molecule, whereas non-polar organic compounds are repelled by water. A hydrophobic compound is also further separated into either a LNAPL (light non-aqueous phase liquid) or DNAPL (dense non-aqueous phase liquid) depending on whether the density of the compound is lower or greater than water respectively.

Table 1: Approximate Classification by pH value

pH Value	**Characteristic**
1 to 4	Acidic
4 to 9	Neutral
10 to 14	Alkaline

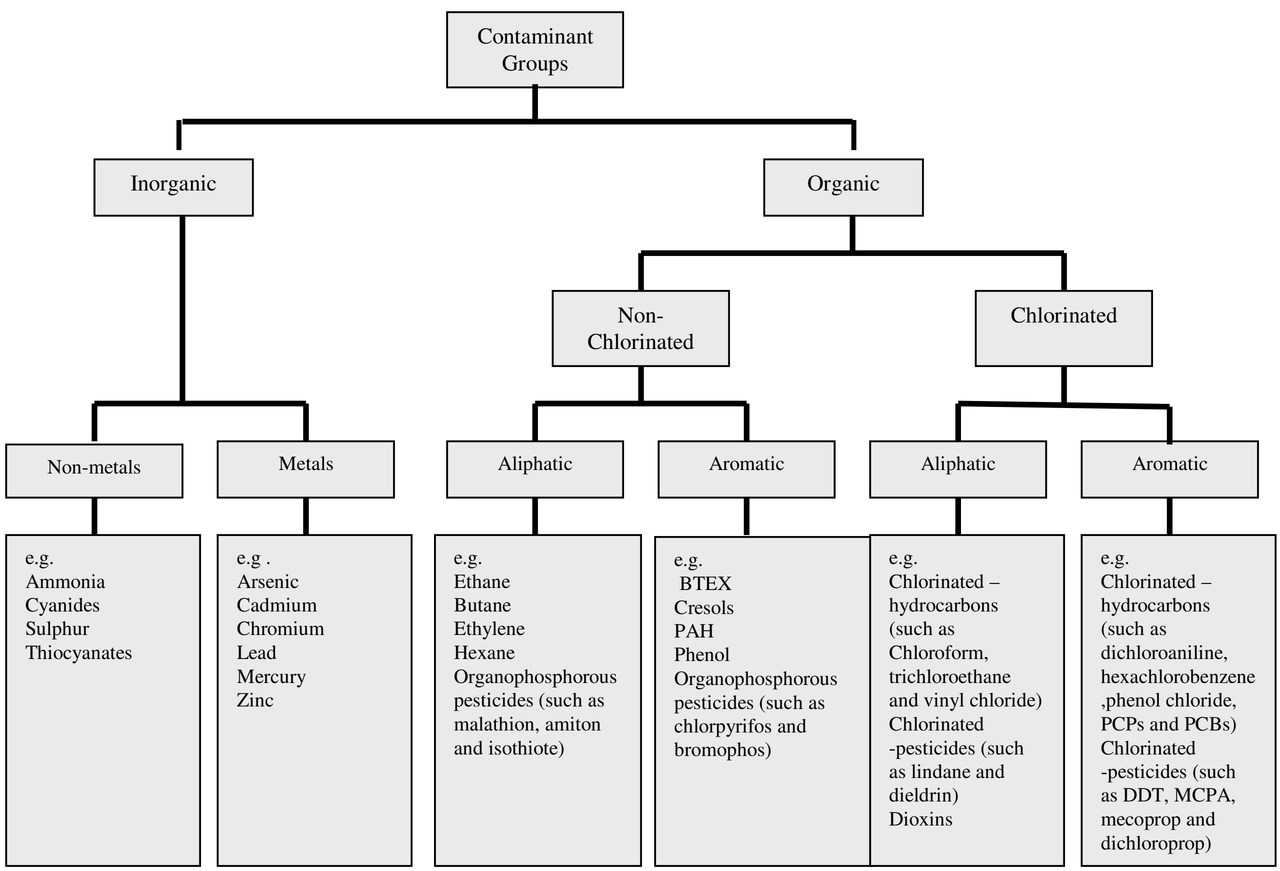


Figure 2: Contaminant Groups

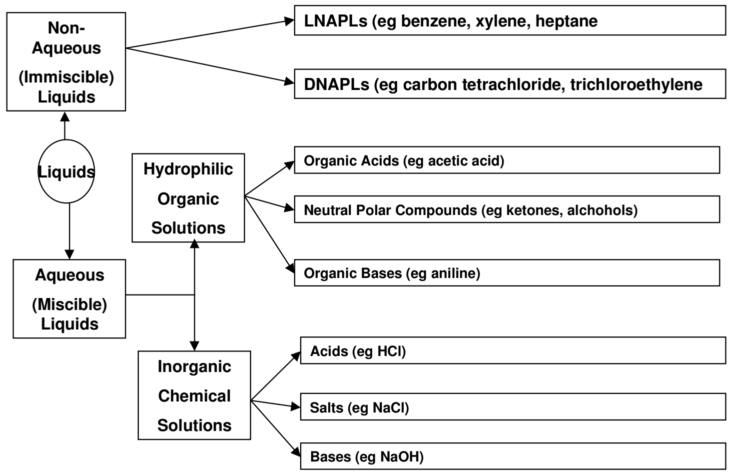


Figure 3. Classification of Liquids

1.3.4 Contaminated Land

1.3.4.1 Site Characterisation

The characterisation of a contaminated site depends on the physical condition of the site (e.g. topography, size, vegetation, etc.), the natural factors that can influence this condition (e.g. soil type, hydrology and hydrogeology) contaminant sources, pathways and the potential receptors that may be at risk. Suggestions regarding possible relevant characteristics about a site are given in Table 2.

In addition to the above it is usually appropriate to characterise the different possible exposure routes and the possible receptors that could be harmed by the contamination. This will include consideration of the current general exposure pattern of the area under consideration (e.g. industrial area, old waste disposal sites, etc.).

Table 2: Parameters useful for Site Characterisation

Parameters	**Description**
Landform	Topography, landform type, area, position, slope, etc.
Land Use and Vegetation	All previous and current land use, residual effects of previous usage, man-made features, vegetation, buildings and hardcover, etc. Future intended land use should also be taken into consideration.
Surface Characteristics	Erosion, surface sealing, surface cracks, rock outcrops, etc.
Hydrology and Hydrogeology	Surface water, rainfall, evapotranspiration, surface runoff, groundwater recharge, presence and depth of water table, moisture conditions, etc.
Geology and Geotechnics	Soil type, soil depth, origin of parent material, sequence and depth of horizons, colour, organic matter content, texture, grading, pH, structure, fracturing, degree of compaction, porosity, geochemistry, roots, biological activity, etc.
Contamination	Types of chemicals, phase, source, depth, concentrations, pattern of distribution, degree of weathering or alteration, availability of the contamination.

1.3.4.2 Soil Classification

Soil characteristics depend on the original rock or geological deposit from which the soil comes, together with other influences such as microbial activity and climate which have the ability to considerably modify the original material, giving distinct horizons within the profile. This modification results in a wide variety of soils differing in physical and chemical characteristics. Even within one soil type, large variations may occur within a short distance or depth. A number of established publications give classifications of soil and these are not reproduced here.

The concentration of a contaminant substance in a soil may not be the only information that is required. Substances present in soil may be bound to the soil matrix by sorptive and binding mechanisms. These mechanisms are likely to affect the availability of the substance with respect to uptake by human and other receptors. The mechanisms referred to above can change with time due to alteration of the soil or to alteration of the binding mechanism.

In addition to the substances known to have entered the ground and/or groundwater, consideration should be given to the possibility of any interaction between these substances which may produce 'daughter' products with completely different properties to the parents, including solubility and migration potential. The production of many manufactured chemicals results from very complex processes. These complex processes do not necessarily cease when chemicals enter the ground. They can react with the ground as well as with each other.

The following presents some of the soil physical, chemical and biological properties which should be taken into consideration when characterising a contaminated site. Not all will apply in every case.

Physical Characteristics: Porosity, permeability, density, solids content, particle size distribution, clay content and moisture content.

Biological Characteristics: The soil concentration of a contaminant at different depths in the soil will depend not only on physical and chemical processes but also on the microbial influence on the degradable components of the soil and the contaminants. It is necessary therefore to know the microbial activity in the soil and the ability of the microbes to alter the soil and/or contaminants.

Chemical Characteristics: A number of basic chemical parameters influence the soil processes that alter the contaminant concentrations, e.g. absorption and precipitation. Some of these are organic carbon content, cation exchange capacity (CEC), pH, redox potential, oxygen content and the presence of gas.

1.3.4.3 Classification of Contamination

Contamination in the ground can take many forms. It can be solid, liquid or gas. Classification of chemicals or chemical solutions is discussed in Section 1.3.3 and these are usually the contaminants of most concern. A classification for solids and gases is not presented here.

1.3.4.4 Classification of Remediation Techniques

There are various ways in which remediation technologies can be classified. This may be according to the objective of the technology (containment vs treatment), the process involved (physical, chemical, biological or thermal) or whether the process is carried out in situ or ex situ. A general classification by Shackelford (1999) using all of these categories is presented in Table 3.

Table 3. Classification of Remediation Technologies Based on Soil as the Contaminated Medium (Shackelford, 1999)

Soil Removal	Technology Category	Technique / Process	Example(s)	Comment(s)
Yes (ex-situ)	Containment	Disposal	Landfills	On-site vs off-site, new vs existing
	Treatment	Chemical	Neutralisation, solvent extraction	Treated soil may require disposal in a landfill or may be returned to the site
		Physical	Soil washing, stabilization, solidification, vitrification	
		Biological	Biopiles, bioreactors	
		Thermal		
No (in-situ)	Containment	Pump & treat	Vertical wells, horizontal wells	Both passive and active containment are possible. In pump & treat, pumping used to control hydraulic gradient and collect contaminated water – treatment is ex situ.
		Capping	Traditional covers, alternative covers, geochemical covers	
		Vertical barriers	Slurry walls, grout curtains, sheet piling, biobarriers, reactive barriers	
		Horizontal barriers	Grout injected liners	
	Treatment	Chemical	Oxidation, chemical reduction	Technologies with (*) require removal of gas and/or liquid phases and ex situ treatment. Both passive and active treatments are possible.
		Physical	Flushing*, stabilisation / solidification, vitrification, air sparging (AS) *, soil vapour extraction (SVE) *, electrokinetics (EK) *	
		Biological	Monitored natural attenuation (MNA), bioventing, bioslurping, biosparging	
		Thermal	Steam injection (*), radio frequency heating (RF) *, vitrification*	

1.3.5 Geosynthetics

Geosynthetics can be defined as planar products manufactured from polymeric material, which are used with soil, rock, or other geotechnical engineering-related material as an integral part of a man-made project, structure, or system (ASTM, 1995). Geosynthetics play an important role in this protective task because of their versatility, cost-effectiveness, ease of installation, and good characterization of their mechanical and hydraulic properties. Furthermore they can offer a technical advantage in relation to traditional liner systems or other containment systems. Geosynthetics systems are nowadays an accepted and well established component of the landfill industry (since at least early 1980's). Containment systems for landfills typically include both geosynthetics and earthen material components, (e.g., compacted clays for liners, granular media for drainage layers, and various soils for protective and vegetative layers).

Geosynthetics applications are very diverse. In order to fulfil different functions in the design of geotechnical, environmental, and groundwater related systems, the geosynthetic industry has developed a number of products. Geosynthetics have numerous material properties. Many of the reported properties are important in the manufacture and quality control of geosynthetics; however, many others are also important in design. The material properties related to the manufacture and quality control of geosynthetics are generally referred to as index properties and those related to the design as design or performance properties. Considering their different properties, the several geosynthetic products can perform different functions and, consequently, they should be designed to satisfy minimum criteria to adequately perform these functions. There are numerous types of geosynthetics, which can be used in waste containment applications and each has a specific function (Zornberg and Christopher, 1999). Functions can include:

- Separation: the material is placed between two dissimilar materials so that the integrity and functioning of both materials can be maintained or improved,
- Reinforcement: the material provides tensile strength in materials or systems that lack sufficient tensile capacity,
- Filtration: the material allows flow across its plane while retaining the fine particles on its upstream side,
- Drainage: the material transmits flow within the plane of their structure,
- Hydraulic/Gas Barrier: the material is relatively impervious and its sole function is to contain liquids or gasses, and
- Protection: the material provides a cushion above (or below) geomembranes in order to prevent damage by punctures during placement of overlying materials.

In some cases, a geosynthetic may serve multiple functions (e.g., a geocomposite layer that serves as a drainage means and a protection layer for an underlying geomembrane). Geosynthetics are manufactured in a factory-controlled environment. They are packaged in sheets, placed in a roll or carton, and finally transported to the site. At the project site the geosynthetic sheets are unrolled on the prepared subgrade surface, overlapped to each other to form a continuous geosynthetic blanket, and often physically joined to each other. The geosynthetic types are as follows:

- Geotextiles
- Geomembranes
- Geogrids
- Geosynthetic clay liners (GCLs)
- Geocomposite sheet drains
- Geocomposite strip (wick) drains
- Geocells
- Erosion control products
- HDPE vertical barrier systems

A description of the different geosynthetic products is provided by Zornberg and Christopher (1999). Geotechnical, environmental, and groundwater systems frequently incorporate several types of geosynthetics, which are designed to perform more than one function in the system. The bottom and cover liners of waste containment facilities are good examples of applications that make use of geosynthetics for multiple purposes. In these facilities, the different geosynthetic products are combined to fulfil the functions of infiltration barrier, filtration, separation, drainage, protection, and reinforcement. The specific function(s) of the different geosynthetic(s) are presented in Table 4. A review of recent advances on the use of geosynthetics in waste containment facilities is provided by Bouazza et al. (2002).

Table 4. Function of Different Geosynthetic Products (Zornberg and Christopher, 1999)

	Geotextile	Geomembrane	Geogrid	GCL	Geocomposite Drain	Geocomposite strip drain	Geocell	Erosion control product	HDPE Vertical barrier
Separation	X						X	X	
Reinforcement	X		X				X		
Filtration	X								
Drainage	X				X	X			
Infiltration barrier	X(1)	X		X					X
Protection	X			X					

(1) asphalt-saturated geotextiles

1.3.6 Waste Materials

1.3.6.1 General

Waste can be considered to be a multiphase medium comprising solid, liquid and gaseous components in a similar manner to soil, although there are also many differences between waste and soil. There are a number of different classification systems that have been put forward by various organisations or individuals, some from a regulatory / administration point of view and others from a geotechnical point of view.

As stated above, for those classification systems that are based on geotechnical behaviour they are not intended to indicate that waste can necessarily be directly assimilated to a soil. However, soil mechanics principles are a good starting point in understanding the behaviour of waste.

1.3.6.2 Classification of Wastes

This section refers to solid wastes rather than liquid wastes. Although Environmental Geotechnics must consider the impact of liquid wastes, one of the main purposes of classification is in respect of the geotechnical properties of solid wastes.

Solid wastes are often classified as inert, municipal, industrial, agricultural and radioactive, with industrial wastes possibly being sub-divided into low, medium and high toxicity. This classification is usually for regulatory or similar purposes and does not assist the specialist in Environmental Geotechnics to assess how the wastes will perform. Radioactive wastes are a specialist area and are not discussed further in this section.

In respect of geotechnical behaviour, Grisolia et al (1995) divided the solid components of municipal solid wastes (MSW) into the categories shown in Table 5. Grisolia & Napoleoni (1996) comment on the choice of design parameters relating to this categorisation.

However, the fluids within the wastes modify the properties of the solid materials and the behaviour of many wastes is controlled by the 'water content'. In respect of water content, Manassero & Shackelford (1994) classify industrial wastes as shown in Table 6 and comment on this classification by saying that Classes (a) and (b) are characterised by source processes in a wide range of grain size distributions for dry particulate material which generally has to be wetted for transportation and storage. On the contrary Class (e) includes fine grained wastes with high water contents near their liquid limits, ie sludges and muds. Wastes in this class can have water contents in excess of 200 to 300 %. Classes (c) and (d) contain a wide range of wastes in respect of both grain size and water content.

Manassero & Shackelford differentiate between soil-like wastes (particulate materials for which soil mechanics principles are applicable, e.g. sludges, ashes, excavated materials, etc.) and non soil-like wastes (materials for which soil mechanics principles are not applicable, e.g. plastic, drums, wood, metal, etc.).

Characterisation of the mechanical features of industrial wastes is proposed by Manassero & Shackelford as shown in Table 7, although this characterisation could also be applied to other wastes.

Table 5. Classification of Municipal Solid Wastes (Grisolia et al., 1995)

Class	Description
A	STABLE INERT MATERIALS: Materials such as glass, metal, masonry rubble, etc. whose overall mechanical behaviour can be assumed as that of a natural coarse-grained and heterogeneous soil.
B	HIGHLY DEFORMABLE MATERIALS: Materials such as fabric, paper and plastic which can arrange themselves in the landfill, especially when in sheets, so as to undergo marked settlements even under modest loads and exhibit an overall mechanical anisotropy.
C	READILY DEGRADABLE MATERIALS: Materials such as food wastes, vegetation, other organic matter, etc. which can cause deep and important physicochemical transformations to occur in waste in a relatively short period of time. Organic matter decomposition undoubtedly affects the overall mechanical behaviour because it causes a volume reduction together with production of gases and very low consistency materials.

Table 6. Classification of Industrial Wastes (Manassero & Shackelford, 1994)

Class	Description
a	Residues from incineration processes (fly and bottom ash from coal power plants and from incineration of municipal and industrial wastes, flue gas ash dry treatments, boiler slag, etc)
b	Residues from metallurgical industry processes (steel slag, blast furnace slag, foundry sands, etc)
c	Residues from construction, oil industries, subsoil treatments and investigations (construction debris, asbestos, contaminated soils, drilling sludges, etc)
d	Dry or quasi-dry residues from physico-chemical treatments (dust from air treatments, oxides, salts from chemicals, metallurgical, pharmaceutical and other industries, etc)
e	Residues from waste liquid and gas treatment plants (slurries from sewage water treatments, slurries and ion exchange resins from waste water and gas treatments of painting, leather, paper, agricultural, metallurgical, mechanical, chemical, pharmaceutical and other industries, flue gas desulphurisation treatments of power plants, etc)

Table 7. Characterisation of the Mechanical Behaviour of Industrial Wastes (Manassero & Shackelford, 1994)

Class	Characteristic
i	SHEAR STRENGTH: pure friction, locking or dilatant behaviour, osmotic and matric suction, cementation, fibres or rigid inclusions reinforcement
ii	DEFORMABILITY UNDER EXTERNAL STRESS VARIATION: intrinsic deformability and/or structural collapse of solid particles and rigid bodies, deformability of solid skeleton
iii	STRAIN: collapsible solid skeleton, positive or negative changes in pore pressure, changes in pore liquid content (degree of saturation), changes in chemical composition of pore liquid (osmotic suction), biological and/or physico-chemical degradation, ravelling (movement of fines into voids), viscosity of the solid skeleton or single solid particles or inclusions

1.4 RISK ASSESSMENT

1.4.1 Principles and Application to Environmental Geotechnics

The objective of a risk assessment is to determine the risk to humans, property and the environment so that an appropriate degree of protection can be incorporated into the design for a specific site or application, rather than relying on a standard specification which might be totally inadequate to provide the level of protection that is required or may be unnecessarily excessive for certain sites. Risk assessment can considerably improve the basis upon which decisions are made.

In environmental geotechnics, risk assessments can be applied to the migration of gases, the egressing of leachates from a landfill and the leaching of contaminants from a contaminated site amongst others, together with the potential effects of these processes.

The assessment can be undertaken in a number of ways from purely subjective to the use of highly complex numerical and statistical methods. However, it is important that those carrying out, or using the results of, a risk assessment are aware that risk cannot be calculated precisely and that all methods, whether subjective or numerical, will produce results with degrees of uncertainty. Thus, in order to make a decision on which method to use it is necessary to consider not only the site situation but also the limitations of each method.

There is an increasing trend towards quantitative risk assessment. In a number of countries it is either a legislative requirement or is included in the guidance documentation for that country.

A risk assessment should consider both human and ecological receptors (and sometimes property). Both human and ecological risk assessments involve toxicology assessments. However, it should be noted that there are fundamental differences between the two types of toxicological data that are used.

The term risk assessment is often confused with a simpler, non-quantitative concept referred to as hazard assessment.

Hazard and risk can be defined as follows:

Hazard:- a situation that could occur which has the potential for human injury, damage to property or the impairment of the environment.

Risk:- the chance (or probability) of a defined hazard occurring and achieving its potential; the probability of a receptor suffering harm or loss.

A hazard is something which may occur, but it is the combination of that consequence with the likelihood or probability of it actually occurring that is the basis of quantitative risk assessment.

Fundamental to the risk-based approach is the source-pathway-receptor concept. Before a hazard (a source of contamination) can pose a risk to a receptor (humans, property or the environment), there has to be a means (a pathway) by which the receptor can come into contact with the contamination (including gas). If no pathway exists, the contamination may well be a hazard, but it does not pose a risk. If a source-pathway-receptor scenario exists, the risk can be mitigated by removing or treating the source, by blocking/removing the pathway or by protecting the receptor.

There are several key stages in developing a risk assessment approach in order to address the following issues:-

- Situation Analysis – Also referred to as the Conceptual Site Model (CSM), which includes the collection of data about the site, the situation and the surroundings to the site. The CSM also illustrates the relationships between the hazard-pathway-receptor(s) specific to the site.
- Hazard Identification - what circumstances could give rise to harm.
- Hazard Analysis - how likely are these circumstances to occur.
- Identification of Consequences - what could happen as a result, what would be the effect.
- Estimation of Magnitude of Consequences - what could be the magnitude of these effects.
- Estimation of Probability of Consequences - what is the probability or frequency of a particular hazard being realised.
- Risk Estimation - what are the magnitudes of the associated risks and what is the uncertainty of the estimate.
- Risk Evaluation - is the risk significant in relation to other risks.
- Risk Management - consideration of any action that needs to be taken to reduce the level of risk and the cost of such action.

The above approach is illustrated in Figure 4, although it should be recognised that risk assessment is an iterative process and once the Risk Management stage has been reached it may then be appropriate to revert back to the Situation Analysis stage and repeat the assessment. There are a number of variations on the above methodology that have been put forward by a variety of authors, although in general the core stages remain the same.

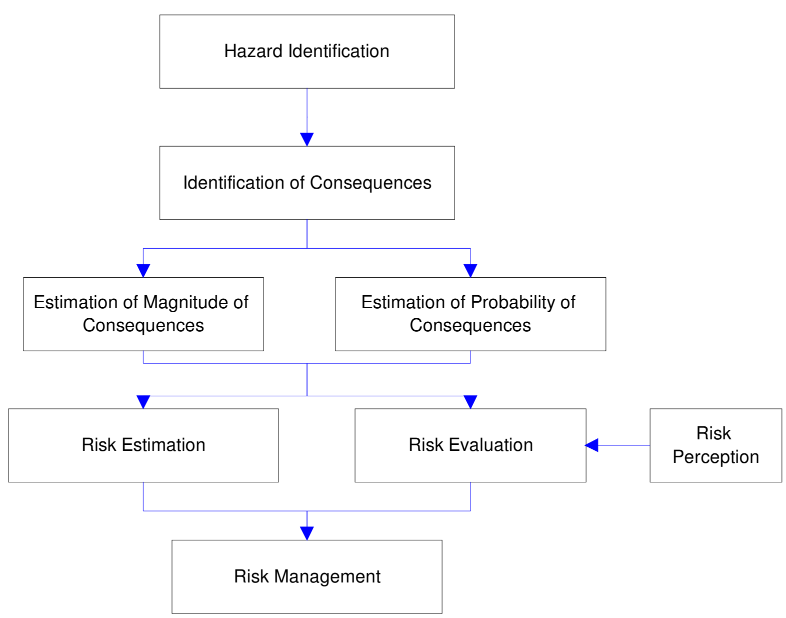


Figure 4. Risk assessment flow diagram

With regard to the number of different potentially harmful chemicals within a landfill or a contaminated site, it is often felt to be unnecessary to consider every chemical when carrying out a risk assessment. Instead certain key contaminants, both carcinogenic and non-carcinogenic, are chosen as being representative on the basis of toxicity, mobility, solubility in water, persistence and vulnerability of receptors. The risk assessment is then based on these selected chemicals.

In addition to data appertaining to the site, reference doses (levels of acceptability) and dose response relationships etc. must be chosen by reference to appropriate publications and databases. The science of deriving these values and relationships is outside the field of environmental geotechnics and requires considerable understanding of the toxicity mechanisms etc. of chemicals.

Environmental geotechnics is primarily concerned with the exposure routes that involve contaminants (including gas) egressing the site through any containment, the surrounding ground or the ground surface. The carrying out of a risk assessment will involve the use of contaminant fate and transport modelling based on the potential mechanisms of contaminant movement.

Risk assessments can be stochastic or deterministic. In a deterministic assessment all the variables are treated as known constants which are often estimates of either average, conservative or worst case conditions. There are a number of deficiencies in carrying out a deterministic risk assessment, for example it is difficult to know the degree of conservatism in the estimated risk and because every value is at or near its respective maximum the assessment considers scenarios that will never happen.

The deficiencies referred to above can be largely overcome by using stochastic methods which use probability distributions (probability density functions) instead of point estimates. Figure 5 shows a comparison between a probability distribution and what might have been taken as a point estimate (in this case the 95 percentile value).

Methods of carrying out risk assessments together with other relevant data are referred to/presented in Rubinstein (1981), Bratley (1983), Kalos & Whitlock (1985), US EPA (1988, 1989a, 1989b), La Gregga et al (1994), McKendry (1995), Jaggy (1996), and McCartney et al. (2004). In all cases minima, maxima and most likely scenarios should be considered.

The process of making decisions based on the results of a risk assessment is termed Risk Management. This will include the results of the Risk Evaluation stage in respect of the significance of the magnitude of the risk calculated. It must be remembered, however, that the results of a risk assessment are a tool in the decision process and should not be the sole means of reaching a decision on acceptability, etc.

The remaining question is what is an acceptable or normal level of risk? However, there first needs to be a distinction between background, incremental and total risk. Background risk is that which would be present in the absence of the source that is being studied, incremental risk is that due to the source under consideration and total risk is the sum of the two. A decision has to be made on whether or not to include background risk, for example when considering remedial alternatives the background risk is not relevant but when considering the maximum acceptable level in order to protect a receptor, it is.

It is now generally recognised that 'no risk' scenarios do not exist. Instead the controls surrounding a particular hazard must achieve a result that is tolerable. In safety law, any risk must be reduced to a level which is as low as is reasonably practicable, normally given as an acronym: the ALARP principle. In practice the tolerability of risk can depend upon the vulnerability of the receptor (target) and the duration of exposure to the hazard.

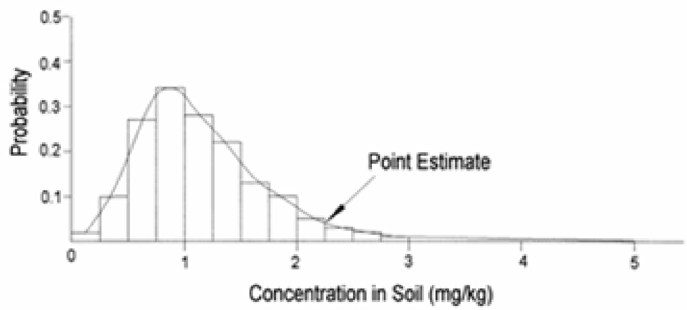


Figure 5. Probability distribution of a contaminant in soil

A tolerable level of risk is often set between a probability of 10^{-4} and 10^{-6}. However, the concepts of acceptable risk and tolerable risk are complex and involve risk perception as well as a technical assessment of risk, which are outside the scope of the present publication.

The quality of any risk assessment is dependent on the quality of input information that is available. There is little point in using techniques such as quantitative risk assessment unless an appropriate amount and quality of information is available in respect of the topography, geology, hydrogeology and climatology of the site and the toxicity, mobility, degradation rate, chemical interaction, concentration and quantity of the contaminants. This inherent prerequisite for complex quantitative risk assessment methodologies can result in an early commitment to specific types of, and detection limits for, analytical testing of contaminants. In addition, disparity of analytical methods between different laboratories can introduce varying degrees of accuracy in contaminant concentrations identified within samples.

Risk assessment provides decision-makers with information on probable contaminant transport / uptake rates and the resulting impacts so that rational, well formulated judgements can be made. It also allows designers to modify various aspects of a containment or remediation system in order to ensure an appropriate level of performance that is consistent with the site situation. It stimulates a review of the appropriateness of existing site investigation data and highlights those aspects which might need further investigation. Finally the cost-effectiveness of alternative designs, methodologies, etc. can be evaluated so that the available fiscal resources are directed towards optimising the performance.

1.4.2 Groundwater Pollution

In general terms, a risk assessment with respect to the potential for groundwater contamination would consider the following:

i) The potential for the generation of leachate from contaminated land or a landfill.

ii) The presence of any natural or artificial barriers to contaminant migration.

iii) The properties of the unsaturated zone such as the processes of seepage flow through the ground matrix, fissure or fracture flow (if applicable), decay of contaminants, dispersion, cation exchange capacity, biological or chemical alteration, and the likelihood that these will be effective.

iv) The properties of the saturated zone which will be similar to those for the unsaturated zone but also include the potential for dilution as a result of the groundwater flow.

v) The vulnerability of the receptor or target, the consequences of contamination and any regulatory requirements with respect to limits on tolerable change. The vulnerability of the groundwater should take into account the effects of bio-accumulation, which may negate the effects of dilution for certain contaminants.

1.4.3 Gas Migration

A similar process with regard to the migration of landfill gas would be along the lines of the following:

i) The generation of landfill gas: Consideration of the probability that suitable conditions could exist at the site for the generation of landfill gas in terms of atmospheric conditions, temperature, moisture, the composition and age of the waste, its burial depth and the quantity of waste.

ii) Barrier competence: Consideration of any barriers that surround the gasing materials such as liners, venting trenches, extraction wells and cut off walls and the probability that these barriers will be effective.

iii) The migration route: Consideration of the geology, hydrogeological and other physical aspects of the migration route and the distance from the gasing ground to a particular receptor; the primary considerations being pressure gradient, distance and ground hydraulic conductivity and the degree of accuracy with which each one of these can be determined.

iv) The ease of entry: Consideration of the means of entry into a building, excavation or other form of confined space in terms of barriers, extraction/ventilation systems, form of construction, quality of workmanship and durability of construction.

v) Internal conditions: Consideration of the degree of ventilation within the building and any alarm systems that have been or are to be installed, the likelihood of an ignition source in respect of methane and the frequency of occupancy/entry in respect of carbon dioxide.

1.4.4 Reliability

As stated above, risk assessment is an iterative process and there are a number of assumptions that have to be made in each of the various stages. The sensitivity of the magnitude of the calculated risk to these assumptions should be assessed and where necessary refinements to the risk assessment should be made (sustained development of CSM). In this context it may be necessary to obtain additional data.

When carrying out a risk assessment it is not possible to enumerate every conceivable outcome and therefore credible worst case scenarios are used. This introduces an inherent conservatism, which often results in assessing scenarios that will never be experienced. The counter to this is that the additive process used in the models in respect of toxicity data may understate the risk of exposure to complex mixes of toxic substances.

Once a risk assessment has been carried out it is important to indicate to the decision-makers who will use the results of the risk assessment, what degree of uncertainty is inherent in the particular calculation of risk. This uncertainty comes from the deficiencies and assumptions within each of the stages that comprise the risk assessment.

1.4.5 Further Work

A considerable amount of work is still required in order to refine and improve the technique of risk assessment associated with landfill containment and contaminated land. In contrast to the nuclear and mining industries, for example, the concept of risk assessment has only relatively recently been applied to these topics.

Techniques of risk assessment have advanced quite considerably over recent years, together with the recognition of the benefits that risk assessment can bring to a decision process. There still remain, however, a number of gaps in our knowledge and understanding, as discussed below.

1.4.5.1 Contaminant Source

Over the years a large number of contaminants have entered soil and groundwater. In terms of appropriately directing research resources we need to focus on those posing the greatest potential risks to human health and the environment.

With regard to petroleum hydrocarbons, for example, research should focus on benzene because of its carcinogeneity, volatility and water solubility. Next would be the other BTEX constituents (toluene, ethylbenzene and xylenes). Polyaromatic hydrocarbons (PAH) should have lesser priority because although they can be carcinogenic, their mobility in the environment is low.

Other organics requiring attention include: chlorinated hydrocarbon solvents (aliphatic and aromatic), phenols, aromatic compounds such as pesticides and the fuel additive methyl-tertiary butyl ether (MTBE).

The output from different risk assessment models primarily varies because of the use of different default parameters, not least of which is toxicity data. These can be sourced from different databases and for some chemicals there is considerable disagreement between databases. There is a need for toxicity data to be reviewed and rationalised to eliminate inconsistencies and produce a peer-reviewed data-set.

The current knowledge of chemical toxicity is primarily based on tests carried out with pure compounds. We know very little about the effects of mixtures of chemicals in the ground.

Further work needs to be carried out in respect of the bio-availability of contaminants. Present risk assessments are generally based on the assumption that all of the contamination is available to cause harm. This is just one of many layers of conservatism.

Speciation is also an important issue which can have a big influence on toxicity. There is a need for standardised methodologies for speciating complex mixtures of petroleum hydrocarbons, although a significant amount of research and guidance has been provided in the Total Petroleum Hydrocarbon Criteria Working Group Series (TPHCWGS).

Many chemicals are produced by very complex processes. This complexity continues when chemicals enter the ground and interact with the ground or each other. Very little is known about the breakdown products of many contaminants. We need to know more about the conditions under which they are produced, what produces what and how the breakdown (daughter) products behave. Some breakdown products may be more toxic and have greater mobility than the original parent compounds.

1.4.5.2 Pathway

Natural attenuation can be a very effective means of restricting a pathway in groundwater. However, more work needs to be done in respect of how effective this process is in different geological and hydrogeological conditions. Those making decisions need to know the time periods over which natural attenuation can occur and how these time periods may be influenced by conditions. The mobility of many contaminants is still poorly understood and there is a need for more work on this aspect.

1.4.5.3 Receptor

The main consideration in respect of the receptor (or target) is the determination of the acceptable level of risk. Tolerable levels of risk need to be published by Governments and Regulatory Authorities for a range of situations .

The success of risk-based approaches depends on gaining the confidence of all stakeholders. There is therefore a need for good mechanisms of risk communication.

Furthermore, some countries would benefit from establishing native risk assessment methodologies to encourage uniformity (within a specific country) in the analysis, interpretation and communication of human and environmental risk associated with landfill and contaminated land.

1.5 MONITORING

1.5.1 Introduction

Monitoring in Environmental Geotechnics is normally associated with either contaminated land or landfills. The term "monitoring" should be considered to mean repeated inspection or survey and/or sampling and testing (e.g.. groundwater quality) at permanent testing points together with the presentation of the results. However, the assignment may also involve discussion of test results and consideration of impacts on human health and the environment. The results can be used to design measures to mitigate or eliminate negative impacts or alternatively to allow / permit / justify natural attenuation.

This section deals mainly with the monitoring of groundwater quality associated with contaminated land or landfills. Most monitoring of groundwater involves either measuring water levels and/or pressure or either taking samples for laboratory analysis or using instruments to measure chemical parameters in-situ, e.g. pH and conductivity. The techniques used can range from extremely sophisticated installations and instrumentation to relatively simple field observations. They can be intrusive or non-intrusive.

1.5.2 Non-intrusive Methods

A variety of non-intrusive monitoring methods are available. These include remote sensing such as aerial photography and satellite imagery as well as geophysical methods such as geomagnetics, electromagnetics, electrical resistivity, seismic, ground probing radar, induction and thermal.

In many countries aerial photographs and satellite imagery are available from Government agencies / departments or commercial operations. From these it is possible to differentiate land use, vegetation, geology and hydrology, etc. Infra-red photography, for example, can be very effective in indicating where vegetation distress is occurring which may be caused by either gas migration or chemical contamination.

Goldman (1992) provides an overview of some of the remote sensing techniques that are available and their usage in monitoring contaminated land. He examines both unmanned and manned orbiting systems and the use of both high and low flying aircraft.

Non-intrusive methods are not generally sufficient on their own to characterise ground and groundwater conditions. However, the methods are generally less expensive than intrusive methods in relation to the areal extent of the information obtained and can be used as a reconnaissance tool to either plan future intrusive monitoring or to extrapolate information from existing intrusive installations, e.g. monitor any change in the boundary of a contaminant plume once the characteristics of the plume are known.

1.5.3 Intrusive Methods

The measurement of groundwater pressure or phreatic surface is normally by means of piezometers. These can be pneumatic, electrical, simple standpipes, etc. A standpipe (or similar) monitoring well can be used for the measurement of pressure (as in a piezometer) and for the taking of water samples or the carrying out of in-situ tests.

A monitoring well normally consists of a response zone (ie a screened or perforated length into which groundwater can flow and which is surrounded by a filter sand or gravel pack which reduces the amount of fines from the surrounding formation that enter the well) above which there is a casing or riser pipe (usually of the same diameter) which isolates the well from the surrounding ground and groundwater above the response zone. If the monitoring well is installed in a borehole, seals (often comprising of bentonite) are formed between the riser pipe and the surrounding ground to prevent the vertical flow of ground or surface water down or up the outside of the casing within the borehole.

For a response zone in an unconfined aquifer, the water level in the monitoring well will equal the water level in the aquifer. For a confined aquifer, the water level in the monitoring well will represent the piezometric head or pressure head in the aquifer. The well is finished at the surface with a cap and some form of protection works.

Care is required in ensuring that the response zone of the installation corresponds to the hydrogeological horizon that is to be monitored. Wells with response zones that cross two or more hydrostratigraphic units are almost useless. Far too many monitoring wells have been constructed ignoring this principle resulting in the data being almost impossible to effectively interpret.

In very variable ground conditions it can be very difficult to determine the correct location (in respect of areal extent and depth) of the response zones. The most difficult is where discontinuous low permeability layers are present that have the potential to redirect the flow direction (vertical and horizontal components) of the contaminant plume. In this situation it is possible to place the response zones too shallow or too deep to detect contamination. In such cases a clusters of wells or multipoint wells (see below) may be needed at a number of monitoring locations across the site. This would also apply where there are head differences between aquifers in a multi-layered aquifer system.

The materials used in the construction of the monitoring well must be non-reactive with the suspected contaminants. Schowengerdt (1987) indicates that stainless steel, Teflon®, and Bisphenol-A Epoxy represent examples of relatively non-reactive casing, whereas PVC casing exhibits sorbent and leaching characteristics with certain contaminant constituents. PVC is resistant to most chemicals except aromatic organics, ketones, esters, etc. According to Kent et al (1985) the amount leached from the casing is small and does not usually affect the integrity of the casing.

The more significant problem is the use of PVC glue to fix couplings to casings and well screens, etc. Tests carried out by Kent et al (1985) found that distilled water subsequently contained methyl ethyl ketone, toluene and tetrahydrofuran as a result of the glue. The alternative is to use threaded connections.

Another example cited by Kent et al (1985) was a project where well casing had come from two different suppliers. Monitoring wells with casing from one of the suppliers showed contaminants not present in the wells constructed with casing from the other supplier. The concentrations could not be explained by natural causes. The most likely explanation was that the supplier had used a chlorinated degreasing solvent to clean the well screens.

Other sources of contamination associated with monitoring well installation are greases applied to the treaded joints of drill pipe, leaking hydraulic fluid from the drilling rig and contaminated material adhering to the drill pipe when it is inserted into the borehole. Complete cleanliness is almost impossible to achieve on many landfill and contaminated land sites. Nevertheless, stringent controls should be applied to ensure that any self contamination is reduced to a minimum and is consistent with the degree of accuracy that is required from the monitoring programme.

An individual borehole may house just one monitoring well or a nest of wells each monitoring at a different elevation in the borehole or a multipoint system may be installed. The latter can consist of a single specially manufactured casing into which an electronic probe is inserted which opens a sealed port at each elevation where monitoring is to be carried out and then either measures pressure or takes a sample or carries out a physical or chemical test. Alternatively the installation may be a pressure driven system utilising very small diameter tubing.

The methods available for taking samples from monitoring wells include bailers, air lifts and pumps. It should be noted that variations in sample analysis results can be caused by using different sampling methods. Ideally the sampling method should be standardised throughout the sampling programme. If a change is unavoidable then samples should be taken with both previous and future methods on at least one sampling visit in order to quantify any differences.

1.5.4 Guidelines for Monitoring Design

The design of any ground related monitoring system should take into consideration the specific topographical, geological, hydrological and hydrogeological conditions prevailing in the vicinity, together with the potential risk to human health and the environment. It must be effective for every individual contamination source or potentially contaminating part of any facility. For new works the design of the monitoring network should be prepared simultaneously with the design of the works. The monitoring should be designed such that it can be extended or otherwise modified, depending on the performance observed.

The design of the monitoring should be preceded by desk studies and, if necessary, field investigations. The design, execution and operation of local groundwater monitoring systems should be managed in the following stages:

Stage I: Preliminary studies including:
- analysis of the historical geological, hydrological and hydrogeological data
- analysis of the historical data about the facility
- reconnaissance of the contamination source and its surroundings,
- preliminary field hydrological and hydrogeological studies of ground and surface waters;

Stage II: Design including:
- further analysis of the historical hydrogeological data
- characterisation of the contamination source
- assessment of the hydrogeological characteristics of the contamination source zone,
- characterisation of the factors and conditions influencing the contamination of the groundwater,
- determination of what monitoring is appropriate,
- design of monitoring network,
- determination of the methodology, scope and frequency of measurements,
- approval of the monitoring system designs by the appropriate regulatory authorities;

Stage III: Construction of monitoring network, which includes:
- installation of monitoring points (e.g. monitoring boreholes with piezometers) or adaptation of existing monitoring points, or the establishing of a programme of survey,
- marking of monitoring points and their protection against damage,
- preparation of as-built documentation including any changes compared to design.

Stage IV: Operation of monitoring system, covering the following activities:
- collection of samples ensuring safety and quality and using appropriate sampling equipment,
- field tests and laboratory analyses,
- carrying out of surveys (e.g. geophysics) if appropriate,
- continuous supply of monitoring results to the owner and the regulatory authorities as necessary;

Stage V: Reporting possibly covering one or a combination of the following:
- full results or summary of results as necessary,
- characteristics of the contamination source,
- quality, degree and extent of groundwater deterioration,
- prediction of contaminants transport with evaluation of hazards to human health and groundwater resources,
- proposals for the protection of groundwater resources against intrusion by contaminants,
- recommendations and time schedule for any longer term monitoring and/or further investigations.

1.5.5 Monitoring Performance Criteria

The design and execution of monitoring systems should be carried out by organisations that possess the following:

- qualified and licensed personnel (as required by regulations in individual countries),
- appropriate monitoring (including sampling) equipment,
- access to accredited laboratory facilities (as required in individual countries) such that there is a guarantee of high accuracy with appropriate limits of detection.

The frequency of samples collection should be determined individually for each site and, where necessary, a particular part of a site, taking into consideration local hydrogeological conditions and the degree of likely hazard to humans and the environment.

Where appropriate the risks to groundwater should be assessed with due consideration of data pertinent to other monitored elements of the environment, e.g. air quality, surface waters, plants and soils.

The need for a monitoring programme that is consistent with performance criteria is a particularly sensitive task for waste containment facilities, which have been designed using prescriptive criteria in the past, but are now being designed using performance-based design criteria. For example, federal and state mandated cover systems for municipal and hazardous waste landfills in the United States have endorsed the use of "resistive barriers". These resistive barriers have also been referred to as "prescriptive" barriers, as their design is based on prescribed dimensions and material properties that have been deemed to lead to acceptable performance. However, regulations allow alternative cover systems if comparative analyses and/or field demonstrations can satisfactorily demonstrate equivalence with prescriptive systems.

The use of alternative (performance-based) systems, has led to the need for comparing the performance of alternative cover systems with that of prescriptive cover systems (i.e. equivalency demonstration). This has required establishing monitoring systems that are consistent with the design criteria.

Design criteria must specifically account for site-specific conditions. McCartney and Zornberg (2002) outline different types of performance criteria that have been put forth for alternative covers. The type of performance criteria should be closely integrated with the design procedures and with the methods of compliance demonstration.

The following three sections discuss monitoring applied to landfills and contaminated land. Many of the principles discussed for landfills also apply to contaminated land and vice versa.

1.5.6 Monitoring of Landfills

Landfills have the potential to impact groundwaters and therefore, with few exceptions, all landfills (new and old) require monitoring. Even landfills designed to a stringent specification require monitoring and this monitoring should be part of the design. The monitoring system should be in place and the initial data (at least three sets) should be obtained and assessed prior to a landfill being commissioned and should take into consideration the possible extent of the contaminant plume that could potentially be created by the landfill.

A possible situation where monitoring would not be required would be a small municipal landfill located within an already well monitored and understood geological and hydrogeological zone where risk assessment can demonstrate that impairment is unlikely. However, each site should be assessed on its own merits.

At least one intrusive monitoring position should be upstream of the landfill so that the quality of groundwater flowing towards the landfill can be determined and a comparison can be made with groundwater downstream of the landfill. When siting these boreholes it should be borne in mind that some contamination can migrate upstream (ie in respect of the regional groundwater flow direction) particularly where either the presence of the landfill or the leakage from the landfill is causing a local mounding of the groundwater.

The determinands analysed (for samples from a range of monitoring boreholes across or around a landfill) can vary between monitoring events. For example, the analyses for the initial monitoring may be for colour, pH, turbidity, odour, dry mass, dissolved solids, hardness of suspension, electrical conductivity, biological oxygen demand (BOD), chemical oxygen demand (COD), ammonia, nitrogen, nitrite nitrogen, nitrate nitrogen, Kjeldhal's nitrogen, sulphates, sulphides, chlorides, phosphorus, cadmium, chromium, copper, lead, zinc, mercury, total hydrocarbons and total organic carbon (TOC), etc. Subsequent analyses may only be for those parameters which specifically indicate whether leaching is occurring from a waste body, e.g. chlorides, sulphates, nitrates, ammonia, BOD, COD, TOC, pH, electrical conductivity, heavy metals).

Periodic use of the full suite may also be appropriate, e.g. quarterly, half yearly or annually, depending on the significance of any impact on humans or the environment. This would also be the case if statistical assessment indicates that contaminant concentrations from the landfill are increasing.

Monitoring of landfill performance has proven useful not only to demonstrate compliance, but also to aid in the design of expansions of the waste containment facility. For example, monitoring of waste properties in an unlined landfill in southern California was useful to evaluate the environmental implications of its proposed vertical expansion (Zornberg et al., 1999). Specifically, monitoring results were used to evaluate if additional compression of the waste will squeeze liquid from the waste. Field monitoring and experimental data were used to evaluate the ability of the landfill to continue to retain moisture after continued waste placement. The evaluation indicated that the moisture content of the waste will not reach its field capacity for the proposed final grading of the landfill and, therefore, that the liquids should remain within the waste mass after the vertical expansion.

Non-intrusive monitoring methods can also be used as discussed in Section 1.5.2 above. Costa el al (2002) used geophysical methods to complement the intrusive methods of monitoring employed at the Bangu Waste Disposal Site in Rio de Janeiro, Brazil. An electromagnetic survey was used to quantify the thickness of the layers saturated with leachate as well as inferring the groundwater flow directions. The extent of the probable contaminant plume was thereby delineated and subsequently monitored by targeted boreholes whose location was chosen by reference to the results of the geophysical survey.

Shrivastava and Mimura (1996) provide an overview of a number of geophysical techniques used with cone penetration technology. Future trends are described and the effectiveness of the available technology is presented. Cossu et al (1990 and 1992) provide a similar overview in respect of surface geophysical methods. They considered that the techniques were of particular use in respect of old landfills where there is generally a lack of basic information.

Landfill gas emissions, mainly methane and carbon dioxide, are an important consideration in the monitoring of landfills. Surface, sub-surface and depth monitoring should be carried out to ascertain the cap (final cover layer) efficiency, the waste decomposition process and any migration through the sides or base of the landfill that may be taking place. Maciel and Juca (2002) presented details of the monitoring carried out at the Muribeca Landfill, Recife, Brazil. They used a flux box to measure the gas emission from the surface of the completed part of the landfill and boreholes to determine the degree of decomposition in the waste mass.

Another aspect of the monitoring of landfills would be the checking of the protection against slope failure or changes in the stability of slopes. Following a large slope failure at a landfill in Bogota, Colombia, Rodriguez and Velandia (2002) describe an extensive monitoring programme involving 162 piezometers, 30 inclinometers and 400 topographical control points. The results in respect of pore pressures, deformations and settlements over a three year period are presented. The information obtained was useful to understand the behaviour of the landslide and its change with time, including the stability and mechanical properties of the wastes. This work highlighted the importance of monitoring in being able to control failure situations and enable decisions on any action to be taken in time for that action to be effective.

A further potential use of monitoring would be in respect of the settlement characteristics of the waste. Gourc et al (1998) describe a proposal for a waste settlement survey methodology. They used transducer settlement gauges buried in the waste mass at two landfills to monitor long term settlement. The use of such information could be for future cap design and/or to enable judgements to be made on how much 'overtipping' to permit on future similar landfills such that surface levels finally reduce back to design levels.

Monitoring activities should be continued after closure of landfill operations and until such time as either regulatory criteria are met or risk assessment indicates that the landfill no longer has the potential to cause harm.

The monitoring should be designed and carried out by specialists. Copies of all of the results of the monitoring should be held in the archive of the landfill owner. Annual (or more frequent) reports may also need to be submitted to the regulatory authorities in the particular region or country.

1.5.7 Monitoring of Contaminated Land (as part of investigations)

Before decisions can be made on the remediation / reuse of contaminated land, it is necessary to determine the extent of any potential impact on human health and the environment. Monitoring should be part of the investigations that are carried out for this purpose in order to provide valuable additional information on the characteristics of the contaminants, the hydrogeological conditions of the site and the surrounding area and any variations in contaminant concentrations.

The final design of the monitoring programme should be based on the results of the investigation. There are unfortunate cases where the monitoring programme was designed at the same time as the design of the investigation and not subsequently modified in accordance with the investigation findings, such that important aspects of the contaminant source and the groundwater regime have not been monitored, resulting in inappropriate remediation or lack of remediation where in reality remediation was necessary.

Examples of monitoring for contaminated land are:

- observations of groundwater levels,
- observations of ground and surface water quality with attention to space and time variations,
- observations related to emissions of gases and dusts, compared with survey of absorptions,
- observations of possible changes due to microbiological activity and chemical and physical processes in transporting media and substances under examination,
- observations of vegetation, cultivation, development of flora and fauna,
- checking of the effectiveness of any improvements that have been implemented, e.g.. tightness of containment (see Sections 1.5.8 and 1.5.9),

The frequency of monitoring should be commensurate with the degree of risk that has been assessed based on existing information. In any event it should not be longer than 1 year and will usually be more frequent. The overall duration of the monitoring should take into account any remediation that has already taken place, the durability of the contaminants, the groundwater regime, the rate of change of the contaminant plume, seasonal variations, the vulnerability of the receptors and any special events that occur (e.g. heavy rain, flood, ecological disaster, etc.).

When collecting contaminated samples, appropriate safety regulations / guidelines should be adhered to, e.g. qualified personnel, verified procedures, calibrated equipment, briefings and inductions, personal protective clothing and equipment, etc. Internationally recognised standards, e.g. ISO, should be used and quality control systems should be introduced. The taking of samples is one of the most important parts of any contaminated land investigation. The samples must be representative of the in-situ conditions and therefore it is important that volatiles, etc. are not lost in the sampling or in the subsequent on-site storage or transportation to the laboratory, etc. Appropriate fixing additives must be used where needed. The samples must be stored and transported at sufficiently low temperatures so that alteration of the sample does not take place or is reduced to an acceptable minimum. Most testing laboratories will give advice on these issues for samples that will be going to their laboratory. These principles also apply to landfills.

It is not unusual on contaminated sites to encounter an upper unconfined aquifer separated from a lower aquifer by an aquitard. The installation of a monitoring well into the underlying aquifer requires special measures to ensure that cross contamination is not caused. These measures can consist of advancing the borehole a small distance into the top of the aquitard and then sealing (by grouting, etc.) a temporary casing into the borehole. Boring then continues through the temporary casing.

It is also extremely important that cross contamination does not occur between monitoring well positions. This requires thorough cleaning of all boring equipment between borehole positions and all monitoring equipment that will be used in the wells. The above also applies where probes are used.

For many contaminated land situations it will be appropriate to use a combination of monitoring techniques in order to obtain the required type and extent of information. Copeland et al (1996) used a combination of active and passive soil gas sampling, air-flux chamber measurements, cone penetrometer tests, HydroPunch and PowerPunch groundwater sampling and a field laboratory to carry out a regional groundwater investigation associated with a contamination plume at San Leandro, California. The methods used enabled the investigators to minimise sampling and sampling costs, to select optimum locations for sampling and well installation, to correlate between hydrogeological stratigraphic units and to evaluate the extent of shallow groundwater contamination. The largest plume (containing a cocktail of volatile organic compounds (VOCs), petroleum compounds, nitrate and metals) in the study area was found to be at least 30 m deep, more than 1.6 km wide and 4 km long, approaching the San Francisco Bay. The monitoring was used to plan further investigations, identify contaminant sources, provide input to risk assessments and facilitate the development of a remediation strategy.

It is often difficult to measure continuous change of contamination at a certain depth or to measure precise distribution of parameters in deeper boreholes. Sano et al (1996) have developed a system to measure water quality in a borehole up to 1500 m deep. The system continuously measures seven parameters, pH, conductivity, salinity, dissolved oxygen, redox, temperature and pressure.

Where contaminant plumes contain VOCs, a component of these contaminants will be present as a vapour or soil gas. This vapour phase can be monitored in boreholes using portable photoionisation detectors to measure VOC concentrations, thereby giving a general indication of the extent of, and variation within, a contaminant plume.

1.5.8 Monitoring of Remediation (during treatment and post treatment)

The monitoring arrangement must allow appropriate assessment of the treatment process. In some cases this could require a complex system to be put in place involving a number of different monitoring methods. The monitoring system should be capable of evaluating changes in the degree of contamination with time, in vertical extent as well as in horizontal direction.

The monitoring systems can, where appropriate, incorporate monitoring installations that were put in place during previous investigations, provided that these meet the requirements of the remediation monitoring design. Previously installed monitoring facilities should not be used for convenience reasons only.

The effectiveness of the treatment process can be related to the initial contaminant concentrations. However, it is more usual for acceptability criteria to be stipulated, ie the maximum residual contaminant concentrations in the soil and/or groundwater that are permissible for the remediation to be considered as complete. These acceptability criteria are normally agreed / approved by the local or country regulatory authorities, e.g. Environment Protection Agency.

Where possible, the monitoring locations should include locations outside the contaminated area. This will enable background groundwater quality to be determined at the start of the remediation and any adverse effects on the surrounding groundwater to be determined during the remediation. In the absence of any acceptability criteria the background groundwater quality will also provide a comparative means of assessing the performance of the remediation.

The analysis of the results from the monitoring system should include the variation of the contaminant concentrations with time. These concentrations usually decrease non-linearly depending on the interrelation of processes such as dilution, dispersion, percolation and gas emission, as well as chemical processes such as coagulation, redox reactions, precipitation, solution, sorption, and desorption.

Remediation of soils can result in contaminants such as organic compounds (previously adsorbed by soil particles) being released into the groundwater. The monitoring in this case should be supplemented by the laboratory analysis of soil samples taken from the treatment zone.

For some remediation schemes a point in time is reached when the continuation of the treatment would result in costs being incurred which would not be commensurate with the improvement in contaminant concentrations that would be achieved. In this situation it may very well be better to allow natural biochemical reactions in the ground to complete the process. Continuing the monitoring can provide confirmation that the natural attenuation processes are having an effect and that rebound of contaminant concentrations has not occurred.

1.5.9 Monitoring of Containment Barriers

The monitoring should be effective in demonstrating the competence of the barrier system in respect of showing that no more than the design leakage is taking place through or around the barrier.

Bieberstein and Saucke (1990) have presented the details of a testing unit developed at the Institute of Soil and Rock Mechanics in Karlsruhe, Germany. This unit is intended to be inserted into wall panels at the time of construction before the slurry hardens. It determines the in-situ permeability of the panels immediately after construction and at any time thereafter.

In the case of reactive barriers, the monitoring will also need to show that no process reversal is taking place, ie that the barrier itself has not begun to release contaminants that it had previously trapped.

1.5.10 Quality Assurance

A monitoring programme is only as good as the quality of the data it produces. Quality assurance is therefore an important aspect of any monitoring. This should extend from the construction of the installations, the calibration and correct use of instrumentation, the use of appropriate sampling techniques, and quality assurance certification of the chemical laboratory through to vetting procedures for the final report.

Undetected errors can have a significant effect, such as remediation measures being implemented when they are not needed and vice versa. A number of extensive legal cases have arisen as a result of errors in monitoring.

Written and verified documentation at all stages of a monitoring programme is very important. This should include installation construction details, sampling and laboratory methodologies, field records and data sheets, spike and duplicates testing in the laboratory, calculations and cross checking that all data included in the report is exactly in accordance with the immediate output from the field and laboratory work. Without this documentation, there may be no third party acceptance of the results of the monitoring and the data may be rejected in regulatory applications, legal claims, etc.

1.5.11 Concluding Remarks

An overview has been presented of monitoring techniques and the principles which should be adopted when designing a monitoring programme. In every case it is essential that the person responsible appropriately designs and specifies the individual components of the monitoring system, the materials to be used, the method of installation and the methodology for carrying out tests and measurements.

Sufficiently extensive monitoring is often overlooked in a desire to get a development project underway. It should be remembered that it has been demonstrated many times over that effective monitoring will more than 'pay for itself' on the subsequent project compared to the additional costs that often result when insufficient data is obtained.

1.6 LIFETIME OF COMPONENTS

1.6.1 Introduction

Engineered barrier systems within landfills and as containment strategies for site remediation projects are constructed to provide protection of groundwater and in some cases surface water. These systems are used to attenuate selected contaminants to a higher level than would otherwise be possible based on the natural hydrogeologic conditions.

The level of attenuation will depend on the nature of the contaminant and requires an assessment of attenuation capacity considering the advective-diffusive contaminant transport as well as appropriate retardation mechanisms. Engineered containment systems tend to minimize the advective movement of contaminants (low hydraulic conductivity of the materials) and some also act as an effective diffusion barriers for certain contaminants.

It is imperative to understand that engineered barrier systems have a service life, which must be considered in safeguarding short and long term groundwater protection. This section establishes the general principles of design lives of various landfill containment components: compacted clay liners, bentonite enhanced soils, geosynthetic clay liners, geomembrane liners, cover materials and drainage layers; as well as in ground cut-off barriers.

As a basic concept, the service life of an engineered component can be taken to be the period of time that it continues to meet its design function. For example, for a leachate collection system this may be the period of time that it can control the leachate head at or below the design value (e.g. 0.3 m). For a compacted clay liner it may be the period of time that the hydraulic conductivity remains at, or below, the design value. It is necessary to design engineered landfill systems and in-ground barriers such that the service life of these systems as a whole exceeds the contaminating lifespan of the landfill or the contaminated ground/groundwater. The contaminating lifespan is the period of time during which the landfill will produce contaminants at levels that could have unacceptable impacts if they were discharged to the environment (Rowe et al., 1995).

1.6.2 Compacted Clay Liners

Compacted clay liners are clayey soils that have been remolded and constructed to obtain a low hydraulic conductivity liner. The quality of a compacted clay liner will depend on: (a) the characteristics of the clayey soil used; (b) the method of compaction and, in particular, the compaction water content; (c) the quality control during construction; and (d) the protection against desiccation after construction. Like most engineered liners, CCLs require protection from the elements (e.g. sun and frost) and consideration must be given to the potential long term effects of differential settlements.

Successful construction of a low hydraulic conductivity CCL using an adequate soil type and therefore design life of this system is highly dependent on: (a) water content control; (b) breakup of clods of soil and homogenization of nonuniform soils; (c) lift thickness; and (d) method of compaction and equipment used. CCLs may experience an increase in hydraulic conductivity with time by several orders of magnitude if not adequately protected against desiccation cracking or frost damage. This is also a particular concern for CCL used as cover material in landfills.

Both desiccation cracking and frost damage protection are key to the design life of these barrier systems and can be mitigated through quality construction control and assurance. For a further discussion on the layout, design and construction procedures for CCLs in landfill barrier systems see Daniel, 1990; Rowe et al., 1995; ETC8, 1993; Manassero et al., 1998; Rowe, 2001. Rowe (2001) indicates that the construction of liner test pads prior to construction of the liner can ensure adequate construction practices without the risk of damage to the actual liner and enables the contractor to become acquainted with potential problems. They also provide a means of calibrating quality control and assurance procedures.

The service life of a clay liner is the period of time during which the bulk hydraulic conductivity of the liner may be expected to fall within the design range. Provided the liner is properly designed and constructed and appropriate attention has been paid to clay-leachate compatibility (e.g. Rowe et al., 1995), it is expected that it will perform within the range of design hydraulic conductivity for thousands of years (MoE, 1998).

1.6.3 Bentonite Enhanced Soils

In situations where suitable natural soils are not available for use in a compacted clay liner, bentonite may be added to a non-cohesive soil (e.g. silty sand) to achieve a liner with the required hydraulic conductivity. Rowe (2001) summarized the key considerations in the selection of bentonite and design of these mixed-soil liners, which are the grain size distribution of the base soil (Evans, 1991; Alston et al., 1997), the amount of bentonite (Kenney et al., 1991; Alston et al., 1997; Santucci de Magistris et al., 1998; Silvapullaiah et al., 2000), and the mineralogy of the bentonite.

Care is needed to examine hydraulic conductivities under the maximum gradient that could occur (e.g. with the development of a leachate mound) to ensure that there will not be an increase in hydraulic conductivity due to internal erosion of the bentonite (i.e. transport of the bentonite out of the liner by water flow; Edil and Erikson, 1984; Buettner, 1985).

Successful construction of low hydraulic conductivity soil -bentonite liners is highly dependent on: (a) obtaining and maintaining a homogeneous mixture of the base soil and bentonite and avoiding segregation prior to and during placement; (b) compaction and water content control (Kenny et al, 1991; Evan, 1991); and (c) lift thickness. A key consideration in the selection and design of soil-bentonite liners is the potential for chemical interaction between the clay and the fluid to be retained causing an increase in hydraulic conductivity (and hence a failure to fulfil its design function). Other important factors include the resistance to shrinkage cracking (Stewart et al., 1999; Kleppe and Olson, 1985) and the need for suitable mechanical properties for structural integrity during construction and operation. Although sand-bentonite specimens have been shown to be more resistant to freeze-thaw cycles than clay or till specimens in laboratory tests (Wong & Haug 1991), as with normal compacted clay liners, it is still prudent to protect the sand-bentonite liners against desiccation and frost.

1.6.4 Geosynthetic Clay Liners

Geosynthetic clay liners provide a convenient and potentially economical low permeability alternative to compacted clay liners both in covers and base liners in many situations. Due to the fact that it is a manufactured product, typically produced using either powdered or granular sodium bentonite, a high level of quality control can be achieved. The main advantages of GCLs are their limited thickness, improved resistance to differential settlement, ease of installation and low cost. The fact that they come in thin sheets that are seamed by overlapping does mean that considerable care is required during construction to avoid tearing the GCL sheets or opening the seams - especially when cover soil is being placed over the GCL.

The liner should be carefully installed in a manner that will avoid holes in the GCL. Shan and Daniel (1991) showed that GCLs have the capacity to effectively self-heal small holes but not large holes or tears. However, it should be recognized that this type of test does not simulate many practical situations. If a stone punches a hole in the GCL, the stone will likely stay in the hole. Secondly, the potential for puncturing will depend on the robustness of the cover geotextile and/or other material.

Additional considerations include the need to place GCLs on a prepared foundation layer, and the stability of liner systems involving GCLs and other geosynthetics such as side slope stability and interface strengths between geosynthetics and/or soil surfaces. Additional concerns associated with the installation of GCLs are their shear strength (Stark and Eid, 1996; Eid and Stark, 1997; Fox et al., 1998; Zornberg et al. 2004). Unreinforced GCLs typically exhibit low internal shear strength upon hydration, making them unacceptable for use on steeper slopes; and the construction requirements associated with their use. A database of 414 large-scale direct shear tests was assembled to evaluate variables governing geosynthetic clay liner (GCL) internal shear strength (Zornberg et al., 2004). Good repeatability of test results was obtained using same manufacturing lot GCL specimens, while comparatively high variability was obtained using different lot specimens. Peak shear strength variability was found to increase linearly with normal stress, but to be insensitive to specimen conditioning procedures. Evaluation of reinforced and unreinforced GCL test results indicates that, in addition to reinforcement variability, bentonite variability contributes to the shear strength variability of reinforced GCLs.

GCLs are subject to potential changes in hydraulic conductivity due to desiccation, however, preliminary work by Boardman and Daniel (1996) indicates that while the bentonite in GCLs did form open cracks in the tests upon drying, these cracks closed due to swelling upon re-wetting. While these results are encouraging, they were performed under a limited range of conditions and more field data is required to verify these findings. GCLs subjected to limited freeze-thaw conditions (Hewitt & Daniel 1997; Kraus et al. 1997) were found to perform well with no evidence of cracking and no significant increase in hydraulic conductivity was observed related to the freeze-thaw of the GCL sheets. Again, there is a need for more field data to verify these encouraging findings. Thus the, albeit limited, available evidence would suggest that GCLs may be preferable to compacted clay liners (CCLs) where the liner can not be protected from desiccation and freeze-thaw (e.g. in covers), Rowe (2001).

Owing to the relatively thin nature of GCLs, significant hydraulic gradients may occur due to ponding of leachate above the liner (a result of clogging of the collection system, discussed below), there is a potential for internal erosion and possible failure due to piping, thus a loss of bentonite from the GCL core. Preliminary results from current research addressing the issue of internal erosion of GCLs by assessing subgrade conditions conducive to piping failure indicate that care is needed in the selection of a suitable GCL product and that problems could occur with an inappropriate product over a severe (e.g. gravel) subgrade (Orsini and Rowe, 2001).

The hydraulic conductivity of a GCL must be assessed in the context of the expected field conditions, type of manufacturing process used, including consideration of hydration, confining stress, permeant characteristics and hydraulic gradient (Petrov and Rowe, 1996; Petrov et al., 1997a; Rowe, 1998b). Rowe (1998b) provided a detailed review of GCL hydraulic conductivity testing. GCL interaction with leachate (see Schubert, 1987; Shan and Daniel, 1991; Daniel et al., 1993; Dobras and Elzea, 1993; Ruhl and Daniel, 1997; Petrov et al., 1997 a,b; Petrov and Rowe, 1997) may increase the hydraulic conductivity of GCL a liner system, however this need not be a problem provided the design was based on the higher values that reflect interaction with leachate.

1.6.5 Geomembrane Liners

Geomembranes are planar, relatively impermeable polymeric sheets. Due to their low permeability, they make excellent liners for fluid retaining structures. There are many different types of geomembranes and the selection of a particular type depends upon the application in which it will be used. Peggs and Thiel (1998) proposed a selection approach based on the identification and the qualification of factors that affect service performance in intended applications of geomembranes by considering polymer performance, geomechanics, constructibility and project specific factors.

An intact geomembrane liner will experience some degradation with time that will lead eventually to its failure. The aging process of HDPE geomembranes can be envisioned as simultaneous combination of physical aging and chemical ageing (Hsuan and Koerner, 1995). For further discussion on physical and chemical ageing of geomembranes, see Petermann et al. (1976) and Schnabel (1981) respectively. Koerner et al. (1990) have provided a detailed description of the types of degradation to which HDPE geomembranes can be susceptible. This includes chemical degradation, oxidative degradation, degradation by swelling, degradation by extraction, biological degradation and UV degradation. Oxidative degradation appears to be the most harmful to HDPE geomembranes, which are the most common geomembranes used in landfill liner applications due to their superior chemical resistance.

Hsuan and Koerner (1995, 1998), Rowe (1998b) and Sangam (2001) reported that the service lives of HDPE geomembranes are essentially controlled by the antioxidants in the material and their service temperature. Most recently Sangam (2001) examined the service lives of HDPE geomembranes under various landfill liner conditions. It was estimated that for the particular geomembrane examined, provided that the landfill is well maintained such that the liner (i.e. geomembrane) temperature is not higher than 15^{o}C, the primary geomembrane would last at least 200 years whereas for the conditions where the temperature is at 33^{o}C (an increase in temperature as a result of leachate mounding, discussed below), the service life is estimated to drop to about 70 years. It was also estimated that geomembranes used as secondary liners will last at least 400 years for a temperature range typical of groundwater, 7 to 10^{o}C.

Concerns regarding the clogging of leachate collection systems (discussed below) have lead to the use of coarse drainage material in these systems. It is essential to ensure that the geomembrane is adequately protected against the potential detrimental effects of coarse gravel indenting the geomembrane, especially under the high overburden pressures in large landfills (Tognon et al., 2000). The development of tensile strains within the geomembrane due to impingement of the coarse drainage material may have serious implications on the service life of the geomembrane and may impair its primary design function as a barrier to advective flow due to the development of holes or tears under increased tensile stress.

1.6.6 In Ground Cut-off Barriers

There is a growing movement towards "containment" as a primary "remediation" strategy for many contaminated sites where alternative approaches have proven ineffective or are too expensive. This "containment" will frequently involve the construction of a vertical cut-off wall around part or all of the contaminated site and possibly (but rarely) a retrofitted "base liner" below the site (O'Donnell et al. 1995). Vertical cut-off walls may consist of slurry trench walls, geomembrane walls or sheet pile walls or a combination of these components.

A detailed discussion of the state-of-practice with respect to these walls is provided by O'Donnell et al. (1995). Daniel and Koerner (1995) provide a discussion of Construction Quality Assurance (CQA) / Construction Quality Control (CQC) for these systems. The key concerns with all of these walls are: (a) ensuring sufficient uniformity and quality of construction that leakage is controlled to an acceptably low level; (b) the difficulty of detecting small leaks; (c) long term durability (e.g. the potential effects of interaction with contaminants).

Slurry trench walls may be used to contain, capture and redirect flow of contaminated water, gases or free phase liquids. There are practical limits to the depths that can be achieved and the type of ground conditions in which they can be constructed. The short and long term hydraulic conductivity that can be achieved requires careful consideration. In particular, CQC and its effect on initial hydraulic conductivity, the effect of changes in groundwater condition and freeze-thaw (i.e. desiccation and cracking), the effect of interaction with the contaminants to be retained and the long term hydraulic conductivity, all need to be considered (Rowe, 2001). In addition, it must be recognized that even if there is no outward flow there is still potential for contaminants to diffuse across the barrier (Rowe, 1996).

Although there is extensive experience in the use of steel sheet pile walls in conventional geotechnical engineering applications, the use of this technique in geoenvironmental applications has been limited. This is primarily due to concerns regarding: (a) providing an interlock that will adequately control movement of contaminated fluids through the wall; and (b) the service life of the wall (i.e. the potential for corrosion in chemically aggressive environmental applications). As discussed in O'Donnell et al. (1995), sealable interlocks have been developed that use a sealant that is chemically compatible with the applicable environmental conditions. Resistance to corrosion can be provided by using: (a) thicker steel sections; or (b) sections with an organic coating (e.g. pitch that has been made more damage resistant than normal pitch by combining with vinyl or epoxy resins).

1.6.7 Landfill Drainage Layers

High transmissivity drainage layers may be used above liners to minimize the hydraulic head acting on the liner (and hence minimize flow through the liner). These drainage layers may be constructed from granular materials or geosynthetics or a combination of both. The maximum liquid thickness must be estimated for two reasons: (1) the liquid thickness is typically limited by regulations (e.g. the Resource Conservation and Recovery Act in the US requires a maximum liquid thickness of 0.3 m), and (2) good design requires that the liquid thickness be less than the thickness of the lateral drain (to avoid confined flow). Regardless of the shape of the liquid surface, the maximum liquid thickness in the liquid collection layer is given by Giroud et al. (2000a). The calculation of the maximum thickness of liquid in a liquid collection layer assumes the following conditions:

- the liquid supply rate is uniform (i.e. it is the same over the entire area of the liquid collection layer) and is constant (i.e. it is the same during a period of time that is long enough that steady-state flow conditions can be reached);
- the liquid collection layer is underlain by a geomembrane liner without defects and, therefore, liquid losses are negligible;

- the slope of the liquid collection layer is uniform (a situation referred to herein as "single slope"); and
- there is a drain at the toe of the slope that promptly removes the liquid.

There are many cases, in particular in landfills, when a liquid collection layer comprises two sections with different slopes. If there is a drain between the two sections, each section can be treated as a liquid collection layer on a single slope, using the method presented by Giroud et al. (2000a). However, there are cases where there is no drain removing the liquid at the connection between the two sections. Those cases are addressed in Giroud et al. (2000b). Also in this case, the determination of the maximum thickness of liquid is an essential design step because the maximum liquid thickness must be less than an allowable thickness.

Regulatory equivalency between natural and geocomposite lateral drainage systems is currently based on equivalent transmissivity. However, Giroud et al. (2000c) have demonstrated that this practice is incorrect and non-conservative. An equivalency based solely on transmissivity will lead to selection of a geosynthetic drainage layer that may not provide adequate flow capacity and may result in the development of water pressure.

Typically the design incorporates a hydraulic conductivity of 10^{-5}m/s. While this may provide the required drainage immediately after construction, a reduction in hydraulic conductivity due to biological, chemical or particulate clogging (Brune et al., 1991; Paksy et al., 1998; Rowe et al., 1997 a,b; Fleming et al., 1999; Peeling et al., 1999; Rowe et al., 2000 a,b) may quickly result in an excessive leachate mound. A result of leachate mounding is an increase in base liner temperature, which may have detrimental effects on the service life of geomembranes (discussed above), the potential of bentonite loss in GCLs due to high hydraulic gradients (discussed above) and increase the diffusive transport of compounds through barrier liners (Barone et al., 1997, 1999;Rowe 1998b).

Special care is required to ensure adequate long-term drainage capacity. Geotextiles are often used as filters between the waste and the drainage layer - especially when either coarse drainage materials (e.g. gravel) or geonets are used to provide a drainage blanket. Key issues in the design of these systems are the need to provide adequate drainage, prevent structural failure (e.g. crushing, or other pipe failure, Moore, 1993, Brachman et al., 2000 a,b, Brachman et al., 2001) and to minimize clogging. There is a growing body of evidence that the performance of these systems can be greatly impaired by clogging and a number of failures have been reported in the literature (see Rowe 1998 a,b for a review of clogging of geotextiles and granular material respectively).

There has also been considerable debate regarding the use of carbonate drainage stone due to the potential for dissolution of the rock by leachate. Bennett et al. (2000) addressed this issue for a landfill in Toronto and showed that for this case carbonate containing stone is an adequate drainage material and that dissolution need not be a concern over the range of pH encountered. The clogging problem arises because municipal solid waste leachate contains nutrients that will encourage bacterial growth (see Rittmann et al., 1996 for a description of leachate chemistry and implications on clogging) in geotextile filters, in granular drainage layers, around the perforations in the leachate collection pipes and within the pipes (Fleming et al., 1999).

Clogging of the leachate collection system involves the filling of the void space between solid particles as a result of a combination of biological, chemical and physical events; particulate clogging, due to the migration of fines from within the waste mass (may be reduced by placement of a suitable geotextile between the waste and transmissive layer) or construction practices. However, a major component of the clogging is microbiologically related. Rowe and Fleming (1998) provide a simple clogging model that could be used to assess the service life of the collection system based on field and laboratory studies. A more sophisticated model is subsequently being developed to address the microbial processes and the development of clog material within the medium, thus decrease in hydraulic conductivity with time for various collection system designs and leachate composition (Rowe et al., 1997c; Cooke et al., 1999, 2000).

1.7 QUALITY ASSURANCE AND CONTROL

1.7.1 Principles

The purpose of Quality Assurance (QA) and Quality Control are to ensure that the quality of the overall structure of a landfill or the implementation of a remediation project, and the individual components of these meet the required quality standards. QA and QC must relate to both the quality of the materials used and to the quality of the workmanship in accordance with the existing state of technology.

EPA 1993 defines the following terms:

- *Manufacturing Quality Assurance (MQA):* A planned system of activities that provides assurance that the materials were constructed as specified in the certification documents and contract plans. MQA includes manufacturing facility inspections, verifications, audits and evaluations of the raw materials and geosynthetic products to assess the quality of the manufactured materials.
- *Manufacturing Quality Control (MQC):* A planned system of inspections that is used to directly monitor and control the manufacture of a material which is factory originated. MQC is normally performed by the manufacturer of geosynthetic materials and is necessary to ensure minimum (or maximum) specified values in the manufactured product.
- *Construction Quality Assurance (CQA):* A planned system of activities that provides the owner and permitting agency assurance that the facility was constructed as specified in the design. CQA includes inspections, verifications, audits and evaluations of materials and workmanship necessary to determine and document the quality of the constructed facility.
- *Construction Quality Control (CQC):* A planned system of inspections that is used to directly monitor and control the quality of a construction project. CQC is normally performed by the geosynthetics installer or for natural soil materials by the earthworks contractor, and is necessary to achieve quality in the constructed or installed system.

In any quality scheme it is essential that definitions (not just those given above) are agreed between the various parties involved. This is particularly relevant where site specific words, phrases or terms may apply.

An organisational chart of MQA/CQA and MQC/CQC activities is also given in EPA (1993) and reproduced here as Figure 6. This chart was devised primarily for use in respect of waste containment but can equally be applied to the remediation/containment of contaminated land. MQA and CQA are normally carried out independently from MQC and CQC. The following text primarily concerns CQA and CQC.

1.7.2 Quality Assurance Plan(s)

As part of the technical design, the scope of the Quality Assurance Plan(s) must be drawn up. This may be a combined plan for manufacture and installation/construction or may be a number of separate plans. The plan(s) should include requirements concerning the materials and methods to be used and the type and frequency of the tests and checks that are to be carried out. The plan may need to be modified as the project proceeds provided that the modifications are justified.

Evidence of suitability should be produced for all materials and methods. Before construction or remediation begins, the suitability of the materials, equipment and methods to be used should be tested under field conditions. The results of the suitability tests should be adopted as reference values in the Quality Assurance Plan for the construction work.

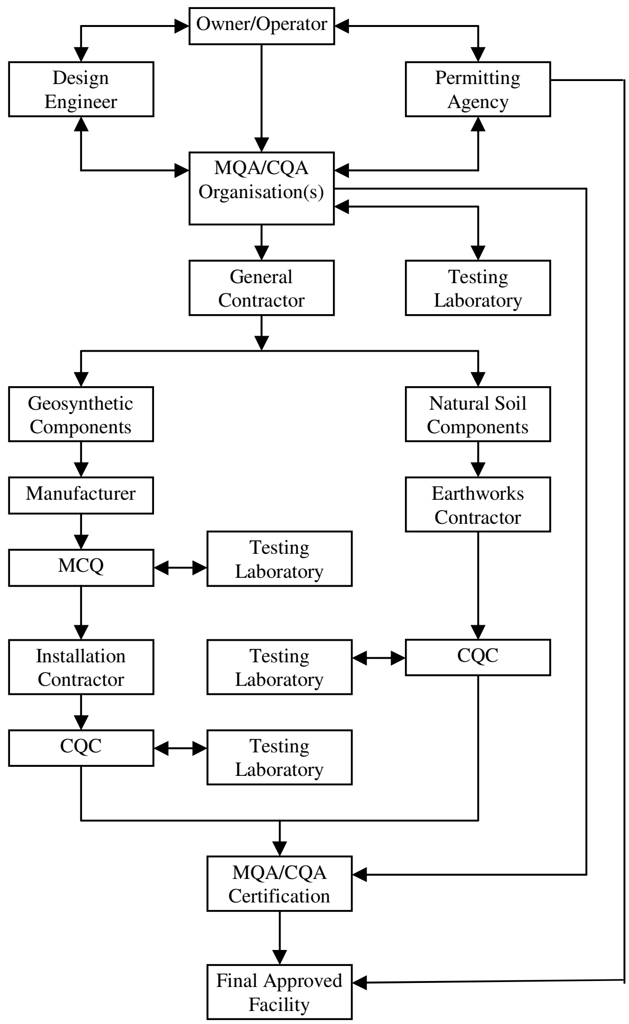


Figure 6. Organisational Structure of MQA/CQA and MQC/CQC Inspection Activities (EPA, 1993)

The testing requirements in the plan should comprise:

- in-house testing by the contractor
- external testing by an independent party

If appropriate, the regulatory authority may also request that testing be carried out as a further independent check.

All testing should be supervised by suitably qualified geoenvironmental specialists with sufficient knowledge in landfill construction or geotechnical construction or remediation processes as appropriate.

The following should be specified in the plan:

- responsibilities and tasks of the supervisors;
- description of the construction or treatment, stating the processes to be inspected;
- type and number of quality tests to be undertaken on the materials supplied (initial tests), on their processing (processing tests) and on the completed components (commissioning tests),
- format and degree of detail required for the reporting of results and inspections.

In some countries the QA plans have to be submitted to the permitting agency as part of the regulatory process and can form part of the permit application. They are a means of the owner/operator/developer demonstrating to the permitting agency that appropriate quality objectives for the project will be met.

1.7.3 QA Personnel

QA should be carried out by an independent organisation from the client and the contractor so that responsibilities are clearly defined and an independent compliance document can be prepared on completion of the works.

The staff should be suitably qualified and should be experienced in this type of work. It is essential that they are appropriately briefed prior to the commencement of the works and are aware of both the design requirements and the overall project goals. Their duties should be clearly defined in writing including types and frequencies of tests, range of responsibility, extent of authority on site, and frequency of reporting. There should be complete familiarity with the testing equipment to be used, including calibration checks and any factors that could lead to inaccuracy in the test results.

The personnel must be fully familiar with the acceptance criteria in respect of the materials to be used. Any contravention of these acceptance criteria should be noted at the time they occur and any corrective measures implemented immediately. An excuse that carrying out such corrective measures would delay the project should never be allowed to prevent appropriate procedures from being followed. There are numerous references in the literature where the cost of subsequent remedial measures has far outweighed the cost of taking action at the time of construction or remediation.

The QA personnel should understand:

- the influence of the surrounding environment on the construction or remediation process from the point of view of hydrology, hydrogeology, geotechnics and meteorology.
- the contractual relationships between the parties.
- the principles of the design of the landfill or remediation process.
- The significance of accurate and systematic record keeping including as-built drawings.
- The need for effective verbal and written communication at all times together with agreed channels for communication.

In addition to the QA personnel responsible for making observations, performing field tests and keeping detailed records, there should be a QA Certifying Engineer. This person is responsible for certifying to the owner and/or operator and permitting agency that they consider the facility to have been constructed (in the case of landfill) or the operation to have been carried out (in the case of remediation) in accordance with the plans and the specification.

The Certifying Engineer may also fulfil other roles in the QA team but must be a registered professional engineer with experience in certifying like installations or operations.

1.7.4 QC Personnel

MQC and CQC personnel are normally employed by the manufacturer or contractor respectively to ensure that the manufacture and construction are taking place in accordance with the approved contract plans and specifications. In some cases, MQC and CQC personnel can be from a separate organisation retained by the manufacturer or contractor.

Many of the comments made above in respect of QA personnel also apply to QC personnel.

1.7.5 QA for Landfill Containment

1.7.5.1 General

QA should be applied to all of the components of landfill containment (subgrade, liner, protective layers, drainage system, gas-venting system and any transitional layers).

The long term performance of a containment system is very dependent on the effectiveness of the liner. However, even though the materials used for liner construction have in some cases been suitable, their use in constructing the containment has been less than satisfactory. For example, there have been cases where for clay liners or the mineral component of composite liners, preconstruction laboratory testing has shown that the required limit on hydraulic conductivity can be achieved but the placement in the field has resulted in a non homogeneous liner with poor compaction. In this situation the liner is just a series of 'clods' of clay which would permit flow paths between them and hence produce poor liquid retention properties.

QA includes; an initial review of the pre-contract laboratory testing and the specification (to ensure that the means of exercising proper control during the works are available), the supervision of a field trial prior to the start of liner construction, carrying out of tests on the placed liner materials during the works and the supervision of the taking of samples for subsequent laboratory testing.

In many countries QA has only recently been implemented in respect of the construction of landfill containments and perhaps in some countries has not been implemented at all. Where it has, there is already evidence that a much better quality of containment has been achieved. Also for a large proportion of the sites that have been lined, but nevertheless leak more than would be expected, poor construction practices have usually been found to be the cause.

For geomembrane liners there are now a number of geophysical techniques available to assist in the QA process. These are normally used once the liner has been completed or a certain section of liner has been completed and rely on the principle of electrical conductivity. On a number of projects these techniques have been able to detect even pin-hole size defects in the membrane. Sealing of these in addition to those found by visual inspection or seam testing has the potential to considerably improve the competence of the completed liner.

1.7.5.2 Subgrade

The subgrade has to satisfactorily support the liner system. The suitability and settlement behaviour of the in situ soil, and of the waste in the case of the capping seal, must be determined and taken into consideration in the design. If necessary, investigations should be specified in connection with QA. The following should be demonstrated by means of in-house and external testing:

- adequate bearing capacity of the subsoil surface or the waste body surface;
- satisfactory settlement characteristics;
- adherence to allowable tolerances in respect to evenness of the subgrade and adherence to design levels and dimensions.

1.7.5.3 Liner

The following range of tests will normally be required:

- characteristics of the materials to be used, including grain-size distribution, shear strength, index properties and moisture content;
- moisture content on placement, homogeneity of the material placed, number of passes with the roller, quantity of water added, if any;
- minimum clod size, cutting depth and quantity of additives or dosage in the case of multiple component mixtures;
- thickness of the individual lifts and adherence to proposed levels and dimensions;
- degree of compaction and homogeneity achieved in each lift by determinations of density, moisture content, grain-size distribution and plasticity;
- determination of the hydraulic conductivity of the sealing layer for each lift.

Random samples of the selected materials should be taken at source, on delivery to site and again on placement. The test results should be compared with the suitability test data (acceptance criteria). In addition, it is necessary to ascertain whether the soil supplied is sufficiently uniform. The thickness of each lift should be determined before and after compaction.

The clod size of cohesive sealing materials should be checked in order to achieve a homogeneous sealing layer. As stated above, compliance with specified density and moisture content values is not, by itself, sufficient.

In cases where additives are used, mixed either with the in situ soil or with an imported soil, the quantity and even distribution of the additives should also be checked by means of a grid system. Spacing should be determined for individual tests by reference to the spreading equipment used. During compaction of such materials the cutting depth and homogeneity of the mix should be checked. The cutting depth should be sufficient (a minimum of 3 cm) to ensure bonding into the upper zone of the underlying lift.

When construction commences, a relationship should be established between compacted lift thickness and cutting depth on the basis of a field trial. Where varying moisture contents can occur, this measurement should be related to the range of moisture content anticipated. Adherence to the required moisture content, together with any measures required to achieve it, should be checked.

Each sealing lift should be properly compacted. During compaction the number of passes with the roller, by reference to data from the field trial, and the uniformity of compaction should be checked. To determine the degree of compaction non-destructive testing should be undertaken. Compaction tests should be undertaken on all lifts of the sealing layer.

If a nuclear probe is used, where the results may be influenced by the type of mineral, the instrument should be calibrated by comparison with data from a sufficient number of density tests (by means of sand replacement or undisturbed sampling).

Moisture content should be checked during construction. It should be determined by oven drying, an in some cases by using a microwave. The nuclear probe can also be used if calibrated regularly.

Hydraulic conductivity can be determined either in the laboratory or in the field provided an appropriate technique is available. The hydraulic conductivity results should be compared with target values related to the test method.

A method should be found which prevents unacceptable delay to the construction work. Hydraulic conductivity tests may not be required prior to acceptance provided other test results relating to QA, particularly grain-size distribution, moisture content and dry density, correspond to data from the suitability tests. Hydraulic conductivity tests are then undertaken for record purposes only.

If weaknesses in the seal are identified from the measuring grid, additional measuring locations should be specified in order to delimit and improve the lower quality zones. A method of backfilling and sealing all sampling locations in the completed liner must be agreed in advance of the construction work to ensure the integrity of the seal.

1.7.6 QA for Contamination Remediation

1.7.6.1 General

All aspects of a remediation project require some form of QA procedures to be applied. The most important components are as follows:

- Elimination of potential for adverse impact on humans, structures or the environment as a result of the remedial operations
- Achievement of target concentrations for contaminants in water and soil
- Removal of potential for recontamination of water and soil
- Effectiveness and durability of solidification and stabilisation processes for immobilising contaminants
- Competence and durability of capping layers and in-ground barriers.
- Long term monitoring

Many of the above involve earthworks or standard geotechnical processes such as the construction of slurry trench walls in which case the QA requirements are adequately documented elsewhere. In the case of caps reference should be made to Section 1.7.5 above.

In respect of the operations which are specific to remediation, many of the QA procedures will be similar to those described above, e.g. documentation, daily observations and inspections, etc. However, a particular aspect of remediation is the need to demonstrate that the contaminant concentrations have been reduced to the specified level. This can be problematic as indicated below.

1.7.6.2 Verification of Achievement of Target Concentrations

Demonstration that contamination has been reduced to the target concentrations stipulated in the design can be influenced by sampling and analytical errors and inconsistencies. It is only by understanding and controlling these errors that the effectiveness of a treatment can be determined.

Many site investigations concentrate on 'hot spots' and hence they do not properly characterise the materials in such a way that this relates to the clean up criteria. For example, many clean up criteria will refer to bulk contaminant concentrations that have to be achieved, which means taking large bulk samples, sub sampling them and then determining overall chemical concentrations including course material. The site investigation may only have preferentially sampled the fine material.

Sampling methods can have a considerable influence on contaminant concentrations and have been known to cause at least a five fold difference in results. Therefore, the same sampling methods should be used before and on completion of remediation. Any regulatory requirements should also sufficiently stipulate sampling methods. Unfortunately there is relatively little literature on quality of sampling and the uncertainties that can occur.

There is far more literature concerning analytical variability. Sub-samples of a sample sent to different laboratories can produce quite variable results. This can be due to different storage conditions, different sample preparation methods prior to test, different protocols for testing for the same analyte, different methods of reporting or the QA personnel inadvertently specifying different tests, e.g. there are many different tests for the presence of hydrocarbons each analysing different characteristics.

Reproducibility of test results is part of the above problem. This can be in the form of variations between replicate determinations on samples analysed:

- as a batch (same equipment, same operator, same method, same conditions, same day, etc.).
- at the same establishment (same method but possibly different equipment, different operator, different day, etc.).

- at different establishments.

It is necessary for personnel carrying out QA to understand these differences and to decide what levels of uncertainty are justifiable and acceptable. Bearing in mind the different perceptions and needs of the various parties that may be involved in a remediation, what is acceptable to one may not be acceptable to another.

1.7.7 Final Certificate and Validation Report

For a project or section of a project, the final QA Certification Statement is normally accompanied by a final QA Validation Report. In some cases the final report may be deemed to fulfil both requirements. This will depend upon the local regulations. Also there may be separate certification statements and reports for MQA and CQA.

The report(s) should contain all the appropriate documentation, including daily observation and inspection reports, sampling locations, test results, photographs, drawings, sketches and any other relevant data which is needed to provide a complete record of the work and demonstrate that the required degree of quality has been achieved. Any design changes during the work should also be documented in the report.

REFERENCES

Alston, C., Daniel, D.E. and Devroy, D.J. (1997). Design and construction of sand-bentonite liner for effluent treatment lagoon, Marathon, Ontario. Canadian Geotechnical Journal, Vol. 34, pp. 841-852.

ASTM. 1995. ASTM Standards on Geosynthetics. Sponsored by ASTM Committee D-35 on Geosynthetics, Fourth Edition, 178p.

August H & Lûders G : 1997 : Investigation into the long-term integrity of protective materials for geomembranes in landfill basal liners. Advanced Landfill Liner Systems. Ed by H August, U Holzlöhmer and T Meggyes. Thomas Telford.

August H & Tatzky R : 1984 : Hydraulic conductivity of commercially available polymeric liners for hazardous landfill leachate organic constituents. Proceedings of the International Conference on FMLs, Vol 1, Denver, USA, IFAI, Minnesota pp 163-8.

Barone, F.S., Costa, J.M.A. and Ciardullo, L. (1997). Temperatures at the base of a municipal solid waste landfill. 50th Canadian Geotechnical Conference, Ottawa, October, Vol. 1, pp.144-152.

Barone, F.S., Costa, J.M.A. and Ciardullo, L. (2000). Temperatures at the base of a municipal solid waste landfill. In Proceeding of the CSCE 2000 Conference, London, Ontario, pp. 41-48.

Bass, J M Lyman, W J & Tratnyek, J P (1985) Assessment of synthetic membrane successes and failures at waste storage and disposal sites. USEPA, EPA/600/2-85/100.

Baykal G, Kavak A & Alpatli M : 1993 : Rubber - kaolinite and rubber - bentonite liners. Green 93 - Waste Disposal by Landfill, Bolton, UK.

Bennett, P.J., Longstaffe, F.J. and Rowe, R.K. (2000). The stability of dolomite in landfill leachate collection systems. Canadian Geotechnical Journal, 37: 371-378.

Benson C H & Daniel D E : 1990 : Influence of clods on hydraulic conductivity of compacted clay. Journal of the Geotechnical Engineering Division, ASCE Vol 116, No. 8.

Benson C H, Hardianto F S & Motan E S (1994) Representative specimen size for hydraulic conductivity assessment of compacted soil liners. Hydraulic Conductivity and Waste Contaminant Transport in Soil. ASTM, SPT 1142, Philadelphia, USA.

Bernhard C, Gouse J P, Le Tellier I & Matichard Y (1994) Compared performance of bottom and slope lining systems of municipal landfills. First International Congress on Environmental Geotechnics, Edmonton, Canada.

Bieberstein A & Saucke U (2002) The Karlsruhe cut-off wall testing unit (KTU) for monitoring in-situ permeability. 4th International Congress on Environmental Geotechnics, Rio de Janeiro.

Boardman, B.T. and Daniel, D.E. (1996). Hydraulic conductivity of desiccated geosynthetic clay liners. Journal of Geotechnical Engineering, ASCE, Reston, VA, Vol. 122, No.3, pp.204-208.

Boels D & Wiebing R : 1990 : Durability of sand bentonite liners in surface cappings of waste disposal sites. The Winand Staring Centre for Integrated Land, Soil and Water Research, Netherlands, Report 42.

Bonaparte R & B.A. Gross (1990) Field behaviour of double liner systems. Waste Containment Systems : Construction, Regulation and Performance, R Bonaparte, ed., ASCE Geotechnical Special Publication No 26, USA.

Bouazza, M., Zornberg, J.G., and Adam, D. (2002). "Geosynthetics in Waste Containment Facilities: Recent Advances." Keynote paper, Proceedings of the Seventh International Conference on Geosynthetics, Nice, France, 22-27 September, A.A. Balkema, Vol. 2, pp. 445-510.

Bowders J J, Daniel D E, Broderick G P & Liljestrand H M (1986) Methods for testing the compatibility of clay liners with landfill leachate. Hazardous and Industrial Solid Waste Testing: Fourth Symposium. Petras J K, Lacy W J and Conway RA Eds., ASTM Special Technical Publication 886.

Brachman, R.W.I., Moore, I.D. and Rowe, R.K. (2000b). Local strain on a leachate collection pipe. Canadian Journal of Civil Engineering. Vol. 27, No. 6, pp. 1273-1285.

Brachman, R.W.I., Moore, I.D., Rowe, R.K. and Tognon, A.R. (2001) Influence of coarse gravel on deflection of landfill leachate collection pipe.

Brachman, R.W.I., Rowe, R.K., and Moore, I.D. (2000a). Drainage pipes for landfill leachate collection systems. 6th Canadian Environmental Engineering Conference, 2000, London, Ontario, pp. 82-88.

Bratley P (1983) A Guide to Simulation. Springer - Verlag

Brune M, Ramke H G, Collins H J & Hanert H M (1991) Encrustation processes in drainage systems of sanitary landfills. Proceedings Third International Landfill Symposium, Cagliari, Italy.

Brune, M., Ramke, H.G., Collins, H., and Hanert, H.H. (1991). Incrustations process in drainage systems of sanitary landfills. Proc. 3rd International Landfill Symposium, Cagliari, Italy, pp. 999-1035.

Budmian J & W.C. Mitts (1992) Effects of temperature cycle on embrittlement of FML. Proc. of the Mediterranean Conference on Environmental Geotechnology, Cesme, Turkey.

Buettner, W.G. (1985) Permeability Testing of Soils With Low Hydraulic Conductivity. M.Sc. Thesis, The University of Saskatchewan, 203 pp.

Cadwallader M & Dixon E (1988) Liner materials for waste containment. Pollution Engineering, Vol 20 Part 7 pp 70-3.

Cazzuffi D, Cossa R, Ferruti L & Lavagnolo C L (1991) Efficiency of geotextile and geocomposites in landfill drainage systems. Proc. Third International Landfill Symposium, Cagliari, Italy.

Chapuis R P (1990) Sand bentonite liners - field control methods. Canadian Geotechnical journal, Vol 27 pp 216-23.

Clark J R (1993) The behaviour of bentonite enriched soils and their use in landfill lining systems. Conference Proceedings for the Engineering Geology of Waste Storage and Disposal. Geological Society, Cardiff.

Clem A G (1985) Factors influencing the stability of smectite sealants. Engineering Geology, Vol 21 pp 273-8.

Cooke, A.J., Rowe, R.K., Rittmann, B.E., and Fleming, I.R. (1999). Modelling biochemically driven mineral precipitation in anaerobic biofilms. Water Science and Technology, Vol. 39, No. 7, pp. 57-64.

Cooke, A.J., Rowe, R.K., Rittmann, B.E., VanGulck J., and Millward, S. (2000). Biofilm growth and mineral precipitation in synthetic leachate columns, ASCE Journal of Geotechnical and Geoenvironmental Engineering (Tentatively accepted 7/8/00).

Copeland WB, Luscher U & Hughes E (1996) The San Leandro plume – A regional study of groundwater contamination. Second International Congress on Environmental Geotechnics, Osaka, Japan.

Cossu R, Ranieri G & Motzo GM (1992) Geophysical methods in surveying and monitoring old landfills. Mediterranean Conference on Environmental Geotechnology, Cesme, Turkey.

Cossu R, Ranieri G, Marchisio M, Sambuelli L, Godio A & Motzo GM (1990) Geophysical methods in surveying old landfills. Third International KfK/TNO Conference on Contaminated Soils, Karlsruhe, Germany.

Costa RGS, Silva Jr GC, Moura JRS and Monteiro AC (2002) Evaluation and monitoring of Bangu waste disposal site, RJ, Brazil. Fourth International Congress on Environmental Geotechnics, Rio de Janeiro, Brazil.

Cowland J W & Leung B N (1991) A field trial of a bentonite landfill liner. Waste Management and Research, Vol 9 pp 277-91.

Daniel D E & Bowders J J (1996) Waste containment systems by geosynthetics. Second International Congress on Environmental Geotechnics, Osaka, Japan.

Daniel D E (1993) Pollution prevention in landfills using engineered final covers. Waste Disposal by Landfill. Green '93, Sarsby (ed), Balkema, Rotterdam.

Daniel, D.E. (1990). Impervious barriers for pollutant containment. Proceedings, Seminar on Impervious Barriers for Pollutant Containment, Italian National Electricity Board-Research Centre, Milano, Italy.

Daniel, D.E. and Koerner, R.M. (1995). Waste containment facilities: Guidance for construction, quality assurance and quality control of liner and cover systems. ASCE, New York.

Daniel, D.E., Shan, H.Y. and Anderson, J.D. (1993). Effects of partial wetting on the performance of the bentonite component of a geosynthetic clay liner. Proceedings, Geosynthetics '93, IFAI, St. Paul, Minn., Vol. 3, pp. 148-1496.

Daniel D E & Koerner R M (1993) Final cover systems. Chapter 18 in Geotechnical Aspects of Waste Disposal, Daniel D E (ed) Chapman and Hall, London.

Department of the Environment, UK (1995) Waste Management Paper 26B. Landfill Design, Construction and Operational Practice. HMSO, London.

Dobras, T.N. and Elzea, J.M. (1993). In-situ soda ash treatment for contaminated geosynthetic clay liners. Proceedings, Geosynthetics '93, IFAI, St. Paul, MN, pp. 1145-1160.

Döll P, Renger M & Plagge R (1997) Non-isothermal water and vapour movement below landfills: laboratory experiments on and computer simulations of desiccation in mineral lines. Advanced Landfill Liner Systems. Ed by H Augus, U Holzlöhmer and T Meggyes. Thomas Telford.

Edil, T.B. and Erickson, A.E. (1984) Procedure and equipment factors affecting permeability testing of a bentonite-sand liner material. Hydraulic Barriers in Soil and Rock, ASTM STP 874, pp. 155-170.

Edwards DA et al. (1997) Development of Fraction Specific Reference Doses (RfDs) and Reference Concentrations (RfCs) for Total Petroleum Hydrocarbons (TPH). Vol 4 TPHCWGS, Amherst Scientific Publishers.

Eid, H.T. and T.D. Stark. (1997). Shear behavior of an unreinforced geosynthetic clay liner. Geosynthetics International, Vol. 4, no. 6, pp. 645-659.

Eith A W, Boschuk J & Koerner R M (1991) Prefabricated bentonite clay liners. Geotextiles and FMLs, Vol 10, pp 575-99.

EPA (1993) Technical Guidance Document. Quality Assurance and Quality Control for Waste Containment Facilities. EPA/600/R-93/182. Environmental Protection Agency.

ETC 8 - European Technical Committee No. 8. (1993). Geotechnics of landfill design and remedial works technical recommendations – GLR, 2nd Edition, Ernst and Sohn.

Evans, E.A. (1991) Municipal waste leachate and organic liquid effects on the hydraulic conductivity of sand-bentonite mixtures. M.E.Sc. Thesis, University of Western Ontario, London, Ontario.

Fang H Y (1995) Bacteria and tree root attack on landfill liners. Waste Disposal by Landfill . Green '93, Sarsby (ed), Balkema, Rotterdam.

Favaretti M, Moraei N & Previatello P (1994) Effects of leachate on the hydraulic and mechanical behaviour of clay liners. First International Congress on Environmental Geotechnics, Edmonton, Canada.

Fleming, I.R., Rowe, R.K., and Cullimore, D.R. (1999). Field observations of clogging in a landfill leachate collection system. Canadian Geotechnical Journal, 36(4), pp. 289-296.

Fox, P.J., M.G. Rowland and J.R. Scheithe. (1998). Internal shear strength of yhree geosynthetic clay liners. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol 124, No. 10, pp. 933-944.

Freeze RA, Massmann J, Smith L, Sperling T & Jones B (1990) Hydrogeological decision analysis, a framework. Ground Water, Vol 28, No.5.

Garlanger J E, Chenug F K, & Tannous B S (1987) Quality control testing for a sand-bentonite liner. Geotechnical Practice for Waste Disposal 87 Proc Speciality Conf ASCE Spec Publ 13, pp 488-91.

Giroud J P & Bonaparte R (1989) Leakage through liners constructed with FMLs. Parts I and II, Geotextiles & FMLs vol 8 No. 1 pp 27-67 and vol 8 No. 2 pp 71-111.

Giroud J P, Badu - Tweneboah K & Soderman K L (1994) Evaluation of landfill liners. Proceedings 5th International Conference on Geotextiles, FMLs and Related Products, Singapore.

Giroud, J.P., Zhao, A. & Bonaparte, R. (2000c). The myth of hydraulic transmissivity equivalency between geosynthetic and granular liquid collection layers. Geosynthetics International, Special Issue on Liquid Collection Systems 7(4-6): 381-401.

Giroud, J.P., Zornberg, J.G. & Beach, J.F. (2000b). Hydraulic design of geosynthetic and granular liquid collection layers comprising two different slopes. Geosynthetics International, Special Issue on Liquid Collection Systems 7(4-6): 453-489.

Giroud, J.P., Zornberg, J.G. & Zhao, A. (2000a). Hydraulic design of geosynthetic and granular liquid collection Layers. Geosynthetics International, Special Issue on Liquid Collection Systems 7(4-6): 285-380.

Goldman J (1992) Detection and monitoring of environmental contamination by remote sensing. Mediterranean Conference on Environmental Geotechnology, Cesme, Turkey.

Gottheil KM & Brauns J (1997) Thermal effects on the barrier efficiency of composite liners - test field measurements. Advanced Landfill Liner Systems. Ed by H August, U Holzlöhner and T Meggyes. Thomas Telford.

Gourc JP, Thomas S & Vuillemin M (1998) Proposal of a waste settlement survey methodology. Third International Congress on Environmental Geotechnics, Lisbon, Portugal.

Grisolia M and Napoleoni Q (1996) Geotechnical characterization of municipal solid waste: Choice of design parameters. Proceedings of the Second International Congress on Environmental Geotechnics. Osaka, Japan.

Grisolia M, Napoleoni Q & Tancredi G (1995) Contribution to a technical classification of municipal solid waste. Proceedings of the Fifth International Landfill Symposium – Sardinia '95, Cagliari.

Gross B A, Bonaparte R & Giroud J P (1990) Evaluation of flow from landfill leakage detection systems, Proc. 4th Int. Conf. on Geotextiles and FMLs. The Hague, vol 2, pp 481-486.

Gustafson BJ, Griffith J & Orem D (1997). Selection of Representative TPH Fractions Based on Fate and Transport Considerations. Vol 3 TPHCWGS, Amherst Scientific Publishers.

Haxo H E & Waller M J (1987) Laboratory testing of geosynthetics and plastic pipe for double liner systems. Proc. of the Geosynthetics 87 Conference, New Orleans, USA Vol. 2 pp 577-94.

Haxo H E, Haxo P D, Nelson N A, White R M & Dakessian S (1987) Liner waste compatibility studies for coal Fired Power Plants.

Haxo H E, Miedema J A & Nelson N A (1984) Hydraulic conductivity of polymeric membrane lining materials. Proceedings of the International Conference on FMLs, Vol 1, Denver, USA, IFAI, Minnesota pp 151-6.

Heibrock G & Jessberger H L (1995) Safety Analysis of a composite liner system. Proceedings "Sardinia '95", 5th International Landfill Symposium, Vol 2, Cagliari, Italy.

Hewitt, R. and Daniel, D. (1997). Hydraulic conductivity of GCLs after freeze-thaw. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 123, pp. 305-313.

Hoeks J, Glas H, Hofkamp J & Ryhiner A H (1987) Bentonite liners for isolation of waste disposal sites. Waste Management and Research, Vol 5 pp 93-105.

Holzlohmer U, August H, Meggyes T & Brune M (1995) Liner landfill systems. A State of theArt Report. Penshaw Press, Sunderland, UK.

Holzlöhner U (1997) Mechanical loading on earthen liners, damage due to subsidence. Advanced Landfill Liner Systems. Ed by H August, U Holzlöhner and T Meggyes. Thomas Telford.

Holzlöhner U, Schossig W, Wuttke W & Ziegler F (1997) Long-term behaviour of earthen layers in landfill basal liners, Moisture Balance under Thermal Effects. Advanced Landfill Liner Systems. Ed by H August, U Holzlöhner and T Meggyes. Thomas Telford.

Hsuan Y.G. and Koerner R.M. (1995). Long Term Durability of HDPE Geomembrane: Part I – Depletion of Antioxidant, GRI Report 16, 35p.

Hsuan Y.G. and Koerner R.M. (1998). Antioxidant depletion lifetime in high density polyethylene geomembranes. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 532-541.

Imamura S, Sueuoka T & Kamon M (1996) Long term stability of bentonite/sand mixtures at LLRW storage. Second International Congress on Environmental Geotechnics, Osaka, Japan.

ISO (1987) Quality Systems. ISO 9000. International Standards Organisation.

Jaggy M (1996) Risk analysis of landfills in Gheorghe, A.V. (Editor) Integrated Regional Health and Environmental Risk Assessment and Safety Management. Published in International Journal of Environment and Pollution, Vol 6, Nos. 4-6.

Jessberger H L & Stone K (1991) Subsidence effects on clay barriers. Geotechnique Vol 41.

Jessberger H L (1994) Emerging problems and practices in environmental geotechnics. Proceedings 13th ICSMFE, New Delhi.

Jessberger HL & Heibrock G (1997) Development of a safety concept for landfill liner systems. Advanced Landfill Liner Systems. Ed. by H August, U Holzlöhner and T Meggyes. Thomas Telford.

Kalos M H & Whitlock P A (1986) Monte Carlo Methods, Vol 1, Basics, Wiley.

Kenney, T.C., Veen, W.A., Swallow, M.A. and Sungaila, M.A. (1991). Hydraulic conductivity of compacted bentonite-sand. Proceedings, 44th Canadian Geotechnical Conference, Calgary, Alberta, pp. 63-1 – 63-10.

Kent R, McMurtry D and Bentley M (1985) Groundwater monitoring, control and analysis. Water Resources Symposium No. 12. Published by the Centre for Research in Water Resources, Austin, Texas, USA.

King K S, Quigley R M, Fernandez F et al. (1993) Hydraulic conductivity and diffusion monitoring of the Keele Valley Landfill Liner, Maple, Ontario. Canadian Geotechnical Journal 30.

Kleppe, J.H. and Olson, R.E. (1985). Desiccation cracking of soil barriers. ASTM STP 874, pp. 263-275.

Koener G R & Koerner R M (1990) Biological activity and potential remediation involving geotextile landfill leachate filters. ASTM Special Technical Publication N.1081.

Koerner R M (1994) Designing with geosynthetics, 3rd Ed., Prentice Hall, New Jersey, USA.

Koerner R M, Lord A E & Hsuan Y H (1992) Arrhenius modelling to predict geosynthetic degradation. Geotextiles and FMLs II.

Koerner R.M., Halse Y.H. and Lord, A.E. Jr. (1990). Long-term durability and aging of geomembrane. In Waste Containment Systems: Construction, Regulation, and Performance. ASCE Geotechnical Special Publication No. 26, R. Bonaparte, Ed., New York, pp. 106-134.

Kraus, J., Benson, C., Erickson, A. and Chamberlain, E. (1997). Freeze-thaw and hydraulic conductivity of bentonitic barriers. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 123, pp. 229-238.

La Gatta MD (1992) Hydraulic conductivity tests on GCLs subject to differential settlement. University of Texas at Austin, Master's Thesis, Austin, Texas, USA.

La Grega M.D., Buckingham P.L and J.C Evans (1994) Hazardous Waste Management. McGraw Hill International Editions, Civil Engineering Series.

Maciel FJ & Juca JFT (1990) Gases monitoring methodology applied in Muribeca solid waste landfill. Fourth International Congress on Environmental Geotechnics, Rio de Janeiro, Brazil.

Madsen F T (1994) Clay and synthetic liners - durability against pollutants attack. XII ICSMFE, New Delhi, India.

Manassero M & Shackelford CD :1994: Classification of industrial wastes for re-use and landfilling. First International Congress on Environmental Geotechnics, Edmonton, Canada.

Manassero M, Van Impe W F & Bouazza A (1996) Waste disposal and containment. State of the Art Report. Second International Congress on Environmental Geotechnics, Osaka, Japan.

Manassero, M., Parker, R., Pasqualini, E., Szabo, I., Almeida, M.S.S., Bouazza, A., Daniel, D.E. and Rowe, R.K. (1998). Controlled landfill design (Geotechnical aspects). Proceedings, 3rd International Congress on Environmental Geotechnics, Lisboa, Portugal. pp. 1001-1038.

McCartney, J.S., and Zornberg, J.G. (2002). "Design and Performance Criteria for Evapotranspirative Cover Systems." Proceedings of the Fifth International Conference on Environmental Geotechnics, Rio de Janeiro, Brazil, 11-15 August, A.A. Balkema, Vol. 1, pp. 195-200.

McCartney, J.S., Zornberg, J.G., Swan, R.H., and Gilbert, R.B. (2004). "Reliability-Based Stability Analysis Considering GCL Shear Strength Variability." Geosynthetics International, June 2004, Vol. 11, No. 3, pp. 188-208.

McKendry P J : 1995 : Landfill design using a quantitative approach to risk assessment. Proceedings Ardinia 95, Fifth International Landfill Synopsium, Cagliari, Italy.

Mitchell J K & Jabar M (1990) Factors controlling the long-term properties of clay liners. Waste Containment systems, R Bonaparte (ed), ASCE, New York, USA.

Mitchell J K & Madsen F T (1987) Chemical effects on clay hydraulic conductivity. Geotechnical Practice for Waste Disposal '87, R D Woods (ed), ASCE, New York, USA.

Mitchell J K (1996) Geotechnics of soil-waste material interactions. Second International Congress on Environmental Geotechnics, Osaka, Japan.

MoE (1998). Landfill standards: a guideline on the regulatory and approval requirements for the new or expanding landfilling sites. Ontario Ministry of the Environment May, Ontario Regulation 232/98. Queen's Printer for Ontario, Toronto.

Moore, I.D. (1993). HDPE pipe for burial under landfills. Sardinia 93, Fourth Intl. Landfill Symposium, Cagliari, Italy, October, pp. 1473-1482.

Murphy W L & Gilbert P A : 1985 : Settlement and cap subsidence of hazardous waste landfill. US Environmental Protection Agency, Cincinnati, Ohio, USA.

Naue Fasertechnik (1997). General Installation Guidelines.

O'Donnell, S.A., Rumar, R.R. and Mitchell, J.E. (1995). Assessment of barrier containment technologies. A comprehensive treatment for environmental remedial applications. Report PB96-180583.

Orsini, C. and Rowe, R.K. (2001). Testing Procedure and Results for the Study of Internal Erosion of Geosynthetic Clay Liners. Proceedings, Geo2001, Portland, Oregon. pp.189-201.

Paksy, A., Powrie, W., Robinson, J.P. and Peeling, L. (1998). A laboratory investigation of microbial clogging in granular landfill drainage media. Geotechnique, Vol. 48, No. 3, pp. 389-401.

Peeling, L., Paksy, A, and Robinson, J.P. (1999). Removal of volatile acids from synthetic landfill leachate by anaerobic biofilms on drainage aggregates: a laboratory study. Waste Management and Research, Vol. 17, pp. 141-149.

Peggs ID, Carlson DS & Peggs S J (1990) Understanding and preventing `shattering' failures of polyethylene FMLs. Geotextiles, FMLs and Related Products. Balkema, Rotterdam.

Peggs, I.D. and Thiel, R. (1998). Selecting a geomembrane material. Proceedings of the Sixth International Conference on Geosynthetics, Atlanta, March, Vol. 1, pp. 381-388, Industrial Fabrics Association International, St. Paul, MN.

Petermann, J., Miles, M. and Gleiter, H. (1976). Growth of polymer crystals during annealing. Journal of Macromolecular Science – Physics, B12 (3), pp. 393-404.

Petrov, R.J. and Rowe, R.K. (1996). GCL-chemical compatibility by hydraulic conductivity testing and factors impacting its performance. Geotechnical Research Report, GEOT-11-96, Geotechnical Research Centre, University of Western Ontario, London, Ontario, Canada.

Petrov, R.J. and Rowe, R.K. (1997). Geosynthetic clay liner compatibility by hydraulic conductivity testing: Factors impacting performance. Canadian Geotechnical Journal, Vol. 34, No. 6.

Petrov, R.J., Rowe, R.K. and Quigley, R.M. (1997a). Selected factors influencing GCL hydraulic conductivity. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 123, 683-695.

Petrov, R.J., Rowe, R.K. and Quigley, R.M. (1997b). Comparison of laboratory measured GCL hydraulic conductivity based on three permeameter types. Geotechnical Testing Journal, ASTM, Vol. 20, No. 1, pp. 49-62.

Potter L. T & Simmons E. K (1998) Composition of Petroleum Mixtures. Vol. 2 TPHCWGS, Amherst Scientific Publishers.

Rittmann, B.E., Fleming, I.R., and Rowe, R.K. (1996). Leachate chemistry: its implications for clogging. In Proceedings of the North American Water and Environment Congress '96, June 1996. American Society of Civil Engineers, Anaheim, California, (CD-ROM).

Rodhe J R & Gribb M M (1990) Biological and particulate clogging of geotextile/soil filter systems. Proceedings of Geosynthetics Testing for Waste Containment Applications. ASTM Special Publication 1081.

Rodringuez JA & Velandia E (1990) Geotechnical monitoring program in Bogota's municipal solid waste landfill after the 1997 landfill slide. Fourth International Congress on Environmental Geotechnics, Rio de Janeiro, Brazil.

Rowe, R.K. (1996). The role of diffusion and modeling and its impact on groundwater quality. In Areal, M. (Ed.) Advances in Groundwater Pollution and Remediation, Kluwer Academic Publishers, Netherlands, pp. 371-403.

Rowe, R.K. (1998a). From the past to the future of landfill engineering through case histories. Keynote Lecture, Proc. Fourth International Conference on Case Histories in Geotechnical Engineering, St. Louis, pp. 145-166.

Rowe, R.K. (1998b). Geosynthetics and the minimization of contaminant migration through barrier systems beneath solid waste. Keynote Lecture, Proc. Sixth International Conference on Geosynthetics, Atlanta, Vol.1, pp. 27-103, Industrial Fabrics Association International, St. Paul. MN.

Rowe, R.K. (2001). Liner Systems. Chapter 25 of Geotechnical and Geoenvironmental Engineering Handbook, Kluwer Academic Publishing, Norwell, U.S.A. pp. 739-788.

Rowe, R.K., Armstrong, M.D. and Cullimore, D.R. (2000a). Mass loading and the rate of clogging due to municipal solid waste leachate. Canadian Geotechnical Journal, 37: 355-370.

Rowe, R.K., Armstrong, M.D. and Cullimore, D.R. (2000b). Particle size and clogging of granular media permeated with leachate. Journal of Geotechnical and Geoenvironmental Engineering, 126(9): 775-786.

Rowe, R.K., Cooke, A.J., Rittmann, B.E., and Fleming I.R. (1997c). Some considerations in numerical modeling of leachate collection system clogging. Lead lecture. Proc. 6th Intern. Symposium on Numerical Models in Geomechanics, Montreal, July, pp. 227-282.

Rowe, R.K., Fleming, I.R. (1998). Estimating the time for clogging of leachate collection systems. In Proceedings of the 3rd International Congress on Environmental Geotechnics, Lisbon, Portugal, September 1998, Vol. 1, pp. 23-28.

Rowe, R.K., Fleming, I.R., Armstrong, M.D., Cullimore, B.R., Rittman, B.E., Bennett, P., and Longstaffe, F.J. (1997a). Recent advances in understanding the clogging of leachate collection systems. Proc. 6th International Landfill Symposium, Cagliari, Italy, Oct. 1997, pp. 383-390.

Rowe, R.K., Fleming, I.R., Armstrong, M.D., Millward, S.C., VanGulck, J., and Cullimore, R.D. (1997b). Clogging of leachate collection systems: some preliminary experimental findings, Proc. 50th Canadian Geotechnical Conference, Ottawa, Canada, pp. 153-160.

Rowe, R.K., Quigley, R.M. and Booker, J.R. (1995). Clayey Barrier Systems for Waste Disposal Facilities. E & FN Spon (Chapman & Hall), London, 390 pp.

Rubinstein R Y (1981) Simulation and the Monte Carlo Methods, Wiley

Ruhl, J.L. and Daniel, D.E. (1997). Geosynthetic clay liners permeated with chemical solutions and leachates. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 123, pp. 369-380.

Sai J O & Anderson D C (1991) State of the art field hydraulic conductivity testing of compacted soils, EPA, Cincinnati, Ohio, USA.

Sangam, H.P. (2001). Performance of HDPE geomembrane liners in landfill applications. PhD Thesis, The University of Western Ontario, Department of Civil and Environmental Engineering, 400 p.

Sano K, Anklovitch D & Ohhashi T (1996) A water quality monitoring system for deep depths. Second International Congress on Environmental Geotechnics, Osaka, Japan.

Santucci de Magistris, F., Silvestri, F. and Vinale, F. (1998). Physical and mechanical properties of a compacted silty sand with low bentonite fraction. Canadian Geotechnical Journal, Vol. 35, pp. 909-925.

Savidis S & Mallwitz (1997) Self-healing behaviour with regard to permeability of mineral sealing materials in disturbed landfill liners / liner systems. Advanced Landfill Liner Systems. Ed by H August, U Holzlöhner and T Meggyes. Thomas Telford.

Schnabel W. (1981). Polymer Degradation: Principles and Practical Applications, Hanser International, Toronto, 227 p.

Schowengerdt RW (1987) An overview of groundwater monitoring techniques. Proceedings of ASCE Symposium on Detection, Control and Renovation of Contaminated Ground Water, Atlantic City, NJ, USA.

Schubert, W.R. (1987). Bentonite matting in composite lining systems. Geotechnical Practice for Waste Disposal '87, R.D. Woods, ed., ASCE, New York, pp. 784-796.

Shackelford C.D. (1994) Waste-soil interactions that alter hydraulic conductivity. Hydraulic Conductivity and Waste Contaminant Transport in Soil, ASTM STP 1142, D E Daniel & S J Trautwein eds., ASTM, West Conshohoken, PA.

Shackelford C.D. (1999) Remediation of contaminated land: an overview. XI Panamerican Conference on Soil Mechanics and Geotechnical Engineering. Foz do Iguassu, Brazil.

Shan, H.Y. and Daniel, D.E. (1991). Results of laboratory tests on a geotextile/bentonite liner material. Proceedings, Geosynthetics '91, IFAI, St. Paul, MN, Vol. 2, pp. 517-535.

Shrivastava AK & Mimura M (1996) Use of geophysical methods in environmental geotechnical in-situ site characterisation. 2nd International Congress on Environmental Geotechnics, Osaka.

Sivapullaiah, P.V., Sridharan, A. and Stalin, V.K. (2000). Hydraulic conductivity of bentonite-sand mixtures. Canadian Geotechnical Journal, Vol. 37, pp. 406-413.

Stark, T.D. and H.T. Eid. (1996). Shear behavior of reinforced geosynthetic clay liners. Geosynthetics International, Vol. 3, no. 6, pp. 771-786.

Stewart, D.I., Cousens, T.W., Studds, P.G. and Tay, Y.Y. (1999). Design parameters for bentonite-enhanced sand as a landfill liner. Proceedings of the Institution for Civil Engineers, Geotechnical Engineering, Vol. 137, pp. 189-195.

Tognon, A.R., Rowe, R.K. and Moore, I.D. (2000). Geomembrane strain observed in large-scale testing of protection layers. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 126, No. 12, pp. 1194-1208.

Trautwein S J & Boutwell G P (1994) In situ hydraulic conductivity tests for compacted soil liners and caps. Hydraulic Conductivity and Waste Contaminant Transport in Soil. ASTM, Special Technical Publication 1142, Philadelphia, USA.

US EPA (1988) Superfund Exposure Assessment Manual, EPA/540/1-88/001. Environmental Protection Agency, USA

US EPA (1989a) Risk Assessment Guidance for Superfund. Vol 1 Human Health Evaluation Manual (Interim Final 9/29/89). Environmental Protection Agency, USA

US EPA (1989b) Exposure Factors Handbook, Office of Health and Environmental Assessment, EPA/600/8-89/043. Environmental Protection Agency, USA

Vogler M, Dorr H, Katzenbach R & Molitor N (1994) Quality and Efficiency Control During Soil Air Sampling and Remediation. First International Congress on Environmental Geotechnics. Edmonton, Alberta, Canada.

Vorhees J.D. et al. (1999) Human Health Risk-Based Evaluation of Petroleum Release Sites: Implementing the Working Group Approach. Vol 5 TPHCWGS, Amherst Scientific Publishers.

Weisman Wade (1998) Analysis of Petroleum Hydrocarbons in Environmental Media. Vol.1 TPHCWGS, Amherst Scientific Publishers.

Weltman AJ and Head JM (1983) Site investigation manual. Ciria Special Publication 25. Construction Industry Research and Information Association, London.

Yong, R. N. (1997). Multi-disciplinarity of environmental geotechnics. *Proceedings of the Second International Congress on Environmental Geotechnics*. M. Kamon (ed), Balkema, Rotterdam, 1255-1273.

Zornberg, J.G., and Christopher, B.R. (1999). "Chapter 27: Geosynthetics." In: The Handbook of Groundwater Engineering, Jacques W. Delleur (Editor-in-Chief), CRC Press, Inc., Boca Raton, Florida, ISBN 0-8493-2698-2.

Zornberg, J.G., Jernigan, B.L., Sanglerat, T.R., and Cooley, B.H. (1999). "Retention of Free Liquids in Landfills Undergoing Vertical Expansion." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 125, No. 7, pp. 583-594.

Zornberg, J.G., McCartney, J.S., and Swan, R.H. (2004). "Internal Shear Strength of Geosynthetic Clay Liners." Journal of Geotechnical and Geotechnical Engineering, ASCE, Vol. 131.

Chapter 2

Managing Contaminated Sites

The context of risks associated with contaminated land

Author:

Stephan JEFFERIS
Environmental Geotechnics Limited, Oxford, United Kingdom

Contributing Authors:

Márcio de Souza S. DE ALMEIDA
Universidad Federal do Rio de Janeiro, Brazil
Micheal LOXHAM
Geo Delft DG Envir. Ltd., Delft, The Netherlands
Trevor ORR
University of Dublin, Ireland
Christian SCHLUCHTER
University of Bern, Switzerland

The authors gratefully acknowledge the review of this chapter and the contribution made to it by:

Akram ALSHAWABKEH, *Northeastern University, Boston, MA, USA*
Ian A. HOSKING, *Coffey Geosciences Pty. Ltd., Australia*

2.1 INTRODUCTION

This document sets out the evolving concepts of risk that have influenced the development of procedures for the management of contaminated land. It is not a manual on contaminated land investigation nor on remediation procedures as these are well presented elsewhere.

It is accepted that, in many countries, the management of contaminated land is now a commodity product underpinned by well established rules and regulations. As a result, clients now expect low cost products such that the involvement of senior experts may be limited to the role of review or the management of large or complex sites where their involvement can be financially justified.

That contaminated land has reached the stage of a commodity product demonstrates that it should now be a standard item in the geotechnical engineer's toolkit. Furthermore, the geotechnical engineer must be able to establish a dialogue with the many environmental disciplines that are also involved today in environmental and sustainability problems. The geotechnical engineer also will have to develop an awareness of the social sciences which are important to the wider sustainability issues required for the development of many of today's projects.

In addition to developing her/his expertise and maintaining links with environmental science professionals, the geotechnical engineer must understand the intellectual basis of risk management as applied to contaminated land.

2.1.1 Environmental risk and the geotechnical engineer

The geotechnical engineer, by training and experience, should be well acquainted with the concepts of risk for the purpose of recognising the impact of unknowns such as soil conditions, supply chain imperfections and contract inadequacies on the functionality, timely delivery and cost of projects. The possibility or fact that a construction site or some of the soil on it is contaminated is simply an added complication that has to be taken into account, but one that until quite recently would not have been covered in university training and indeed in some parts of the world is still not included in civil engineering courses.

It is unlikely, and highly undesirable, that the geotechnical engineer who has not been specially trained in this field of contaminated land will work alone to solve the problem. Instead, the geotechnical engineer more likely will be part of a team deployed to actually remediate the contamination or at least bring the impact of the contamination within acceptable limits. This teamwork approach will require the geotechnical engineer to attain additional specialist services of contamination hydrogeologists, chemists, biologists, toxicologists, chemical engineers and sometimes and in some countries, agricultural engineers. The geotechnical engineer will also almost always have to interface with lawyers, regulators and all manner of other disciplines.

2.1.2 The presence of contamination on a site

In relation to contaminated land management, two situations are generally distinguished:

- Where the ground is known to be contaminated before the construction starts; and
- Where the ground is discovered to be contaminated in the course of the construction.

The risk profiles for these two situations are very different. In the first case, the problems will have been analysed beforehand and an appropriate action plan will have been formulated and agreed with all concerned. In the latter case, the project could be seriously impeded by the discovery, nobody is properly prepared, and the orderly conduct of business can be severely disrupted, often leading to substantial delays and cost overruns. In this case, it is essential that the professional geotechnical engineer clearly understands his position and his role.

In both situations, the geotechnical engineer will almost certainly be called upon to contribute to the practical solution of the problems posed by the contamination. The solution will be generated and implemented within a complicated framework of interacting social, environmental and economic factors. It is the purpose of this text to make the geotechnical engineer aware of the requirements and constraints operating in what is becoming a common but specialised field.

2.1.3 The role of the geotechnical engineer in contaminated land management

The role of the geotechnical engineer in contaminated land management will include recognising and managing all the forms of harm that can be done by contamination in land (N.B. In this respect harm is a general term used to describe all loss of quality). The geotechnical engineer must ask himself: could there be harm to the health of:

- Those investigating the site for contamination e.g. by skin contact, or by inhalation, etc.?
- Those working on the site during remediation and/or construction work?
- Neighbours and those at a distance form the site, e.g. from dust or vapours?
- Future users or visitors to the site, e.g. by build-up of gases such as carbon dioxide or methane?
- Children e.g. by skin contact, ingestion when playing on the site during or after development?
- Gardens, allotment users e.g. by skin contact, and the consumption of contaminated vegetables?
- Personnel involved in the maintenance of underground services or the installation of new services?
- The ecology of the site, its surroundings, flora and fauna?
- Domestic and farm animals etc.

and the geotechnical engineer must ask could there be harm to:

- Groundwater and surface water? Ground and surface waters are a resource contamination may be particularly difficult and costly to remediate.
- The works, buildings etc. e.g. by explosion, fire or chemical;/physical reactions?
- Service cables and pipes etc.? Organic contaminants and gases can diffuse through the walls of plastic pipes and there have been many incidents of drinking water being tainted so as to be unpleasant / unfit to drink after domestic heating oil has leaked from underground pipes or has been spilled.
- Foundations and structures? Generally this requires the presence of significant quantities of contaminants or particularly aggressive contaminants. Structural damage is more often associated with chemicals naturally present in the ground such as sulfates.
- Building, e.g. from combustion of contaminant materials?
- Neighbouring sites, by migration of contaminants?

Note: see also section 6.3

The longevity/degradability of contaminants is an important issue but it should not be assumed that degradable contaminants will have been lost. Nor should it be assumed that soluble compounds will have been dissolved and flushed away. Contaminants can persist for surprisingly long times in the ground and their presence may become obvious only when there is some change of circumstance: earth moving, installation of soakaway pits for roof water drainage from buildings etc.

2.2 THE PERCEPTION OF POLLUTION

Before considering the risks associated with contaminated land, it is necessary briefly to consider the social implications of pollution.

The concerns raised by pollution can be presented as spectrum of issues between two extremes: the emotional and the technical. The emotional response is that all pollution represents a risk and no risk is acceptable. Public debate has tended to harden the emotional view. Thus, at the extreme emotional position, societies have moved from the NIMBY syndrome (not in my back yard) to the BANAANA syndrome (build absolutely nothing at all near anyone). For politicians involved in decisions on controversial projects, NIMTOO (not in my term of office) can be an important consideration. The emotional response of an electorate can be a very potent stimulus for both action and inaction. At worst, the action may be new legislation merely to show government is doing something and the inaction may be the side-lining of decisions on major developments. The cynical will view the situation as one which will continue to oscillate depending on the contemporary focus of the electorate. For example, environmental consciousness has waned in earlier generations when recession hit. However, although consciousness may wane, the current world-wide momentum is such that the environment is now an item on nearly every industrial/development agenda. However, sensitivities do of course vary. It has also been observed in recent years that a focus on land contamination problems has been a luxury of the developed world. Areas where the livelihoods for generations have been industry and mining are often less sensitive to environmental issues than say an urban electorate driven more by what seem to be ideals than real risk. Also, political imperatives can be quite different in poorer nations where the focus may be on food and clean water rather than contamination of the land.

Of course, perception and reality can merge. The owner of a site may not be able to sell or develop it if it is thought to be contaminated and could have to expend much money to prove that it is clean before the negative perception can be removed. It will be of little consolation to the owner that there is no proof that the site is contaminated if influential people believe or fear that it is. This has led to the need to remediate sites to very high standards in order to realise the inherent 'uncontaminated' real estate value and the coining of the term 'site polishing'.

2.2.1 Some questions

Unfortunately for environmental problems, the quantification of risk can be a time consuming and costly undertaking. The nature and extent of contamination must be reasonably estimated. Exposure pathways must be identified and consequences investigated. Even after extensive investigations and assessment, the uncertainties associated with the cost and time of remediation will be much greater than those associated with the civil and even the geotechnical design.

2.2.1.1 When does the harm occur?

It is very important to realise that for pollution, there may be a very extended period between dose and demonstrable response. For example, asbestos has a very long 'incubation period' after exposure before lung disease becomes apparent. As an example, the mercury poisoning at Minamata, Japan took years to be identified and longer again for any compensation to be ordered. The geotechnical engineer should recognise that some contaminants give very poor warning (i.e. have a very long period between exposure and identifiable response). These contaminants may be not only harmful to the geotechnical engineer, but also to all those who have been exposed. Contaminants with poor warning could be the 'asbestos' of the future.

Risk, dose and response are becoming the keywords for the management of all forms of pollution including contaminated land. Of course, the risks are not only to human life. Animal and plant life must be considered as well as damage to structures, foundations, services etc. Indeed, with many current models, the ecotoxicological risks are found to be higher than the human health risks. Thus, the exposure and sensitivity of different receptors (or targets) needs to be considered. Table 1 outlines some of the general questions that must be asked about any site today.

Table 1: Some of the questions

- What types of site are potentially hazardous?
- Is my site a potential hazard?

- What are the risks associated with toxic materials - during the site investigation, the construction phase, in use?
- What hazards should one look for at any site?
- How should one go about assessing a site?
- How should the site investigation be carried out?
- How should one manage the contaminants? Should they be treated in-situ, contained, treated ex-situ, removed?
- What about the future? Will the control measures be durable?
- What validation is required for any remedial action?
- What monitoring is required to confirm the future performance of the site and any remediation?
- Could future legislation or changes in regulation force further works at the site?
- Could improvements in chemical analytical precision force further works (e.g. by reducing 'non-detect' levels)?

2.2.2 Environmental audits and assessments

Financial profit and loss accounts are a part of life and environmental audits are set to achieve similar status - though engineers need to be more rigorous and consistent in preparing them. Also, engineers must be prepared to go back to environmental audits in later years and re-evaluate them. For example, environmental impact assessments are required in many countries before major developments can proceed. How many of these assessments have been reviewed after the development to see how closely the predictions met reality? This back-analysis must be done for all types of assessments, including those by developers and those by protest groups. It is only in this way that the environmental audit/assessment system can gain credibility.

2.3 GENERAL STRUCTURE OF CONTAMINATED LAND LEGISLATIONS

First and foremost, it must be acknowledged that the management of contaminated land is highly regulated worldwide and the geotechnical engineer would be extremely unwise to take any action in contaminated land management without a thorough knowledge of the local regulatory regime. Three distinct situations may be identified:

- Management of historic contamination;
- Remediation of contamination and the need to avoid exacerbating or spreading the problem; and
- Prevention of future pollution.

Furthermore, as the management of contaminated land can generate substantial quantities of waste, it will be essential for the geotechnical engineer involved in contamination work to have a sound knowledge of waste management regulations.

2.3.1 The enforcing Agency

The regulations in some countries are enforced by a civil agency (e.g. in the UK, enforcing agencies include the English and Scottish Environment Agencies and the Health and Safety Executive). However, in other countries (e.g. The Netherlands) the policy driving the legislation recognises soil as a resource (similar say to groundwater), and deserving of inherent protection such that the force of criminal law may be used, exposing both the problem owner and his advisors to serious penalties including imprisonment.

There also may be further complications due the division of responsibility between national and local regulation. For example, in the USA, prevention of future contamination is federally regulated by the Resource Conservation and Recovery Act (RECRA) of 1976, while management and remediation of historic sites is governed by the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA or Superfund) of 1980. The Potentially Responsible Parties (PRPs) for contamination include: the owners, operators, generators, and transporters. There are few other areas where the geotechnical engineer can be so quickly and so deeply exposed to civil claims for large damages by litigious parties. Proper knowledge of the legislation applying at the point in question is an absolute requirement before embarking on any action at all.

It should be noted that in many jurisdictions, the regulator may not be empowered to approve the remedial designs proposed for a site. Approval of a design can bring with it liability if the design fails. The regulator's powers may be limited to not disapproving a design. This can be a source of frustration to site owners and remedial designers, but it is a necessary protection of the regulatory agency.

2.3.2 Legislative control levels

In the following discussion, some general points will be made on the structure of the legislative controls on the pollution from contaminated land. Firstly, there is a general relationship between environmental standards and legislative requirements. In fact most countries have formulated their legislation in terms of an absolute requirement that can be modified where necessary by functional considerations and sometimes cost-benefit considerations. This has strong echoes of the geotechnical debate between prescription (method statements) and functional (performance specifications) design methods. The regulator usually will try to achieve a balance between the desirable and the possible whilst responding to pressures to reduce risk in the community and address their perceptions of the risk – these can be very different from those of the technical person. However, it is important to remember that this is a world of finite resources (financial, technical, institutional, infrastructure, human and natural resources, etc.), so that wasting resources on trivial risks make the world less safe; i.e., it denies resources to higher risk areas. It is the engineer's role to help identify those sites which pose a high risk to society and where the expenditure of resources will bring sustainable benefits. However, the choice may not lie with the geotechnical engineer. We also live in a world which is becoming ever more litigious and the driver may prove to be litigation or the threat/fear of it by the empowered versus wider benefits to all, empowered and impoverished.

In passing, it may be noted that there is a need for better decision tools to help balance the potentially conflicting demands of the social, environmental and economic aspects of contaminated land management, i.e., to develop a discipline of 'sustainable geotechnics' as the logical next step after 'environmental geotechnics'.

2.3.2.1 Prescriptive controls

Legislation can be written in absolute terms such 'the concentration of some chemical X will not exceed value S mg/kg in the soil or W mg/litre in the groundwater', or 'the concentration in drinking water must be less than D microgram/litre irrespective of the individual contaminated sites contributing to the pollution', or 'the leachable concentration based on leachability test T must be les than L mg/litre'. Note that there are many different leachability tests, and results from different testing regimes may be very different. It is fundamental that appropriate regimes are specified and used.

The USEPA established limits on the concentrations of certain contaminants in public drinking water supplies referred to as 'maximum contaminant levels (MCLs)'. For example, the USEPA MCL standard for trichloroethylene (TCE), one of the most common groundwater contaminants in developed countries, is 0.005 mg/L. The USEPA also identifies contaminated soil as a hazardous material if the leachable concentration based on the Toxicity Characteristic Leaching Procedure (TCLP) exceeds a specific value, For example, soil is a hazardous material if the lead concentration in the TCLP extract exceeds 5 ppm. These standards are absolute in the sense that they do not take into account who or what may be affected by the contamination, where the site is situated, or what is possible in practice or can be economically justified. The controls would be the same for a site in the middle of a desert or next to a hospital for children. These standards are clear, unambiguous and generally useless in that they cannot be met in many real situations, unless the site is only marginally contaminated; i.e. where there is no real problem. A particularly pernicious variant of these types of standards is where the legislation calls for the concentrations to be below ‘detection level’, i.e., that the contaminant must be effectively absent. This case can turn a clean site into a contaminated one simply by changing the laboratory equipment used to analyse the contaminated soil. Also, developments in analytical equipment can lead to re-classification of a site as contaminated long after it has been apparently successfully remediated. Contamination standards based on detection levels are not appropriate for the management of contaminated land. Whilst this problem is now recognised, it should not be assumed that the regulator is empowered, by the legislation under which he must work, to accept anything but ‘non-detect’. The problem lies with the legislation, not its application.

The old Dutch ‘A, B, C’ values were also of a prescriptive nature. The UK guide levels produced by the Interdepartmental Committee on the Reclamation of Contaminated Land (ICRCL), although clearly stated to be guide levels, were soon interpreted as prescriptive control levels as no other figures or procedures were available in the UK. Indeed, many countries still base their initial assessment of a site on these so-called look-up tables, and the first step for the engineer in addressing a contamination problem is usually to compare the chemical analysis results with the local look-up table, if available, or to ‘borrow’ recognised standards such as those from the Netherlands.

2.3.2.2 Best practice controls

At the other end of the spectrum are those standards reflecting some kind of Best Practice. The most well known is perhaps the concept of the Best Available Technique Not Entailing Excessive Costs (BATNEEC), which sets out to balance cost and benefit, a laudable theory but difficult in practice and leading to debates such as whether the best available technique should be same for all industries: should a cement works have the same stack emission clean-up techniques as an incinerator? The procedure to establish what is best technology and what are excessive costs is one for local or national negotiation. This makes it very difficult, especially at the outset of a project or in the crisis of newly discovered pollution, to achieve a clear program of action. Nonetheless, these negotiations often lead to very practical and pragmatic solutions to the problem. To bystanders, however, there is always a credibility gap between what is said and what people perceive as being done. The cynical may view BATNEEC as a procedure by which to approach CATNAP / CATNIP – cheapest available techniques narrowly avoiding prosecution / cheapest available techniques not involving prosecution. In Victoria, Australia, the equivalent is CUTEP (Clean-up to the extent practicable), which is applied to groundwater in that state. BATNEEC type regulation is perhaps most appropriate for continuing processes such as chemical or manufacturing industries as it can help to promote a steady improvement in pollution emissions. BATNEEC is more difficult to apply in the generally one-off situations of contaminated land management.

2.3.3 Risk and harm

A modern development in addressing contaminated land issues is the legislative framework introduced in England in April 2000 under Part IIA of the Environment Act. The underlying methodology embedded in this and the laws of many other nations is that of Risk Analysis. The legislation focuses on the effect of the pollutant at the so-called receptor. The receptor may be far removed from the contaminated land forming the source. The legislation in England states that land

is contaminated if:
(a) significant harm is being caused or there is a significant possibility of such harm being caused; or
(b) pollution of controlled waters (these include surface and ground waters) is being, or is likely to be caused.

Note that the test relating to pollution of groundwater (b) could have been included in test (a) relating to harm, but under English law it was necessary because the laws relating to soil and water are quite separate. There was an older law relating to waters alongside which a newer law on soil has had to be fitted (though since 2000 there has been further development of the law so that the test for pollution of groundwater is based on harm – in the former law pollution was defined by the entry into groundwater of polluting material and there was no test for harm, e.g. quantity or concentration). This is an example of a regular problem, the different rates of advance of technical understanding and legal coding. It should not be assumed that, in any legislature, developments in the law will match developments in the technical understanding of problems such as contaminated land, because the more likely scenario is that the law once codified is difficult to change! Actually this is sometimes an advantage as technical understanding can be an iterative process and rapid iterations in the law would be socially unacceptable.

The harm is assessed at the receptor, and a key concept is that there can be no harm to the receptor unless there is a pathway between the source and the receptor, as illustrated in Figure 1. The relationship between a source and a receptor via a pathway may be called a linkage.

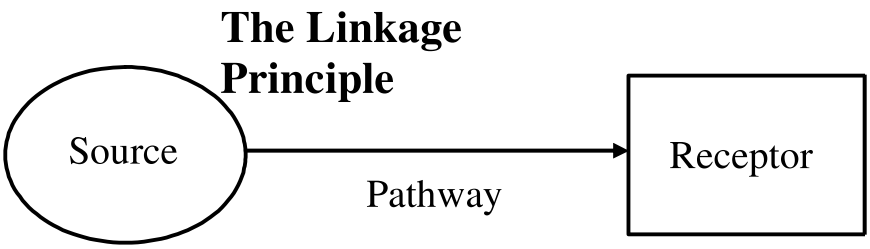


Figure 1. A Source-Pathway-Receptor linkage

If a linkage does not exist, that is there is not a significant chance of significant harm at the receptor, then for the purposes of this risk based law, the site is not contaminated. Nonetheless it should be recognised that even though the site is not contaminated legally, the use value, sale value or further exploitation could well be seriously impaired now or in the future. That is, the receptor may one day be on or at the site, such that the pathway is established. Furthermore, it should not be assumed that the Source-Pathway-Receptor concept is the only risk based assessment procedure.

For example, in the USA, a Hazard Ranking System (HRS) has been developed by the USEPA as the principal mechanism to include uncontrolled waste sites on the National Priorities List (NPL). The HRS is a numerically based screening system that uses information from limited investigation, including the preliminary assessment and the site inspection, to assess the relative potential of sites to pose a threat to human health or the environment. The HRS uses a structured analysis approach that assigns numerical values to factors that relate to risk based on conditions at the site. The factors are grouped into three categories: (1) likelihood that a site has released or has the potential to release hazardous substances into the environment; (2) characteristics of the waste (e.g. toxicity and waste quantity); and (3) people or sensitive environments (targets) affected by the release. The pathways that can be scored under the HRS include (a) ground water migration (drinking water); (b) surface water migration (drinking water, human food chain, sensitive environments); (c) soil exposure (resident population, nearby population, sensitive environments); and (d) air migration (population, sensitive environments). The scores are usually calculated for one or more pathways and are combined using a root-mean-square equation to determine the overall site score. If all pathway scores are low, the site score is low. However, the site score can be relatively high even if only one pathway score is high. This is an important requirement for HRS scoring, because some extremely dangerous sites pose threats through only one pathway. However, the priority in determining the funding for USEPA remedial actions is not based on the HRS, because the information collected to develop HRS scores is not sufficient to determine either the extent of contamination or the appropriate response for a particular site. Remedial Investigation/Feasibility Studies (RI/FS) usually follow listing of the sites on the NPL.

2.3.4 Liability for contaminated land

Contaminated land legislation must consider not only control of the contamination itself but also who to blame for it and who should pay to remediate the situation (or more generally, who are the Potentially Responsible Parties, PRPs). Almost all national legal frameworks are based on polluter pays principle with the implication that this is ethically the correct approach. This may be the case, but a note of caution is necessary. Much of the contamination which is being paid for today would not have been regarded as pollution at the time it was created, and it involved no breach in the law at that time; what was done was legal at the time. To put it another way, there has been a retrospective imposition of liability for actions that were previously legal. Retrospective liability is a complex legal imposition. Once established, it may be further extended, and today's management of contaminated land may yet be held by future generations, as pollution for which someone must pay and the engineer is at risk to be the first target. Also, in former times, society, or at least some parts of it, will have benefited from pollution through slightly lower product prices (e.g. through cheaper energy prices) or more rapid exploitation of new technologies. However, when the costs have to be paid, society in the form of government may be more reluctant to be involved. A final and important feature of legislation based on the polluter-pays principle is that it can lead to long legal wrangles and substantial delays to site clean-up. The Danish legal system recognises this problem and places less emphasis on 'polluter pays'.

2.3.4.1 Finding the funds

In practice, the situation is yet further complicated. First of all, the construction priorities at a site may make it impossible to wait to investigate and establish who the polluter(s) really were. Work has to go forward such that the lawyers are left to sort out the problem retrospectively. Secondly, it may be difficult to find somebody who is liable for and can pay for the historical pollution. Many countries have orphan site legislation and funds (e.g. the Superfund in the USA) to cover such issues, but times to access these funds are typically measured in years or even decades rather than weeks.

2.3.4.2 Creating new liabilities

The geotechnical engineer has to be very careful not to be involved in any activities that cause the contamination, or that allow the contamination to spread such that the geotechnical engineer is placed in the position so as to be held responsible for future problems caused by the site. The English legislation is especially delicate in this respect because a seemingly unrelated geotechnical action could complete a potential, but up to then not actual, Source-Pathway-Receptor linkage leaving the engineer open to prosecution or legal action. The engineer who inadvertently joins up someone else's source to a receptor may be liable for the consequences of creating contaminated land. For example, a poorly sealed borehole may link contamination seeping from a neighbouring site into an aquifer. Other legislatures may not impose such heavy liabilities, but the engineer will be failing if the potential for creation of new pathways is not considered or the types of new receptors that could be introduced are not recognised. To ensure that the engineer has a sufficient understanding of the site, it is essential that he properly assesses the setting of the site.

2.4 ASSESSMENT OF THE SITE

An understanding of the geological and hydrogeological setting is an essential prerequisite to the management of a contaminated site. The geology and hydrogeology may provide a useful last and final safety barrier against contaminant migration (e.g. a clay geology – a situation that will be sought when siting waste disposal landfills) or a pathway for rapid dissemination of the contamination to groundwater (e.g. a major aquifer in fissured chalk).

Any management or monitoring, either short or long term, has to start from a proper understanding of these issues and recognition that groundwater can be not only a pathway but also an important receptor for harm.

2.4.1 Source and type of contamination

In geological terms, there are two types of contamination:

(a) the contaminant has been spilled or otherwise released to the ground as a result of accidents or industrial activity. On sites such as these, the range of contaminants may be quite limited and sometimes can be identified from a review of the history of the site and the activities on it. The contaminants will have the potential to spread and migrate from the source zone, although they may be attenuated by degradation mechanisms or sorption onto the soil (e.g. metals onto clays or organics onto soil organic matter);

and

(b) the contaminant is part of a deliberately created landfill. In the case of old, non-engineered landfills, the local geology may have been considerably disturbed, be highly permeable and offer minimal attenuation (e.g. in former times, the void used for a landfill was often the result of gravel extraction). A wide and unidentifiable range of contaminants may be present, potentially including some very difficult and recalcitrant materials such as polychlorinated biphenyls (PCBs). For newer, engineered landfills, the hope is that there will be more reliable controls on contaminant migration, but the range of contaminants is still likely to be very large.

In both cases, the chemical and physical nature and behaviour of the contaminant has to be evaluated in detail or confirmed from the history of the site. Contamination may be present in many different states including:

- a gas or mixtures of gases, lighter, neutrally buoyant or denser than air;
- a free phase liquid phase lighter or denser than water (i.e. a light or dense non-aqueous liquid, LNAPL, DNAPL) or a separate phase which is neutrally buoyant or which can be a DNAPL or LNAPL depending on the temperature;
- in solution in the groundwater;
- soluble solids which cause both chemical harm and physical settlement if groundwater reaches them, for example as a result of changes to the surface water management regime following development of a site (e.g. installation of soakaways); and
- a viscous semi-solid which may appear rigid but which will ooze to the surface if buried.

Note that the significant level of solubility at which a solid can cause harm will depend on the nature of the harm. If the harm is physical settlement, then the solubility must be such that significant amounts can dissolve into the groundwater. If the harm is because of toxicity, then solubilities measured in milli or micro grams per litre may be significant. The solubility of NAPLS also will be significant in terms of toxicity.

To prepare a decision analysis for remedial work, engineers need to know if the contaminant is potentially harmful to any of the receptors previously identified (see Sections 1.3 and 6.3). It also will be necessary to consider whether the contamination could, over time, degrade to harmful or more harmful by-products.

2.4.1.1 Site history

A crucial factor in evaluating a contaminated site is its history: is it an accident that requires an immediate action or is it an 'inherited' situation which has evolved over weeks, months, years or decades. In the case of a site 'with a history', much valuable information may be obtained from a reconstruction of the history by review of site documents, maps, aerial photographs, etc., and interviews with current or former employees. A full history is rarely possible, but even a very limited history can be of enormous value when planning site investigations and remedial strategies and just simply understanding why the contaminants are there, how much and whereabouts.

In the case of the contamination from old landfills, there is in most cases at least some information such as tipping records (source of waste and period of tipping). Aerial photographs also may be available, if by chance the site was overflown at useful times. A detailed search in the local archives or at state agencies may be useful. Tracing former employees, either those who created the void (e.g. excavated gravel) or those who filled it (landfill employees), may be difficult as once excavation or filling is complete they will have moved on. Also, landfill employees may have little information on the properties of the waste that was placed.

For landfill sites, mapping by geophysical methods (e.g. ground probing radar, electrical resistivity, gas flux measurements at the ground surface) may give very useful data on the extent and depth of the contamination and the spread of any leachate plume.

2.4.2 Geology and hydrogeology

Important geological information will be the presence of:

- permeable features such as gravel or sand aquifers or lenses;
- fissured rocks such as fissured chalk; and
- low permeability materials such as clays or silty clays, these may be particularly important in limiting vertical penetration of the contamination.

Data on the hydrogeological setting also will be required including:

- the groundwater level (including seasonal variation). If remedial systems which exploit the local hydrogeology are planned (e.g. permeable reactive barriers) then continuous groundwater level monitoring data from down-hole logging devices may be very useful at the design stage;
- local and regional ground and surface water flow patterns;
- data on hydraulic parameters (porosity, permeability);
- location of ground and surface water extraction points; and
- location of effluent discharges.

In some countries/locations, hydrogeological maps showing groundwater contours, natural springs etc. may be available. Licence details on water extraction points and effluent discharges also may be available.

It is also noteworthy that often the most significant geological boundary beneath a site is the boundary between the natural soil and man made fill. This often (but by no means always) represents the boundary between contaminated and non-contaminated (or lesser contaminated) soil. As the geotechnical engineer knows, this boundary can be highly irregular, and there have been many cases of surprises when contaminated fill is excavated, only to find deep trenches between boreholes, in-filled basements and the like, leading to large increases in site remediation costs.

The geotechnical engineer must also be wary of investigations that suggest some of the filled portion of a site is contaminated, whilst some is not. The distribution of contamination in fill soils, particularly those that were contaminated prior to be being deposited on a site, can be (and generally is) highly variable spatially. There have been many cases where remedial planning has assumed a percentage of the fill to be contaminated, where attempted validation of remaining fill has resulted in the need to remove all of the fill.

2.4.2.1 Minimum programme in area with an aquifer

In an area with an aquifer (where groundwater is a key receptor and a major contaminant migration pathway), any contamination should be monitored or managed at least to some extent (depending on the type and extent of the contamination). If the contaminant is not confined to the unsaturated zone and if it cannot be removed, a thorough hydrogeological investigation has to be completed with at least three boreholes (one upstream, one downstream and a third to confirm flow direction). All boreholes must be logged geologically to determine the host environment. A simple 3D reconstruction of the main lithological units should be attempted (e.g. simple layer structures; more complex structures, lenses etc. will require many more than three boreholes). A full 3D model will require very many boreholes and possibly geophysics as well.

Boreholes equipped for pumping tests also may be necessary to establish the flow regime in the aquifer. Early advice from a hydrogeologist, particularly one specialising in contaminant transport, is advisable.

It must be remembered that plumes of contaminated groundwater can be of irregular shape, including long and narrow, floating plumes and sinking (DNAPL) plumes. For example, a careful case study by CSIRO in Perth, Australia involving multiple, short screened wells, defined a benzene plume that, whilst only being about 30m wide, was some 500m long.

2.4.2.2 Minimum programme in an area without a significant aquifer

The absence of an (important) aquifer needs to be verified by appropriate investigations (desk based and field). At least one borehole will be necessary. Special care needs to be taken if a contamination occurs in a karst area which may require high resolution regional geological mapping.

2.4.3 Overall checklist

- visit the site and walk over it. If permitted, photograph key features. If access to the site is denied the geotechnical engineer should consider whether to continue with the services;
- site specific and/or regional geological mapping; search for archive documents;
- simple 3D – geological reconstructions including aquifer depth, consider whether a more detailed reconstruction is necessary;
- construct map of aquifer and flow regimes (contaminant migration pathways) ;
- consider groundwater resources and current contamination status;
- reconstruct hydraulic properties of soil or rock with special attention to fissured rock, chalk, karst;
- reconstruct history of contamination (accident, long term evolution and hazards);
- consider contaminant properties (e.g. chemistry, solubility, LNAPL, DNAPL, biodegradability etc.;
- consider the plausible extent of the contamination in relation to hydraulic properties;
- consider site topography and accessibility for plant, drill rigs, excavators;
- identify location of services; and
- synthesise of an overall conceptual model of the contamination, geology and hydrogeology.

The final step, the synthesis of a conceptual site model is a key step in site risk assessment and will be a key tool for the dissemination of information on the site to the client, other engineers and the wider panoply of stakeholders around any site. The conceptual model should be capable of presentation both in technical detail for experts and in non-technical terms for lay people.

2.5 CONSIDERATIONS FOR REMEDIAL DESIGN AND IMPLEMENTATION

It has become very clear over the last twenty years that full clean-up of contaminated land is rarely possible when dealing with sites with serious pollution. The one major exception to this is where a so-called dig and dump strategy can be applied (but remember that at another site, the dump area is further contaminated) – and this is only possible where the contaminated materials are readily accessible as can be the case on sites undergoing redevelopment.

However, this is not the case for many sites and other strategies have to be developed. Fortunately, other options are available and are now well tried. The following discussion sets out some of the very basic concepts. The reader is directed to specialist texts for fuller information (see, for example, La Grega et al, 2003)

2.5.1 Isolation or containment

This is where the contaminated land is prevented from releasing toxic material to the environment or where the toxic material is intercepted along the emission pathway before it reaches the receptor. Examples include:

- cut-off walls or grouting to limit groundwater migration;
- management of groundwater pathways by pumping (geohydrogeological isolation);
- interception of hydrocarbons e.g. floating contaminants by cut-off walls;
- source solidification by soil mixing or grouting;
- cover layers to prevent the escape of chemical vapours and/or downward infiltration of surface or rain water; and
- capillary break layers to control the upward migration of contaminants by capillary rise.

In many cases, the construction itself can be used as the container for the isolated material. An example is to concentrate the contaminated material in an on-site landfill which may be designed into the geotechnical works. However, this may require a waste management licence. For example, the construction of an on-site landfill in England will require that the site is licensed as a landfill. This will require a considerable geological and hydrogeological investigation and may take several years to achieve the necessary planning consents. Whilst this procedure for licensing is quite proper, it should be recognised that it effectively prevents on-site landfill and forces disposal to other licensed landfills – a process which separates the liability (the contamination) from the asset (the land) and may disadvantage future generations. Also, as a general principle, the more that is regulated, the less are the options for risk based decisions, designs or for stakeholder involvement. The state takes on the role sole stakeholder.

2.5.2 In-situ treatment

The essential characteristic of in-situ treatment is that it is done in the ground. This can have major advantages in that the site and any construction are left largely undisturbed.

The technologies are directed at extracting, degrading, stabilizing or changing the chemical nature of the contamination such that it is either immobile or non-toxic. In situ treatment techniques can be physical, such as soil vapour extraction; chemical, such as in situ oxidation or reduction; biological, such as in situ bioremediation or phytoremediation; or even electrochemical, such as electrokinetic techniques. While in situ techniques are favourable, they are often complex, can be expensive, and are challenged by the physical, chemical and biological heterogeneity of the site and contaminants.

2.5.3 Pump and treat techniques

These are essentially the same as geohydrological isolation by pumping except that the intention is now to treat the source rather than control migration from it. This requires that the contaminant is mobile. For example, it may be used to recover the mobile fraction of free-phase hydrocarbons floating on the groundwater. However, full source removal may be very slow, and effectively may never be achieved. The recovered contaminant is usually mixed with groundwater and this has to be treated before it can be discharged either to sewer or back into the ground. It is likely that some form of licence will be required for the water abstraction and that a discharge consent will be required if the 'cleaned' groundwater is to be returned to the ground or surface water. Water quality standards are likely to be stringent for the cleaned water – it may have to meet drinking water standards or an environmental quality standard. If only small amounts of water are pumped, it may be economic to return the water to foul sewer – subject to the agreement of the sewerage undertaker. Note that in some jurisdictions including England, the sewerage undertaker is entitled to refuse to accept groundwater – though he must accept liquid waste from domestic or industrial premises.

2.5.4 On-site, ex-situ soil cleaning

It is often desirable to retain as much of the soil on site as is possible to avoid the costs of transporting polluted soil from the site and the import of clean soil. With on-site but ex-situ techniques, the contaminated soil is excavated and treated on-site typically in windrows for bio-remediation or in a skid mounted treatment unit for process based technologies such as soil washing. Techniques include: soil washing whereby the soil is divided into a clean fraction, often the coarse fraction and a dirty fraction which will require subsequent treatment or disposal; ex-situ bioremediation; incineration (thermal destruction) and low thermal treatment (effectively distillation followed by thermal destruction of the vapours released). The potential for secondary environmental effects from contaminated material and gaseous discharges from the treatment plant should be considered. The treatment plants can become quite complex and chemical engineering input is required at the design stage. Figure 2 shows a schematic of an on-site plant.

2.5.5 Natural attenuation

The soil system itself can act as a chemical reactor and attenuate the toxicity of the contaminants moving from the source along the pathway to the receptor. Chemical and biochemical effects will be important and regulators will be unlikely to accept attenuation arising solely from dilution and dispersion as treatment. They are likely to accept natural attenuation as a clean-up technique only if there is a demonstration of destruction of the contamination at a useful rate. Thus, there will be a requirement for monitoring which likely will be extensive and may have to be continued for as long as the source emission is significant. The costs of this monitoring can seriously militate against the use of natural attenuation as a remedial process.

2.5.6 Summary on remediation processes

All the above remediation techniques involve the modification or management of the release characteristics of the source and often rely on the attenuation effects in the pathway(s) between the source(s) and receptor(s). The technologies all have a serious drawback in terms of the absolute clean-up achieved and in terms risk perception by the lay public, if they leave elements of the source in place – they seem to contain an element of either sophistry or alchemy. Furthermore, regrettable applications in the past, perhaps through ignorance as experience was developing, have given ample reason for deep suspicion. This places a heavy burden of proof on the engineer proposing anything other than total source removal as a remediation technology. Today, this burden of proof may be best discharged in an open forum allowing wide consultation based on a transparent methodology.

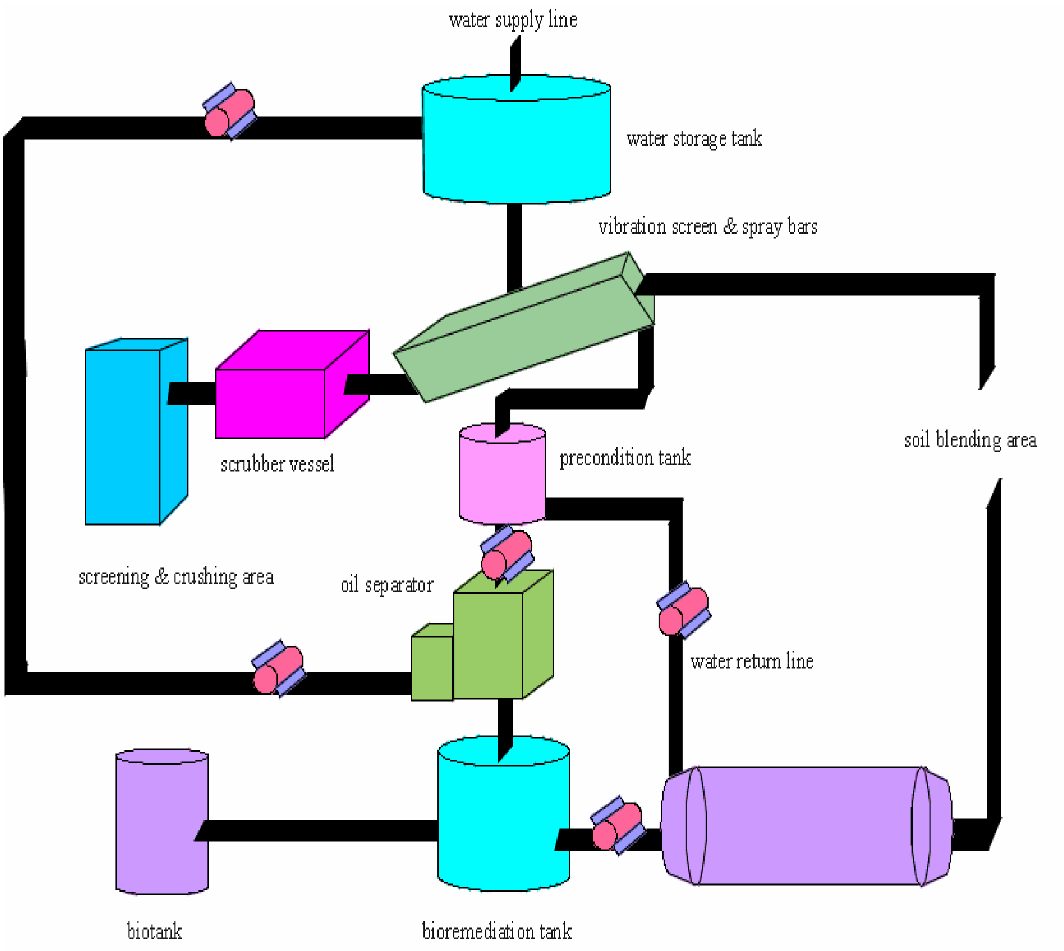


Figure 2. Schematic diagram of a soil washing and bioremediation plant

There are several other aspects that need to be considered when integrating remedial technologies into an active civil construction programme or simply cleaning up a dangerous site. One of these is that the retention of contaminated material on the site can have a profound impact on the total life cycle cost of the construction from the perspective of the site owner. If at some future point the works are recycled or demolished, the owner will again be confronted with the contaminated material which may now have increased in quantity as materials that were initially clean have been contaminated. From this standpoint, technologies which leave the source in place can be questioned as to their sustainability – they will be passing on contamination to a future generation. However, of course, some of the money or resources saved by leaving a source in place also may be passed onto the next generation and this could prove to be their preferred option, but we cannot yet know.

If the source is removed by dig and dump, then there will be a counter-concern by those adjacent to the landfill where the waste is disposed about its sustainability. We actually need decision tools that allow us to properly assess the bigger picture and not just the immediate contaminated land. These same problems apply to all techniques that require the provision, operation and maintenance of plant such as pumps or monitoring systems over long time periods.

2.5.7 Insurance

In addition to questions of sustainability, the more mundane problems of the long to very long term financial and liability issues have to be addressed. A new generation of insurance based financial instruments is emerging to help manage the financial consequences, although few products will offer cover for anything approaching 30 years and most legal and social frameworks cannot address liabilities extending more than about the period of a human lifetime.

The developments in legislation in the longer term are also unpredictable. Land is likely to become a progressively more pressured resource and ideas for regulations which today seem unthinkably restrictive, tomorrow may seem a matter of course.

2.5.8 The regulator

About 20 years ago, it became clear that integrated regulation of the environmental media (air, water and land) was essential to the proper management of environmental issues. However, it has now become clear that an integrated regulator, such as an environmental protection agency, is not sufficient. There also must be integrated law which is in line with the current environmental issues and not those of decades ago. New laws come out all the time, but rationalisation of old law is much slower and rarer.

For example, in England, waste management legislation has become a major hurdle for the sustainable re-use of materials. Waste soil from one site may not be re-used on another site without a waste management licence and ash from coal fired power stations (pulverised fuel ash, pfa) is a waste until 'recovered'. Pfa is often used in grouts, but in this role it will remain a waste and subject to licensing until 'recovered'. For pfa in a grout, recovery is defined as the point at which the grout has set. As noted earlier, waste management licensing is important, but so also is the need to ensure that the law and its ramifications are not so complex that landfill is a simpler option than re-use.

2.6 RISKS AND RISK BASED METHODOLOGIES

2.6.1 Risk analysis as a starting point

There are several possible starting points for developing procedures for the management of contaminated land including the application of the Observational Method. However, the most promising approach to date seems to be the one that takes Risk Analysis applied to the impact at specified receptors as its starting point. Furthermore, as has been noted above, risk analysis is at the heart of the English legislative regime – a regime which has been widely reviewed and it will be interesting to see the extent to which it is followed in other countries.

Risk analysis provides a context for the whole analysis and management of the contaminated site. It provides a common, if sometimes difficult, forum for communication of the problem to all concerned and it can be coupled with cost-benefit techniques to provide a very powerful management tool. Equally, the technique can be used in the field of pollution prevention design to assess the need for pre-investment and to avoid possible future problems.

Risk analysis accepts that whatever clean-up or management strategy is applied to a site, the contamination will not be totally removed nor will the site be returned to its pre-pollution state. There will always be residual risks after the corrective actions. The risk based methodologies described below are ideally suited to predicting and managing these residual risks.

In what follows, a description of this approach and its application will be presented. The methodology has several major disadvantages or issues and these will also be presented.

2.6.2 Risk and harm

The key distinguishing factor between risk based design methodologies and other more conventional ones is that they are based on a recognition that soil contamination is only relevant because it has the potential to cause harm. Or to put it another way, the existence of a chemical with hazardous properties in the soil is not an issue in itself. For this discussion, risk in environment related areas is most conveniently defined as:

Risk = Sum over all hazards (Probability of occurrence x Hazard that may occur)

The probability will be in the range 0 (hazard cannot occur) to 1 (the hazard will occur).

The risks may be to human health, financial, ecological etc. Thus, for example, at the conclusion of a risk assessment, risks may have been identified to human health (quantified as excess cancers); financial risks and risks to be wildlife (quantified as loss of local biodiversity). When presenting risk assessment results, there is often a desire to convert risks of all types a common measure e.g. money. This can be done if conversion factors can be derived. However, these factors are always contentious. There can be no absolute values and conversions will depend on contemporary social circumstances. Conversion to a single number can be useful when comparing alternatives, for example, Project A may cause damage of X money units but Project B, Y units. However, it must be recognised that for this type of comparison whether X > Y or Y > X will depend on the conversion factors adopted (except for the special case of X being better or worse than Y for all risk categories). A better form of presentation will set out all risks and benefits so that there is opportunity to consider mechanisms for the reduction of all risks and the optimisation of all benefits. Thus, reduction of risk to a single measure can be counterproductive.

Risk based methodologies assume that as a risk reduction procedure, the reduction of the probability of occurrence of a hazard is as acceptable as the reduction of the severity of the hazard itself. The management of probability by engineering design requires that the engineer has had appropriate training.

However, the management of probability may be codified in regulations and not left to the design of the engineer. For example, the European Union Landfill Directive defines wastes by their properties (i.e. the hazard they pose – flammable, harmful, toxic, corrosive etc.). The probability of harm from these properties is assumed to be related to the type of landfill to which they can be disposed e.g. hazardous waste, inert etc.

The risk based approach is in marked contrast to earlier procedures that sought to reduce the contaminant level at the site to a prescribed, and often legislated, value (i.e. which regulated just the hazard). It is this paradigm shift from source contaminant parameters to receptor impact values that brings with it the many advantages of risk based methodologies.

An important consequence of risk based approaches is that it is no longer possible to use design procedures that are based on prescribing management techniques or the properties of components of the remediation engineering. Only the functionality counts.

2.6.3 Types of risk from contaminated land

As discussed earlier (see Section 2.1.3) contaminated land may pose many types of hazard (harm) and so many risks, including:

- risk to human health;
- risk to flora and fauna including uptake in food chains;
- risk to the eco-system as a whole including bio-diversity;
- risk to the asset value of the site even though realisation of this value is not currently planned;
- risk to the use value of the site restricting its economic value;
- risk of incurring liabilities to others by cross boundary migration;
- risk of legislative non-compliance leading to fines or imprisonment;
- risk to the reputation of the owner or user of the site;
- risk to ground resources such as groundwater;
- risk to surface water bodies; and
- risk of loss of bearing capacity of the soil if highly soluble or reactive chemicals are present in the ground. Problems are relatively rare but have occurred.

Note that it will be necessary to consider all these risks both in the short term and in the long term.

It is clear from this list that the span of any suite of risk analysis techniques will have to be quite wide for them to be effective and will involve many different skills including chemistry, biology, toxicology, ecology etc.

Furthermore these risks often will occur concurrently, and as many of the categories overlap strongly, there is the potential for double counting.

The picture is further complicated by the fact that no two people appreciate and deal with risk in a same manner – it is difficult to be entirely rational and avoid all emotion in relation to risks (see the discussion on risk perception). It is important that risk assessors are properly trained in risk analysis. Unfortunately, far too often training is very limited. Risk assessors will also gain valuable insights if they have experience of accident investigation.

2.6.4 Key questions

Soil contamination problems give rise to five basic questions that lead to five basic problems:

1. What contamination is present? – The site investigation problem.
2. Where are the contaminants going to? – The migration problem.
3. If the contaminants get there, will there be a problem? – The impact problem.
4. If the impact is significant, what can be done about it? – The remediation problem.,
5. Has the remediation achieved the design objectives? – The validation and monitoring problems.

The first three problems can be integrated in the Source-Pathway-Receptor methodology, a key subset of risk analysis techniques. The last two problems are concerned with remediation technologies themselves rather than risk concepts and will not be considered further here.

The risk assessment methodology starts by identifying the three elements of the contamination management problem: the Source, the Pathway and the Receptor. The objective is to relate events, such an emission of toxic substances or other harm from a contaminated site, to their effects at some sensitive point, or 'receptor' in the environment. This relationship is a chain of sub-events, such as leaching followed by transport by groundwater and subsequent abstraction as drinking water, and is referred to as the 'Pathway' connecting the Source of risk to the Receptor where its impact will be felt. In the general methodology, the pathway will be represented by mathematical transfer functions which are more abstract than the physical flow of toxic substances.

2.6.4.1 The source of contamination

The Source is characterized by the nature and emission strength of the chemicals in the site. The Pathway is the chain of pollutant migration events as described above and is a vector. In general, there are multiple pathways between source and receptor, and the further removed these two are from each other, and the more members in the chain of events forming the Pathway that have to be considered, the more difficult the analysis.

The source is characterised by its nature and potential for emission as a function of time. This is controlled by the chemicals in the site, their properties such as solubility and mobility in the soil and their concentration. The concentration alone says little of the emission strength. For example, a site with 500 ppm of cadmium precipitated as a sulphide in reducing conditions is a very different risk than 5 ppm of the same cadmium present as a highly mobile sulphate in oxidising conditions. This distinction is of particular importance, as most sites contain a cocktail of pollutants whose behaviour can change with time or development. For example, dewatering during or as a result of development may allow oxygen ingress and so produce oxidising conditions whilst flooding may promote reducing conditions. The change of conditions can result in mobilisation of heavy metals and change of the speciation of sulphur, nitrogen, iron, manganese, arsenic etc.

The mobility of a pollutant in a site also will be influenced by physical factors such as temperature and pressure.

On any site there may be multiple sources and therefore multiple pathways as shown in Figure 3.

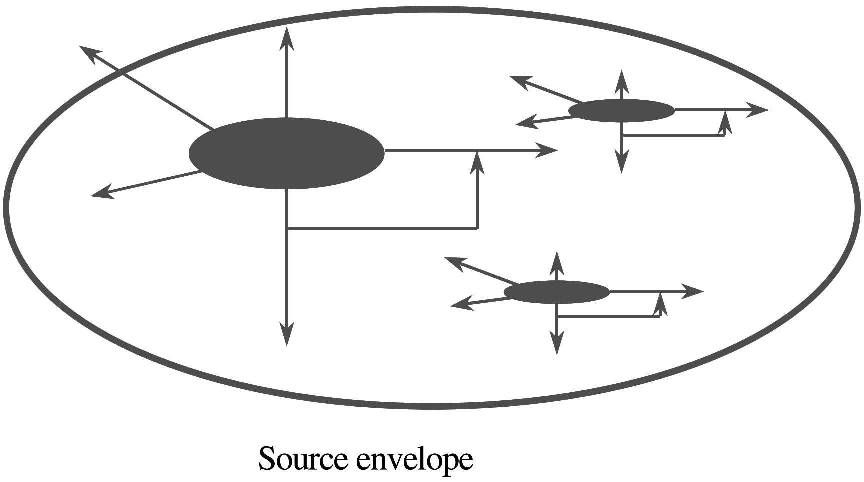


Figure 3. The envelope surrounding multiple sources

The source is the point on which risk perception tends to be focused. The language used can be very emotive: 'toxic dump', 'chemical time bomb', 'disaster waiting to happen'. Whilst the source may be the focus, emissions are the key to risk assessment.

Systematic and quantitative application of the Source-Pathway-Receptor methodology makes it possible to translate the environmental problem at the receptor to an emission characteristic at the source. In turn, this translation makes it possible to approach the soil remediation problem in terms of manipulating the emission characteristics at the source to achieve a desired risk level at the receptor.

2.6.4.2 The receptor

The receptor is defined by its sensitivity and its position in the environment. This is expressed as a maximum allowable impact value (harm) and is receptor specific.

For a living receptor (as opposed to a structure or service pipe etc.), the sensitivity of the receptor can be defined in toxicological terms if the detailed nature of the chemicals arriving at the receptor from the source and their required dose-effect relationships are known.

However, authoritative dose-effect relationships are not always available and mixture effects such as can occur with the complex cocktail of substances found on many sites are poorly understood. Nonetheless, there has been rapid progress for single contaminant analyses (and a few mixed systems such as hydrocarbons) and several risk analysis models such as RBCA from the American Society of Testing Materials or Risc Human from the van Hall Institute in Holland have extensive data bases associated with them. These allow very useful estimates of the impact at the receptor.

Although notionally starting from a risk concept, many countries have approached the problem of the limited available toxicological data and understanding by setting chemical concentration standards rather than impact values. These are the maximum allowable concentration, or MAC, based standards of which the old Dutch A, B, and C framework or the new target and intervention levels have received wide application. This can result in a drastic simplification of the problem which has the advantage of cost and speed for small sites, but it sacrifices some of the advantages and possible financial savings of the impact analysis route especially for large sites or significant contamination.

The recently published English Contaminated Land Exposure Assessment model (CLEA) provides a coherent framework within which guideline values can be developed, but as yet data toxicological data are limited. It may be noted that the CLEA model has introduced a new framework for the assessment of non-threshold carcinogens in England. The process for developing remedial criteria for these non-threshold carcinogens is no longer based on earlier ideas of acceptable risk levels (e.g. 1 in 10^5). The new criterion is that the dose to human receptors should be as low as reasonably practicable (ALARP) – a concept common in the nuclear industry. The no-threshold assumption means that there is no safe dose. In the DEFRA and the Environment Agency report, CLR9, 2002, it is stated that “for these species there is no theoretical reason why a single molecular exposure should not result in a tumour or a mutation, possibly expressed in subsequent generations though the lower the dose, the lower probability”. The ALARP criterion has the potential substantially to reduce the required clean-up criteria for non-threshold contaminants such as benzene and arsenic.

A serious issue when using the Source-Pathway-Receptor methodology is where along the pathway the MAC, ALARP or other impact values are to be applied. For example, if the receptor is groundwater, where should it be located, at an abstraction well or beneath the source?

The conceptual position around the problem where the impact values are to be applied is called the ‘receptor envelope problem’. Changing the position of this envelope can have profound influence on the range of remedial countermeasures available and the costs involved (see Figure 4).

For the risks other than those to health a different set of maximum allowable impact values has to be set. These can be financial or less quantifiable parameters which reflect elements of perceived risk as well as real risk.

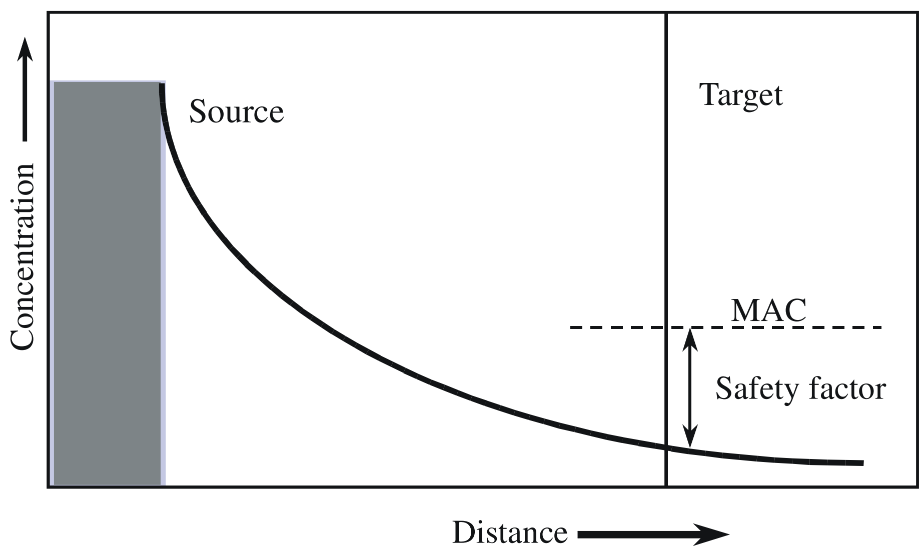


Figure 4. Effect of distance from source on contaminant concentration at the receptor (target)

2.6.4.3 The Pathway

A pathway connects a source with a receptor and is a vector characterised by direction and speed. A schematic of the general pathways for contamination is shown in Figure 5. The pathway may be simple, as in advection and dispersion through the groundwater system, or more complex and multi-membered such as a food chain pathway. Although physical pathways can be easily visualised, some of the risk areas mentioned above require more abstract pathway notions that are very difficult to work with and may be time varying as for air pathways where the dose may vary with wind characteristics and extreme values may be very different from average values.

The physical pathways can dilute, disperse and delay toxic emissions from the site, can provide opportunities for biodegradation or chemical stabilisation or conversely can result in re-concentration of toxins by bio-accumulation or sorption – possibly to higher values than in the original source.

For a typical site, at least the following pathways have to be considered:

- Leachate through the shallow groundwater system and local drains;
- Leachate to the deeper groundwater system especially where this is an important aquifer;
- Surface run-off of by erosion or leaching;
- Vapour or gas phase dispersion into surroundings;
- Dust (particulate) dispersion into the surroundings;
- Uptake into food chains; and
- Intrusion into the site (human or animals digging holes) and physical displacement by ground movements.

All these pathways can be operative at some point in the history of the site, from initial contamination, through the remedial actions to the long-term inevitable loss of institutional control of the contamination on the site.

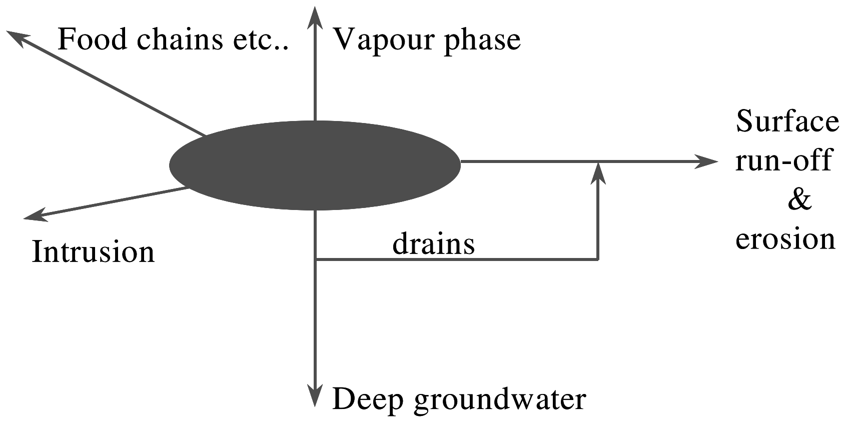


Figure 5. Pathways for contamination

2.6.5 Check list for Source-Pathway-Receptor analysis

The key steps in a Source-Pathway-Receptor analysis will include:

1. List all the possible Source-Pathway-Receptor combinations, keep an open mind, and include even those that may seem implausible or non-quantifiable.
2. Construct a data-set based on available information, common sense, similar sites and incidents or just plain guesswork.
3. Use the data set and an appropriate risk analysis model to assess the impact of the sources at the receptors. At this point many if not most of the potential source– path–receptor linkages will be eliminated leaving only a sub-set of plausible combinations.
4. Conduct a sensitivity analysis of the plausible linkages to identify those where the evaluation is sensitive to parameters that are poorly known. ('the phase 1 sensitivity analysis').
5. Carry out a cost-benefit analysis between the consequences of not knowing a parameter to a higher level of certainty against the cost of obtaining more information ('the need to know' principle).
6. Refine the 'need to know' parameters and repeat the steps to this point until there is sufficient confidence in the parameter database.
7. Evaluate the source emission characteristics that will reduce the impact at the receptor to below those maximally allowed.
8. Design and cost a remedial management scheme that will achieve such emission characteristics.
9. Evaluate the sensitivity of the impact at receptor to errors or failures in the design and/or implementation of the management scheme. ('the phase 2 sensitivity analysis').

10. If there is a significant chance of unacceptable failure, go on to perform a cost benefit analysis on reducing that chance. (The 'discrimination analysis').
11. Consider the extra cost of extra security. Will a small extra outlay significantly improve the reliability of the design?
12. Converge, by more than one cycle if necessary, to the final optimum management strategy.
13. It is very advantageous to formalise the above process so that it can be easily integrated into an IT based environment with an adequate quality system.
14. Consider the effects of time and degradation of materials or performance.

It is important to address the correct problem with the methodology. Many sites contain multiple sources (see Figure 3) and it is necessary to consider whether these should be treated together or separately in the analysis.

In summary, the Source-Pathway-Receptor design process changes the focus from the source to the receptor and thereby makes the source emission characteristic a key parameter. However, it is always necessary to keep failure states in mind. Even the best designs involve some risk of failure sooner or later and they had better fail safe.

2.7 COMPUTER BASED RISK ANALYSIS MODELS

In general, conducting a risk analysis as part of the risk based design task is too complicated and too prone to error (for example, when converting units etc.) to be done by hand. There are many risk analysis models, and the agency in many countries responsible for regulating contaminated land will provide its own model. In this section a brief overview will be given of the common features of some of these models.

All the modelling strategies approach the problem in a tiered fashion, applying steadily more complex models (called tiers or levels of analysis) until an answer can be given with a sufficient degree of confidence (or more often that sufficient site specific reduction parameters have been used to allow a remedial solution at an affordable cost). This answer has to have the required level of discrimination, by which is meant the ability to distinguish between the results of differing courses of action and a set level of confidence. This level of confidence will be established by sensitivity analyses.

The starting point of all the models is what is known as the Tier 1 level of analysis (see Figure 6). This level is essentially a look-up table in which the source soil concentrations are compared with what are assumed to be acceptable values by the regulators concerned. These values are called screening values. Sometimes the screening values are adjusted to reflect a few soil properties such as the organic matter content and the clay content and a few land uses (housing, industry, public park etc.). In the Americas, the USEPA or Canadian lists are usually used, whereas in Europe, there is a tendency to use the Dutch screening levels, although the impact of the new English CLEA model is yet to be seen. In fact, the screening levels are very similar for most models. However, as noted above, the ALARP requirement introduced for non-threshold carcinogens in English CLEA model has the potential significantly to reduce the values for these contaminants.

It is necessary to establish whether the mean, maximum or some other statistical measure of the source concentrations are to be used to decide on the site's compliance to the Tier 1 criteria. For example, a statistical procedure might be used to estimate the probable maximum concentration for each contaminant on the site. Where data are limited, estimated maximum values will be high – thus encouraging fuller site investigation.

The main problem with the Tier 1 analysis is that it is generic, and takes limited account of soil properties and land use. However, it has the advantage of simplicity and reduced effort and cost involved.

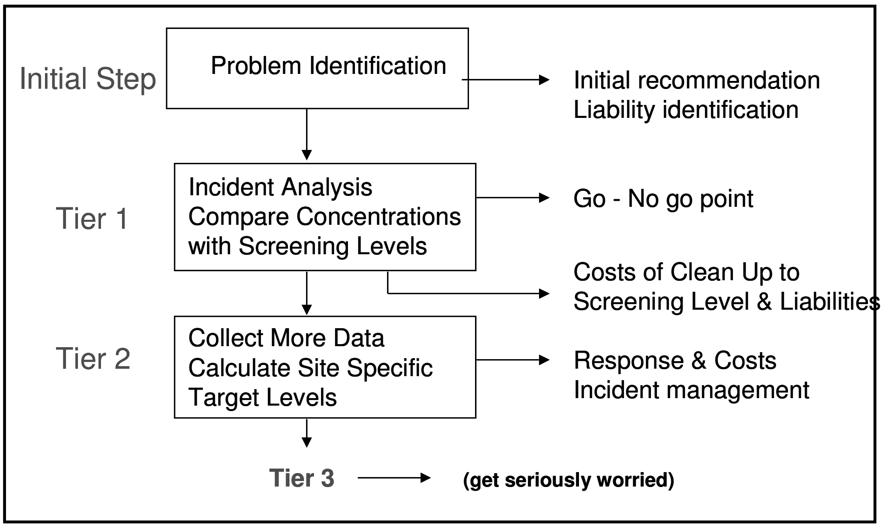


Figure 6. Tiered approach to computer-based risk assessment.

If the site fails the Tier 1 criteria, remediation may be undertaken to reduce site concentrations to the screening values. This is usually a poor strategy leading to excessive remediation and costs as it uses so little site specific information.

As more site specific data are used, involving more specific fitness for purpose information, the level of analysis moves from Tier 1 to Tier 2.

The Tier 2 analysis models work on the basis of calculating the impacts at one, or several, receptors that can be remote from the source. They assess the dynamics and the properties of the Source-Pathway-Receptor Linkage.

What distinguishes a Tier 2 model from a full custom designed, site specific, Tier 3 risk analysis model, is that many of the transport and impact functions are fixed in the model itself. This is called a ‘hard wired’ model. For example, the sub-model calculating the movement of vapour from the source to a receptor may be fixed even though there might be (and often is) a more appropriate model for the type of construction being built.

These models take the source data and problem dimensions and calculate the transport of contaminant to the receptor via the air, water and soil systems, finally making a total assessment of the impact on the receptor (normally just human health is considered) via the dermal, ingestion and inhalation routes. The physical and chemical parameters needed in the model are usually built in to its own database. In the model, the calculated dose results are then compared with the relevant toxicological standards to assess the need for contamination management.

The models can be run in inverse mode where the user sets up the Linkage and the model calculates the maximum allowed concentration or emissions from the source thereby giving a site-specific remediation objective to design to.

There are many Tier 2 models, most are PC based, and some can be downloaded from the Web. It is anticipated that soon, some of these models will be able to be run remotely over the Web alleviating the need for the geotechnical engineer to purchase them directly.

In their simplest form, these models are deterministic by which is meant that for fixed inputs they give a single fixed outcome. Some of the more sophisticated Tier 2 models can handle probability distributions as inputs and obviously give a probabilistic prediction of impacts. Probabilistic models include ConSim, LandSim and GasSim (contaminated land, landfill and landfill gas models, respectively) developed by Golder Associates for the UK Environment Agency.

The growing complexity of Tier 2 models is accompanied by heavier demands on the databases required to run them in a meaningful way. This is often the limitation to the model use.

If the problem is especially demanding or if there are special circumstances (e.g. large and complex sites, parts of which may fail a Tier 2 analysis), then it may be necessary to call on a professional contaminated land risk modeller who can construct a dedicated model from the basic transport and impact physics and chemistry. This is the Tier 3 model. Such modelling exercises are expensive, time consuming and should not be attempted lightly by the geotechnical engineer unless appropriately trained.

2.8 WIDER CONSIDERATIONS

Unfortunately, every methodology has its limitations and failings and this is neatly encapsulated in a saying attributed to the American statistician George Box: 'all models are wrong but some are useful'.

One of the general issues of the risk based methodology is its relationship to the precautionary principle and what risks are acceptable in society. The precautionary principle states in its most extreme form that, if all the effects and impacts of an action cannot be understood and predicted to be acceptable with certainty, then the action must not be taken. For contaminated land, this effectively requires the most rigorous clean-up but often forgets that the remedial measures themselves can bring uncertainties of their own. In relation to contaminated land, responses to the precautionary principle include:

- although not everything is known with absolute certainty, enough is known to discriminate between different courses of action;
- that doing nothing will lead to worse impacts than carrying out the course of action proposed. This leads on to the topic of conditional certainty and the application of the observational method in environmental engineering; and
- that resources expended on trivial risks makes the world less safe as it denies to more important risks, thus an assessment of relative risk is fundamental.

It should be remembered that the risk models themselves are fallible and in particular, they may be wrong because of:

- limited understanding of the processes at work (ignorance of the processes);
- limited available data and ignorance due to limited data; and
- the impossibility of fully incorporating random events such as floods, earthquakes etc.

This means that the actual outcome of designs can be different from that predicted and agreed with the authorities or client. In order to monitor and where necessary influence these outcomes it is necessary to set up a system of Risk Mapping. This means following key processes in the project that can lead to significant deviations from the desired results and only not intervening if the probabilities are below certain agreed 'conditional' levels. This process has very close parallels with the observational methods being employed in other areas of civil engineering and insights in methodology gained there can be applied here.

An issue that has already been touched upon and is related to the inadequacy of the relevant databases is the issue of toxicological data. Toxicological data are often very limited and can very difficult to apply especially for chemical cocktails. Models of the future characteristics of the emission curve, the failure mechanisms and the migration along the multiple pathways are all open to question and few if any have been verified for the time-scales involved. An especially difficult area is the incorporation of these uncertainties into a statistical assessment of the design performance. This situation is made all the more difficult by the heterogeneous nature of the highly disturbed soils in the vicinity of construction. Failure to cater for these uncertainties adequately can lead to massive over-design of the management strategy as large safety factors are built in at each stage of the design.

2.8.1 Extension to hazards other than human health

The argument to this point has concentrated on the risk to human health of the contaminants in the site and this is always the first starting point in any analysis (after all, at least it must be safe to go on the site to investigate). However, there are several other areas that the geotechnical engineer needs to be aware of.

The first is the impact of the contaminant on the ecology around the site. Sometimes large areas have been designated as areas of ecological importance or sites of special scientific interest. This means that the conventional risk analysis will have to be supplemented with an Ecological Risk Analysis. This is a highly specialised field (and still can be contentious) and the engineer will have to seek help. There are several ecological risk analysis models available but again they should only be deployed by specialists.

In principle, at least the bearing capacity and shear strength of soils could be influenced by contamination. This is a poorly reported area and effects are seldom seen except in extreme or bizarre circumstances. In contrast, the effect of the contaminants and their by-products on underground structures and services is better reported and understood. Well known examples of damage to services include the deterioration of concrete piles, the corrosion of steel sheet piling and the tainting of water supplied through plastics pipes.

Some solvents have a severe effect on geosynthetics. The geotechnical engineer should be careful to assess the compatibility of his proposed materials with the contaminants on the site. A conventional fault tree based risk analysis can be used for the risk mapping exercise here.

There are financial risks associated with contaminated land or the use of contaminated material in a construction. Next to the obvious ones of delay and extra costs in the work itself mention has to be made of the impact of perceived risk on the value of the finished product. Houses built on brownfield sites may be worth less and sell, to the first and perhaps second purchasers, more slowly than the same houses built of uncontaminated sites. Later purchasers may be unaware of the contamination unless problems have developed and thus sale values may be re-established. The situation largely depends on confidence – the perceived expertise, financial status and reliability of the developer. The geotechnical engineer should examine any strategies that might reduce the risk perception profile of the site when selecting the design and construction teams. Quality of work will be a key driver.

Finally there is the general risk to the reputation and standing of the geotechnical engineer. Again, this can be deserved as a consequence of inadequate professionalism or underserved as a product of his perceived role in the process. Many good reputations (and careers) have been irreparably damaged on contaminated sites.

2.9 METHODOLOGIES FOR DEVELOPING COUNTRIES

The text so far has concentrated on the position of more developed countries, developed both in terms of regulatory frameworks and income per capita to expend on pollution control. This is partly because regulatory frameworks tend to be more developed in countries with higher per capita incomes – the more people have the more they want including quality of the environment or inversely the less they have the less than can be concerned with the environment – there will be more pressing needs.

The principles for the management of contaminated sites in developing countries should not differ from those in industrialised countries. However, the implementation of these principles needs to take into account the applicable technical and institutional circumstances in the developing country concerned, the resources available and the other demands on these resources. The legacy of historically contaminated land in developing countries is generally not as great as in countries with a longer industrial heritage but the contamination which does occur can be no less intense and can pose similar problems of risks to the environment.

Furthermore, it should be recognised that contaminated land may not only be the result of an industrial heritage. For example, in Brazil it is estimated (Rose & Chih, 2001) that 74% by weight of domestic solid waste goes to uncontrolled dump landfills whilst only 24% goes to controlled and sanitary landfills and just 2% is recycled. However, much material may be recycled before being placed in a waste container as the average weight of garbage generated per person in developing countries is about half of that generated in developed countries. In Brazil, this amount to an average of 0.6 kg/day, although in developed regions the quantity generated is similar to that in developed countries – demonstrating the existing social contrasts.

The Source-Pathway-Receptor concept is the framework within which a strategy for management of a contaminated site is conventionally considered and is as valid in developing as in developed countries. This framework necessarily incorporates aspects of the climate and hydrology on a site, these usually provide the 'energising fluids' (the carriers) for the emissions which impact on the receptors. Developing countries are often within the tropical, semi-arid or arid climatic zones and the strategy for remediating contaminated sites needs to take into account the nature of the relevant hydrology and hydrometeorology. For example, the option of isolating a site using cut-off walls, liners or caps may not be appropriate if the identified contamination is not mobile simply for lack of a consistent leaching or driving fluid (e.g. water). For the same reason, natural attenuation may not be a realistic option also. Moreover, frequently in tropical countries, the groundwater table may be deep, below lateritic soils and subsoils of a low infiltration capacity, rendering the vulnerability of the groundwater relatively low though the result may be impact on surface water bodies. Furthermore, the sometimes arid conditions coupled with occasional intense rain can make very difficult conditions for containment measures such as caps and cut-offs which can become desiccated and cracked in arid times and will not have time to swell and seal (if they are capable of this) to be effective in times of intense rain. In short, to be effective, the technical strategy for dealing with contaminated sites in many developing countries may need to take particular account of the prevailing hydrological circumstances which may be quite different from those in northern latitudes.

Almeida et al. (2002a, b) report two examples of contaminated land investigation in Brazil. One of them relates to leakage of hydrocarbons in the oil production areas located in the wetlands in Northeast Brazil and the other with soil contamination by PCBs in a petrochemical industry in Guanabara Bay, Southeast Brazil.

Aside from technical considerations, equally important, in practice, in developing countries is the lack of an institutional and regulatory framework governing environmental protection in general and contaminated sites in particular. This requires effective policies and institutions to be in place by which site investigation and remediation managed.

Various reviews of environmental management in developing countries have been carried out in recent years (e.g. OECD, 1995), and the recommendations from such studies usually emphasise the opportunity for prevention of environmental ‘damage’ at the early stages of development prevailing in many such countries. However, the institutional and political capacity may not be in place to institute the appropriate protective measures. In relation to the proactive prevention of continuing or new pollution, there can be no justification for continuing pollution from existing facilities or new pollution from new facilities. Preventing pollution will add minimal extra cost to a process. The major requirement is a change of mindset for those operating plants or brining in new plants. This can be a major hurdle in pollution prevention.

Therefore, it is not uncommon in developing countries that a long time lag occurs between the site investigation (and risk analysis) and the actual site remediation. Almeida et al (2002c) report a clear example of this.

There is often a consequential need to strengthen the planning processes through which the development of contaminated land takes place. In turn, the national capacity to carry out environmental assessments is often weak through lack of human resources and infrastructure. For this capacity to develop, there are clear needs for training but also for raising environmental awareness in the community.

The direct benefits of remediating and developing contaminated land have to be appreciated in the technical and sociological context of the developing country concerned recognising the competing demands for financial, institutional and regulatory resources.

2.10 IRELAND - CURRENT STATE OF REMEDIATION

2.10.1 Background

The current situation in Ireland provides a useful case history of contaminated land activity in small country with a significant agricultural component to the economy.

The legacy of contaminated land in Ireland is small, largely because the industrial revolution and subsequent growth did not generate substantial industrial development. However, there are significant local areas of contaminated land in some urban areas such as Cork, Dublin and Limerick. Moreover, it has only been with recent development pressures in these urban areas that significant attention has been given to appropriate remediation strategies. Under the Irish Environmental Protection Agency (EPA) Act, 1992, the Environmental Protection Agency has overall responsibility for regulating such remediation, mainly through waste licensing, although some regulatory powers are delegated through the local authorities.

2.10.2 Key Sites

The most significant contaminated sites in Ireland are gasworks - around 40 in the Republic and a similar number in Northern Ireland. A variety of old chemical works (relatively small), fuel handling facilities and agriculturally related industries make up the remainder. It is difficult to estimate the size of the contaminated land problem but, at the outside limit it would not exceed a few hundred million euro.

2.10.3 Guidelines

The continuing lack of Irish national guidelines for soil and water remediation has hampered developers and often resort is made to imported guidelines from the UK or the Netherlands. The Irish EPA has formulated a tentative set of guidelines for soil and water remediation, which are at the consultation stage. The initial approach has been to prepare specific guideline concentrations, based on multifunctionality (multifunctionality is a concept developed in Holland requiring that the site should be fit for any use – e.g. an industrial works or domestic housing), but there have been recent developments on particular sites for which a risk-based approach has been accepted. As with other countries, the developers are not keen on such approaches because of the desire to declare a site 'clean' after development. However, the final approach to be taken by the regulators is not yet clear.

2.10.4 Present Strategies

The current approach generally adopted to remediate contaminated soil is to 'dig and ship' - apart from soil with relatively low levels of contamination. The options for removal to landfill are limited, partly because of extremely limited landfill capacity and also because of an Irish national ban on the disposal of hazardous waste to landfill. The option taken for all hazardous, and some less than hazardous, soils is to ship them abroad for treatment (landfill or, more likely, incineration) in the Netherlands, Belgium and UK. Some localized in situ bioremediation has been undertaken, particularly on hydrocarbon spill sites and, on one site, soil washing has been undertaken.

For groundwater, pump-and-treat has been a common approach, using a variety of filtration and treatment technologies, but such sites are relatively localized. On at least one site in Dublin, however, a permeable reactive barrier approach is being developed with control via a waste licence from the Irish EPA. The groundwater is contaminated by a variety of LNAPLs and the site is close to an estuary.

Under the economic pressures to develop inner city areas, there has been an increasing use isolation of as a remediation strategy – i.e. capping and cut-off walls. This has been justified through the use of risk analysis. Thus, in summary, the initial approach of adopting multifunctionality as a criterion for remediation has given way to a more risk-based strategy, albeit driven by economic imperatives rather than regulation so far.

11. SUMMARY

This chapter has presented a detailed analysis of the thinking that has gone behind the development of risk assessment as a procedure for the management of contaminated land. Risk based contaminated land management is now generally accepted as preferred procedure. Indeed, risk based methodologies have become so well developed and codified in many countries that contaminated land management is a low cost commodity product. In these countries, the geotechnical engineer may undertake assessment and remedial work according to a prescribed formula driven by the regulator or the company's practice. As a result, the geotechnical engineer may not be exposed to the mindset that developed risk based procedures and it is therefore hoped that the preceding text will be useful to the 'developed country' geotechnical engineer in explaining the thinking behind current practice which may be very important when working on leading edge remedial techniques or major sites.

For the 'developing country', geotechnical engineer the text may help to highlight some of the problems and pitfalls of developing contaminated land management and regulation procedures. Contaminated land management is necessarily not only expensive but also absorbs scarce material and institutional resources. Management procedures must be appropriate to the facilities available and not merely a copy of procedures in developed countries where priorities may be very different.

Since the time this text was drafted, sustainability has become perhaps a more important issue than purely environmental concerns. At its simplest, sustainability requires consideration of financial and social issues as well as the environmental/technical. The present text considers these issues and, therefore, is consistent with sustainability criteria. However, it is clear that sustainable geotechnics is the next step and Task Force 2 aims to address it in due course.

REFERENCES AND BIBLIOGRAPHY

As this chapter is concerned with the development of risk based approaches to contaminated land and not just current practice, the bibliography set out below includes some of the important works that have influenced the development process but may now seem rather 'dated'.

Adams, J., Risk, UCL Press, 1995.

Almeida, M.S.S, Barbosa, M.C., Schmidt, C.B, Souza, A. (2002a), Site investigation of fine soils contaminated with hydrocarbons, to appear in 4th International Conference on Environmental Geotechnics, Rio de Janeiro, August, 2002.

Almeida, M.S.S, Mesquita, A.C., Miranda, M.I. and Barbosa, M.C. (2002b), Geo-environmental investigation of a PCB plume contamination in a mangrove area, to appear in 4th International Conference on Environmental Geotechnics, Rio de Janeiro, August 2002.

Almeida, M.S.S, Barbosa, M.C., Almeida, J.L.D.S., Mariz, D.F. (2002c), Studies of contaminated channel sediments, to appear in 4th International Conference on Environmental Geotechnics, Rio de Janeiro, August, 2002.

Alloway, B.J., Editor, Heavy Metals in Soils, Blackie and Son Ltd., 1990.

Arvin, E., Degradation, retention and dispersion of pollutants in groundwater, Water Science and Technology, Vol 17, No. 9, 1985.

Cairney, T. editor, Contaminated Land Problems and Solutions, Blackie Academic and Professional Publishers, Glasgow, 1993 also 2nd edition, 1998.

Construction Industry Research and Information Association (CIRIA), Remedial Treatment for Contaminated Land, Published in 12 Volumes, London, 1994 on.

DocTer International, European environmental yearbook, London, 1990.

Hesse P.R., 1971, A textbook of soil chemical analysis, John Murray.

Freeze, R.A. & Cherry, J.A., Groundwater, Prentice Hall, 1979.

Gribble, G., The Chemophobia conundrum, Chemistry and Industry, 19th August 1991.

Institution of Civil Engineers, UK , Guidance on Contaminated Land, London, 1995

Jorgensen, S.E., Modelling in Environmental Chemistry, Elsevier, 1991.

Joyce, F.E., Strengths and weaknesses of Europe's environmental protection industry, Network for Environmental Technology Transfer Conference, Brussels, November 1989.

Kimbrough, R.D., Mahaffey, K., Grandjean, P., Sandoe, S-H., Rutstein, D.D., Clinical effects of environmental chemicals, Hemisphere Publishing Corporation, 1989.

LaGrega, M.D, Buckingham, P.L. and Evans, J.C., Hazardous waste management, Mc Graw Hill Inc., 1994 & 2nd Ed 2003.

Montgomery, J.H. & Welkom, L.M., Groundwater Chemicals Desk Reference, 1989.

OECD, Environmental Management in Developing Countries, pp417, Paris, 1995.

Rose R. and Chih K., Guide of Environmental Technologies Brazil Germany 2001-2002 (in Portuguese), Brazil, 2001.

Sheppard, C. Jefferis, S.A. and Jacobs, A., Tolley's Environmental Law and Procedures Management, Chapter on Contaminated Land Management, 2000 and later editions.

Smith, M.A., Contaminated land reclamation and treatment, Plenum Publishing Corporation, 1985.

The Hague Ministry of Housing, Physical Planning and Environment, Soil Clean-up interim act (Interimwet bodemsanering), 1984 and later publications.

The Hague Ministry of Housing, Physical Planning and Environment, Soil Protection Act (Wet bodembescherming), 1986 and later publications.

Examples of English publications on the regulation of contaminated land.

Note there is a separate legal system in Scotland and the regulations in Wales and Northern Ireland can differ from that in England.

Department of the Environment, A guide to risk assessment and risk management for environmental protection, 1995.

Department of the Environment Transport and the Regions, Circular, Environmental Protection Act 1990: Part IIA, Contaminated Land, 1 April 2000.

Department of Health, Report on Health and Social Subjects 42, Guidelines for the Evaluation of Chemicals for Carcinogenicity, HMSO, 1991.

Environment Agency and Department for the Environment, Food and Rural Affairs, CLR Publication Series, includes guidance on the UK contaminated land exposure assessment model, CLEA.

Her Majesty's Stationery Office, Planning (Hazardous Substances) Act, 1990, HMSO.

Her Majesty's Stationery Office, Environment Act, 1990.

Her Majesty's Stationery Office, Environmental Protection Act, 1995.

Health and Safety Executive, 1991 Protection of workers and the general public during the development of contaminated land, HMSO, 1991.

Interdisciplinary Committee 0n the Reclamation of Contaminated Land (ICRCL) Guidance Note 23/79, Notes on the redevelopment of sewage works and farms, 2nd edition November 1983.

Interdisciplinary Committee 0n the Reclamation of Contaminated Land (ICRCL), Guidance Note 59/83, Guidance on the assessment and redevelopment of contaminated land, 2nd edition July 1987.

Ministry of Agriculture, Fisheries and Food, The Analysis of Agricultural Materials, Reference Book 427, 1986.

APPENDIX 1 - GLOSSARY OF TERMS

INTRODUCTION

This glossary sets out some of the chemical and related terms used in contaminated land practice. It is based on UK practice and it should be noted that slightly different terminology or contaminant groupings may be used in other countries.

CONTAMINANT GROUPINGS

It has become common practice in contaminated land discussions to group chemicals together. Reasons for the groupings include:

- they tend to occur together in contaminated land;
- analytical techniques may be similar;
- they have similar physical properties;
- they are similar chemically.

Use of chemical groupings can save time in discussions, for example, with analytical laboratories. However, it should not be assumed that everyone will have the same understanding of the definition/members of a group. Examples of groups and possible definitions are as follows:

Aliphatics	Organic hydrocarbons (chemicals containing carbon and hydrogen and/or other species) with straight or branched chains but no aromatic ring structures.
Aromatics	Hydrocarbons containing one or more six carbon, benzene rings.
BTEX	Benzene, toluene, ethyl benzene, xylene typical of the aromatic fraction of hydrocarbon mixtures such as petrol.
Chlorinated aliphatics	Aliphatic compounds with chlorine atoms attached to one or more carbon atoms. Examples include vinyl chloride, dichloromethane, trichloromethane, tetrachloromethane (i.e. methane with one to four hydrogens substituted by chlorine), trichloroethene, tetrachloroethene, 1,2 dichloroethane – all these examples are also VOCs and also may be referred to as chlorinated solvents.
Chlorinated aromatics	Aromatic compounds containing chlorine, e.g. chlorobenzenes, hexachlorobenzene, PCBs, PCDDs, PCDFs, chlorophenols, chloronaphthalene.
DNAPLs	Dense non aqueous phase liquids. Any chemical or mixture of chemicals that can form a separate phase which sinks through water (i.e. is denser than water and is present in sufficient quantity that it is not all dissolved into any available water, e.g. local groundwater). See also LNAPLS and NAPLS.
DROs	Diesel range organics, heavy hydrocarbons of low volalility.
Dry residue	Total solids in a water determined by evaporation to dryness at a specified temperature, expressed as mg/litre.
EC	Electrical conductivity, units are likely to be milli or micro Siemens. See also total dissolved solids (TDS).

Heavy metals	Literally metals with a high specific gravity, for example >5. Generally potentially toxic metals, typically transition metals such as chromium, manganese, nickel and copper. However may include other elements such as lead, mercury, selenium, boron, arsenic.
LNAPLs	Light non aqueous phase liquids. Any chemical or mixture of chemicals that can form a separate floating phase on water (i.e. is less dense than water and is present in sufficient quantity that it is not all dissolved into any available water, e.g. local groundwater).
MTBE	Methyl tertiary butyl ether an oxidant used in petrol – especially unleaded petrol.
NAPLS	Non aqueous phase liquids (see DNAPLS and LNAPLS). Note: some non aqueous phase liquids may be of similar density to water and thus be neutrally buoyant (no tendency to float or sink). However, they may have different temperature coefficients of expansion and thus become DNAPLS or LNAPLS depending on temperature.
PAHs	Polycyclic aromatic hydrocarbons, species with two or more benzene rings fused together with at least two common carbons e.g. naphthalene (2 rings), anthracene and phenanthrene (3 rings) and benzo(a)anthracene, chrysene and pyrene (4 rings) and benzo(a) pyrene with 5. Fluoranthene, benzo(k)fluoranthene and indeno(1,2,3-cd)pyrene are examples of more complex systems including a five membered ring.
PCBs	Polychlorinated biphenyls. Biphenyl consists of two benzene rings linked through one carbon atom on each ring. In chlorinated biphenyls, some or all of the remaining the carbon sites are chlorinated. There are 209 different PCBs (different congeners) depending on the degree of chlorination and the position of the chlorines on the benzene rings. Contamination will consist of a mixture of PCB congeners of different degrees of chlorination and different toxicities. PCBs were used in transformers, capacitors etc.
PCDDs	Polychlorinated dibenzo-p-dioxins. A class of chlorinated compounds containing two benzene rings linked at two carbon atoms through oxygen atoms.
PCDFs	Polychlorinated dibenzo-furans (furans include five membered rings, four carbons and an oxygen).
Phenols	A class of aromatic hydrocarbons in which one or more hydroxyl groups (OH) are attached directly to a benzene ring. The simplest member of the class, phenol consists of a benzene ring with a single hydroxyl group. It is also known as carbolic acid.
SVOCs	Semi volatile organics compounds. SVOCs will include PAHs.
Tars	By-products of the distillation of coal and other organics. Likely to be rich in PAHs.
TDS	Total dissolved solids determined by evaporation to dryness, expressed as mg/litre. May be determined by evaporating a water sample to dryness (see Dry residue) but often estimated from electrical conductivity.

TEM Toluene extractable material. An indicator of the quantity of low volatility organics in a sample. The sample is extracted with toluene and the extract allowed to evaporate and the residue weighed. May be useful for coal tars etc. Elemental sulphur is extractable and will be included in the TEM.

TPH Total petroleum hydrocarbons. May be subdivided by carbon chain length e.g. aliphatic C_5 - C_8, C_9 - C_{16}, C_{17} - C_{35} and aromatic C_6 - C_8, C_9 - C_{16}, C_{17} - C_{35}.

VOCs Volatile organic compounds. Any organic compound which evaporates easily at ambient temperatures. VOCs may cause stratospheric ozone depletion. Examples include chlorinated solvents and benzene.

OTHER TERMS

ADI Acceptable daily intake.

ALARP As Low As Reasonably Practicable – a standard for assessing necessary control measures taking into account the practicalities of the task in hand. ALARA As Low As Reasonably Achievable.

Anthropogenic Artificially produced (man made)

CERCLA Comprehensive environmental response, compensation and liability Act, 1980 (USA).

CLEA Contaminated land exposure assessment model, a key risk assessment tool for use with the Part IIA Statutory Guidance on contaminated land.
See www.environment-agency.gov.uk/subjects/landquality/ and the DEFRA website

Contaminant A substance which if present on, in, or under the land has potential to cause harm or water pollution.

COSHH Control of substances hazardous to health regulations, 1995 (UK).

Ingestion Contaminant entering the stomach and gastrointestinal tract through eating contaminated food or hand to mouth contact.

Inhalation Breathing in through the nose or mouth, e.g. particulate material and vapours.

IRIS Integrated risk information system toxicity data on chemicals.

HEAST Health effects assessment summary tables.

LEL, UEL Lower, Upper explosive limit. Volume concentration of a gas in air at which an explosion can occur. An explosion may not occur if there is too little of the gas (lower limit) or too little air (i.e. to much gas - upper limit). These limits will be important for gases such as methane and hydrogen.

LOAEL	Lowest observed adverse effect level. A term used in toxicology, the lowest dose at which some adverse effect is seen.
MAC	Maximum acceptable concentration, an upper limit on concentrations of chemicals in water (or soil) – used as a control values in certain jurisdictions.
MCL	The USEPA has established Maximum Contaminant Levels (MCLs) for concentrations of certain contaminants in public drinking water supplies.
MTR	Maximum tolerable risk.
NOAEL,	No observed adverse effect level, see LOAEL.
NOEL	No observed effect level.
Pathway	A route or means by or through which a receptor is or could be exposed to a contaminant.
PPE	Personal Protective Equipment. Often erroneously considered as the only necessary defence against contamination. Should be used only as a last line of defence after other procedures such as elimination or reduction of risk of exposure.
RBCA	Risk-based corrective action. A procedure for contaminated land assessment developed in the USA but now used more widely.
Receptor	Something which could be harmed by a contaminant such as human health, other living organisms, ecological systems, property or controlled waters. Formerly referred to as a target
RfC, RfD	Reference concentration or dose.
Target	See receptor
TDI	Tolerable daily intake. A dose parameter used in risk modelling.

Chapter 3

Traditional and Innovative Barriers Technologies and Materials

PART 1 – TRADITIONAL BARRIERS

Authors:
Craig H. BENSON
University of Wisconsin-Madison, USA
Abdelmalek BOUAZZA
Monash University, Australia
Evelina FRATALOCCHI
Università Politecnica delle Marche, Ancona, Italy
Mario MANASSERO
Politecnico di Torino, Italy

PART 2 – INNOVATIVE BARRIERS

Authors:
Charles D. SHACKELFORD
Colorado State University, Fort Collins, USA

Contributing Authors:
Craig H. BENSON (§ 3.1.1)
University of Wisconsin-Madison, USA
Abdelmalek BOUAZZA (§ 3.3.3)
Monash University, Melbourne, Australia
Luiz DE MELLO and Maria Eugenia BOSCOV (§ 3.3.7)
University of Sao Paulo, Brazil
John BOWDERS (§ 3.3.4)
University of Missouri-Columbia, USA
Rolf KATZENBACH (§ 3.3.5)
Darmstadt University of Technology, Germany
Horace MOO-YONG (§ 3.2.4)
Lehigh University, USA
Tom ZIMMIE (§ 3.2.4)
Rensselaer Polytechnic Institute, USA
Jorge G. ZORNBERG (§ 3.2.1, 3.2.2)
The University of Texas–Austin, USA

The authors gratefully acknowledge the review of this chapter and the contribution made to it by:

John BOWDERS, *University of Missouri-Columbia, USA*
Russell JONES, *Golden Associated, Nottinghamshire, United Kingdom*

PART I – TRADITIONAL BARRIERS

3.1 TRADITIONAL LINERS AND LINER MATERIALS

At present, the overall layout and general design and construction procedures of barrier systems can be considered framed and addressed (Daniel, 1993; ETC8, 1993; Rowe et al., 1995a; Rowe, 1998a; TC5, 1998). The main research streams are concentrated on the assessment of potential contaminant impact on the subsoil environment:

- evaluation and quantification of some key factors which govern the field scale performances of mineral (CCL and GCL) and polymeric (GM) barrier components;
- evaluation of the service life of mineral and polymeric barrier components;
- stability of slopes involving composite liners;
- set up of specific models and related parameters for risk assessment of pollution potential;
- adequacy of the present regulations to address the use of new products and alternative design options.

Some of these aspects will be discussed in this chapter.

The design of bottom barriers of modern landfills should be based on the following main principles:

- The mineral barrier is the basic component of traditional sealing systems referring in particular to the long-term performance.
- The requirements and characteristics of the mineral sealing layer in order of importance are: (1) low hydraulic conductivity (HC) at field scale, (2) long-term compatibility with the chemicals to be contained, (3) high sorption capacity, and (4) low diffusion coefficient.
- Composite lining systems using geomembranes can give important advantages both in the short and long-term due to: (1) reduction of HC as a result of the attenuation of defects of both geomembrane and compacted clay (Giroud & Bonaparte, 1989a;b; Giroud et al., 1992; Daniel, 1993); (2) better biogas control; (3) minimization of desiccation problems; (4) enhancement of flow within the drainage layers toward the collection pipes (i.e. minimization of ponding leachate on the liner) and (5) the geomembrane on the top of the clay barrier delays contact between clay and leachate long enough for consolidation of the clay when the waste is landfilled, thus reducing compatibility problems (Rowe et al., 1995a).
- Construction procedures play a fundamental role in the final efficiency of the lining system in terms of field-scale HC.

3.1.1 Compacted clay liners (CCL)

3.1.1.1 Hydraulic Conductivity

The HC of CCLs at the field scale has been discussed and developed in the late 80's and 90's. The main results and recommendations are related to the effects of compaction procedure, clay water content and pretreatments (Daniel, 1993; Daniel & Koerner, 1995).

Uncertainties in construction, flow through macropores and spatial variability of the hydraulic properties of compacted clay liners have been treated statistically and validated via field tests (Figure 1) by Jessberger et al. (1993), Benson & Daniel (1994a, b) Benson et al. (1999) among others. Their main conclusions are:

- A mineral sealing layer, consisting of four or more lifts, compensates for the effect of spatial variability of the HC. So, the recommended minimum thickness for compacted soil liners is four to six lifts or 0.6 to 0.9 m.. There is little benefit if the number of lifts is increased above 4 to 6.
- High quality of CCLs can be considered achieved if the randomly measured HCs using standard tests during construction is 3 to 5 times smaller than the value expected for the field-scale liner.

Minimum testing frequencies are summarized by Daniel (1990).

The kinds of field and laboratory tests for the assessment of CCLs HC have been analyzed by Daniel & Trautwein (1994). The representative dimensions of samples for reliable tests depends on the method and quality of construction (Benson et al., 1994; Trautwein & Boutwell, 1994). If the soil is compacted poorly, the representative specimen size should be very large. When the soil is well compacted, the representative size is close or equal to the dimension of standard laboratory test specimens. Similar results and observations have been confirmed by field data (Figure 2).

The index properties of some CCLs are reported in Figure 3 together with the limit values, suggested by Jones et al. (1993) and Daniel (1989) in order to succeed in obtaining low HC at the field scale. The database of Daniel (1997) shows that there is little influence of the index properties on k_{field}. Other factors appear to be far more important for complying with $k_{field}<10^{-9}$ m/s requirement. Considering that 75% of the CCLs did achieve the objective of $k_{field}<10^{-9}$ m/s and that 25% of the total did not, it is interesting to show and comment the results by Daniel (1998) as reported in Figures 1.b and 4.

The Daniel's data in Figure 1, together with the results of field large scale tests from some Italian landfills (Manassero et al., 2000), are in agreement with the range of variation of k_{field} vs CCL thickness found by Benson and Daniel (1994a;b). There is a clear trend showing that hydraulic conductivity decreases with increasing thickness of the clay liner. It is worth to note that most of CCLs examined by Daniel (1998) lay in the field of good to excellent construction. In spite of these evidences Fuleihan & Wissa (1995) strongly argued against the supposed influence of the thickness on the actual HC of CCL showing experimental data that in some cases seem to contradict the aforementioned trend. According to Manassero et al. (2000) it is physically and statistically sound to find a significant influence of the thickness and number of lifts on the actual HC of CCLs.

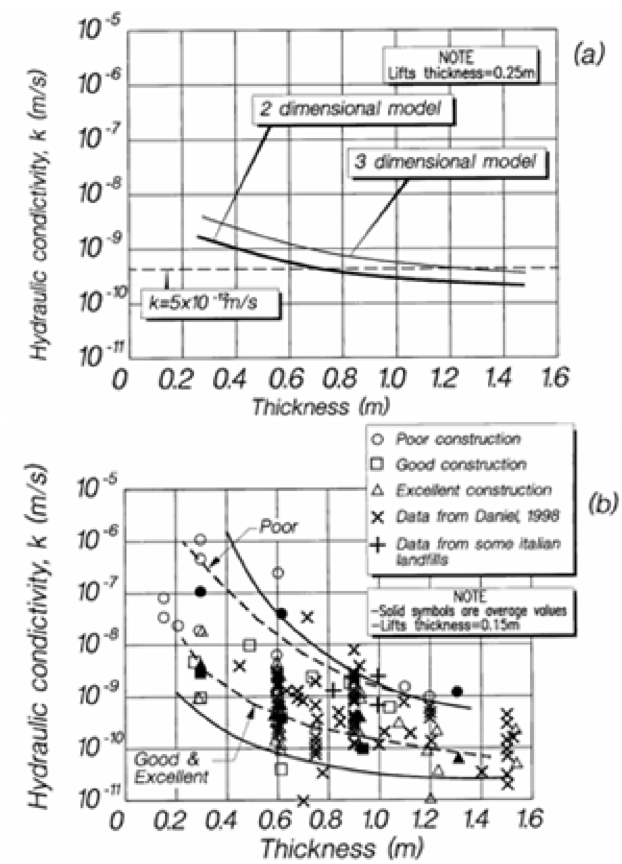


Figure 1. Hydraulic conductivity of compacted soil liners versus thickness: (a) theoretical assessment, (b) experimental trend from field data (Jessberger et al., 1993; Benson & Daniel, 1994a, b)

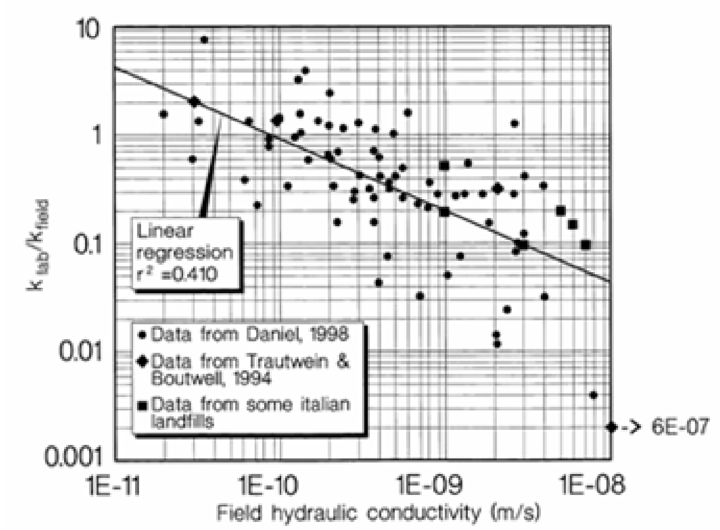


Figure 2. Ratio of laboratory (k_{lab}) to field (k_{field}) HC vs field HC (Manassero et al., 2000)

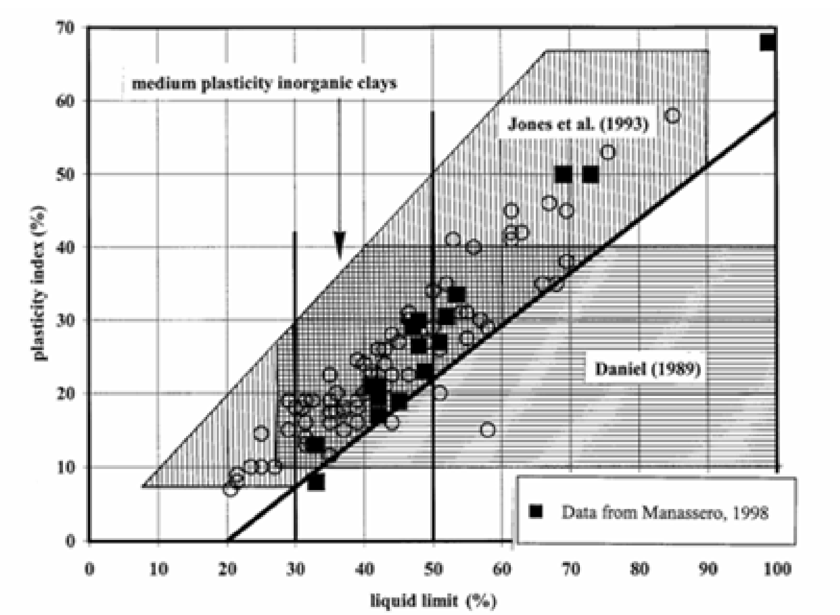


Figure 3. Index properties of some compacted clays (Manassero et al., 2000)

Figure 4 shows field HC versus the percentage of water content-dry density (w-γ_d) points laying above the line connecting the peaks of compactions curves of different compacting energy. A rather strong correlation between these two parameters is evident. Nevertheless, also a comparable influence on k_{field} of the simple water content referred to the optimum standard Proctor has also been observed by Daniel (1998).

Summing up, the two most important parameters for a good performance in terms of field HC, identified by Daniel (1998), are the water content (w) and the total thickness of the liner (L), given a suitable clay and a sufficient compaction energy.

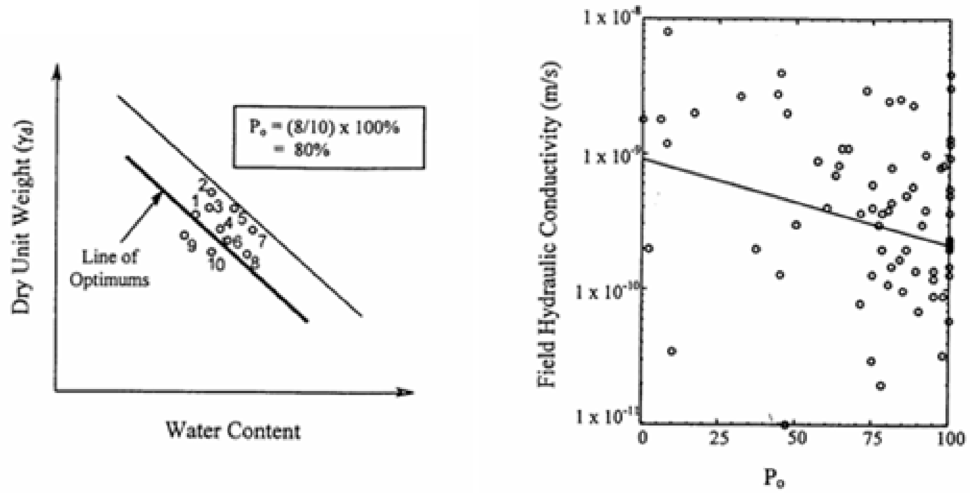


Figure 4. Relationship between field HC and percentage (P_0) of tests results laying above the line of optima (Daniel, 1998)

3.1.1.2 Compatibility

As chemicals migrate through clay soils that contain appreciable amount of clay minerals, such as CCLs and GCLs, interaction between the chemicals in pore water of the soil and the clay particles can result in significant increase in the HC of the soil (Shackelford, 1994a; Shackelford et al., 2000). While the potential effects of these chemical-soil interactions are well recognized, and may be significant, such interactions are rarely included in the modeling of contaminant transport through waste containment liners mainly because of the lack of standardized procedures and reliable parameters. Nevertheless many studies and research programs have been performed on chemical compatibility (Mesri & Olson, 1971; Acar & Seals, 1984; Fernandez & Quigley, 1985, 1989, 1991; Bowders et al., 1986; Mitchell & Madsen, 1987; Dragun, 1988; Quigley et al., 1988; Quigley & Fernandez, 1989; Wagner et al., 1990; Manassero & Shackelford, 1994a,b; Rowe et al., 1995a; Pasqualini & Fratalocchi, 2000). On the basis of these studies, several types of direct and indirect compatibility tests have been proposed, such as modified Atterberg limits, sedimentation, swelling, cracking pattern, setting, exposure or immersion and permeation (Shackelford, 1994b). However, a lack of quantitative and practical indications exist to define conditions that can be critical for compatibility problems. A procedure for addressing these aspects has been attempted by Manassero & Shackelford (1994a). This procedure, valid for organic contaminants, allows taking into account the activity of the compacted soil, the dielectric constant of the pollutant solution, the contaminant concentration and density. Further research and validation are needed before the proposed procedure can be used reliably. A possible extension to inorganic pollutants can also be addressed. For more details see Shackelford (1994b). In general, soil liners perform much better at high compressive stress; this is important to keep in mind when compatibility tests by permeation are performed for design purpose.

The results by Kaczmarek et al. (1997) indicate that the potential effects of compatibility problems are controlled by the magnitude of the concentration at the source boundary of the clay barrier. For low to moderate concentration of the contaminants at the source boundary, the diffusive component controls the transport such that no significant changes of HC occur in the clay barrier. Significant increases in HC are likely to occur only in the case where an extremely high concentration is imposed at the source boundary. In this case, the HC of the clay barrier may increase to the extent that the advective component of the transport eventually dominates the diffusive component. Thus, some consideration should be given to the potential influence of contaminant concentration in the leachate on the long term performances of the barrier (Bowders and Daniel, 1987).

3.1.1.3 Desiccation

Desiccation of CCLs can occur before placing the geomembrane, after placing the geomembrane and before covering with wastes, and after placement of waste (Rowe, 1998a).

Desiccation due to water evaporation before the placement of a geomembrane on the CCL can be readily prevented by appropriate construction procedures and, if it does occur, can be observed and rectified by removal and replacement of the cracked portion.

A high temperature of the geomembrane has the potential to cause evaporation of water from compacted clay into any air space between the clay and the geomembrane (e.g. wrinkles and waves), and moisture movements from the region of higher temperature to the region of lower temperature (Rowe, 1998a). These phenomena can be particularly critical along landfill slopes and in the case of heating and cooling cycles. In order to avoid desiccation of CCL after the placement of the geomembrane it is important to avoid as much as possible wrinkles and waves of the geomembrane and to cover as soon as possible the geomembrane with the protection layer and/or the leachate collection layer (Rowe, 1998a; Bowders et al., 1997).

The temperature of the waste in close contact with the lining system of the landfill can have a significant effect on the rate of clogging of the leachate collection system, on the service life of geopipes and geomembrane liners, on diffusion through low permeability liners and, on desiccation problems of compacted clay liners. The temperature of the waste body appears to be related to the water content of the waste and the level of leachate mounding (Rowe, 1998a). This highlights once again the importance of the design, construction and operation of the leachate collection system.

The risk of CCL desiccation after waste emplacement depends on: properties of the clay, properties of the underlying subgrade, overburden pressure, temperature gradient across the liner, and the depth of groundwater (Holzlohner, 1989, 1995; Doll, 1996).

The basic influencing properties of clay are the initial saturation degree, the tensile and shear strengths in both drained and undrained conditions, the matrix suction and the parameter of stress distribution between pore water and air pores. Overburden pressure can help to reduce cracking problems in conjunction with a sufficiently low shear strength of the clay (ETC 8, 1993). The distance of the water table, the unsaturated HC of the subsoil below the liner and the capillary rise can play a fundamental role on potential CCL desiccation together with the thermal gradient.

Heibrock (1997) performed thermodynamic analyses which predict no cracking in 50 years with a temperature at the top of the CCL of 25°C but predicts cracking to a depth of 1 m in 20 years with a temperature at the top of the liner of 40°C with all the other parameters being constant. These calculations are strongly dependent on the clay of the barrier, subsoil parameters, and boundary conditions, therefore it is not possible to generalize the aforementioned example. Nevertheless these results do underline the need to carefully consider the potential for temperature induced cracking after waste emplacement and the desirability of limiting the temperature at the top of the liner to the extent allowed by the design and operation of the landfill.

Finally, it is important to outline that the potential water movement due to temperature gradient could be controlled by installing a geomembrane below the CCL as well as above it (Rowe, 1998a). It is also important to take into account that a drainage layer below the CCL, not protected by a geomembrane, can increase the potential for desiccation both by preventing any movement of underlying groundwater upward into the CCL and by transporting the vapor from the subsoil to the drainage layer and then out of the drainage layer due to the changes in atmospheric temperature and pressure. Therefore it would seem prudent to design landfill lining systems such that the potential for moisture loss from the secondary drainage layer due to atmospheric pumping is minimized and to have a thick rather thin compacted clay primary liner (Rowe, 1998a). Moreover the introduction of water inside the secondary leachate collection system could be positive for both reducing desiccation problems and minimizing advective transport via decreasing hydraulic gradient through the primary liner.

3.1.1.4 Sorption

Provided that clay-leachate compatibility problems are not induced, the sorption capacity can be considered one of the most effective properties of fine grained soils for the control of pollutant migration through mineral barriers. Although there is a wide range of possible chemico-physical interactions between contaminants, mineral porous media and pore liquids, the main sorption mechanisms which are considered for pollutant migration modeling through containment mineral barriers are:

- ionic exchange on clay particle surface in the case of inorganic compounds (e.g. cations such as: K^+, Na^+, Pb^+, Cd^+, Fe^{2+}, Cu^{2+}, Ca^{2+}, Mg^{2+}).
- sorption of organic compounds by the organic carbon of the solid skeleton on the basis of laws as:

$$K_d = K_{oc} \cdot f_{oc} \tag{1}$$

where K_d is the distribution coefficient; K_{oc} is the carbon-octanol partition coefficient and f_{oc} is the organic carbon content of the solid skeleton.

In general, the sorption phenomena in the advection-dispersion-reaction equation (ADRE) solutions are modelled with linear equilibrium isotherms. However, highly non-linear isotherms as well as non-equilibrium sorption can be observed in batch test sorption kinetics when some clayey soils and chemical compounds are combined. When sorption kinetics are significant for contaminant transport, the following non-equilibrium first-order model can be used (Rabideau et al., 1996):

$$\frac{\partial S}{\partial t} = \alpha(S_\infty - S) \tag{2}$$

where S is the solute mass fraction in the sorbed phase, α is the sorption rate coefficient, S_∞ is the sorption capacity at $t=\infty$ if concentration, c, is kept constant with time and can be either defined according to the linear isotherm $(K_d \cdot c)$ or according to the Freundlich isotherm $(K_F \cdot c)^\beta$ or Langmuir isotherms $(S_m \cdot b \cdot c \,/\, (1+b \cdot c))$; with k_F [L^3/M] and β [-] the Freundlich's constants, and b [L^3/M] and S_m [-] the Langmuir's constants. In particular, S_m is the maximum sorption at high equilibrium concentrations, k_F and b are the affinity parameters between solid and solution; while β is an empirical parameter that varies with the degree of heterogeneity of charges onto the solid surface. The concentration c in this case is defined as mass of solute per unit volume of solution.

When these kinds of sorption models are used, the ADRE in the most general form can be solved only via numerical techniques. The sorption capacity of materials for mineral sealing layers can be assessed performing batch tests or from interpretation of simple diffusion tests and column tests (Shackelford, 1994b).

As already mentioned, most of the available ADRE closed-form solutions used for diffusion and column test interpretation consider only linear isotherms. Therefore, the interpretation results should be referred to an average concentration in the pore liquid of the mineral sample. In the case of diffusion tests performed with a single reservoir with decreasing concentration, Manassero et al. (1998) suggest to refer to the average solution concentration in the pore liquid in the soil sample at the end of the test. In the case of advective-diffusive column tests, the reference concentration can be taken equal to one half of the source concentration c_0, as a first approximation, if c_0 is constant during the test.

Laboratory batch contact tests carried out by Manassero et al. (1998) using a potassium-bromide (KBr) solution and a natural clayey-silt led to the following considerations which should be related only to the materials and methods used for this research:

- as far as sorption kinetics is concerned, the standard contact time of 24 hours is enough to approach satisfactory equilibrium conditions for any soil to solution ratio;

- even though some variations can be noticed on the measured solute concentrations from centrifuged or squeezed samples, the observed differences can be considered negligible in terms of modeling contaminant transport through mineral barrier materials;
- the soil to solution ratio (s/w) is the most important parameter influencing the sorption capacity results; therefore, the s/w ratio must be always taken into account when determining the sorption parameters to be used in the contaminant transport modeling;
- the shape of sorption isotherms can be significantly influenced by a slow and progressive increase of the solute concentration in contact with the clayey soil; in any case, the different shape of the sorption isotherms plays an appreciable role within the contaminant transport model used for the interpretation of a decreasing source single reservoir diffusion (DSSRD) test, at least referring to the considered range of solute concentrations.
- Freundlich's and Langmuir's isotherm parameters obtained by batch-contact tests at s/w ratios ranging from 1/4 to 1/1 and extrapolated to s/w ≥ 3/1 seem to give consistent and reliable input data for modeling contaminant transport through compacted clay and silty materials (Figure 5).

It is possible to generalize the observations related to the aforementioned specific sorption tests to a more general context stressing the strong influence on the bulk sorption capacity of parameters such as pH and the soil-to-solution mass ratio (s/w) (Shackelford & Redmond, 1995; Rowe et al., 1995a; Manassero et al., 1996). In particular, the soil-to-solution mass ratio (s/w) must be taken into account when results from standard batch tests are compared with those from column or diffusion tests or used to model mineral barriers where the s/w ratio ranges around 3-4 instead of 0.25 as required for standardized batch tests.

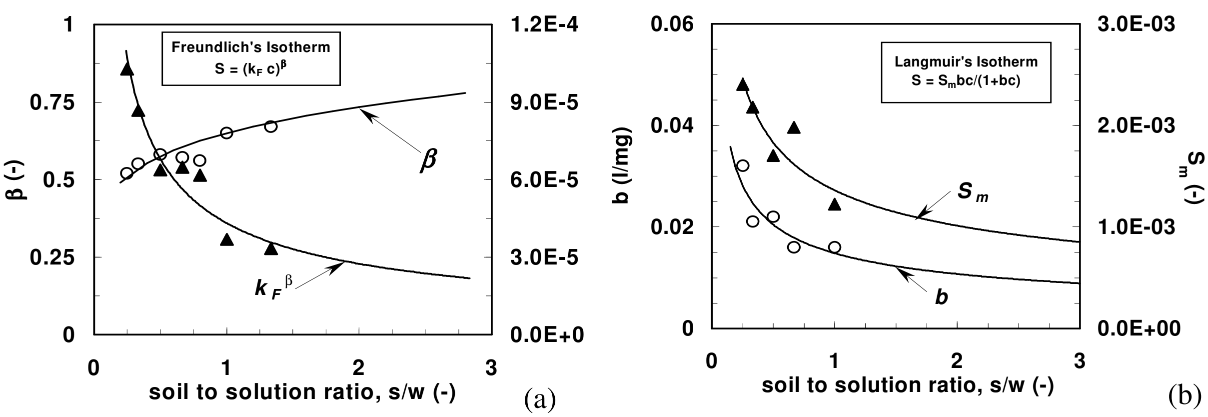
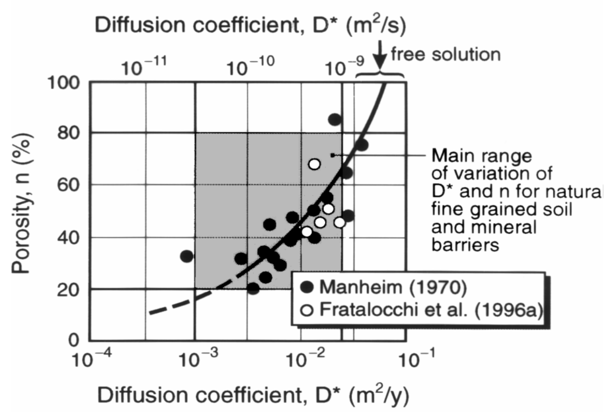


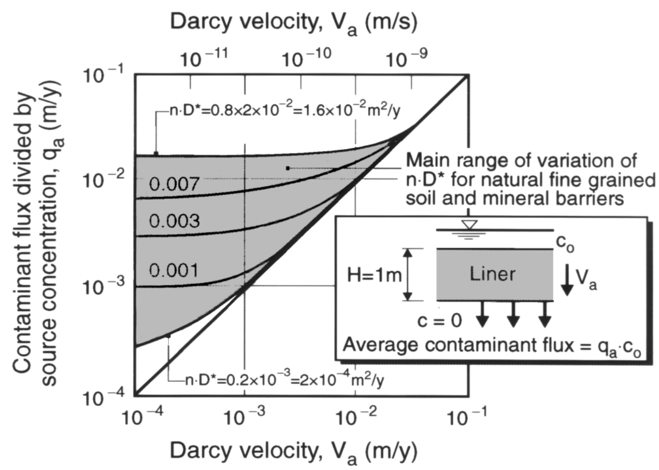
Figure 5. Influence of soil to solution ratio on the parameters of (a) Freundlich's and (b) Langmuir's sorption isotherm (Manassero et al., 1998)

3.1.1.5 Dispersion – Diffusion

It is well known that the effective diffusion coefficients (D^*) become significant for pollutant transport through containment barriers if, and only if, advection transport is low. The common range of variation of porosity and diffusion coefficient of some chemical species (e.g. chloride, ethanol, etc.) for typical materials for mineral barriers is indicated in Figure 6.a. The same range of variation for D^* and n is reported in Figure 6.b where the total contaminant flux through a 1 m thick mineral barrier is given as a function of Darcy velocity that, in turn, is proportional to the HC provided that the gradient is constant. Diffusion starts to play the predominant role in the total contaminant flux only when HC value (k) is lower than 10^{-9} m/s for the common range of hydraulic gradients, as evidenced in Figure 6b.



(a)



(b)

Figure 6. (a) Range of diffusion coefficients for soils and mineral barriers; (b) relative importance of diffusion and advection through mineral barriers (from Manheim, 1970; Rowe, 1988; Jessberger, 1995)

The contribution to the contaminant flux due to the hydrodynamic dispersion (D_d) which is proportional to the effective seepage velocity (v) can be appreciated in Figure 7. The example refers to a landfill bottom barrier underlain by a flushing aquifer where it is assumed a uniform exit concentration. The system can be analysed in a first approximation referring to one dimensional steady state conditions with downward pollutant migration through the landfill liner ending with a perfect mixing with the underlying flushing groundwater. It is possible to observe that using current input parameters for the analyzed mineral barriers material, the contribution of dispersion to the total contaminant flux and concentration is always negligible whenever the seepage velocity is taken into account. In fact at low seepage velocity, pure diffusion prevails, whereas at higher seepage velocity it is the advection, which mainly influences both contaminant concentration and flux.

The scenario of laboratory tests for the determination of diffusion-dispersion parameters is rather wide and is described by Shackelford (1991), Shackelford & Daniel (1991a, b), Jessberger (1994) and Rowe et al. (1995a). The decreasing-source-single-reservoir-diffusion test and the column test considering all its possible variations (constant hydraulic gradient, constant flow rate, etc.) are the most common. The use of these tests can provide indications in terms of both diffusion-dispersion and sorption parameters. In the case of the considered parameters the scale effect is not so important such as in the case of HC determination. On the other hand, the boundary conditions imposed in the different types of laboratory tests must be carefully considered in order to obtain a reliable interpretation of each test in terms of both concentration and contaminant flux. In fact, some column test results singling out, for example, significant sorption properties with non reactive pollutants, should be re-interpreted in order to check if the used algorithms have been appropriate. Referring in particular to the column tests on GCL characterized by very low Peclet number (P = vL/D*, L = sample thickness), it is mandatory to carry out their interpretation using models able to take into account the appropriate boundary conditions.

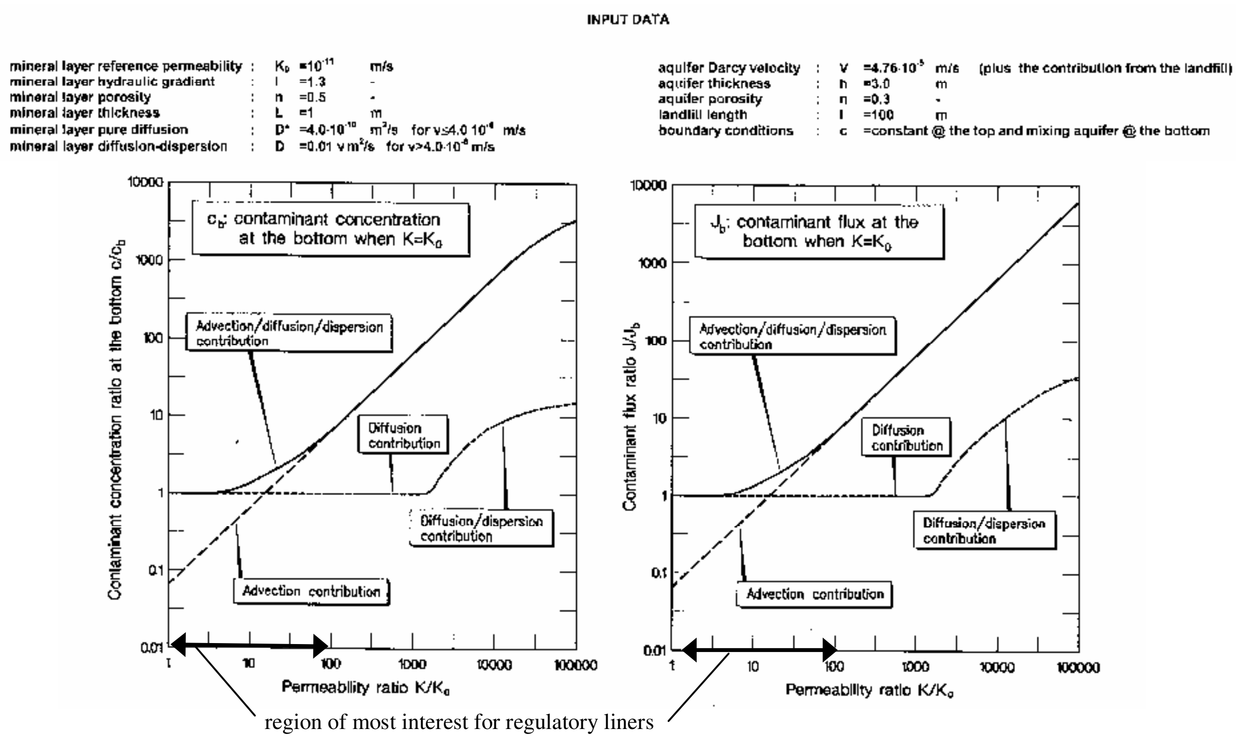


Figure 7. Contribution of advection/diffusion/dispersion to contaminant migration in terms of concentration and flux (Manassero, 1997)

3.1.2 Geosynthetic Clay Liners (GCLs)

An alternative type of mineral liner for landfills consists of geosynthetic clay liners, GCLs (Figure 8); they are a combination of geosynthetics with dry bentonite, typically 5 to 10 mm thick with approximately 5 kg/m^2 of bentonite. The main advantages of this kind of sealing layer are the limited thickness, the good compliance with differential settlements of underlying soil or waste, easy installation and low cost. On the other hand, the limited thickness of this barrier can produce vulnerability to mechanical accidents, limited sorption capacity and an expected significant increase of diffusive transport if an underlying attenuation mineral layer is not provided. Moreover, when hydrated with some types of leachates instead of water, bentonite will show a minor swelling that will reduce the efficiency of the hydraulic barrier.

The use of GCL can be a good alternative to CCL for cover systems and in some cases for bottom liners but the use of GCL alone should be carefully evaluated in the case of waste producing gas unless the GCL is wetted soon after installation and it is not subjected to drying cycles.

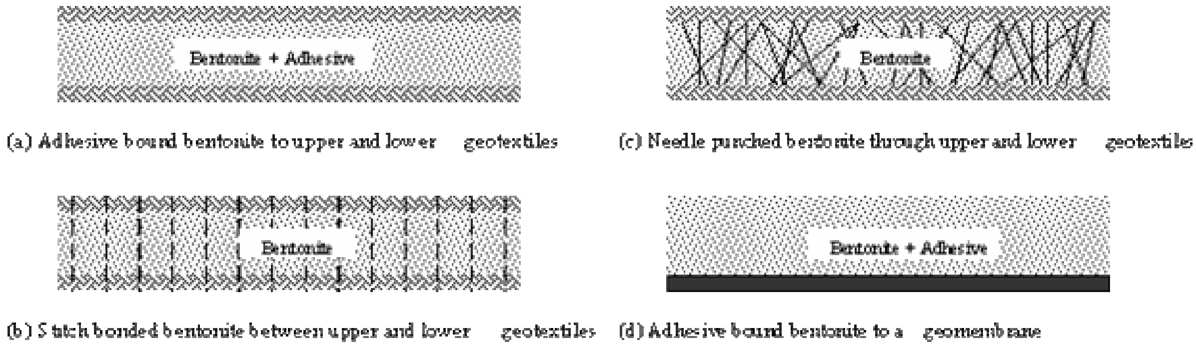


Figure 8. Cross section of geosynthetic clay liners (Daniel & Koerner, 1995)

A qualitative comparison of GCLs and CCLs, provided by different authors referring to different criteria is presented in Table 1. The performance of a GCL, for most criteria, should be either equivalent to or exceed that of a CCL. However, in terms of liner applications, the considerations of solute flux and breakthrough time, compatibility, and attenuation capacity favour CCLs. Some exceptions can be made for GCLs that use geomembrane supports instead of geotextiles and when an attenuation layer (AL) is provided. Given the aforementioned observations about GCL in comparison with a CCL it is recommended that the GCL installed below wastes must be used in conjunction with an attenuation soil layer with a thickness in the range of that of a CCL.

Table 1. Potential equivalency between geosynthetic clay liners (GCLs) and compacted clay liners (CCLs) (Daniel, 1995; Shackelford & Nelson, 1996)

Category	Criterion for Evaluation	Equivalency of GCL to CCL			
		GCL Probably Superior	GCL Probably Equivalent	GCL Probably Inferior	Site or Product Dependent
Construction Issues	*Ease of Placement*	X			
	Material Availability	X			
	Puncture Resistance			X	
	Quality Assurance	X			
	Speed of Construction				
	Subgrade Condition	X			X
	Water Requirements	X			
	Weather Constraints				X
Contaminant Transport Issues	*Attenuation Capacity*				
	Gas Permeability			X(1)	X
	Solute Flux and				X
	Breakthrough Time	X(2)		X	
Hydraulic Issues	*Compatibility*	X(2)		X	
	Consolidation Water	X			
	Steady Flux of Water		X		
	Water Breakthrough Time				X
Physical/ Mechanical Issues	*Bearing Capacity*				X
	Erosion				X
	Freeze-Thaw	X			
	Settlement-Total		X		
	Settlement-Differential	X			
	Slope Stability				X
	Wet-Dry	X			

(1) Based only on total exchange capacity, TEC (2) Only for GCLs with a geomembrane

In order to quantify the comparison between GCLs and CCLs, it is necessary to evaluate the following main features and parameters of GCLs which govern the pollutant transport: HC of GCL permeated with non standard liquids; effect of holes on GCL hydraulic conductivity; diffusion and sorption parameters.

Rowe (1998a), Shackelford et al. (2000) and Lake & Rowe (2000) have developed these topics and a short summary of these research works and some specific aspects will be given.

- The GCL features which influence their HC with liquids other than water, are: aggregate size, content of montmorillonite, thickness of adsorbed layer, prehydration and void ratio of the mineral component. The main factors related to the permeant that influence the HC are: concentration of monovalent and divalent cations. The test duration must assure the achievement of the chemical equilibrium.
- Shan & Daniel (1991), Mazzieri (1998) and Didier et al. (2000a) carried out experimental research on GCLs in order to evaluate the effects of holes and tears. The results of these tests suggest that GCL had the capacity of effectively self heal holes or tears up to 30 mm in diameter.
- Care must be paid to the combination of puncturing and presence of strong cations in the permeating liquids. Mazzieri & Pasqualini (1997; 2000) reported the case history of an artificial basin for collecting rain water that was constructed with a GCL lying below a 30 cm thick calcareous gravel blanket. One year after installation abundant leakage occurred putting the basin out of service. In order to investigate the failure, laboratory tests were carried out with the specific purpose of quantifying the influence of puncturing (possibly due to root penetration of vegetation growing when the basin was empty) and the interaction between the sodium-bentonite GCL and the overlaying layer of calcareous gravel. The results showed that the coupled effect of root penetration and cation exchange were the main causes of the failure of the GCL, and moreover a single phenomenon such as puncturing or cation exchange could not alone lead to the complete failure of the basin.
- The experimental research works to assess diffusion and sorption characteristics of GCLs are just at the beginning. The main findings can be summarized as follows: (1) void ratio and related confining stress have a strong influence on diffusion coefficient (Figure 9); (2) the concentration level of the solute gives significant variation to the diffusion coefficients due to the modification of the micro-structure of the sensitive mineral component (in particular sodium bentonite). Both types of simple diffusion tests and GCL manufacture did not significantly affect the diffusion coefficient. A list of diffusion and sorption coefficients is given in Tables 2 and 3 referring to different types of GCL and organic and inorganic potential pollutants.

Critical aspects about service life of GCL can be related to the limits of the geosynthetic component, to lateral movements of the overlap, to long term compatibility problems and to the localized loss of bentonite during placement or due to piping phenomena. Well supported and validating experiences about these problems are still not available today.

The comparison of GCL versus CCL in terms of actual performance is today one of the hot topics for the engineers involved in landfill design, construction and management. When comparison between different products must be carried out, it is important to keep in mind that it is not possible to generalize about "equivalency" of liner systems since "equivalent" depends on what is being compared and how it is being compared (Rowe, 1988). Apart from their own features, the performances of liner systems are related to the contaminant amount, concentration and decay parameters, the aquifer characteristics and its distance from the bottom of the landfill, and the efficiency of capping and drainage systems. A tentative procedure to compare the performance of a CCL and a GCL is given in Manassero et al. (2000) referring to steady state conditions of contaminant flux (the most critical, at least, in terms of amount of contaminant flux), taking into account advection and diffusion phenomena. The results of the comparison between these two types of liners allow the following comments:

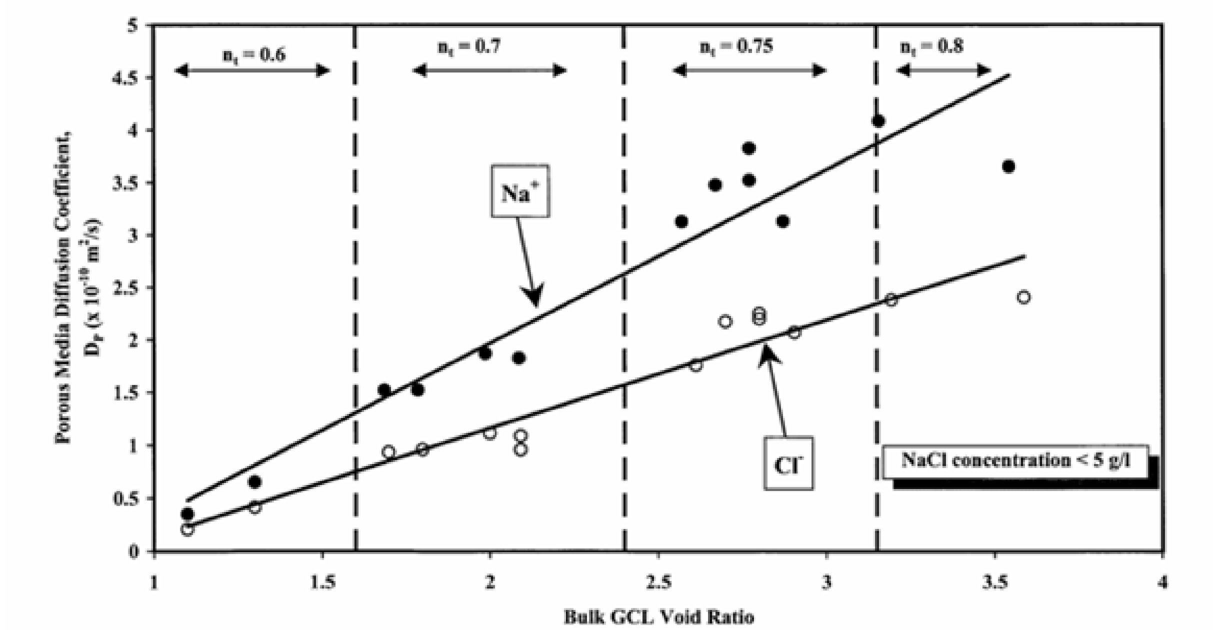


Figure 9. Typical values of total porosity (n_t) and porous media diffusion coefficient of sodium and chloride vs bulk GCL void ratio (Rowe, 1998a)

Table 2. Chloride diffusion characteristics of some GCLs (Rowe, 1998a)

GCL	Applied Stress σ_v' (kPa)	Hydrated Thickness t_{GCL} (mm)	Effective Diffusion Coefficient D^* (m^2/s)	Porosity n (-)
BF3	20	11.1	$3.0*10^{-10}$	0.80
	65	9.1	$2.0*10^{-10}$	0.77
	100	7.1	$1.5*10^{-10}$	0.71
	350	5.6	$0.4*10^{-10}$	0.51
BF2	25	9.1	$2.5*10^{-10}$	0.77
	140	7.1	$1.6*10^{-10}$	0.68
	280	5.6	$0.7*10^{-10}$	0.64
BF4	29	11.1	$2.9*10^{-10}$	0.83
	100	7.1	$1.3*10^{-10}$	0.74

Table 3. Diffusion and linear sorption coefficients of natural and treated clay used in GCLs (Lo, 1992)

Liner Material	Effective Diffusion Coefficient D^* (m^2/s)			K_d value (mL/g)			
	Chloride	Lead	1.2 DCB	Lead (pH-7)	1,2 DCB	1,2,4 TCB	1,2,4,5 TECB
CL	$2.4*10^{-10}$	$5.9*10^{-10}$	$9.8*10^{-11}$	6000	1.4	2.2	10
Organo-Clay	$4.9*10^{-10}$	$9.0*10^{-10}$	$1.5*10^{-10}$	140	609	1320	4500
HA-A1OH-Clay	$3.6*10^{-10}$	$7.6*10^{-10}$	$1.2*10^{-10}$	417	20	38	254

- a good CCL plus AL is able to give better performance in the long-term in comparison with GCL plus AL of the same total thickness;
- the higher contaminant concentration and flux shown by the GCL is mainly due to the higher permittivity (k/L) and therefore to the advective transport, whereas the two diffusive contributions are comparable.
- the advective transport is largely the prevailing contribution to the contaminant migration referring to the barriers considered in the example; therefore, in this case, a further reduction of diffusion coefficient of both CCL and GCL is useless.

The permittivity is still the critical issue for GCLs looking in particular at compatibility problems with leachates (Figure 10). The diffusive transport is reduced by the contribution of the AL and by the good performance of GCL in itself from this point of view (Tables 2 and 3).

In § 2.3 it will be shown that the reduction of advective pollutant transport by a geomembrane placed on the top of these mineral barriers can significantly change the conclusions of the comparison shown in the previous example.

Considering both steady state and transient conditions, Manassero et al. (2000) show that in the presence of heavy metals, the GCL barriers can perform better than CCLs, at least in the short term due to the high sorption capacity of bentonite and special clays used for these products. It is worth stressing that the simplified equivalency criteria can give reliable indications only on a case by case basis and referring to specific conditions related to both time and space domains. For more details about GCL parameters and comparison between CCL and GCL see Rowe (1998a).

Referring to the steady state landfill model illustrated in Manassero et al. (2000) it can be observed that there is a minimum in the function of aquifer relative concentration versus the thickness of soil layers separating the bottom barrier from the groundwater surface (Olinic, 2000). This is due to the combination of a positive contribution in terms of diffusive transport when the AL is thickened and a corresponding decreasing efficiency in terms of advective transport at a given hydraulic gradient since the k_a value of AL is in general higher than the k_b of the landfill liner.

Figure 11 shows the ratio between the thickness of attenuation layer (L_a) and barrier system (L_b) that minimizes the groundwater concentration in the aquifer versus the Peclet number of the barrier evaluated assuming unit hydraulic gradient ($P_{b1}=k_b/n_b \cdot D_b$). Current ranges of variation of the main parameters have been considered referring in particular to equivalent hydraulic conductivity and diffusion coefficients ratios between the barrier and the attenuation layer (k_b/k_a; n_bD_b/n_aD_a).

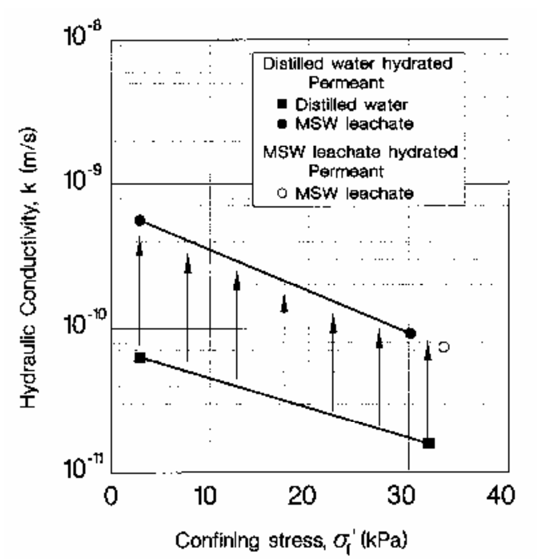


Figure 10. Hydraulic conductivity of a GCL vs static confining stress for permeation of synthetic MSW leachate (modified from Petrov & Rowe, 1997)

Figure 11 has been plotted assuming a hydraulic leachate head (δh) referred to the barrier thickness equal to 0.3. Nevertheless, the plots of Figure 11 can be used with other $\delta h/L_b$ ratios, by simply evaluating the permittivity and diffusivity parameters of an equivalent barrier, which include part of the attenuation layer or, vice versa, of an AL which includes part of the barrier, in order to get a total barrier thickness which, for the actual value of δh, is consistent with the ratio $\delta h/L_b = 0.3$.

Even though it is unlikely that, in the case of a thick and unsaturated attenuation layer, a steady state flow will take place, within the times of interest, for the considered landfill, the plots of Figure 11 can be useful in order to calibrate and optimize the position of the landfill bottom (Figure 12) avoiding excessive thickness of the AL without getting advantages in terms of minimization of pollutant impact on the groundwater.

Finally, it is still interesting to observe that using the current parameters for modern composite liners (i.e. $P_b \cong 1$), the optimized AL thickness ranges from 3 to 10 m. That is the thickness of attenuation layers or the minimum distance between the landfill bottom and the groundwater surface indicated by the regulations of some countries such as Germany, Italy or states such as Ontario (Canada).

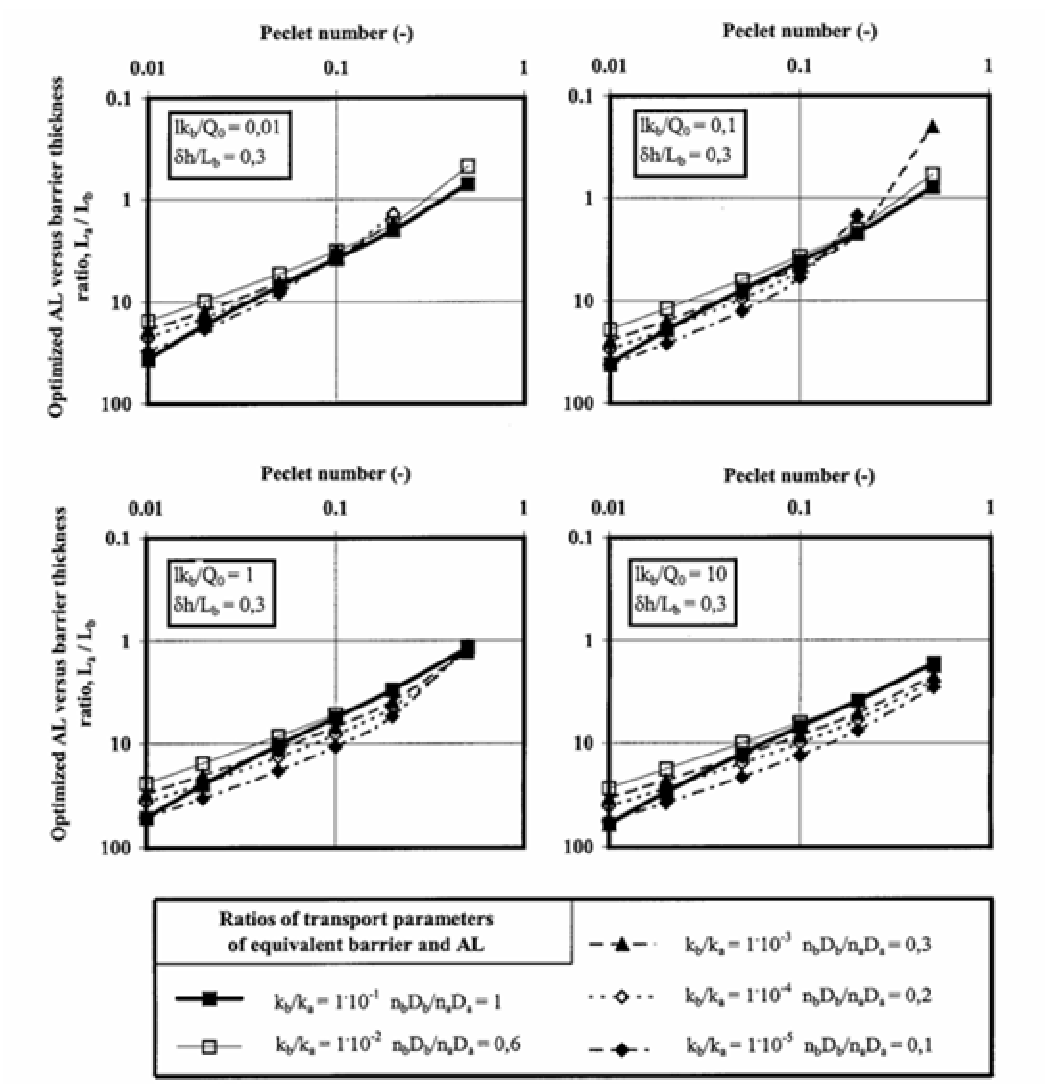


Figure 11. Optimized thickness of AL referring to steady state conditions (Manassero et al., 2000)

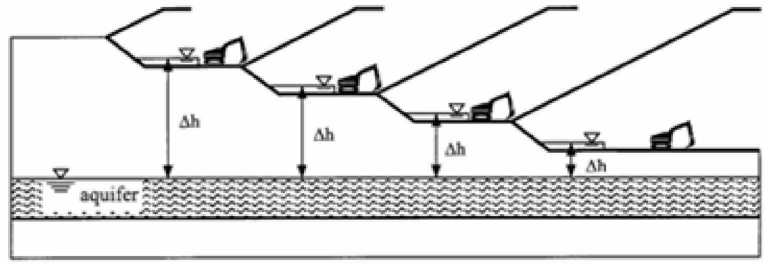


Figure 12. Increase of landfill capacity vs decrease of attenuation layers thickness (Manassero et al., 2000)

The simplified analytical models illustrated above, beyond their use by designers for a preliminary evaluation of the barriers performances, could also be seen as a practical tool for public authorities in order to compare and check, in a quick and generally conservative manner, the performance of alternative liners. This can be done before the approval of some proposals, which do not foresee the current barriers indicated by prescriptive regulation.

An experimental investigation was carried out by Mazzieri et al. (2000) in order to evaluate the compatibility between a commercial GCL and ethyl alcohol (ethanol). The hydration of the GCL prior to exposure was found to play a major role in GCL compatibility with ethanol. Tests results indicated that if the hydrated GCL is permeated after hydration by pure ethanol, the GCL hydraulic conductivity may be increased significantly but not dramatically. Conversely, in the case of unhydrated GCL, the hydraulic performance is strongly dependent on ethanol concentrations. Concentrated ethanol solutions (>50 %) prevent significant swelling of bentonite and impact dramatically the hydraulic conductivity, whereas diluted concentrations (<25 %) have a minor influence on permeability. The effect of initial contact with concentrated solutions was found to be reversible when the GCL was again permeated with water. It is suggested that controlled re-hydration with pure water may be effective as a potential rehabilitation procedure for GCLs contaminated by miscible organic pollutants. As a result, if the actual concentration of the pollutant to be contained is not known, which is often the case, prehydration is strongly recommended.

As final remark, the experimental work allowed to point out that the hydraulic performance of the GCL in the presence of organic pollutants strongly depends on the hydration and permeation sequences, pollutant concentrations and exposure times. Therefore, a reliable prediction of the actual performance of the GCL requires a close simulation of the exposure sequences to be expected in the field.

3.1.3 Composite Barriers

Composite liners consist of a polymeric geomembrane (usually high density polyethylene, HDPE) overlying, in close contact, a mineral barrier (usually a CCL or, in some cases, a GCL). The advantages of composite liners in terms of advective transport, are apparent especially for a poor quality mineral barrier ($k > 10^{-9}$ m/s) as observed by Daniel (1991). Other very interesting and useful information about the full scale field performance of composite liners obtained by monitoring systems are summarized by Rowe (1998a) (Tables 4 and 5). Referring to these tables, it is necessary to observe that in the case of a CCL most of the leakage collected by the secondary leachate collection and removal system (SLCRS) is attributed, by the authors of the papers quoted in the references, to the consolidation water. Hence it would not be due to leakage through the geomembrane.

Table 4. Average flow rates in PLCRS and SLCRS for landfills with composite liners involving GM and CCL (in lphd). (Othman et al., 1996) [simple average of data after about 3 years]

Primary Liner GM		Primary Liner Clay		SLCRS		PLCRS Flow Rates		SLCRS Flow Rates		
type	thickness (mm)	type	thickness (mm)	material	thickness (mm)	average (lphd)	peak (lphd)	average (lphd)	peak (lphd)	period
CSPE	0.9	CCL	600	Sand	450	1120	2076	113	260	41-93
HDPE	2.0	CCL	450	Sand	300	4400	5790	59	152	35-54
HDPE	1.5	CCL	900	GN	5	1142	3985	167	275	42-66
HDPE	2.0	CCL	450	GN	5	53	170	1.5	10	34-58
HDPE	1.5	CCL	900	GN	5	1144	1371	60	102	30-37

PLCRS: Primary leachate collection and removal system; SLCRS: Secondary leachate collection and removal system

Table 5. Mean and standard deviations of flow in PLCRS and SLCRS for 6 landfill cells with a GCL as part of a composite primary liner (in lphd) (Bonaparte et al., 1996)

		Average Flows				Peak Flows			
		PLCRS		SLCRS		PLCRS		SLCRS	
	Cells	mean	sd	mean	sd	mean	sd	mean	sd
Initial period	25/26	5.350	3.968	36.6	68.5	14.964	11.342	141.8	259.9
Active operation	18/19	276	165	0.7	1.1	752	590	7.7	13.7
Post-closure	4	124	-	0.2	-	266	-	2.3	-

Many excellent attempts have been carried out in the past in order to assess the rate of leakage through composite liners by calculations based on the fundamental parameters that govern the problem, (see, for example, Giroud & Bonaparte, 1989a, b; Giroud et al., 1992, 1998). In particular, Rowe (1998a) set up a calculation procedure which allows taking into account the presence of holes in correspondence to a wrinkle of the geomembrane (Figure 13). It also assumed that the length (L_w) of the wrinkle is far greater than the width (2b) so that the effect of leakage at the ends of the wrinkle can be neglected.

Using proper equations and a series of input parameters evaluated by the best estimate based on the present state of knowledge, the results in terms of leakage through composite liners adopting CCLs and GCLs are reported in Table 6 (Rowe 1998a). It is possible to observe a rather good agreement of these leakage amounts with the values reported in Tables 4 and 5. Failing to take into account the presence of wrinkles, it is not possible to obtain reliable results by theoretical calculation with reasonable parameters whichever procedure or equation is used. In particular, if poor contact is assumed (Giroud et al., 1992) the predicted leakage amount is far less than the observational data. This comparison outlines the importance of considering a certain amount of wrinkles over other defects for a correct simulation of full scale performances of composite lining systems.

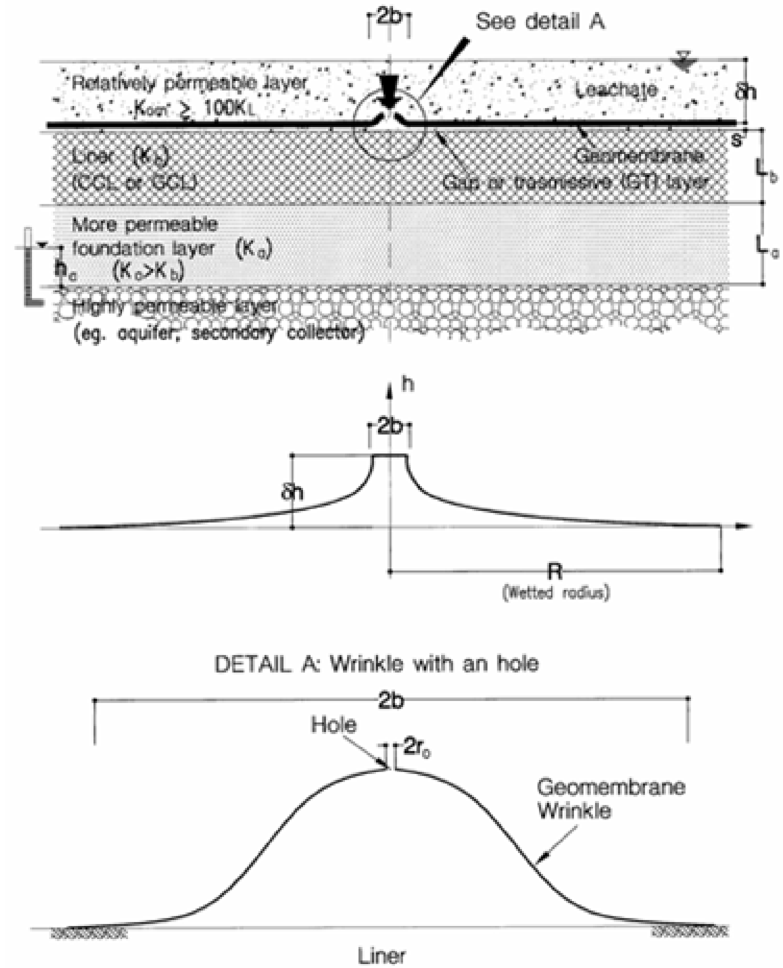


Figure 13. Schematic showing a hole of radius r_0 in a wrinkle of a geomembrane and the underlying strata together with the head distribution between the geomembrane and clay liner (Rowe, 1998a)

Referring to the average leakage rate measured in the field or evaluated by equations reported in Rowe (1998a), Manassero et al. (2000) compare the performance of composite liners using CCL and GCL via the procedure mentioned in § 2.2. The diffusion coefficient of a HDPE geomembrane (GM) has been evaluated from Table 7. The results show that the CCL and GCL composite barriers are practically equivalent, with the contaminant migration being largely governed by diffusion. Therefore, the importance of the geomembrane is evident in reducing the advective migration of contaminants when typical thickness and related HC of mineral barriers are taken into account. In this case the geomembrane hides the higher permittivity of the GCL in comparison with the CCL and this is the main reason why the overall performance of the two types of composite liners are almost fully equivalent under the given assumptions. On the other hand the pure diffusion coefficient of the geomembrane is in general some order of magnitude lower than that of the mineral layers. However, since the geomembrane is generally very thin, its contribution in reducing the diffusive flux is limited, in particular for some organic compounds.

Table 6. Calculated leakage from a small hole (r_0=0.001m) in a wrinkle: L=3m, 2b=0.2m, h_a=0, $q_0=1*10^{-9}$ m/s (Rowe, 1998a)

Liner H K_L θ	h_w (m)	Q (m^3/s)	Leakage* (lphd)
CCL 0.6 m $1*10^{-9}$ m/s $1*10^{-7}$ m/s	0.03 0.3 3 30	$3.0*10^{-9}$ $2.4*10^{-8}$ $2.4*10^{-7}$ $2.4*10^{-6}$	0.6 5.2 51 510
CCL 0.6 m $1*10^{-9}$ m^2/s $1.6*10^{-8}$ m^2/s	0.03 0.3 3 30	$1.6*10^{-9}$ $1.0*10^{-8}$ $9.7*10^{-8}$ $9.6*10^{-7}$	0.3 2.2 21 210
GCL 0.01m+ 0.59m AL $2*10^{-10}$ m/s $1*10^{-10}$ m^2/s	0.03 0.3 3 30	$7.8*10^{-9}$ $1.3*10^{-8}$ $6.9*10^{-8}$ $6.2*10^{-7}$	1.7 2.9 15 130

* Assumes 2.5 holes in wrinkles/ha

Table 7. Values of D_g and S_{gf} used in modeling of contaminant migration across an HDPE GM (Rowe, 1998a)

Leachate	D_g (m^2/s)	S_{gf} (-)	Reference
Pure Toluene	$4.4*10^{-12}$	0.09	(a)
Toluene*	$0.47*10^{-12}$	96	(a)
DCM	$2.2*10^{-12}$**	1	(b)
(aqueous)	$1*10^{-12}$	2.3	(c)
Chloride	$3.2*10^{-15}$**	1	(c)
	$2*10^{-15}$**	1	(c)
	$4*10^{-13}$	0.008	(c)
	$1*10^{-13}$	0.008	(c)

LEGEND: S_{gf}=Henry's coefficient; D_g=geomembrane diffusion coefficient

* Aqueous Solution: Average values 5-90% of solubility; ** D_g calculated assuming S_{gf} = 1

(a) Park & Nibras, 1993; (b) Rowe et al., 1995a; (c) Rowe, 1998a

On the basis of the above observations it becomes fundamental to know the service life of the geomembranes in order to optimize the landfill liner design. The design life of the geomembrane is influenced primarily by the synergistic effects of chemical and physical stresses over an expected period of time (Rowe, 1998a). Primary considerations are the effect of temperature and the effect of tensile stresses over and above the calculated design value (e.g. stresses induced by wrinkles). On the basis of both experimental data, field monitoring results and direct observations from literature, Rowe (1998a) reached the conclusion that the service life of a geomembrane is related to the type and amount of antioxidant used in the geomembrane, the presence of stress concentration (e.g. at wrinkles, due to indentation by stones, etc.) and stress crack resistance. Based on existing data the service life of a properly formulated HDPE geomembrane is projected to be in the order of 150 years at a temperature of around 25^o (e.g. the primary liner of a municipal waste landfill) and 350 years at a temperature of 12^{o}C (e.g. the secondary liners). These service lives have been predicted assuming, in general, good working conditions in a well managed landfill and in particular: (a) good design and construction practice; (b) the specified minimum oxidative time and minimum stress crack resistance indicated by Koerner et al. (1993); and (c) negligible tensile stress concentration in the geomembrane. It is noted that an increase in the temperature of the liner may substantially reduce the service life of the geomembrane to around 10 to 20 years.

Given the above indications, it can be fully acceptable to design a landfill liner with a certain confidence on the performance of geomembranes in the medium and in the long term (i.e. 50 to 350 years). Moreover this conclusion can also be strengthened by the fact that in many cases, after landfill closure (assuming that a low permeability capping system has been used) and at the end of the service life of the leachate collection system, the seepage velocity through the basal lining system, and therefore, the advective transport of the pollutants toward the underlying aquifer, is mainly governed by the capping system and by the annual precipitation and climate conditions of the considered region (i.e. hydrological balance of the landfill) whereas the bottom barrier layers play a certain role only in terms of diffusive transport. This means that the main function of the geomembrane, i.e. the reduction of seepage velocity and therefore limitation of the advective transport can be completely exploited during the active life of the landfill and during the active management of the post closure period. That means up to a maximum of around 50 years; this time is fully compatible with the service life of a well designed and installed geomembrane of the bottom barrier. After this period the amount of leachate reaching the aquifer below a well performing landfill is mainly governed by other factors but not by the basal lining system.

Using the long term simplified model and the equations reported previously, it is possible to investigate the effect of the number of wrinkles (with holes) per unit area of the geomembrane on the performance of composite liners including a CCL or a GCL.

Figures 14 and 15 show the concentration in the groundwater versus the number of wrinkles (with holes) referred to the concentration in the case of pure diffusive flux (i.e. geomembrane without defects). The two analyzed barrier systems, of the same total thickness, consist of an attenuation layer and a composite barrier including CCL or GCL. The other input parameters of the examples are reported in the same figures.

Looking at the plots of Figures 14 and 15, it is possible to observe that the contribution of advection on the liner efficiency due to a single wrinkle (with holes) per hectare is negligible if organic pollutants (i.e.high diffusion coefficient of the geomembrane) are considered. Of course, the reduction could also be 100%, in the case of inorganic pollutant that cannot diffuse through the geomembrane. For up to 10 holes per hectare, the efficiency decrease is modest, thereafter it becomes significant up to reach the maximum gradient in log scale within the range between 10 to 100 wrinkles (with holes) per hectare.

The comparison between Figures 14 and 15 also show that the behaviour of composite liners with CCL or GCL is fully equivalent under the assumption of the given examples. It is still worthwhile to point out the possibility of describing, by this simplified procedure, the trend with time of geomembrane degradation effects once some relationships between wrinkles with holes or cracking and elapsing time will be available through field observations or other experimental data.

In conclusion it is important to stress again that the geomembrane has important positive effects on the drainage of the leachate in order to keep the leachate head on the sealing layer as low as possible and maintain in this way low temperatures in the waste body (Rowe, 1998a). As far as the global efficiency of the drainage system is concerned, it is of paramount importance to point out that the potential of microbial clogging can be drastically reduced if the seepage velocity of the leachate is increased, this can be achieved for example by the use of a smooth geomembrane surface (Rowe et al., 1995b).

Another advantage given by the geomembrane is the retardation of the direct contact of the leachate and the mineral liner until the end of the geomembrane active life. Generally the geomembrane active life ends up after the mineral liner is confined by the whole overburden pressure of the entire waste body. Experimental studies point out the positive effect of confinement stresses in terms of compatibility particularly if applied to a water saturated soil before contact with chemicals that can modify the soil microstructure (Shackelford, 1994a; Rowe et al., 1995a).

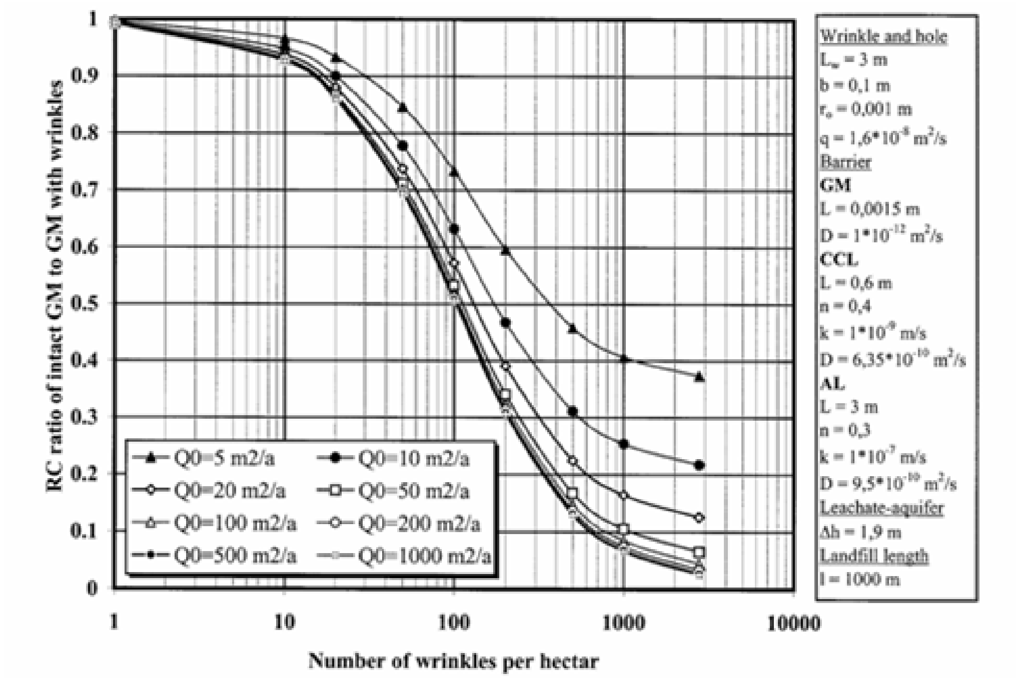


Figure 14. Efficiency of GM+CCL+AL barrier systems vs the number of wrinkles (with holes) in the GM

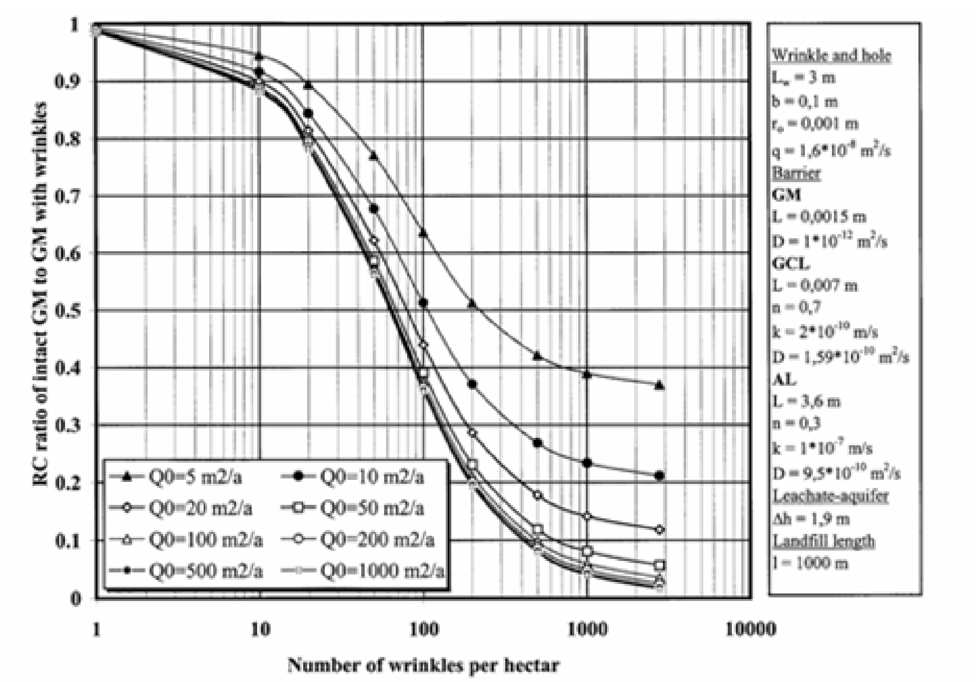


Figure 15. Efficiency of GM+GCL+AL barrier systems vs the number of wrinkles (with holes) in the GM

Due to the need to reduce the superficial area of impact of new landfills, the inclination of side slopes is generally increased to improve the ratio between the volumetric capacity and the foot print of the landfill. Different types of sidewall lining systems that are able to achieve the same safety level as the bottom composite liners and to allow the construction on slope angles up to around 50^o.

Three different alternatives for steep slope side liners are shown in Figure 16. In the first case (Figure 16a) the mineral layer has been constructed with natural clay in horizontal lifts achieving the final slope profile by means of a finishing excavation. The natural clay can be mixed on site before compaction with 2% to 10% by weight of cement in order to achieve the strength that assures the stability of the slope (Manassero & Pasqualini, 1993). For further details on compacted soil-cement mixtures see Chapter 3.b.

The second type of liner for steep sides (Figure 16b) consists of composite geotextile-geomembrane bags filled with plastic concrete or cement-bentonite (CB) slurries. The main advantages of this technique are the reduction of discontinuities between different phases of casting operations and the improved contact between the geomembrane of the geocomposite and the mineral filler. On the other hand, this kind of liner is generally expensive.

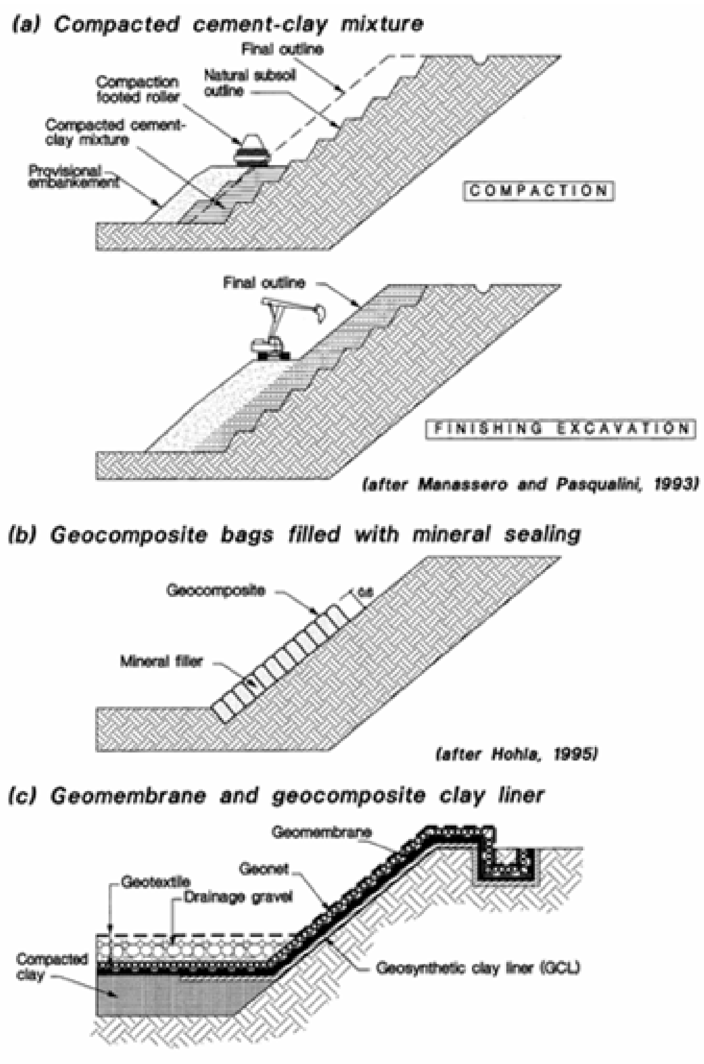


Figure 16. Alternative liners for steep side slopes (Manassero & Pasqualini, 1993; Hohla, 1995)

The third type of steep slope liners (Figure 16c) comprises geosynthetic clay liners. The main advantages of this kind of sealing layer are the easy emplacement and low cost. On the other hand, the limited thickness of this barrier can produce vulnerability to mechanical accidents and an expected significant increase of diffusive transport in absence of an AL.

The need for a more careful design for the lining systems on the slopes has been stressed by the recent failures generated by slip surfaces along liner interfaces (Mitchell et al., 1990; Seed et al., 1990; Mitchell et al., 1995; Stark et al., 1998). Various ranges of interface strengths between geosynthetics and mineral liners or geosynthetics collected from available literature are given in Table 8. It is important to stress the fact that published values of interface friction cannot be used for design of a specific project, without at least careful review of test materials, conditions and methods. It is of paramount importance to determine the interface strength on site specific basis for design purposes, considering the environmental conditions (temperature, leachate, etc.) which can appreciably modify the shear strength. In particular, with reference to the geotextile-geomembrane interface, no significant differences were observed by Stella (1998) between the shear resistance measured at room temperature (20-25°C) and at the maximum temperature (45°C) of testing (Figure 17); this suggests that the results of direct shear test usually performed at room temperature can be assumed as representative of the GM-GT interface behaviour in bottom liners of municipal solid waste landfills where high temperatures are expected. On the contrary, the shear resistance at low temperature (less than 5°C) can be significantly different from that measured at room temperature (Figure 17); in some cases, an increase of shear resistance was observed, in other cases a decrease, depending on the materials in contact. The behaviour of the geotextile-geomembrane at low temperature needs further research, as it can be important for the prediction of stability of covers where low temperatures can occur.

Referring to the shear resistance at the interface between compacted clay and smooth geomembrane, a constant trend of decrease in the shear strength with increasing temperature was observed (Figure 18), making this aspect critical for stability of composite barriers.

Table 8. Strength parameters of different interfaces in landfill liner systems (Manassero et al., 1996)

GEOSYNTHETIC - SOIL INTERFACE	
Geomembrane (HDPE) - Sand	ϕ = 15° to 28°
Geomembrane (HDPE) - Clay	ϕ = 5° to 29°
Geotextile – Sand	ϕ = 22° to 44°
Geosynthetic clay liner - Sand	ϕ = 20° to 25°
Geosynthetic clay liner - Clay	ϕ = 14° to 16°
Textured HDPE – Compacted clay	ϕ = 7° to 35° c' = 20 to 30 kPa
Textured HDPE - Pea gravel	ϕ = 20° to 25°
Textured HDPE – Sand	ϕ = 30° to 45°
Geotextile – Clay	ϕ = 15° to 33°
GEOSYNTHETIC - GEOSYNTHETIC INTERFACE	
Geonet – Geomembrane (HDPE)	ϕ = 6° to 10°
Geomembrane (HDPE) - Geotextile	ϕ = 8° to 18°
Geotextile – Geonet	ϕ = 10° to 27°
Geosynthetic clay liner - Textured HDPE	ϕ = 15° to 25°
Geosynthetic clay liner - Geomembrane (HDPE)	ϕ = 8° to 16°
Geosynthetic clay liner - Geosynthetic clay liner	ϕ = 8° to 25° c' = 8 to 30 kPa
Textured HDPE – Geonet	ϕ = 10° to 25°
Textured HDPE – Geotextile	ϕ = 14° to 52°

.

A method to improve the shear resistance at the interface between CCL and smooth GM consists in spreading cement powder (100-200 g/m²) onto the compacted clay surface before placing the geomembrane (Figure 19); this benefit is particularly important in submerged condition that is the most critical one.

The influence of aging in leachate on the shear resistance at geomembrane-geotextile interface is analyzed by Pasqualini et al. (2002) considering different kinds of geosynthetics. Results of laboratory direct shear tests (Figure 20) show that when non-aged geosynthetics are tested, the GMS-GT interface shear resistance measured in presence of leachate is very close to that obtained in presence of water; the same results show that the aging in leachate of geosynthetics produces a significant improvement of the shear resistance of the GMS-GT interface; this improvement tends to increase with time.

Apart from the stability problems, the design of inclined linings using compacted soils is similar to the design of bottom liners in terms of sealing capacity. Therefore, several concepts on pollutant transport previously developed can be applied.

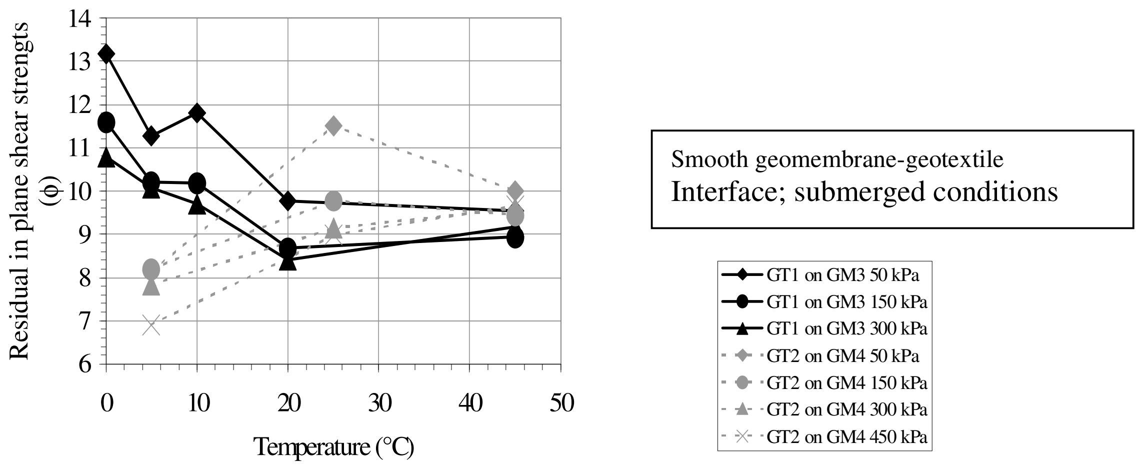


Figure 17. Influence of temperature on the residual equivalent friction angle at the interface between different geotextiles and geomembranes (Pasqualini et al., 2002).

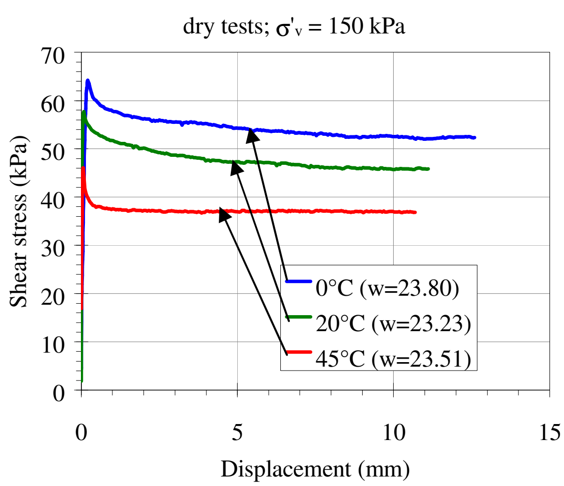


Figure 18. Influence of temperature on the interface shear strength at the interface between a compacted clay and a smooth geomembrane (Stella, 1998).

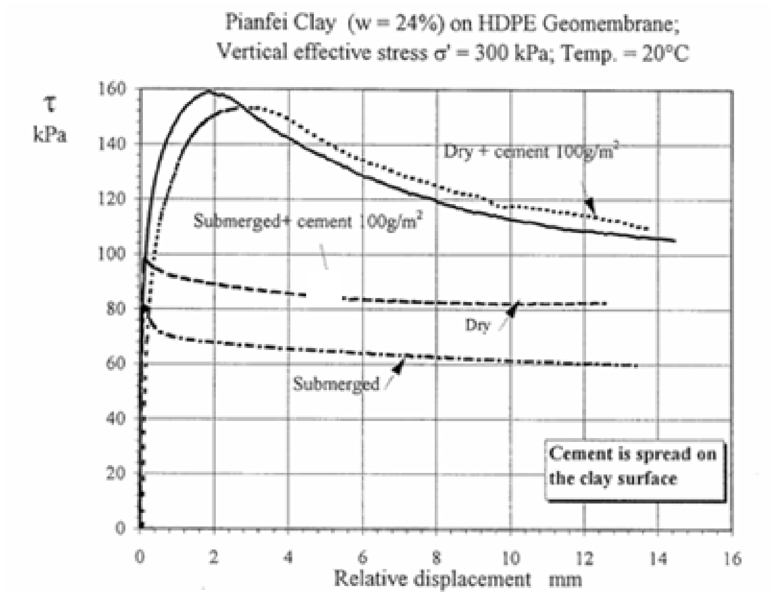


Figure 19: Direct shear tests results on compacted clay-geomembrane interfaces (Pasqualini & Stella, 1994)

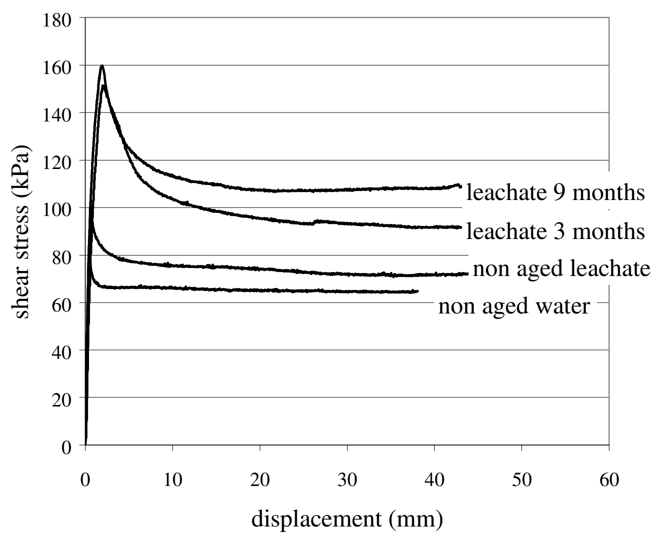


Figure 20. Influence of aging in leachate on the shear strength at smooth geomembrane-geotextile interface (Pasqualini et al., 2002).

3.2 TRADITIONAL COVERS AND COVER MATERIALS

Covers for waste containment facilities must serve three primary functions:
- isolate the waste from the surrounding environment,
- control ingress or egress of gases (e.g., egress of decomposition gases from municipal solid waste or ingress of oxygen into sulphidic mining wastes),
- limit percolation of water into the underlying waste.

For most waste containment systems, control of percolation is the most important function. Also, when a cover controls percolation effectively, the waste is isolated as well and gas movement is controlled.

3.2.1 Compacted Clay Covers

The first covers for waste containment systems consisted of a compacted clay barrier (thickness > 60 cm) overlain by a drainage layer (except for arid climates) and a vegetated surface layer. The low saturated hydraulic conductivity of the compacted clay layer (10^{-8} to 10^{-10} m/s) is expected to limit deep percolation of water, enhance runoff, and promote evapotranspiration. In this context, the primary factor influencing the performance of a clay cover is the hydraulic conductivity of the clay barrier layer.

3.2.1.1 Weathering

Weathering of cover systems consists of erosion, wet-dry cycling and freeze-thaw cycling. Erosion is generally limited to the surface layer, and is controlled through routine maintenance. In contrast, wet-dry cycling and freeze-thaw cycling affect the barrier layer in a manner that precludes simple repair through routine maintenance activities.

The effects associated with wet-dry and freeze-thaw cycling include cracking of the compacted clay, which results in large increases in saturated hydraulic conductivity (Corser and Cranston, 1991; Benson and Othman, 1993; Othman et al., 1994). Frost-related damage is caused by desiccation induced as the freezing front moves downward and by the formation of ice lenses (Benson and Othman 1993). Hydraulic gradients driving flow to the growing lenses cause desiccation of the underlying clay, which results in vertically oriented shrinkage cracks. Horizontal cracks are created as ice lenses form. The horizontal and vertical cracks form a permeable network responsible for the increase in hydraulic conductivity (Othman and Benson, 1994).

Laboratory and field testing (Zimmie & La Plante, 1990; Benson et al., 1995) has shown that most of the damage caused by frost occurs within three to five freeze-thaw cycles (Figure 21). The magnitude of these changes can be predicted reasonably well using the standard laboratory methods in ASTM D 6035.

Desiccation has a severe impact on the hydraulic conductivity of compacted clay by inducing shrinkage and cracking (Albrecht 1996). Large-scale tests (Drumm et al., 1997) and observations in the field (Montgomery and Parsons, 1990; Corser and Cranston, 1991; Benson and Khire, 1995; 1997; Melchior, 1997) have shown that shrinkage cracks in clay result in preferential flow paths and substantial increases in hydraulic conductivity. Results by Albrecht (1996) show that shrinkage, cracking, and increases in hydraulic conductivity caused by desiccation are larger in more plastic clays (Figure 22). Silts and clays, clayey silts and clayey sands with low plasticity are the most resistant to damage by desiccation.

Soil-bentonite mixtures have been suggested as being less susceptible to damage caused by frost (Figure 23) and by desiccation (Albrecht, 1996; Kraus et al. 1997; Abichou et al., 2000). The rigid matrix of the base soil reduces volume change and swelling of the bentonite heals any cracks that may form. Ice segregation does not occur in soil-bentonite mixtures, eliminating cracking due to ice lenses (Kraus et al., 1997).

A method used to limit damage to a compacted clay layer due to desiccation and frost is to thicken the superficial layer, up to 1-m thick. Nevertheless, experience has shown that compacted clay layers still experience desiccation even with thick protective layers during drier periods (Benson and Khire, 1995; Albrecht, 1996; Khire et al., 1997). However, thick surface layers are effective in preventing frost damage. Benson and Othman (1993) indicate that the thickness of frost protection should be at least the maximum depth of frost penetration plus an additional 0.3 m, the latter to account for desiccation ahead of the freezing front.

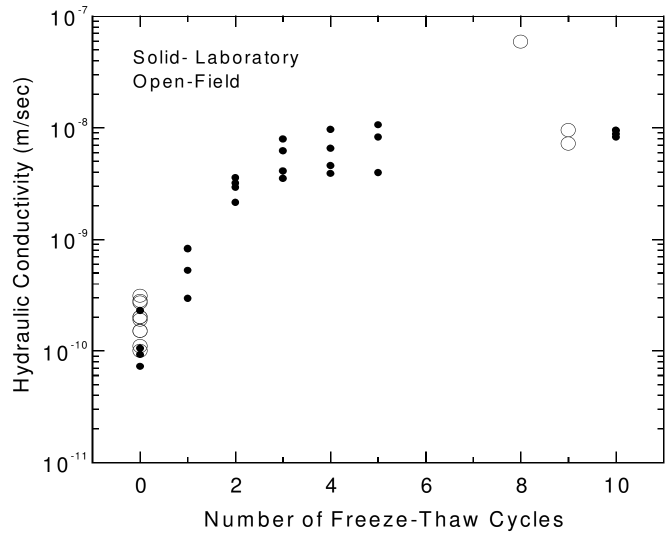


Figure 21. Hydraulic conductivity of compacted clay vs number of freeze-thaw cycles (Benson et al., 1995).

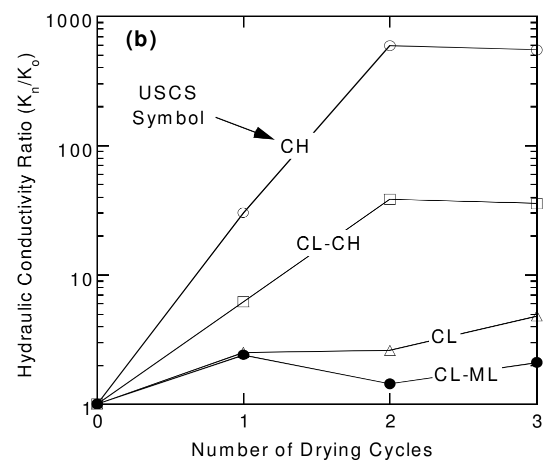


Figure 22. Hydraulic conductivity ratio (hydraulic conductivity after a specified number of wet-dry cycles, K_n, divided by the initial hydraulic conductivity, K_o)vs. number of wet-dry cycles (Albrecht, 1996).

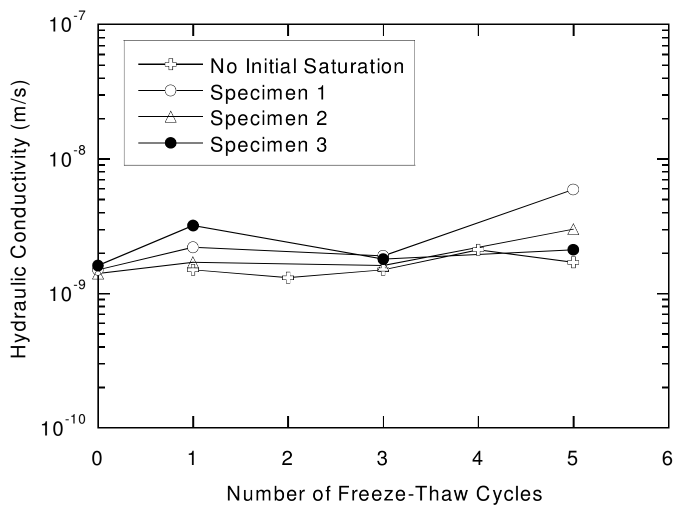


Figure 23. Hydraulic conductivity vs freeze-thaw cycling for a soil-bentonite mixture (Kraus et al., 1997)

3.2.1.2 Settlements

Large settlements (5 to 30%) often occur in MSW landfills as the waste degrades (Edil et al., 1990; Bouazza & Pump, 1997; Bowders et al., 2000). Heterogeneity in the waste inevitably results in differential settlement, which is evident in the undulating surface of most covers on MSW landfills.

Differential settlement can affect the integrity of compacted clay barriers since compacted clays are brittle in tension and have low tensile strength. Jessberger & Stone (1991) conducted model tests in a geotechnical centrifuge and found that flow rate through a barrier was very low and remained unchanged until angular deformation, θ, in the compacted layer reached 6^o (Figure 24); at 6^o the flow rate jumped by a factor of 80. Examination of the clay barrier after testing showed that cracks formed at the point of maximum tensile strain and penetrated the entire thickness of the barrier (Figure 24).

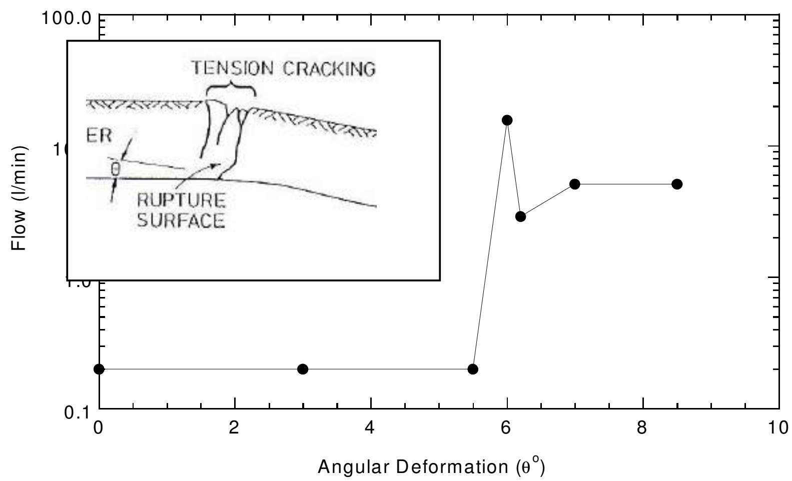


Figure 24. Flow rates measured at various angular deformations (Jessberger and Stone, 1991).

The practical implication of these findings is that differential settlements can compromise compacted clay barriers. The data from Jessberger & Stone (1991) suggest that differential settlements resulting in a distortion (differential settlement Δ divided by the length along the barrier, L, x 100%) greater than 9.5 % will result in cracking of compacted clay barriers and an increase in percolation rate. If distortions of this magnitude are likely to occur, the compacted clay can be reinforced or supported from below using geogrids or high strength woven geotextiles. Alternatively, the cover can be monitored and maintenance can be conducted once distortions approaching 9.5% are reached.

No field data exist to confirm that differential settlement causes cracking or increases in percolation rate when compacted clay barriers undergo differential settlement. However, the results from Jessberger & Stone (1991) are consistent with recommendations in LaGatta et al. (1997), which are based on data describing the tensile characteristics of clays used for cores of earth dams. For the data reviewed by LaGatta et al. (1997), the tensile strain at failure (ε_{tf}) ranges from 0.07% to 0.84% and averages 0.32% when data for bentonite are excluded (i.e., only typical compacted clays are considered). Elastic theory for vertical differential distortion of a horizontal beam with rigid connections at each end shows that Δ/L is 7% for ε_{tf} of 0.32%, which is in reasonable agreement with the Δ/L of 9.5% from Jessberger & Stone (1991).

3.2.1.3 Field Hydraulic Performance

Field studies by Montgomery and Parsons (1990), Melchior (1997), and Khire et al. (1997) describe the hydraulic performance of compacted clay covers of landfills in Wisconsin, USA (annual precipitation ~ 1000 mm/a), in Hamburg, Germany (annual precipitation ~ 850 mm/a) in Georgia, USA (annual precipitation ~ 1200 mm/a) and in Washington, USA (annual precipitation ~ 250 mm/a). For humid climates (the first three sites) the ratio of precipitation (P) to potential evapotranspiration (PET) is greater than 0.5, whereas P/PET is less than 0.5 for semi-arid climates (UNESCO 1979). Each study employed test sections conceptually similar to the one shown schematically in Figure 25, a design that is being employed in USEPA's Alternative Cover Assessment Program (ACAP) (Benson et al., 1999). These test sections are instrumented so that all components of the water balance can be measured.

As shown in Figure 26, annual percolation for the covers spans a large range (1 to 200 mm/a). Test pits at each site showed that the compacted clay barrier cracked, which resulted in an increase in percolation rate. These sites are in very different climates ranging from wet to dry; thus, cracking is likely to occur in most compacted clay covers. The data also suggest that *intact* compacted clay covers are likely to transmit between 10 to 50 mm/a of percolation in humid climates (~1 to 4% of precipitation) and about 1-4 mm/a in semi-arid climates (~1 to 2% of precipitation). When the clay barriers are *cracked*, compacted clay covers are likely to transmit about 100 to150 mm/a in humid climates (10-20% of precipitation) and 30 mm/a in semi-arid climates (~12% of precipitation).

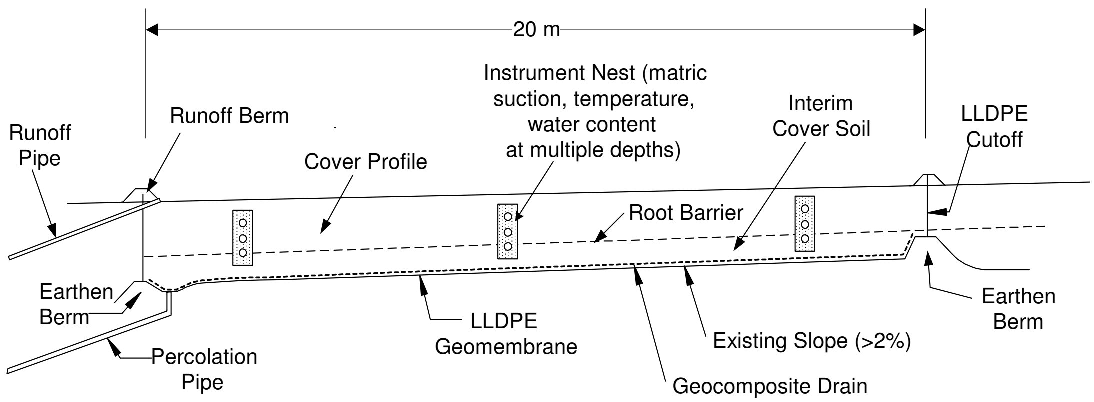


Figure 25. Cross-section of a typical test section used in USEPA's Alternative Cover Assessment Program (Benson et al., 1999)

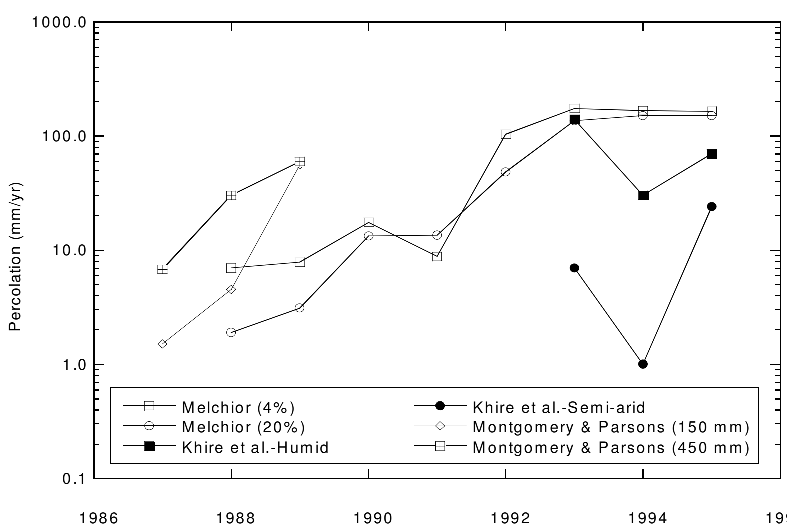


Figure 26. Annual percolation from test sections simulating compacted clay covers (data from Montgomery and Parsons, 1990; Melchior, 1997; Khire et al., 1997).

3.2.2 Composite Covers

Covers employing a composite barrier layer (geomembrane placed directly on a compacted clay layer) began being used when composite liners were deployed in lining systems. Today, all new MSW landfills in the United States are required to use a composite cover unless another type of cover can be constructed that has equivalent hydrologic performance. A drainage layer generally is not prescribed for composite covers placed on MSW landfills.

Composite covers have some significant advantages over compacted clay covers. The geomembrane is very effective in limiting flow through the clay barrier, even though some defects in the geomembrane are inevitable (Giroud & Bonaparte, 1989a; b). The geomembrane can also limit drying of the clay barrier and potentially limit intrusion of roots into the underlying clay barrier. There are disadvantages as well, such as additional cost for materials and construction, and the potential for stability problems along interfaces between geosynthetic layers or soil and geosynthetic layers.

3.2.2.1 Field Hydraulic Performance

Field data suggest that covers employing composite barrier are very effective at minimizing percolation (Melchior, 1997; Dwyer, 1998). Percolation rates from Melchior (1997) are shown in Figure 27 for two composite cover test sections along with data from one of his compacted clay covers (i.e., from Figure 26). The layering was the same as the compacted clay cover, except a textured HDPE geomembrane 1.5 mm thick and a sand drainage layer 300-mm thick were placed above the compacted clay layer.

Percolation from the composite covers gradually increased and then leveled off between 2 and 3 mm/a, which is approximately two orders of magnitude less than percolation from the compacted clay covers (~ 200 mm/a). Test pits excavated in the composite cover test sections showed that the geomembrane prevented desiccation cracking of the clay. The compacted clay beneath the geomembrane was moist, pliable and homogeneous even after the cover had been exposed to drought.

Dwyer (1998) constructed test sections in Albuquerque, New Mexico, USA (annual precipitation ~ 280 mm/a) as part of the US Dept. of Energy's Alternative Landfill Cover Demonstration (ALCD) project. The test sections are similar to those in Melchior (1997). Several different cover designs are being tested at the ALCD, including a compacted clay cover and a composite cover. Percolation rates reported by Dwyer (1998) are summarized in Table 9.

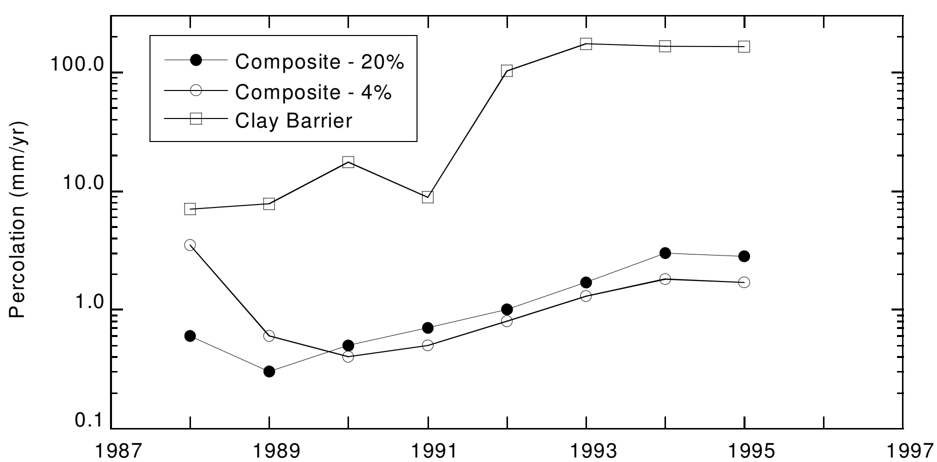


Figure 27. Percolation data from Melchior (1997) for test sections representing composite and compacted clay covers.

Table 9. Percolation from test sections at the ALCD (Dwyer, 1998).

Design	Relative Cost	Percolation @ 10 mos. (mm)	Relative Percolation
Clay Cap	0.32	10.3	147
GCL Composite Cap	0.57	0.88	13
Composite Cap	1.00	0.07	1
Capillary Barrier	0.59	1.24	18
Capillary Barrier with Lateral Drain	0.48	0.97	14
Monolithic	0.47	0.12	2

Percolation from the composite cover at ALCD is 147 times lower than percolation from the compacted clay cover, which is comparable to the ratio of percolation rates observed by Melchior (1997). The high percolation rates obtained from the compacted clay cover are attributed to desiccation cracks that formed in the clay during construction and during subsequent dry periods, which are common in Albuquerque.

Corser and Cranston (1991) describe another field study that illustrates how geomembranes in composite covers protect the compacted clay layer. They constructed three test pads at the Kettleman Hills hazardous waste landfill in southern California, USA, which has a semi-arid climate. One test pad simulated a compacted clay cover, another a composite cover, and the third a compacted clay barrier covered only with a geomembrane. The profile for the composite cover consisted of 610 mm of vegetated fill, a 1.5-mm-thick HDPE geomembrane, and 915 mm of compacted highly plastic clay. The compacted clay cover had the same profile without the geomembrane.

After six months of exposure to ambient conditions, cracks 2-6 mm wide and 25-100 mm deep were found in the compacted clay layer. The water content had also dropped 6% in the upper 100 mm of the clay. Cracks of similar size were also found in the compacted clay layer covered only with a geomembrane. However, these cracks were located in isolated regions where a gap existed between the geomembrane and the compacted clay. At locations where the geomembrane and clay were in firm contact, the clay was moist. Solar heating of the geomembrane caused water to evaporate in regions where a gap existed between the clay and geomembrane. This water condensed on the underside of the geomembrane, and flowed along the surface of the geomembrane to regions where the clay and geomembrane were in contact. Unlike the other two test pads, the compacted clay barrier in the composite cover was devoid of cracks. The clay was moist, soft, and pliable as if it had just been placed. Three years after construction, in the compacted clay cover and the compacted clay overlain only with a geomembrane, cracks existed that were 5-10 mm wide and penetrated the entire thickness of the clay barrier. In contrast, the compacted clay in the composite cover was still moist and un-cracked (Patrick Corser, personal communication, Montgomery-Watson, Steamboat Springs, Colorado, USA, 1997). These findings indicate that the geomembrane in a composite cover is very effective in protecting the clay from desiccation cracking, as was reported by Melchior (1997). Without a geomembrane, a compacted clay barrier is likely to crack and become very permeable. Montgomery and Parsons (1990), Albrecht (1996), Benson and Khire (1997), and Melchior (1997) have made similar observations. Simply covering the clay with a geomembrane is inadequate. A surface layer must be placed as soon as possible to ensure that firm contact exists between the geomembrane and clay.

3.2.2.2 Root Intrusion Through Geomembranes

Geomembranes are often assumed to be effective in limiting intrusion of roots into underlying clay barriers, but little field data exist to verify this assumption. For intact geomembranes, roots are unlikely to penetrate through the polymer. However, laboratory experiments conducted by USEPA suggest that holes in geomembranes quickly become conduits for root penetration.

3.2.3 Geosynthetic Clay Liners

Difficulty in compacting clay on compressible waste, the high cost of clay at some locations, and the aforementioned problems with compacted clay covers has resulted in increasing use of geosynthetic clay liners (GCLs) as a replacement for compacted clay in covers.

3.2.3.1 Frost Damage

Kraus et al. (1997) conducted laboratory and field studies to determine if freezing and thawing affect the hydraulic conductivity of GCLs. Laboratory tests were conducted on 150-mm-diameter specimens of three types of GCLs that were repeatedly permeated, frozen, thawed, and then re-permeated. Field tests were conducted in square (1.3 m x 1.3 m) HDPE test pans that contained a double-ring drainage system beneath the GCL. The outer ring of the drainage system was used to check whether preferential flow occurred between the GCL and the test pan. GCLs in the test pans were overlain with a layer of gravel and exposed to two winters of weather. After the thaw each spring, water was placed on top of the GCLs and the outflow was measured to calculate the hydraulic conductivity.

Typical results of the laboratory tests are summarized in Figure 28. Essentially no change in hydraulic conductivity occurred. Similar results were obtained for all but one of the field tests. The field test that performed differently was for a GCL containing no additional bentonite in the seam. The hydraulic conductivity of this GCL increased by a factor of 25 and dye testing showed that preferential flow through the seam was responsible for the increase in hydraulic conductivity rather than damage to bentonite in the GCL. Nearly identical results were obtained in a bench-scale study conducted by Hewitt and Daniel (1997).

Kraus et al. (1997) indicate that GCLs are not damaged by frost because the hydrated bentonite is soft, and readily consolidates around ice lenses and other defects during thawing. Their findings along with those of others, suggest that GCLs are undamaged by freeze-thaw cycling provided that the seams contain additional bentonite. However, a word of caution: no field tests have been conducted to determine *the long-term performance* of GCLs in cold regions.

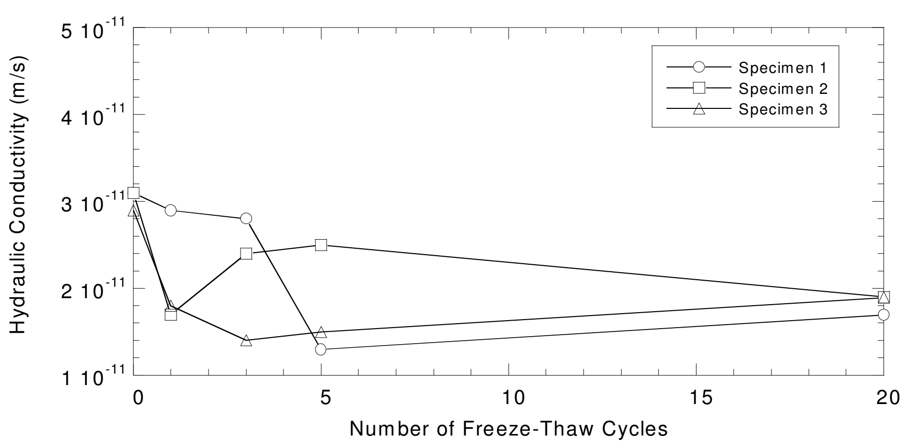


Figure 28. Hydraulic conductivity of GCLs vs freeze-thaw cycling (lab tests; Kraus et al. 1997).

3.2.3.2 Desiccation

GCLs have been suggested as a superior alternative to compacted clay because cracks that form in a GCL during desiccation should swell shut during hydration due to the high swell potential of bentonite. However, mixed results have been obtained regarding the effect of desiccation on the hydraulic conductivity of GCLs.

Boardman and Daniel (1996) conducted large-scale laboratory tests on a GCL placed on top of a drainage layer and sealed against a tank, even during drying. Gravel was placed on top of the GCL to simulate a leachate collection layer. Pipes were installed in the gravel to route heated air for drying the GCL. Electrical resistance probes were installed in the GCL to determine when it ceased losing water during drying.

Tap water was initially placed in the tank to hydrate the bentonite and to determine the initial hydraulic conductivity of the GCL. Afterwards, the water was drained and hot air was circulated through the gravel until probes in the GCL indicated the water content was no longer decreasing. The tank was then re-filled with tap water for permeation to define the hydraulic conductivity after drying. Results of the tests showed that the GCLs initially were very permeable, having a saturated hydraulic conductivity of approximately 10^{-5} m/s. However, the hydraulic conductivity gradually dropped and after two days it returned to its value before drying (~10^{-10} m/s).

Melchior (1997) and James et al. (1997) describe case histories where GCLs apparently were damaged by desiccation. In both cases, GCLs buried in covers began to leak excessively. Exhumation revealed that the GCLs contained fine cracks that apparently formed during desiccation. Tests on bentonite from the exhumed GCLs showed that the exchange complex was primarily calcium and magnesium, which are the predominant cations in natural pore waters. Apparently sodium ions initially in the exchange complex were replaced by calcium and magnesium cations as pore water hydrated the bentonite. As a result the swell potential of the bentonite decreased. After calcium-for-sodium exchange, cracks that formed in the bentonite during dry periods did not swell shut when the bentonite was subsequently re-hydrated. Percolation from the covers increased due to preferential flow through these cracks.

The findings of Melchior (1997) and James et al. (1997) prompted Lin and Benson (2000) to conduct a series of laboratory tests under controlled conditions to assess how GCLs are affected by desiccation and re-hydration with waters having concentrations of divalent cations typical of humid climates. Results of swell and hydraulic conductivity tests they conducted are shown in Figure 29. Bentonite subjected to wet-dry cycling using deionized water (DI) as the hydrating liquid showed no change in swell or hydraulic conductivity even after seven wet-dry cycles. In contrast, swell decreased when 0.0125 M $CaCl_2$ solution was used for re-hydration and, after four wet-dry cycles, the hydraulic conductivity increased by a factor of 4000. In some cases the bentonite was hydrated with DI water or tap water during the first wetting cycle to determine if a first exposure effect would prevent damage to the bentonite. However, these specimens behaved in the same manner as the other specimens hydrated with $CaCl_2$ solution.

Desiccation cracks that did not close during re-hydration were the cause of the increase in hydraulic conductivity. Lin and Benson (2000) also suggest that Boardman and Daniel (1996) did not see an increase in hydraulic conductivity since they conducted only a single drying cycle and used water having a concentration of divalent cations below that typical of natural pore waters.

The results reported by Lin and Benson (2000) confirm the case histories reported by Melchior (1997) and James et al. (1997). That is, exchange of divalent cations in natural pore waters for sodium in the bentonite ultimately results in the bentonite being unable to swell sufficiently to close cracks that form during desiccation. As a result, GCLs that are exposed to wet-dry cycling are likely to fail in the long term unless cation exchange can be prevented. From a practical perspective, these findings suggest that GCLs should not be used in covers unless they will be placed directly beneath a geomembrane.

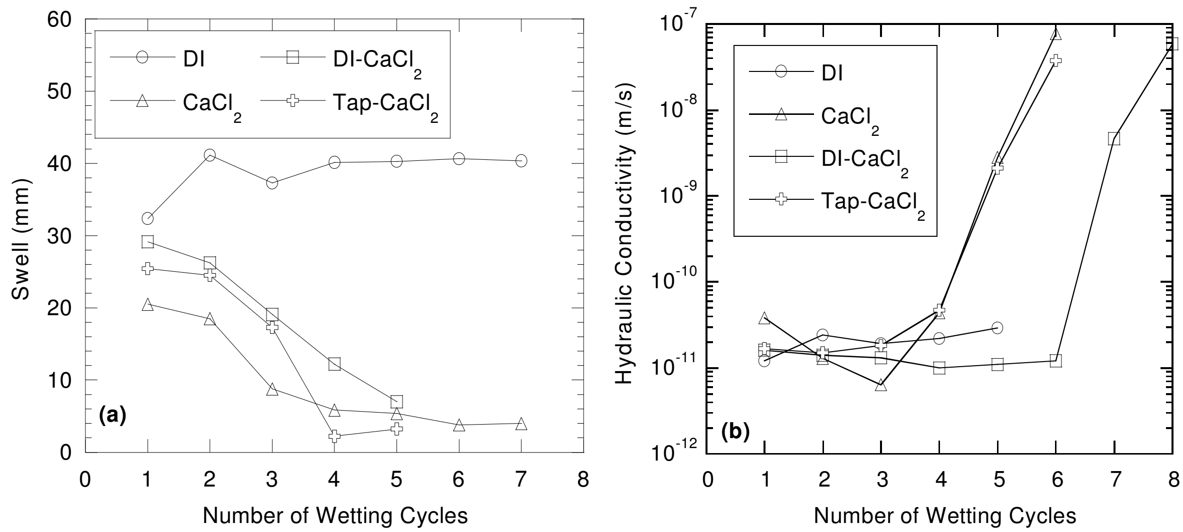


Figure 29. Swell (a) and hydraulic conductivity (b) of GCL subjected to wet-dry cycling using DI water and 0.0125 M $CaCl_2$ solution (adapted from Lin and Benson 2000).

A GCL employed in a cover over a fly ash landfill in southwestern Wisconsin, USA recently failed as a result of calcium for sodium exchange (Manassero et al., 2000). Percolation collected in two lysimeters (BL1 and BL2) is shown in Figure 30. Excessive percolation was first noticed during the spring after construction. The following inspection suggested that thinning of the GCL due to pressure applied by gravel in the lysimeter was the cause for the failure, and the exchange complex of the bentonite was not examined. A layer of sand was added to the lysimeter above the gravel as a cushion, a new GCL was placed, and the surface layers were replaced. Percolation monitoring continued after the lysimeters were re-built. Approximately 15 months after re-construction, the leakage rate became excessive again (Figure 30) and the GCL was exhumed. Inspection of the interior of the GCL revealed that the bentonite was dry and cracked. Analysis of the exchange complex of the bentonite showed that the bentonite contained approximately 14 calcium cations for every sodium cation. In contrast, when the GCL was new, the exchange complex contained approximately 1 sodium cation per 1.4 calcium cations. The in-service hydraulic conductivity of the GCL was estimated from the leakage rate and found to range between 3 $x10^{-9}$ to 7 $x10^{-9}$ m/s. A specimen of the exhumed GCL was tested and found to have very similar hydraulic conductivity (2 $x10^{-9}$ m/s). Swelling of the exhumed bentonite was also comparable to that of calcium bentonite. These findings suggest that calcium-for-sodium exchange reduced the swelling capacity of the bentonite, and prevented cracks in the bentonite from healing after desiccation during the summer months, as was observed by Melchior (1997) and James et al. (1997).

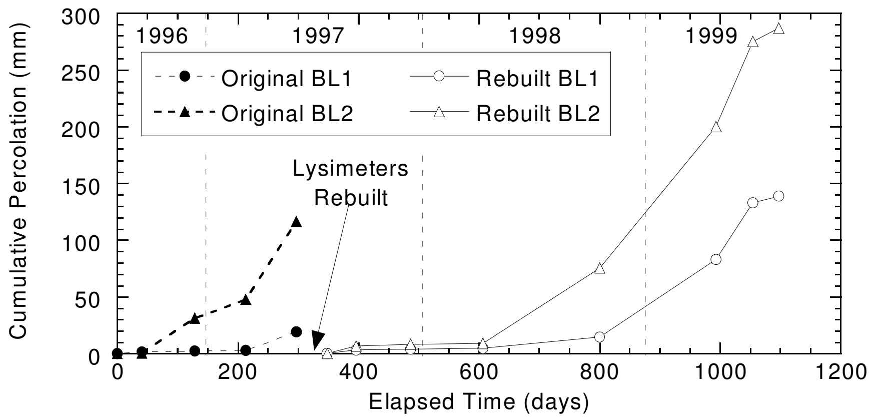


Figure 30. Percolation rates recorded by two lysimeters beneath the GCL cover over a fly ash landfill in Wisconsin, US (Manassero et al., 2000).

3.2.3.3 Gas Permeability and Diffusion

Although GCLs are usually installed to limit advection of fluids (e.g., water through a cover system) they may also serve an important role in covers as a gas barrier. Recent work has shown that the manufacturing process and the form of bentonite (powdered or granular) have a significant effect on the gas permeability of GCLs (Didier et al., 2000b; Bouazza and Vangpaisal, 2000; Vangpaisal and Bouazza, 2001; 2004).

The movement of gases in porous media such as soil or GCLs occurs by two major transport mechanisms: advective flow and diffusive flow. In advective flow, the gas moves in response to a gradient in total pressure. To equalize pressure, a mass of gas travels from a region of higher pressure to a lower one. In the context of landfills, the primary driving force for gas migration, especially through cover systems, is pressure differentials due to natural fluctuations in atmospheric pressure (barometric pumping). Falling pressures tend to draw gas out of the landfill, increasing the gas concentration near the surface layers. Conversely, high or increasing barometric pressure tends to force atmospheric air into the landfill, diluting the near surface soil-gas and driving gas deeper into the landfill. A change in the leachate/water table or temperature changes can also give rise to pressure differences and lead to gas migration. A number of recent events have brought the hazards associated with landfill produced methane very much into public view. The best known of these were the Loscoe, U.K, (Williams and Aitkenhead, 1991); Skellingsted, Denmark, (Kjeldsen and Fisher, 1995) and Masserano, Italy (Jarre et al., 1997) incidents, which resulted in extensive property damage and loss of lives. The Loscoe explosion in the United Kingdom for example, took place after atmospheric pressure dropped by 29 mbars in approximately 7 hours. The same phenomenon caused the Skellingsted and Masserano explosions. Another area of concern is the presence of a geomembrane in a cover system, where even nominal amounts of gas can be troublesome. One concern, amongst many others, is the possibility of landfill gas accumulation, which can gradually increase positive pressure in the landfill. This may induce geomembrane extrusion in landfill composite covers at points of inadequate overburden (Sherman, 2000). In addition, the positive gas pressure under the barrier layers may induce the reduction of interface shear strength between the geomembrane and the underlying layer due to the insufficient normal stress acting on the barrier layer which in this case can contribute to a slope failure (Koerner and Daniel, 1997). Therefore, it is not surprising that nowadays gas pressures have been recognized as a design issue for landfill covers and a methodology has been put forward to address this issue (Thiel, 1999). In this respect, the effectiveness of the mineral barrier component to control gas migration becomes of a paramount importance and needs to be addressed in much more detail.

Gas movement by diffusion occurs due to molecular interactions. When a gas is more concentrated in one region of a mixture than another, it is likely that this gas diffuses into the less concentrated region. Thus the molecules move in response to a partial pressure gradient or concentration gradient of the gas. This is a key issue (diffusion) in the performance of cover systems for milling wastes and mined rocks where sulphidic minerals should not come into contact with atmospheric oxygen in order to prevent acidification of leachate. In this circumstance, the oxygen diffusion coefficient is a critical parameter for designing the capping (Yanful, 1993; Shelp and Yanful, 2000).

Mechanism of gas transport due to advection

Dullien (1975) pointed out that the flow of gas in porous media has different characteristics from the flow of liquids. First, the compressibility of gases has an important contributing factor to the occurrence of unsteady state flow in porous media. Second, the velocity of gas flow at the pore walls cannot generally be assumed to be zero. The nonzero flow velocity at the pore walls is termed “slip flow” or “drift flow”. This effect results in greater flow than predicted by Darcy’s law, which governs the viscous flow of liquids in porous media where the velocity along the pore walls is zero. Third, the adsorption of gases on the pore surface can lead to large difference in gas permeability when determined with highly adsorbing gases comparing to non-adsorbing gases. Flow measurement performed by Alzaydi and Moore (1978) showed that Darcy’s law could provide a fair approximation of gas flow in a low permeability material. Furthermore, Izadi and Stephenson (1992) confirmed that contrary to coarse grain soils, the gas slippage flow through clay soils decreased as the degree of saturation decreased. This indicates that the magnitude of slip flow is very small relative to viscous flow. Brusseau (1991) also indicated that slip flows are not observed when the pressure difference is lower than 20 kPa (similar to the one encountered in landfills) and can, on this basis, be excluded from the modeling process for gas advective transport conditions. The same study also stressed the fact that for low pressure differences the assumption of incompressible flow of gas in porous media is valid. Thus models developed for water flow can be used for gas flow. Massmann (1989) indicated that groundwater flow model provided good approximation for gas transport up to a differential pressure of 50 kPa.

Based on Darcy’s law, the one-dimensional volumetric flow (Q) of gas in porous media is given as:

$$Q = -k\frac{K_r}{\mu}A\frac{dP}{dx} \tag{1}$$

where k is the intrinsic permeability of the porous material, K_r is the relative permeability for the permeant gas, μ is the dynamic viscosity of the gas, A is the cross section of the porous material, and dP/dx is the pressure gradient. It is assumed that the intrinsic permeability is a function only of the properties of the porous material, not the permeating gas. To avoid the complex calculation of K_r, Darcy’s law can be formulated using the gas permeability K as follow:

$$Q = -\frac{K}{\rho g}A\frac{dP}{dx} \tag{2}$$

where g is the acceleration due to gravity and K is given by:

$$K = \frac{\rho g}{\mu}k\,K_r \tag{3}$$

For gases, the rate of flow changes from one point to another point as the pressure decreases due to their compressibility. However, it can be assumed that landfill gases behave like ideal gases and the continuity equation of ideal gas can be written as:

$$\frac{\rho_0 T_0}{P_0} = \frac{\rho T}{P} \tag{4}$$

where ρ_0 is the gas density at standard pressure P_0 and standard temperature T_0, and ρ is the gas density at pressure P and temperature T. Assuming the rate of mass flow (ρQ) is constant and the law of mass conservation is applied. A steady state flow ($d(\rho Q)/dt=0$) of gas can be written as:

$$\frac{d}{dx}(\rho Q) = 0 \tag{5}$$

From equations 2, 4, and 5, a linear differential equation for the one-dimensional steady state flow in an isotropic homogeneous porous medium under isothermal conditions is obtained:

$$\frac{d^2}{dx^2}(P^2) = 0 \tag{6}$$

For a sample of length *L*, the solution to equation 6 is subject to the boundary conditions, $P = P_1$ at $x = 0$ and $P = P_2$ at $x = L$, hence:

$$P^2 = P_1^2 + \left(\frac{P_2^2 - P_1^2}{L}\right)x \tag{7}$$

From equation 2 and 7 the volumetric flow of gas at distance *x* can be obtained from the following equation:

$$Q_x = -\frac{K\,A}{2\rho\,g\,L}\frac{(P_2^2 - P_1^2)}{\sqrt{P_1^2 + \frac{\left(P_2^2 - P_1^2\right)}{L}x}} \tag{8}$$

Considering the volumetric flow of gas at a distance *L*, equation 8 becomes:

$$Q_L = -\left(\frac{K}{\rho\,g}\right)A\frac{(P_2^2 - P_1^2)}{2LP_2} \tag{9}$$

Gas Permeability

The variation of the gas permeability of partially hydrated GCLs has been investigated in detail by Vangpaisal & Bouazza (2004). Their results show that the decrease of gas permeability is associated with the increase in gravimetric moisture content (Figure 31). For the range of gravimetric moisture contents studied, a decrease of around 5 to 7 and 4 to 6 orders of magnitude in the gas permeability was observed for confined hydration and free swell hydration, respectively. It appears from Figure 31 that the gas permeability of GCL-3 (granular bentonite) is more sensitive to moisture variation than other GCLs. In the case of confined hydration, a variation of 7 orders of magnitude in the gas permeability of GCL-3 was obtained for a gravimetric moisture content varying from 18% to 138%. This was due to the large difference in the bentonite form, which changed from coarse bentonite grains with large interconnected voids in the dry state to soft continuous bentonite gel at high gravimetric moisture content.

Figure 31 also shows that the gas permeability of the GCLs varies according to the mode of sample hydration. The GCLs exposed to a surcharge during hydration tended to have lower gas permeability than the GCLs hydrated under zero confinement, particularly at medium to high gravimetric moisture content (>80%). This can be attributed to the fact that the application of a surcharge limited the swelling of hydrated bentonite and induced a more uniform distribution of moisture content throughout the GCL specimens. As a result, pore size and the interconnected voids in the bentonite component were likely to reduce, therefore, the lower gas permeability. This implies that the GCL should be subjected to confinement at time of installation or hydration. Interestingly, the modes of sample hydration appeared to have no effect on the gas permeability of GCL-2 (NWGT impregnated bentonite). This was probably due to the effect of bentonite impregnation into the non-woven geotextile in GCL-2. The presence of the surcharge during hydration had less effect on the gas permeability at lower gravimetric moisture content. A common value of gas permeability was attained in the dry state (gravimetric moisture content as received from manufacturer).

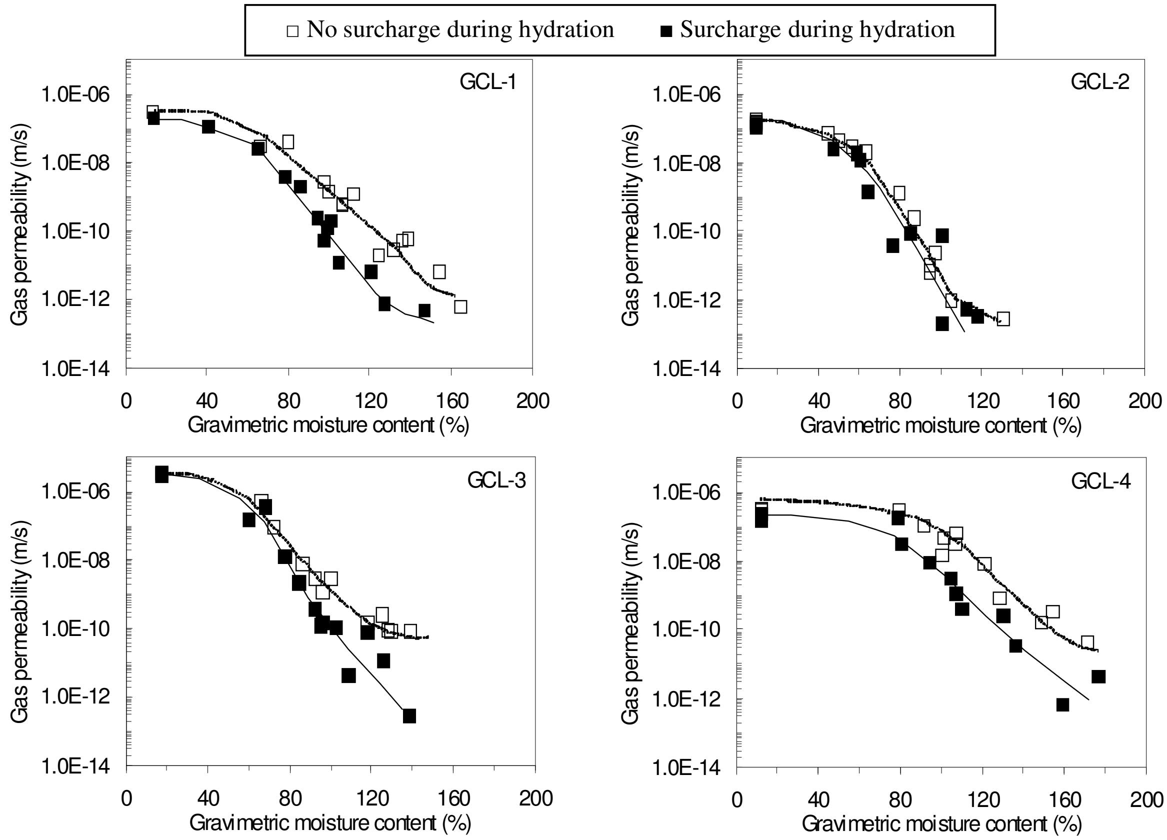


Figure 31. Relation between gas permeability and gravimetric moisture content for 4 different GCLs.

The ability of GCLs to allow the flow of gas can also be expressed in terms of gas permittivity (Ψ). The permittivity is defined as the cross plane permeability (K) divided by the GCL thickness (L), $\Psi = K/L$. The variation of the permittivity against gravimetric moisture content is plotted in Figures 32 and 33 for the confined hydration and free swell hydration, respectively. It can be observed that the variation of permittivity followed the same trend for both hydration conditions. The higher permittivity values were obtained at lower gravimetric moisture content and lower permittivity values were obtained at higher moisture content.

The effect of the bonding mechanism of GCLs on the gas permeability can be seen in the GCLs containing powdered bentonite (GCL-1, 2 and 4), in which GCL-4 (stitch bonded) tended to have higher permittivity for the range of gravimetric moisture contents investigated in the present study. This difference is probably linked to the way that the GCLs are held together as a composite material. Stitch bonding is used in GCL-4, whereas needle punching is used in GCL-1 and GCL-2. As bentonite hydrates and swells GCL-4 tends to form pillow like shapes. This results in zones (at the stitch bonding level) with less bentonite to mitigate gas flow. In contrast, the bentonite tends to swell uniformly in the needle punched GCLs. In the case of dry GCLs, the effect of the holding mechanisms is insignificant because the large interconnected air voids in dry bentonite overrides the effect of needle punching and stitch bonding.

It can be seen that at the same level of gravimetric moisture content, GCL-2 tends to have lower permittivity than other GCLs for both hydration conditions. This may be a result of the impregnation of bentonite in the non-woven geotextile, which induces an additional form of confinement, from the non-woven fibres, to the hydrated bentonite. The contribution of bentonite impregnation in lowering the gas permittivity was significant, particularly in the case of free swell hydration.

For the range of gravimetric moisture contents investigated, the variation of gas permittivity between different types of GCLs is more than one order of magnitude, and varies up to 3 orders of magnitude in the case of free swell hydration. It also shows from Figures 32 and 33 that the permittivity is leveling off at higher gravimetric moisture contents, suggesting that gas advection becomes less significant (the measured flow rate approaches zero flow) and gas diffusion probably becomes the governing transport mechanism. The boundary of gravimetric moisture content, which was attained for zero advective flow measurement depended on the types of GCLs, i.e. gravimetric moisture content higher than 120% for GCL-2, and higher than 180% for GCL-4. At this level of gravimetric moisture content, the measured flow rate approaches the lower limit of the measuring technique and the advective flow is assumed as approaching zero flow rate. As expected, GCL-3 had an exceptionally high permittivity in the dry condition. This is because gas can easily flow through the large pore spaces of dry granular bentonite. The effect of bentonite form on the gas permeability of GCLs is clearer when the gas permittivity is plotted against volumetric water content as shown in Figures 34 and 35. The variations of gas permittivity with volumetric water content follow a similar trend as the plot with moisture content, in other words the permittivity decreases as the volumetric water content increases.

Interestingly, among the needle punched GCLs (GCL-1, 2 and 3) investigated, GCL-3 (bentonite in granular form) tends to have higher permittivity than GCL-1 (powdered bentonite) and 2. This is due to the large difference in the nature of bentonite form. The hydrated granular bentonite is stiffer than the hydrated powdered bentonite, and it is clearly visible as soft grains particularly at the lower level of volumetric water content. This indicates the presence of larger inter-granular pore spaces, which provide preferential gas flow paths. As indicated earlier, each bentonite grain is slowly hydrated from the surface and as the volumetric water content increases the bentonite forms a gel surface and becomes softer, hence, the interconnected voids are decreased, and as a result the difference in the permittivity of GCL-3 to GCL-1 and GCL-2 is lower.

For the conditions tested, the effect of stitch bonding (in GCL-4) and the form of bentonite (in GCL-3) on gas permittivity are comparable at volumetric water content greater than 50%. However, the effect of the GCL structures and bentonite forms on gas permittivity tends to decrease as the volumetric water content increases when the GCLs are hydrated under a confining stress. In this case the effect of the surcharge overrides the effect of the differences between the GCLs, and a common permittivity value can be obtained, for all types of GCLs tested, at volumetric water content greater than 70%. At this level of volumetric water content, the advective flow is approaching a so-called zero advective flow condition and diffusive flow probably becomes significant. On the other hand, the gas permittivity of GCLs approaches the zero advective flow condition at different volumetric water content levels for the free swell hydration.

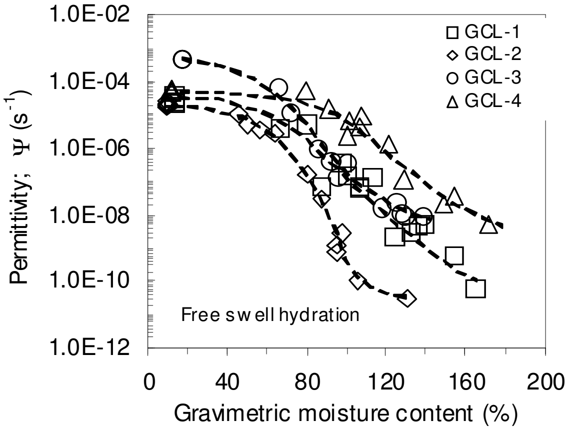


Figure 32 Variations of gas permittivity with gravimetric moisture content for GCLs under confined hydration. (from Vangpaisal & Bouazza, 2004)

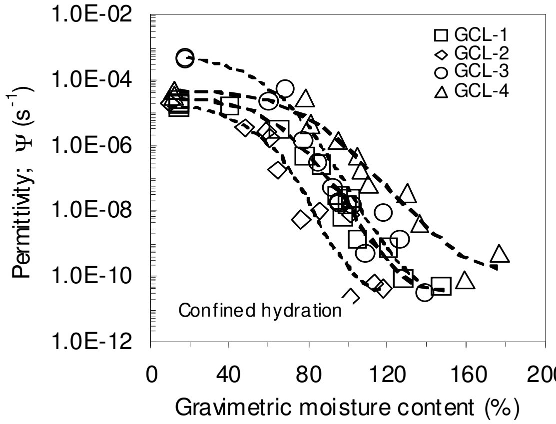


Figure 33 Variations of gas permittivity with gravimetric moisture content for GCLs under free swell hydration. (from Vangpaisal & Bouazza, 2004)

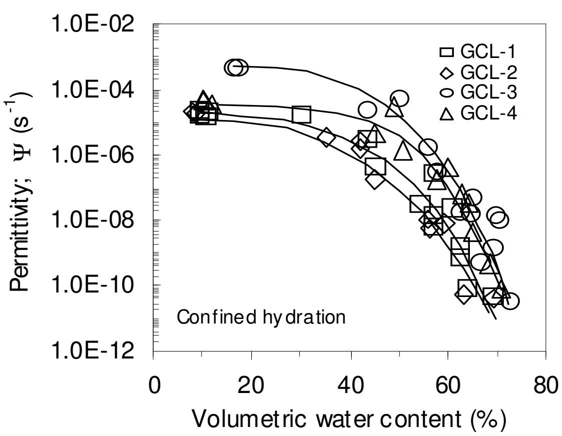


Figure 34 Variations of gas permittivity with volumetric water content for GCLs under confined hydration. (from Vangpaisal & Bouazza, 2004)

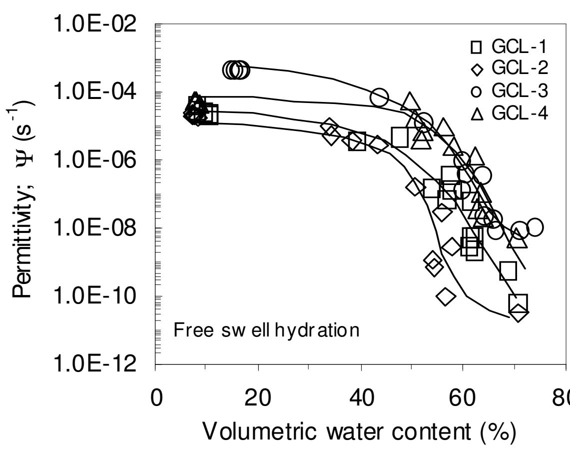


Figure 35 Variations of gas permittivity with volumetric water content for GCLs under free swell hydration. (from Vangpaisal & Bouazza, 2004)

Diffusion

Knowledge of the gas diffusion coefficient is useful in analysis of a variety of transport processes such as oxygen movement through cover systems for milling wastes and mined rocks where sulphidic minerals should not come into contact with atmospheric oxygen to prevent acidification of leachate, and radon or methane movement through the basement of new buildings. The diffusive transport of gases in a GCL or in any porous media can occur following two scenarios. 1) the medium is partially saturated, in this case diffusion will occur mostly within the air filled pores; 2) the medium is fully saturated, in this case the diffusion will occur partly in the gaseous phase and partly in the liquid phase. Both mechanisms of transport are reviewed in detail in Aubertin et al. (2000).

Fick's first law states that the diffusive mass flux of a chemical species across a unit area in an isotropic, steady state of non-reactive solutes, i.e., without undergoing adsorption on to the solids, precipitation and degradation is proportional to the negative concentration gradient measured normal to the area, and can be expressed in one dimensional form as:

$$J_g = -n_e D_e \frac{\partial c}{\partial z} \tag{10}$$

where, J_g is the mass diffusive flux of a gas [M/L^{2}T], n_e is the soil porosity available for solute diffusion (*i.e.* effective porosity), c is the gas concentration in the gaseous phase [M/L^3], D_e is the effective diffusion coefficient of gas [L^2/T], z is a distance (thickness, or height, etc.) [L], $\partial c/\partial z$ is the concentration gradient [M/L^4]. The minus sign in Equation 10 means that mass transfer over time occurs in the direction of decreasing concentration. It can be seen that Equation 10 holds a linear relationship between the flux and the concentration gradient but in reality diffusion is not a function of concentration only and may be affected by the force fields around the molecules (Reid et. al., 1977). Equation 10 is the fundamental equation for diffusion containing four variables. It is difficult to use the above equation in real diffusion problems and a more simple equation is required. The basic principle of Equation 10 can be used for deducing a fundamental differential equation for diffusion. Under one-dimensional transient conditions, the principle of conservation of mass requires that the change in mass flux of a diffusing solute across an infinitesimal soil element ($\partial J/\partial z$) must be equal to the time rate of change of concentration within the element, i.e.,

$$\frac{\partial J}{\partial z} = n_e \frac{\partial c}{\partial t} \tag{11}$$

Equation 11 assumes that there is no change in porosity of the soil element with respect to time (i.e. $\partial n/\partial x = 0$). Equating 1 and 2 for the mass flux and eliminating n_e from both sides, gives:

$$\frac{\partial c}{\partial t} = D_e \frac{\partial^2 c}{\partial^2 z} \tag{12}$$

which is the well known equation for diffusion for non-reactive solute under transient condition (Fick's second law). Equations 10 and 12 can be used for the determination of the effective diffusion coefficient $D_{e,}$ which is dependent on the pores and fluid characteristics such as total porosity, tortuosity, degree of saturation, molecular weight, etc.

Bouazza & Rahman (2004) showed that a decrease of around 3 orders of magnitude in the diffusion coefficient could occur if the degree of saturation increased from 40% to 97% (Figure 36). Similar trends have been reported for other types of needle punched GCLs (Figure 37) and soils (Figure 38). In the case of needle punched GCLs, The differences in GCL structures (i.e. bentonite impregnation, bentonite mass per area, bentonite distribution, needle punching) might explain the differences observed in Figure 37.

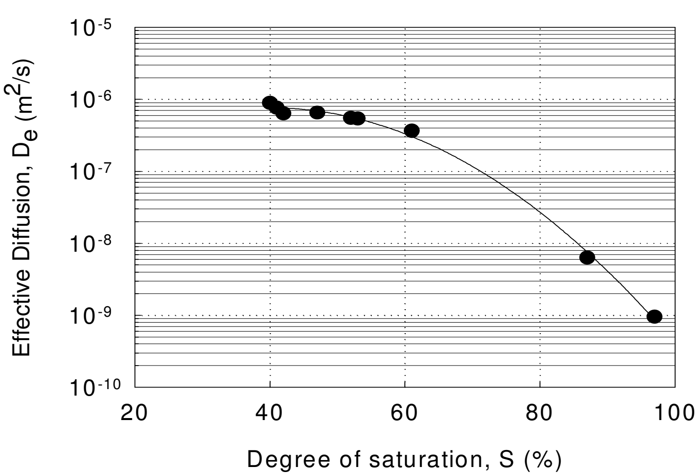


Figure 36. Effective diffusion coefficient of oxygen vs degree of saturation (Bouazza and Rahman, 2004)

Soils on the other hand are different in textures and are heavily dependent on the way they are packed. Soils data collected from several studies reported in the literature seems to plot above the GCL investigated by Bouazza & Rahman (2004) (Figure 38). A larger difference in terms of D_e is noticed at the lower range of saturation where soils per nature contain larger air filled pores. Interestingly, at very high saturation (S≥90%) it seems that this difference is largely reduced and there is not much variation between the two materials. However, more data need to be included before a final conclusion can be made.

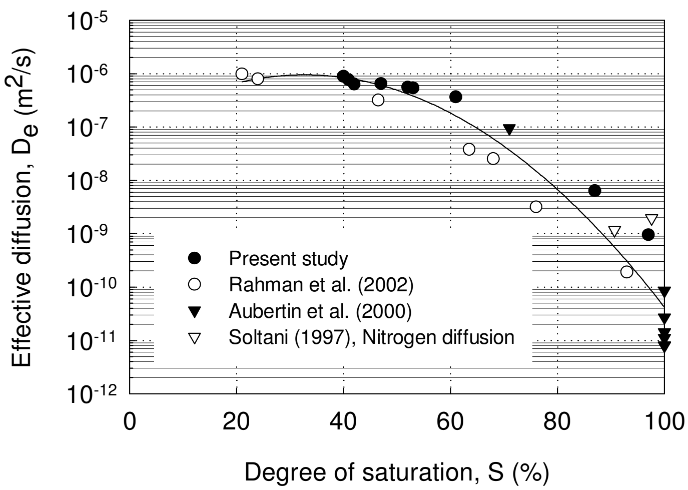


Figure 37. Effective diffusion coefficient of oxygen vs degree of saturation in different GCLs (Bouazza and Rahman, 2004)

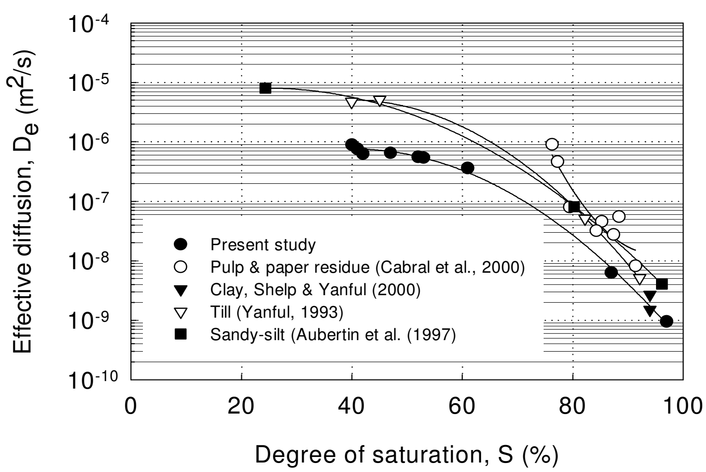


Figure 38. Effective diffusion coefficient of oxygen vs degree of saturation: comparison between study and soils (Bouazza and Rahman, 2004)

3.2.3.4 Shear Strength and Stability

Covers employing GCLs are prone to instability because the drained friction angle of fully hydrated sodium bentonite is approximately 4^o (Olson, 1974). Consequently, reinforced GCLs are used in nearly all cover applications.

The internal shear strength of reinforced GCLs strongly depends on the type of bonding (needled or stitched fibers that penetrate through the thickness of the GCL, or the adhesive to bond the clay to the geotextiles). Figure 39a shows a comparison of stress-strain behaviour for an unreinforced, a stitched and a needle-punched GCL; for displacements less than 1 mm, each product show a similar increase in shear stress with horizontal displacement. For larger displacements, the curves diverge, as after that displacement reinforcements start to be mobilised. After peak strength is reached, the shear strength for all three specimens decreased to residual values that essentially depend on bentonite.

In general the Mohr-Coulomb envelope was found to well fit the experimental data (Figure 39b). The different peak shear strengths are mainly due to the kind of reinforcement and manufacturing which give different failure mechanisms: for unreinforced GCLs the failure occurs in the bentonite layer or at the bentonite-woven geotextile interface; in the needle-punched GCLs, if the fibers are thermally bonded to the carrier geotextile, they are broken under a shear stress, whereas fibers tend to be pull out from the woven geotextile if they are simply tangled with it; as far as stitched GCLs are concerned, the peak strength of stitched GCLs can strongly depend on the direction of shear: results by Fox et al. (1998) showed that when shear stress is applied in the "standard" direction, the stitches locked and failure occurred when the lines of stitching ripped through the woven geotextile; when shear stress was applied in the "reverse" direction, the stitches progressively unravelled and the peak strength was strongly reduced, up to nearly half. Therefore, the in situ placement of stitched GCLs also requires a more careful supervision.

One of the key issue related to the internal shear strength of GCLs is the testing. The shear box, which is the most common equipment, has great advantages consisting in the possibility to control testing conditions in terms of stress level and history, hydration conditions, but these tests suffer the important disadvantages to be expensive, to require experienced personnel to be properly performed and they are complicated to set up. As a consequence, proper testing equipment are rarely available and there is a great variability in the testing apparatus among laboratories. Moreover, when internal shear strength of the GCL is high (reinforced GCLs) and a low normal stress is applied, difficulties in testing are mainly due to the gripping surface that does not always guarantee perfect bonding with the GTs.

Considering the difficulties in testing the needle-punched GCLs, the proposal to use the peel test as an indirect measure of the peak shear strength of needle-punched GCL (e.g. Heerten et al., 1996) could be convenient for pre design assumption. The peel test (recently standardised by ASTM and used for manufacturing quality control) has the advantage to be quick and easy to perform and it can be appropriate considering that no significant influence of the shear rate was observes on the peak shear strength of needle-punched GCLs (Fox et al., 1998).

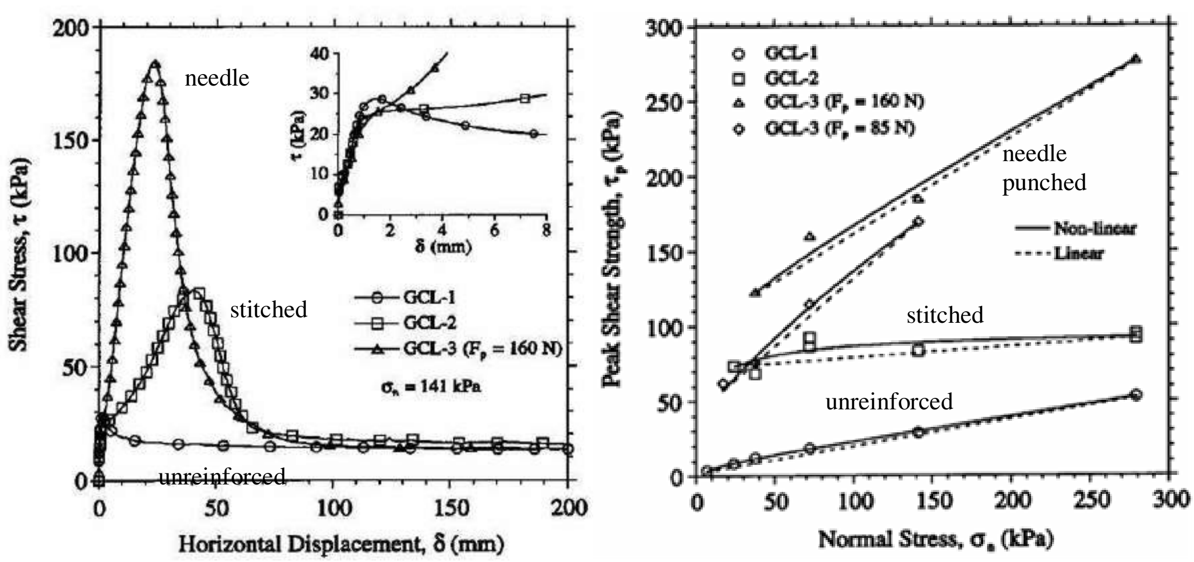


Figure 39. Shear strength of different types of GCLs (Fox et al., 1998)

As GCLs are frequently placed beneath geomembranes to act as composite liners, the shear strength at the interface can be critical for the liner stability. Among the factors governing the interface shear strength, the kind of polymer (both GM and GT) significantly affects the shear strength at the interface.

In Figure 40, the internal shear strength of a needle-punched reinforced GCL is compared with the interface shear strength between the same GCL and a smooth and a rough geomembrane. It is evident that smooth GM-GCL interface gives very low shear strength, generally found to be very close and even less than internal shear strength of unreinforced GCLs. Therefore, a reinforced a GCL should not be used with a smooth geomembrane. On the other hand, it is generally found that the use of rough or textured geomembranes assure higher interface shear strength, that can be very close to the internal shear strength of GCLs.

As far as the influence of the type of GT is concerned, for textured geomembranes, the non-woven side of GCL gives interface with higher shear strength (Triplett & Fox, 2001). This difference becomes much lower or even negligible if a smooth geomembrane is used (Figure 41). However, if a GCL is coupled to a geomembrane at the non-woven site geotextile, the bentonite can pass through the GT if the non woven GT is lower than 200-220 g/m² (Rowe & Jones, 2000), and this can reduce the shear strength at the interface (Figure 42).

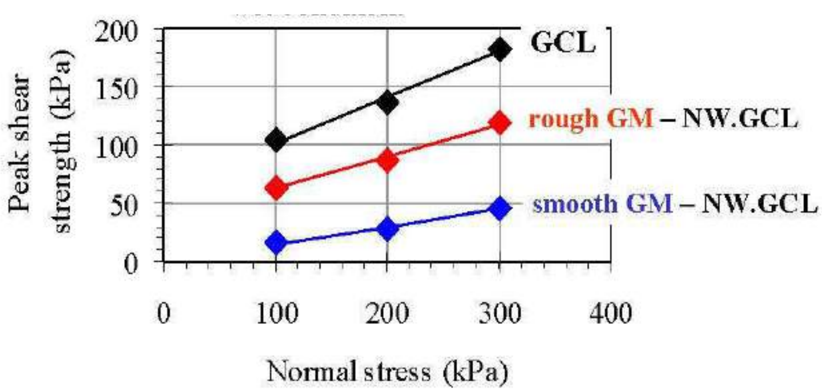


Figure 40. Comparison between peak shear strength of a needle-punched GCL and the shear strength at the interface with a smooth and a rough geomembrane.

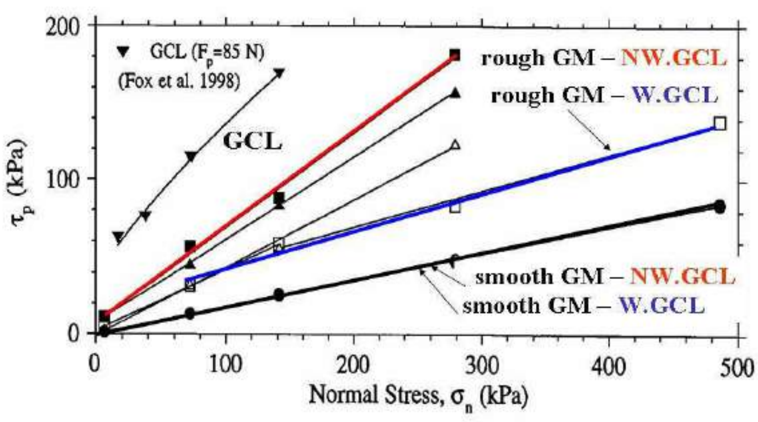


Figure 41. Influence of the type of GM and GT on the shear resistance at the interface between GM and GCL (Triplett & Fox, 2001)

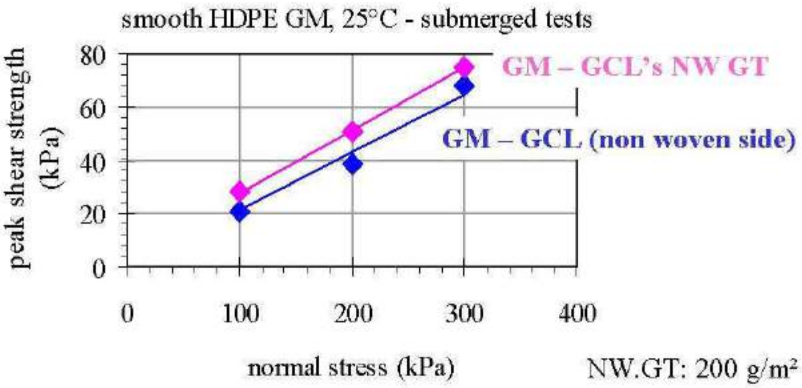


Figure 42. Influence of the bentonite at the interface between geomembrane and GCL.

As far as the operative shear strength of GCLs in final covers is concerned, Daniel et al. (1998) conducted a field study for USEPA using various GCLs and arrangements of soil and geosynthetic layers to construct fourteen test sections on 2:1 and 3:1 slopes. A schematic of a typical test section is shown in Figure 43. After construction, the geosynthetics at the top of the slope were cut, resulting in an infinite slope condition. The test sections were monitored for two years, during which three test sections failed. None of the failures were due to internal failure of the GCL reinforcement but on the geosynthetic interfaces.

Direct shear tests were conducted on all of the interfaces in a large-scale (300 mm x 300 mm) direct shear machine following methods in ASTM D 5321. Friction angles corresponding to peak shear strength (τ_p) and the shear strength at a large (50 mm) displacement (τ_{ld}) were used to back-calculate factors of safety for each test section using an infinite slope analysis. Eight of the test sections that were stable had $F_s > 1.3$ based on τ_p and $F_s > 1.0$ based on τ_{ld}. All of the test sections that failed had $F_s \leq 1.0$ based on τ_p and $F_s < 0.9$ based on $\tau_{ld.}$ Based on these results, a reasonable recommendation for design is to ensure that all slopes have a static factor of safety greater than 1.3 based on peak strength and 1.0 based on the large-displacement strength measured using ASTM D 5321. However, some practitioners prefer that the factor of safety based on the large-displacement strength be at least 1.3.

An important point is that the strength at large displacement measured in ASTM D 5321 is not the residual strength. Displacements greater than 1 m are required to obtain fully residual conditions in GCLs and for geosynthetic interfaces (Stark and Poeppel, 1994; Eid et al., 1999); so large Displacements cannot be induced using ASTM D 5321.

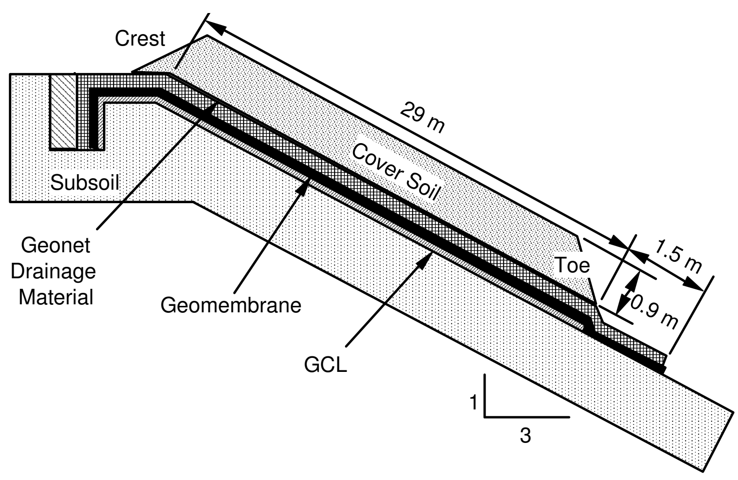


Figure 43. Test section used to assess stability in field study conducted by Daniel et al. (1998).

REFERENCES

Abichou T., Benson C. & Edil T. (2000) Foundry Sands as Hydraulic Barriers: Laboratory Study, *J. of Geotech. and Geoenvir. Engr.,* ASCE.

Acar Y.B. & Seals R.K. (1984) Clay Barrier Technology for Shallow Land Waste Disposal Facilities. *Hazardous Waste*, 1(2).

Albrecht B. (1996) Effect of Desiccation on Compacted Clay. MS Thesis, University of Wisconsin, Madison, USA.

Alzaydi A.A. & Moore C.A. (1978). Combined pressure and diffusional transition region flow of gases in porous media. *AIChE Journal*, Vol. 24, No.1, pp. 35-43.

Aubertin M., Aachib, M., Authier, K. (2000). Evaluation of diffusive gas flux through covers with a GCL. *Int. Journal of Geotextiles & Geomembranes*, Special Issue on GCLs, Ed. A. Bouazza, Vol. 18, No 2-4, pp. 215-233.

Benson C.H. & Daniel D.E. (1994a). Minimum Thickness of Compacted Soil Liners: I. Stochastic Models. *Journal of Geotechnical Engineering*, ASCE, 120(1).

Benson C.H., Daniel D.E. (1994b). Minimum Thickness of Compacted Soil Liners: II. Analysis and Case Histories. *Journal of Geotechnical Engineering*, ASCE, 120(1).

Benson C.H., Hardianto F.S., Motan E.S. (1994). Representative Specimen Size for Hydraulic Conductivity Assessment of Compacted Soil Liners. *Hydraulic Conductivity and Waste Contaminant Transport in Soil,* ASTM STP n. 1142, Philadelphia.

Benson, C. and Khire, M. (1995). Earthen Covers for Semi-Arid and Arid Climates. *Landfill Closures*, ASCE, GSP No. 53, J. Dunn and U. Singh, Eds., 201-217.

Benson, C. and Khire, M. (1997), Earthen Materials in Surface Barriers, *Barrier Technologies for Environmental Management,* National Academy Press, National Research Council, Washington, DC, USA, D79-D89.

Benson, C. and Othman, M. (1993), Hydraulic Conductivity of Compacted Clay Frozen and Thawed In Situ, *J. of Geotech. Engr.,* ASCE, 119(2), 276-294.

Benson, C., Abichou, T., Olson, M., and Bosscher, P. (1995). Winter Effects on the Hydraulic Conductivity of Compacted Clay. *J. of Geotech. Engr.,* ASCE, 121(1), 69-79.

Benson, C., Abichou, T., Wang, X., Gee, G., and Albright, W. (1999). Test Section Installation Instructions Alternative Cover Assessment Program, Environmental Geotechnics Report 99-3, University of Wisconsin, Madison, USA.

Boardman, T. and Daniel, D. (1996). Hydraulic Conductivity of Desiccated Geosynthetic Clay Liners. *J. of Geotech. and Geoenviron. Engr.*, ASCE, 122(3), 204-208.

Bonaparte, R., Othman, M., Rad, N., Swan, R., and Vander Linde, D. (1996). Evaluation of Various Aspects of GCL Performance. *Report of 1995 Workshop on Geosynthetic Clay Liners*, USEPA Report No. EPA/600/R-96/149, Washington, DC, USA, F1-F34

Bouazza, A. and Rahman, F. (2004) Experimental and numerical study of oxygen diffusion through a partially hydrated needle punched geosynthetic clay liner. *Advances in Geosynthetic Clay Liner Technology: 2nd Symposium*, *ASTM STP 1456*, R.E. Mackey and K. Von Maubeuge, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2004.

Bouazza, A. and Vangpaisal, T. (2000). Advective Gas Flux Through Partially Saturated Geosynthetic Clay Liners. *Advances in Transportation and Geoenvironmental Engineering Systems Using Geosynthetics, Geotechnical Special Publication No 103*, 54-67.

Bouazza, A. and Vangpaisal, T., (2003). An Apparatus to Measure Gas Permeability of Geosynthetic Clay Liners. *Geotextiles and Geomembranes,* 21(2): 85-101.

Bouazza, A., Pump, W. (1997). Settlement and the design of municipal solid waste landfills. *Proceedings 1st ANZ Conference on Environmental Geotechnics,* Melbourne, pp. 339-344.

Bowders J.J., Daniel D.E. (1987) Hydraulic conductivity of compacted clay to dilute organic chemicals. Journal of Geotechnical Engineering, Vol. 113, No. 12, 1432-1448.

Bowders J.J., Daniel D.E., Broderick G.P. Liljestrand H.M. (1986) Methods for testing the compatibility of clay liners with landfill leachate. ASTM STP 886, 233-250.

Bowders J.J., Daniel D.E., Wellington J., Houssidas V. (1997) Managing desiccation cracking in compacted clay liners beneath geomembranes. Proc. Geosynthetics ’97, Vol. 1, 527-540.

Bowders, J.J., Bouazza, A. , Loehr, E, Russel, M. (2000). Settlement of municipal solid waste landfills.

Creation of a New Geoenvironment, Fourth Kansai Geotechnical Forum, Japanese Geotechnical Society, pp.101-106.

Brusseau, M.L. (1991). Transport of organic chemicals by gas advection in structured or heterogeneous porous media: Development of a model and application to column experiments. *Water Resources & Research*, Vol. 27, No.2, pp. 3189-3199.

Corser, P. and Cranston, M. (1991) Observations on The Performance of Composite Clay Liners and Covers. *Proc.Geosynthetic Design and Performance*, Vancouver Geotechnical Society, British Columbia, Canada, 1-16.

Daniel D.E. (1989). In Situ Hydraulic Conductivity Tests for Compacted Clay. *Journal of Geotechnical Engineering,* ASCE, Vol. 115 (9), pp.1205-1226.

Daniel D.E. (1991). Compacted Clay Liners and Geosynthetic Clay Linings. Proc., *15th Conferences of Geotechnics of Torino*, Italy.

Daniel D.E. (1993). "Clay Liners". Chapter 7, *Geotechnical Practice for Waste Disposal*, Chapman & Hall, London.

Daniel D.E. (1995). Pollution prevention in landfills using engineering final covers. Proc. *Green '93,* Balkema, Rotterdam.

Daniel D.E. (1997). Critical Issues for Waste Containment in Landfills and Contaminated Sites. Proc., *XIV ICSMFE*, Balkema, Rotterdam.

Daniel D.E. (1998). Landfills for solid and liquid wastes. Proc., *3rd ICEG,* Balkema, Rotterdam, pp. 1231-1246.

Daniel D.E., Koerner R.M. (1995). *Waste Containment Facilities: Guidance for Construction, Quality Assurance and Quality Control of Liner and Cover Systems*. ASCE, New York.

Daniel D.E., Trautwein S.J. (1994). Hydraulic Conductivity and Waste Contaminant Transport in Soil. ASTM STP n. 1142, Philadelphia.

Daniel, D., Koerner, R., Bonaparte, R., Landreth, R., Carson, D., and Scranton, H. (1998). Slope Stability of Geosynthetic Clay Liner Test Plots. *J. of Geotech. and Geoenviron. Engr.*, ASCE, 124(7), 628-637.

Didier, G., Al Nassar, M., Plagne, V. , Cazaux, D. (2000a). Evaluation of self healing ability of geosynthetic clay liners. *Int. Conf. Geotech. & Geological Engng*., GeoEng.2000, Melbourne, Australia

Didier, G., Bouazza, A., Cazaux, D. (2000b). Gas permeability of geosynthetic clay liners. *Int. Journal of Geotextiles & Geomembranes*, Special Issue on GCLs, Ed. A. Bouazza, Vol. 18, No 2-4, pp. 235-250.

Doll P. (1996). Modelling of moisture movement under the influence of of temperature gradients: desiccation of mineral liners below landfills. Ph.D. Thesis, Technical University of Berlin, 251 p.

Dragun J. (1988). The Soil Chemistry of Hazardous Materials. The Hazardous Materials Control Research Institute, Silver Spring, Maryland.

Drumm, E., Boles, D., and Wilson, G. (1997) Desiccation Cracks Result in Preferential Flow. *Geotechnical News*, 15(2), 22-25.

Dullien, F.A.L., (1975). Single Phase Flow Through Porous Media and Pore Structure. *Chemical Engineering Journal,* 10, 1-34.

Dwyer, S. (1998), Alternative Landfill Covers Pass the Test, *Civil Engineering*, 68(9), 30-52.

Edil, T., Ranguette, V., and Wuellner, W. (1990) Settlement of Municipal Refuse. *Geotechnics of Waste Fills - Theory and Practice*, STP 1070, A. Landva and G. Knowles, Eds., ASTM, 225-239.

Eid, H., Stark, T., and Doerfler, C. (1999) Effect of Shear Displacement Rate on Internal Shear Strength of Reinforced Geosynthetic Clay Liner. *Geosynthetics International*, 6(3), 219-239.

Estornell, P. and Daniel, D. E. (1992) Hydraulic Conductivity of Three Geosynthetic Clay Liners. *J. of Geotech. Engrg.*, ASCE 118(10): 1592-1606.

ETC 8, European Technical Committee n. 8 (1993). Geotechnics of landfill design and remedial works technical recommendations-GLR. 2nd edition, Ernst & Sohn (Publ.).

Fernandez F., Quigley R.M. (1985). Hydraulic Conductivity of Natural Clays Permeated with Simple Liquid Hydrocarbons. *Canadian Geotechnical J*, 22.

Fernandez F., Quigley R.M. (1989). Organic Liquids and the Hydraulic Conductivity of Barrier Clays. Proc. *12th ICSMFE*, Rio de Janeiro, A.A.Balkema, Rotterdam.

Fernandez F., Quigley R.M. (1991). Controlling the Destructive Effects of Clay-Organic Liquid Interactions by Application of Effective Stresses. *Canadian Geotechnical J*, 28.

Fuleihan N.F., Wissa A.E.Z. (1995). Discussion on *Minimum Thickness of Compacted Soil Liners: II. Analysis and Case Histories by C.H. Benson and D.E. Daniel (J. of Geotech Engineering, ASCE, vol. 120*

no. 1, 1994). J. of Geot. Engineering, vol 121(6) ASCE, pp. 504-509.
Giroud J.P., Badu-Tweneboah K., Bonaparte R. (1992). Rate of Leakage through a Composite Liner due to Geomembrane Defect". *Geotextiles and Geomembranes*, 11(1).
Giroud J.P., Bonaparte R. (1989a,b). Leakage through Liners Constructed with Geomembranes - Part I, Geomembrane Liners; Part II, Composite Liners. *Geotextiles and Geomembranes*, 8, Elsevier.
Giroud J.P., Soderman K.L., Khire M.V., Badu-Tweneboah (1998). New developments in landfill liner leakage evaluation. Proc. *6th International Conference on Geosynthetics,* Atlanta, IFAI:
Giroud, J., Gross, B., and Durasse, H. (1992) Flow in Leachate Collection Layers, *Design and Performance of Geosynthetic Lining Systems for Waste Containment*. IFAI Publishers, St. Paul, MN.
Harpur, W.A., Wilson-Fahmy, R.F., Koerner, R.M. (1993) Evaluation of the Contact between Geosynthetic Clay Liners and Geomembranes in Terms of Transmissivity. *Proceeding of the 7th GRI Seminar on Geosynthetic Liner Systems*, Philadelphia, GRI.
Heibrock G. (1997). Desiccation Cracking of Mineral Sealing liners. Proc. *2nd Int. Landfill Symposium,* S. Margherita di Pula, Cagliari, CISA.
Hohla M. (1995). Sealing of the Ash Landfill Karlschacht II, Voitsberg, Styria. Proc. *9th Young Geotechnical Engineering Conference,* Ghent, Belgian Member Society of ISSMFE.
Holzlohner U. (1989). Moisture behaviour of soil liners and subsoil beneath landfills. Proc. *2nd International Landfill Symposium,* Porto Conte, 1, pp.X1-X11.
Holzlohner U. (1995). Moisture Balance, Risk of Desiccation in Earthen Liners. Chapter H of the *State of the Art Report: Landfill Liner Systems,* Eds., Penshaw Press.
Izadi, M.T., Stephenson, R.W. (1992). Measurement of gas permeability through clay soils. *Current Practices in Ground Water and Vadose Zone Investigations*, ASTM STP 1118, pp.331-345.
James, A., Fullerton, D., and Drake, R. (1997), Field Performance of GCL Under Ion Exchange Conditions, *J. of Geotech. and Geoenviron. Engr.*, ASCE, 123(10), 897-901.
Jarre, P., Mezzalama, R. and Luridiana, A. (1997). Lessons to be Learned from a Fatal Landfill Gas Explosion. *Proceedings 6th International Landfill Symposium*, Cagliari, Italy, 497-506.
Jessberger H.L. (1994). Emerging Problems and Practices in Environmental Geotechnics. Proc. *13th ICSMFE*, New Delhi, Oxford & IBH Publishing Co. Pvt. Ltd., New Delhi.
Jessberger H.L. (1995). Waste Containment with Compacted Clay Liners. Proc. *The Geoenvironment 2000*, ASCE, GSP n. 46, New York.
Jessberger H.L., Demmert S., Heibrock G. (1993). Safety Concepts for Landfill Liners. Proc. Int. Conference *The Environment and Geotechnics*, Paris, Presses de L'Ecole Nationale des Ponts et Chaussees.
Jessberger, H. and Stone, K. (1991) Subsidence Effects on Clay Barriers. *Géotechnique*, 41(2), 185-194.
Jones R.M., Murray E.J., Rix D.W., Humphrey R.D. (1993). Selection of clays for use as landfill liners. Proc., *Green '93 – Waste Disposal by Landfill,* Bolton, UK, Balkema, Rotterdam.
Kaczmarek M., Hueckel T., Chawla V., Imperiali P. (1997). Transport through a clay barrier with the contaminant concentration depending permeability. *Transport in Porous Media,* Kluwer, Dordrecht, The Netherlands, 29(2), 159-178.
Khire, M., Benson, C., and Bosscher, P. (1997) Water Balance Modeling of Earthen Landfill Covers. *J. of Geotech. and Geoenviron. Engr.,* ASCE, 123(8), 744-754.
Kjeldsen, P. and Fisher, E.V., (1995). Landfill Gas Migration - Field Investigations at Skellingsted Landfill, Denmark. *Journal of Waste Management and Research*, 13, 467-484.
Koerner R.M., Lord A.E., Hsuan Y.K. (1993). Stress cracking behaviour of HDPE geomembranes and its prevention. GRI Report 9, 94 p.
Koerner, R.M. and Daniel, D.E. (1997). *Final Covers for Solid Waste Landfills and Abandoned Dumps*. ASCE Press, 256 pp.
Kraus, J., Benson, C., Erickson, A., and Chamberlain, E. (1997) Freeze-Thaw and Hydraulic Conductivity of Bentonitic Barriers. *J. of Geotech. and Geoenviron. Engr.,* ASCE, 123(3) 229-238.
LaGatta, M., Boardman, B., Cooley, B., and Daniel, D. (1997) Geosynthetic Clay Liners Subjected to Differential Settlement. *J. of Geotech. and Geoenviron. Engr.*, ASCE, 123(5), 402-410.
Lake, C.B., Rowe, R.K. (2000). Diffusion of Sodium and Chloride Through Geosynthetic Clay Liners. *Geotextiles and Geomembranes*, 18 (2-4), 103-131.
Lin, L. and Benson, C. (2000) Effect of Wet-Dry Cycling on Swelling and Hydraulic Conductivity of Geosynthetic Clay Liners. *J. of Geotech. and Geoenviron. Engr.,* ASCE, 126(1), 40-49.
Lo M.C. (1992). Development and Evaluation of Clay Liner Material for Hazardous Waste Sites. *Ph.D.*

Thesis, University of Texas.
Manassero M. (1997). Pollutants Containment Via Passive Barriers. Proc. *14th ICSMFE*, Discussion Session 6.1, Hamburg.
Manassero M., Benson C.H., Bouazza A. (2000) Solid waste containment systems. Proc. GeoEng2000, Melbourne, Australia.
Manassero M., Pasqualini E. (1993) Design and Construction of Barricalla Hazardous Waste Landfill. *Geotechnical News*, BiTech Publishers, Richmond, B.C., 11(4).
Manassero M., Pasqualini E., Sani D. (1998). Potassium Sorption Isotherms of a Natural Clayey-Silt for Pollutant Containment. Proc., 3rd ICEG, Lisbon, Balkema, Rotterdam.
Manassero M., Shackelford C.D. (1994a). Classification of Industrial Wastes for Re-Use and Landfilling. Proc., *1st ICEG*, Edmonton, BiTech Publishers, Richmond B.C.
Manassero M., Shackelford C.D. (1994b). "he Role of Diffusion in Contaminant Transport through Soil Barriers. *Rivista Italiana di Geotecnica*, AGI, 28(1).
Manassero M., Van Impe W.F., Bouazza A. (1996). Waste Disposal and Containment. State of the Art Report, Proc., *2nd ICEG*, Osaka, A.A. Balkema, Rotterdam.
Manheim F.T. (1970). The Diffusion of Ions in Unconsolidated Sediments. *Earth and Planetary Science Letters,* North Holland Publishing Company, 9.
Massmann, J.W., (1989). Applying Groundwater Flow Models in Vapor Extraction System Design. *Journal of Environmental Engineering*, ASCE, 115(1), 129-149.
Mazzieri F. (1998) Hydraulic performance of GCLs. PhD Thesis, University of Ancona, Italy (in Italian).
Mazzieri F., Pasqualini E. (1997). The long term performance of GCLs: a case study. Proc., *Geoenvironment '97,* 1st International Australian New Zealand Conference on Environmental Geotechnics, Melbourne.
Mazzieri F., Pasqualini E., Van Impe W. (2000) Compatibility of GCLs with organic solutions. Proc. GeoEng2000, Melbourne, Australia.
Mazzieri, F.P., Pasqualini, E., 2000. Permeability of damaged geosynthetic clay liners. *Geosynthetics International* 7(2), 101-118.
Melchior, S. (1997). In Situ Studies on the Performance of Landfills Caps. *Proc. Intl Containment Technology Conf.,* US Dept. of Energy, Germantown, Maryland, USA, 365-373.
Mesri G., Olson R.E. (1971) Mechanisms Controlling the Permeability of Clays. *Clays and Clay Minerals*, 19.
Mitchell J.K., Bray J.D., Mitchell R.A. (1995). Material Interactions in Solid Waste Landfills. Proc., International Conference *The Geoenvironment 2000*, ASCE GSP n. 46, New York.
Mitchell J.K., Seed R.B., Seed H.B. (1990). Kettleman Hills Waste Landfill Slope Failure. I: Liner System Properties. *Journal of Geotechnical Engineering*, ASCE, 116(4).
Montgomery, R. and L. Parsons (1990), The Omega Hills Cover Test Plot Study: Fourth Year Data Summary, *Proc. 22nd Mid-Atlantic Industrial Waste Conf.*, Drexel University, Philadelphia, Pennsylvania, USA.
Olinic, E. (2000). Landfill bottom lining systems. Master of Science Thesis. Politecnico di Torino.
Olson, R. (1974), Shearing Strengths of Kaolinite, Illite, and Montmorillonite. *J. of the Geotech. Engr. Division*, ASCE, 100(11), 1215-1229.
Othman M.A., Bonaparte R., Gross B.A. (1996). Preliminary Results of Study of Composite Liner Field Performance. Proc., 10th GRI Conference *Field Performance of Geosynthetics and Geosynthetic Related Systems*.
Othman, M. and Benson, C. (1994) Effect of Freeze-Thaw on the Hydraulic Conductivity and Morphology of Compacted Clay. *Canadian Geotech. J.*, 30(2), 236-246.
Othman, M., Benson, C., Chamberlain, E., and Zimmie, T. (1994) Laboratory Testing to Evaluate Changes in Hydraulic Conductivity Caused by Freeze-Thaw: State-of-the-Art. *Hydraulic Conductivity and Waste Containment Transport in Soils,* STP 1142, ASTM, S. Trautwein and D. Daniel, Eds., 227-254.
Pasqualini E. & Fratalocchi E. (2000). In situ and Laboratory tests in environmental geotechnics. *Italian Geotechnical Journal*, (in Italian) 34, n. 3 (suppl.), 17-37.
Pasqualini E. & Stella M. (1994). Le tensioni di contatto tra i singoli componenti strutturali e sovrapposti. Proc. Updating Seminar *Meccanica dei rifiuti e analisi strutturale dei componenti di una discarica controllata*, S. Margherita di Pula, CISA, Cagliari. (in Italian).
Pasqualini E., Fratalocchi E., Stella M. (2002) Stability of liners: some particular issues. Proc. IV Int. Conf. Environmental Geotechnics, Rio de Janeiro, Brasil.

Petrov R.J., Rowe R.K., (1997). Geosynthetic Clay Liner Compatibility by Hydraulic Conductivity Testing: Factors Impacting Performance. *Canadian Geotechnical Journal*, vol. 34(6).

Quigley R.M. Fernandez F., Rowe R.K. (1988). Clayey Barrier Assessment for Impoundment of Domestic Waste Leachate (Southern Ontario) Including Clay-Leachate Compatibility by Hydraulic Conductivity. *Canadian Geotechnical J.,* 25(8).

Quigley R.M., Fernandez F. (1989). Clay/Organic Interactions and their Effect on the Hydraulic Conductivity of Barrier Clays. Proc. *International Conference Contaminant Transport in Groundwater*, A.A. Balkema, Rotterdam.

Rabideau A., with contributions from Mercer J. (session co-leader), Andrews C., Chiang C., Grathwohl P., Hayworth J., Culligan-Hensley P., Manassero M., Mott H., Schroeder P., Shackelford C., Teutsch G., van Meurs G., Wilhelm R. e Zheng C. (1996). Contaminant Transport Modeling. *International Containment Technology Workshop*. Sponsored by U.S. Department of Energy, U.S. Environmental Protection Agency, DuPont Industries - Baltimore.

Reid R.C., Prausnitz, J. M. and Sheerwood, T. K. (1977) *The Properties of Gases and Liquids*, 3rd edition, New York: McGraw Hill.

Rowe R.K. (1988). Contaminant Migration through Groundwater: The Role of Modelling in the Design of Barriers. *Canadian Geotechnical .J*, 25(4).

Rowe R.K. (1998a). Geosynthetics and the Minimization of Contaminant Migration through Barrier Systems Beneath Solid Waste. Keynote Lecture, *6th Int. Conf. on Geosynthetics*, Atlanta, IFAI.

Rowe R.K., Fleming I., Cullimore D.R., Kosaric N., Quigley R.M. (1995b). A Research Study of Clogging and Incrustation in Leachate Collection Systems at Municipal Solid Waste Landfills. *Rep.,* Submitted to Interim Waste Authority.

Rowe R.K., Quigley R.M., Booker J.R. (1995a). *Clayey Barrier Systems for Waste Disposal Facilities*. E & FN Spon, London.

Rowe RK., Jones C.J.F.P.(2000) Geosynthetics: innovative materials and rational design. *Proc.GeoEng2000*, Melbourne, Australia.

Seed R.B., Mitchell J.K., Seed H.B. (1990). Kettleman Hills Waste Landfill Slope Failure. II: Stability Analyses. *Journal of Geotechnical Engineering*, ASCE, 116(4).

Shackelford C.D. (1991). Laboratory Diffusion Testing for Waste Disposal. a Review. *Journal of Contaminant Hydrology, Elsevier,* 7(3).

Shackelford C.D. (1994a). Critical Concepts for Column Testing. *Journal of Geotechnical Engineering,* ASCE, 120(10).

Shackelford C.D.(1994b). Waste-Soil Interaction That Alter Hydraulic Conductivity. *Hydraulic Conductivity and Waste Contaminant Transport in Soil*, ASTM STP n. 1142, Philadelphia.

Shackelford C.D., Benson C.H., Katsumi T., Edil T.B., Lin L. (2000). Evaluating the hydraulic conductivity of GCLs permeated with non-standards liquids. *Geotextiles and Geomembranes,* 18, pp. 133-161.

Shackelford C.D., Daniel D. E. (1991a). Diffusion in Saturated Soil. I: Background. *Journal of Geotechnical Engineering,* 117(3), ASCE New York.

Shackelford C.D., Daniel D. E. (1991b). Diffusion in Saturated Soil. II: Results for Compacted Clay. *Journal of Geotechnical Engineering,* 117(3), ASCE New York.

Shackelford C.D., Nelson J.D. (1996). Geoenvironmental design considerations for tailings dams. Proc. *Seismic and environmental design of earth dams, concrete dams and tailings dams*. Santiago, Chile.

Shackelford C.D., Redmond P.L. (1995). Solute Breakthrough Curves for Processed Kaolin at Low Flow Rates. *Journal of Geotechnical Engineering*, ASCE, 121(1).

Shan H.Y., Daniel D.E. (1991). Results of laboratory tests on a geotextile/bentonite liner material. Proc., *Geosynthetics '91,* IFAI, Saint Paul, MN, 2, pp. 517-535.

Shelp, M.L. and Yanful, E.K., (2000). Oxygen Diffusion Coefficient of Soils at High Degrees of Saturation. *Geotechnical Testing Journal*, 23(1), 36-44.

Sherman, V.W., (2000). GCL Durability and Lifetime - A State's Perception. *Proceedings 14th GRI conference: Hot Topics in Geosynthetics - I.,* Las Vegas, USA,. 238-241.

Stark, T. and Poeppel, A. (1994) Landfill Liner Interface Strengths from Torsional-Ring-Shear Tests. *J. of Geotech. Engr.*, ASCE, 120(3), 597-615.

Stark, T.D., Arellano, d., Evans, W.D., Wilson, V.L., Gonda, J.M. (1998). Unreinforced geosynthetic clay liners case history. *Geosynthetics International*, Vol. 5 (5), pp. 521-544.

Stella M. (1998). Shear strength at interfaces of composite liners. PhD Thesis, University of Ancona, Italy (in Italian).

Thiel R.S. (1999). Design of gas pressure relief layer below geomembrane landfill cover to improve slope stability. *Geosynthetics International*, Vol.5, No.6, pp.589-617

Trautwein S.J., Boutwell G.P. (1994). In Situ Hydraulic Conductivity Tests for Compacted Soil Liners and Caps. *Hydraulic Conductivity and Waste Contaminant Transport in Soil,* ASTM STP 1142.

UNESCO (1979) Map of the World Distribution of Arid Regions. MAB Technical Note 7, United Nations Educational, Scientific, and Cultural Organization, Paris.

Vangpaisal, T. (2003) *Gas* Permeability of Geosynthetic Clay Liners under Various Conditions. PhD Thesis, Department of Civil Engineering. Monash University, Melbourne.

Vangpaisal, T. and Bouazza, A. (2001). Gas Permeability of Three Needle Punched Geosynthetic Clay Liners. *Proceedings 2nd ANZ Conference on Environmental Geotechnics*, Newcastle, Australia, 373-378.

Vangpaisal, T. and Bouazza, A. (2003). Gas leakage rate through a circular defect in a GM/GCL composite. Proceedings 9th International Waste Management and Landfill Symposium, Cagliari (CD Rom)

Vangpaisal, T. and Bouazza, A. (2004). Gas permeability of partially hydrated geosynthetic clay liners. *Journal of Geotechnical and Geoenvironmental Engineering, Vol. 130 No1.*

Wagner J.F., Egloffstein Th., Czurda K.A. (1990). The Pros and Cons of Different Suitability Tests for Clay Liners. Proc. 3rd International Conference *Contaminated Soil '90*, Karlsruhe, Kluwer Academic Publishers, Dordrecht.

Williams, G.M. and Aitkenhead, N., (1991). Lessons from Loscoe: The Uncontrolled Migration of Landfill Gas. *Quarterly Journal Engineering Geology*, 24,191-207

Yanful, E. (1993). Oxygen diffusion through soil covers on sulphidic mine tailings. *J. of Geotech. Engng.,* Vol. 119, N0 8, pp. 1207-1228.

Zimmie T.F. & La Plante C. (1990) The effect of freeze-thaw cycles on the permeability of a fine-grained soil. Proc. Mid-Atlantic Industrial waste Conference, Philadelphia, Pennsylvania, USA.

PART II – INNOVATIVE BARRIERS

3.1 INTRODUCTION

This report provides a brief overview of some current and promising technologies for waste containment applications that are considered to be innovative from the viewpoint that their use has not gained widespread acceptance at the present time. The technologies discussed are classified into two broad categories: (1) innovative covers and cover materials, and (2) innovative liners and liner materials.

Innovative covers and cover materials pertain to the use of covers (or caps) for final closure of landfills or other waste disposal scenarios, as well as for use as horizontal surface barriers to minimize the leaching of contaminants into the ground water. With respect to waste disposal scenarios, the purposes of the cover system is to prevent the generation of leachate by minimizing the amount of precipitation percolating through the waste during the inactive (post closure) period, and to provide for containment and prevent physical dispersion by wind and water. With respect to applications involving remediation, a cover or cap may be considered as the only technology needed in cases where the climate is arid, the water table is deep, and the site is relatively isolated (Shackelford, 1999; Shackelford and Jefferis, 2000). An example of this application is the Hanford Barrier that has been designed to isolate single-shell tank wastes and transuranic-contaminated soil sites for a minimum of 1000 years at the U. S. Department of Energy's Hanford site near Richland, Washington, USA. A cover or cap also may be used as an *in situ* barrier to prevent the potential for overtopping of the contaminated ground water due to infiltration within a vertically contained zone, an occurrence known as the "bathtub effect" (Shackelford, 1999; Shackelford and Jefferis, 2000).

Innovative liners and liner materials pertain to waste disposal applications requiring the use of liners to prevent leachate or a contaminant waste stream resulting from a waste disposal practice from migrating directly into the underlying soil during both the active disposal period as well as the post-closure, or inactive, period of the containment facility. The liner technologies described herein are in contrast to the use of the more traditional compacted clay and/or geosynthetic liners and materials.

3.2 INNOVATIVE COVERS AND COVER MATERIALS

3.2.1 Alternative Earthen Final Covers (AEFCs)

3.2.1.1 *Introduction*

Alternative earthen final covers (AEFCs) are earthen covers designed on water storage principles that perform equally as well as their traditional counterpart (i.e., composite or compacted clay covers), have greater durability, or lower cost. The high cost associated with composite final covers (~$400,000US to $500,000US/ha) and the frequent failure of compacted clay covers has led to interest in alternative earthen final covers (AEFCs) in drier regions. In addition, AEFCs are perceived to be more harmonious and congruent with nature. AEFCs can be as simple as a monolithic layer of vegetated finer-grained soil or as complex as a multilayer anisotropic capillary barrier (Benson and Khire, 1995; Stormont, 1995; Gee and Ward, 1997). AEFCs currently are receiving significant attention in North America, particularly in western regions that have semi-arid and arid climates.

AEFCs act like a sponge in several ways. First, AEFCs store water during periods of elevated precipitation and limited evapotranspiration in the same way as a dry sponge stores water that is wiped from a countertop. Subsequently, the stored water in AEFCs is released to the atmosphere during drier periods with higher evapotranspiration just as a sponge when not in use. Finally, deep percolation from an AEFC may occur if increase in moisture storage takes place within the cover. AEFCs are suitable in drier regions where potential evapotranspiration (PET) far exceeds precipitation, and can be designed to have sufficient storage capacity to retain water during wet periods without transmitting appreciable percolation. Although a variety of design concepts have been considered for AEFCs, the most common design concepts can be classified as either monolithic barriers or capillary barriers.

3.2.1.2 Monolithic or Evapotranspirative Covers

Monolithic covers (MCs) consist of a thick vegetated layer of comparatively fine-textured soil that has high water storage capacity. Monolithic covers are also referred in the literature as monocovers, soil-plant covers, or evapotranspirative covers. The terminology "evapotranspirative cover", results from the fact that MCs are usually vegetated with native plants that survive on the natural precipitation.

The superior performance in arid climates of MCs relative to conventional resistive covers can be attributed to their comparatively low hydraulic conductivity under the unsaturated conditions that prevail in arid areas. In addition, an MC is made sufficiently thick so that water contents near the base of the cover remain fairly low. Under this condition, percolation from the base of the cover can be small enough to meet target percolation rates. Additional advantages of MCs over typical clay barrier systems are that they typically are less vulnerable to desiccation and cracking during and after installation, they are relatively simple to construct, and they require low post-closure maintenance. Also, MCs are economical to implement since, as they can be constructed of a reasonably broad range of soils, they are typically constructed using soils from a nearby area. Finally, MCs may represent a technically superior alternative relative to traditional covers if the cover design is governed by stability considerations, as is the case for the design at the OII Superfund landfill described herein.

The target percolation rate is selected based on the percolation rate associated with the prescriptive cover that the monolithic cover is to replace. The Alternative Cover Assessment Program (ACAP) sponsored by the US Environmental Protection Agency (EPA) has defined target percolation rates for humid climates and semi-arid (or drier) climates that correspond to compacted clay and composite covers. These percolation rates are summarized in Table 1. A method to design a monolithic cover using site-specific meteorological data and soil-water characteristic curves (SWCCs) for the soil is described by Chen (1999).

Table 1. Equivalent percolation rates for prescriptive final covers (Benson, 1999; Manassero et al., 2000)

Type of Prescriptive Cover	Maximum Annual Percolation (mm/yr) [1]	
	Semi-Arid and Drier ($P/PET \leq 0.5$)	Humid ($P/PET > 0.5$)
Compacted Clay (or lesser)	10	30
Composite	3	3

[1] P = precipitation; PET = potential evapotranspiration

A rational basis for selection of parameters that govern the design of MCs, such as the thickness of the soil cover layer and the rooting depth of the vegetation, is provided by Zornberg et al. (2003). A sensitivity analysis of the components of the water balance was performed using unsaturated flow modeling of a generic MC system for weather conditions typical of southern California. The parametric evaluation indicates that a MC with a thickness as small as 600 mm in the semi-arid climate of southern California satisfies stringent percolation design criteria. Generic cover evaluations can provide the basis for site-specific unsaturated flow investigations, such as the one undertaken for compilation of the design of an evapotranspirative cover system at the Operating Industries, Inc. (OII) Superfund landfill in southern California. In this case, equivalence demonstration to evaluate compliance of the proposed alternative cover with the prescriptive cover system was based on unsaturated flow analyses performed for both covers using site-specific weather conditions and soil-specific hydraulic properties.

3.2.1.3 Capillary Barriers

A significant amount of study recently has been devoted to the use of capillary barrier covers (CBCs) for waste disposal (e.g., Aubertin et al., 1994; Khire et al., 1994; Benson and Khire, 1995; Stormont et al., 1996; Watkins, 1996; Stormont and Anderson, 1999). A capillary barrier effect results when unsaturated flow occurs through a relatively fine layer of soil overlying a relatively coarse layer of soil, as illustrated in Figure 1a. The fine layer does not have to be a fine-grained soil, but rather simply must be finer than the underlying coarser soil. When the incipient wetting front reaches the interface between the two soil layers, the wetting front does not pass unabated into the lower, coarser soil due to the residual suction remaining in the finer layer after passage of the incipient front (Figure 1b), and the comparatively lower unsaturated hydraulic conductivity of the coarser layer relative to that of the finer layer with increasing matric suction (Figure 1c). As a result, only a fraction of the incipient wetting flux is transmitted into the underlying coarser material, and the infiltrated water begins to fill the remaining air voids in the finer layer (Figure 1d). Breakthrough occurs when the matric suction at the interface between the layers reaches a value corresponding to the sharp bend in the SWCC of the coarser soil near residual water content (Khire et al., 2000).

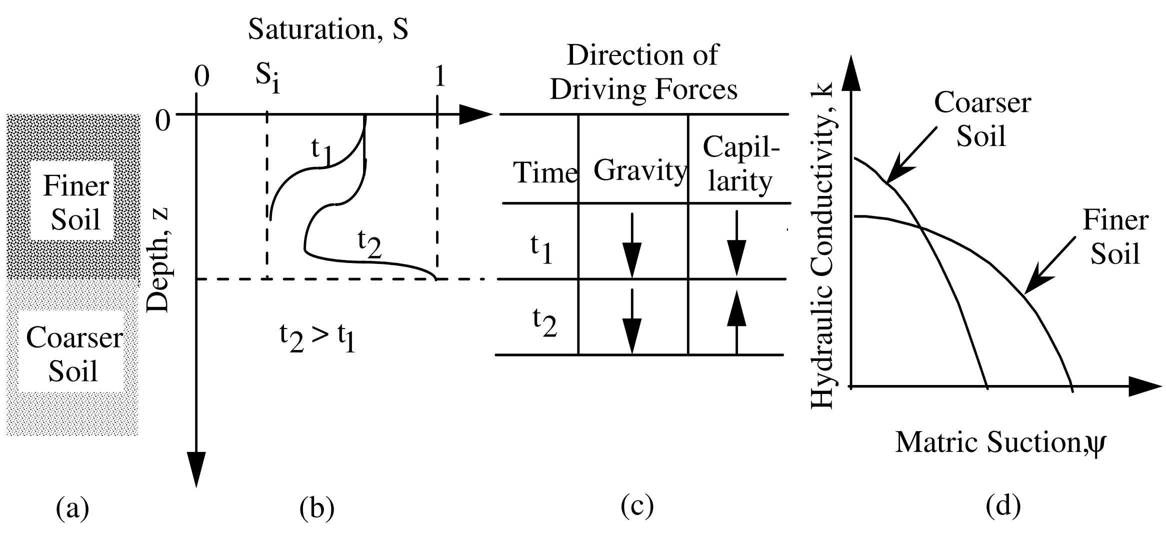


Figure 1. Conceptual layering in a capillary barrier cover: (a) cross-section; (b) wetting fronts; (c) driving forces; and (d) unsaturated hydraulic conductivity relationships (Shackelford and Nelson, 1996).

In general, the effectiveness of a CBC increases with an increase in the contrast in soil properties (e.g., unsaturated hydraulic conductivity) between the finer and coarser layers and when the water storage capacity or the residual suction of the finer layer is maximized. Complete saturation of the finer layer during migration of the incipient wetting front will destroy the capillary barrier effect, and the CBC will fail. Thus, a CBC generally is considered plausible only in regions with relatively small precipitation events, such as in arid and semi-arid climates. However, even in arid and semi-arid regions, provision must be made for adequate lateral drainage of infiltrating water to minimize the potential for saturation of the finer layer particularly when the finer layer is relatively thin.

The thickness of the finer layer (L_f) is sized to have sufficient water storage capacity to store water during cooler and wetter months while limiting percolation to below a prescribed threshold (Stormont and Morris, 1998; Khire et al., 2000). The required storage capacity can be conservatively assumed to equal the amount of precipitation received outside the growing season during the wettest year on record and the snowiest year on record.

The thickness of the coarser layer is not nearly as important as that of the finer layer since the coarser layer provides little storage capacity. The coarser layer only needs to be thick enough to provide a good working platform for placement of the finer layer and, in some cases, adequate lateral drainage capacity. A layer 300-mm thick is generally adequate (Khire et al., 2000).

The major design considerations for CBCs can be summarized through the *capillary barrier pyramid* shown in Figure 2. In general, an increase in CBC order correlates with an increase in the contrast of the properties (e.g., unsaturated hydraulic conductivity) of the soils used in the capillary barrier. However, an increase in CBC order also correlates with an increase in the potential for piping (i.e., migration of finer soil particles through the coarser soil particles). Lateral drainage through the top layer has been considered in some CBC designs to minimize the potential for saturation of the top layer and subsequent failure of the CBC. In this regard, 1st order CBCs with sand as the top layer usually are preferred. However, this approach may require excessively thick top layers to ensure that the water holding capacity during lateral drainage is not exceeded causing failure of the CBC. An alternative approach is to design a lateral drainage layer immediately above

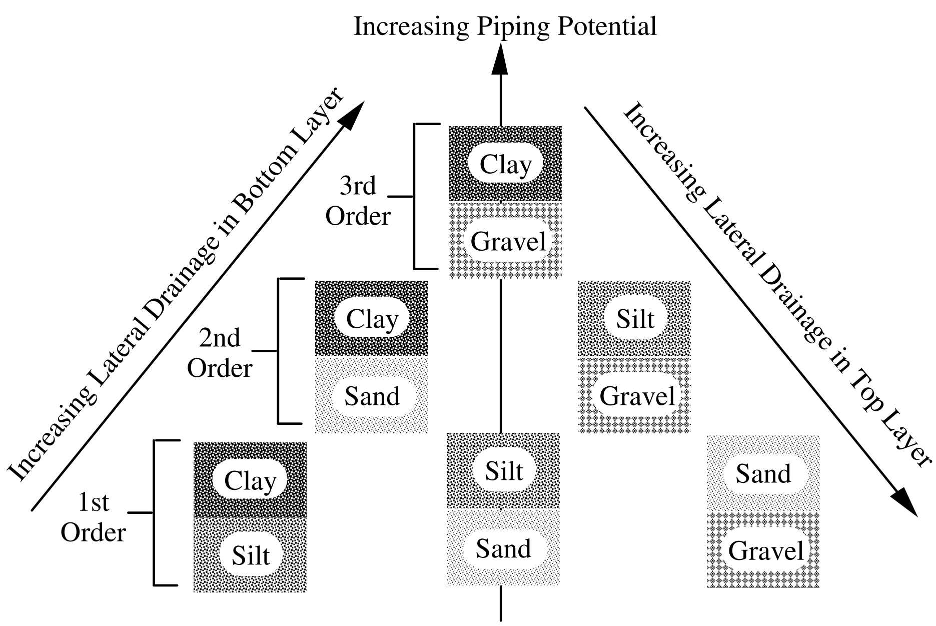


Figure 2. The capillary barrier pyramid (Shackelford and Nelson, 1996)

the top layer of a CBC to minimize the amount of ponded water. As previously noted, lateral drainage through the bottom layer is not a significant design consideration except in the case of failure of the CBC in which case the lower drainage layer helps to minimize infiltration of water into underlying waste. The design of liquid collection layers is provided by Giroud et al. (2000).

Another major factor governing the selection is the concern for desiccation of clays in arid and semi-arid regions that typically plagues the performance of the traditional, prescriptive covers. In this regard, dry-side compaction of a fine-grained soil used as the finer layer helps to reduce the potential for desiccation while simultaneously increasing the water storage capacity (Shackelford and Nelson, 1996). Also, CBCs that use a silt or sand as the finer layer also offer a measure of protection against desiccation cracking. Thus, given all of the above design considerations, silt/sand 1st order, clay/sand 2nd order, or silt/gravel 2nd order CBCs probably offer the best potential CBC profiles.

3.2.1.4 Site-Specific Design Considerations for AEFCs

An aspect of AEFC design that differs from the design of prescriptive covers is that the cover profile is a function of meteorological conditions and vegetation. Thus, the cover thickness varies with location. For example, Winkler (1999) conducted a modeling study evaluating how the thickness of MC systems varies with location throughout the western United States. He used site-specific meteorological data and vegetative properties in his analysis. A contour map of the thickness of silt loam required to achieve an annual percolation rate of 10 mm/yr resulting from Winkler's analysis is shown in Figure 3. The silt loam had a saturated hydraulic conductivity of 10^{-7} m/s.

The map in Figure 3 indicates that very thin covers may be possible in the desert southwestern United States (e.g., Arizona, Nevada, New Mexico) whereas much thicker covers probably are required in the cool deserts where significant snowfall occurs (e.g., Montana, Wyoming, North Dakota). Zornberg et al. (2003) emphasize the need for site-specific weather and soil-specific hydraulic input parameters for the design of AEFCs. Such analyses provide insight into design aspects such as the effect of irrigation programs to sustain permanent vegetation, of increased natural precipitation, of the initial moisture content of the cover, and of the potential increase in the hydraulic conductivity of cover soils induced by root penetration.

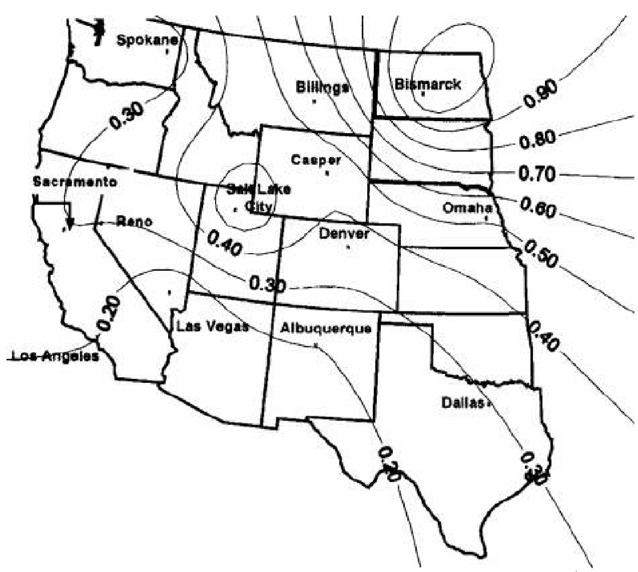


Figure 3. Contour map of the western US showing required thickness (m) of silt loam monolithic cover required to meet a target percolation rate of 10 mm/yr (adapted from Winkler, 1999).

3.2.1.5 Suitable Soil Properties and Placement Conditions for AEFCs

Vegetation plays an important role in the performance of AEFCs since vegetation facilitates removal of stored water (Fayer et al. 1996). Although evaporation from the soil surface can remove water, transpiration by plants is necessary in all but the driest regions if percolation is to be limited to very small amounts (e.g., less than 3 mm/yr). Consequently, soils that are suitable for vegetation should be used for the portion of AEFCs where vegetation will be established. In addition, soils that have low potential for desiccation cracking and frost damage should be used so that preferential flow will not be problematic. Soils meeting these criteria normally classify as silty sands, silts, silty clayey sands, clayey silty sands, and similar materials. Erosion of these soils can be problematic in some cases. Thus, vegetation should be established as soon as possible after construction is complete and nurtured to maturity. Maturation normally requires three to five years.

Since the soil is intended to be a medium for vegetation growth, only modest compaction should be used. This may lead to the use of stabilization methods other than high compaction in projects involving steep cover systems. Compaction specifications for AEFCs typically stipulate that the finer layer be compacted to approximately 85 % and at most 90 % of maximum dry unit weight based on standard Proctor (RMA, 1997; Benson et al., 1999; Zornberg et al., 2003). This level of compactive effort can usually be delivered using rubber-tire construction equipment. Modest compaction also ensures that the pore structure of the soil is not prone to large changes caused by shrinking and swelling or frost action; i.e., the pores are reasonably large after compaction and thus pedogenesis is unlikely to cause a major change in pore size. The use of geosynthetic reinforcements to stabilize steep AEFCs is reported by Zornberg et al. (2001).

3.2.1.6 A Case History

Results from a generic evaluation of a baseline evapotranspirative cover (ETC), performed using site-specific weather information for southern California, were used as the basis for the design of the cover system at the OII Superfund site (Zornberg and Caldwell, 1998; Zornberg et al., 2003). The site is located in the city of Monterey Park, California, approximately 16 km east of downtown Los Angeles. Before implementation of the final closure system at the site, the refuse mass reached over 76 m above grade with slopes as steep as 1.3H:1V. The landfill, a former sand and gravel quarry pit excavated up to 60 m deep in places, was filled with solid and liquid wastes over a 40-year period. There is no evidence indicating that subgrade preparation or installation of a liner system took place prior to the placement of solid waste in the quarry. The maximum vertical thickness of the solid waste in the landfill is approximately 100 m. The landfill received waste until 1984, when an interim soil cover of variable thickness (1 to 5 m), consisting of silty clay to silty sand, was placed on top of the landfill. The site has been undergoing final closure under the US EPA Superfund program since 1986.

Selection of the final cover system at the site was driven by stability concerns, which led to the identification of alternative covers such as an exposed geomembrane cover and an ETC system. Although an exposed geomembrane cover would be stable under both static and seismic conditions, evaluation of the uplift by wind of the geomembrane becomes a key design consideration (Zornberg and Giroud, 1997). An ETC system then was selected because of aesthetic, economical, and technical considerations. This system constitutes the first evapotranspirative cover approved by the USEPA for construction at a Superfund site. Selection of this system allowed use of geogrid reinforcements on steep portions of the landfill, which were designed to satisfy static and seismic stability design criteria. The infiltration design criteria for the cover system at the OII Superfund site required that the percolation through the proposed alternative, ETC be less than the percolation through the prescriptive, resistive cover. The prescriptive cover consisted of a 1200-mm-thick system, which included a 300-mm-thick vegetative layer, a 300-mm-thick clay layer having a saturated hydraulic conductivity of 1×10^{-8} m/s, and a 600-mm thick foundation layer. The vegetative layer and the foundation layer were both assumed to have a saturated hydraulic conductivity of 1 x 10^{-6} m/s. Equivalence demonstration procedures using site-specific weather conditions and soil-specific hydraulic properties were developed to evaluate the compliance of the proposed alternative cover with the prescriptive cover system.

Construction of an ETC at least 1200-mm thick resting on top of a 600-mm-thick foundation layer was completed in April 2000. Performance monitoring of the cover, consisting of a series of time domain reflectometry probes, was implemented during the three years following construction to monitor moisture variations and percolation trends within the cover.

3.2.2 Exposed Geomembrane Covers

3.2.2.1 Introduction

Exposed geomembrane covers (EGCs) have been recently analyzed, designed, and constructed to provide temporary and final closure to waste containment facilities. Significant cost savings may result from elimination of topsoil, cover soil, drainage, and vegetation components in typical cover systems. Additional advantages include reduced annual operation and maintenance requirements, increased landfill volume, easier access to landfilled materials for future reclamation, and reduced post-construction settlements. In addition, if the landfill slopes are steep, the use of EGCs may provide solutions to erosion concerns and to stability problems associated with comparatively low interface shear strength of typical cover components. Disadvantages associated with the use of EGCs include increased vulnerability to environmental damage, increased volume and velocity of stormwater runoff, limited regulatory approval, and aesthetics concerns. However, EGCs have been particularly applicable to sites where the design life of the cover is relatively short, when future removal of the cover system may be required, when the landfill side slopes are steep, when cover soil materials are prohibitively expensive, or when the landfill is expected to be expanded vertically in the future. Key aspects in the design of EGCs are assessment of the geomembrane stresses induced by wind uplift and of the anchorage against wind action.

3.2.2.2 Geomembrane Stresses Induced by Wind Uplift

The resistance to wind uplift of an EGC is a governing factor in its design. Wind uplift of the geomembrane is a function of the mechanical properties of the geomembrane, the landfill slope geometry, and the design wind velocity. Procedures for the analysis of geomembrane wind uplift have been developed by Giroud et al. (1995) and Zornberg and Giroud (1997). A number of EGCs have been designed and constructed using these procedures (Gleason et al., 2001; Bouazza et al., 2002), particularly in the US.

Wind uplift design considerations involve assessment of the maximum wind velocity that an exposed geomembrane can withstand without being uplifted, of the required thickness of a protective layer that would prevent the geomembrane from being uplifted, of the tension and strain induced in the geomembrane by wind loads, and of the geometry of the uplifted geomembrane. The fundamental relationship of the geomembrane uplift problem is the "uplift tension-strain relationship," defined by (Zornberg and Giroud, 1997):

$$\varepsilon_w = \frac{2T}{S_e L} \sin^{-1}\left[\frac{S_e L}{2T}\right] - 1 \qquad (1)$$

where ε_w = geomembrane strain component induced by wind uplift, T = total geomembrane tension, S_e = effective wind-induced suction, and L = length of geomembrane subjected to suction. Figure 4 shows a schematic representation of an uplifted geomembrane. It should be noted that the uplift tension-strain relationship (Equation 1) relates the strain induced only by the wind (ε_w) with the total tension in the geomembrane (T) induced also by other sources like temperature or gravity. In other words, Eq. 1 is not a relationship between the wind-induced strain (ε_w) and the wind-induced tension (T_w).

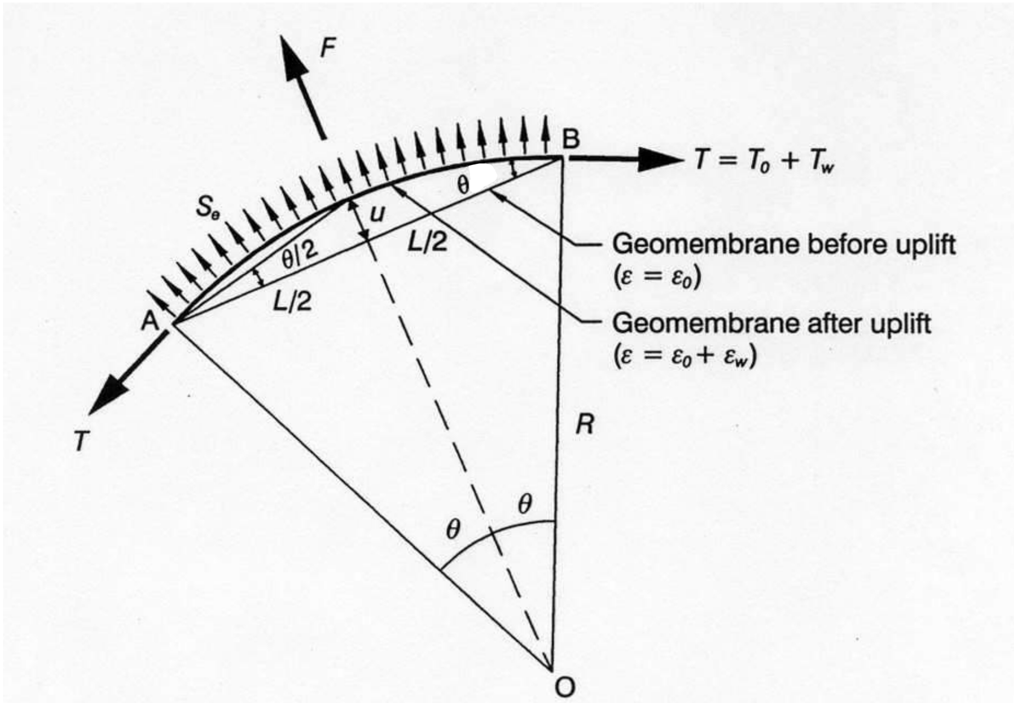


Figure 4. Schematic representation of an uplifted geomembrane (Zornberg and Giroud, 1997).

3.2.2.3 Anchorage Against Wind Action

A method for designing anchor benches and trenches used to secure geomembranes exposed to wind action is presented by Giroud et al. (1999). Three potential failure mechanisms are identified: (i) sliding of the anchor bench or trench in the downslope direction; (ii) sliding of the anchor bench or trench in the upslope direction; and (iii) uplifting of the anchor bench or trench. It is shown that the first mechanism is the most likely and that the third mechanism is the least likely. Criteria are provided by Giroud et al. (1999) to determine what is the potential failure mechanism in each specific situation. This is defined by the geometry of the slope on which the geomembrane is resting and the geomembrane tensions induced by wind action. It is also shown that a simple method, consisting of only checking the resistance of anchor benches and trenches against uplifting is unconservative as lateral sliding is more likely to occur than uplifting.

3.2.2.4 Case Histories

A number of EGCs have been recently designed using the aforementioned procedures for wind uplift analysis. Four of the recently constructed EGCs in the US are listed below (Gleason et al. 2001; Bouazza et al. 2002). At each landfill, the design and operations criteria for the EGC, as well as the rationale for constructing the EGC were significantly different. The sites are:

- Delaware Solid Waste Authority, Sussex County, Delaware: an EGC was designed and installed over a 17-ha landfill to provide a long-term cover system (i.e. 10 to 20 years) over waste that may be reclaimed at a later date.
- Crossroads Landfill, Norridgewock, Maine: an EGC was designed and installed over a 2-ha landfill that had reached its allowable interim grades based on site subsurface stability. With

time, the subsurface strata of clay beneath the landfill will consolidate and gain shear strength, thus allowing for additional waste placement.

- Naples Landfill, Naples, Florida: an EGC was designed to provide a temporary cover for a 9-ha landfill for two purposes: (i) the EGC was constructed a year prior to the planned construction of a typical final cover system in order to control odors associated with landfill gas; and (ii) on two of these slopes, the EGC was installed over areas that will be overfilled in the near future.
- Sabine Parish Landfill, Many, Louisiana: an EGC was designed and installed over a 6-ha landfill that had severe erosion because of long steep side slopes that could not be reasonably closed using conventional closure system technology.

In addition, a feasibility evaluation of the use of an EGC was conducted for the OII Superfund landfill (see section on Alternative Earthen Final Covers). The main reason for having considered an exposed geomembrane cover at this site was the difficulty in demonstrating adequate slope stability, under static and seismic conditions, in the case of conventional covers where geosynthetics are overlain by soil layers. Although a MC system was finally adopted at the site, an EGC was also considered because it would have been stable under both static and seismic conditions.

3.2.3 Geochemical Covers

3.2.3.1 Introduction

Blowes et al. (1991) evaluated the formation of cemented (hardpan) layers near the surface of two inactive sulfidic tailings impoundments in Canada to evaluate the potential effect of hardpan layers on reducing the rate and magnitude of sulfide oxidation and H^+, Fe^{2+}, and SO_4^{2-} production. Blowes et al. (1991) found that the hardpan layer at one of the sites consistently occurred at a depth where there is an abrupt increase in the solid-phase carbonate content.

3.2.3.2 Application in Acid Drainage Problems

For example, consider the problem of acid drainage resulting from the oxidation of sulfidic tailings, such as pyrite ($FeS_{2(s)}$), in accordance with the following chemical reaction (e.g., Nicholson et al., 1989; Evangelou and Zhang, 1995; Ribet et al., 1995):

$$FeS_{2(s)} + 3.5O_2 + H_2O \rightarrow 2SO_4^{2-} + 2H^+ + Fe^{2+} \tag{2}$$

The result of this chemical reaction is the production of a low pH solution (e.g., pH < 6) containing relatively high concentrations of potentially toxic heavy metals associated with the tailings, such as Fe^{2+}. In the presence of sulfate (SO_4^{2-}) resulting from dissolution of pyritic tailings (Eq. 1), calcite ($CaCO_{3(s)}$) dissolves resulting in gypsum ($CaSO_4{\cdot}2H_2O_{(s)}$) precipitation as follows:

$$SO_4^{2-} + CaCO_{3(s)} + 2H^+ + H_2O \rightarrow CaSO_4 \cdot 2H_2O_{(s)} + CO_2 \tag{3}$$

The ferrous iron (Fe^{2+}) resulting from dissolution of pyritic tailings (Eq. 2) oxidizes to ferric iron (Fe^{3+}) in the presence of *Thiobacillus ferrooxidans* as follows (Blowes et al., 1991):

$$Fe^{2+} + 0.25O_2 + H^+ \rightarrow Fe^{3+} + 0.5H_2O \tag{4}$$

The abundance of ferric iron from the reaction in Eq. 4 results in a hardpan layer of precipitated iron oxyhydroxide ($FeO(OH)_{(s)}$) and/or ferric hydroxides ($Fe(OH)_{3(s)}$) due to an increase in pH (Eqs. 3 and 4) as follows (Evangelou and Zhang, 1995; Chermak and Runnells, 1996):

$$\left.\begin{array}{l} Fe^{3+} + 2H_2O \rightarrow FeO(OH)_{(s)} + 3H^+ \\ Fe^{3+} + 3H_2O \rightarrow Fe(OH)_{3(s)} + 3H^+ \end{array}\right\} \qquad (5)$$

Thus, the existence of $CaCO_{3(s)}$ (Eq. 3) results in an increase in pH and the consequent precipitation of $FeO(OH)_{(s)}$ and/or $Fe(OH)_{3(s)}$.

Based on their investigations, Blowes et al. (1991) concluded that the persistence of hardpan layers near the surface of sulfidic tailings can reduce substantially the rate of sulfide oxidation and the generation of acid drainage. They also noted that the hardpan layer formation may be enhanced by adding carbonate minerals to the tailings near the end of disposal operations.

3.2.3.3 Creation of Geochemical Covers

Chermak and Runnells (1996) evaluated the potential for creating a low-permeability geochemical cover by evaluating the effect of the addition of lime ($CaO_{(s)}$) and crushed limestone ($CaCO_{3(s)}$) on the hydraulic conductivity of an acidic sulfide-rich overburden material from a gold mine. The measured hydraulic conductivity values for two columns of overburden material without surface amendment permeated with a simulated rainwater (1.5 x 10^{-6} m HCl, pH = 5.80) were 1.4 x 10^{-5} m/s and 1.3 x 10^{-6} m/s. The measured hydraulic conductivity values for various surface amended columns ranged from 1.6 x 10^{-7} m/s to 5.7 x 10^{-7} m/s as the result of the formation of hardpan layers with hydraulic conductivity values ranging from 2.8 x 10^{-9} m/s and 7.8 x 10^{-9} m/s.

Chermak and Runnells (1996) also found that an important consideration in the development of the hardpan layer is that the sulfide overburden and limestone and/or lime be in direct contact to allow the chemical reactions to occur. Thus, the addition of solid-phase carbonate minerals to acidic sulfide-bearing tailings shows promise in terms of creating a low-permeability geochemical cover. Also, the hardpan layer formation should be self-perpetuating and self-hardening provided sufficient amounts of carbonate minerals are available.

Organic-rich materials, such as sewage sludge, composted municipal waste, peat, or sawdust, also have been proposed as geochemical cover materials. These material consume oxygen and prevent or reduce the oxidation of the underlying sulfidic tailings (Ribet et al., 1995). The major advantages of this approach are low cost and availability of materials. A potentially significant disadvantage is that aerobic and anaerobic biodegradation of the organic-rich waste can release soluble organic compounds to percolating water that subsequently result in reductive dissolution of ferric-bearing precipitates resulting in release of metals from the tailings.

3.2.4 Covers from Pulp and Paper Mill Sludge

3.2.4.1 Introduction

The pulp and paper industry generates large quantities of sludge from wastewater treatment. The United States Department of Commerce estimated that in 1974, 560 to 630 pounds of solid waste was generated per ton of paper production (Springer, 1986). More recent estimates indicate that wastewater treatment is now more efficient in removal of solids, yielding about 1780 kN of paper sludge (also referred in the literature as fibre clay®, paper clay, paper residuals, paper biosolids, and short paper fiber (spf®) per ton of paper produced (Gregg et. al., 1997). This production equates to a total of over 10.8 billion kg of paper sludge generated each year by the United States paper industry alone (Springer, 1986; Gregg et. al., 1997).

Approximately 70-75 % of the paper sludge is disposed in landfills (Springer, 1986; Zander et. al., 1996). Another 15-20 % is currently being incinerated for both power production and volume reduction (Springer, 1986; Zander et. al., 1996). A much smaller portion of the paper sludge stream is being disposed as soil amendments and as low-permeability covers for landfills. With the shrinkage of available landfill space, and the difficulty in locating and permitting new solid waste landfills, an alternative is clearly needed to handle the paper sludge that is still being placed into landfills.

The high price of disposal has sparked interest in the development of alternative uses for paper sludge. Numerous studies have been conducted to find alternative uses for paper sludges (Stoeffel and Ham, 1979; NCASI, 1984; NCASI, 1985; NCASI, 1989; NCASI, 1990; NCASI, 1992a; NCASI, 1992b; Floess et al., 1995, Moo-Young and Zimmie, 1996a; 1997a; 1997b; Kraus et al., 1997; Floess et al., 1998). Compacted clays have been widely used as the barrier layer in landfill covers. However, when an abundant source of clay is not readily available, the cost of landfill closure is greatly increased. The elevated cost of waste disposal may be reduced by the use of unconventional material in the construction of landfills. Moreover, since paper sludges are considered a waste product, they are provided to the landfill owner at little or no cost. This may reduce the cost of construction by $50,000US to $125,000US per hectare.

3.2.4.2 Sludge Composition and Properties

The composition of paper sludges is a result of the type of paper produced and the wastewater treatment system at the mill. In general, the paper making process is categorized as integrated (I), non-integrated (N), or recycling (R). An integrated paper mill makes magazine quality paper by grinding fresh timber into ground wood, whereas a non-integrated plant purchases it as ground wood.

The water treatment plant at the paper mill typically provides primary and secondary treatment for the wastewater. In a typical wastewater treatment scheme, tissues and fibers are removed from the wastewater in the primary clarifier. Secondary treatment is employed to reduce the BOD (Biochemical Oxygen Demand). Disinfectants are used to kill bacteria after secondary treatment. Dewatering mechanisms such as belt presses, centrifuges, and vacuum filters are used to dewater the paper sludges. In general, paper sludges are characterized as primary, secondary or blended sludges. The differences in the paper making process combined with the variety of wastewater treatment options have led to the recommendation of thoroughly characterizing paper sludges prior to their utilization in environmental geotechnics applications (Moo-Young and Zimmie, 1997a).

3.2.4.3 Geotechnical Index Properties

Paper sludges consist of organic fibers and tissues, and inorganic clay fillers. Organic material generally consists of lignin and pulp fibers. Trace materials can also be found in paper sludges such as resins and starch for strengthening the paper product, and pigments and chalk for coloring and surface treating of the final paper. The inorganic component, also referred to as ash, can consist of kaolinite, calcium carbonate, titanium oxide, or other materials used in pulp and paper production (NCASI, 1989). Clay is usually the principal component of the ash.

Geotechnical index properties, such as gravimetric water content, organic content, specific gravity, and Atterberg limits, have been utilized to compare paper sludges to natural soils (i.e., clays). Paper sludges are characterized by typically higher gravimetric water contents (100 % to 250 %), organic content (30 % to 70 %), liquid limits (120 % to 300 %), and plastic limits (48 % to 130 %) relative to typical clay soils (Moo-Young and Zimmie, 1996a; 1997d). Moreover, paper sludges are characterized by a lower specific gravity (1.8-2.1) than typical clays. As a result of the high organic matter in paper sludges, the oven temperature used to dry samples must be lowered from 105 ^{o}C to 70 ^{o}C to avoid burning off the organic matter (Alvi and Lewis, 1987; Moo-Young, 1992; Moo-Young, 1995), a larger sample (200-300 grams) is utilized (Moo-Young and Zimmie, 1996a), and two to three days typically are required to completely dry the specimen.

3.2.4.4 Chemical Analysis

Chemical analysis using TCLP (Toxicity Leachate Characterization Procedure) on paper sludges has shown that they are non-toxic (Izu et al., 1998). The chemical characteristics of the liquid phase in paper sludge are related to the paper making and wastewater treatment processes. Trace metal leachability from paper mill ashes and paper mill sludge, and the effects of a soil on their leachability, were determined by leaching the residues packed on top of a soil in a column (Xiao and Sarigumba, 1999). The results showed that organic matter enhances metal sorption when present as a solid phase, but organic matter also increases metal leachability when present in dissolved form under alkaline conditions.

3.2.4.5 Compaction

Laboratory compaction tests have been conducted by numerous researchers to determine the relationship between dry unit weight and molding water content (Moo-Young and Zimmie, 1996a; Kraus et al., 1997; Izu et al., 1998). Because of the high water content, tests were conducted from the wet side rather than from the dry side of the compaction curve as stipulated by American Society of Testing and Materials (ASTM) procedure D 698 (ASTM D 698). When water was added to dry sludge, large clods formed, the clods were difficult to break apart, and the sludge lost its initial plasticity. At higher water contents, the dry density obtained from the compaction test for the various sludges is similar. At the optimum density (6.0 to 9.0 kN/m^3) and moisture content (40 % to 60 %), the sludge is dry, stiff, and unworkable. Quiroz and Zimmie (1999) suggested that paper sludge be compacted at a dry density ranging between 7 kN/m^3 and 4.5 kN/m^3, which generally corresponds to a water content ranging from 90 % to 160 %, respectively.

During the construction of the landfill cover system in Hubbardston, Massachusetts, USA, and test plots in Erving, Massachusetts, USA, different types of equipment were used to place the sludge cap. Four types of equipment were used: a small ground pressure vibratory drum roller, a vibrating plate compactor, a sheepsfoot roller, and a low ground-pressure track dozer. The vibratory methods did not provide homogeneous mixing and did not compact the sludge effectively. The small ground-pressure dozer provided the best method for placement and compaction from the toe of the landfill towards the top. This equipment successfully eliminated large voids from the sludge material and kneaded the material homogeneously (Moo-Young, 1998; Floess et al., 1998; Floess et al., 1995).

3.2.4.6 Hydraulic Conductivity

Paper sludges generally have several characteristics that make their use as hydraulic barriers promising. Hydraulic conductivities in the range of 10^{-9} m/s to 10^{-10} m/s have been reported (Moo-Young, 1995; Moo-Young and Zimmie, 1996a; Kraus et. al., 1997). The hydraulic conductivity of paper sludge is a function of the organic content, effective stress, and initial water content. As the organic content of paper sludge decreases, the hydraulic conductivity decreases. The following relationship can be utilized to estimate the hydraulic conductivity from the organic content (Moo-Young, 1996d):

$$k = 10^{(0.022OC - 8.03)} \tag{6}$$

where k = hydraulic conductivity (cm/s) and OC = organic content (%).

Paper mill sludge can be compacted to a low hydraulic conductivity ($<10^{-9}$ m/s) at water contents 50 % to 100 % wet of the optimum water content. At or near the optimum water content, the hydraulic conductivity of paper sludge increases (Moo-Young and Zimmie, 1996a; Kraus et al., 1997). Studies also have shown that an increase in effective stress on paper sludge corresponds to a decrease in the hydraulic conductivity (Moo-Young and Zimmie, 1997a; Moo-Young and Zimmie, 1996a; Kraus et al., 1997).

Some special precautions are required when measuring the hydraulic conductivity of paper mill sludges. For example, Kraus et al. (1997) placed triaxial cells in a refrigerator at 4°C to reduce the production of gases from organic decomposition near the porous stone. Also, Zimmie and Moo-Young (1995) suggest that the test specimen be purged every twenty-four hours to reduce the buildup of gases.

3.2.4.7 Compressibility

Paper mill sludges are highly compressible, and the water content and organic content are a useful indicator of the consolidation characteristics. Moo-Young and Zimmie (1996a; 1996d) present relationships between the compression index (C_c) and the initial water content (w_o, %) and organic content (OC, %) as follows:

$$C_c = 0.009(w_o) \tag{7}$$

and

$$C_c = 0.027(OC) \tag{8}$$

Landva and LaRochelle (1983) established a relationship between compression index and water content for peats, which is similar to the one obtained for paper mill sludges.

3.2.4.8 Shear Strength

Laboratory undrained triaxial compression tests conducted on paper sludge indicate that the effective angle of internal friction for paper sludge varies from 25° to 40° and the effective stress cohesion ranges from 2.8 to 9.0 kPa (Moo-Young and Zimmie 1996a, 1997b). Field vane shear tests conducted at the Montague Landfill in Montague, Massachusetts, resulted in undrained shear strength ranging from 2 to 35 kPa with a water content ranging from 200 % to 122 %, respectively (Quiroz and Zimmie, 1998).

3.2.4.9 Freeze and Thaw Susceptibility

Laboratory specimens frozen one-dimensionally to a desired number of freeze-thaw cycles, and subsequently permeated in a flexible-wall permeameters at effective stresses of 34, 69, and 138 kPa using a hydraulic gradient of 21 have shown that freezing and thawing increased the hydraulic conductivity of paper sludge by one order of magnitude (Moo-Young and Zimmie 1996b). The change in hydraulic conductivity at lower effective stresses due to freeze-thaw appears to be similar to that for higher effective stresses. Similar results were obtained by Kraus et al. (1997).

3.2.4.10 Case Histories

Since 1990, more than 14 landfills in the US (52.6 hectares total) and 10 landfills in Finland have been covered using paper sludge as the barrier construction material (Maltby 1999, Saarela 1999). The landfill closure projects in the US are summarized in Table 2.

Table 2. Summary of full-scale closures using paper mill sludges in the United States.

State	Landfill Type	Area (ha)	Barrier Thickness (m)	Hydraulic Conductivity (m/s)
Alabama	Industrial	0.76	0.61	$<10^{-10}$
Maine	Municipal	0.076	0.76	10^{-10} - 10^{-11}
Maine	Municipal	0.081	0.76	10^{-11}
Maine	Municipal	0.038	0.76	10^{-9}
Massachusetts	Municipal	0.20	0.76	10^{-12} - 10^{-13}
Massachusetts	Municipal	0.19	0.76	10^{-12} - 10^{-13}
Massachusetts	Municipal	0.41	0.76	10^{-12}
Massachusetts	Municipal	0.077	0.76	10^{-12} - 10^{-13}
Massachusetts	Municipal	0.077	0.76	10^{-12} - 10^{-13}
Massachusetts	Municipal	0.41	0.76	10^{-12} - 10^{-13}
Michigan	Industrial	0.23	0.81	10^{-11}
New Hampshire	Municipal	0.077	0.76	10^{-12} - 10^{-13}
New York	Municipal	0.33	1.2	10^{-11}
New York	Municipal	0.30	0.61	10^{-11}

3.3 INNOVATIVE LINERS AND LINER MATERIALS

3.3.1 Reactive Liners and Barriers

3.3.1.1 Introduction

Attenuation refers to the reduction in the rate and/or magnitude of contaminant migration due to physical, chemical, and/or biological reactions (Shackelford and Nelson, 1996). Geochemical attenuation refers to attenuation that results from geochemical interactions between natural geological material and chemical constituents in the pore water. Some possible geochemical attenuation mechanisms include cation and anion exchange with clays, adsorption of cations and anions on hydrous metal oxides, such as iron and manganese, sorption on organic matter or organic carbon, precipitation of metals from solution, and co-precipitation by adsorption (Shackelford, 1999; Shackelford and Jefferis, 2000).

Although most low-permeability clay soil barriers have some intrinsic attenuation capacity, the concept of designing liners or barriers with an enhanced attenuation capacity, referred to as reactive liners or barriers, recently has gained momentum (Shackelford, 1999). For example, the use of additive barrier materials, such as zeolites, high carbon fly ash, organically modified clays, and tire chips, has been proposed to enhance the attenuation capacity of waste containment liners (Shackelford, 1999).

3.3.1.2 Design Considerations

The design of reactive barriers requires knowledge of not only the physical properties of the barrier materials (e.g., hydraulic conductivity) but also the chemical properties of the barrier materials that will affect the migration rate of the contaminants in the pore water. Thus, an understanding of the potential attenuation mechanisms for the principal chemical species of interest is required. Thornton et al. (1993) have identified the principal attenuation mechanisms for many of the inorganic chemical solutions of concern as ion exchange (sorption), precipitation, dilution, and neutralization. However, the two primary attenuation mechanisms with respect to heavy metal migration are ion exchange, or adsorption, and precipitation.

Ion exchange or adsorption can be enhanced in a barrier by using additive materials that will increase the overall cation exchange capacity (CEC) or adsorption capacity of the liner. For example, consideration of the use of zeolites with CEC as high as 250 meq/100 g as a barrier material recently has been described by Evans et al. (1990) and Allerton et al. (1996). Zeolites are hydrated alumino-silicates with a cage structure that acts as a sieve in that metal ions in solutions that are passed through the structure are trapped by ion exchange reactions. Due to the cage structure, the CEC of zeolites is very high at about 250 meq/100 g. As a result, zeolites can be used to enhance the adsorption capacity of other clays (e.g., bentonite) with respect to heavy metal cations.

The precipitation potential of a passive barrier can be enhanced by adding materials that will increase the pH of the permeant liquid. For example, the addition of quicklime [CaO], hydrated high calcium lime [$Ca(OH)_2$], dolomitic quicklime [CaO· MgO], or monohydrated dolomitic lime [$Ca(OH)_2 \cdot MgO$] may aid in increasing the pH of an acid solution during migration through the liner to (a) neutralize the pore solution pH, (b) precipitate metals from solution, (c) clog the pores, and (d) decrease the hydraulic conductivity thereby enhancing the overall performance of the liner. Processed clay soils, such as bentonite and attapulgite clay, that have been pre-treated to maintain a relatively high pH (> 9) for stability considerations (e.g., for use as drilling muds in high TDS environments), also may be used to enhance the precipitation capacity of the liner materials. In most cases, laboratory column and/or batch equilibrium adsorption tests (BEATs) will be required with barrier specific materials and site-specific chemical solutions to determine the viability and optimum amounts of reactive materials being considered for use in reactive liners.

Adsorption of neutral, non-polar organic compounds (e. g., benzene) is correlated directly with the amount of organic carbon, OC, in the soil through the well-known "hydrophobic effect". Thus, increasing the amount of organic carbon in the barrier, for example, by adding high carbon fly ash, rubber tire chips, or granular activated carbon, may increase the attenuation capacity of the barrier with respect to this important class of organic compounds. Also, organically modified clays may be used in the barrier to increase the adsorption capacity of the barrier for organic compounds. Organically modified clays, or organo-clays, are naturally occurring clays in which a portion of the inorganic exchangeable cations are exchanged with a suitable organic cation, such as the quaternary ammonium organic cations benzyltriethylammonium (BTEA) bromide and dodecyl-trimethylammonium (DDTMA) bromide (e.g., Evans et al., 1990; Gray, 1995; Smith et al., 1995). This substitution enhances the ability of the clay to adsorb organic chemicals (e.g., benzene, dichlorobenzene, and perchloroethyleane) that migrate through the clay and also may enhance the ability to achieve a low hydraulic conductivity (Smith et al., 1995).

The high organic content and low hydraulic conductivities associated with paper mill sludges also make it an ideal candidate for a low permeability reactive barrier. The organics within the sludges potentially act as a carbon source for heavy metal attenuation (Moo-Young and Gallagher 1998, Moo-Young et al. 2000). Work done by Moo-Young and Gallagher (1997) showed that the sorption capacity of the paper sludge was greater than that of kaolin clay.

3.3.1.3 Benefits of Reactive Barriers

The potential benefits of reactive barriers with respect to attenuation of contaminants was previously shown by Shackelford (1999) by considering diffusive transport with the potential for sorption through the simplified barrier scenario shown in Figure 5. This simplified scenario obviates the need to consider the hydraulic conductivity of the barrier, and represents the limiting case of pure diffusion expected to govern miscible transport of contaminants through low-permeability barriers (e.g., Shackelford, 1988).

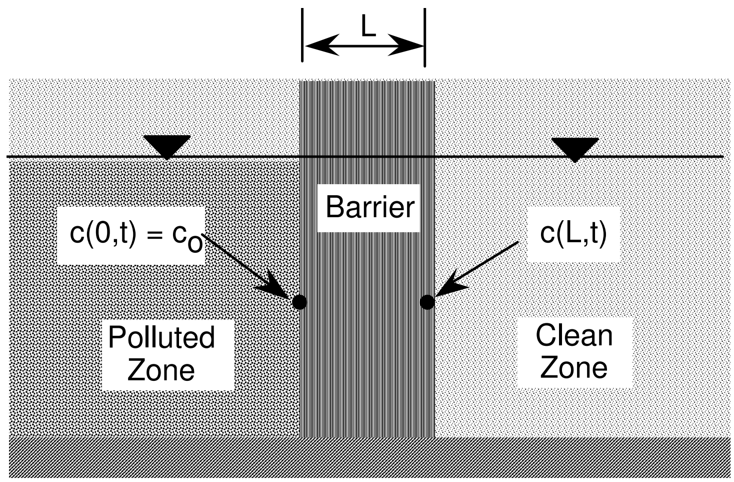


Figure 5. Schematic cross-section of passive containment barrier (Shackelford, 1999)

The analysis of the scenario depicted in Figure 5 by Shackelford (1999) was presented in terms the thickness of the barrier required for a given retardation factor, R_d, representing the attenuation capacity of the barrier, for a given containment time, or the required containment time as a function of R_d for a given thickness, as illustrated in Figure 6. The results shown in Fig. 6 are based on limiting the concentration at the outer extent of the barrier, c(L,t), resulting from diffusion with reaction to $\leq$ 10 percent of a constant source concentration, c_o, assuming a representative value for the effective diffusion coefficient of the contaminant, D^*, as 5×10^{-10} m^2/s (see Shackelford, 1999).

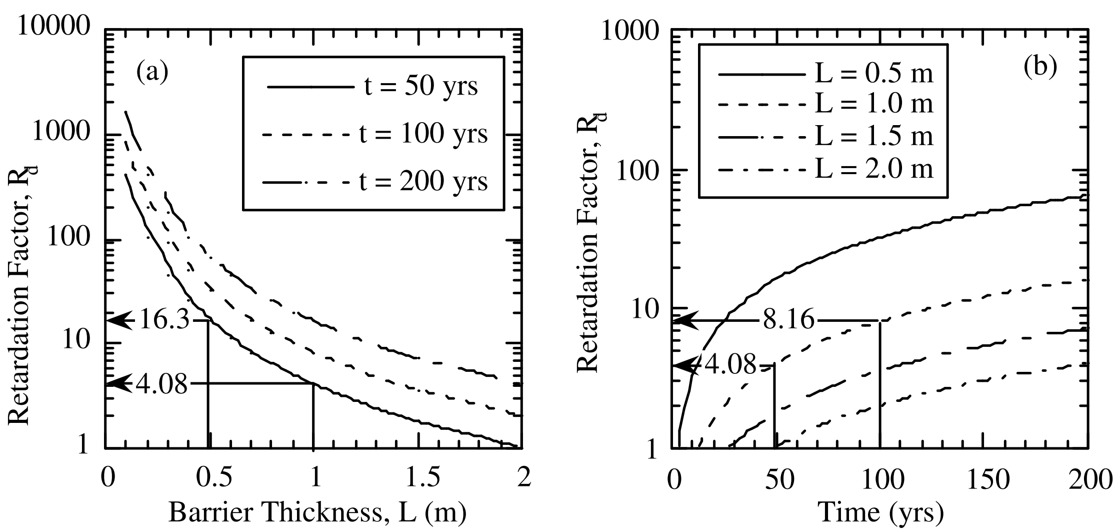


Figure 6. Retardation factor (a) as a function of thickness for a given containment time, and (b) as a function of containment time for a given thickness for the case of pure diffusion and a concentration at x = L equal to 10 % of that relative to that at x = 0 (i.e., $C_L/C_o = 0.1$) (Shackelford, 1999).

As illustrated in Figure 6, the barrier thickness can be halved by increasing the retardation factor (i.e., attenuation capacity) by 4X for a given containment period or, conversely, the containment period can be doubled for a barrier with a given thickness by doubling the retardation factor of the barrier material. Thus, the potential benefits of utilizing the reactive nature of passive containment barriers, particularly for long-term containment, are apparent.

3.3.2 Geochemical Barriers

Morrison et al. (1995) described the conceptual design of a chemical barrier for retarding the migration of U^{6+} from a uranium mill tailings impoundment. The design involves mixing a solution of water and hydrous ferric chloride ($FeCl_3{\cdot}6H_2O_{(s)}$) and simply spraying the solution containing dissolved $FeCl_3$ on liner solids containing carbonate minerals (e.g., $CaCO_{3(s)}$) to cause precipitation of amorphous ferric oxyhydroxide ($FeO(OH)_{2(s)}$) that subsequently provides exchange sites for adsorption of U^{6+} in leachate from the tailings. Modeling simulations performed by Morrison et al. (1995) indicated that the chemical barrier would be effective in limiting U^{6+} concentrations to ≤ 0.05 mg/L to within 27 m (88 ft) of the repository boundary for a period of at least 216 yrs, and that the effective performance time could be increased by more efficient distribution of the $FeCl_3$ solution.

3.3.3 Biobarriers

3.3.3.1 Introduction

The concept of using bacteria to form biobarriers in otherwise highly permeable media (e.g., sands) to contain or reduce the migration of contaminant plumes recently has gained attention (e.g., Mitchell, 1997). Biobarriers are created by reducing the hydraulic conductivity of the porous medium through particulate or pore clogging using bacterial cells, by the formation of biofilms that develop over time thereby reducing the void space, and/or by the filling of voids through the use of viscous biopolymers (Bouazza et al, 2001). Reductions in the hydraulic conductivity from one to three orders of magnitude have been reported for a variety of porous media using many types of bacteria and different treatment methods including stimulation of indigenous bacteria (biostimulation), and injection of full-sized living and dead bacteria as well as ultramicrobacteria (bioaugmentation) (Dennis and Turner, 1998).

3.3.3.2 Bacteria Induced Reductions in Hydraulic Conductivity

As shown in Figure 7, Shackelford (1999) and Shackelford and Jefferis (2000) summarized data reported by Dennis and Turner (1998) on the results of several studies that showed the relative decrease in hydraulic conductivity of different porous media resulting from treatment with bacteria. All of the results shown in Figure 7 represent tests performed on materials with high initial values of k, ranging from 2.5 x 10^{-3} to 2 x 10^{-7} m/s, and the results indicate that final values of hydraulic conductivity < 10^{-9} m/s generally were not achieved.

As shown in Table 3 and Fig. 8, Bouazza et al. (2001) also summarize the results of several studies showing a decrease in the hydraulic conductivity of several different porous media using several different types of treatment. The results in Table 3 show that low hydraulic conductivities (<10^{-9} m/s) can be achieved with clayey silts and sands. Dennis and Turner (1998) also report results showing that the hydraulic conductivity of a compacted silty sand (SM) decreased from between ~ 10^{-7}and10^{-8} m/s to ~ 10^{-10} m/s after treatment with the biofilm-producing bacterium *Beijerinckia indicica*. In addition, they show that the hydraulic conductivity of ~ 10^{-10} m/s for soil specimens with well-established biofilm barriers was essentially unaffected after permeation with a 0.5 N saline solution, an acidic solution (pH 3), or a basic solution (pH 11). Thus, the results of the studies by Dennis and Turner (1998) and Bouazza et al. (2001) show promise for the use of biobarriers for containment applications when the soil contains a sufficient amount of fines (silt or clay).

The results in Figure 8 indicate the relative effectiveness of three different types of biopolymers - *Xanthan Gum*, *Sodium Alginate*, and *Guar Gum*. As shown in Figure 8, *Xanthan Gum* was more effective than *Sodium Alginate*, and *Guar Gum* was found to be the least efficient biopolymer for the study by Bouazza et al. (2001) in that 2 % biopolymer was required to reach the target hydraulic conductivity of 10^{-9} m/s.

The reduction of hydraulic conductivity has been shown to be dependant on the initial porosity of the material, the mechanism of clogging, the concentration and variety of biosubstance applied, the compaction (molded) moisture content, and the curing time (Bouazza et al., 2001). In general, the a greater reduction in hydraulic conductivity has been shown for a lower initial hydraulic conductivity (lower initial porosity) of the medium, an increase in compaction water content to several percentage points wet of optimum, and a longer curing time.

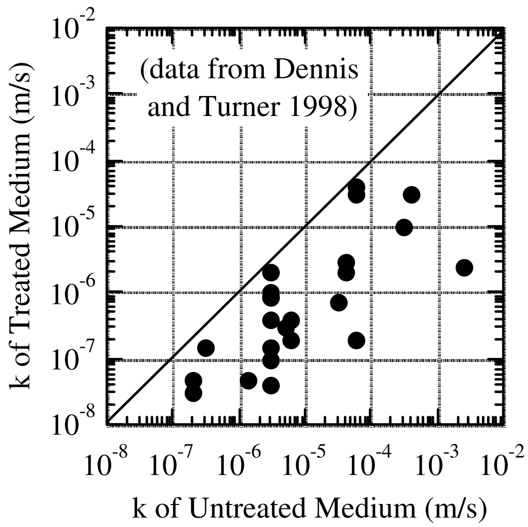


Figure 7. Hydraulic conductivity (k) of porous media before and after treatment with bacteria (Shackelford, 1999; Shackelford and Jefferis, 2000).

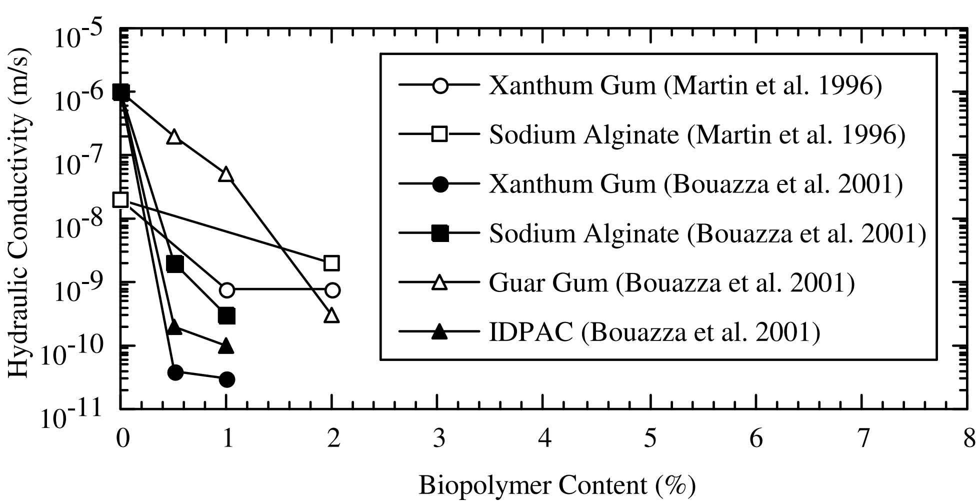


Figure 8. Hydraulic conductivity of soil biopolymer mixtures (Bouazza et al., 2001)

Table 3. Summary of selected results indicating reduction in hydraulic conductivity of porous media (modified from Bouazza et al., 2001).

Matrix	Hydraulic Conductivity (m/s) Initial	Final	Material added
Berea sandstone	1.3×10^{-6}	5×10^{-8}	*Bacillus subtilis* (dead)
Sandstone cores	3×10^{-6}	9×10^{-7}	*Klebsiella pneumoniae UMB*
Sandstone cores	3×10^{-6}	4×10^{-8}	*Klebsiella pneumoniae UMB* (resuscitated)
Clayey silt	5×10^{-8}	8×10^{-10}	*Xanthan Gum* (1%) at 30% moisture content
Clayey silt	10^{-6}	1×10^{-8}	*Xanthan Gum* (1%) at 13.7% moisture content
Clayey silt	10^{-8}	5×10^{-10}	*Xanthan Gum* (1%) at 20% moisture content
Clayey sand	10^{-6}	3×10^{-10}	*Guar Gum* (2%)
Clayey sand	10^{-6}	2×10^{-10}	Sodium alginate (1%)
Clayey sand	10^{-6}	1×10^{-10}	*IDPAC* (1%)
Clayey sand	10^{-6}	3×10^{-11}	*Xanthan Gum* (1%)
Silty sand	10^{-7} to 10^{-8}	1×10^{-10}	*Beijerinckia indica*

3.3.3.3 Increase in Material Strength

Due to the viscosity of the biopolymer and biofilm and the adhesion to soil surfaces, it is reasonable to expect that there may be an increase in shear strength of soil due to the addition of biopolymers or biofilms. Bouazza et al. (2001) summarize results reported by Martin et al. (1996) that show a general increase in undrained strength of specimens treated with biopolymers or the growth of biofilm.

3.3.4 Asphaltic Barriers

3.3.4.1 Introduction

Asphalt barriers have been used in containment applications for more than 70 years. Prior to the mid-1960s, asphalt barriers were primarily used to control water seepage from facilities such as impoundments and earth dams (Sherard et al., 1963; Monismith and Creegan, 1996). Asphalt was applied as hot-sprayed asphalt membranes and as asphalt concrete for the barrier layer. In the late 1960s and early 1970s, asphalt was on the way to becoming the state-of-the-practice landfill liner (Asphalt Institute, 1976). Several US facilities were constructed using asphalt concrete (hot-mix asphalt) liners that, in some cases, were combined with a sprayed-on fluid applied asphalt layer. The petrol shortage of the 1970s along with the establishment of rules for hazardous and solid waste landfill designs that focused the industry toward composite liners consisting of geomembranes and compacted soil contributed to the decline of the use of asphalt for containment. Resurgence into the use of asphalt for waste isolation was initiated by the US Department of Energy (DOE) in their quest for very long-term hydraulic barriers (1000+ years) for radioactive and mixed waste sites (Wing and Gee, 1994a).

3.3.4.2 Hydraulic Conductivity

Historical analog information collected by the US DOE indicated that asphalt can have lifetimes of greater than 1000 years. DOE developed performance criteria for a cover system that included an asphalt barrier for mixed waste sites (Wing and Gee, 1994b). A test cover including a fluid applied asphalt (FAA) layer above the asphalt concrete was constructed and evaluated. The initial results indicated hydraulic conductivity of cores from the asphalt concrete layer ranging from 1.3 x 10^{-11} to 1.2 x 10^{-12} m/s and field-measured conductivities ranging from 1.1 x 10^{-9} to 1.9 x 10^{-11} m/s. The higher values in the field are likely attributed to measuring techniques and may not be representative of the asphalt conductivity. The conductivity of the FAA was measured and reported to be 1.8 x 10^{-13} m/s. The asphalt barrier looked very promising.

Results of laboratory and field efforts with asphalt concrete and fluid applied asphalt have illustrated that low hydraulic conductivities can be achieved with these barriers given proper design and high level construction quality control (Bowders et al. 2000, 2003). Several lessons learned from the existing data include: the percentage of air voids must be below 4 % (v/v) to achieve low hydraulic conductivity, asphalt cement content must be above 6 % (w/w) to achieve low hydraulic conductivity, fines content (fraction < 0.02 mm) must be increased to 8 % - 15 % to ensure a dense graded mixture, at least two layers of asphalt concrete should be used with a minimum thickness of 50 mm/layer to minimize continuity of potential defects and lateral spreading of any seepage, an asphalt cement tack coat should be applied between layers, the joints should be staggered and sloped for good compaction, the fluid asphalt applied layer should be between 1- and 3-mm thick, and the subgrade must be stable and adequately drained.

3.3.4.3 Case Histories

There are numerous citations for the use of asphalt in hydraulic containment structures (Monismith and Creegan, 1996). Most of the citations, beginning around the 1940s, refer to the use of hot-sprayed buried asphalt membranes (HSBAM) for controlling seepage of water. In many instances the HSBAM were used for potable water supplies. Asphalt concrete has also been used for hydraulic barriers and in at least one case was used for the low hydraulic conductivity liner for a municipal waste landfill (Asphalt Institute, 1976). The literature cases with specific application to waste isolation were summarized by Bowders et al. (2001), and are summarized in Table 4.

3.3.4.4 Recent Experience

In the US, several containment barriers incorporating asphalt have been constructed primarily for cover systems for existing landfills and contaminated sites (Bowders et al., 2000; 2003). The designs follow closely with the lessons learned as cited above. Bowders et al. (2000; 2003) used a systematic methodology to develop an asphalt mixture design, and quality control-quality assurance measures followed by *in situ* testing to document the field performance of an asphalt barrier. The program is described below.

An alternative barrier system was designed that incorporates 100 to 150 mm of asphalt concrete overlain by a 2- to 3-mm-thick fluid applied asphalt/geotextile (FAA/GT). Hydraulic conductivity tests were performed separately on laboratory prepared FAA/GT and asphalt concrete specimens. The FAA/GT specimens had measured hydraulic conductivities of less than 1 x 10^{-13} m/s. Conductivity tests on asphalt concrete specimens indicated that specimens having 7 % or more asphalt cement and unit weights of 22 kN/m^3 or more have conductivity of less than 1x10^{-11} m/s (Bowders et al., 2002; 2003; Neupane et al., 2005)).

A full-scale test pad (60 m x 18 m) was constructed and tested for barrier performance. The specification for the asphalt concrete was 7 to 7.5 percent asphalt cement and an *in situ* density ≥ 22 kN/m^3. The top surface of the asphalt concrete was sprayed with hot fluid applied asphalt and a paving geotextile was applied followed by a surface coating of hot fluid applied asphalt cement. FAA/GT and asphalt concrete samples were retrieved from the test pad to measure the hydraulic conductivity. Measurements on field installed FAA/GT and asphalt concrete specimens revealed conductivities comparable to that of the lab prepared specimens. Hydraulic conductivity of the asphalt concrete cores was 10^{-12} to 10^{-13} m/s. The conductivity of the FAA/GT specimens was 10^{-13} to 10^{-14} m/s. These values represent the lower limit for accurately measuring conductivity using standard procedures (ASTM D 5084).

Table 4. Asphalt concrete hydraulic barriers for waste isolation (Bowders et al., 2001).

Project/Location	Application	Hydraulic conductivity (k)	Status
Winnebago County Landfill, Rockford Illinois, 1972	-MSW liner -50 mm asphalt concrete -Tar emulsion surface -150 mm sand layer LCS	None reported	-1972 receiving 500 metric tons of waste/day -Current status unknown.
Liner Exposed to Simulated Landfill Leachate, 1976	-9 % asphalt cement -60 mm asphalt concrete	-Water: 3 x 10^{-11} m/s -Use >100 mm thick for leachates	-Liner in good condition after 4.6 years of exposure
Flue Gas Sludge Leachate/Liner Compatibility, 1977	-11 % wt basis asphalt cement -50 mm asphalt concrete liner	None reported	-Met k requirements
Superfund site, Montana, 1985	-Cover for former surface impoundment	-Lab: 7.5 % asphalt – 2 x 10^{-8} m/s -Lab: 8 % asphalt cement <10^{-12} m/s -Field cores: 1 x 10^{-8} to 9 x 10^{-10} m/s	-Poor construction quality control
Western Processing Company, Kent Washington, 1987	-Cover for waste site -6 % asphalt cement -Hi-way paving mix	-3 x 10^{-3} to 1 x 10^{-4} m/s cores from cover	-12 to 17 % air voids -Insufficient compaction
Landfill Cover, Oregon, 1990	-Cover and roadway	-Field test: <1 x 10^{-9} m/s (SDRI)	-Sealed double ring infiltr. field test
Hanford Permanent Isolation Barrier Program, Hanford, Washington, 1994	-Prototype cap -7.5 % asphalt cement -Two 150-mm layers of asphalt concrete -FAA on surface	Asphalt concrete: -Field cores: 1.3 x 10^{-11} to 1.2 x 10^{-12} m/s - Field SRIs: 1.1 x 10^{-9} to 1.9 x 10^{-11} m/s FAA: 1.8 x 10^{-13} m/s	-Variation in single ring k values likely due to measurement technique (SRI)
Rocky Flats, Denver, Colorado, 1997	-Fluid applied asphalt above the asphalt concrete	-FAA: 1.0 x 10^{-13} m/s or lower	-Lower limit of k test device. -No effect on k of gravel embedment.
Industrial waste, pulp & paper ash landfill, British Columbia, Canada 1998	-Cover for landfill -Asphalt cement included petroleum contaminated soils -150 mm of AC	None reported	-1999, cover performing well.
Port of Tacoma, Washington, 1999	-Cover for a slag dump	<1 x 10^{-9} m/s	-Cover serves as a parking lot

In situ hydraulic conductivity measurements were also performed. Specially designed and constructed sealed double-ring infiltrometers were used to measure the infiltration in the field. Hydraulic conductivity was calculated from the infiltration rates. The average *in situ* hydraulic conductivity measured was 1 x 10^{-12} m/s. The *in situ* value represents the lower limit for accurately measuring field infiltrations. The hydraulic conductivities measured on the cores from the test pad are thought to be more representative of the overall hydraulic conductivity of the asphalt liner.

3.3.4.5 Summary
The asphalt barrier presents an alternative to existing liner technology for waste containment. Given the longevity of buried asphalt and the high level of barrier performance shown by lab and field testing, asphalt-based liner materials are equivalent and in some regards superior to the present prescriptive Subtitle-D liner. In addition, the asphalt liner is thinner than the conventional Subtitle-D composite liner therefore saving valuable airspace.

3.3.5 Glass Liners

3.3.5.1 Introduction
The high chemical resistance of glass offers fascinating advantages for structures serving technical environmental protection. The development of the Integrated-Glass-Sandwich-Sealing system enables the use of those advantages for base liner systems for waste containment. In this sealing system, flatglass-elements are integrated into mineral layers so that the longlasting impermeability of glass can be combined with the mechanical protection of the flatglass-elements. The feasibility of the Integrated-Glass-Sandwich-Sealing has been proven by extensive experimental investigations on its mechanical resistance, as reported by Katzenbach and Weiler (1997).

3.3.5.2 System Properties
The sealing-system contains flatglass-elements integrated into mineral sealing layers. The properties of the mineral layers and the dimensions of the flatglass-elements should be adapted to site specific requirements. The cross joints between the flatglass-elements are about 10-mm wide and can be sealed by natural and synthetic sealing materials (bentonite, bentonite cement, silicon etc.). The area of the joints is only about 0.5 % of the waste site ground area.

Customary structural glass is used for the flatglass-elements. It consists of silica (71-75 %), soda (13-15 %) and lime (8-9 %). Since glass is an amorphous, homogenous, isotropic material, its properties are independent of the direction, and the load-deformation behavior is linear-elastic. An extensive overview of the mechanical properties of glass is given by Petzold et al. (1990) and Wörner and Sedlacek (1991).

The strength of glass is determined by the surface conditions, especially macroscopic cracks. Stresses on the surface lead to load concentration at the crack tip. If the load concentration exceeds the molecular strength of glass, crack growth and breaking will arise. This process is influenced by water decreasing the bond energy of the atoms at the crack tip. Water also reacts with glass under stress and causes a time-dependent reduction of strength. Further details of the mechanical properties of glass are provided by Katzenbach and Weiler (1997).

Glass, especially glass with a high rate of silica, is known for its chemical inertness and general resistance. The resistance of soda-lime-silica-glass is affected by the electrolyte pH. A summary of chemical reactions with glass is given by Scholze (1988).

Contact of glass with water and acids leads to a lixiviation of the alkali and alkaline earth elements. This interaction results in an insoluble SiO_2-protective coat on the surface, which stops further corrosion. The resistance against alkaline solutions depends essentially on the temperature. Hot alkaline solutions attack the [SiO_4]-netting resulting in dissolution of the glass. However, this effect is unlikely because the leachate at waste disposal sites typically is in the form of aqueous solutions. As the rate of reaction in contact to aqueous solutions is negligible, glass can be considered as chemical resistant (Scholze, 1988). Further details regarding the chemical resistance of glass are provided by Katzenbach and Weiler (1997).

The hydraulic conductivity of the Integrated-Glass-Sandwich-Sealing system is small (e.g., $\leq 10^{-13}$ m/s) compared to the standard design sealing systems because the possible water flow and contaminant transport is reduced to the area of the joints. Also, the hydraulic conductivity of the Integrated-Glass-Sandwich-Sealing system is less susceptible to incompatibility with the waste leachate because of the resiliency of the glass.

3.3.5.3 Field Test

Katzenbach and Weiler (1997) report the results of a large-scale field test performed to evaluate the ability of the flatglass-elements to withstand the loading that occurs during construction of the Integrated-Glass-Sandwich-Sealing system. The results indicated that all flatglass elements were placed without breaking, the covering layer was compacted to the required degree of compaction, the construction of the sealing system did not lead to residual stresses in the flatglass elements, and the horizontal position of the flatglass-elements stayed unchanged.

3.3.5.4 Summary

Based on their evaluation, Katzenbach and Weiler (1997) concluded that the use of flatglass elements in waste containment liners elements is technically feasible, the glass ensures a long-lasting sealing system, and the Integrated-Glass-Sandwich-Sealing significantly reduces the permeability compared to other standard German sealing systems.

3.3.6 Clay Membrane Barriers (CMBs)

3.3.6.1 Introduction

Clay membrane barriers, or CMBs, are clay barriers that exhibit membrane behavior by restricting the passage of solutes (Malusis et al. 2001, Shackelford et al., 2001; 2003). Restricted movement of charged solutes (ions) through the pores of a clay is attributed to electrostatic repulsion of the ions by electric fields associated with the diffuse double layers (DDLs) or adsorbed layers of cations of adjacent clay particles. Non-electrolyte solutes (uncharged species) also may be restricted from migrating through clays if the size of the solute molecule is greater than the pore size. This latter type of restriction commonly is referred to as *steric hindrance*, and occurs more often in the case of relatively large organic molecules (i.e., aqueous miscible organic compounds). The existence of clay membrane behavior also results in chemico-osmosis, or the movement of liquid in response to a solute concentration gradient. Chemico-osmosis resulting from membrane behavior in clays has been shown to (1) influence volume change, (2) cause apparent deviations from Darcy's law in laboratory hydraulic conductivity testing, (3) result in anomalous pore-fluid pressures in low-permeability geologic formations, and (4) affect the rate of solute migration through aquitards.

3.3.6.2 Factors Affecting CMBs

The ability of clays to act as CMBs is affected by several factors, including the state of stress on the soil, the types and amounts of clay minerals in the soil, and the types (species) and concentrations of the solutes in the pore water (Shackelford et al., 2003). In general, clay membrane behavior increases with an increase in stress (lower porosity), an increase in the amount of high activity clay minerals, and a decrease in the valence and concentration of the solute. In particular, several studies indicate that membrane behavior is significant in clay soils containing an appreciable amount of sodium montmorillonite, such as sodium bentonite (Malusis et al., 2001; Shackelford et al., 2001;

2003). Clay soils containing a significant amount of sodium montmorillonite, such as sodium bentonite, also are frequently used in waste containment applications (e.g., soil-bentonite cutoff walls, geosynthetic clay liners, sand-bentonite liners), due to the low hydraulic conductivity (e.g. ≤ 10^{-9} m/s) typically required in these applications. Thus, the existence of clay membrane behavior in such materials should not be surprising.

However, due to the reactive nature sodium montmorillonite, any factor that causes a compression of the DDLs will also result in a reduction in the efficiency of the membrane. Such factors include an increase in salt concentration in the pore liquid, an increase in the valence of the predominant cations in the pore liquid, a decrease in the dielectric constant of the pore liquid, and a decrease in the pH of the pore liquid (Shackelford et al., 2003). For example, Shackelford and Lee (2003) attributed a time-dependent decrease in the observed membrane behavior of a 5-mm-thick geosynthetic clay liner (GCL) due to diffusion of $CaCl_2$ from a 5-mM source solution into the pore liquid of the GCL such that all of the membrane behavior was effectively destroyed after ~ 35 days of diffusion. However, in general, the extent of membrane degradation due to diffusion decreases with decrease in salt concentration and/or salt cation valence (Shackelford et al., 2003). For example, Malusis and Shackelford (2002) found little or no membrane GCL specimens ranging in thickness from 8 to 13 mm when subjected to potassium chloride (KCl)solutions with KCl concentrations ranging from 3.9 to 47 mM.

3.3.6.3 Benefit of Clay Membrane Barriers

The potential benefit of membrane behavior is illustrated in Figure 9, where the solute mass flux at steady state through a 1-m-thick clay barrier that behaves as a semi-permeable membrane (J_m) relative to that which exists for non-membrane behavior (J_{nm}) is plotted as a function of ω and the hydraulic gradient (i_h). The results shown in Figure 9 are based on simulations using a coupled solute transport model and measured values for the effective salt diffusion coefficient (D^*) and chemico-osmotic efficiency (i.e., reflection) coefficient (ω) as described by Malusis and Shackelford (2004). The chemico-osmotic efficiency coefficient is a measure of the extent of solute restriction or membrane efficiency and ranges from zero in the case of no membrane behavior to unity in the case of an ideal or perfect semi-permeable membrane, whereby all solute migration is restricted ($0 \leq \omega \leq 1$). Of course, in the case of waste containment or isolation barriers, $\omega = 1$ is the desired result.

As shown in Figure 9, in the absence of membrane behavior, $\omega = 0$ such that $J_m = J_{nm}$. However, as membrane behavior becomes more prevalent (i.e., as ω increases), the solute mass flux exiting the barrier is reduced increasingly such that $J_m < J_{nm}$. For example, at a membrane efficiency of 60 % ($\omega = 0.6$), the exit solute mass flux at steady state is only about 30 % of that which would exist in the absence of membrane behavior (i.e., $J_m / J_{nm} \approx 0.3$). In the limit as $\omega \rightarrow 1$, $J_m / J_{nm} \rightarrow 0$ because, by definition, there can be no solute mass transport through an ideal or perfect membrane. The results for the two cases where a hydraulic gradient is applied (i.e., $i_h = 10$ and $i_h = 100$) are essentially the same as the results for the pure diffusion case ($i_h = 0$) because transport through the barrier is controlled by diffusion due to the very low hydraulic conductivity of the barrier material ($k \approx 10^{-11}$ m/s).

3.3.6.4 Summary

Clay membrane barriers are clay barriers that are able to restrict passage of solutes (contaminants). Although the existence of clay membrane behavior has been recognized for a long time, the use of this behavior for containment applications has only been proposed recently (e.g., see Malusis et al., 2001; 2003, Shackelford et al., 2001; 2003).

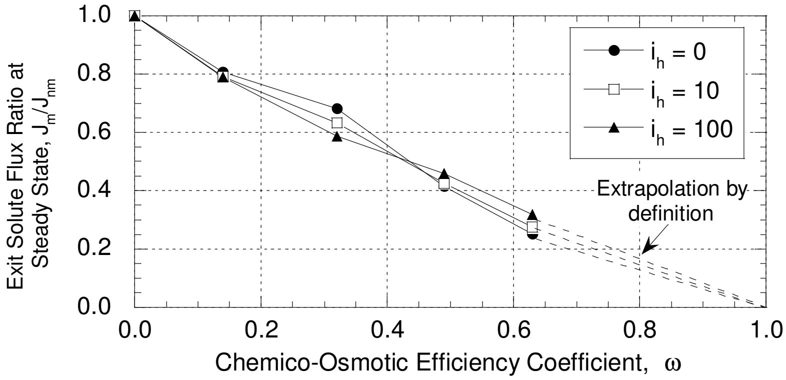


Figure 9. Effect of membrane behavior on solute mass flux at steady state through a 1-m-thick clay barrier (i_h = hydraulic gradient; J_m = membrane steady-state solute flux; J_{nm} = non-membrane steady-state solute flux) (results from Malusis 2001).

3.3.7 Residual Soil Liners

3.3.7.1 Introduction

In Brazil, practice in design and licensing of barriers underneath solid waste and other residues in disposal areas still follows obedience to prescriptions, instead of allowing sound discussions about performance predictions based on behavior simulations. The existence of imposed geometric prescriptions, i.e. limit minimum dimensions in a disposal site, to which engineering features must conform in order to be licensed by environmental Authorities, has also influenced Brazil. The environmental protection agency for the State of São Paulo, CETESB, has led the process, establishing minimum geometry for different classes of wastes since 1992; for municipal solid waste (MSW), considerations on local meteorological water balance, aquifer depth, and unsaturated superficial natural hydraulic conductivity, allow for 4 categories of covers and bottom liners (de Mello and Boscov, 1998). Practice has developed in such a way that many distinct liner concepts based on specific technical concepts from around the World have been used. CETESB's prescriptions have flexibility, and design based on performance criteria is being licensed.

3.3.7.2 General Characteristics of Lateritic Soils

The properties and behavior of tropical soils are peculiar relative to non-tropical (sedimentary and temperate climate) soils, due to the presence of geological and pedological processes typical of humid tropical regions. For a soil to be classified as tropical, therefore, it is not enough to have been generated in the tropics or in a region with humid tropical climate since it also must show peculiarities in its geotechnical behavior (Nogami and Villibor, 1995), which are typical for each family of tropical soils. Within the broad group of tropical soils, two big families are easily identified: lateritic soils, and saprolites or saprolitic soils.

A soil is considered to be lateritic when (a) it is situated in the well-drained superficial layers of a subsoil profile that have been developed in a humid tropical climate, and (b) its clay fraction is dominated by kaolinite clay minerals and iron or aluminum hydroxides and oxides, resulting in a peculiar porous structure and highly stable concretions. Lateritic soils are not necessarily residual soils in that transported superficial layers may also be lateritic if the above definition applies.

On the other hand, a soil is classified as saprolitic if (a) routine classification soil tests can be performed on it, (b) it clearly shows a relic structure of the matrix rock from which it originated, and (c) it has not been removed from its original location. Saprolitic soils are often simply referred to as residual soils.

Saprolitic soils can be very heterogeneous with relation to their basic geo-mechanical behavior, which is highly influenced by matrix rock and degree of weathering, whereas lateritic soils tend to allow for acceptance and behavior prediction, yielding compacted materials of low hydraulic conductivity, as acknowledged from their utilization in conventional embankment dams and documented by Cruz (1996), de Mello (1977) and many others. Lateritic soils can be found all over Brazil.

In the silt and sand fractions of lateritic soils, quartz usually predominates, together with lateritic concretions and heavy minerals, such as magnetite, ilmenite, rutile, tourmaline and zircon (Nogami and Villibor, 1995). The clay fraction of lateritic soils is mainly composed of kaolinite and hydrated oxides of iron and/or aluminum due to the laterization. The hydroxides and oxides precipitate on the surface of clay particles and cement groups of clay particles, reducing their water adsorption capacity and forming the so-called lateritic concretions, that are stable in the presence of water (Bernucci, 1995). The resulting microstructure resembles popcorn grains in scanning electron microscopy (SEM).

Because of the concretions, lateritic soils are porous, with low natural density and high natural hydraulic conductivity. They do not present a typical grain-size curve, since the quartz percentage is variable and different deflocculating agents may interfere differently; however, a high percentage of grains are smaller than 2 mm. High iron concentrations generate pebble like material or layers in a subsoil profile. Even with SEM, it is difficult to distinguish individual grains, which are linked by an apparently amorphous mass; they do not present the typical plate-like shape, and their apparent contours are rounded.

Lateritic soils may be residual or not, as the genetic factor does not predominate in a soil mechanics perspective. They are usually red, yellow, orange or brown, alone or with bands of each of these colors, as imposed by iron and aluminum ions, which are responsible for, respectively, red and yellow tones.

3.3.7.3 Geotechnical Properties of Lateritic Soils

Lateritic soils show well-defined compaction curves, with a steep slope on the dry side, even for clayey materials. These characteristics influence field compaction, as small water content variations may generate significant changes of dry densities.

Although lateritic soils show high values of hydraulic conductivity in their natural state, these values are greatly reduced by compaction. Small changes in water content or dry density of compacted lateritic soils can produce great changes in hydraulic conductivity, as observed from permeability tests carried out at different relative densities and molding water contents.

In general, lateritic soils, even if classified as clays or very clayey soils, show small expansion when compacted at the optimum water content, even if submerged in water. Furthermore, expansion is not dependent on the existing surcharge. The same soils, when compacted wet of optimum may show high expansion, above 1%.

3.3.7.4 Lateritic Soils as Clay Liners

Compacted lateritic clays with the typically low hydraulic conductivity recommended by most regulations for the construction of barriers at waste disposal sites are seldom found in Brazil. Nonetheless, these soils have been used as borrow material for liner construction and are being more thoroughly researched for this purpose in the last decade. Other alternative materials that have been considered are mixtures of natural soils, mixtures of sandy soils with bentonite and rolled concrete.

Migration of pollutants through natural foundation soils at disposal sites has also been researched, such as the thick layers of soft organic clay which occur along Brazilian coast, where many big cities are located (Barbosa, 1994; Barbosa et al., 1996).
Numerical modeling has been developed to contemplate particular issues of tropical soils, such as unsaturated condition and other retention processes (Vargas 1999, Queiroz 1999), and centrifuge modeling has proven to be an interesting tool (Almeida and Gurung, 1999; Villar and Merrifield, 1992). In situ measurements and determination of parameters by back-analyses, or at least published data from practical experience, are still needed for further development in design of waste disposal sites in Brazil.

3.3.7.5 Summary

Lateritic soils present unique properties and, therefore, potential problems with respect to their use as waste containment barriers. Nonetheless, the abundance of these soils in certain regions of the World, such as Brazil, means that reliance on their use for such purposes is required. As a result, lateritic soils represent innovative barrier materials in the sense that their use as waste containment barriers will require innovative design concepts. Research to pursue the use of lateritic soils as clay liners for waste containment currently is being emphasized.

3.3.8 Chemically Stabilized Clay Liners

3.3.8.1 Introduction

Amended or chemically stabilized soils may be required when the unamended soil is a fine-grained soil (e.g., a clay soil) with an unacceptably large hydraulic conductivity and/or is not compatible with the liquid waste or leachate (Shackelford and Nelson, 1996). For example, soil-bentonite (SB) slurry cutoff walls are constructed using a backfill mixture consisting of sodium bentonite slurry mixed with soil excavated from slurry trenches (i.e., instead of quarried soil). In addition, if a clay soil is not suitably stabilized against attack by a liquid waste the initially low hydraulic conductivity of the clay soil permeated with water may increase significantly as the liquid waste permeates the clay soil resulting in an unsuitable containment system. In such cases, a stabilizing additive, such as lime, cement, and/or attapulgite clay, may increase the compatibility of the clay soil to the liquid waste. For example, Broderick and Daniel (1990) evaluated the use of attapulgite clay as a stabilizing additive in terms of compacted clay soil.

3.3.8.2 Cement Stabilized Clay Liners

Cement-modified soils are mixtures of granular or silt-clay soils with a small percentage of cement used primarily to reduce plasticity and improve strength. Plastic soil-cement is a thorough mixture of soil and Portland cement combined with sufficient water to produce a consistency similar to that of plasticizing mortar. Soil cement has a higher cement content to produce a concrete-like material providing strength and durability. In general, cement-treated soils will exhibit a reduction in plasticity, an increase in strength, a decrease in hydraulic conductivity, and a reduction in volume change (Broderick, 1987).

3.3.8.3 Fly Ash Liners and Fly Ash Stabilized Clay Liners

Coal fly ash from electrical power plants has been considered for use as both a liner material and a stabilizer for soil liners in waste containment facilities (Vesperman et al., 1985; Edil et al., 1987; Bowders, 1988; Bowders et al., 1987; 1990; Usmen and Bowders, 1990; Almes and Bowders, 1991; Creek and Shackelford, 1992; Bowders et al., 1994; Palmer et al., 2000). For example, Vesperman et al. (1985) and Edil et al. (1987) report k values $\leq 1.0 \times 10^{-9}$ m/s for fly ash/quartz sand mixtures, whereas Bowders et al. (1987) report k ~ 8.5×10^{-10} m/s for a fly ash sample stabilized with 10 percent bentonite. Fly ash particles (typically < 0.075 mm) fill the pores between the larger sand particles to reduce the hydraulic conductivity of the mixture. The leaching of toxic heavy metal species associated with fly ash represents a potential "pit-fall" in the use of fly ash as a liner material (e.g., Creek and Shackelford, 1992).

3.3.8.4 Lime Stabilized Clay Liners

The addition of lime to fine-grained soils initiates several physico-chemical reactions that are not fully understood. However, improvements in the properties and characteristics of lime-treated soils include an increase in strength, a reduction in compressibility, and a decrease in hydraulic conductivity. Lime has been used to stabilize the hydraulic conductivity of clays against chemical attack by organic solutions (Broderick and Daniel, 1990), and as an additive to reduce the hydraulic conductivity of fly ash (Bowders et al., 1987).

3.3.8.5 Other Stabilizers

Other materials, such as synthetically produced polymers, also can be added to clay soils to decrease the hydraulic conductivity of the soil. For example, Haug and Bolt-Leppin (1994) evaluated the effect of the addition of a specially formulated commercial polymer on the hydraulic conductivity, k, of sand/bentonite mixtures containing 8 percent (dry wt.) of a "marginal quality" powdered bentonite. An anionic polyacrylamide polymer specifically designed to improve the low k nature of bentonite was used in the study. The results of flexible-wall (triaxial-cell) hydraulic conductivity tests using distilled water (DW) as the permeant liquid indicated that a polymer addition of only 0.05 percent was sufficient to reduce the hydraulic conductivity of the unamended sand-bentonite specimen by as much as 4 orders of magnitude to a value of 1.5×10^{-11} m/s, which was identical to the measured hydraulic conductivity of an unamended high quality bentonite-sand mixture. Thus, the potential effect of polymer material addition on the hydraulic conductivity of otherwise poor quality clay soils may be significant.

3.3.9 Polymer Gel Barriers

The potential use of polymers gels as barriers in the form of a grout or slurry has recently been proposed (Darwish et al., 2004). As described by Darwish et al. (2004), polymer chains are macromolecules consisting of a high number of units (monomers), either charged or neutral, that are bonded together. The charged polymers, referred to as polyelectrolytes, can be either negatively or positively charged, with the magnitude of the charge depending on the number of charged units. A gel is formed when water-soluble polymers are dissolved in water to form a polymer solution that contains polymer chains that are subsequently connected together at a number of points after the addition of a specific chemical referred to as a cross-linker. This process results in the formation of a three-dimensional polymer chain network analogous to a porous medium with pores containing the water used in the process. In the case of polyelectrolyte solutions, the charges are neutralized by counter-ions in the pores with charges opposite to that of the polyelectrolytes, in much the same way as cations balance the net negative charges of clay particles via cation exchange. Preliminary results show that polymer gels may have hydraulic conductivities that are as low, if not lower, than many natural clays (e.g., as low as 2×10^{-12} m/s), diffusion coefficients similar to that of compacted clay, and sorption capacities greater than that of a typical compacted clay (Darwish et al., 2004). However, these results are preliminary and more research is required to confirm the viability of the use of polymer gels as containment barriers.

REFERENCES

Allerton, D. K. et al. (1996). Waste containment barrier enhancement with zeolite. *Tailings and Mine Waste '96,* Balkema Publ., Rotterdam, The Netherlands, 255-264.

Almeida, M.S.S. and Gurung, S.B. (1999). Migration of some pollutants through clayey and sandy soils: centrifuge modelling. Proc. 4º Congresso Brasileiro de Geotecnia Ambiental, 297-306.

Almes, W. S. And Bowders, J. J. (1991). Stabilized fly ash cover for a municipal waste landfill. *Energy in the 90's*, ASCE Specialty Conference, ASCE, Reston, VA, 215-220.

Alvi, P.M. and Lewis, K.H. (1987). Geotechnical properties of industrial sludge. *Proc. First International Symposium on Environmental Geotechnology*. H.Y. Fang, Ed. Bethlehem, PA., Vol.2, 57-76.

Asphalt Institute (1976) *Asphalt In Hydraulics*, Asphalt Institute, College Park, MD, 68 p.

Aubertin, M. , Chapuis, R. P., Aachib, M., Ricard, J. F., Tremblay, L., and Bussiere, B. (1994). Cover technology for acidic tailings: Hydrogeological properties of milling wastes used as capillary barrier. *First International Congress on Environmental Geotechnics*, Edmonton, Alberta, Canada, July 10-15, BiTech Publ. Ltd., Richmond, British Columbia, Canada, 427-432.

Barbosa, M.C. (1994). *Investigação Geoambiental do Depósito de Argila Sob o Aterro de Resíduos Urbanos de Gramacho-RJ" (Geoenvironmental Investigation of the Clay Deposit Under MSW Gramacho Landfill-RJ).* Doctorate thesis, COPPE/UFRJ, Universidade Federal do Rio de Janeiro, Brazil.

Barbosa, M.C., Almeida, M.S.S. & Ehrlich, M. (1996). Ions diffusion through an organic saline clay soil. *2nd International Congress on Environmental Geotechnics, IS-Osaka '96*, M. Kamon, ed., Osaka, Japan, Nov. 5-8, 1996, Balkema, Rotterdam, Vol. 1, 7-12.

Benson, C. H. (1999). Final covers for waste containment systems: A North American perspective, *Proc. XVII Conference of Geotechnics of Torino, Control and Management of Subsoil Pollutants*, Italian Geotechnical Society, Torino, Italy, 1-32.

Benson, C. H. and Khire, M. V. (1995). Earthen covers for semi-arid and arid climates. *Landfill Closures*, J. Dunn and U. Singh, eds., Geotechnical Special Publication No. 53, ASCE, Reston, VA, 201-217.

Benson, C. H. and Khire, M. V. (1997), Earthen materials in surface barriers, *Barrier Technologies for Environmental Management,* National Academy Press, National Research Council, Washington D.C., D79-D89.

Benson, C., Abichou, T., Wang, X., Gee, G., and Albright, W. (1999). *Test Section Installation Instructions Alternative Cover Assessment Program*, Environmental Geotechnics Report 99-3, University of Wisconsin, Madison, Wisconsin, USA.

Blowes, D. W. ,Reardon, E. J., Jambour, J. L., and Cherry, J. A. (1991). The formation and potential importance of cemented layers in inactive sulfide mine tailings. *Geochimica et Cosmochimica Acta*, 55(4), 965-978.

Bouazza, A., Richards, S., and Chan, I. (2001). Soil-biopolymer barriers and soil-biofilm barriers. XVth ICSMGE, Istanbul, Balkema Publ., Rotterdam.

Bouazza, M., Zornberg, J.G., and Adam, D. (2002). Geosynthetics in waste containment facilities: Recent advances. (Keynote paper) Proceedings of the *Seventh International Conference on Geosynthetics*, Nice, France, 22-27 September, A.A. Balkema, Vol. 2, pp. 445-510.

Bowders, J. J. (1988). Fly ash liners for waste disposal facilities. *Geotechnical News*, 6(4), 26-29.

Bowders, J. J., Gabr, M. A., Boury, E., and Baker, R. C. (1994). Engineering evaluation of amended fly ash for hydraulic barriers. *International Conference on Land Reclamation & Mine Drainage*, April, 24-29, 1994, Pittsburgh, Pennsylvania, United States Department of the Interior, Bureau of Mines Special Publication SP 06B-94, Vol. 2, 226-233.

Bowders, J. J., Gidley, J. S., and Usmen, M. A. (1990). Permeability and leachate characteristics of stabilized Class F fly ash. *Transportation Research Record 1288*, Transportation Research Board, National Research Council, National Academy Press, Washington, DC, 70-77.

Bowders, J. J., Loehr, J. E., Mooney, D. T. and Bouazza, A. (2000). Asphalt barriers for waste containment. *International Conference on Geotechnical and Geoenvironmental Engineering (GeoEng2000),* Melbourne, Australia, Nov. 19-24, Technomic Publ. Co., Inc., Lancaster, PA, Vol. 2, 243.

Bowders J.J., Loehr J.E., Neupane D. and Bouazza A. (2003) Construction quality control for asphalt concrete hydraulic barriers. *Journal of Geotechnical and Geoenvironmental Engineering*, 129(3), 219-223.

Bowders, J. J., Neupane, D., and Loehr, J. E. (2002). Sidewall leakage in hydraulic conductivity testing of asphalt concrete specimens. *Geotechnical Testing Journal*, 25(2), 210-214.

Bowders J, Neupane D, Loehr, J. E, Mooney D. T.and Bouazza, A.(2001).Asphalt-based lining system for landfills, *Proceedings of the MissouriWaste Control Coalition Conference, 2001 A Waste Odyssey*, July 7-9,Columbia, Missouri, 8 p.

Bowders, J. J., Usmen, M. A., and Gidley, J. S. (1987). Stabilized fly ash for use as low-permeability barriers. *Geotechnical Practice for Waste Disposal*, R. D. Woods, ed., Geotechnical Special Publication No. 13, ASCE, Reston, VA, 320-333.

Broderick, G. (1987). *Stabilization of Compacted Clay Against Attack by Concentrated Organic Chemicals*. Ph. D. Dissertation, University of Texas, Austin.

Broderick, G. and Daniel, D. E. (1990). Stabilizing compacted clay against chemical attack. *Journal of Geotechnical Engineering*, 116(10),1549-1567.

Bernucci, L.L.B. (1995). *Considerações Sobre o Dimensionamento de Pavimentos Utilizando Solos Lateríticos Para Rodovias de Baixo Volume de Tráfego (Considerations on Pavement Design Using Lateritic Soils for Low Volume Roads).* Doctorate thesis, Escola Politécnica, Universidade de São Paulo, Brazil (in Portuguese).

Chen, C. (1999). Meteorological Conditions for Design of Monolithic Alternative Earthen Covers (AEFCs), MS Thesis, University of Wisconsin, Madison, Wisconsin, USA.

Chermak, J. A. and Runnells, D. D. (1996). Self-sealing hardpan barriers to minimize infiltration of water into sulfide-bearing overburden, ore, and tailings piles. *Tailings and Mine Waste '96,* Balkema Publ., Rotterdam, The Netherlands, 265-274.

Creek, D. N. and Shackelford, C. D. (1992). Permeability and leaching characteristics of fly ash liner materials. *Geoenvironmental and Engineering Properties of Rock, Soil, and Aggregate*, Transportation Research Record No. 1345, National Academy Press, Washington, D.C., 74-83.

Cruz, P.T. (1996) *100 Barragens Brasileiras. Casos Históricos. Materiais de Construção. Projeto" (100 Brazilian Dams: Case History, Construction Materials, Design).* Edited by Oficina de Textos, São Paulo, Brazil (in Portuguese).

Darwish, M. I. M., Rowe, R. K., van der Maarel, J. R. C., Pel, L., Huinink, H., and Zitha, P. L. J. (2004). Contaminant containment using polymer gel barriers. *Canadian Geotechnical Journal*, 41(1), 106-117.

De Mello, V. F. B. (1977). Reflection on design decisions of practical significance to embankment dams. *Géotechnique*, 27, 279-355.

De Mello, L. G. & Boscov, M. E. G. (1998). Discussão da prática brasileira de disposição de resíduos à luz das tendências internacionais (Discussion of Brazilian practice of waste disposal in the context of international trends). *XI Congresso Brasileiro de Mecânica dos Solos e Engenharia Geotécnica, Proceedings*, (in Portuguese), Vol. 4.

Dennis, M. L. and Turner, J. P. (1998). Hydraulic conductivity of compacted soil treated with biofilm., *Journal of Geotechnical and Geoenvironmental Engineering*, 124(2), 120-127.

Edil, T. B., Berthouex, P. M., and Vesperman, K. D. (1987). Fly ash as a potential waste liner. *Geotechnical Practice for Waste Disposal*, R. D. Woods, ed., Geotechnical Special Publication No. 13, ASCE, Reston, VA, 447-461.

Evangelou, V. P. and Zhang, Y. L. (1995). A review: Pyrite oxidation mechanisms and acid mine drainage prevention. *Critical Reviews in Environmental Science and Technology*, 25(2),141-149.

Evans, J. C., Sambasivam, Y., and Zarlinski, S. (1990). Attenuating materials for composite liners. *Waste Containment Systems: Construction, Regulation, and Performance*, ASCE Geotechnical Special Publication No. 26, R. Bonaparte, ed., ASCE, Reston, VA, 246-263.

Fayer, M., Gee, G., Rockhold, M., Freshley, M. and Walters, T. (1996), Estimating recharge rates for a groundwater model using a GIS, *Journal of Environmental Quality*, 25, 510-518.

Floess, C.H., Smith, R.F.J., and Hitchcock, R.H. (1995), Capping with fiber flay. *Civil Engineering*. ASCE. 65(8), 62-63.

Floess, C.H., Moo-Young, H.K., Zimmie, T.F., and Harris, W.A. (1998). A municipal landfill cover with a paper sludge barrier layer. *Fourth International Conference on Case Histories in Geotechnical Engineering*. S. Prakash, ed. 1009-1017.

Gee, G. and Ward, A. (1997). Still in quest of the perfect cap, *Proceedings, Landfill Capping in the Semi-Arid West: Problems, Perspectives, and Solutions*, Environmental Science and Research Foundation, Idaho Falls, Idaho, USA, 145-164.

Giroud, J.P., Gleason, M.H., and Zornberg, J.G. (1999). Design of geomembrane anchorage against wind action. *Geosynthetics International*, 6(6), 481-507.

Giroud, J.P., Pelte, T., and Bathurst, R.J. (1995). Uplift of geomembranes by wind. *Geosynthetics International*, 2(6), 897-952.

Giroud, J.P., Zornberg, J.G., and Zhao, A. (2000). Hydraulic design of geosynthetic and granular liquid collection layers. *Geosynthetics International*, 7(4-6), 285-380.

Gleason, M.H., Houlihan, M.F., and Palutis, J.R. (2001). Exposed geomembrane cover systems: technology summary. *Proceedings of the Geosynthetics' 01 Conference*, Portland, Oregon, Industrial Fabrics Association International, Minnesotta, 905-918.

Gray, D. H. (1995). Containment strategies for landfill wastes. *Geoenvironment 2000*, ASCE Geotechnical Special Publication No. 46, Y. B. Acar and D. E. Daniel, eds., ASCE, Reston, VA, 484-498.

Gregg, J. J., Zander, A. K. and Theis, T. L. (1997). Fate of trace elements in energy recovery from recycled paper sludge." *TAPPI Journal*. 80(9), 157-162.

Haug, M. D. and Bolt-Leppin, B. (1994). Influence of polymers on the hydraulic conductivity of marginal quality bentonite-sand mixtures. *Hydraulic Conductivity and Waste Contaminant Transport in Soil, ASTM STP 1142*, D. E. Daniel and S. J. Trautwein, eds., ASTM, West Conshohoken, PA, 407-421.

Izu, P., Zulueta, A., and Salas, O. (1998). Laboratory testing of several paper sludges as raw materials for landfill covers. *First International Workshop for the Use of Paper Industry Sludge in Environmental Geotechnology and Construction*. Painopaikka, Helsinki, Finland, 23-29.

Katzenbach, R. and Weiler, H. (1997). Base liner systems of dump sites with a longlasting contaminant protection by flatglass-elements. *Proceedings, 14th International Conference on Soil Mechanics and Foundation Engineering*, Hamburg, Germany, Sept. 6-12, 1997, Balkema, Rotterdam, Vol.3, 1915-1918.

Khire, M., Benson, C., Bosscher, P., and Pliska, P. (1994). Field-scale comparison of capillary and resistive landfill covers in an arid climate. *14th Annual American Geophysical Union Hydrology Days*, H. J. Morel-Seytoux, ed., Colorado State University, Fort Collins, Colorado, April 5-8, Hydrology Days Publ., 57 Shelby Lane, Atherton, California 94027-3926, 195-209.

Khire, M., Benson, C., and Bosscher, P. (2000). Capillary barriers: Design variables and water balance. *Journal of Geotechnical and Geoenvironmental Engineering*, 126(8), 695-708.

Kraus, J.F., Benson, C.H., Van Maltby, C. and Wang, X. (1997). Laboratory and field hydraulic conductivity of three compacted paper mill sludges. *Journal of Geotechnical and Geoenvironmental Engineering*, 123(7), 654-662.

Landva, A.O. and LaRochelle (1983). Compressibility and shear characteristics of Radford peats. *Testing Peats and Organic Soils*. P.M. Jarrett, ed., ASTM STP 820, ASTM, West Conshohoken, PA, 157-191.

Maltby, V. (1999). Study of the use of paper industry residuals in landfill covers systems. *Second International Workshop on the Use of Paper Industry Sludges in Environmental Geotechnology and Construction*, Rensselaer Polytechnic Institute. Troy, NY., 80-84.

Malusis, M. A. (2001). *Membrane Behavior and Coupled Solute Transport through a Geosynthetic Clay Line*. PhD Dissertation, Colorado State University.

Malusis, M. A. and Shackelford, C. D. (2002). Chemico-osmotic efficiency of a geosynthetic clay liner. *Journal of Geotechnical and Geoenvironmental Engineering*, 28(2), 97-106.

Malusis, M. A. and Shackelford, C. D. (2004). Explicit and implicit coupling during solute transport through clay membrane barriers. *Journal of Contaminant Hydrology*, 72(1-4), 259-285.

Malusis, M. A., Shackelford, C. D., and Olsen, H. W. (2001). Flow and transport through clay membrane barriers. *Geoenvironmental Engineering, Geoenvironmental Impact Management*, R. N. Yong and H. R. Thomas, eds., Thomas Telford Publ., London, UK, 334-341.

Malusis, M. A., Shackelford, C. D., and Olsen, H. W. (2003). Flow and transport through clay membrane barriers. *Engineering Geology*, 70(2-3), 235-248.

Martin, G. R., Yen, T. F., and Karimi, S. (1996). Application of biopolymer technology in silty soil matrices to form impervious barriers" *7th Australia-New Zealand Conf. on Geomechanics,* Adelaide, 814-819.

Monismith, C. L. and Creegan, P. J. (1996*) Asphalt Concrete Water Barriers for Embankment Dams*, ASCE Press, Reston, VA, 196p.

Moo-Young, H. K. (1992). *Evaluation of the Geotechnical Properties of a Paper Mill Paper Sludge for Use in Landfill Covers*. MS Thesis, Rensselaer Polytechnic Institute, Troy, N. Y.

Moo-Young, H. K. (1995). *Evaluation of Paper Mill Paper sludges for Use As Landfill Covers*. Ph.D. Thesis, Rensselaer Polytechnic Institute, Troy, NY.

Moo-Young, H. K. and Zimmie, T. F. (1996a). Geotechnical properties of paper of mill paper sludges for use in landfill covers. *Journal of Geotechnical and Geoenvironmental Engineering*, 122(9), 768-775.

Moo-Young, H. K. and Zimmie, T. F. (1996d). Effects of organic decomposition on paper mill sludges used as landfill covers. *2nd International Congress on Environmental Geotechnics, IS-Osaka '96*, M. Kamon, ed., Osaka, Japan, Nov. 5-8, 1996, Balkema, Rotterdam, Vol. 2, 827-832.

Moo-Young, H. K. and Zimmie,T. F. (1997a). Utilizing a paper sludge barrier layer in a municipal landfill cover in New York. *ASTM STP 1275*, West Conshohoken, PA, 125-140.

Moo-Young, H. K. and Zimmie, T. F. (1997b). Waste minimization and reuse of paper mill sludges as landfill covers: A case study. *Journal of Waste Management and Research*, 15(6), 593-605.

Moo-Young, H. K. and Zimmie, T. F. (1997d). A comparison of clay to paper sludge as hydraulic barrier in municipal landfill covers." *Proceeding of 14th International Conference Soil Mechanics and Foundation Engineering Conference*. Hamburg, Germany, 1923-1926.

Moo-Young, H. K. (1998). Designing waste containment systems utilizing paper sludge sapping layers. *First International Workshop for the Use of Paper Industry Sludge in Environmental Geotechnology and Construction*, Painopaikka, Helsinki, Finland, 40-48.

Moo-Young, H. K. and Gallagher, M. (1998). Determination of the adsorption capacity of paper sludge as a reactive impermeable barrier. *First International Workshop for the Use of Paper Industry Sludge in Environmental Geotechnology and Construction*. Painopaikka: Helsinki, Finland, 73-80.

Moo-Young, H. K., Gallagher, M. J., Amadon, A. J., Polaski, J., and Wolfe, M. (2000). Use of paper clay as a barrier for treatment of contaminated ground water. *ASTM STP 1374*, West Conshohocken, PA.

Morrison, S. J., V. S. Tripathi, and Spangler, R. R. (1995). Coupled reaction/transport modeling of chemical barrier for controlling uranium(VI) contamination in groundwater. *Journal of Contaminant Hydrology*, 17(4), 347-363.

NCASI, National Council of the Paper Industry for Air and Stream Improvement Inc. (1984), *The Land Application and Related Utilization of Pulp and Paper Mill Paper Sludges*, NCASI Technical Bulletin No. 439.

NCASI, National Council of the Paper Industry for Air and Stream Improvement Inc. (1985) *Recent Studies and Experience with the Use of Paper Sludge and Fly Ash on Farm Crop and Forest Lands*. NCASI Technical Bulletin No. 478.

NCASI, National Council of the Paper Industry for Air and Stream Improvement Inc. (1989) *Experience with and Laboratory Studies of the Use of Pulp and Paper Mill Solid Wastes in Landfill Cover Systems*. NCASI Technical Bulletin No. 559.

NCASI, National Council of the Paper Industry for Air and Stream Improvement Inc. (1990) *A Field-Scale Study of the Use of Paper Industry Paper Sludges in Landfill Cover Systems: First Progress Report*. NCASI Technical Bulletin No. 595.

NCASI, National Council of the Paper Industry for Air and Stream Improvement Inc. (1992a) Progress in Reducing Water Use and Wastewater Loads in the U.S. Paper Industry. NCASI Technical Bulletin No. 603.

NCASI, National Council of the Paper Industry for Air and Stream Improvement Inc. (1992b) *Solid Waste Management and Disposal Practices in the U.S. Paper Industry*. NCASI Technical Bulletin No. 641.

Neupane, D., Bowders, J. J., Loehr, J. E., Bouazza, A., and Trautwein, S. J. (2005). Sealed double ring infiltrometers for estimating very low hydraulic conductivity. *Geotechnical Testing Journal*, in review.

Nicholson, R. V. et al. (1989). Reduction of acid generation in mine tailings through the use of moisture-retaining cover layers as oxygen barriers. *Canadian Geotechnical Journal*, 26, 1-8.

Nogami, J. S. and Villibor, D. F. (1995). "Pavimentos de baixo custo com solos lateríticos" (Low cost pavements with lateritic soils). Editora Villibor, ed., 213 p. (in Portuguese).

Petzold, A., Marusch, H., Schramm, B. (1990). *Der Baustoff Glas.* Verlag für Bauwesen, Berlin.

Palmer, B. G., Edil, T. B., and Benson, C. H. (2000). Liners for waste containment constructed with class F and class C fly ashes. *Journal of Hazardous Materials*, 76(2-3), 193-216.

Queroz, P. I. B. (1999). Sobre a modelagem de transporte de contaminantes no solo (About modeling pollutant transport through soils). *4º Congresso Brasileiro de Geotecnia Ambiental, Proceedings*, São José dos Campos, Brazil, 307-314, (in Portuguese).

Quiroz, J. D. and Zimmie, T. F. (1998). In-situ strength testing on paper mill sludge landfill cover. *First International Workshop on the Use of Paper Industry Sludges in Environmental Geotechnology and Construction*, Painopaikka, Helsinki, Finland, 30-38.

Quiroz, J. D. and Zimmie, T. F. (1999). Paper mill sludge landfill cover design. *Second International Workshop on the Use of Paper Industry Sludges in Environmental Geotechnology and Construction*, Rensselaer Polytechnic Institute, Troy, NY., 26-35.

Ribet, I., Ptacek, C. J., Blowes, D. W., and Jambor, J. L. (1995). The potential for metal release by reductive dissolution of weathered mine tailings. *Journal of Contaminant Hydrology*, 17(3), 239-273.

RMA (1997). *Rocky Mountain Arsenal, Final RCRA-Equivalent Cover Demonstration Project Comparative Analysis and Field Demonstration Scope of Work*, Dec. 1997, Rocky Mountain Arsenal Remediation Venture Office, Commerce City, Colorado, USA.

Saarela, J. (1999). Developments in controlled closuring of landfills with paper sludges in Finland. *Second International Workshop on the Use of Paper Industry Sludges in Environmental Geotechnology and Construction*. Rensselaer Polytechnic Institute, Troy, N.Y., 21-25.

Scholze, H. (1988). *Glas: Natur, Struktur und Eigenschaften.* 3. Aufl., Berlin: Springer-Verlag

Shackelford, C. D. (1988). Diffusion as a transport process in fine-grained barrier materials. *Geotechnical News*, Bi-Tech Publ. Ltd., Richmond, British Columbia, Canada, 6(2), 24-27.

Shackelford, C. D. (1989). Diffusion of contaminants through waste containment barriers. *Geotechnical Engineering 1989*, Transportation Research Record 1219, Transportation Research Board, National Research Council, Washington, D. C., 169-182.

Shackelford, C. D. (1997). Modeling and analysis in environmental geotechnics: An overview of practical applications. *2nd International Congress on Environmental Geotechnics, IS-Osaka '96*, M. Kamon, ed., Osaka, Japan, Nov. 5-8, 1996, Balkema, Rotterdam, Vol. 3, 1375-1404.

Shackelford, C. D. (1999). Reactive nature of passive containment barriers, Discussion 6.1, *Proceedings, 14th International Conference on Soil Mechanics and Foundation Engineering*, Hamburg, Germany, Sept. 6-12, 1997, Balkema, Rotterdam, Vol. 4, 2535-2536.

Shackelford, C. D. (2000). Remediation of contaminated land: An overview, *Proceedings, XI Pan-American Conference on Soil Mechanics and Geotechnical Engineering*, Iguasu Falls, Brazil, Aug. 8-13, 1999, Vol. 4, 371-430.

Shackelford, C. D. and Jefferis, S. A. (2000). Geoenvironmental engineering for in situ remediation, *International Conference on Geotechnical and Geoenvironmental Engineering (GeoEng2000),* Melbourne, Australia, Nov. 19-24, Technomic Publ. Co., Inc., Lancaster, PA, Vol. 1, 121-185.

Shackelford, C. D. and Lee, J.-M. (2003). The destructive role of diffusion on clay membrane behavior. *Clays and Clay Minerals*, 51(2), 187-197.

Shackelford, C. D. and Nelson, J. D. (1996). Geoenvironmental design considerations for tailings dams, *Proceedings, International Symposium on Seismic and Environmental Aspects of Dams Design: Earth, Concrete and Tailings Dams*, Santiago, Chile, Sociedad Chilena de Geotecnia, Vol.1, 131-187.

Shackelford, C. D., Malusis, M. A., and Olsen, H. W. (2001). Clay membrane barriers for waste containment, *Geotechnical News*, 19(2), 39-43.

Shackelford, C. D., Malusis, M. A., and Olsen, H. W. (2003). Clay membrane behavior for geoenvironmental containment. *Soil and Rock America Conference 2003* (Proceedings of the joint *12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering* and the *39th U. S. Rock Mechanics Symposium*), P. J. Culligan, H. H. Einstein, and A. J. Whittle, eds., Verlag Glückauf GMBH, Essen, Germany, Vol. 1, 767-774.

Sherard J. L., Woodward, R. J., Gizienski, S. F., and Clevenger, W. A. (1963) *Earth and Earth-Rock Dams*, John Wiley and Sons, New York, 725 p.

Smith, J. A., Li, J., and Galan, A. (1995). Organobentonites as components of earthen landfill liners to minimize contaminant transport. *Geoenvironment 2000*, Geotechnical Specialty Publication No. 46, Y. B. Acar and D. E. Daniel, eds., ASCE, Reston, VA, 806-814.

Springer, Allen M. (1986). *Industrial Environmental Control – Pulp and Paper Industry*, John Wiley and Sons, New York, New York.

Stoeffel, C.M. and Ham, R.K. (1979). *Testing of High Ash Paper Mill Sludge for Use in Sanitary Landfill Construction. Prepared for the City of Eau Claire,* Wisconsin by Owen Ayers and Assoc., Inc. Eau Claire, WI.

Stormont, J. (1995). The performance of two capillary barriers during constant infiltration. *Landfill Closures*, GSP No. 53, J. Dunn and U. Singh, eds., ASCE, Reston, VA, 77-92.

Stormont, J. and Anderson, C. (1999). Capillary barrier effect from underlying coarser soil layer. *Journal of Geotechnical and Geoenvironmental Engineering*, 125(8), 641-648.

Stormont, J. and Morris, C. (1998). Method to estimate the water storage capacity of capillary barriers, *Journal of Geotechnical and Geoenvironmental Engineering*, 124(4), 297-302.

Stormont, J. C., Morris, C. E., and Finely, R. E. (1996). Capillary barriers for covering mine wastes. *Tailings and Mine Waste '96, Third International Conference*, Colorado State University, Fort Collins, Colorado, Jan. 16-19, Balkema Publ., Rotterdam, The Netherlands, 201-210.

Thornton, S. F. et al. (1993). The role of attenuation in landfill liners. *Proceedings, Fourth International Landfill Symposium*, Sardinia, Italy, Oct. 11-15, 1993, CISA, Environmental Engineering Sanitary Centre, Cagliari, Italy, 407-416.

Usmen, M. A. and Bowders, J. J. (1990). Stabilization characteristics of Class f fly ash. *Transportation Research Record 1288*, Transportation Research Board, National Research Council, National Academy Press, Washington, D. C. , 59-69.

Vargas Jr, E. A. (1999). Modelagem de Transporte de Contaminantes em Solos (Modeling of Pollutant Transport through Soils)". Invited Speech, Escola Poltécnica, Universidade de São Paulo, Brazil.

Vesperman, K. D., Edil, T. B., and Berthoux, P. M. (1985). Permeability of fly ash and fly ash sand mixtures. *Hydraulic Barriers in Soil and Rock, ASTM STP 874*, A. I. Johnson, R. K. Frobel, N. J. Cavalli, and C. B. Pettersson, eds, ASTM, West Conshohoken, PA, 289-298.

Villar, H. and Merrifield, C. (1992). Centrifuge modelling of radioactive waste migration through soil using radiotracers. *Environmental Geotechnology*, (*Proceedings of the Mediterranean Conference on Environmental Geotechnology*, Turkey, 1992). M. A. Usmen and Y. B. Acar, eds., Balkema, Rotterdam, 87-92.

Watkins, R. I. (1996). A prototype barrier for long term waste isolation. *Tailings and Mine Waste '96, Third International Conference*, Colorado State University, Fort Collins, Colorado, Jan. 16-19, Balkema Publ., Rotterdam, The Netherlands, 279-287.

Wing, N. R. and Gee, G. W. (1994a). The development of surface barriers at the Hanford Site. *Proccedings of the 33rd Hanford Symposium on Health and the Environment, In Situ Remediation: Scientific Basis for Current and Future Technologies*, Part 1, G. W. Gee and N. R. Wing, eds., 427-440.

Wing, N. R. and Gee, G. W. (1994b). Quest for the perfect cap. *Civil Engineering*, 64(10), 38-41.

Winkler, W. (1999). *Thickness of Monolithic Covers in Arid and Semi-Arid Climates*, MS Thesis, University of Wisconsin, Madison, Wisconsin, USA.

Wörner, J.-D. and Sedlacek, G. (1991). *Der Baustoff Glas im Konstruktiven Ingenieurbau.* Sachstandsbericht des Institutes für Konstruktiven Glasbau, Nr. 1, Gelsenkirchen (in German).

Xiao, C., L. Q. and Sarigumba, T. (1999). Effects of soil on trace metal leachability from paper mill ashes and sludge. *Journal of Environmental Quality*, 28(1), 321-333.

Zander, A. K., Theis, T. L. and Brennan, M. (1996). Energy from paper sludge: Criteria and hazardous air pollutants. *Journal of Environmental Engineering*, 122(8), 758-760.

Zimmie, T. F. and Moo-Young, H. K. (1995). Hydraulic conductivity of paper sludges used for landfill covers. *Geoenvironment 2000*, ASCE Geotechnical Special Publication No. 46, Y. B. Acar and D. E. Daniel, eds., ASCE, Reston, VA, Vol. 2, 932-946.

Zornberg, J.G., and Caldwell, J.A. (1998). Design of monocovers for landfills in arid locations. *Proceedings, Third International Conference on Environmental Geotechnics, Lisbon,* Portugal, September 1998, P.S. Seco e Pinto (Editors), A. A. Balkema, Rotterdam,Vol.1, 169-174.

Zornberg, J. G. and Giroud, J. P. (1997). Uplift of geomembranes by wind – Extension of equations. *Geosynthetics International*, 4(2), 187-207.

Zornberg, J.G., LaFountain, L., and Caldwell, J.C. (2003). Analysis and design of an evapotranspirative cover for a hazardous waste landfill. *Journal of Geotechnical and Geoenvironmental Engineering,* 129(5), 427-438.

Zornberg, J.G., Somasundaram, S., and LaFountain, L. (2001). "Design of Geosynthetic-Reinforced Veneer Slopes." Proc. of the *International Symposium on Earth Reinforcement* (IS Kyushu 2001), Fukuoka, Japan, A.A. Balkema, 14-16 November, Tokyo, Vol. 1, 305-310.

Chapter 4

Underwater Geoenvironmental Issues

Author:

Masashi KAMON
Kyoto University, Japan

Contributing Authors:

M. BOUTOUIL
Caen Technical College, France
W. VAN IMPE
Ghent University, Belgium
G. VAN ROEKEL
Fugro-Milieu Consult B.V., The Netherlands

The Authors gratefully acknowledge the review of this chapter and the contribution made to it by:

Andy FOURIE*, School of Civil Environmental Eng., Johannesburg, South Africa*
Mark VAN DEN BROECK*, Dredging International, Zwijndrecht, Belgium*

ABSTRACT: Underwater geoenvironmental issues include a variety of aspects in Environmental Geotechnics. Waste sludge and dredging management and utilization are the focus of this report. Some recent developments of dredging operations and containment techniques are also described. Waste and dredged sludge contamination can still be a daunting problem from a technical and regulatory standpoint. Many utilization techniques are available under carefully controlled operation systems. In particular, beneficial use of dredged materials as reclamation is introduced along with case studies for land use.

4.1 GENERAL VIEW OF UNDERWATER PROBLEMS

Dredged material or waste sludge is discharged from dredging undertaken for cleanup of sediments in rivers and lakes and for maintaining the navigation depth in ports, as well as from the process of installing foundations, such as cast-in-place concrete piles, continuous diaphragm walls, shield tunnels, etc. The quantity of some forms of waste sludge is increasing, which can lead to an increased frequency of illegal dumping of waste sludges. The quality of waste sludge varies, depending on the types and degree of contamination by toxic substances. Further, disposal sites are being filled faster than originally predicted such that the original disposal life and integrity of the disposal system needs to be confirmed to avoid contamination of the surrounding ground and ground water (Kamon et al. 1998).

In the 1970s, protocols for the control of dredged material were set up, two of which are the London (Dumping) Convention and the Oslo and Paris Convention. They were set up primarily to regulate the disposal of noxious substances into the oceans, but they included the regulation of dredged sediment as well. The inclusion of contaminated dredged materials within this report is to be expected, given that the annual volume of dredged material disposed at sea greatly exceeds any other material.

Within the United States, about 500 million m^3 of sludge is dredged annually (US Army Corps of Engineers: COE, 1987). Throughout the Great Lakes, about 5 million m^3 of sludge is dredged annually to maintain navigation in channels and harbors for commercial, military, and recreational users, and for environmental remediation projects (EPA 1990). About one half of the total sludge dredged in the Great Lakes is sufficiently contaminated to be problematic sludge and require placement in a confined disposal facility (CDF). The contaminated sludge requires special consideration during dredging and disposal operations because of the potentially adverse impact on water quality and local organisms. As for European countries, the amounts of sludge dredged annually in millions m^3 are as follows: France: 50 (among which 75 % are silt); England: 40; Germany: 50; and Netherlands: 45. The largest volumes are concentrated in the major ports of these countries such as: Rotterdam, Hamburg, Antwerp, Portsmouth, Le Havre, and Marseilles (Alzieu et al. 1991).

In the northeastern Atlantic/North Sea regions, approximately 150 million m^3 of dredged material were disposed of in 1990 compared with 10 million m^3 of sewage sludge and less than 2 million m^3 of chemical waste. In Japan in 1995, waste sludge amounting to 30 million m^3 was generated from dredging works and 10 million m^3 was generated from construction works. However, the contamination level of the waste sludge in Japan is rather low and almost all pf the dredged materials were reclaimed in wetlands. About 90 % of waste sludge from construction works was disposed because of difficulties associated with reuse.

Recognizing that dredged material consists mainly of natural sediment and that only a small proportion of the total volume dredged is contaminated, a number of organizations launched a campaign to change the perception of this material such that the term gradually dredged "spoil" has been substituted for the term dredged "material". This substituted term is now embedded in convention and has been a contributory in getting dredged material treated as a special case.

4.2 BASIC CHARACTERISTICS OF THE UNDERWATER MATERIALS

Properties of waste sludges vary greatly due to differences in origin, sedimented area, dredging and excavation methods employed, additive materials, etc. One of the main characteristics of sludges is a high water content. In particular, sedimented sludges in lakes, rivers or seas, have extremely high gravimetric water contents on the order of 200 - 250%. The unit densities of sludges also are closely related to their water contents, as shown in Figure 1. The upper part of sedimented sludges is very soft. The sludges also have high organic contents. The ignition-loss of most dredged materials ranges from 5 to 15 %. In particular, paper sludges in Japan have a high organic content with ignition losses on the order of 45 %. Figure 2 shows the relationship between ignition loss of paper sludge and the chemical oxygen demand (COD).

Some discharged sludges with high water content can be treated by dehydration resulting in volume decrease. Many types of inorganic or organic flocculants have been developed and utilized in many dehydration plants.

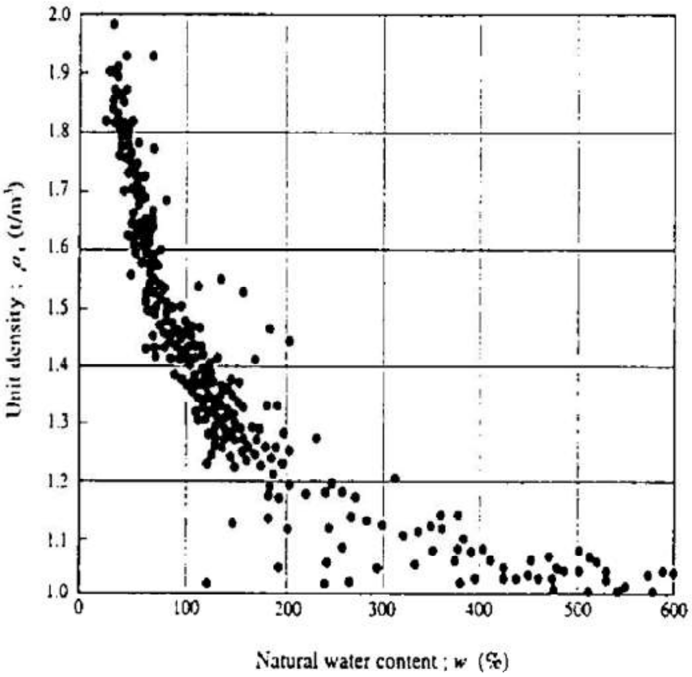


Figure 1. Physical properties of dredged sludges

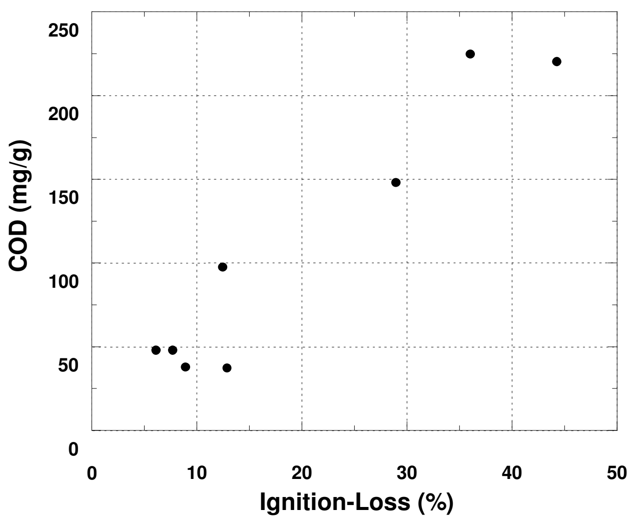


Figure 2. Chemical properties of dredged sludges

In addition to water content and organic matter, the grain-size distributions for dredged sludges typically correspond to those for clayey and silty soils. For example, ~ 90 % of the sludges from La Seine and La Loire estuaries consist of clay- and silt-sized particles (< 75 or 63 μm), whereas dredged materials in France are normally ~ 75 % fines, while the fines content of the dredged materials from Hamburg Harbor is only ~50 %. The grain-size distribution can not only influence the mechanical behavior (i.e. shear strength) but also control the interaction between the sediment and the pollutants. Thus, if the sediment is contaminated, the pollutants (i.e., heavy metals) are likely to concentrate in the fine fraction, as for a case in which 99 % of mercury was reported to be contained in particles less than 75 μm in size (Tsunoda, 1998). Nevertheless, many researchers have pointed out that contamination could be present over a wide range of grain sizes up to the sand sizes.

As remediation strategies, such as capping or natural burial for the contaminated sludge, develop, the strength and compressibility of the materials become important. There are a number of laboratory and in situ tests that have been developed for soft aquatic sludge, including the vane shear, static and cone penetration devices to measure the undrained shear strength.

Usually, characterization of the contamination of dredged materials focuses only on chemical pollution. However, due to sewage water from urban areas, these materials could contain another type of pollution, i.e. bacteria and virus, which may be transmitted to humans via shells and swimming. Alzieu et al. (1999) report that particle size, organic matter content, and temperature have a great influence on microbial contamination, quantitatively and qualitatively. Fine sediments that offer a high surface area for colonization can be considered as shelters for bacteria. While fixated on particles, bacteria appear to have more metabolic activities than those in a free form, and are responsible for degradation of a great part of organic matter and for production of 90 % of the carbon. Viruses are also abundant in the environment. In sediments, viruses are essentially adsorbed with cations (Na^+, Ca^{2+}) onto clay particles in suspension in water. In this state, they are relatively protected from external chemical attack. Furthermore, high pH enhances virus elution while with low pH viruses are more adsorbed. However, soluble organic matter competes with viruses for adsorption on clay particles. Dredging and disposal of dredged materials bring changes to the phenomena of dilution, resuspension, and sedimentation that affect the growing behavior of bacteria and viruses. Figure 3 describes the main interactions that occur in a dredged material among solid particles, water, suspended particles and inorganic contaminants.

4.3 DREDGING OPERATIONS

Operation systems are designed in such a way that no spillage occurs during dredging operations. Dredging technology exists that is capable of greatly reducing turbidity and resuspension during the dredging of bottom sediment; however, special equipment has to be deployed and modified operational methods must be used (Herbich, 1992 and 1995). It became obvious that dredging would be the most efficient and cost-effective way to cleanup contaminated sediment. A number of appropriate dredging methods have been developed, principally in Italy, the Netherlands and Japan. These include the Pneuma pump (available in Italy), the Oozer dredge, the Cleanup dredge and Refresher system (available in Japan), and the Dutch-designed "Matchbox-head" dredge (available in the United States). A combination dredge employing mechanical, hydraulic, and pneumatic concepts has been developed in Japan.

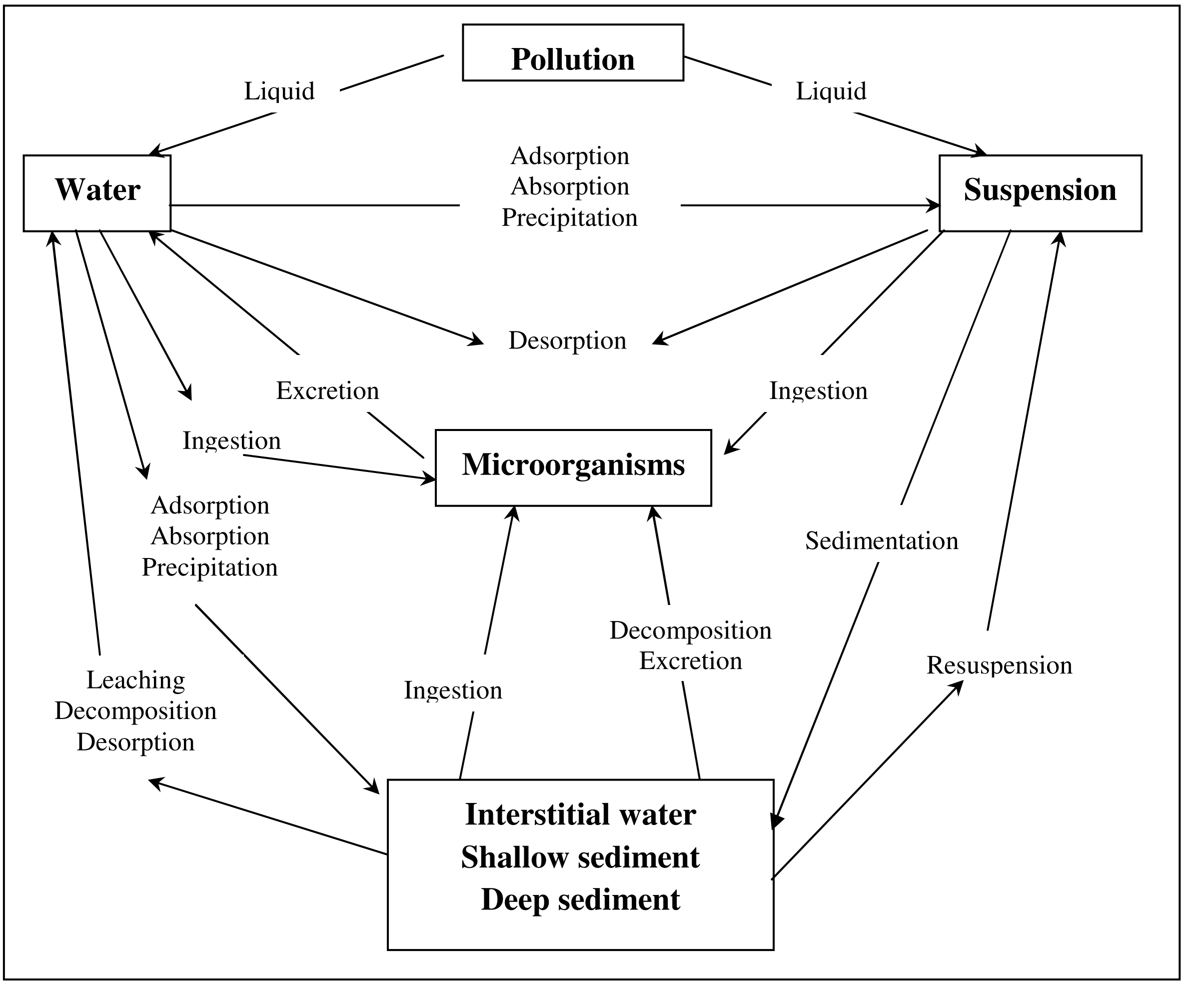


Figure 3. Behavior of heavy metals spilled in the water-sediment system (Robbe et al. 1985)

One example of the dredging machine "Cleanup System" is shown in Figure 4. It was developed for dredging highly contaminated sediment to reduce or minimize resuspension of the sediment (Sato 1984). The Cleanup head consists of a shielded auger that collects sediment when the dredge swings back and forth; the auger guides the sediment toward the suction of a submerged centrifugal pump. The auger is shielded and a movable wing covers the sediment as it is being collected by the auger. Cleanup dredges have been used for excavating soft mud and sand containing various contaminants such as mercury, cadmium, PCB, oily and organic substances.

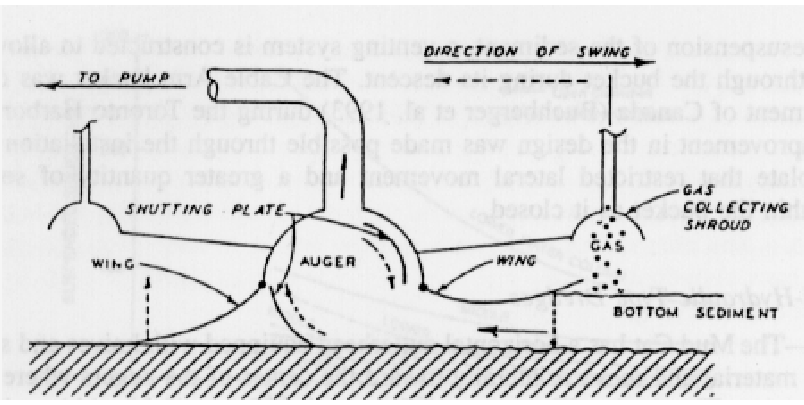


Figure 4. The "cleanup system" developed in Japan

Table 1 summarizes the dredging equipment capabilities in removal of contaminated sediment. The resuspension of sediments while dredging results in from 2 to 1600 mg/L total suspended solids (TSS). Consequently, the most important factors for preventing negative environmental impact include a low rate of sediment resuspension by dredging equipment to prevent secondary contamination and low water content in dredged sediment to reduce the volume of slurry to be treated. Since there are many sites where contaminated sediment will have to be removed, refinement in design of dredging equipment and removal methods is necessary.

4.4 DREDGING AND CLEAN-UP UNDERWATER MATERIALS

During the last two centuries, industrial activities generated pollutants substances that were discharged in air, water, soils and marine sediments. Data from the Environment Agency of Japan (1972), for example, indicate that sediments are much more contaminated than seawater. This contamination has a serious effect on fish, resulting in malformed fish and shells.

Because the concentration levels of contaminants are subjected to variations from place to place, and because of the degree of pollution that is dangerous for living organisms, definition of background levels may be the best way to estimate the dangerous level of hazardous substances (Fukue et al., 2000). Such background of contaminants in sediments can be determined from the profile of a heavy metal (e.g., copper) concentration in the different layers, as shown in Figure 5.

Table 1. Dredging capabilities used for removal of contaminated sediment

Type	Production	Depth limitation, m (ft)	Resuspension	Comments
Mechanical				
Open clamshell watertight	Low	9.1-12.2(30-40)	High	
Watertight clamshell bucket	Low	9.1-12.2(30-40)	Low	Experiments conducted in St-Johns River
Cable-arm bucket	Low	9.1-12.2(30-40)		Experiments conducted in Canada
Mechanical-Hydraulic				
"Mud Cat"	Moderate	4.6-7.6(15-25)	Low to moderate	Extensively used
"Mud Cat ENV"	Moderate		Low	Experiments conducted in Canada
Remotely controlled "Mud Cat"	Low	4.6 (15)	Low to moderate	New development
"Cleanup system"	Moderate	21.3 (70)	Low to moderate	Extensively used in Japan
Cutter head	Moderate to high	12.2 (40)	Low	Pilot study in New Bedford, Massachusetts
Hydraulic-Suction				
"Refresher"	Moderate to high	18.2-35.0(60-115)	Low	Extensively used in Japan
"Matchbox"	Moderate to high	25.9 (85)	Low to moderate	Experiments conducted at Calumet Harbor
"Wide Sweeper"	Moderate	30.5 (100)	Low	Used in Japan
Pneumatic				
"Pneuma"	Low to moderate	60.9 (200)	Low	Evaluated by USAE Waterways Exper. Station
"Oozer"	Moderate to high	18.0 (59)	Low	Used extensively in Japan
Mechanical-Hydraulic-Pneumatic				
Screw-impeller	Low to high	6.1 (20)	Low	Used in Japan (high density)
Airtight bucket wheel	Low to moderate	4.6 (15)	Low	Used in Japan (high density)

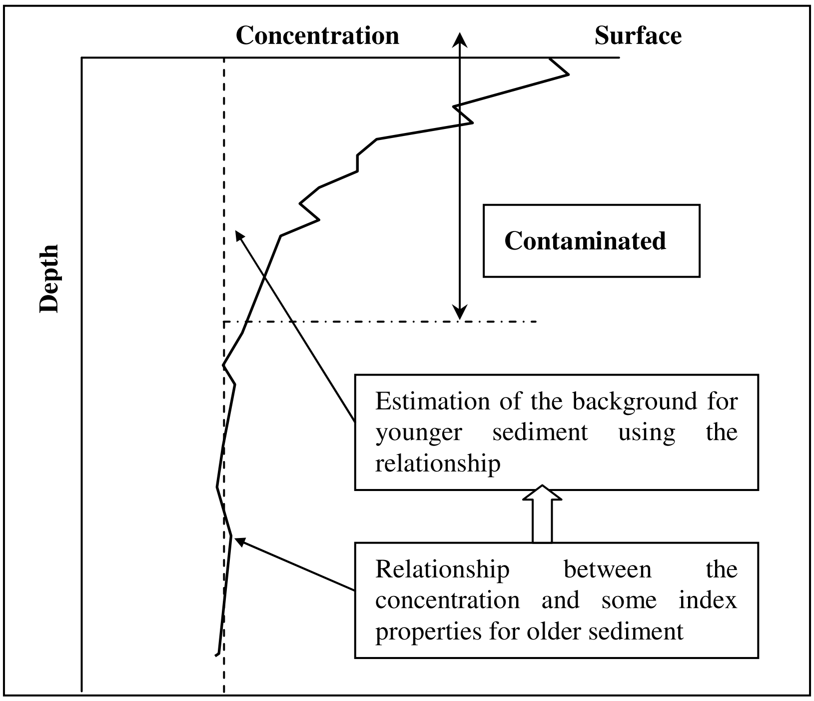


Figure 5. Concentration profile contaminated by a heavy metal and determination of the background

Since the background depends on soil type, it is possible to establish a relationship between the background and the soil properties such us grain size and Atterberg limits. If the background is well-known (significant number of sites and results), then the degree of pollution (P_d) can be defined by the following relation (Fukue, et al, 1999):

$$P_d = (C_i - B_g)/B_g \tag{1}$$

where C_i is the concentration of the element in the sediment and B_g is the background concentration.

The concentration profiles show also that the polluted thickness is usually limited to 1 m in the Japanese bays, with an average thickness of 0.5 m. In order to prevent living organisms from contacting the polluted sediment, several solutions may adopted. The first solution consists of dredging the polluted layer. However, one cannot just use any technique of dredging for this purpose. Grab dredging, for example, is preferred to pumping. The latter technique results in a large quantity of muddy water which cannot be treated properly. Another solution involves covering the polluted bottom sediments with a clean material. Nevertheless, the cover could be subjected to erosion and removal due to marine current action.

Considering dredging of polluted sediment, scarcity of land disposal prompts the reduction in volume and/or the reusing of polluted sediment and, at same time, the reduction of harmful potential. Utilization of dredged materials depends mainly on their geotechnical characteristics and their level of contamination. Clean dredged sediments can be used either as construction materials in the case of sand or as a clay liner for muddy sediment due to their high capacity of sorption. One method for the cleanup of contaminated sediment is solidification as described by Yamasaki et al. (1995) and Fukue et al. (2000). The objectives of the solidification method are (1) to decrease the volume and to solidify the high water content of the dredged sediment (water content of 243 to 279 %) and (2) to ensure a leachate water quality that satisfies the level of standards in Japan. This is achieved by adding agents to solidify the sediment and then extracting water and compressing the solidified sediment.

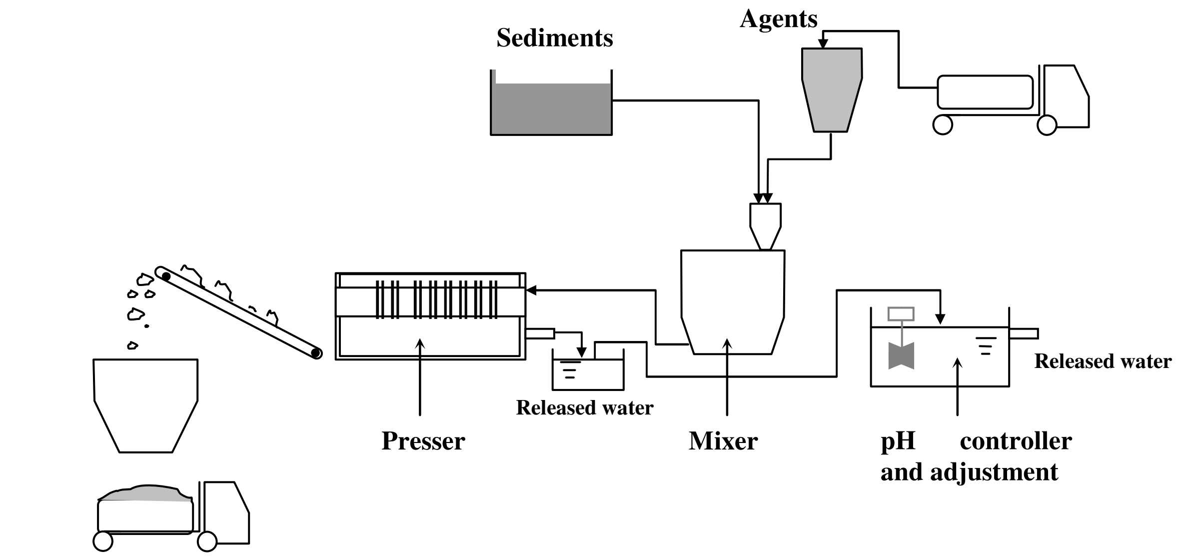


Figure 6. Scheme of the solidification plant (Yamasaki et al. 1995)

As shown in Figure 6, the solidification system consists mainly of a mixer, presser with water extractor and pH controller. For the solidification and mixing phase, several solidification agents mixed into the sediments were evaluated, including slaked lime (1.3 or 5% by wet weight), Portland cement (1.3 or 5%), a synthetic resin called UKC-H (1.3 or 5%), polyaluminum chloride (PAC) (0.5, 0.75 or 1.5%) and natural polymer called L-fresh (0.1%). The mixing was carried out using vanes that rotated for 10 min. In the next phase, the mix is brought into the press chamber where it is subjected to water extraction and compression under a filter pressure of 600 to 630 kPa. The treated sediment is then routed to site disposal or used as reclamation material. The performance of the technique was assessed on the basis of the following indicators:

1- decreasing rate of water content
2- compressive strength
3- water quality

The results are summarized in Table 2. After an elapsed time of 40 minutes, the lowest water content (38%) was obtained with the PAC and the highest for the polymer (L-fresh). The latter result is the effect of increasing the viscosity of the pore water that makes the water hard to extract. The results of compressive strength tests show that the best value (1.4 MPa) was obtained with the resin UKC-H that was designed to strengthen soft soil. Cement and slaked lime treated sediments indicated similar values of 400 kPa. Furthermore, although the mixture prepared with 1.5 PAC had the lowest water content, it was too soft to measure the compressive strength. As regards the water quality, the pH was either higher or lower than the original sediment pH of 6.3, depending on the type of added agent.

Therefore, lime, Portland cement and the resin UKC-H increased the pH to greater than 12, while the PAC and the polymer L-fresh decreased the pH to 5.5. Hence, the extracted water was neutralized and pH adjusted to meet the standard for discharge (7.2 to 7.7). The TSS and dissolved oxygen (DO) values were found to satisfy the standard for discharge, whereas the biological oxygen demand (BOD) was unsatisfactory resulting in the need for a second treatment.

Table 2. Results of the assessment of the treated sediment (Yamasaki et al. 1995)

Agent → / Properties ↓	Without agents	Slaked lime	Portland cement	Resin UKC-H	PAC	Polymer L-fresh
Adding level (%)	-	3.0	3.0	3.0	0.75	0.1
Poured volume (m^3)	0.69	1.04	1.01	1.03	0.969	0.633
Treated sediment						
Water content (%)	82	45	44.8	48.1	37.8	77.0
UCS (kPa)	-	400	450	1420	-	-
pH	6.3	11.2	11.8	11.4	7.14	7.2
Density (g/cm^3)	1.39	1.73	1.7	1.73	1.81	1.54
Leachate water						
Leachate pH	6.3	12.6	12.6	12.0	5.4	5.6
Neutralized pH	7.2	7.3	7.7	7.3	5.4	7.3
SS (mg/l)	5.0	6.3	6.0	6.3	5.6	6.3
DO (mg/l)	8.5	6.3	7.5	6.3	5.6	6.3
BOD (mg/l)	15.7	16.7	16.2	16.4	16.2	16.4

Concerning the mechanisms and efficiency of cement-based solidification towards heavy metals, immobilization occurs principally by two means: physical and chemical. Physical immobilization results from the formation of a solid matrix while chemical stabilization refers to changes in the chemical forms of contaminants from less stable and more soluble forms to more stable and less soluble forms. The major fixation mechanisms include 1) hydroxide precipitation 2) substitution reactions and incorporation of metals in the hydrated products and 3) adsorption. The major parameters that control these mechanisms are pH, redox potential and pore-water composition. When many amphoteric heavy metals are present in sludge (e.g., Cd and Cr(III) are less soluble at different pH, respectively, 11-11.5 and 8.5), adjustment of pH (by addition of cement for example) may not achieve an efficient immobilization. The degree of stabilization depends also on the form of the metal in the cement matrix. For example, mercury which is bound to the crystalline structure of cement and substituted to Al and SO_4 should not be easily released, whereas precipitated hydroxide mercury should be more sensitive to the external environmental conditions such as pH. On the other hand, the *USEPA (1990)* indicated that, although hydroxides of cadmium and lead have comparable and weak solubilities, leaching tests from solidified/stabilized wastes by cement showed different stabilization degrees, i.e., Cd was more stabilized than Pb. The tentative explanation for this behavior is that the cement/Cd system enhances rapid formation of $Cd(OH)_2$ that is incorporated in the nucleation site of CSH and, thus, is protected with an impermeable envelope. As for Pb, precipitation reactions are complicated such that precipitants are in form of salts containing hydroxide, sulfate and nitrate ions, and these salts retard cement hydration by forming an envelope around anhydrous cement grains. As the pH of pore-water solution varies during hydration, Pb salts undergo solubilization and precipitation that result in their precipitation on the surface of hydrated cement particles and, therefore, become more accessible for leaching.

4.5 SPILLAGE WATER TREATMENT

Spillage water from highly organic sludge contains elevated concentrations of ammonia and soluble BOD and chemical oxygen demand (COD) substances that result from the anaerobic decomposition of organic matter in the bottom sediments and concentrate in the pore water. Changes in flow rate, in water quality and in water temperature make biological treatment hard to realize. Thus, Tsunoda (1998) reports a physico-chemical treatment which is described hereafter as an alternative. To decompose $N\text{-}NH_4$, sodium hypochlorite is used and residual Cl_2 is measured to control the efficiency of the treatment. According to the results shown in Figure 7, NH_4-N decreases rapidly with an increase in sodium hypochlorite concentration and variation in residual Cl_2 indicates a break point at which Cl_2 increases abruptly. The method using this control point in chlorine treatment is known as a break point method. Before releasing the treated water, residual chlorine is removed by activated carbon adsorption.

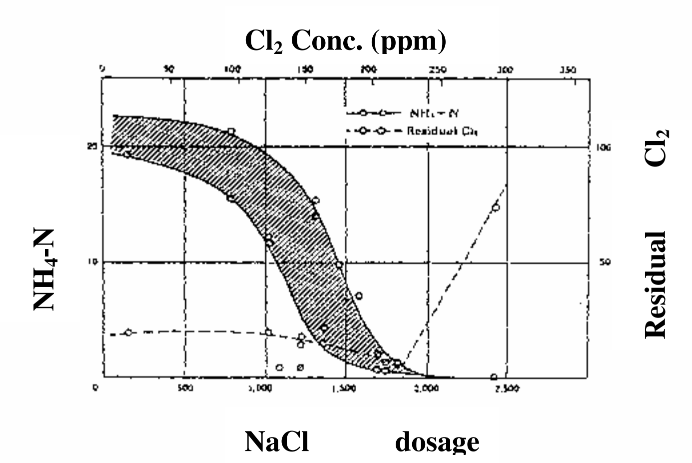


Figure 7. Breakpoint chlorination (Tsunoda, 1998)

4.6 ODOR CONTROL

Under anaerobic conditions, organic sludge produces malodorous substances such as hydrogen sulfide and ammonia. Tsunoda (1998) indicated that this secondary pollution can be treated with ferric chloride and slaked lime as a neutralizer. The suggested method is based on the relationships between the total sulfide content of the sludge (related to the malodorous compounds) and the required amount of ferric chloride and between the amount of added ferric chloride and the quantity of slaked lime to neutralize the former. Figures 8 and 9 show these relationships.

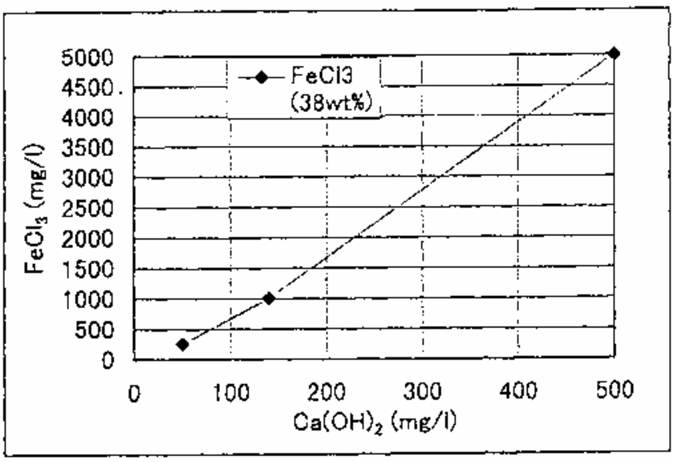


Figure 8. Amount of ferric chloride in relation to the total sulfide content.

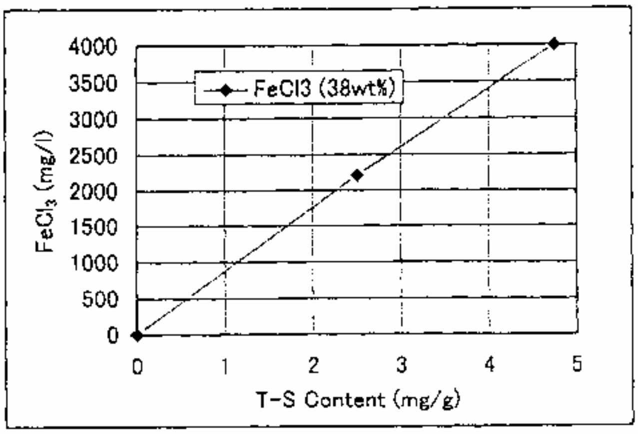


Figure 9. Amount of lime required for neutralization of ferric chloride.

4.7 CONTAINMENT AND ISOLATION

4.7.1 Confined disposal facility

The regulatory requirements for the disposal of dredged material are determined by both the type and level of the contaminants associated with the dredged material. Based on the degree of contaminant partitioning to the water associated with the sludge, there are three conceptual categories under which sludge disposal can occur with associated disposal regulations. These three approaches to sludge disposal are labeled "beneficial use or open water disposal," "solids retention," and "hydraulic isolation." The confined disposal facility (CDF) design criteria based on contaminated level and pathway is shown in Figure 10 (Richardson et al. 1995).

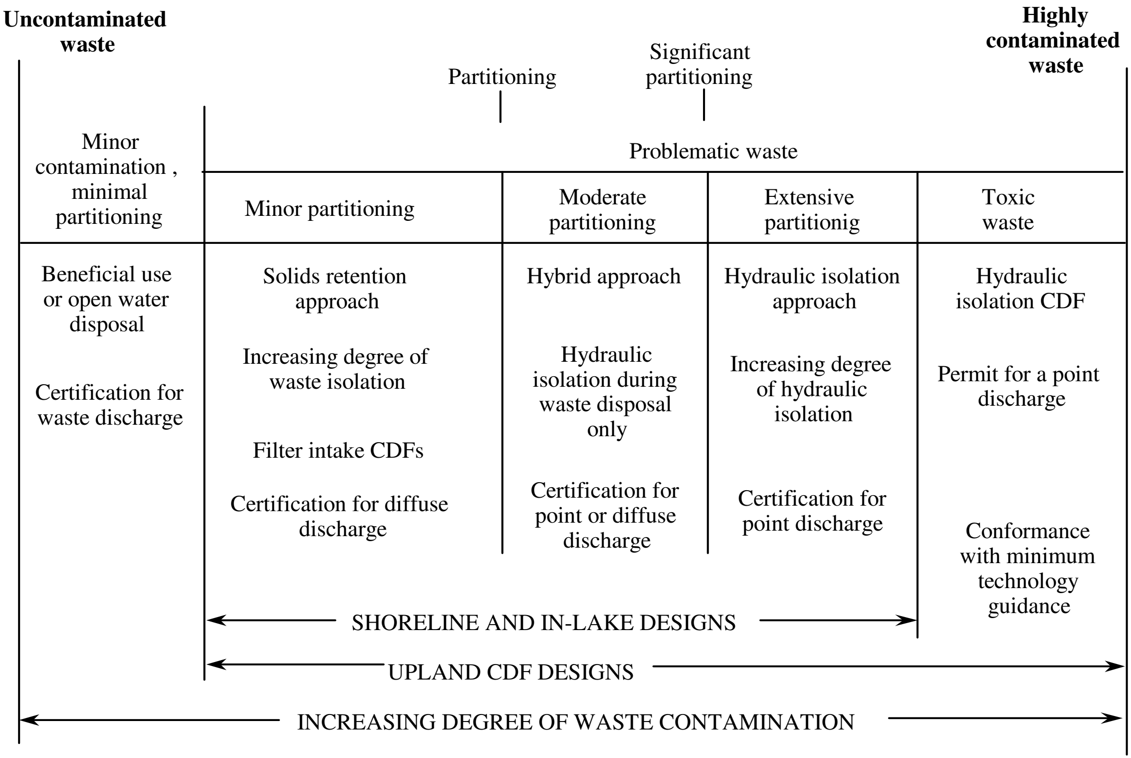


Figure 10. CDF design criteria based on contaminant level and pathway

The CDF should be used for containing contaminated sludge that cannot be released without control to the environment. CDFs can be located at both upland and in-lake sites depending on the level of isolation that the sludge under consideration warrants. Contaminants within the CDF can be discharged to the environment via six potential pathways (Richardson et al. 1995). These pathways are shown in Figure 11 and include three waterborne pathways, two pathways related to the direct uptake of the contaminants by plants or animals, and an airborne emission of contaminants.

The methods for limiting contaminant pathways from CDFs for problematic dredged materials include the addition of engineered barrier and/or water-balance components in the dikes, basin, and cover. The operational alternatives for establishing the pathway barriers are also important. Thus, the basin of the CDF could be lined using and engineered compacted clay liner (CCL) or it could be sealed by placing an initial layer of clean fine-grain dredged material in the CDF. Either barrier layer could be effective in limiting the movement of leachate from the dredged material into the ground water beneath the CDF.

A highly watertight revetment (CDF) was used in order to contain dredged materials contaminated by methyl mercury (Yoshinaga, 1995). About 1,000,000 m^3 of contaminated sludge were removed from Minamata Bay (Japan) using suction dredges, and they were disposed along the shoreline area. After about ten years, the effort to remediate the mercury contaminated sludge that caused the Minamata Disease was completed in 1990 with the final surface covering by soils, and no outbreak of pollution or related disease has occurred since then.

In the area of seabed designated for contaminated mud disposal, the sand capping method where the bottom mud is covered with a thin sand layer is applied as a solution of environmental preservation. Brand et al. (1994) reported a typical section of the contaminated mud pit (CMP). The pits are dredged as deep as can be readily achieved. For example, in Hong Kong, this means that the pits are dredged to the base of soft Holocene (post-glacial) marine deposits, commonly about 15 m below the seabed. In order to prevent diffusion of contaminants to the open area, a 2-m-thick mud cap was chosen. To facilitate placement of the mud cap onto the semi-fluid contaminated mud, a 1-m-thick layer of sand is first sprinkled onto the surface of the contaminated mud. This sand sinks differentially into the surface, and so thickens and strengthens the mud surface prior to placement of the clean capping mud. The full cap design, therefore, comprises a 1-mr-thick sand layer followed by 2 m or more of uncontaminated mud to reinstate the seabed to its original level. Gomyoh et al. (1994) also investigated the shear strength of typical bottom mud measured by vane shear and cone penetration tests together with the procedure for measuring the thickness of the thin sand layer for capping.

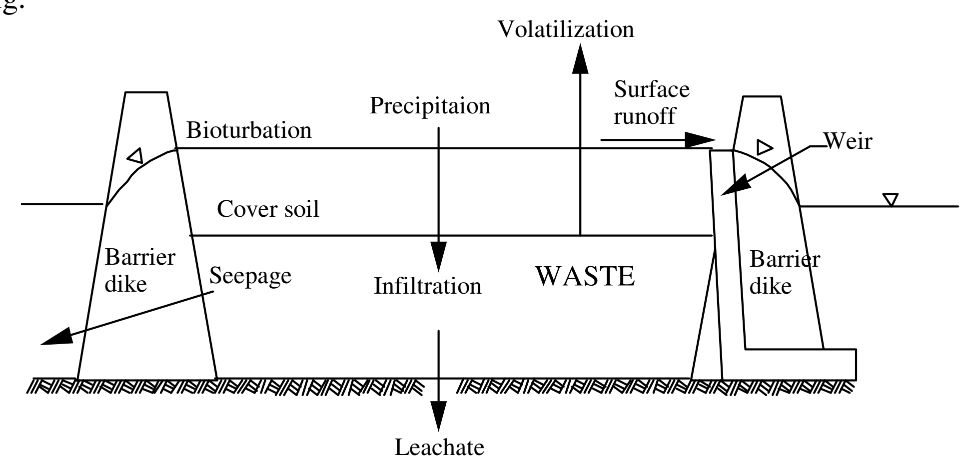


Figure 11. Example of the contaminant pathway

4.7.2 Sludge containment structure

Waste disposal facilities are facing various difficulties such as increasing volume of wastes and inherent problems (noise transportation, bad smell, dust, and potential pollution risk for underground water). Disposal sites in sea areas are becoming more important to discharge hazardous wastes. Concerning the containment structure, the barrier used to prevent hazardous waste leaching into the open sea, is made more safely and must be doubled based on failsafe concept. The impermeable soil layer in the sea is defined as "clay layer of more than 5 m thickness whose permeability is less than 10^{-5} cm/s". Tsuchida et al. (2000) reported the design and the construction of a new seawall structure which has been constructed in Tachibana Port and which complies with the new revised standard. The related disposal site has a capacity of 630,000 m^3 including domestic wastes (290,000 m^3), industrial wastes (178,000 m^3), waste soil (90,000 m^3) and dredging (72,000 m^3). In this project, a seawall of 600-m length was constructed and the barrier consists of polyvinyl chloride (PVC) sheets and cement treated soil. The cross section of the seawall and the barrier is shown in Figure 12.

Using sand compaction piles to improve the clay will result in an increase in the permeability. This increase was numerically assessed by means of analyses based on the boundary element method. It was found that, in order to insure a permeability less than 10^{-5} cm/s and considering the construction practice defects, the sand replacement ration (defined as an area ratio) should be less than 50 %. The aim of the utilization of cement treated soil was to relieve the stress on the PVC sheet and to prevent leakage in case of damage of sheet. Consequently, the treated soils should have ductile stress-strain properties, and the minimum shear strength necessary to keep the designed shape and lower permeability. Results of consolidation and permeability tests show that, for the same consolidation pressure, the permeability of treated soil was greater than that of the untreated soil. On the other hand, for the same void ratio, the permeability of the treated soil was lower than that of the untreated soil. Hydrated products are assumed to clog the voids and, thus, prevent water flow. The role played by the cement treated soil became more evident when seepage through the barrier was analyzed (modeled) and compared to the case of utilization of a sand layer. Overall, the analyzed (predicted) seepage of water was significantly greater in case of the sand layer (1100 m^2/year) relative to the case of treated soil only (0.943 m^2/year). The barrier composed of two PVC sheets and an intermediate cement treated soil with a seepage of 0.241 m^2/year was considered to be safe towards leakage.

Based on finite element analysis, the settlement and the deformation of clay layer were investigated. Results indicate that the PVC sheets and the cement treated soil will have maximum deformations about 2 and 0.7 % respectively. Accordingly, it is considered that PVC sheets and cement treated soil will be stable in the long term.

In terms of the construction of underwater works, the new technique of on-ship heat bonding the PVC sheets is noteworthy, which is completed prior to laying the PVC sheets in a single piece on the seawall. Cement treated soil should be stable on slopes of from 1:3 to 1:8 and should possess non brittle but deformable characteristics. According to the shear strength results, the initial water content of the soil was determined to be about $1.4w_L$ and the cement content is 50 kg/m^3. The total volume of treated soil was 63,000 m^3 and the cement treatment was carried out on a special vessel with a 250 m^3/h capacity.

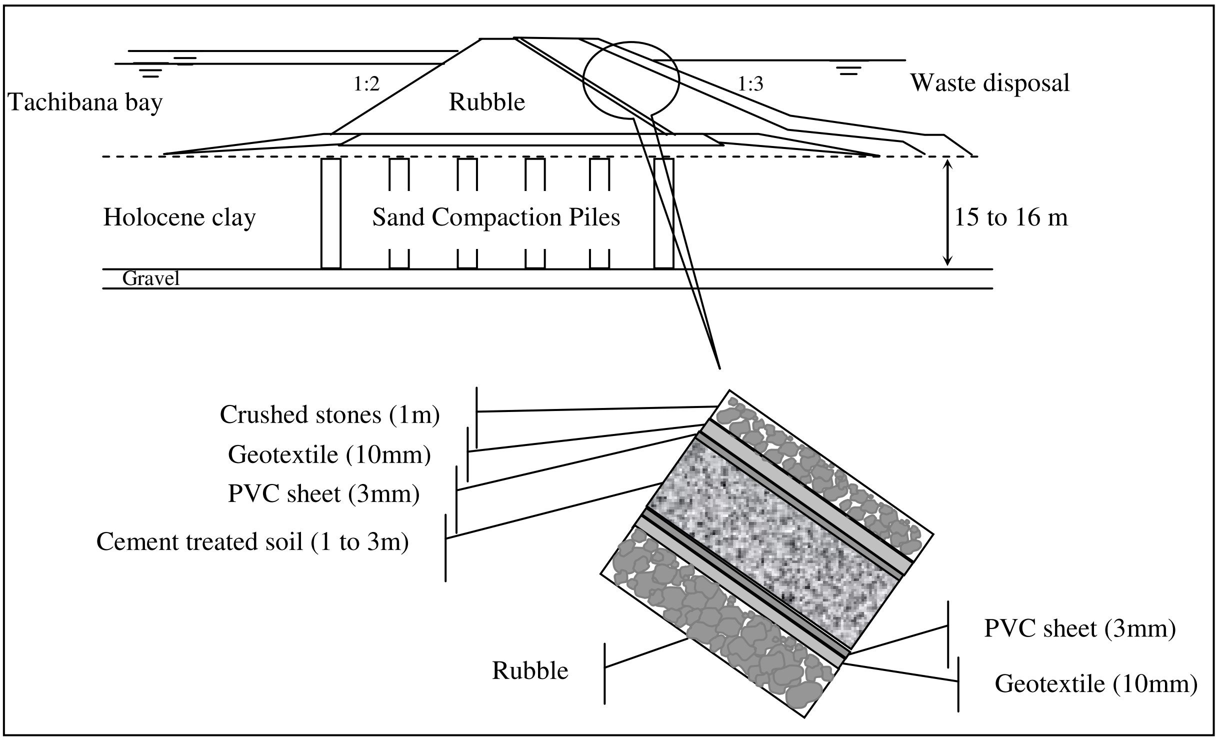


Figure 12. Cross section of the seawall and the barrier.

4.7.3 Environmental impact from CDF

The main issue with dredged materials pertains to the environmental impact when dealing with dredging and reclamation. In terms of the nitrogen and phosphorus present in the sediments, Tsunoda (1998) noticed that the dredging process has an effect on the release of these two species to water. The given example indicates that, after dredging and assuming that total nitrogen content is 2-2.5 g/kg and the release rate is in proportion to nitrogen content in the bottom sediment, the released nitrogen will be reduced by ~40 % by removal of sediment. As for phosphorus, the amount released by dredging was estimated to be ~ 50 % assuming phosphorus content of 0.5-0.9 g/kg. However, aerobic conditions enhance soluble phosphorus such that the released amount increased to ~90 %.

An in-situ treatment of contaminated sediment consists of a sand covering or capping where the contaminated layer is covered with a 30 to 50 cm layer of clean sand. Hence, the release of COD substances, such as nitrogen and phosphorus, is decreased by the chemical and biological processes that take place in the pores of the sand layer, as shown in Figure 13. Experiments on capping show that, immediately after application of a sand layer, reduction of about 80-90 % in the amount of N and P released was observed. However, although the sand layer seemed to be effective over a period of 6 years, the amount of released phosphorus appears to increase, probably due to new sedimented layers.

In the case of heavy metals release, Kamon et al. (2000) conducted an experimental study on heavy metal leaching from Zn contaminated clay slurry that simulates the dredged sediment characterized in Table 3. In this study, two types of leaching tests were performed: batch leaching test (BLT) and consolidation leaching test (CLT). The first test aims at determining the adsorbed mass of Zn on the clay, while the second one was proposed to reveal the leaching mechanism in the dehydration and consolidation processes.

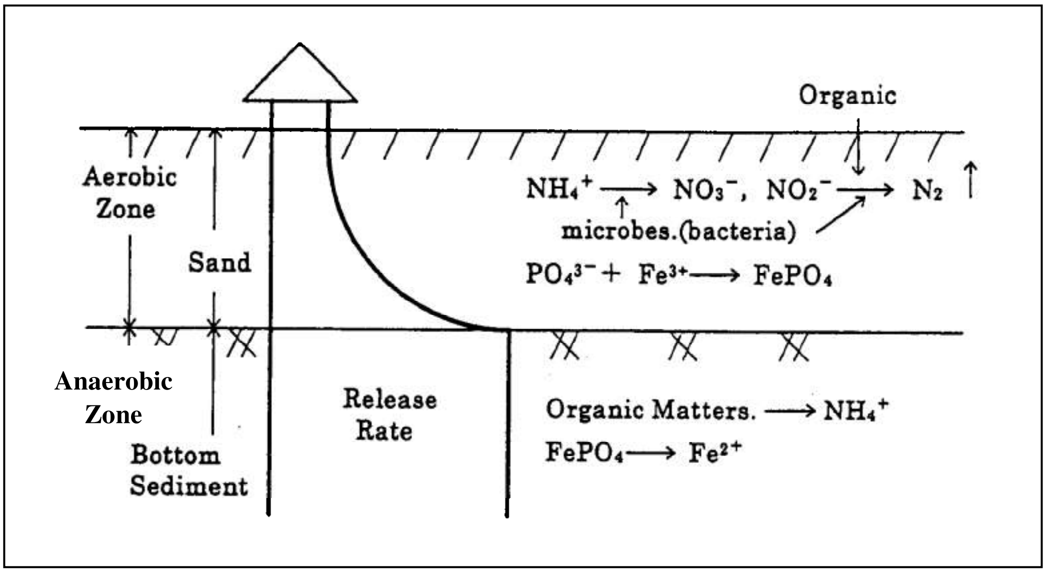


Figure 13. Effect of sand capping on N and P release

According to the results from the batch leaching tests (BLT), Zn adsorption appeared to be dependent on the liquid-to-solid (L/S) ratio. The greater the L/S ratio, the higher the concentration of adsorbed Zn and the higher the equilibrium concentration (Figure 14).

To explain the involved immobilization mechanism, Kamon et al. (2000) suggested cation exchange as the dominant mechanism. Calcium and magnesium ions, which initially adsorbed to the clay particles, have been changed by Zn^{2+} in water. Formation of insoluble compounds (carbonate, hydroxide) and adsorption onto organic matter are unlikely to occur.

By adding several amounts of Zn to the clay slurry and measuring the adsorbed Zn, a relationship between to the two amounts was established. Zn adsorption is dependent on Zn addition according to the following relationship:

$$M_A = 0.403 * M_D^{0.649} \qquad (2)$$

where M_A is adsorbed Zn on 1 g soil (mg/g) and M_D is doped Zn per 1 g soil (mg/g). In general, 10 % of dosed Zn is adsorbed on clay and 90 % remains in water.

Table 3. Properties of Fukakusa clay (under 75 μm)

Solid density	2.72 g/cm^3
Liquid limit	47.4%
Plastic limit	26.0 %
Grain size	
Silt fraction (5-75 μm)	38.2 %
Clay fraction (< 5 μm)	61.8 %
CEC	23.6 meq/100g
Exchangeable cations	
Na	0.01 meq/100g
Mg	13.5 meq/100g
Al	1.4 meq/100g
K	0.06 meq/100g
Ca	8.7 meq/100g

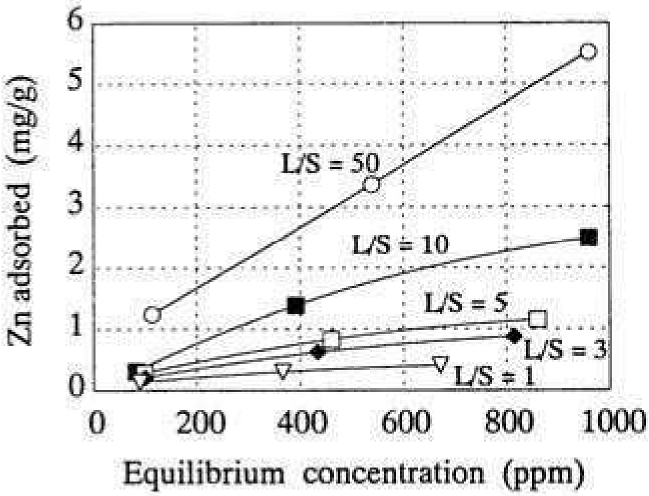


Figure 14. Zinc (Zn) adsorption versus equilibrium concentration for different liquid-to-solids (L/S) ratios based on batch leaching tests (BLT).

The consolidation leaching tests (CLT) were run on Zn doped clay at 100 % water content, using an oedometer cell with vertical pressure ranging from 1.48 to 90.16 kPa. Results, shown in Figure 15, indicate that leaching concentrations are almost equal to equilibrium concentrations and so for the adsorbed Zn amount. This phenomenon could be explained by the fact that the equilibrium constant K should be kept constant by increasing adsorbed Zn concentration while the exchangeable ions concentration are extracted from soil-water system with the drained water. Furthermore, the concentrations of leached Zn appear to be the same at the beginning and at the end of the consolidation. These results suggest that the leached and adsorbed amount of Zn from the consolidation process can be predicted from the batch leaching test with the same value of L/S as in the initial condition prior to the consolidation.

It is of some importance to estimate the mass of leached heavy metal during the successive steps of treatment such as: dredging, dehydration, reclamation and consequent consolidation. During all these steps, the water content decreases, and the concentration of heavy metal remains the same as after dredging. According to the experimental results, dehydration has no effect on the concentration of leached heavy metals.

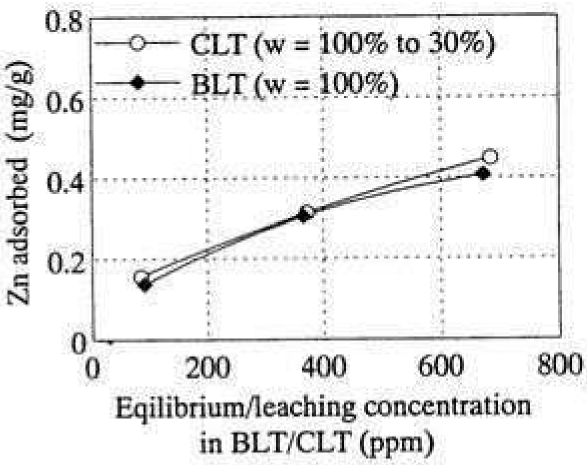


Figure 15. Zn adsorption in CLT and BLT

As for the mass of heavy metals, it decreases after dehydration and reclamation because some amount of heavy metals are drained out with water. Based on this process (Figure 16), the estimated mass of leached heavy metal can be expressed as:

$$M_L = M_2 - M_3 = [(w_2 - w_3) * w_0/w_1] * C_0 \tag{3}$$

where M is mass of heavy metal, w is water content and C is concentration of heavy metal. The meaning of suffix is as follows; 0: initial, 1: dredging, 2: dehydration, 3: reclamation.

Figure 17 shows the results of calculated mass of leached heavy metal. It can be seen that the last amount is related to both water content after dredging and water content achieved by dehydration. To achieve the same mass of leached heavy metal, for example 10 mg/kg dry soil, the dredged sediment has to be dehydrated to low water content of 40 % if the water content after dredging is only 100 %, while in case of water content after dredging of 500 %, the sediment has to be dehydrated at a water content of only 80 %. Therefore, to reduce the mass of leached heavy metal, either an effort should be made to dredge the sediment with a relatively low water content and also to achieve a low water content after dehydration, or the sediment should be dredged at a high water content. However, this latter option may be technically unsuitable.

The aforementioned results are valid under the following assumptions: results were obtained for Zn only and effects of chemical conditions (Eh, pH) and sediment components (organic matters) were not taken into account. Furthermore, interaction between heavy metals (e.g., only ion exchange mechanism is considered) and treatment cost of contaminated dehydrated water was omitted.

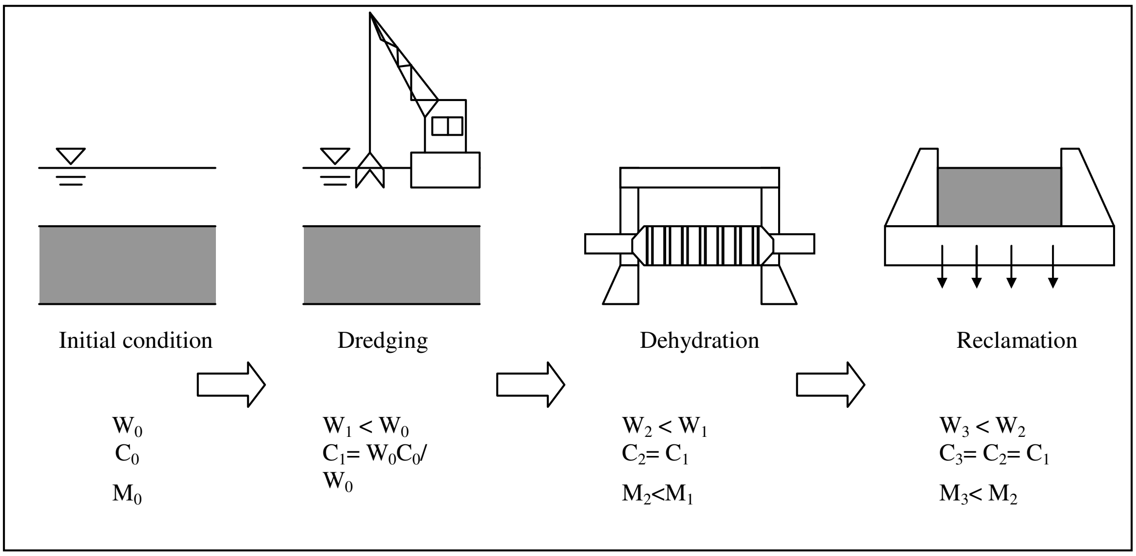


Figure 16. Mass of heavy metal existing in the sediments during dredging, dehydration and reclamation.

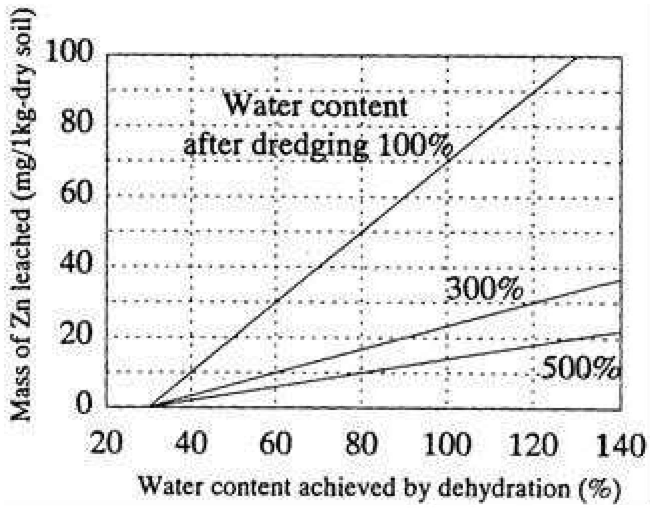


Figure 17. Calculated mass of leached heavy metal after reclamation.

4.8 REMEDIATION OF CONTAMINATED SLUDGE

Soils and sediments in Japan contaminated with polychlorinated biphenyls (PCBs) have been treated primarily by the addition of cement and disposed in landfills. However, the landfill sites containing the treated sediments must be maintained safe for a long-term period and the environmental impact has to be monitored. Hosomi et al. (2000) reported innovative technologies to treat and to decompose PCB and dioxin contaminated soils and sediments.

For example, a technology referred to as Gas-phase Thermal Reduction was used by Eco logic International (1992) to treat sediment samples. The sediment was pretreated in a 600°C thermal desorption unit prior to injection into the gas-phase thermal reduction chamber at 900°C. Efficiency results (Table 4), based on dioxins and furans analysis before treatment and in the residual solid, show that the thermal desorption was not very efficient in reducing the concentrations of 2,3,7,8-TCCD and total dioxins in the feed sample.

Solvent extraction has been used in a bench-scale test to treat dioxin-contaminated sediment. This technology, which is referred to as BEST (Basic Extractive Sludge Treatment), uses triethylamine to extract the contaminants. A sequential extraction process was used with 6 extractions. Results show that the efficiency is better than for gas-phase thermal reduction. It should be noted that the extraction removes the contaminant and concentrates them in a small volume of liquid (oil) which should be treated subsequently.

Base-Catalyzed Decomposition (BCD) mixes the dioxin-contaminated sediment with two mixtures. The first mixture is comprised of water, sodium hydroxide, tetraethylene glycol and a proprietary catalyst. This sediment mixture is heated to 320-345°C for 90 minutes. The second mixture contains sodium hydroxide, high boiling point hydrocarbon oil and USEPA propriety catalysts, and is heated to same temperature for a period of 4 h. Both mixtures result in a non-detectable level of dioxin in the treated residues (Table 4).

Small-scale incineration tests were performed on dioxin-contaminated sediment. The destruction and removal efficiency were calculated by combining sediment feed rates and flue gas discharge flow rates with the reported sediment and flue gas concentrations. Removal efficiency results indicated very high percentages. However, due to the low concentrations of dioxins in the feed sample, the measured flue gas was found to be typical for the incineration of any wastes containing chlorine compounds.

For the above-mentioned treatment technologies, the based-catalyzed dechlorination and incineration are the most efficient in destroying or reducing dioxins to below detection limits.

The BCD process was developed by the USEPA to chemically decompose PCBs in a liquid-liquid phase reactor (Rogers et al. 1991). The process consists of desorption and recovery of PCBs in a soil reactor (300-350°C) and the decomposition of recovered PCBs in the liquid phase reactor and removal efficiency of PCBs and dioxins from the PCB-contaminated bottom sediments and the dioxin-contaminated soils by heated soil reactor.

Table 4. Results from bench-scale tests of dioxin-contaminated sediments (Hosomi et al. 2000).

Process	2,3,7,8-TCDD (pptr)			Total TCDD (pptr)		
	Feed	Product	Efficiency (%)	Feed	Product	Efficiency (%)
Gas-Phase	159	99.7	37.3	193	144	25.4
Thermal Reduction	130[1]	50[1]	61.5	130[1]	50[1]	61.5
Solvent Extraction	119	16.5	86.1	136	21.3	84.3
B.E.S.T	217[1]	15[1]	93.1	276[1]	15[1]	94.6
Based-Catalyzed	268[4]	<1.9[2]	>99.3	309[4]	<11.1	>96.4
Dechlorination 1	67.5[1]	<0.996[1]	>98.5	83.7[1]	<0.301[1]	>99.6
Based-Catalyzed	268	<33.1	>87.7	309	<47.8	>84.5
Dechlorination 1	67.5[1]	11.0[1]	>83.7	83.7[1]	<11.0[1]	>86.9
Incineration	18.5	<0.61[2]	>96.7	252	<1.2[2]	>99.5

1: Analysis performed by technology developer
2: Not detected, reported as analysis detection limit
3: % efficiency = (Feed Con. – Product Conc.)*100/Feed Conc.
4: Average of 2 samples.

4.9 BENEFICIAL USES OF DREDGED MATERIALS

4.9.1 Case study of land use 1: Dewatering method (horizontal drainage and floating PBD)

A new accelerated consolidation method combining the dewatering and Plastic board Drain by Floating system (PDF) is described by Kiyama et al. (2000). This method was designed to improve the consolidation process of dredged clay disposed at a reclamation coastal site without using pre-loading weights. The PDF system provides the possibility to increase the volume of dredged clay that can be accommodated at the site, thereby extending its service life. The principal of this method is based on increasing the effective stress in the case of the use of dewatering combined with vertical drains, as shown in Figure 18. Therefore, the degree of consolidation settlement is greater than in the case of the self-weight process.

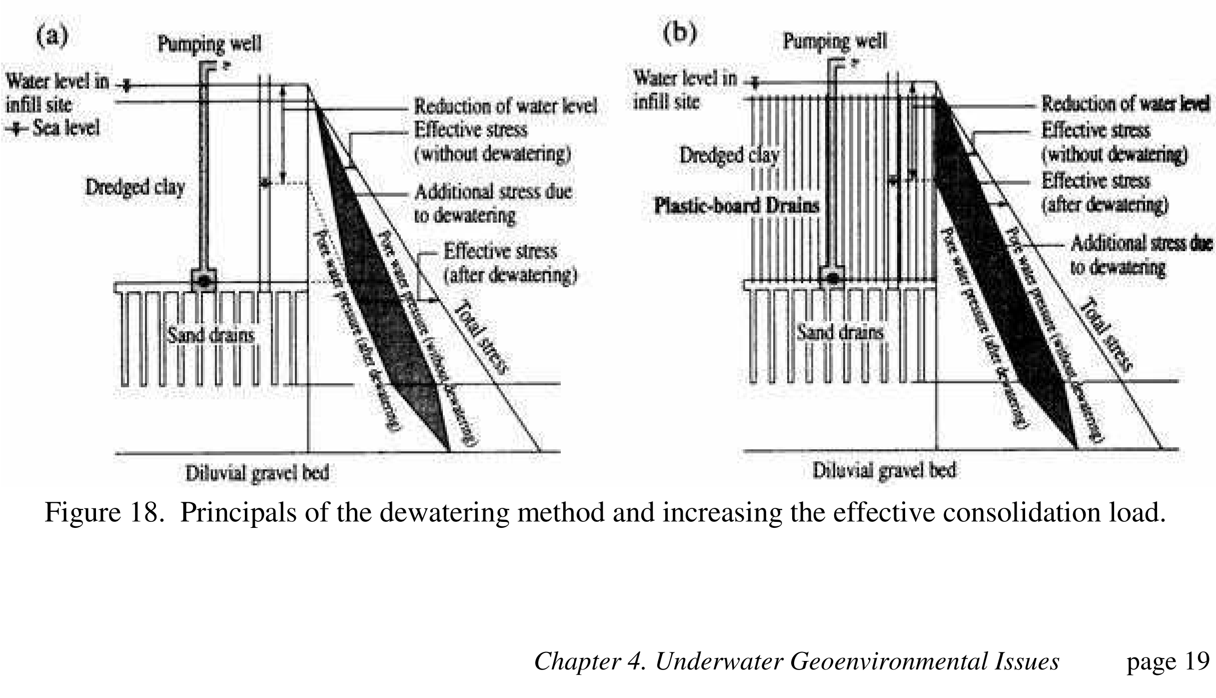


Figure 18. Principals of the dewatering method and increasing the effective consolidation load.

As for the implementation of the installation, the plastic board drains were installed through the dredged clay using a pile driving device which is assembled on a new designed floating vessel (Figure 19). The advantage of this system is its possible utilization on muddy ground or through shallow water which has proved difficult in the past. The implementation steps are as follows:

1- The vessel is fixed in position using 4 winches.
2- The equipment is positioned for drain installation.
3- A vertical drain is installed: penetration, withdrawal and cutting-off.
4- The installation is shifted laterally and repositioned for the next drain installation. Steps 3 and 4 are repeated.
5- Then, the vessel is repositioned.
6- After installing plastic drains, the pumps were set up in the sand mat.

It has to be noticed that the plastic board drains must reach the sand mat to create continuity to the water flow that is draw down by the plastic board. The water is than drawn out from the sand mat by activating pumps. The flow water diagram is shown in Figure 20. Furthermore, the drainage channels at the top of the drains must be sealed so as to prevent the seawater permeating the upper layers of the dredged clay and penetrating the drains.

Laboratory tests including consolidation properties (settlement and pore pressure) were carried out on samples taken from the dredged clay in the improved area by plastic drains and also in the unimproved ground (Figure 21). Monitoring results of the settlement show that, after 8 months of activating the drainage, ground improvement was evident. Indeed, more than 4 m of consolidation settlement was recorded in the improved area, while only about 50 cm of settlement was observed in the unimproved ground. It can be seen that the calculated settlement values match the average of measured values even if they are slightly higher. If we focus on the point T-11, we can note that the curve of this point is the most representative of the improvement brought by the plastic drains. Point T-11 is in the center of the implementation area (Figure 22) and is less affected by the boundary with the unimproved ground than the other points.

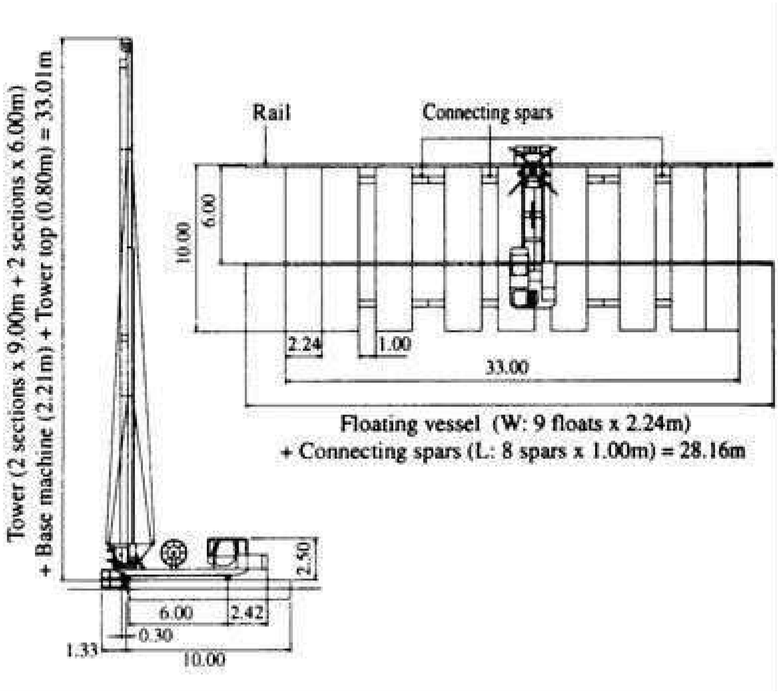


Figure 19. PDF implementation assembly.

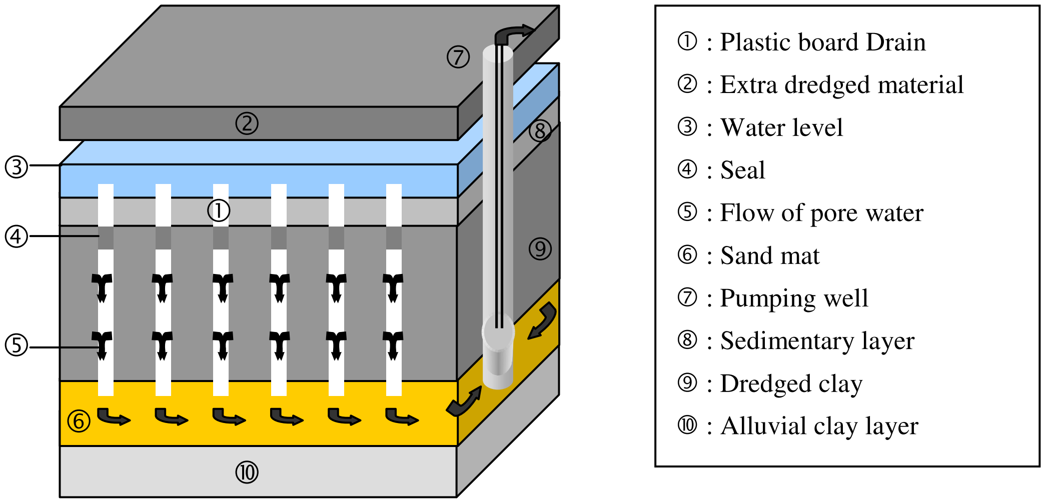


Figure 20. Ground Improvement concept diagram.

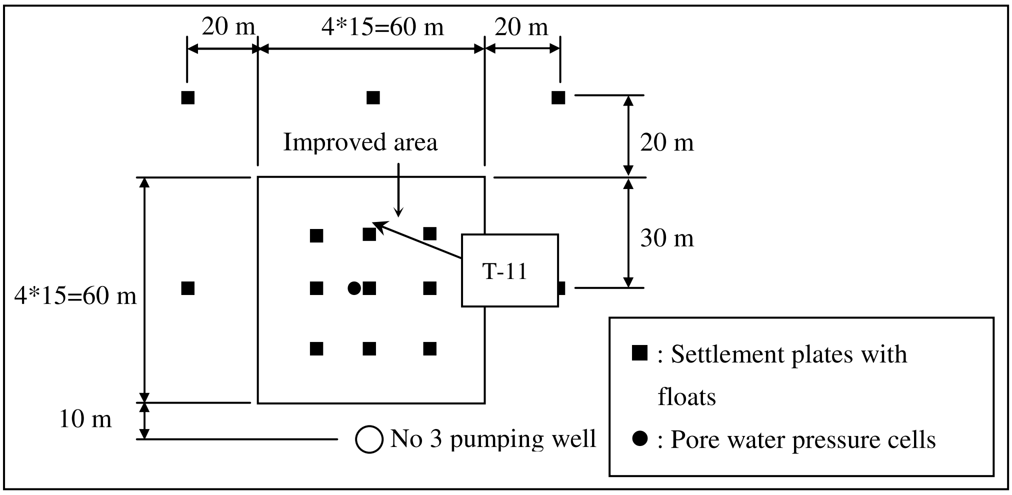


Figure 21. Site tests survey plan.

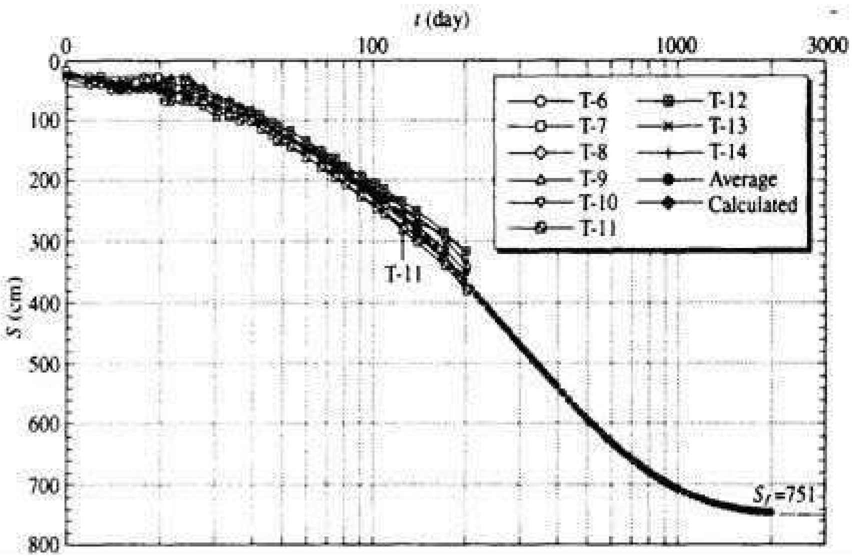


Figure 22. Measured and calculated settlement.

4.9.2 Case study of land use 2: Ketelmeer Project

A large-scale dredging project in the Ketelmeer Lake in the Netherlands was undertaken (IADC. 1999). It involves the construction within the Ketelmeer of a storage depot for contaminated silt dredged from the lake itself. The Ketelmeer, which is located at the mouth of the River IJssel, has become polluted over recent decades as a result of discharges from factories along the Rhine and IJssel rivers. A large number of pollutants have entered the river. These substances have attached themselves to the silt which is carried downstream by the river, and have ultimately ended up in the Ketelmeer. The result is that, of the total of 3,800 hectares of the lake bed, 2,800 hectares are covered in a layer of highly polluted silt 50 cm thick. The clean-up project must be carried out in accordance with strict environmental standards. It was decided to construct a depot in the lake itself, primarily in order to avoid high transport costs. In the centre of the Ketelmeer, a 45-m-deep circular basin is being constructed, measuring more than one km across at the water line. The surrounding ring dike will stand 10 m above sea level, as measured by 'Normal Amsterdam Level' (NAP). A circular shape of the depot was adopted because the surface contact between the polluted silt and the surroundings is minimized. The result will be a basin with a capacity to store 23 million m^3 of polluted silt. The construction phases are illustrated in Figure 23.

Before construction of the depot started, a sheet piled wall was built along the boundary of the working area, over a length of 8.5 km, to prevent cleaned areas within the construction site from being recontaminated by polluted water and silt from the Ketelmeer. With the piling complete, the contaminated silt from the enclosed area – a total of 2 million m^3 – was removed and then stored in the temporary depots.

Special environmental dredgers have been built for cleaning up the construction site. The latest technology and most advanced software have been incorporated into the design. The number and variety of dredgers make this project unique. They include:

- an auger dredger;
- an environmental disc cutter;
- a slice cutter suction dredger

(a) Phase 1 (1996) (b) Phase 2 (1997) (c) Phase 3 (1998)

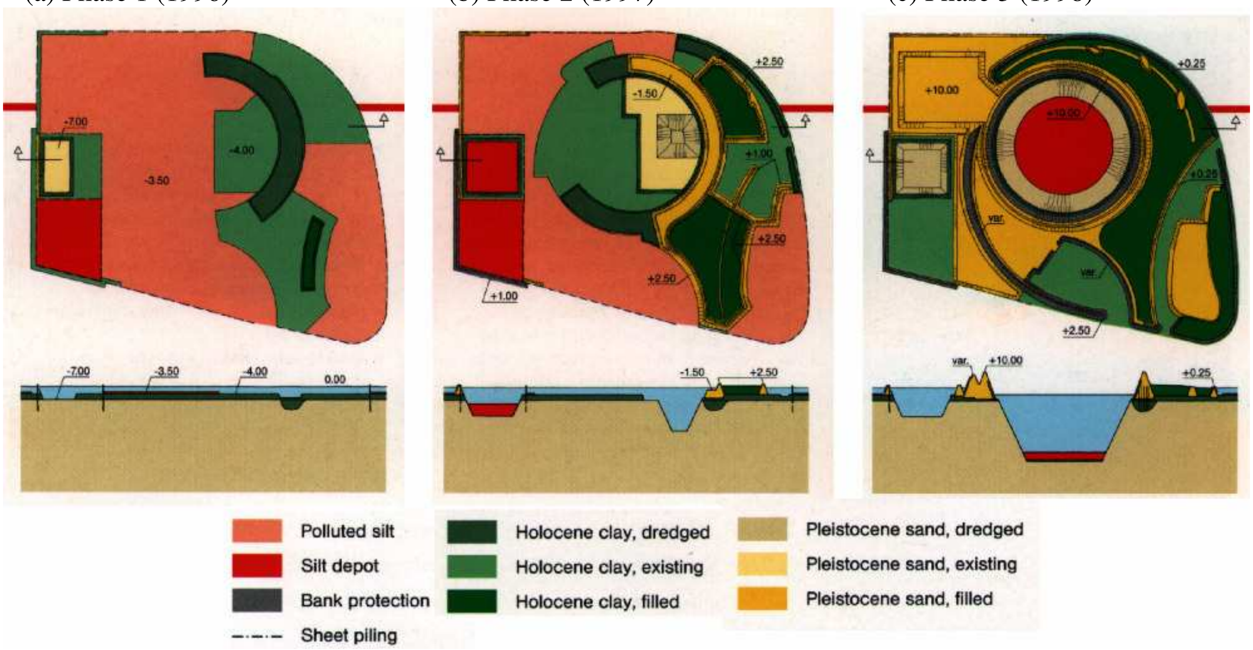


Figure 23. The construction phases of Ketelmeer Project

All of the environmental dredgers have the ability to dredge to within an accuracy of one cm. This level of accuracy is important to ensure that neither too much nor too little silt is removed. Too much removal leads to wastage of storage and processing capacity, while too little removal means that the lake bed is not entirely clean. A cm of dredging can have major consequences. The extreme accuracy of the dredging operation is achieved using satellite-based positioning. The new technique limits the maximum deviation to only 1 or 2 cm, both horizontally and vertically. During dredging activities, it is normal for silt to be disturbed and to mix with the surrounding water (clouding), but the most modern techniques can avoid almost entirely. Spillage of contaminated silt is also reduced to a minimum with these environmental suction dredgers. Moreover, the equipment can remove high silt concentrations. This is important because it avoids water being dumped in the depot along with the silt.

After the dredging of the contaminated silt, the clean clay and sand from the storage basin and from under the proposed ring dike, port area and secondary bank, were dredged out and used for the dike construction. The ring design of the dike demands a special working method, with the sand being deposited layer by layer. This gives the underlying material sufficient time to settle. Depositing the sand will take almost one and a half years, but because of the process of settlement, the construction of the dike will take almost two years. The bottom of the fully excavated depot is sealed with a non-permeable layer. Since the water levels of the Flevopolder, the Ketelmeer and the depot all differ, seepage could occur as the water tries to reach the lower area. In order to prevent this possibility and, thus, reduce the risk that contaminated material could escape from the depot mixed with water, pumps will be installed to maintain the water level in the depot. The depot will be filled by the contaminated silt. It is anticipated that the depot will be full in 20 years, at which time it will be covered with a clean layer of clay or sand. The three islands which have been constructed as nature reserves and for recreation will by that time be covered with established reeds and marshland vegetation, and the area around the depot will have become filled with undergrowth. It will then be a small paradise for water and marshland birds.

4.9.3 Reuse of dredged materials

Reuse of dredged materials (contaminated or not) is being increasingly considered as new methods and techniques evolve. From this point of view, dredged materials are being recognized as a significant, cost effective and sustainable natural resource. Up to now, applications and beneficial uses of treated dredged materials that show promise to meet environmental and materials standards are typically shown in Figure 24:

- Fill, structural and non- structural, from several processes
- Bricks, load and non-load bearing
- Solidified and stabilized materials for capping, parking lots, walkways…
- Manufactured soils for golf courses, parks, landscaping…
- Wetland for multiple purposes, including to safely filtering runoff
- Molded objects, including statuary from glass or bonding additives
- Aggregates for concrete, asphalt and road barriers
- Pre-cast infrastructure, including road barriers
- Retaining walls and other cement-like materials

Concerning specific reuse of contaminated sediment, it is believed that the intended use is determined by the degree of contamination such that the full range of dredged sediment can be considered. For example, cleanliness criteria for playgrounds are likely to differ from those for surface-mine reclamation material. This implies that, besides decontamination techniques, increasing interest is given on processes that make contaminated dredged materials environmentally safe by preventing leaching and volatilization of contaminants above acceptable levels. The above mentioned treatment techniques and underwater environmental issues are closely related to reuse of dredged materials.

Safe, treated dredged materials can also help to overcome environmental problems. Some examples of projects are as follows: preventing acid-mine drainage into groundwater by injecting treated sediment (with binders and additives) into abandoned mines, capping contaminated soils on brownfields to prevent leaching, engineering wetlands to filter leachates from landfills on waterways, and producing sediment based materials that control riparian erosion (PIANC. 1996 and 1997).

4.10 CONCLUSIONS

Underwater geoenvironmental issues were described based on waste sludge and dredging management and utilization. Dredging operations and the containment techniques are the most important issues. Waste and dredged sludge contamination are widely distributed in urban areas and, thus, the careful treatment is required. Many utilization techniques are available under carefully controlled operation systems. In particular, beneficial use of dredged materials as reclamation is also an effective issue with case studies supporting this approach for land use.

Agencies at various levels, including industries and the public, have made progress in developing regulatory and technical approaches to the cleanup of the most contaminated sites and to identify sites that require the most rapid action. However, no single regulatory or technical approach will work in all situations. According to a variety of authorities, additional resources and new approaches that are being applied on all fronts, the possibility of environmentally friendly reclamation is proposed in order to solve the problems concerning a contaminated environment and waste management.

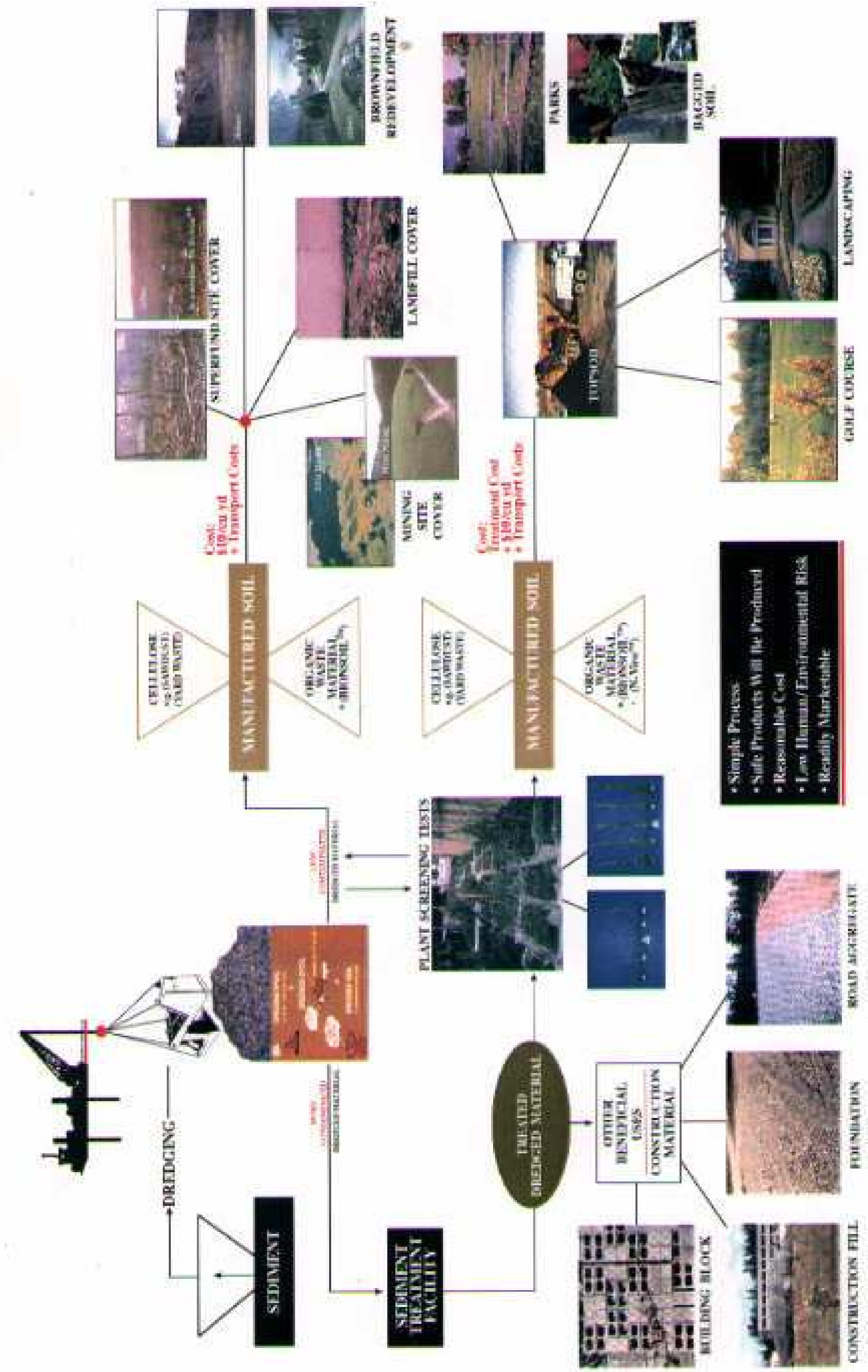


Figure 24. Range of ways to reuse dredged materials

REFERENCES

Alzieu, C. et al. 1999. Dredging and Marine Environment: State of the Art. IFREMER Edition, France, 223p.

Brand, E.W., Massey, J.B. & Whiteside, P.G.D. 1994. Environmental aspects of sand dredging and mud disposal in Hong Kong, *Proc. 1st ICEG*, Edmonton, 1-10.

COE. 1987. Confined disposal of dredged material, *Engineering Manual EM 1110-5027*, Dept. of Army.

EPA. 1990. Report on Great Lakes confined disposal facilities, *EPS/905/9-90/003*.

Fukue, M., Nakamura, T., Kato, T. &. Yamasaki, S. 1999. Degree of pollution for marine sediments, *Engineering Geology,* 53: 131-137.

Fukue, M., Yanai, M., Yamasaki, S., &. Kuboshima, S. 2000. Strategy for cleanup of polluted marine sediments, *Proc. 4th Kansai International Geotechnical Forum, Creation of New Geo-Environment,* Kyoto, 233-241.

Gomyoh, M., Hanzawa, H. & Pradhan, T. 1994. Investigation and measurements associated with environmental improvement of inner bay areas by covering polluted soft mud with a thin sand layer, *Proc. 1st ICEG*, Edmonton, 241-246.

Herbich, J.B. 1992. *Handbook of Dredging Engineering,* McGraw-Hill.

Herbich, J.B. 1995. Removal of contaminated sediments: Equipment and Recent Field Studies, *Dredging, Remediation, and Containment of Contaminated Sediments*, ASTM., STP1293, 77-111.

Hosomi, M., Takada, M., Taniguchi, S., Toda, H. & Miyamura, A. 2000. Chemical dechlorination of PCB- and dioxin-contaminated bottom sediment, *Proc. 4th Kansai International Geotechnical Forum, Creation of New Geo-Environment,* Kyoto, 243-252.

IADC 1999. From Hand-drag to Jumbo: A millennium of dredging. *Terra et Aqua*, No. 77, 48p.

Kamon, M. 1996. Geotechnical utilization of industrial wastes. *Preprint of 2nd ICEG; IS-Osaka'96*, 3: 1293-1309.

Kamon, M. & Katsumi, T. 1994. Utilization of waste slurry from construction works, *Proc. of 13th Int. Conf. on Soil Mech. and Foundation Eng.,* New Delhi, 4, 1613-1616.

Kamon, M., Katsumi, T. Sawa, N. & Ito, K. 2000. Minimization of heavy metal leaching effect from dredged sediments involving reclamation. *Proc. 4th Kansai International Geotechnical Forum, Creation of New Geo-Environment,* pp.253-258, Kyoto.

Kamon, M., van Roekel, G. & Bluml, W. 1998. Assessment of geo-environmental hazard from dredged materials. *Environmental Geotechnics*, ed. by P. S. Seco e Pinto, Vol. 3, pp. 1057-1074, Balkema

Kiyama, M., Oshima, A., Higashi, S., & Harada, K 2000. The new "accelerated consolidation method" combining the dewatering and plastic-board-drain by floating system (PDF) methods, *Proc. 4th Kansai International Geotechnical Forum, Creation of New Geo-Environment,* Kyoto, 221-226.

PIANC. 1996 and 1997. Handling and Treatment of Contaminated Dredged Material from Ports and Inland Waterways, *Report of Working Group I-17 of the Permanent International Association of Navigation Congresses*, Volume 1 and 2

Richardson, G.N., Petrovski, D.M., Chaney, R.C. & Demars, K.R. 1995. State of the Art: CDF contaminant pathway control, *Dredging, Remediation, and Containment of Contaminated Sediments*, ASTM., STP1293, 195-219.

Robbe, D., Marchandise, P. & Gouleau, D. 1985. Heavy metals in the sediment of La Loire estuary. *Water Resource*. 19-12: 1555-1563

Rogers, C.D., Kornel, A. & Sparks, H.L. 1991. Base catalyzed decomposition of toxic and hazardous chemicals. *Hazardous Waste Management in the Pacific Basin, HAZPAC'91*, 233-244.

Sato, E. 1984. Bottom sediment dredge Cleanup. *Proc. 8th US/Japan Experts Meeting on Management of Bottom Sediments Containing Toxic Substances,* 403-418.

Tsuchida, T., Watabe, Y., & Yuasa, H. 2000. Construction of seawall for waste landfill in Tachibana Port, Tokushima, *Proc. 4th Kansai International Geotechnical Forum, Creation of New Geo-Environment,* Kyoto.

Tsunoda, S. 1998. The history of technologies concerning the management of bottom sediments containing toxic substances. *Proc. 18th US/Japan Expert Meeting on Management of Bottom Sediment Containing Toxic Substances*, 3:1-28.

Yamasaki, S., Yasui, S. & Fukue, M. 1995. Development of solidification technique for dredged sediments, *Dredging, Remediation, and Containment of Contaminated Sediments*, ASTM., STP1293, 136-144.
Yoshinaga, K. 1995. Mercury-contaminated sludge treatment by dredging in Minamata Bay, *Dredging, Remediation, and Containment of Contaminated Sediments*, ASTM., STP1293, 182-191.

Chapter 5

Seismic Design of Solid Waste Landfills and Lining Systems

Authors:

Michele MAUGERI
University of Catania, Italy

Pedro Simão SÊCO E PINTO
National Laboratory of Civil Engineering (LNEC), Lisbon, Portugal

The authors gratefully acknowledge the review of this chapter and the contribution made to it by:

George BUCKOVALAS, *National Technical University of Athens, Greece*
Edward KAVAZANJIAN, *Arizona State University, USA*

ABSTRACT: The performance of solid waste landfills and lining systems during earthquakes is addressed. Analysis of solid waste landfill stability during earthquakes is presented. Experimental methods and the mathematical methods are referred. Selection of design earthquakes is presented. Also, determination of material properties for dynamic analysis is treated. The seismic response analysis is discussed. The assessment of liquefaction potential of landfills or foundation is referred. The dynamic response of geomembranes liners is addressed. Some case histories to illustrate the performance of solid waste landfills during earthquakes are presented. The monitoring and safety control of landfills are analysed. Risk analysis and safety are addressed.

5.1 INTRODUCTION

In this chapter, the seismic behaviour of solid waste landfills is considered. This behaviour is analysed by experimental and mathematical methods. The characterization of material properties for seismic design is difficult due the heterogeneity of the material, requiring the procurement of large samples.

Analysis of the seismic performance of solid waste landfill follows generally the same procedures for the design of embankment dams, but the methods and safety requirements differ. Nonetheless, both pseudo-static and deformational analysis methods are used for solid waste landfills as well as dams. An assessment of the behaviour of the geosynthetic elements used in the cover systems and bottom lining systems of landfills with respect to the potential for seismic induced permanent displacements is important. The monitoring tasks and the safety control of landfills are analysed. Risk analysis and safety are addressed.

5.2 PERFORMANCE OF SOLID WASTE LANDFILLS DURING EARTHQUAKES

From the lessons learned from past earthquakes, such as Loma Prieta earthquake (Johnson et al., 1991; Buranek and Prasad, 1991; Sharma and Goyal, 1991) and Northridge earthquake (Matasovic et al.,1995; Stewart et al., 1994; Augello, 1995) modern solid waste landfills have generally shown a good ability to withstand strong earthquakes without damages to human health and the environment. Experience has shown that well-built waste landfills can withstand moderate peak accelerations up to at least 0.2g with no harmful effects. Nonetheless, the integrity of solid waste landfills during strong earthquakes to achieve environmental and public health objectives deserves more consideration.

From well documented case histories, the failure mechanisms for landfills under earthquake loadins are as follows:

- Sliding or shear distortion of landfill or foundation or both;
- Landfill settlement;
- Transverse and longitudinal cracks of cover soils;
- Cracking of the landfill slopes;
- Damage to the gas system header pipes;
- Tears in the geomembrane liners;
- Disruption of the landfill by major fault movement in foundation;
- Differential tectonic ground movements;
- Cracks through the contact between refuse landfill and canyon; and
- Liquefaction of landfill or foundation.

These failure mechanisms are not necessarily independent of each other.

5.3 ANALYSIS OF SOLID WASTE LANDFILLS STABILITY DURING EARTHQUAKES

5.3.1 Introduction

The stability analysis of solid waste landfills can be established by following the procedures outlined in the flow chart presented in Figure 1. In general, the behaviour of solid waste landfills during the occurrence of earthquakes can be analysed by experimental methods or mathematical methods. Seismic design of solid waste landfills uses the same principles of seismic design of embankment dams (Sêco e Pinto,1993). The capabilities and limitations of these methods are briefly discussed.

5.3.2 Experimental Methods

Experimental methods are used to test predictive theories and to verify mathematical models. The most popular techniques for solid waste landfills are the shaking table and centrifuge models.

Yegian et al (1995) conducted shaking table tests and, based on the results, concluded that: (i) relative displacement (slip) at the geosynthetic interface may reduce the level of the acceleration pulses of the ground motion; and (ii) the geosynthetic interface acts as base isolator absorbing the wave energy through interface slip**.** Centrifuge model tests have been carried out to understand the principle of waste-structure interaction and to investigate deformation induced stress redistributions within the waste body near a structure (Kockel et al., 1997).

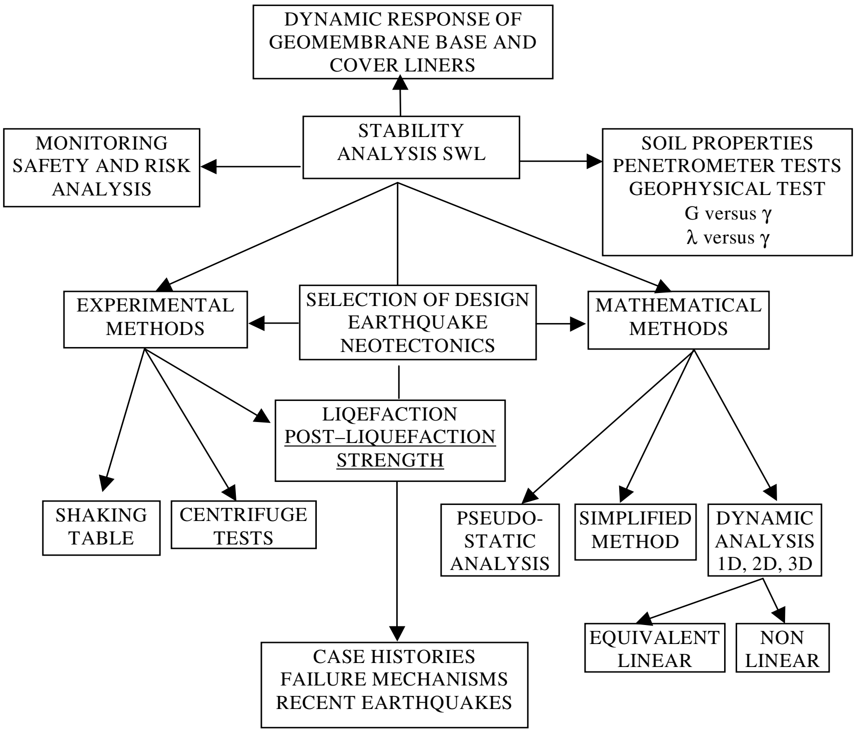


Figure 1. Flowchart for solid waste landfills

5.3.3 Mathematical Methods

The following methods for dynamic analysis of embankment dams are also used in landfill engineering (Sêco e Pinto et al., 1995):

i) pseudo-static analyses;
ii) simplified procedures to assess deformations; and
iii) dynamic analysis.

The slope stability of waste landfills is generally evaluated by limit equilibrium slope stabilility analyses.

For the pseudostatic analyses, a seismic coefficient value equivalent to the peak ground acceleration divided by 1.5 can be considered (Sêco e Pinto et al., 1998). For solid waste landfills, an acceptable seismic behaviour is antecipated if the calculated pseudo-static factor of safety ranges from 1.3 to 1.5.

Simplified procedures to assess landfills deformations were proposed by Newmark (1965), Sarma (1975) and Makdisi and Seed (1977) and have given reasonable answers in areas of low to medium seismicity. Newmark´s original sliding block model that considered only the longitudinal component was extended by Elms (2000) to include the lateral and vertical components of earthquake motion. The use of dynamic pore pressure coefficients along with limit equilibrium and sliding block approaches for assessment of stability of earth structures during earthquakes was demonstrated by Sarma and Chowdhury (1996).

Several finite element computer programs assuming an equivalent linear model in total stress have been developed for 1D (Schanabel et al., 1972; Idriss and Sun, 1992), 2D (Idriss et al., 1973; Lysmer et al., 1974) and pseudo 3D (Lysmer et al., 1975) analysis. Since these models are essentially elastic, the permanent deformations cannot be computed by this type of analysis and are estimated from static and seismic stresses with the aid of strain data from laboratory tests (cyclic triaxial tests or cyclic simple shear tests).

To overcome these limitations, nonlinear hysteretic models with pore water pressure generation and dissipation have been developed using incremental elastic or plasticity theory. The incremental elastic models have assumed a nonlinear and hysteretic behavior for soil, and the unloading-reloading has been modeled using the Masing criterion incorporating the effect of both transient and residual pore-water pressures generated by seismic loading (Lee et al., 1978; Finn, 1987).

5.3.4 Selection of Design Earthquakes

5.3.4.1. The design earthquake

The Code of Federal Regulations (United States 1991) requires new municipal solid waste landfills to be designed either for a maximum horizontal acceleration taken from a published seismic map for a 10 percent probability of exceedance (90 percent probability of non exceedance) in a 250-year exposure period or on the basis of a site specific analysis. The related return period for the map-based acceleration is 2375 years. The criterion of a site specific analysis is not specified in the regulation, but rather is left up to the individual states and may be probabilistic or deterministic. Because of the lower uncertainty, the return period for a site specific analysis may be less than 2375 years. For comparison, EC8 (1995) recommends a return period of 475 years for the seismic design of buildings and bridges.

The selection of seismic design parameters for municipal solid waste landfills, following the procedures used for dam projects, depends on the geologic and tectonic conditions at and in the vicinity of the site. Attenuation relations can be separated into 3 main tectonics classifications: (1) shallow crustal earthquakes in active tectonics regions, (2) regions subduction earthquakes, and (3) shallow crustal earthquakes in stable continental regions.

In terms of attenuation relations, the Idriss model (1995) and the Sadigh et al. model (1997) have only horizontal components, and Abrahamson and Silva (1997) relation has been used for vertical component. Overall, directivity has a significant effect on long-period ground motions for sites in the near-fault region.

5.3.4.2 Neotectonics

The tectonic conditions should include tectonic mechanisms, location and description of faults (normal, stryke and reverse) and estimation of fault activity (average slip rate, slip per event, time interval between large earthquake, length, directivity effects, etc). All of these factors are important to assess the involved risk.

Determination of neotectonic activity implies first the qualitative geomorphologic analysis based on air photos and topographic maps. The global position satellite (GPS) system is another powerful means of monitoring crustal mobility.

Cluff et al.(1982) have proposed the following classification for slip rates: extremely low to low for 0.001 mm/yr to 0.01 mm/yr, medium to high for 0.1 mm/yr to 1 mm/yr and very high to extremely high for 10 mm/yr to 100 mm/yr.

The most dangerous manifestation concerning landfill stability and integrity is the surface fault breaking, and intersecting the landfill site. In this regard, the current practice is a deterministic approach in which the seismic evaluation parameters are ascertained by identifying the critical active faults which show evidence of movements in Quaternary time. As per ICOLD (1989), an active fault is a fault, reasonably identified and located, known to have produced historical fault movements or showing geologic evidence of Holocene (11 000years) displacements and which, because of its present tectonic sitting, can undergo movements during the anticipated life of man-made structures.

Recently, a fault investigation method other than trenching has been developed, called the long Geo-slicer method in which long, iron, sheet piles with a flat U-shaped cross section are driven into an unconsolidated bed, iron plate shutters are inserted to face these iron sheet piles and the piles and shutters are pulled out to take undisturbed samples of strata of a certain specified width. This method is advantageous in regard to the ease of securing land for conducting investigations compared with trenching, and the ease of bringing the strata samples back to the laboratory for detailed observations (Tamura et. al, 2000).

When active faults are covered with alluvium, geophysical explorations such as the seismic reflection method, sonic prospecting, electric prospecting, electromagnetic prospecting, gravity prospecting and radioactive prospecting can be used (Takahashi et al.,1997). Of these methods, the seismic reflection method can locate faults if geological conditions are favourable, and confirm the accumulation of fault displacements based on the amount of displacements in the strata, which increases with strata age. However, even if the geometry and the position of the fault is known at bedrock level, you cannot know in advance whether and where the rupture plane will emerge at the ground surface, as well as what is the differential ground displacements that will develop. This problem can be approached either numerically (with the Finite Element and the Finite Difference method) or experimentally (with the aid of centrifuge tests or shaking table tests).

5.3.5 Selection of Soil Properties for Dynamic Analysis

The shear strength properties of waste landfills are not easily determined since the physical composition of the mixture makes it unsuitable for the conventional laboratory strength testing. The size of testing equipment is too small relative to the normal size of the refuse. To overcome this situation, the waste properties are established based on the type of waste, the waste processing and the placement procedures.

Some properties are measured directly, such as dry density and water contents, whereas other properties, due the difficulties related with sampling, are obtained from indirect methods combining with the existent knowledge of waste properties (Sêco e Pinto, 1997). A state-of-art on the evaluation of MSW properties using field measurements is given by Kavazanjian (2003).

Total unit weights of the material are determined from in-place testing or laboratory compaction tests. Kavazanjian (1995, 2003) presents a unit weight profile with depth (Figure 2). From a literature survey, the particle-size distribution of municipal solid waste is shown in Figure 3 (Jessberg, 1994). From results of laboratory and field tests the shear parameters of municipal waste exhibits a differentiation between fresh and old waste (Jessberg, 1996) (Figure 4). This differentiation is not quoted by some other Authors (Kavazanjian, 2001; Kavazanjian et al., 2001). A wide range of reported Vs values for MSW compiled by Kavazanjian et al.(1996) is shown in Figure 5.

To characterize the strength of solid waste, dynamic penetrometer tests were performed at the Grândola landfill and the obtained results are shown in Figure 6.

The measurement of the shear wave velocity (Vs) by cross-hole and down-hole techniques requires drilling boreholes in landfills. Spectral analysis surface waves (SASW) provide relatively accurate Vs profiles without the need for drilling and sampling the landfill material. Limitations in these measurement approaches include health and safety constraints on sampling and testing of solid waste, and the small size of test specimens relative to the size of the waste constituents.

Taking this into consideration, geophysical measurements to estimate dynamic strain-dependent materials of solid wastes of Grândola landfill were implemented, and a values of Vs between 330 - 350 m/ s were obtained (Figure 7) (Sêco e Pinto et al., 1999). The obtained results have not shown a variation of Vs with depth, probably because the height of landfill is only 12m, and are in reasonable agreement with the results reported by Kavazanjian et al. (1995).

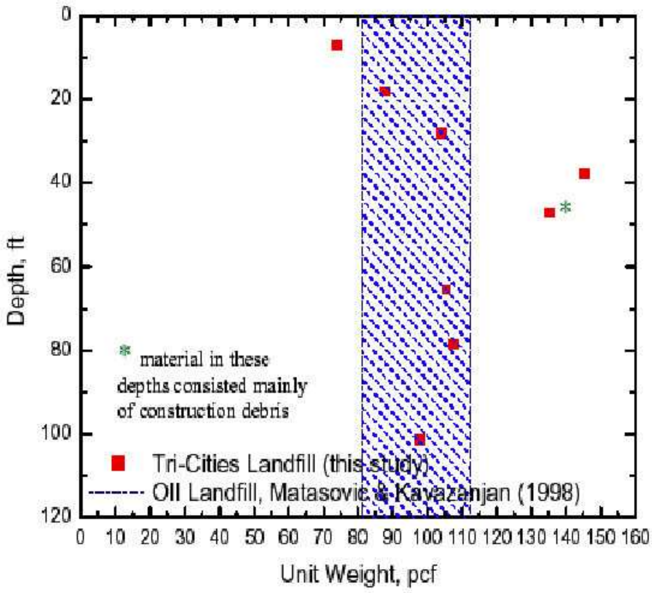


Figure 2. Unit weight of MSW (U.C. Berkeley, 2003)

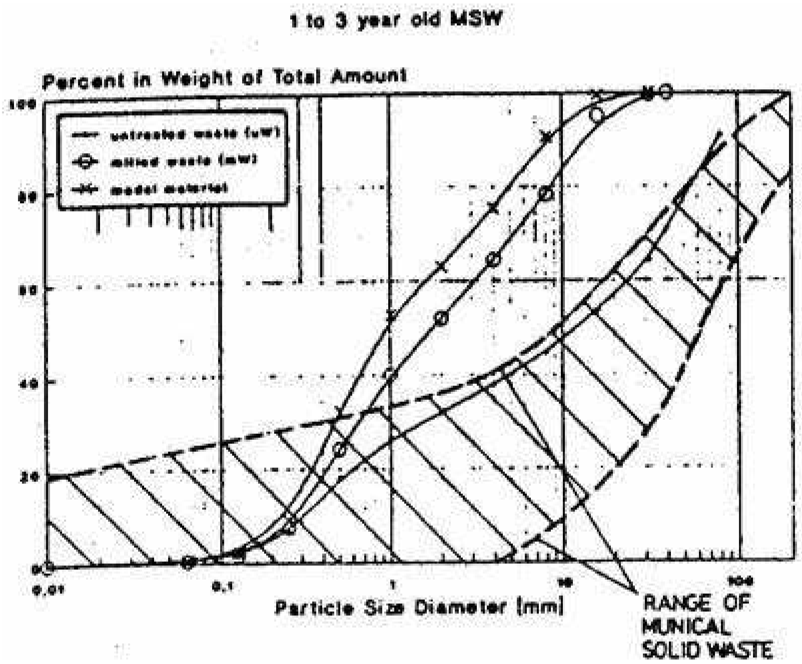


Figure 3. Particle size distribution of waste for laboratory tests (Jessberg, 1994)

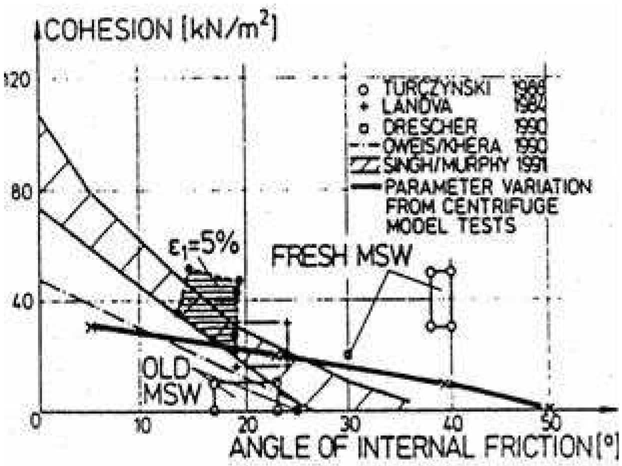


Figure 4. Shear parameters of municipal solid waste (Jessberg, 1996)

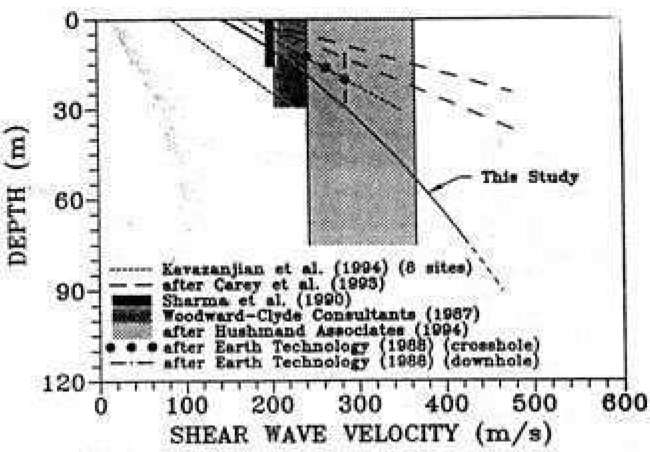


Figure 5. Shear wave velocity of MSW (Kavanzanjian et al., 1996)

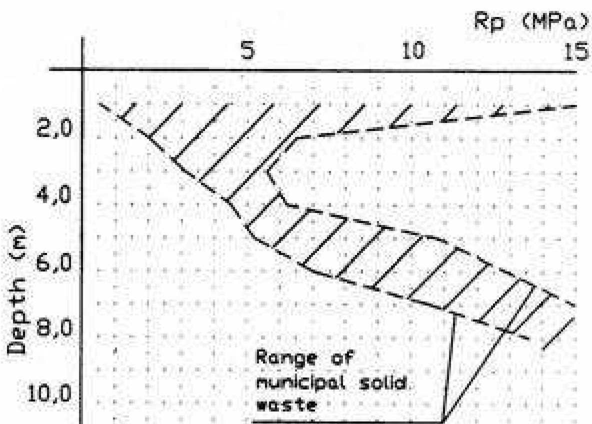


Figure 6. Dynamic penetrometer tests (Sêco e Pinto et al., 1999)

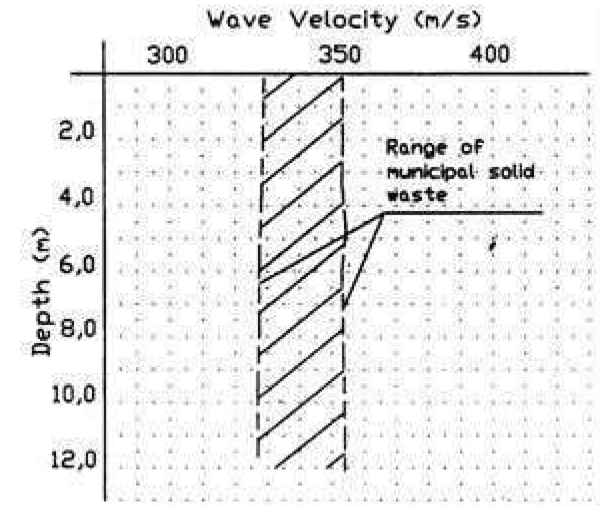


Figure 7. Shear wave velocities of Grândola solid wastes (Sêco e Pinto et al., 1999)

The variation of shear modulus G and damping ratio λ with shear strain can be derived by laboratory tests (Sêco e Pinto, 1990). For the variation of shear modulus and damping characteristics of waste materials with shear strain for sandy silt materials and silty materials, some curves proposed by Vucetic and and Dobry (1991) or by Matasovic and Kavazanjian (1998) are presented in Figure 8. When solid waste landfills incorporate construction demolition debris the curves proposed for rockfill and gravel materials can be used.

The shear interface resistance for liner and cover systems in landfills has deserved increasing attention and will be treated in section 5.4.2. Yegian and Kadakal (1998) developed a method where the dynamic properties of the geosynthetic liner can be replaced by the dynamic properties of an equivalent soil layer based upon shaking table tests. Horizontal geosynthetic interfaces have the potential effect of modifying the seismic response of overlying material. Smooth HDPE geomembrane/geotextile liners reduce significantly the accelerations and shear stresses transmitted through the landfill profile, especially when the base acceleration exceeds 0.2g, as pointed by Yegian and Kadakal (1998). These effects should be taken into account to avoid unrealistic estimates of seismic acceleration, shear stresses and permanent deformations in a landfill.

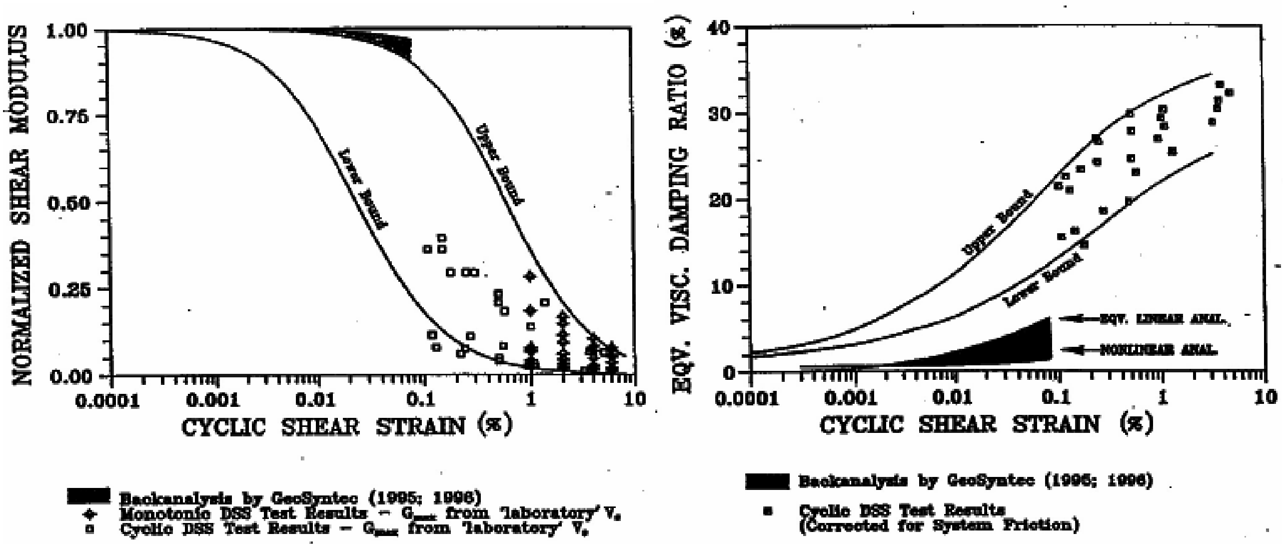


Figure 8. Waste modulus degradation and damping at OII from field measurements (Matasovic and Kavazanjian, 1998)

5.3.6 Seismic response analysis

The seismic responses obtained by computer finite element 1D programs are considered reasonable. These analyses are based on the solution of the equation of motion considering a homogenous and continuous soil deposit composed of horizontal soil layers and assuming a vertical propagation of shear waves. Since the slopes of landfills are usually flatter than slopes of earth dams, and landfill decks are larger than dam crests, two-dimensional response effects in landfills should be less significant than in earth dams. For the soil behaviour the equivalent linear method is used and the shear modulus and damping ratio are adjusted in each iteration until convergence has occurred. Due to the uncertainties related to the material properties and the foundation geometry, the influence of the seismic action generally is evaluated via parametric and sensitivity studies.

The shear stress distribution and the acceleration distribution for the solid waste at the Grandola solid waste landfill (SWL) for three foundation geometries are presented in Figures 9 and 10 (Sêco e Pinto et al., 1999). The basic data for the Grandola SWL are reported by Sêco e Pinto et al. (1999). Due to the geometry of the landfill (height and slopes), the effect of the HDPE geomembrane/geotextile liner was ignored; i.e. the dynamic properties of the geosynthetic liner was not replaced by the dynamic properties of the equivalent soil layer.

A comparison the results of the analyses performed by SHAKE 91 and QUAD 4M codes Rathe and Bray (1999) have concluded that: (i) the maximum seismic loading for base sliding within a landfill can be estimated conservatively with 1D analysis; (ii) the 1D analysis underpredicts the surface maximum horizontal acceleration (MHA) along the slope of a landfill by 10% on average, and by as much as 40 %; (iii) at the crest, 1D analysis consistently under predicts the MHA by about 25%; (iv) along the deck, the analysis is only moderately unconservative and the effect of base rock topography is not captured with 1D analysis.

It is important to stress that the dynamic characteristics of solid waste materials play an important role on the seismic response of landfill, and this area deserves more consideration (Sêco e Pinto et al., 1998). It also is important to assess the dynamic shear strengths of liner materials due the effect of inertial forces in the refuse mass.

5.3.7 Liquefaction assessment

The methods available for evaluating the cyclic liquefaction potential of landfills or foundation are based on laboratory tests and field tests. In general the following laboratory tests are used: (i) cyclic triaxial test, (ii) cyclic simple shear tests, (iii) torsional cyclic shear tests. Due the difficulties in obtaining high quality undisturbed samples, field test such as SPT tests, CPT tests, seismic cone, flat dilatometer and methods based on electrical properties of soil are used.

To estimate liquefaction resistance from shear wave velocity, there are two approaches: (i) methods based on a combination of in situ shear wave velocity measurements and laboratory tests on undisturbed tube and in situ freezing samples as described by Tokimatsu et al. (1991); and (ii) methods based on in situ shear wave velocity measurement and a correlation between liquefaction resistance and shear wave velocity deduced from the degree of liquefaction in the field based on Stokoe et al.(1999). The assessment of liquefaction resistance from shear wave crosshole tomography was proposed by Furuta and Yamamoto (2000).

Liquefaction resistance of silty sands during seismic liquefaction conditions for various silt contents and confining pressures was investigated by Amini and Qi (2000). The post-liquefaction strength of loose silty sediments is commonly less than that of sands, but moderately dense silts at shallow depths are generally dilative, making them more resistant to ground deformation than cleaner sands (Youd and Gilstrap, 1999). A probabilistic method considering the uncertainty in the liquefaction criterion was proposed by Todorovsha and Trifunac (1999).

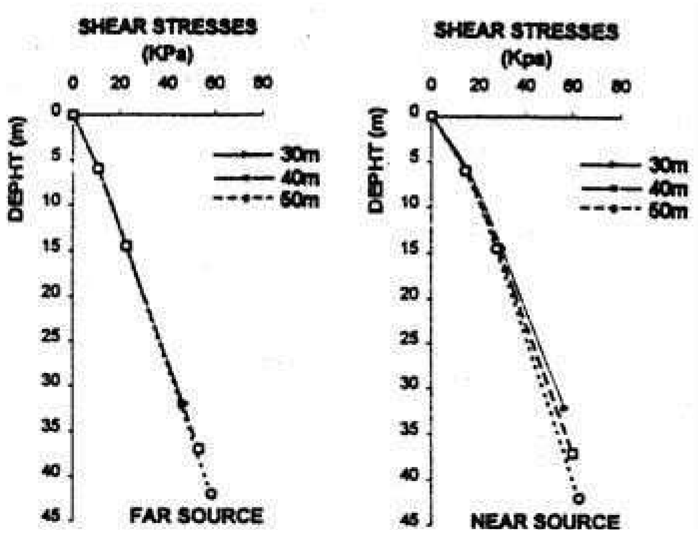


Figure 9. Shear stresses distribution (Sêco e Pinto et al., 1999)

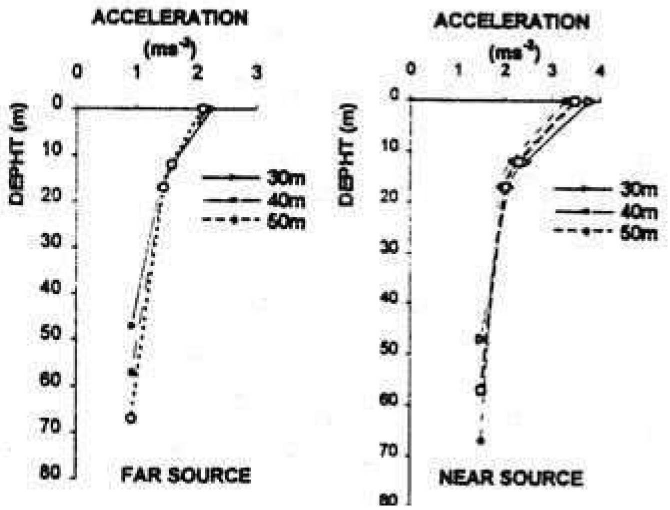


Figure 10. Acceleration distribution (Sêco e Pinto et al., 1999)

5.4 DYNAMIC RESPONSE OF GEOMEMBRANE LINERS

5.4.1. Introduction

Geomembrane liners are widely used in waste landfills mainly as part of a base liner or a cover liner to avoid groundwater impacts and environmental contamination. The key aspect of the response of geomembrane liners to earthquakes is the behaviour of geosynthetic interfaces. Slippage along interface may produce attenuation of earthquake induced acceleration to the cove liner, but also may cause some stability problems during and after earthquake loading.

5.4.2. Dynamic Geosynthetic Interface Behaviour

The use of geosynthetics in landfills, such as in other civil engineering applications, causes the presence of potentially week continuous surfaces, that must be carefully investigated to guarantee the static and dynamic stability of the entire landfill system. The main failure mechanism, which occurs along these discontinuities, is slippage. To avoid this phenomenon, it is necessary to control the interface strength or the limiting value of the shear stress available along the interface in static and cyclic/dynamic condition, considering the different geosynthetic/geosynthetic and geosynthetic/soil combinations that can be used in the field.

As far as the static interface strength analysis is concerned (Martin et al., 1984; Williams and Houlihan, 1986; Negussey et al., 1989; Mitchell et al., 1990), the interface friction angle between the geosynthetic and soil is lower than the internal friction angle for the soil itself. Also, the interface friction angle between two geosynthetics is sometimes lower than that between a geosynthetic and soil. Moreover, the residual interface strength may be completely mobilised at very low strain levels. Finally, the static interface shear strength is influenced by several different factors, such as the moisture content at the interface, the disposition of the geosynthetics relative to the loading direction, the polishing grade in case of geomembranes, the geosynthetic structure, the surface roughness, and the nature of the soil (Jewell, 1990; Stark and Poeppel, 1992; O'Rourke et al., 1990).

From comparison between the static and the cyclic/dynamic conditions, considerable divergences in the interface shear strength occur, even if some factors, such as the moisture content, have similar effects. For dynamic loads, the difference in the shear strength from the static to the dynamic condition is due to the inertial and viscous effects linked to the load velocity and to its time variation (Carrubba and Massimino, 1998).

In particular, the most common device used to evaluate the frictional properties of the typical landfill interfaces is the shaking table (Hushmand and Martin, 1990; Yegian and Lahlaf, 1992; Strano, 2000). The normal stress on the considered interface is generated by means of a concrete block. No relative displacement occurs between the block and the material being tested until the dynamic force reaches the interface limiting shear force, at which point a relative displacement occurs reflected by a clear break in the block acceleration/table acceleration curve.. Considering the limiting condition and assuming a Mohr-Coulumb failure criterion, it is possible to evaluate the friction angle ϕ_d, as follows:

$$\phi_d = \tan^{-1}(a_b / g) \tag{1}$$

where a_b is the block acceleration and g is the acceleration due to gravity.

With regard to the other widely used cyclic/dynamic laboratory tests to check the geosynthetic/geosynthetic or geosynthetic/soil interfaces, cyclic direct shear tests (Pasqualini et al., 1995; De, 1996) and shaking table tests on geotechnical centrifuge (Zimmie et al., 1994) have been used. The type of test to apply depends on the field use and the nature of the considered interface. For example, De and Zimmie (1998) suggest that the cyclic direct shear devise be used for seismic event problems, because the first 5-30 cycles are the most important for this event. Table 1 summarizes the tests recommended for eight typical geosynthetic/geosynthetic interfaces (De and Zimmie, 1998).

In performing direct shear tests, shaking table tests or tests in the geotechnical centrifuge to evaluate interface shear strength, applying the same stress levels as in the prototypes is an important consideration. In landfill applications, for example, a bottom liner system may be subjected to normal stresses variable between 100 and 800 kPa, considering a waste height of from 8 to 60 m, respectively.

Table 1. Tests for estimating geosynthetic interface dynamic friction angles (De and Zimmie, 1998).

Interface description	Recommended testing procedures to estimate dynamic friction angle	
	Seismic excitation (1)	Machine foundation(2)
Nonwowen geotextile over smooth geomembrane	ST or CDS consider reduction in ϕ	ST or CDS consider reduction in ϕ
Smooth geomembrane over geonet (oriented transversely)	CDS at proper σ	ST or CST at proper σ
Smooth geomembrane over geonet (oriented longitudinally)	CDS	ST or CST
Smooth geomembrane over geonet (oriented aligned)	CDS	ST or CDS
Nonwowen geotextile over geonet (oriented transversely)	CDS at proper σ and ϕ	ST or CST at proper σ and ϕ
Nonwowen geotextile over geonet (oriented longitudinally)	CDS at proper σ and ϕ	ST or CST at proper σ and ϕ
Nonwowen geotextile over geonet (oriented aligned)	CDS	ST or CDS
Smooth geomembrane over smooth geomembrane	CDS	ST/CST at proper ϕ
Notes: ϕ=friction angle; σ= normal stress; f = frequency; (1)Seismic excitation = small number of cycles; (2)Machine excitation = large number of cycles; CDS = cyclic direct shear for geosynthetics; ST = shaking table test; CST = centrifuge shaking table.		

Table 2 shows the average values of the peak friction angle for different geosynthetics interface and different tests in static and dynamic conditions. Analysing some typical geosynthetic/geosynthetic interfaces, the following observations can be made (Elgamal et al., 1990; Carrubba and Massimino, 1998).

1) The initial values of friction angle in static and dynamic conditions are very similar, even if due to though the dynamic shear strength of the soil interface is influenced by the strain rate. This leads to slightly higher values of the friction angle in dynamic conditions than in static conditions.

2) For geotextile/smooth geomembrane inter-faces, the peak dynamic friction angle decreases with increase of the number of excitation cycles, especially for low values of the number of cycles (Pasqualini et al., 1995; De and Zimmie, 1998). This reduction is very probably due to a polishing action. The polishing effect increases with the addition of moisture, which is common in landfill liners and covers because of the presence of leachate or other fluids (Von Pein and Lewis, 1991).On the contrary, for smooth geomembrane/geonet interfaces and smooth geomembrane/smooth geomembrane interfaces, a significant increase of the peak dynamic friction angle with cycle numbers is possible.

In the first case, the increase in peak dynamic friction can be due to a possible increased roughness of geomembrane caused by the geonet. In the second case, the increase in peak dynamic friction could be due to the occurrence of abrasion along the shaking direction (De and Zimmie, 1998). Finally, for goetextile/geonet interfaces, the peak dynamic friction angle appears independent from the number of excitation cycles.

3) The magnitude of the normal stress is not important for geotextile/smooth geomembrane and smooth geomembrane/smooth geomembrane interfaces, but the magnitude of the normal stress influences significantly the behaviour of smooth geomembrane/geonet and geotextile/geonet interfaces. In the latter case, the lower the normal stress, the higher the peak dynamic friction angle. The reason of the normal stress influence is until now not completely understood, but in some cases the reason could be related to the high deformability of non woven geotextiles. The normal stress in

relation to the material hardness can produce a penetration effect of one material to the other. This last phenomenon causes a significantly non-linear behaviour of interfaces (Carrubba and Massimino, 1998).
4) For interfaces including geonets, the mesh orientation can affect greatly the results. Of course, the lower interface shear strength occurs when the strands are aligned in the same direction of the motion.
5) As in static conditions, an increase in moisture at the interface has the effect to reduce the friction angle.
6) Tests performed on HDPE geomembrane-nonwoven geotextile interface showed that the maximum slip displacement is higher than the permanent one.
Thus, for a safe design of landfill liners and leachate collection systems, it is important to estimate not only the permanent slip displacement, as very frequently happens, but also the maximum dynamic displacement resulting from the displacement versus time history.

Lai et al. (1998) have performed cyclic shear tests on samples of a geomembrane supported geosynthetic clay liner. The dry material showed no degradation in shear strength during cyclic loading, but the hydrated material was found to reduce the shear strength by cyclic loading.

Table 2. Average peak friction angle of the interfaces (De and Zimmie, 1998)

Interface Description	Static friction angle		Dynamic friction angle		
	Tilt table test	Direct shear tests	Direct shear tests	Shaking table test 1g	Shaking table test 10g to 40g
Nonwowen geo-textile over smooth geomembraane	11.8°	12°	Decreases from 12.5° to 10.5°	12°	11°
Smooth geo-membrane over geonet (transverse)	10.1°	11.3°	Increases from 11° to 18° (for low σ) or 14° (at high σ)	12°	7°
Smooth geo-membrane over geonet (longitudinal)	9.8°	11.3°	Increases from 10° to 18° (for low σ) Or 16.5° (at high σ)	12°	11°
Smooth geo-membrane over geonet (aligned)	8.1°	8.1°	Increases from 9° To 18° (for both low and high σ)	---	---
Nonwowen geo-textile over geonet (transverse)	24.5°	Ranges from 22° (at low σ) to 14.5° (at low σ)	Ranges from 24° (at low σ) to 17° at high σ)	24°	8°
Nonwowen geo-textile over geonet (longitudinal)	13.9°	Ranges from 17° (at low σ) to 14° (at low σ)	15°	19°	11°
Nonwowen geotextile over geonet (aligned)	11.2°	10.5°	11° to 10°	---	---
Smooth geomembrane over smooth geomembrane	13.1°	8.8°	Increases from 10.3° to 19.5°	19°	13°

5.4.3. Predicting Landfill Response

To predict landfill response, knowledge of the bedrock motion and duration and the dynamic property of waste materials is required. In the case of landfills resting on soft clay (Figure 11a) or alluvial soil (Figure 11b), the site amplification effect must be considered to select a design earthquake (see section 5.3.4).

Figure 12 shows the potential failure mechanisms according to different soil foundations. The failure mechanism for waste fill resting on soft clay soil is controlled by foundation-soil failure.
Because of large deformations for geomembrane base liners in static (Figure 12a) and dynamic (Figure 12b) conditions, a failure of the geomembrane base liner can occur resulting in pollution of underground water.

In the case of landfills resting on alluvial soils, the potential mechanism of failure is linked to the translation of the side base liner system (Fig. 12b) if it is not well anchored. For landfill resting on rock foundations, a translation of waste fill due to the seismic loading can occur producing damages to both the base and the cover of landfill systems. The cover landfill system will be damaged regardless of the potential failure mechanism.

The performance of solid waste landfills during recent earthquakes (see section 5.5) shows that the cover system has been damaged. In regard to the waste material characterisation according to section 3.5, knowledge of the the unit weight, shear wave velocity, shear modulus and damping ratio, dynamic shear strength of waste material and dynamic geosynthetic interfaces behaviour is required (see section 5.4.2). Selecting a design earthquake (see section 5.3.4) and evaluating the dynamic soil waste landfill properties (section. 5.3.5) and geosynthetic interface properties (section 4.2), the seismic response of solid waste landfills can be performed by experimental methods (section 5.3.2) and mathematical methods (section 5.3.3).

The primary concern pertaining to seismic response of solid waste landfills is the dynamic response of geomembranes liners, because the eventual failure of the base liner and the cover liner systems causes the loss of landfill serviceability and creates the potential for environmental damages due to water and air pollution.

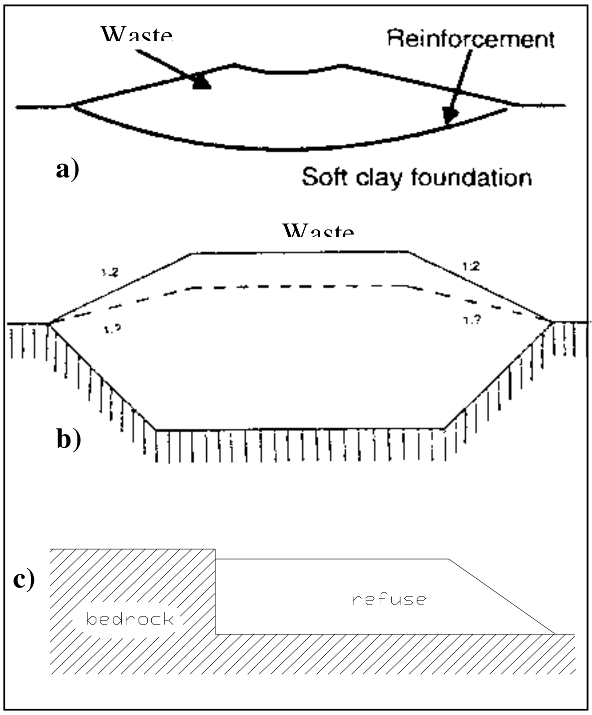


Figure 11. Types of waste landfill resting on different soil: a) soft clay, b) alluvial soil, c) rock.

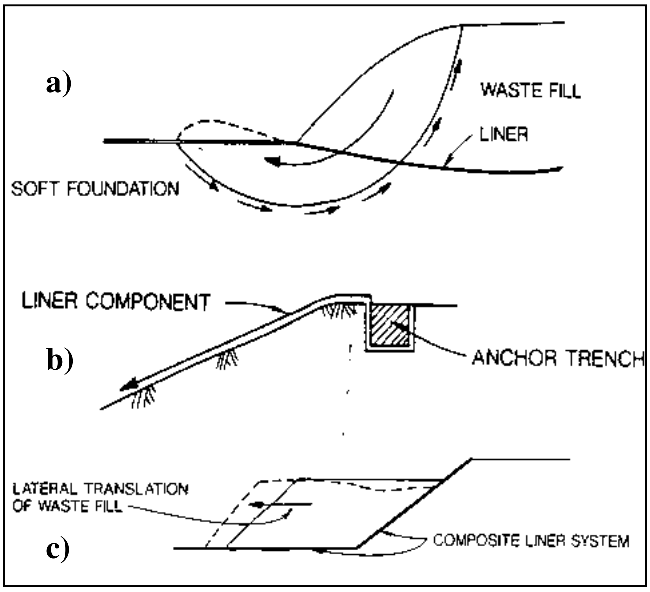


Figure 12. Potential failure mechanism according to soil foundation: a) failure of soft soil, b) translation of linear component, c) lateral translation of waste fill.

5.4.4. Dynamic Response of Geomembrane Base Liner

The dynamic response of geomembrane base liners can be determined by numerical analysis based on FEM modelling or by numerical analysis based on Newmark sliding block model (1965). Using the computer code Shake91 (Idriss & Sun 1992), Bray et al. (1995) found that the maximum horizontal earthquake acceleration (MHEA) for base sliding depends primarily on the fundamental period of the waste fill and on the maximum horizontal acceleration and predominant period of the input earthquake rock motion. Bray & Rathje (1998), taking into account the non linear response of solid waste material and considering the mean period of earthquake loading ($T_{m\text{-}EQ}$) instead of the predominant period, reviewed the response of over 300 hypothetical landfill profiles resting on rock sites to provide a better representation of the overall frequency content of a ground motion

The normalised response factor ($NRF=MHA_{site}/MHA_{rock}$) is reported versus the ratio $T_{s\text{-}waste}/T_{m\text{-}EQ}$ in Figure 13. The scatter in Figure 13 is primarily due to the non linear solid waste behaviour. For the same ratio, the lower values of MHA_{rock} (up to about 0.3g) tend to induce amplification or less attenuation (NRF>1), while for higher value of MHA_{rock} (more than about 0.5g) more attenuation takes place (NRF<1).

As shown in Figure 13, there is a dispersion of the response factor values near the resonance condition where strain levels are higher. This situation also must be checked for the evaluation of the permanent dynamic displacements. The permanent dynamic displacement can be evaluated from the displacement time history by a numerical analysis based on Newmark's sliding block model (Newmark 1965).

Considering a possible failure mechanism shown in Figure 12c, the normalised base liner sliding displacement versus the ratio between the critical acceleration K_y over the maximum earthquake acceleration K_{max} (evaluated as the MHA from Figure 13) is reported in Figure 14. Figure 14 allows for the evaluation of a average permanent base liner displacement U, normalised to K_{max} and to the significant duration $D_{5\text{-}95}$ of the input motion. From a theoretical point of view, when k_y/k_{max} is equal to unity, no seismic displacement can occur; however, since average values are used, base liner sliding displacement appears to be greater than zero for $k_y/k_{max}=1$.

5.4.5. Dynamic Response of Geomembrane Cover Liner

As the dynamic response of the geomembrane base liner (section 5.4.4), the dynamic response of the geomembrane cover liner can be performed by FEM numerical analysis or Newmark's sliding block model (Newmark 1965).

Using the computer code Shake 91 (Idriss & Sun 1992), Bray & Rathje (1998) found the normalised cover liner acceleration versus normalised fundamental period reported in Figure 15, similar to the normalise base liner acceleration reported by Bray et al. (1995).

The results reported in Figure 15 clearly show that, for cover liner system, the amplification of the rock motion is higher than the amplification for base liner system, causing a possible failure due to the translation cover liner component.

As shown in Figure 15, such as in the case of base liner, there is a dispersion of the response factor for a given period ratio due primarily to non-linear effects. The evaluation of the strain level and permanent dynamic displacement is a key point for design of the cover liner system.

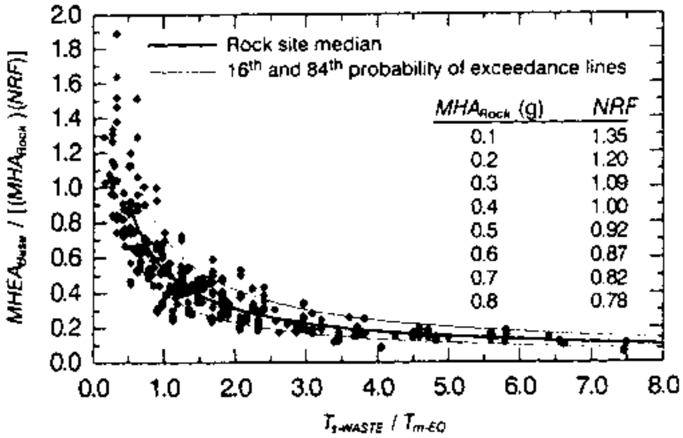


Figure 13. Normalised base liner acceleration versus normalised fundamental period (Bray & Rathje, 1998).

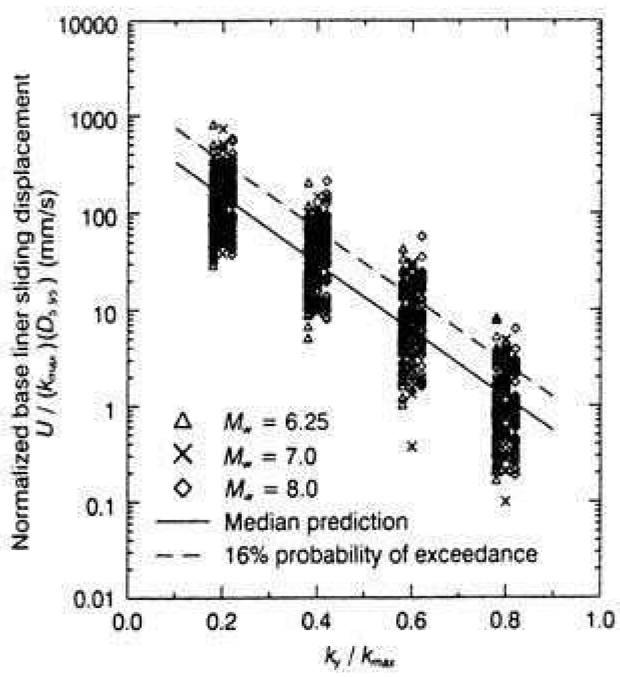


Figure 14. Normalised base liner displacement vs normalised critical acceleration (Bray & Rathje, 1998).

The cover liner displacement versus the normalised critical acceleration, evaluated similarly to the normalised base liner displacement, is reported in Figure 16. As can be seen by comparing Figure 14 with Figure 16, the cover liner displacements are higher than the base liner displacements. Bray et al (1998) report that cover displacements are usually significantly greater than those calculated for the base liner due to the higher seismic loading acting at the cover of the landfill.

Figure 14 allows for the evaluation of an average permanent displacement U of the base liner, normalised to K_{max} and to the significant duration $D_{5\text{-}95}$ of the input motion.

However, the dynamic geosynthetic interface behaviour depends on the maximum dynamic displacement, as reported in section 5.4.2, rather than an average permanent displacement. Consequently the maximum dynamic displacement of cover liner, must be carefully evaluated as reported in the following section.

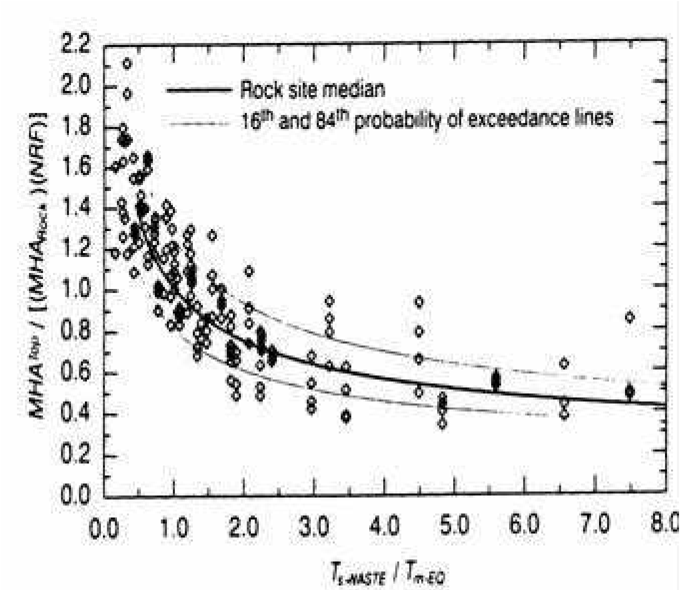


Figure 15. Normalised cover liner acceleration vs normalised fundamental period (Bray & Rathje, 1998).

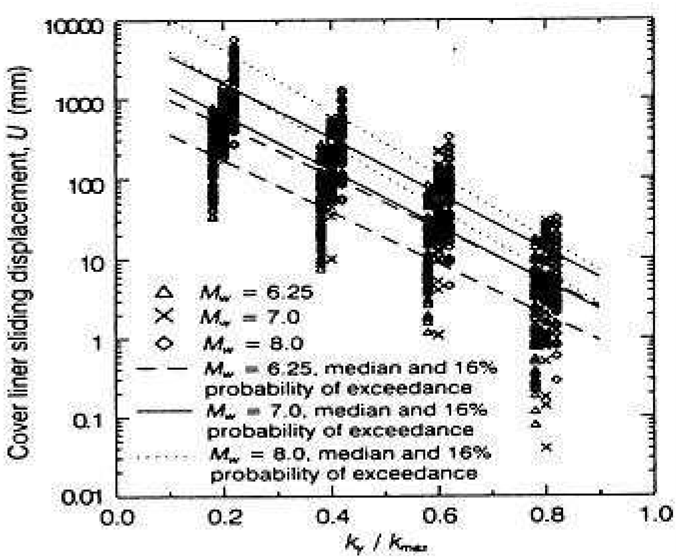


Figure 16. Normalised cover liner displacement vs normalised critical acceleration (Bray & Rathje, 1998).

5.4.6. Stability Analysis During and After Seismic Loading

Stability analysis during and after seismic loading is a key point for design of solid waste landfills in seismic areas. The seismic response of municipal solid waste (MSW) landfills are regulated in the USA by the Resource Conservation and Recovery Act (RCRA) Subtitle D regulation (US-EPA, 1994). According to this regulation, requirements for siting, design criteria, ground water monitoring, and closure/post closure maintenance of landfill facilities are established. It must be clearly demonstrated that all structures, including base and cover liner systems, leachate collection systems and surface water control systems, are designed to resist to the maximum horizontal acceleration. However, what constitutes "resisting" the earthquake is not defined (e.g., no damage or release of contaminants). The siting restrictions preclude the location of new MSW landfills within 60 m of a fault areas, unless it is clearly demonstrated that an alternative setback distance of less than 60 m will prevent damages to the structural integrity of the MSW and will be protective of human health and environment. Siting restrictions include also requirements for detailed engineering analyses in areas susceptible to liquefaction, induced instability and other modes of seismically induced displacement of natural slopes and foundation soils.

The waste containment regulations in most, if not all, European countries do not give specific requirements about the design of landfill in seismic areas, because the seismicity is moderate in most European countries, especially in the northern and in the central Europe. In Italy, where seismicity is moderate in the north and relatively high in the south, no restrictions are stipulated for MSW landfills, whereas the noxious and toxic (hazardous) waste landfills are forbidden in seismic areas with maximum horizontal acceleration equal to or greater than 0.1g.

The maximum cover liner displacement can be evaluated by FEM 1-D displacement time history analysis or by Newmark's sliding block model (Newmark 1965).

Figure 17 shows the results of a 1-D non-linear analysis of the Lentini (Sicily, Italy) MWS landfill performed using the GEODIN code (Frenna & Maugeri 1995). The results obtained with this code have been validated (Frenna & Maugeri, 2000) by back analysis of the behaviour of OII landfill impacted by the 1987 Whittier Narrows earthquake and by the 1994 Northridge earthquake. From Figure 17, it is possible to see that a slippage of the geomembrane cover interface occurs because of insufficient shear strength due to the limited normal stress level provided by the vegetative cover soil. The slippage of geomembrane cover, produces the positive effect of decreasing the seismic acceleration at the cover liner system, but it may also have the negative effect of increasing the horizontal maximum displacement.

Alternatively to FEM analysis, the time history of seismic displacement also can be evaluated by a numerical analysis based on Newmark sliding block model (1965). This model has been modified by Matasovic et al.(1997), to take into account two-way sliding, vertical motion and shear strength reduction from peak to residual at large displacements. Maugeri & Motta (1986) extend Newmark's block model to the case of circular slip failure. Cascone et al. (1998) also takes into account shear strength reduction due to cyclic loading for a circular slip failure. Figure 18 shows a comparison between the results of classical and modified Newmark's analysis performed taking into account the shear strength degradation. From Figure 18, it clearly can be seen that the classical Newmark analysis performed using peak parameter values for the waste can be conservative, while the modified analysis gives more realistic values of permanent displacements. A modified Newmark sliding block model has been proposed by Biondi and Maugeri (2002) and applied to the dynamic stability analysis of the cover system, taking into account cyclic shear strength degradation.

The acceptable displacements reported by Seed & Bonaparte (1992) are 150 mm and 300 mm for design of the geosynthetic base and cover systems, respectively, as will better described in the following section 5.6.

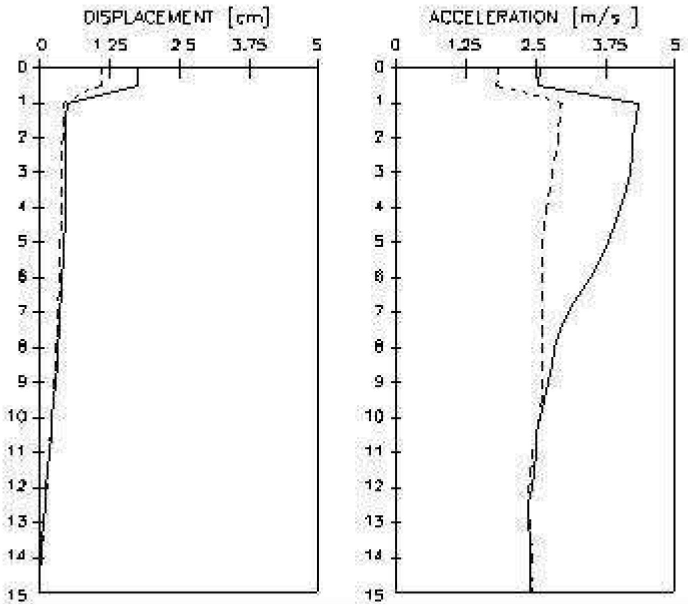


Figure 17. Non linear seismic response of Lentini landfill in terms of displacements and acceleration: [----] Northridge (1994) earthquake, [—] Sortino (1990) earthquake (normalised to Northridge maximum acceleration).

5.5 MONITORING AND SAFETY CONTROL OF LANDFILLS

Landfill behaviour during construction and operation is monitored to check methods and the results of analyses and model tests, and to analyse for safety against deterioration or failure. Seismic down-hole array data provide a unique source of information on actual soil behaviour over a wide range of loading conditions. Correlation and spectral analyses are performed to evaluate shear wave propagation characteristics, variation of shear wave velocity with depth, and site resonant frequencies and modal configurations (Elgamal et al., 1995).

With regard to seismic instrumentation of the response of landfills to such seismic activity, the type of instruments currently designated include strong - motion accelerographs, peak recording accelerographs and seismoscopes.

In comparison with manual readings, the automatic data acquisition systems allow for rapid data processing of results for a great number of instruments. Once in operation, an automatic system allows a reduction of personal, both in the field and office. The automatic system and central data processing allow a quicker updating of the information. An automatic system implies an increase of complexity, with electronic equipment to be installed in the unfavourable environment of low (e.g., below freezing) or high temperatures and high humidity. The seismic instrumentation and its maintenance, mandatory for dams, may be too expensive for landfills. However, the OII landfill has been instrumented with two accelerometers; one placed at the top of the landfill and another placed outside the landfill (Anderson et al., 1992).

For data validation, a preliminary check on the raw values (following functionality tests on measurement equipment) by comparing the actual values from the sensor readings with the established limits and data reduction (computation of engineering quantities) is performed. For the interpretation of the measurements, it is necessary to establish a mathematic model that can be a statistical model a deterministic model or a hybrid model.

Safety control refers to the group of measures taken in order to have an up-to-date knowledge of the condition of the landfill and to detect in due time the occurrence of any anomalies to define actions to correct the situation or, at least, to avoid serious consequences.

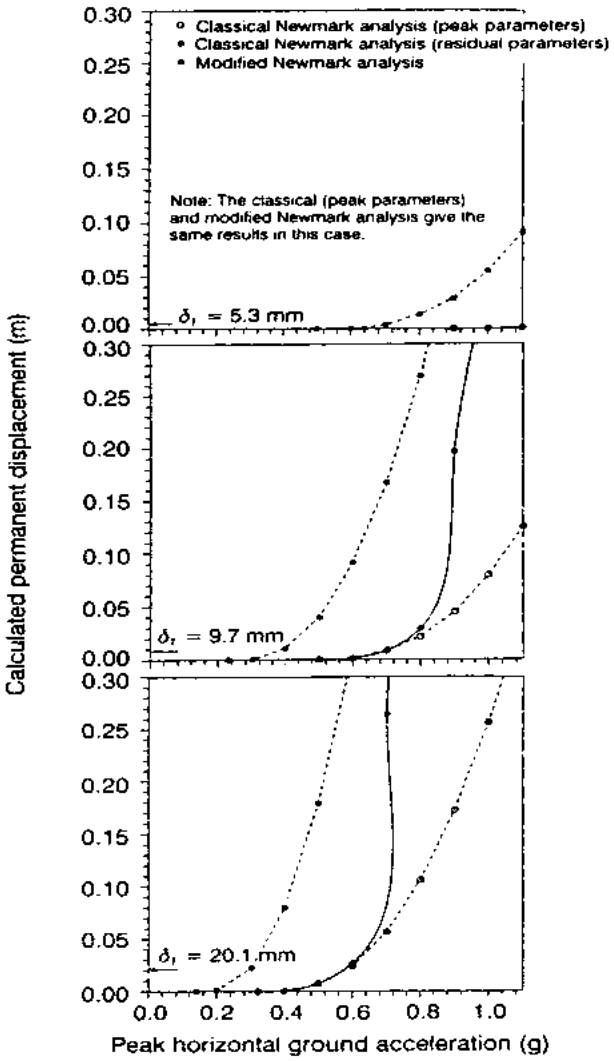


Figure 18. Cover liner permanent seismic displacement by modified Newmark analysis (Matasovic et al., 1998).

5.6 SAFETY AND RISK ANALYSES

Safety analysis for geotechnical structures, such as slopes, retaining walls, piles and shallow foundations implies the verification of limit states: ultimate limit states and serviceability limit states. For dams, two levels of safety also are considered, depending of whether they correspond to normal conditions for use of the structures (current scenario) or are associated with an exceptional occurrence (failure scenarios). Based on the above considerations, a level of damage for solid waste landfills apparently can be accepted provided there is no harmful discharge of contaminants to the environment.

The allowable value for the calculated permanent seismic displacement of geosynthetic liner systems is typically 150 to 300mm.

The upper value of 300 mm is generally considered appropriate for simplified analyses which use upper bound displacement curves for generic Newmark displacement charts, residual shear strength and/or simplified seismic analyses (Kavazanjian, 1998). The lower value of 150 mm is usually considered more appropriate for more sophisticated analyses and formal Newmark displacement analyses.

For cover systems, large displacements can be accepted taking into consideration that most cover failures can be detected and repaired at reasonable costs.

The allowable values for deformation of landfill systems depend on several factors related to geosynthetic liner systems and gas recovery system.

Municipality waste landfills owners, regulatory authorities and consultants typically are interested in carrying out a risk analysis. The purpose of this analysis is to identify the main risks associated with each type and height of landfill for all circumstances.

Such analyses can be conducted: (i) as extensive risk analysis of very large landfills, to substantiate reliably the probabilities chosen in event trees; (ii) as simplified risk analysis of smaller landfills, to focus low-cost risk analysis on a few main risks; or (iii) for identifying possibilities for reducing these risks through low-cost structural or non-structural measures.

Consideration of human behaviour is essential when assessing the consequence of failures. Since well organized emergency planning and early warning systems can decrease the number of victims, the study of human behaviour plays an important role in assessment of risk analysis.

The results of a risk analysis can be used to guide future investigations and studies, and to supplement conventional analyses in making decisions on safety improvements for waste landfills. With increasing confidence in the results of risk analyses, the level of risk may become the basis for safety decisions.

5.7 FINAL REMARKS

In the preceding sections, the different methods to analyse solid waste landfills and the stability of lining systems during earthquakes were presented.

The tools described are very important to assist the design engineer in incorporating adequate design measures to prevent deleterious effects of earthquake shaking.

All the essential steps of good analyses, whatever type of material or type of analysis are involved, shall be performed with a sufficient degree of accuracy that the overall results can be extremely useful in guiding the engineer in the final assessment of seismic stability.

This final assessment is not made by numerical results, but shall be made by experienced engineers who are familiar with the difficulties in defining the design earthquake and the material characteristics, who are familiar with the strengths and limitations of analytical procedures, and who have the necessary experience gained from studies of past landfill performance.

ABBREVIATIONS AND ACRONYMS

ASCE:American Society of Civil Engineers
CGJ : Canadian Geotechnical Journal
EERC:Earthquake Engineering Research Center
EESD:Earthquake Engineering and Soil Dynamics
ICCHGE:International Conference on Case Histories in Geotechnical Engineering
ICNMG: International Conference on Numerical Methods in Geomechanics
ICOLD: International Conference on Large Dams or International Committee on Large Dams
ICRAGEESD: International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics
ICSMFE: International Conference on Soil Mechanics and Foundation Engineering
JGED: Journal of the Geotechnical Engineering Division
JSMFD: Journal of the Soil Mechanics and Foundation Division
LNEC: Laboratório Nacional de Engenharia Civil
PACSMFE:Pan American Conference on Soil Mechanics and Foundation Engineering
SF: Soils and Foundations
WCEE: World Conference on Earthquake Engineering

REFERENCES

Abrahamson, N. A. and W. J. Silva 1997 “Empirical response spectral attenuation relations for shallow crustal earthquakes”. Sei. Res. Lett. 68, nº 1, pp. 94-127.

Amini. F. and G. Z. Qi. 2000 “Liquefaction testing of stratified silty sands”. J.G.G.E., Vol. 126, nº: 3, pp. 208-217.

Ambraseys, N.N. 1960. The Seismic Stability of Earth Dams, Proc., 2nd WCEE, Tokyo, Vol. II., pp. 331-336.

Anderson.,D.G., Hushmand, B. and Martine G.R. 1992. Seismic response of landfill slopes. Proc. Stability and Performance of Slopes and Embankments – II, Vol.2. ASCE, Geotechnical Special Publication, No. 31, Berkeley, California, pp.973-989.

Anderson.,D.G. and Kavazanjian, E.,Jr. 1995. State of the Art (SOA 14) Performance of landfills under seismic loading. Proc. 3rd ICRAGEESD, St.Louis, Vol.3, pp.1557-1587.

Augello, A.J. ,Bray.,J.D. Seed., R,D. ,Matasovic, N and Kavazanjian, Jr.,E. Solid waste landfill performance during the 1994 Northridge earthquake 1995. Proc. 3rd ICRAGEESD, St.Louis, Vol.3, pp.1395-1401.

Biondi G, Maugeri M. 2002. Modified Newmark analysis of cover system. Seventh International Conference on Geosynthetics, Nice, France, 22-27 September 2002, pp. 651-654.

Bray, J.D., Augello,A., J. Leonards, G. A. Repetto, P. C. and Byrne R. J. 1995. Seismic stability procedures for solid waste landfills. JGE, Vol. 121, Nº2, pp 139-151.

Bray J.D., Rathje E.M. 1998. Earthquake induced displacement of solid waste landfills. *Journ. Of Geot. Eng.,* ASCE, Vol. 124, No. 3, pp- 242-253.

Bray J.D., Rathje E.M., Augello A.J., Merry S.M. 1998. Simplified seismic design procedure for geosynthetic-lined, solid-waste landfills, G*eosynthetics International,* Special Issue on Geosynthetic in Earthquake Engineering, vol. 5 Nos. 1-2, pp.203-235

Buranek, D. and Prasad, S. 1991. Sanitary landfill performance during the Loma Prieta earthquake. Proc. 2nd ICRAGEESD, St. Louis, Vol. 2, pp. 1655- 1660.

Carrubba P., Massimino M.R., 1998. Dynamic geosynthetic interfaces behaviour. *Proc. XII Italian National Conference on Geosynthetic and Earth construction for Environmental protections,* In Journal of Engineers and Architects, Vol. 5, n. 1-3, pp. 33.36 (in Italian).

Cascone E., Maugeri M., Motta E., 1998. Seismic response of clay slopes. *Proc. 11th European Conference on Earthquake Engineering*, Paris. pp. 6-11

De A., 1996. Study of Interfacial Friction of Landfill Geosynthetics: static and Dynamic. Ph.D. Thesis, Rensselaer, Polytechnic Institute, Troy, New York, Usa, 245 p.

De A., Zimmie T.F., 1998. Estimation of dynamic interfacial properties of geosynthetics. G*eosynthetics International,* Special Issue on Geosynthetic in Earthquake Engineering, vol. 5 Nos. 1-2, pp. 17-39

EC8 (Eurocode 8) (ENV 1998-5.1994) Design provisions for earthquake resistance of structures- Part 5 : Foundations, retaining structures and geotechnical aspects.

Elgamal A.-W M., Scott R.F., Succarieh M.F., Yan L., 1990. “La villita” dam response during five earthquakes including permanent deformations. *Journ. Of Geot. Eng.,* ASCE, Vol. 116, No. 10, pp. 1443-1462.

Elgamal, A.W., M. Zeghal, and E. Parra 1995 “Identification and modeling of earthquake ground response”. First International Conference on Earthquake Geotechnical Engineering, Japan.

Elms, D. 2000 “ Refinements to the Newmark sliding block model”. Paper nº 2132. 12th WCEE, Auckland, New Zealand.

Finn, W.D.L. 1987 “Finite Element Handbook - Chapter 3 - Geomechanics”. Mc. Graw-Hill Editors H. Hardestuncer, .

Frenna S.M., Maugeri M., 1995. GEODIN: A computer code for seismic soil response. *Proc. IX Italian Congress of Computational Mechanics,* Catania 20-22 June, (in Italian), pp 145-148.

Frenna S.M., Maugeri M.,. 2000. On solid waste landfills seismic response analysis. *Second European Geosynthetic Conference, EuroGeo,* Bologna.

Furuta and Yamoto 2000 “ Liquefaction assessment by shear wave crosshole tomography tests ”. Paper nº 831. 12th WCEE, Auckland, New Zealand.

Hushmand B., Martin G.R., 1990. Final Report SBIR, Phase I study, Layered Soil-synthetic Linear Base Isolation system. Report submitted to the National Science Foundation Small Business Innovative Research Program, The Earth Technology Corporation, Long Beach, California, USA, 86 p.

ICOLD 1989. Selection Seismic Parameters for Large Dams - Guidelines. Bulletin nº 72.

Idriss, I. M. 1995 “An overview of earthquake ground motions pertinent to seismic zonation”. Proc. 5 th Int. Conf. Seismic Zonation, Nice, pp. 2111-2126.

Idriss I.M., Sun J.I. 1992. User's manual for Shake91. Centre for Geotechnical Modelling, Department of Civil and environmental engineering, University of California, Davis, California, USA 12p

ING (Istituto Nazionale di Geofisica). Contribution to the study of the 13-th December 1990 Eastern Sicily Earthquake. Editors E. Bosche & A. Basili, publication No. 537, 162 p (in Italian) .

Jessberg, H. L. 1994. Emerging problems and practices in environmental geotechnology . Proc. XIII ICSMFE Vol. 1 , pp 271- 281.

Jessberg, H. L. 1996. TC5 activities - Technical Committee on Environmental Geotechnics, ISSMFE., Proc. 2nd.International Congress on Environmental Geotechnics., Osaka, pp. 1- 28.

Jewell R.A. 1990. Strength and deformation in reinforced soil design, *Proc. 4th Int. Conf. on Geotextiles. Geomembranes and Related Products,* Vol. 3, The Hague, Netherlands, May 28 June 1.

Johnson, M. Lew, M. ,Lundy, J. and Ray, M.,E. 1991. Investigation of sanitary slope performance during strong ground motion from the Loma Prieta Earthquake of October 17. 1989. Proc. 2nd ICRAGEESD, St.Louis, Vol.2, pp 1701-1708.

Kavazanjian, E. Jr. 1995. Evaluation of MSW properties for seimic analysis. Geoonvironmental Geotechnics, pp 1126-1141.Balkema, Rotterdam.

Kavazanjian, E. Jr. 1998 Current issues in seismic design of geosynthetic cover systems. Proc. of

the 6th Int. Conference on Geosynthetics, Vol. 1, pp. 219-226.
Kavazanjian, E., Jr. 2001. Mechanical Properties of Municipal Solid Waste. Proc. Sardinia ‘01 - 8th International Waste Management and Landfill Symposium, Cagliari, Italy, October, Vol. III, pp. 415-424.
Kavazanjian, E. Jr. 2003. Evaluation of MSW properties using field measurements. Proc. 17th GSI/GRI Conference: Hot Topics in Geosynthetics – IV, Las Vegas, Nevada, 15-16 December, 2003, 40 p.
Kavazanjian, E. Jr., Hendron, D. and Corcoran, G.T. 2001. Strength and Stability of Bioreactor Landfills. Proceedings, *6th Annual Landfill Symposium*, Solid Waste Association of North America, Silver Springs, Maryland, 18-20 June, San Diego, pp. 63-72.
Kavazanjian, E. ,Jr., Matasovic, N. Bonaparte, R. and Schmertmann, G. R. 1995. Evaluation of MSW properties for seismic analysis. Proc. Geoenvironment 2000, ASCE Specialty Conference, New Orleans, Louisiana, 22-24, February
Kavazanjian, E. ,Jr., Matasovic, N. 1995. Seismic analysis of solid waste landfills. Proc. Geoenvironment 2000, ASCE Specialty Conference, New Orleans, Louisiana, 22-24, February
Kavazanjian, E. ,Jr., Matasovic, N., Stokoe, K. and Bray, J. 1996. In situ shear wave velocity of solid waste from surface wave measurements. Proc. of 2nd International Congress on Environmental Geotechnics,pp 97- 102, Vol.1.Edited by Masashi Kamon.
Kockel. R. ,Konig , D. and Syllwasschy, O. 1997. Three Basic Topics Mechanics on Waste Mechanics.Proc. 14 th ICSMFE, Vol.3 , pp 1831- 1837.
Lai, S. S. and H.B. Seed [1985] “Dynamic Response of Long Valley Dam in the Mammoth Lake”. Earthquake series on May 25-27, 1980. Report nº UCB/EERC-85/12, University of California, Berkeley.
Lee, M.K.W. and W. D.L. Finn 1995 “ DESRA-1: Program for the dynamic effective stress response analysis of soil deposits including liquefaction evaluation. Rep Nº 36, Mech. Ser. Department of Civil Engineering , University of British Columbia, Vancouver.
Ling, H. I. Leshchinsky, D. 1997. Seismic stability and permanent displacement of landfill cover systems, JGGE, Vol. 123, Nº2, pp 113- 122.
Lysmer, J., T. Udaka, H.B. Seed and R. Hwang 1974 “ LUSH 2- A computer program for complex response analysis of soil-structure systems”. Report nº UCB/EERC 75-30. University of California, Berkeley.
Lysmer, J., T. Udaka, C. Tsai and H.B. Seed 1975 “ FLUSH-A computer program for approximate 3D analysis of soil-structur interaction problems.”. Report nº UCB/EERC 74-4. University of California, Berkeley.
Makdisi, F.I. and Seed, H.B. 1977. A Simplified procedure for estimating earthquake-induced deformations in dams and embankments. Report Nº EERC 79-19. University of California, Berkeley.
Martin J.P., Koerner R.M., Whitty J.E. 1984. Experimental friction evaluation of slippage between geomembranes, geotextiles, and soils, *Proc. Int. Conf. on Geomembranes,* Denver, Colorado.
Matasovic, N.and Kavazanjian, E. 1996. Dynamic Properties of Solid Waste from Field Observations. Proc. of the First International Conference on Earthquake Geotechnical Engineering, Vol. 1, pp 549- 554. Kenji Ishihara Editor.
Matasovic, N.and Vucetic, M. 1996. Seismic Response of Soil Deposits Composed of Fully- Saturated Clay and Sand Layers. Proc. of the First International Conference on Earthquake Geotechnical Engineering, Vol. 1, pp 611- 616. Kenji Ishihara Editor.
Matasovic, N.,Kavazanjian, E. Jr. 1998. Cyclic characterization of OII landfill solid waste. *Journal of geotechnical and Geoenvironmental Engineering, ASCE*, Vol. 124, No. 3, pp. 197-210.
Matasovic, N.,Kavazanjian, E., J., Augello, A.J., Bray, J.D. and Seed; R.B. 1995. Solid Waste Landfill Damage Caused by 17 January 1994 Northridge Earthquake, In Woods, Mary C. and

Seiple, Ray W. Eds., The Northridge, California, Earthquake of 17 January 1994: California Department of Conservation, Division of Mines and Geology Special Publication 116, Sacramento, California, pp. 43-51.

Matasovic N., Kavazanjian E., Giroud J.R., Giroud J.P,1997. Newmark deformation analysis with degradation of yield acceleration. *Proc of* G*eosynthetics '97,* IFAI, vol. 2, Long Beach, California, Usa, pp.989-1000.

Matasovic N., Kavazanjian E., Giroud J.R., Giroud J.P.,1998. Newmark seismic deformation analysis for geosynthetic covers. G*eosynthetics International,* Special Issue on Geosynthetic in Earthquake Engineering, vol. 5 Nos. 1-2, pp.237-264.

Maugeri M., Motta E., 1986. On seismic design of clayey embankments and slopes. *Proc. Int. Symp. on Engineering Geology Problems in seismic areas*. Bari, Italy, pp. 277-288.

Mitchell J.K., Seed R.B., Seed H.B., 1990. Kettleman hills waste landfill slope failure. I: liner-system properties, *Journ. Of Geot. Eng.,* ASCE, Vol. 116, No. 4.

Mitchell, J. K. ,Seed, R.B. and Seed, H. B. 1990. Kettleman Hills waste landfill slope failure I: Liner-System Properties. JGE Vol. 116, Nº4, pp 647- 668.

Mononobe, N., Takata, A. and Matumura, M. 1936. Seismic stability of the earth dam. 2nd ICOLD, Washington, Vol. IV.

Negussey D.M Wijewickreme W.K.D., Vaid Y.P., 1989. Geomembrane interface friction, *Canadian Geotechnical Joural,* 26.

Newmark, N.M. 1965. Effects of earthquakes on dams and embankments, Geotechnique, Vol. 15, Nº 2, pp. 139-160.

O'Rourke T.D., Druschel S.J., NetravaliA.N., 1990. *Journ. Of Geot. Eng.,* ASCE, Vol. 116, No. 3, pp. 451-469.

Pasqualini E., Sani D., Roccato M., 1995. Factors influencing geomembrane interface friction. *Green '93*, Sarsby, R.W., Editor Balkema, Proceedings of a conference held in Bolton, United Kingdom, pp. 349-356.

Pasqualini, E. Stella, M. and Roccato 1996. Composite liners- improvements of the interface shear resistance. Proc. of 2nd International Congress on Environmental Geotechnics, pp 591- 596. Vol.1.Edited by Masashi Kamon.

Rathje, E.M. and Bray, J.D. 1999 " Two dimensional seismic response of solid waste landfills. Proc. of 2nd International Conference on Earthquake Geotechnical Engineering Lisboa. Vol. 2, pp.655-660. Edited by Pedro S. Sêco e Pinto. Published by A. Balkema.

Sadigh, K., C.Y., Chang, J.A., Egan, F. Makdisi, and R.R. Youngs 1997 " Attenuation relationships for shallow crustal earthquakes based on California strong motion data". Seis. Res. Lett. 68, nª 1, pp 180-189.

Sarma, S.H. 1975 "Seismic stability of earth dams and Embankments". Geotechnique, Vol 25, nº 4, pp. 743-76.,

Sarma, S.H., R. Chowdhury, 1996 "Simulation of pore pressure in earth structures during earthquakes". 11th WCEE, Acapulco, Mexico.

Schnabel, P.B., J. Lysmer and H.B. Seed 1972 "Shake: A computer program for earthquake response analysis of horizontally layered sites". Report nº UCB/EERC 72-12. University of California, Berkeley.

Sêco e Pinto, P.S. 1990. Dynamic characterization of soils. Natural Hazards and Engineering Geology - Prevention and Control of Landslides and other Mass Movements". European School of Climatology and Natural Hazard Course.

Sêco e Pinto, P.S 1993. Dynamic analysis of embankment dams. Proceedings of the Seminar on Soil Dynamics and Geotechnical Earthquake Engineering, pp 159-269. Edited by Pedro S. Sêco e Pinto. Published by A. Balkema.

Sêco e Pinto, P. S. 1997. Analysis of the behaviour of solid waste landfills (in portuguese). Seminar on

Geoenvironmental Aspects. of Industrial Solid Waste Landfills,Lisboa .

Sêco e Pinto, P. S., Dakoulas, P. C. Harder, L. Watanabe, H. and Chugh, A. 1995 Stability of slopes and earth dams under earthquakes-General Report Session VI. Proc. 3rd ICRAGEESD, St. Louis, Vol. 3, pp 323 - 332.

Sêco e Pinto, P. S. Lopes, L. Agostinho, and Vieira, A. 1998. Seismic analysis of solid waste landfills. Proc. of 3rd. International Congress on Environmental Geotechnics, Lisboa. Edited by Pedro S. Sêco e Pinto. Published by A. Balkema.

Sêco e Pinto, P. S. Lopes, L. Agostinho, and Vieira, A. 1998. Seismic behaviour of solid waste Grândola landfill. Proc. of 2nd International Conference on Earthquake Geotechnical Engineering Lisboa. Vol. 2, pp.661-666. Edited by Pedro S. Sêco e Pinto. Published by A. Balkema.

Seed, R. B., Mitchell, J. K. and Seed, H. B. 1990 Kettleman Hills waste landfill slope failure II stability analyses, Journal of Geotechnical Engineering, ASCE Vol. 116, Nº 4, pp. 669-690.

Sharma, H.D. and Goyal, H. K. 1991. Performance of a hazardous waste and sanitary landfill subjected to Loma Prieta earthquake, Proc. 2nd ICRAGEESD, St. Louis, Vol.2 pp 1717- 1725.

Singh, S. and Murphy, B. 1990. Evaluation of the stability of sanitary landfils. ASTM STP 1070 Geotechnics of Waste Landfils- Theory and Practice. A. Landva, and G:D. Knowles, eds. ASTM, Philadelfia, Pa. pp. 240-258.

Singh, S. and Sun, J.I. 1995. Seismic evaluation of municipal solid waste landfills". Proc. Geoenvironment 2000, AsCE Specialty Conference, New Orleans, Louisiana, 22-24 February.

Somerville, P. G. N. F. Smith, R. W. Graves, and N. A. Abrahamson 1997 "Modification of empirical strong ground motion attenuation relations to include the amplitude and duration effects of rupture directivity". Sei. Res. Lett. 68, Nº 1, pp. 199-222.

Stokoe, K.H. II, M. B. Darendeli, R.D., Andrus and L.T., Brown 1999 "Dynamic soil properties: Laboratory, field and correlation studies". Theme Lecture. Proc. of 2nd International Conference on Earthquake Geotechnical Engineering, Lisboa. Edited by Pedro Sêco e Pinto. Published by A. Balkema, Vol.3, pp. 811-845.

Soydemir,C.1995. Seismic Design of landfills for NE United States Proc. 3rd ICRAGEESD, St.Louis, Vol.1, pp 457-462.

Seed H.B., Bonaparte R., 1992. Seismic analysis and design of lined waste fill: current practice. Stability and performance of slopes and embankment-2, Seed H.B. and Boulancer R.W. Editors. Special publication n.31 ASCE, pp. 1152-1187.

Stark T.D., Poeppel A.R., 1992. Landfill linear interface strengths from torsional-ring-shear tests. *Journ. Of Geot. Eng.,* ASCE, Vol. 120, No. 3.

Stewart, J. P., Bray, J.D., Seed, R. B. and Sitar, N. 1994 Preliminary report on the proncipal geotechnical aspects of the January 17, 1994 Northridge Earthquake, Report Nº. UCB/EERC-94/08, College of Engineering, University of California at Berkeley, Berkeley, California, 238 p.

Strano A., 2000. Soil-Geosynthetics interface shear strength evaluation by shaking table tests. *Diploma Thesis*, University of Catania, Italy (in Italian).

USEPA, 1994. RCRA Subtitle D(258) Seismic design guidance for Municipal Solid Waste Landfill Facilities, Final Draft. *United States environmental Protection Agency.*

Takahashi, T., K. Mimoto, and T. Hayakawa 1997 "Present state of applications of geophysical methods to characterization of active faults". Journal of Japan Society of Engineering Geology, 38, pp.118-129.

Tamura, C., S. Kanyo, T. Uesaka, I. Nagayama and Y. Wakizaka 2000 "Survey and evaluation of active faults on dam construction in Japan". Paper nº 2493. 12th WCEE, Auckland, New Zealand.

Todorovska, M. I. and M. D. Trifunac 1999 "Liquefaction opportunity mapping via seismic wave energy". J.G.G.E., Vol. 125, nº 12, pp. 1032-1042.

Tokimatsu, K., S. Kuwayama and S. Tamura 1991 "Liquefaction potential evaluation based on Rayleigh wave investigation and its comparison with field behavior". Proceedings 2nd Int. Conf.

On Recent Advances in Geotech. Earthquake Eng. And Soil Dynamics, March 11-15, St. Louis, Vol. I, pp. 357-364.

U.C. Berkeley (2003). Preliminary report on drilling and sampling activities, tri-cities landslides. *U.C. Berkeley, Geosyntec, U.T. Austin Collaborative Research Project on Static and Dynamic Properties of Municipal Solid Waste.* Geotechnical Engineering Group, University of California, Berkeley.

Von Pein R.T., Lewis S.P., 1991. Composite Lining System Design Issues. *Geotextiles and Geomembranes*, Vol. 10, Nos. 5-6, pp. 125-131.

Williams N.D. & Houlihan M., 1986. Evaluation of friction coefficients between geomembranes, geotextiles and related products, *Proc. 3rd Int. Conf. on Geotextiles,* Vienna, Austria.

Yegian, M. K.. and Kadakal, H. V., 1998. Seismic response of landfills with geosynthetic liners. Proc. of the 6th Int. Conference on Geosynthetics, Vol. 1, pp. 227-230.

Yegian, M. K., and Lahlaf, A. M. 1992. Dynamic interface shear strength properties of geomembranes and geotextiles. Journal of Geotechnical Engineering, ASCE, Vol. 118, Nº. 5, pp. 760-779.

Yegian, M. K., Yee, Z. Y. and Harb, J. N. 1995. Response of geosynthetics under eartquake excitations. Proceedings of Geosynthetics´95, Vol 2, pp. 677-689.

Youd, T. L. and S. D. Gilstrap 1999 "Liquefaction and deformation of silty and fine-grained soils". General Report. Proc. of 2nd International Conference on Earthquake Geotechnical Engineering, Lisboa, Vol.3, pp. 1013-1020. Edited by Pedro Sêco e Pinto. Published by A. Balkema.

Zimmie T.F., De A., Mahumud M.B., 1994. Centrifuge modelling to study dynamic friction at geosynthetic interfaces. *Proc. of the Fifth Int. Conf. on Geotextiles, Geomembranes and Related Product,*Vol. 1, Singapore, pp. 415-418.

Chapter 6

Education in Environmental Geotechnics

Author:

Mario MANASSERO
Politecnico di Torino, Italy

Contributing Authors:

Chiara DEANGELI
Politecnico di Torino, Italy
Claudia SPANNA
I.G. Ingegneria Geotecnica S.r.l., Torino, Italy

The authors gratefully acknowledge the review of this chapter and the contribution made to it by:

Pietro JARRE*, Golder Assiciates, Torino, Italy*
Hywel THOMAS, *Cardiff University, United Kingdom*

ABSTRACT: The paper begins by considering various points of view expressed by outstanding geotechnical engineers and professors, on the teaching of traditional and advanced geotechnics. Thereafter, a case is developed by comparison of related studies and publications with the key thoughts of these geotechnicians that proposes a consistent layout for teaching environmental geotechnics concepts. A revised version of Burland's (1987) triangle is proposed that includes the new elements. From this structure, example environmental geotechnics course structures are proposed. A summary of observations from an investigation of the primary features of undergraduate and graduate education programs in a number of European and United States universities is presented, in order to frame the main trends arising from the different basic schools.

6.1 INTRODUCTION

Before discussing environmental geotechnics education, it is worthwhile to keep in mind the basic principles leading to a solid traditional geotechnical education. This can be summarised by following quotes from some of the outstanding figures of our discipline.

An overview of their thoughts can help us in order to frame, within the main aims of environmental engineering education, the role of environmental geotechnics and, thereafter, to address the disclosure and teaching of this specialised subject. In particular, this inspirational source can help us to define, in the most appropriate way, the basic geotechnical elements that are necessary for dealing with the specialised topics that comprise environmental geotechnics.

As a first step, it is important to realise the complexity of the behaviour of the multiphase system forming soil and its pedagogic function for many other advanced engineering topics. In consideration of these aspects, the following sentences are enlightening:

"I believe that in many ways soil mechanics lends itself splendidly to the correct training of the minds of engineers-to-be. It lifts them to a level from where they can see the subject of mechanics, theory of elasticity, the properties of materials in their true and very interesting relationships. Especially the nature of "stress and strain" in its full meaning can be more fully understood from the study of materials with so wide range of possible combination, as is the case with soil" Casagrande (1958).

Burland (1987) is in the habit of introducing his lectures in Soil Mechanics for undergraduate students with the following sentences:

"In soil mechanics we are dealing with a highly complex and variable material. In the first place we have the challenge of geology. Unlike most other construction materials the geotechnical engineer has to make do with what he finds and the first thing he has to do is to find, or deduce, what is there. Nature is seldom straightforward and it often requires a considerable amount of detective work coupled with an understanding of geological process to unravel the geological and groundwater complexities of a site and, above all, appreciate their engineering significance".

Referring to mechanical soil behaviour, Burland (1987) also points out that, "*We are dealing primarily with particulate material having an extraordinarily wide range of particles sizes and shapes with an enormous variety of gradings, packing arrangements and stress histories. Moreover, being particulate materials, soils are very difficult to sample and test without influencing their properties. Hence great ingenuity is required to devise appropriate sampling and testing techniques."* Moreover "… b*ecause of their particulate nature, the development of mathematical models to describe and predict the behaviour of soils is a formidable task which must rank with any of the most esoteric sciences".*

Apart from the aforementioned difficulties, concerning the characterization of soils and the idealization of their behaviour through theoretical models, Burland found an illuminating comparison between the skills of the geotechnical engineer and those necessary to be a good craftsman: "*A distinct feature of a craftsman* (carpenter, blacksmith or stonemason) *is that he 'knows' his material. He may not be able to quote its Young's modulus, yield strength, degree of anisotropy or homogeneity but he knows from handling it and working it far more about its likely behaviour than would be revealed by measuring a dozen different properties".*

This last sentence leads to the following conclusion by Peck (1960): "*The application of soil mechanics in practice requires knowledge in three quite separate areas. These are a thorough grounding in the theoretical and experimental concepts of soil behaviour; a thorough knowledge of the accumulated experience of our contemporaries and predecessors in the field; and a working knowledge and appreciation of aspects of geology related to soil materials. Not one of these fundamental aspects could be eliminated from our training without serious damage".*

6.2 FROM TRADITIONAL SOIL MECHANICS TO ENVIRONMENTAL GEOTECHNICS EDUCATION

Among others, these concepts, expressed by Peck, inspired Burland's triangle (1987) which summarizes the key aspects of a correct and appropriate approach to a traditional soil mechanics and geotechnics education, as follows:

(i) the ground profile;
(ii) soil behaviour;
(iii) applied mechanics, which are at the apexes of the triangle; and
(iv) empiricism occupying the centre with the function of linking and synthesising the information and the idealization schemes arising from the other points.

With reference to the Burland triangle, the Technical Committee on Education (TC31) of the ISSMGE discussed and studied the aspects of "Geotechnical engineering education toward 2000", with the aim of establishing a starting point for an introduction to environmental geotechnics education. The main conclusions of the study are listed as follows:

(i) Generally, undergraduate educational programs in which one or more courses in geotechnics would be given include:
   - civil engineering, specialising in geotechnical engineering (CEGE);
   - general civil engineering not specialising in geotechnical engineering (CE);
   - mining engineering (ME); and
   - environmental engineering (EE).

(ii) The amount and nature of geotechnical courses will generally differ in each case; however, there will nevertheless be a series of desirable objectives, which will include the following (Poulos, 1999):
   - a proper understanding of the principles of soil mechanics, and in particular of the effective stress principles;
   - an appreciation of the process of application of theory to practice;
   - an understanding of the shortcomings of both theoretical and practical design methods; and
   - an appreciation of the means by which soil parameters may be assessed, such as laboratory tests, in-situ tests and empirical correlations.

(iii) It has been suggested by TC31 that each of the four different undergraduate educational programs should ideally contain at least two among the following courses in geotechnical engineering:
   - basic soil mechanics (SM);
   - soil and foundation engineering (SFE);
   - applied geotechnical engineering (AGE);
   - mining geotechnical engineering (MGE); and
   - environmental geotechnics (EG).

(iv) The distribution scheme of the basic geotechnical courses among typical undergraduate education programs should be as follows:
   - CEGE ⇒ SM, SFE, AGE;
   - CE ⇒ SM, SFE;
   - ME ⇒ SM, MGE; and
   - EE ⇒ SM, EG

(v) From the point of view of TC31, basic soil mechanics should be considered a mandatory course for all of the education programs listed above. The main course topics forming a basic soil mechanics course are presented in Table 1.

Table 1: Basic Soil Mechanics – Main Course Topics (Poulos, 1998)

Lecture and Tutorial
- Soil formation and Geomorphology - Phase relationship and basic definitions - Soil classification - Effective stress principle - Soil permeability - Seepage analysis - Stress path, Mohr circles - In-situ stress, pre-consolidation pressure - Soil compressibility and stress-strain - Soil strength - Critical state concepts - Undrained and drained strength
Demonstrations and Laboratory work
- Effective stress principle (Demo) - Seepage/quicksand (Demo) - Soil classification (Lab) - Seepage model (Lab) - Direct shear test on sand (Lab) - Direct shear test on clay (Lab)

From a review of the course topics presented in Table 1, it is immediately apparent that it is not possible to properly address education in environmental geotechnics, without first ensuring a sufficient level of basic understanding of the stress-strain behaviour of soils, solid skeleton and pore fluids interaction, soil testing and the interpretation of soil characteristics.

Postgraduate education courses can vary widely. In the authors' opinion, it is generally necessary to increase the level of knowledge and understanding of the basic principles governing the behaviour of micro, macro and mega structures of soils as well as the modern theories and numerical tools for modelling complex soil behaviour. Once these principles and theories are understood, it is possible to introduce new topics. Referring, for example, to environmental geotechnics, new topics could include the chemico-physical interaction of mineral barriers and pollutants and the electro-kinetic remediation of polluted soil.

From the previous statements delineating the ISSMGE approach to geotechnical engineering education, it is possible to define the required prerequisite subjects that would provide post graduate students with the necessary background, prior to application for entry to specialized environmental geotechnics courses. In particular, the following subjects should be considered mandatory:

- Advanced Soil Mechanics integrated with courses including Laboratory and Field Investigation, Slope stability, Retaining structures and Foundation design, Dewatering and Groundwater control, and Soil improvement;
- basic Chemistry, Physics and Thermodynamics;
- Continuous Mechanics of Solids;
- Mixture Theories and Multiphase Fluid Dynamics;
- Bonding, Crystal Structure and Surface Characteristics of Soils and Soil Mineralogy; and
- Soil Formation and Geological Aspects.

Moreover, it is useful to be able to deal with differential analysis and numerical modelling. Finally, as complementary matter, a basic knowledge of industrial, agricultural, waste and water treatment processes and plants can be considered very appropriate (see Table 2).

Table 2: Basic Knowledge for Environmental Geotechnologists (Fang, 1997)

Bacteriology, Biology
Chemical Engineering
Climatology, Geohydrology
Geophysics, Geochemistry
Hydrogeology, Mechanics
Micro geology, Physico-Chemistry
Soil Science, Soil Engineering
Toxicology
Statistics
Legislation

6.3 BASIC CONTENTS OF ENVIRONMENTAL GEOTECHNICS

"Environmental Geotechnics", in the broadest sense of the word, can include topics that range from the prevention and minimization of natural hazards, such as floods, earthquakes and slides, to landfill design and in situ confinement for pollutant control, remediation of contaminated sites and re-use of by-products for civil works (Brumund, 1995; Carrier et al., 1989; Daniel, 1993; Morgenstern, 1985; Sembenelli & Ueshita, 1981).

Some of the aforementioned topics belong to the traditional geotechnical field (e.g. slope stability), which deals with mechanical and hydraulic behaviour of particulate media; others, such as pollutant control via mineral barriers, are typical examples of recent developments within the emerging area of environmental geotechnics. Environmental geotechnology, as defined by Fang (1997), is an *"interdisciplinary science which covers soil and rock and their interaction with the various environmental cycles, including the atmosphere, biosphere, hydrosphere and lithosphere"*.

Environmental geotechnology can be considered to include environmental geotechnics and to be an emerging science, as illustrated in Figure 1, where the contribution of geotechnics and other disciplines are listed together with the time history of the main developments.

There are two major reasons for the development of Environmental Geotechnology as an engineering discipline (Fang, 1997): (i) the adverse environmental conditions due to soil pollution created by industrial activity and (ii) the decrease in available landfill areas and locations due to the increase in areas used for residential housing and other construction projects. To cope with soil pollution and adverse environmental conditions, conventional construction technology must, by necessity, take a new direction.

In consideration of these factors, the assessment of soil should not be based on a mechanical and hydraulic approach alone (see Figure 2). Other phenomena in the chemical, biological, electrical, magnetic and thermal fields must be taken into account for a comprehensive assessment of soil behaviour (Manassero & Shackelford, 1994).

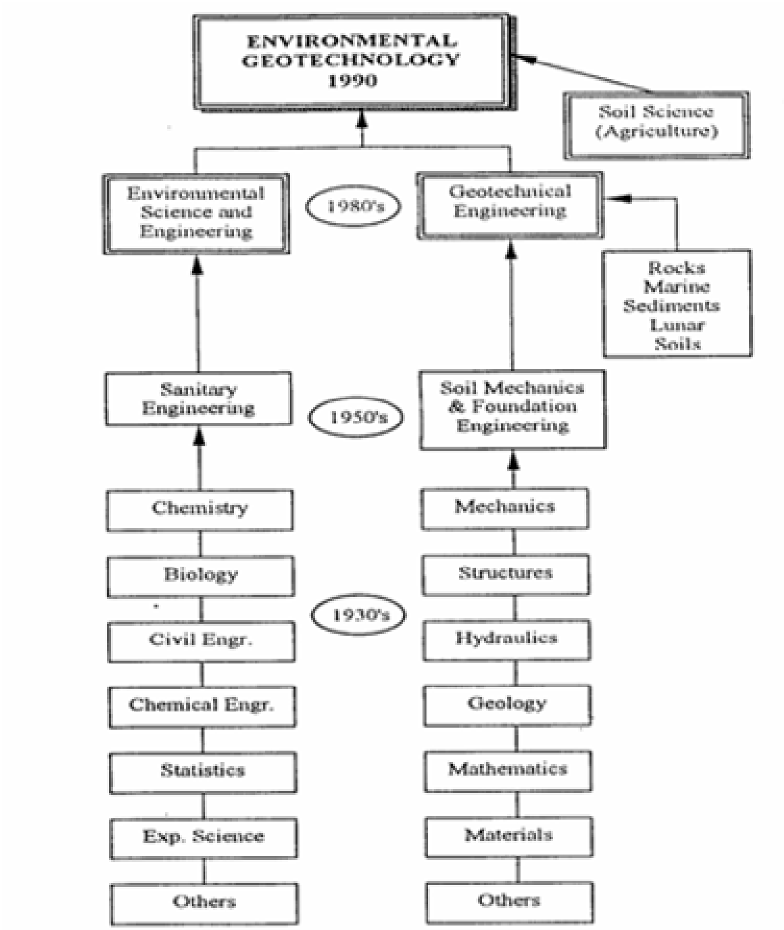


Figure 1. Environmental Geotechnology – An emerging Science (Fang, 1997; Koerner, 1987)

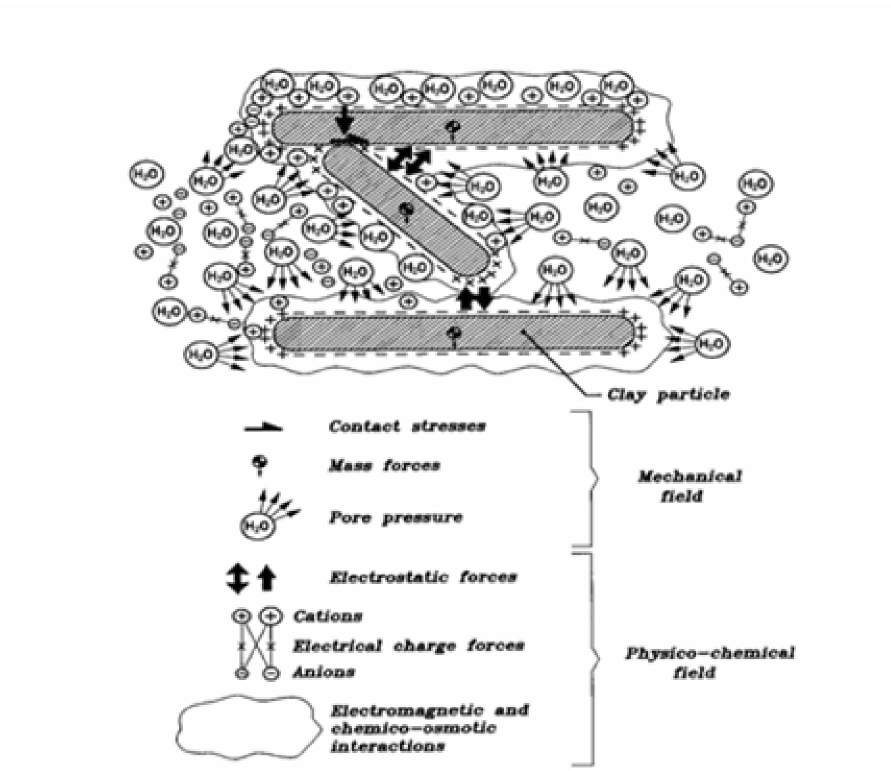


Figure 2. Multiphase particulate porous media system (Manassero, 1998)

Considering the complex interactions within the soil and the factors requiring assessment, the following subjects pertinent to environmental geotechnics, in terms of investigation, design and construction control activities (in some cases the contribution of other disciplines is obviously necessary) can be proposed:

- landslide and subsiding areas;
- solid and liquid waste landfills;
- polluted subsoil and abandoned landfills;
- re-use of by-products; and
- in situ improvement of the mechanical behaviour of wastes.

With the exception of the first point related to traditional landslide, subsidence and erosion problems, the subsequent points have been included in the first report of TC5 (1998). The index of this document is presented in Table 3.

Table 3. Index of the first report of Technical Committee N.5 of the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) on Environmental Geotechnics

Sub-Committee	*Title*
TC5-SC1	Monitoring system for: - performance assessment of landfill liners - containment of polluted areas - pollutant extraction in contaminated soils - mechanical behaviour of landfilled waste and embankments of by-products
TC5-SC2	Contaminant migration - test methods, model-ling and monitoring
TC5-SC3	Waste stability - waste classification and characterization - stability and bearing capacity of solid waste landfills
TC5-SC4	Controlled landfills - design of lining systems - capping systems and sealing barriers - regulatory aspects and quality assurance - leachate collection and leak detection systems
TC5-SC5	Contaminated land reclamation (design, construction and management)
TC5-SC6	Assessment of geo-environmental hazards from dredging materials
TC5-SC7	Assessment of geo-environmental hazards from non-traditional geotechnical construction materials
TC5-SC8	Long term behaviour of containment systems including risk assessment
TC5-SC9	Landfill behaviour under extreme loading assessment of possible scenarios

From a consideration of the pertinent factors and subject areas covered by environmental geotechnics, the Burland (1987) triangle, originally developed to frame and address the traditional aspects of a geotechnical engineering education, can be modified to include additional elements that together provide the key components of an environmental geotechnics education. The modified Burland triangle detailing these elements is presented in Figure 3.

From a review of Figure 3, it can be seen that some basic topics of soil mechanics have been developed toward a more general and some toward a more comprehensive approach. This approach is necessary due to the complicating factors introduced by multiphase fluid-dynamics, electro-chemical surface forces, coupled flows, etc. These aspects of education can be generally neglected in traditional geotechnical problems, but are very important when dealing with environmental geotechnics applications.

It is important to note that almost all of the topics listed in Figure 3 are direct developments of the basic geotechnical aspects listed in Burland's original triangle. We can observe, for example, that the phenomena of diffusive-dispersive migration of pollutants can be described by the basic theory that is formally the same as the theory used for modelling fine grained soil consolidation, as taught within basic soil mechanics undergraduate courses. Moreover, it is interesting to observe how two apparently very different phenomena have a common theoretical background that can be exploited to improve student comprehension and competence in dealing with both elements.

Given the previously delineated background from undergraduate courses, a well stated environmental geotechnics postgraduate program should include the following specialized subjects:

- Advanced Environmental Geotechnics;
- Identification and Characterization of Subsoil, Groundwater and Contamination Phenomena via Field and Laboratory Tests;
- Landfill Design and Management;
- Subsoil Remediation Technologies;
- Geo-Environmental Utilization of Industrial By-Products; and
- Surveying and Monitoring Systems for Geotechnical and Geo-environmental Applications.

If one goes into further detail, it is possible to propose the basic contents of some of the aforementioned subjects.

The content for the course on “Advanced Environmental Geotechnics" could include the following topics:

- principles and modelling of contaminant transport (advection, dispersion, diffusion);
- principles and modelling of contaminant interaction with solid, liquid and gas phases in soils;
- systems for waste and pollutant control;
- mechanics applied to complex particulate media;
- waste classification;
- improvement of waste behaviour;
- principles of remedial works for polluted sites;
- investigation and monitoring; and
- overview of environmental legislation, regulations and recommendations.

The course for "Landfill Design and Management" could include:

- waste type and relevant regulations;
- basic elements for sitting;
- hydrological balance;
- design procedures for key landfill components (e.g. liner systems, see Figure 4, leachate detection and collection systems, gas extraction system, capping system, geotechnical stability issues);
- construction specifications and quality control;
- surveying and monitoring systems; and
- waste management (Figure 5).

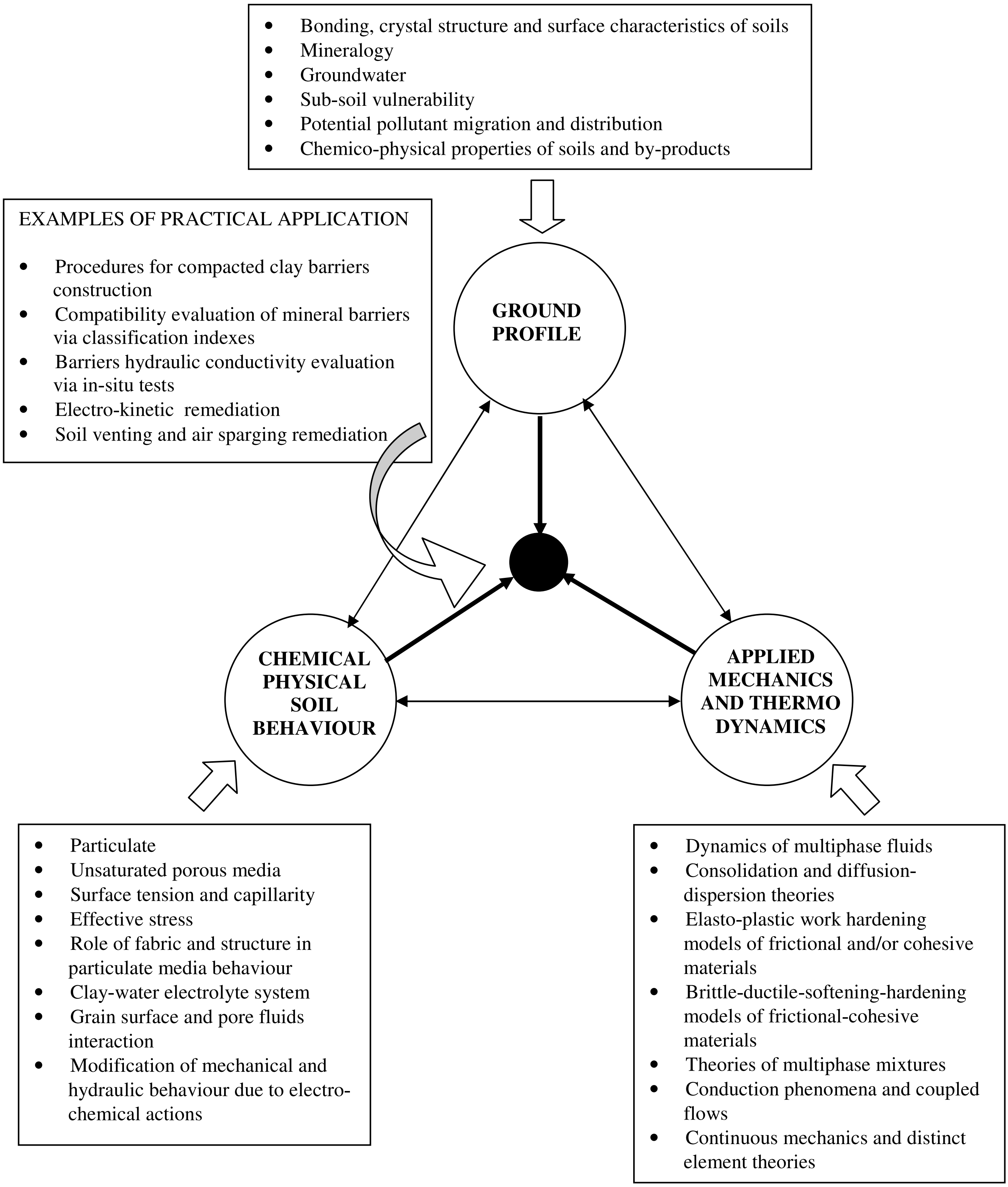


Figure 3: The Burland's triangle, modified for environmental geotechnics education.

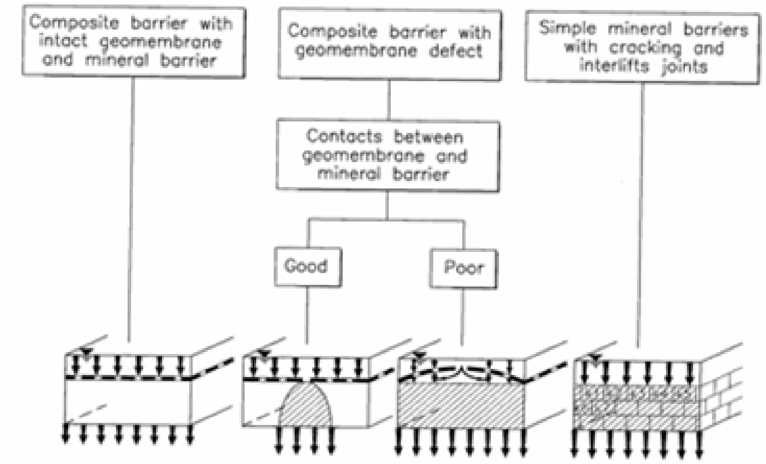


Figure 4. Advective transport through mineral and composite barriers (Demmert, 1993)

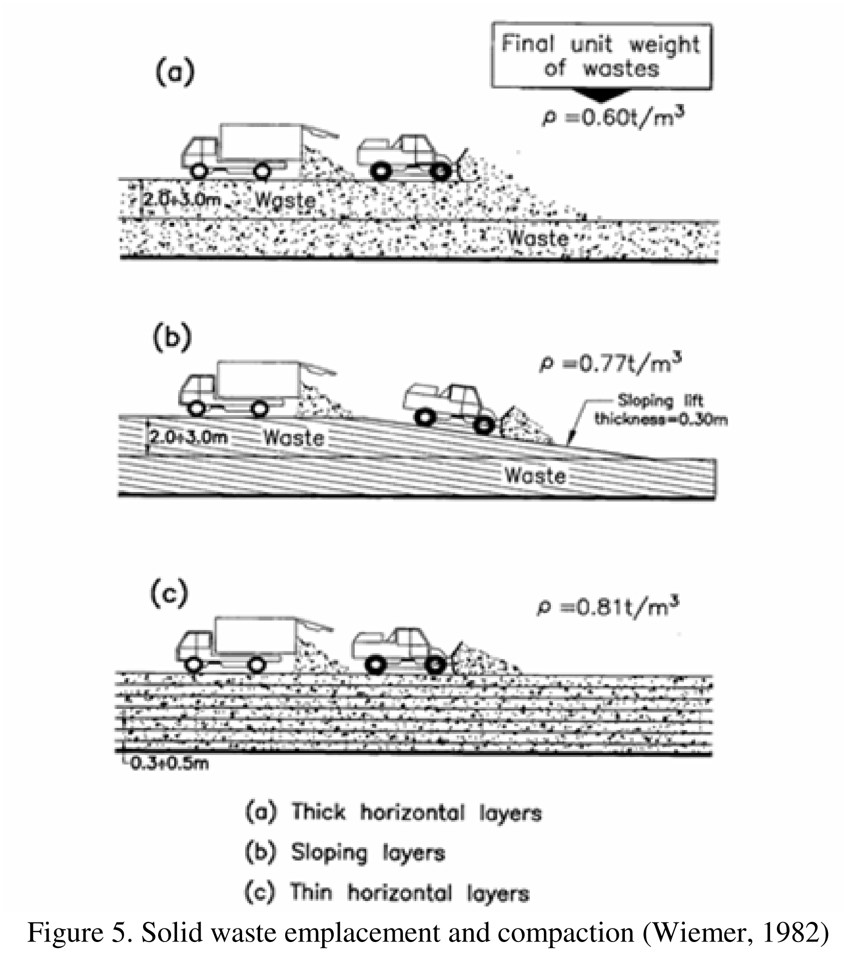


Figure 5. Solid waste emplacement and compaction (Wiemer, 1982)

Courses on "Subsoil Remediation Techniques" could contain the following basic topics:

- multi phase multi component subsurface transport processes;
- coupled flow theory and applications;
- non-aqueous-phase contaminant distribution in the subsoil;
- containment technologies (e.g. diaphragm walls, recovery wells, bottom barriers,

capping systems);

- extraction systems (e.g. pump & treat, reactive barriers, electro kinetics, air stripping, air sparging, steam extraction, use of co solvents and surfactants); and
- in situ attenuation, degradation, and immobilization (e.g. bio-remediation, vitrification, stabilization and solidification by soil mixing and injections).

Courses on "Geo-Environmental Utilization of Industrial By-Products" could contain the following basic topics:

- constitutive models for the mechanical behaviour of frictional and cemented particulate media (see, for example, Figure 6);
- chemical, biological, mechanical and hydraulic characteristics of process by-products (fly-ash, saw dust, mine spoil, furnace slag, paper sludge);
- assessment of contaminant release characteristics of process by-products via laboratory and in situ tests; and
- geotechnical and geo-environmental applications.

In addition to the mandatory subjects taught in the main environmental geotechnics education programs, some elective courses should be included in order to provide the opportunity to develop specialist knowledge on specific topics.

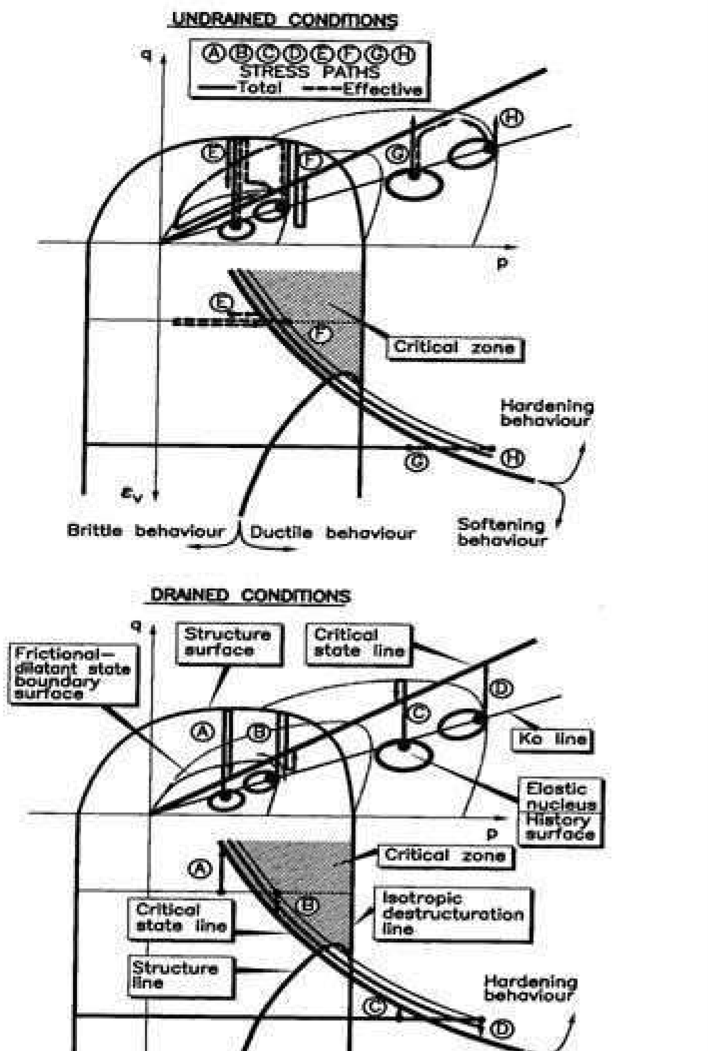


Figure 6. Conceptual modelling of the mechanical behaviour of frictional-cemented particulate materials (Manassero and Shackelford, 1994)

An example of such a course is the following on "Clay Liners and Covers for Waste Disposal Facilities" (Daniel & Trautwein, 1991), the contents of which include the following topics:

- nature of clay, classification, crystal structure, liner and cover systems, composite liners, leak rates through soil, geomembranes, and composite liners;
- leachate collection and removal systems, materials for clay liners, bentonite-soil blends, factors affecting permeability, influence of overburden stress, desiccation, frost action, required thickness of soil liners;
- manufactured clay liners i.e. geosynthetic clay liners;
- construction of clay liners and covers, concepts; equipment; pre-processing of soil, moisture control, sieving, clod control, crushing/ pulverization, compaction, test pads;
- quality assurance, tests, frequency tests, sampling patterns, criteria for water contents - density acceptability, outliners, checklist;
- laboratory permeability testing, equipment, testing procedures, testing errors, variables that influence results, standardization, keys to good testing protocol;
- field permeability tests, infiltration tests, sealed double ring infiltrometer, installation monitoring and field data interpretation, borehole tests and porous probes, Boutwell permeameter, constant-head borehole tests, Guelph permeameter, BAT permeameter, lysimeter pans;
- effects of chemicals on clay, acid, base, metals, miscible organics, immiscible organics, diluted versus concentrated liquids, reagent-grade versus real-world chemical wastes and leachates, stabilization of soils from chemical attack;
- transport of chemicals through clay, attenuation processes, advection, diffusion, effective porosity, batch adsorption tests, column tests;
- cover systems, settlement, special problems;
- water balance calculations by hand and via Computer; and
- documented case histories.

6.4 UNIVERSITY EDUCATIONAL PROGRAMS IN ENVIRONMENTAL GEOTECHNICS

This section presents the results of a survey carried out in the period 2000 to 2002 to assess the status and content of graduate educational programs in environmental geotechnics within Europe (EU), the United States (US) and Southern American (SA) universities.

In order to assess the developments in graduate education in environmental geotechnics, a specific questionnaire was developed and disseminated via email to universities in Europe and in North and South America.

The questionnaire takes into account two degree levels and is divided into two parts concerning undergraduate programs and postgraduate programs.

The first two questions refer to the structure of the academic year: quarters, semesters, annual or other; and if the education system is parallel, tree or ladder, as indicated by Manoliu (1998) and presented in Figure 7.

The participants were asked to specify the duration of studies (years), the number of credit hours required for undergraduate or postgraduate degree and specific requirements for graduation: thesis, research, practical stages or other.

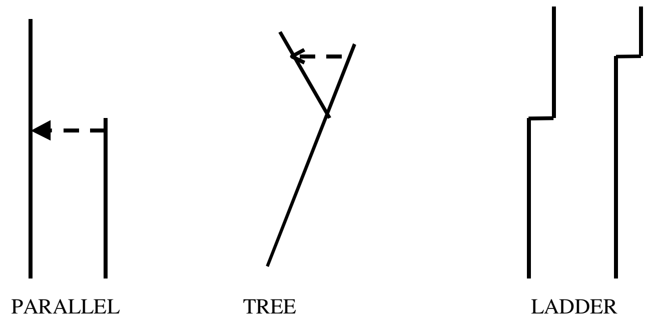


Figure 7. Different types of university education systems

Most of the European and North American universities include the specialty areas of Environmental Geotechnics within the Civil Engineering and/or the Environmental Engineering curricula as reported by Shackelford (1998), and Manassero & Spanna (2000).

The questionnaire included the following possible answers in order to assess civil engineering programs within the different universities:

- geotechnics courses are present;
- a specialty area in geotechnics is present;
- environmental geotechnics courses are present in the geotechnics specialty area; and
- a specialty area in environmental geotechnics is present;
- For environmental engineering programs the answer options were:
- geotechnics courses are present;
- environmental geotechnics courses are present; and
- a specialty area in environmental geotechnics is present;

A flow chart for the questionnaire is presented in Figure 8.

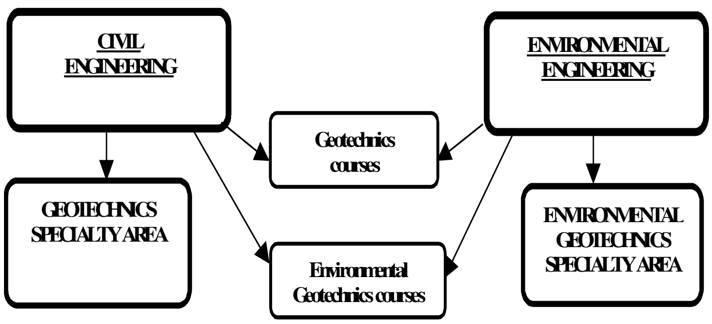


Figure 8. Flow chart of the questionnaire

The questionnaire included a list of compulsory and elective courses dealing with geotechnics and environmental geotechnics, with a short description of contents and an indication, for each course, of the semester and number of hours per week, considering lecture, laboratory work and the relative credits.

If no specific environmental geotechnics courses were present, the participants were required to indicate if some of the following topics were developed in other more general geotechnics courses:

- landfills for solid and liquid waste;
- containment barriers for subsoil pollutants;
- remediation for polluted sub soils and abandoned landfills;

- re-use of by-products for structural or other filling; and
- mechanical behaviour, settlement and stability of waste deposits.

The participants were required to list any other complementary courses within the geotechnical engineering specialty area (or program) and within the environmental geotechnics specialty area (or program). A total number of 45 questionnaires were sent by e-mail. A total of 12 responses were received from the universities listed in Table 4. The results of the survey are summarized in Table 4 and Figure 9.

Table 4: Universities Responding to the Survey on Environmental Geotechnics Education

University	**Civil Engineering Program**	**Environmental Engineering Program**	**Duration of studies and Education system**	**Structure of the Academic Year**
Aachen University of Technology (Germany)	X	X	2+3 ladder	2 semesters
Ancona University (Italy)	X	X	5 ladder	2 semesters
Bergische University-Wuppertal (Germany)	X		4+1 tree	2 semesters
Colorado State University (USA)	X		4	2 semesters
Darmstadt University of Technology (Germany)	X		3+2 tree	2 semesters
Hannover University (Germany)	X		3+2 tree-ladder	2 semesters
Joseph Fourier University Grenoble 1 (France)	X		2+3 parallel	2 semesters
Ghent University (Belgium)	X	X	3+2+1	2 semesters
Monash University (Australia)	X	X	4	2 semesters
Munchen Technische Universitat (Germany)	X		/	2 semesters
Pontificia Universidad Catolica de Chile	X		5.5 ladder	2 semesters
Torino Polytechnic (Italy)	X	X	3+2 ladder	2 semesters

It is necessary to point out that different universities organize courses on a semester or quarter system. Moreover, a very different amount of material can be supplied by different courses. For example, the credit hours per course reported in the questionnaire range from a minimum of 1.5 to a maximum of 12. In order to give some consistency to the investigation results, the number of courses has been evaluated with reference to a standard course of 6 credit hours (e.g. courses of 3 and 12 credit hours correspond to 0.5 and 2 courses respectively, in the tables of Figure 9).

From a review of the survey results, the following observations can be made:

- all the universities offer civil engineering educational programs for post graduates and, within these programs, 58% of the universities offer a specialty area in geotechnical engineering. Only one university offers a specialty area in geotechnical engineering for undergraduate education. Although this percentage seems low, all the civil engineering curricula include courses dealing with geotechnics. Furthermore, compulsory geotechnical engineering courses for all civil engineering students are available in 83% of the universities for undergraduate programs and in 58% of the universities for postgraduate programs;
- environmental geotechnics courses are available in 50 % and 58% of the universities in undergraduate and postgraduate civil engineering programs, respectively;
- as far as the geotechnical contribution to environmental engineering is concerned, only one university provides an undergraduate program, and 5 universities provide postgraduate

programs. Geotechnical engineering courses are available in all universities. Environmental geotechnics courses are present in 80% of their programs, whilst only one university provides a specialty area in environmental geotechnics;

- it is positive to observe that the geotechnical engineering course, within the environmental engineering programs, consist of at least a number of subjects, the first of which is devoted to the introduction of the basic soil mechanics principles as suggested by the TC31 guidelines on geotechnical education; and
- as far as the course contents summarized in the questionnaire by the participants, the following primary areas are covered: soil and rock mechanics, soil dynamics, foundations and retaining structures, slope stability, soil improvements, field and laboratory investigations, principles of environmental geotechnics, contaminant migration, groundwater flow and seepage, landfills and containment, remediation principles and technologies and geosynthetics.

6.4.1 University education programs in North America

A detailed survey carried out by Shackelford (1998) of second-level degree programs from 30 different universities in the United States (US) and Canada has indicated that geo-environmental engineering/environmental geotechnics represents, in the majority of cases a specialty area within traditional geotechnical engineering programs.

Of the several points investigated by Shackelford, it is interesting to note the answers of the different universities to the questions that asked to list: (i) the traditional geotechnical courses, (ii) the environmental geotechnics courses, and (iii) the courses related to environmental geotechnics but taught outside the geotechnical engineering programs.

The subjects related to the aforementioned groups are presented in Tables 5, 6 and 7. From a careful examination of the lists, the following comments can be made:

- most of the contents of the subjects taught in the geotechnical engineering programs in the US and Canadian universities can be found within the EU universities programs. It is important to observe that, in general, the US and Canadian university subjects consist of 3 credit hours instead of the 6 credit hours established by the Torino Technical University (PdT). This is the main reason why the headline definitions of EU university subjects are on average more comprehensive and less detailed than those of US and Canadian universities;
- in general, education programs in US and Canadian universities are very updated and specialized; therefore, adaptation to professional market requirements is faster than in Europe. Moreover, this flexibility is also due to the fact that the students can choose and combine a large number of subjects thereby establishing very specific education pathways;
- on the other hand, European universities (e.g. PdT) pay more attention to the basic contents and to the consistency of the education curricula. The subjects are therefore set up to provide the fundamentals of different physico-chemical phenomena, leaving the task of updating the application state of the art to specialized post graduate courses and to the professional practice;
- the differing characteristics between US and Canadian versus European university education programs, outlined in the previous points, are converging due to the increasing attention that European universities are paying to the market trends. At the same time, some US universities are reconsidering, particularly at the undergraduate level, the importance of a strong theoretical basis in a multidisciplinary field such as that of Environmental Engineering.

Table 5. Subjects related to traditional geotechnical engineering in some US and Canadian universities (Shackelford, 1998)

Category	Subject
Soil mechanics	Elementary soil mechanics
	Soil mechanics
	Shear strength and soil behaviour
	Advanced soil mechanics
Foundations	Foundation engineering
	Advanced foundation engineering
Experimental methods	Engineering properties of soils
	Engineering soil tests
	Experimental methods in soil behaviour
	Experimental methods in soil mechanics
	Experimental soil mechanics
	Laboratory characterization of geomaterials
Geotechnical engineering	Geotechnical engineering
	Advanced geotechnical design
	Advanced geotechnical engineering
Earth pressure	Earth and earth retaining structures
	Earth retaining structures
	Air photo interpretation
Other	Engineering geology
	Ground improvement
	Highway design
	In situ testing/Site characterization of geomaterials
	Numerical groundwater monitoring
	Numerical methods in geomechanics
	Seepage and drainage

6.5 SUMMARY

Environmental geotechnics is generally considered to be that branch of technology dealing with the application and adaptation of geotechnical engineering principles and expertise to land environmental engineering problems and situations (Yong, 1997). As such, the list of problems include those disasters associated with natural causes (i.e. natural disaster) and those disasters resulting principally from anthropogenic activities: floods, earthquakes, landslides, land disposal of waste, in situ confinements for pollutant control and remediation of contaminated sites (Brumund, 1995; Carrier et al., 1989; Daniel, 1993; Morgenstern, 1985; Sembenelli & Ueshita, 1981).

Typical situations or activities which have been considered as common practice in geotechnical engineering (i.e. design and construction measures against landslides) are generally not viewed as part of environmental geotechnics because they have become common activities in geotechnical engineering practice. Therefore, environmental geotechnics issues involve engineering activities and processes related to the pollution of engineered soils, namely:

- solid and liquid waste landfills;
- polluted subsoil and abandoned landfills;
- re-use of by-products; and
- in situ improvement of the mechanical behaviour of wastes.

Environmental geotechnics can be considered a discipline that moves from traditional geotechnics.

Classical concepts and principles of geotechnics do not fail when applied to soil-waste systems; however, they must be applied with the understanding that concurrent processes and interactions must be taken into account. It is the reactions and interactions among materials that most complicate analyses and that introduce the greatest uncertainty into results.

Education in environmental geotechnics must consider the aforementioned aspects, in order to provide to engineers the necessary knowledge to properly address and solve soil pollution problems.

From a consideration of the pertinent factors and subject areas covered by environmental geotechnics, the Burland triangle, originally developed to frame and address the traditional aspects of a geotechnical engineering education, has been modified to include additional elements that together provide the key components of an environmental geotechnics education.

On the basis of the previous considerations and considering the environmental problems common to most industrialised countries, a complete environmental geotechnics education program, with the specification of the courses and their contents, has been presented.

In order to assess the "state of the art" of environmental geotechnics education in European, North American, and South America universities, a survey has been conducted. The survey consisted of a questionnaire disseminated via email to Universities. The results of this work are encouraging; in fact they show the relevant presence of geotechnics courses in civil and environmental engineering university programs. Although only one EU university, among the surveyed, provide a specialty area in environmental geotechnics, nevertheless environmental geotechnics courses are available in all of these university programs, with an average number of 4 courses in civil engineering postgraduate programs and 3 in environmental engineering postgraduate programs.

Finally, on the basis of a previous survey conducted by in the US, it was observed that education programs in the US are regularly updated and specialised, whilst EU university programs pay more attention to the basic contents and to the consistency of education curricula.

Table 6. Geoenvironmental engineering subjects taught within the Geotechnical Engineering programs of some US universities (Shackelford, 1998)

Category	**Subject**
Groundwater flow and seepage	Groundwater hydrology/Modelling
	Groundwater and seepage
	Hydrogeology
Landfill and containment	Landfill design
	Waste containment
	Waste management
General course	Environmental geotechnics
	Geoenvironmental engineering
In situ applications	Soil improvement/Ground modification
	In situ testing
Contaminant migration	Contaminant transport/Modelling or hydrogeology
Geosynthetics	Geosynthetics
Remediation technologies	Remediation
Geology	Geophysics
Other geotechnical and geology	Engineering soil properties
	Filtration and drainage
	Soil behaviour
	Unsaturated soils
	Applied geotechnical analysis
	Industrial by-product utilization
	Intermediate soil mechanics
	Stability of waste fills
	Uncertainty in geologic environment
Other	Environmental engineering
	Soil physics laboratory
	Surface water
	Water in the environment

Table 7. Geoenvironmental engineering subjects taught at some US universities outside the Geotechnical Engineering program (Shackelford, 1998)

Category	Subject
Environmental or Chemical Engineering	Hazardous, industrial municipal or solid wastes
	Aqueous or environmental chemistry
	Microbiology, biological processes, bioremediation
	Environmental laboratory
	Treatment and separation
	Unit operation and processes
	Remediation
	Sanitary/Environmental analysis
	Transport/Reaction analysis
	Environmental behaviour organic pollutants
	Environmental monitoring and sampling
	Environmental toxicology
Hydrology and Hydraulics	Contaminant transport (Migration)
	Groundwater hydrology/Hydraulics/Modelling
	Surface water hydrology
	Hydraulics
	Fate and contaminant in subsurface
	Karst hydrology
	Non-point pollution
	Stochastic hydrology
	Water resources systems
Geology	Hydrogeology
	Geophysiscs
	Engineering geology
	Geochemistry/Groundwater chemistry
	Geomorphology
	Glacial geology
	Environmental geology
Soil sciences	Soil chemistry
	Soil physiscs
	Soil microbiology
Chemistry	Organic chemistry
	Thermodynamics
Analysis	Finite element methods (FEM)
	Geographic information systems (GIS)
	Satellite imagery
Probability and statistics	Risk assessment
	Applied probability
Other	Marine pollution
	Physicochemical processes
	Soil classification
	Soil and groundwater pollution
	Waste containment

UNDERGRADUATE EDUCATION PROGRAMS CIVIL ENGINEERING 6/12 (50%)

SPECIALTY AREA IN GEOTECHNICAL ENGINEERING
available in 1/6 (16%)

GEOTECHNICAL ENGINEERING COMPULSORY COURSES FOR ALL CIVIL ENGINEERING STUDENTS
available in 5/6 (83%)

ENVIRONMENTAL GEOTECHNICS COURSES
available in 3/6 (50%)

Total number of available Geotechnical Engineering courses for each education program
Min: 2 Max 6
Total number of available Environmental Geotechnics/ Engineering courses for each education program
Min: 0 Max 2

POSTGRADUATE EDUCATION PROGRAMS CIVIL ENGINEERING 12/12 (100%)

SPECIALTY AREA IN GEOTECHNICAL ENGINEERING
available in 7/12 (58%)

GEOTECHNICAL ENGINEERING COMPULSORY COURSES FOR ALL CIVIL ENGINEERING STUDENTS
available in 7/12 (58%)

ENVIRONMENTAL GEOTECHNICS COURSES
available in 9/12 (75%)

Total number of available Geotechnical Engineering courses for each education program
Min: 3 Max 12
Total number of available Environmental Geotechnics/ Engineering courses for each education program
Min: 0 Max 3

UNDERGRADUATE EDUCATION PROGRAMS ENVIRONMENTAL ENGINEERING 1/12 (8%)

GEOTECHNICAL ENGINEERING COURSES
available in 1 University

SPECIALTY AREA IN ENVIRONMENTAL GEOTECHNICS
not available

ENVIRONMENTAL GEOTECHNICS COURSES
not available

Total number of available Geotechnical Engineering courses for each education program
Min: 0 Max 3
Total number of available Environmental Geotechnics/ Engineering courses for each education program
Min: 0 Max 0

POST GRADUATE EDUCATION PROGRAMS ENVIRONMENTAL ENGINEERING 5/12 (42%)

GEOTECHNICAL ENGINEERING COURSES
available in 5/5 (100%)

SPECIALTY AREA IN ENVIRONMENTAL GEOTECHNICS
available in 2/5 (40%)

ENVIRONMENTAL GEOTECHNICS COURSES
available in 4/5 (80%)

Total number of available Geotechnical Engineering courses for each education program
Min: 2 Max 7
Total number of available Environmental Geotechnics/ Engineering courses for each education program
Min: 1 Max 5

Figure 9. Results of the survey questionnaire

REFERENCES

Brumund W.F. (1995). "Environmental Geotechnology-Examples Of Where We Are And How We Got Here". Proc., *Geoenvironment 2000,* GSP no. 46, ASCE, New York.

Burland J. B. (1987). Nash lecture: The teaching of soil mechanics - A personal view. *Proc. 9th ECSMFE*, Dublin, Balkema, Rotterdam.

Carrier W.D.III, De Mello L.G., Moh Z.C. (1989). "Environmental Impact in Geotechnical Engineering". Special Lecture A. Proc., *12th ICSMFE,* Rio de Janeiro, Brazil, Balkema, Rotterdam.

Daniel D.E. (1993). "Geotechnical Practice for Waste Disposal". Chapman & Hall, London.

Daniel D.E., Trautwein S.J. (1991). "Clay Liners and Covers for Waste Disposal Facilities". *Short Course,* The University of Texas at Austin, Austin.

"Declaratation conjointe sur l'harmonisation de l'architecture du systeme europeen de l'enseignement superieur par les quatres mininstres en charge de l'enseignement superieur en Allemagne, en France, en Italie et au Royame Uni". Paris, en Sorbonne, le 25 mai 1998.

Demmert S. (1993). "Analyse des Emissionsverhaltens einer Kombinationsabdichtung im Rahmen der Risikobetrachtung von Abfalldeponien". *Dissertation,* Schriftenreihe des Instituts fur Grundbau der Ruhr-Universitat Bochum. Heft 21

Fang H.Y. (1997). "Introduction to Environmental Geotechnology". CRC Press

Koerner R.M. (1987). Keynote Address. Proc., *1st Int. Symp. Environ. Geotechnol.,* v. 2, Envo Publ. Co. Bethlehem, PA.

Manassero M., Shackelford C.D. (1994). "Classification of Industrial Wastes for Re-Use and Landfilling". Proc., *1st ICEG*, Edmonton, BiTech Publishers, Richmond B.C.

Manassero M., Van Impe W.F. and Bouazza A. (1997). Waste disposal and containment. State of the art report. *Proc. 2nd t ICEG*, Osaka, Balkema, Rotterdam.

Manassero M. (1998). "Education in environmental geotechnics – European practice". *Proc. 3rd ICEG*, Lisbon, Balkema, Rotterdam.

Manassero M. and Spanna C. (2000). Basic aspects of Environmental Geotechnics education. *Proc. Geotechnical Engineering Education and Training,* Sinaia, Balkema Rotterdam.

Manoliu I. (1998). "Education in Geotechnical Engineering". *Workshop* at the Politecnico di Torino.

Morgenstern N.R. (1985). "Geotechnical Aspects of Environmental Control". Proc., *11th ICSMFE,* San Francisco, USA, Balkema, Rotterdam.

Poulos H.G. (1999) Geotechnical Education toward 2000. *Proc. 12nd ICSMFE* , Hamburg, Balkema, Rotterdam.

Reible D. (1999) Fundamentals of Environmental Engineeering, Springer, Lewis Publishers.

Reddi L.N. and Inyang H.I. (2000) Geoenvironmental Engineering Principles and Applications. Marcel Dekker, Inc.NY.

Sembenelli P., Ueshita K. (1981). "Environmental Geotechnics-State-of-the-Art Report". Proc. *10th ICSMFE,* Stockholm, Balkema, Rotterdam.

Shackelford C.D. (1998). "Educational programs in Environmental Geotechnics/Geoenvironmental Engineering at USUCGER universities". Proc. 3rd ICEG, Lisbon, Balkema, Rotterdam.

Technical Committee no. 5 "Environmental Geotechnics" (1998). "First report". Proc. Proc. 3rd ICEG, *Environmental Geotechnics* Lisbon, Balkema, Rotterdam.

Uehara H., Sueoka T. and Ueshita (1997) Environmetal impact assessment for geotechnical problems *Proc. 2nd ICEG,* Osaka, Balkema, Rotterdam.

Wiemer K. (1982)."Qualitative und quantitative Kriterien zur Bestimmung der Dichte von Abfallen in geordneten Deponien". *Dissertation,* TU Berlin.

Yong R.N. (1997) Multidisciplinarity of Environmental Geotechnics, *Proc. 2nd ICEG,* Osaka, Balkema, Rotterdam.

Chapter 7

Nuclear Waste

Author:

Antonio GENS
Technical University of Catalonia, Spain

Contributing Author:

Hywel R. THOMAS
University of Wales, Cardiff, United Kingdom

7.1 INTRODUCTION

Issues relating to nuclear waste disposal are, in many ways, similar to those associated with the disposal of more conventional waste. The amount of nuclear waste is in fact small when compared with the existing quantities of all other wastes. It is also true, however, that nuclear waste has some particular features that may possibly require specific solutions. The following can be mentioned:

- the number of relevant interacting phenomena is high, resulting in the need of considering systems of high complexity
- there is little chance of remedial actions, so final design concepts should be established with a high degree of confidence
- the long design life of the disposal schemes (tens of thousands of years and more) is well beyond most projects in conventional civil or geoenviromental engineering

Disposal of radioactive waste has become a contentious public issue and this fact places the technical problems in a context somewhat different form those of other areas of environmental engineering. Perhaps partly due to this, there has been a large amount of international research carried out in this area. In fact, in many ways, scientific knowledge related to this waste disposal topic has outstripped advances in other waste disposal fields. It has sometimes been argued that many of the approaches now prevalent in the nuclear waste field should be usefully transferred to the management of other types of toxic and dangerous wastes.

The solution of the problems involved in nuclear waste disposal often requires strong geotechnical input. Much of the geotechnical work in this field of application is disseminated in publications issued by the National Agencies for Nuclear Waste and by International Organisations such as the International Atomic Energy Agency (United Nations), the Nuclear Energy Agency (OECD) and the European Commission. There are also specific Conferences and Symposia devoted to nuclear waste disposal that also present interesting geotechnical work (American Nuclear Society, 1998; Materials Research Society, 1997; Canadian Nuclear Society, 1996).

The Chapter starts considering the specific features of nuclear waste and available disposal strategies. Then the main issues associated with deep geological disposal of radioactive waste are discussed with special attention to topics related to construction, the effects of excavation on the surrounding rock and safety assessment analysis. The chapter closes outlining the procedures for the disposal of intermediate and low level nuclear waste and the remediation of uranium mining sites.

7.2 NUCLEAR PROCESSES AND NUCLEAR WASTE

7.2.1 Nuclear processes and radioactivity

The two main nuclear processes that may give rise to energy production are fission and fusion. In nuclear fission, a heavy nucleus captures a neutron and, as a consequence, splits in two releasing a number of additional neutrons and large amounts of energy. The use of this chain reaction in a controlled manner is the basis for energy production. In nuclear fusion, two light nuclei join together to form a heavier helium nucleus and a large amount of energy is released. However, and in spite of a large research effort invested in fusion, nuclear energy production must rely on fission for the foreseeable future.

According to the IAEA (International Atomic Energy Agency), at the end of 2005 there were 443 operating nuclear reactors (total power 369,552 MW) and 24 under construction. The latest available figures (2003) show that nuclear power accounted for an electricity production of 2525 TWh, about 16 % of the total electricity generated in the world (and about 24% in OECD countries). This proportion has remained basically constant since the late eighties. The increased nuclear power output has largely arisen from enhanced plant productivity rather than from new facilities coming into operation. Concern about climate change associated with the emission of "greenhouse" gasses has recently brought back nuclear power as a potentially valuable energy source for the future (NEA, 2002a).

Radioactivity is often associated with the use of nuclear power but it is also a natural phenomenon arising from the instability of the nuclei of some elements in the earth crust and from the effects of cosmic rays on the atmosphere and solid earth. Those radionuclides will spontaneously transform to other nuclides that, in turn, may or may not be stable. The transformation involves radiation emissions and, sometimes, fission of the nucleus. The main types of radiation emitted are: α (helium nuclei), β (electrons), $\beta+$ (positrons), n (neutrons) and γ (high-energy electromagnetic radiation). Radioactive substances decay over periods of time that may vary enormously depending on the type of element, some are extremely short lived (halftimes of the order of less than a millionth of a second) and others are very long lived (halftimes of the order of millions of years). In the last sixty years or so, human activity has added radioactive sources to the already existing natural ones. They range from fallout from explosions of nuclear weapons to more mundane applications such as energy production or medical exploration and treatments.

There are a number of units and parameters that can be used to measure variables associated with radioactive phenomena. For instance, the 'activity' is measured in Bequerel (Bq). One Bq corresponds to one disintegration or decay event per second. For practical purposes, however, the most important variables are related to the effects of radioactive radiation on living bodies. The amount of ionising radiation (or 'dose') received by a person is measured in terms of energy absorbed in the body tissue and is expressed in Gray (1 Gray is one joule deposited per kilogram of mass). However, equal exposure to different types of radiation does not necessarily produce equal biological effects (for instance α and neutron radiation are considered to be twenty times more damaging than β or γ radiation). Generally, those effects are expressed in terms of "effective dose equivalent" measured in Sievert (Sv). To provide an order of magnitude, a typical annual effective dose equivalent from natural sources of radiation lies in the range of 1.5 - 4 mSv/year although there are very large differences from one location to another; some sites exhibiting natural radiation of 40 mSv/year and more. At sea level, about 0.3 mSv/year dose comes from cosmic rays and this value roughly doubles every time that elevation increases by 1,500 m. Also, the average annual effective dose equivalent arising from medical sources is reckoned to be around 0.4 mSv (UNEP, 1985). The Sievert has replaced the former dose unit, the rem (radiation equivalent man), 1Sv equals 100 rem.

7.2.2 Sources and types of nuclear waste

The main source of nuclear waste is the production of electricity power by means of nuclear fission. Nuclear power generation must not be considered in isolation but as part of the entire nuclear cycle that involves, at least, the following operations:

- extraction of mineral ores containing suitable fissile materials, the most common by far is uranium
- chemical purification, isotope enrichment and fuel manufacturing
- reactor operation
- spent fuel management.

All those activities give rise to radioactive wastes that must be disposed in an adequate way. Perhaps the most difficult problem is concerned with spent fuel management and disposal. Countries adopt a variety of nuclear fuel management strategies that can be grouped in three different cycles (Figure 1):

- Open cycle. Spent irradiated fuel is considered as waste and is not subject to reprocessing. Spent fuel is disposed of directly. Before disposal, however, spent fuel must be stored in temporary facilities during 40-50 years in order to achieve the required cooling of the irradiated fuel and, also, to allow time to build the final repository. Some countries plan to increase the period of temporary storage to longer terms (50-100 years).
- Present closed cycle. In this case the spent irradiated fuel is not considered as waste, spent fuel is reprocessed to utilize part of its radioactive content, fundamentally uranium and plutonium. The resulting fuel (MOX) is then used again in nuclear reactors for power generation. Reprocessing produces large volumes of waste. Some of this waste is a highly radioactive liquid that must be properly conditioned before disposal. Vitrification is the favoured conditioning method.
- Advanced closed cycle. In this case not only uranium and plutonium are separated but the rest of long-lived fission products as well. The idea is to use transmutation to transform the long lived and highly toxic radionuclides into new ones of lower mass and shorter lifetimes, reducing in this way their toxicity. Advanced closed cycle strategies are currently in a research phase and no industrial-scale facility is operational at present.

Significant amounts of radioactive waste also arise from dismantling nuclear reactors, an activity that will increase sharply in coming years due to the achievement of the planned lifetimes of power plants. Additional sources of nuclear waste are military nuclear activities and the use of radionuclides in medicine, research and industry. These types of waste are not that dissimilar from those arising from electrical power generation and the planned disposal methods are in fact the same. In contrast, the treatment of uranium mine waste is quite specific and is discussed at the end of the chapter.

For waste management purposes, it is usual to distinguish three different waste types:

- High Level Waste (HLW). High activity wastes that, in addition to many short half-life radionuclides, also contain large amounts of long-lived radionuclides. They are also strong heat emitters. Usually they are constituted by spent fuel rods or solidified high-level waste from reprocessing. Although they contain most of the radioactivity (over 95 % of the total), the waste quantities are relatively low, of the order of 10,000 tonnes per year at present (Mc Combie et al., 2000). As indicated above, HLW waste is generally left to cool off before disposal for a number of years. After 40 years, the level of radioactivity goes down to about one thousandth its initial value.
- Medium or Intermediate Level Waste (MLW or ILW). This is an intermediate class of waste that contains non-negligible amounts of radioactivity and normally requires shielding. They usually result from enrichment and fuel fabrication, reactor operation, reprocessing, and nuclear plant decommissioning. Heat emissions are low. A special type of intermediate level waste is that containing Long-Lived radionuclides (LL-ILW). They arise mainly from reactor operations, from reprocessing spent fuel and from decommissioning nuclear facilities. LL-ILW is also a product of producing and dismantling nuclear weapons. Because of the long lives of the radionuclides involved, disposal of II-ILW will be generally similar to that of HLW.
- Low Level Waste (LLW). Low activity waste is normally associated with radionuclides of short half-life and comprises the bulk of waste of the nuclear fuel cycle. In fact, all nuclear activities, not only power generation, generate amounts of LLW in significant quantities. Also, wastes resulting from mining and ore processing normally belong to this category. Worldwide they make

up 90% of the waste volume but contain only 1% of the total radioactivity of all radioactive wastes.

It should be stressed that there is no internationally agreed standard about classification of wastes in this very small number of categories. Often this classification is associated with the type of disposal planned (or even with national legal provisions) and, so, a number of differences between various countries arise. Having said that, the above classification is useful as a ready way to relate a waste to the specific problems arising from their disposal and the qualitative definitions given above are sufficient for this purpose.

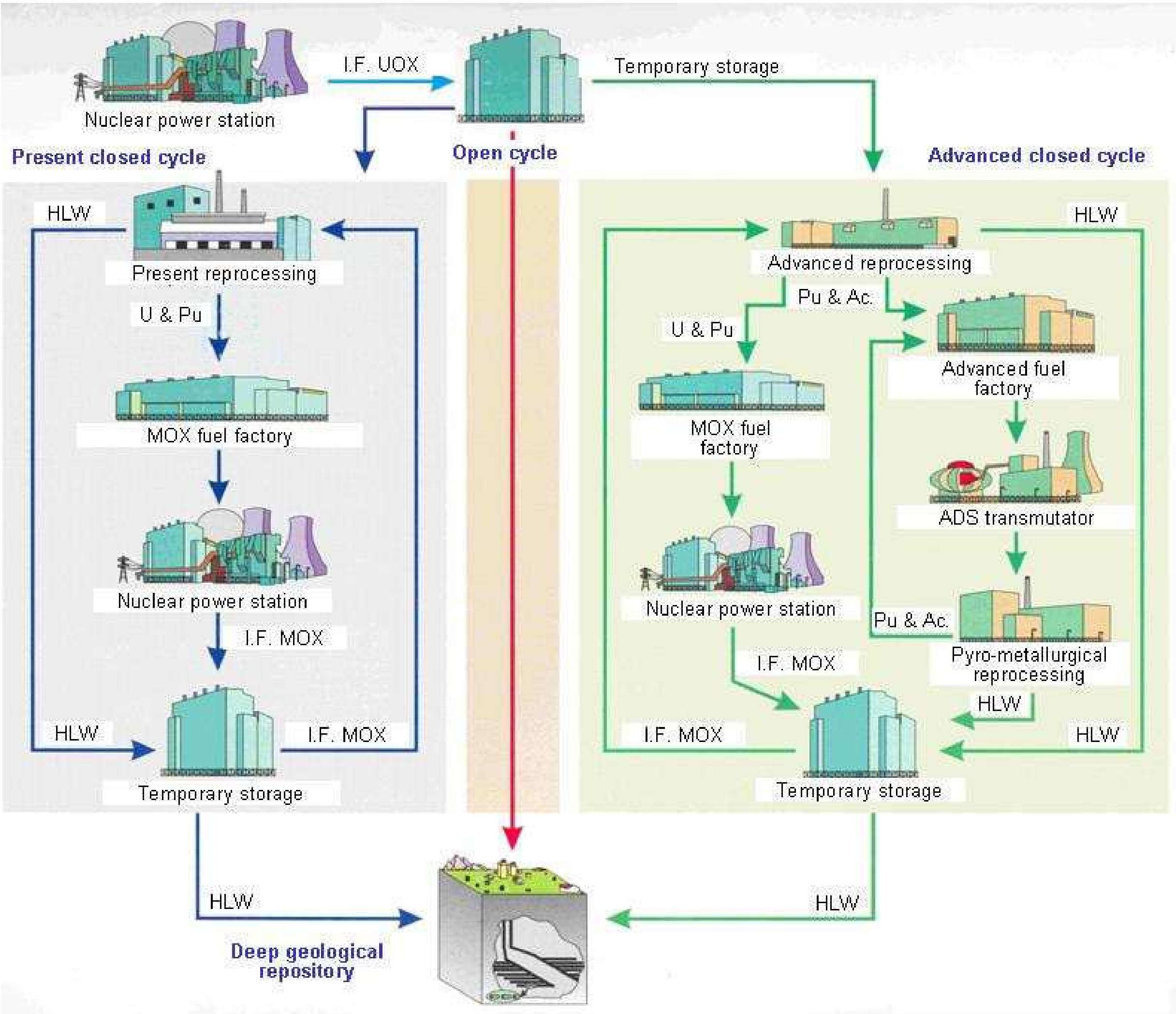


Figure 1. Nuclear fuel cycles and disposal requirements (adapted from Astudillo, 2001).

7.3 DISPOSAL OF HIGH LEVEL NUCLEAR WASTE

7.3.1 Disposal strategies

Generally the management or disposal of high level nuclear waste (HLW) involves issues quite different from those associated with the storage of intermediate level (ILW) and low level waste

(LLW), so they are considered separately in this chapter. Although some countries are planning to store together all waste types, most policies envisage solutions for HLW different from those adopted for LLW and ILW. An exception is LL-ILW that tends to be assimilated to HLW for disposal purposes.

As for any other waste management problem, there are two basic strategies to approach the problem:

- Dilution/dispersion of contaminants
- Concentration and isolation of contaminants

Both strategies have been considered although, at present, the second one is dominant. A role for the dilution/dispersion case is, however, also envisaged, especially for the long-lived radionuclides for which a sufficiently long period of isolation may be impossible to ensure.

Any disposal system should in principle guarantee that:

- the waste is removed from the human environment,
- the waste is isolated and contained over long periods of time (depending on waste type),
- only small release rates will occur once the complete isolation period is over.

A variety of procedures have been put forward to solve the problem of radioactive waste disposal (e.g. BNWL, 1974): a) space disposal, b) ice sheet disposal, c) ocean bed disposal, d) disposal beneath the seabed, e) nuclear transmutation, and f) geological disposal

Currently, only options e) and f) are the subject of intense research activity. Transmutation of the most harmful long-lived radionuclides does offer the possibility to reduce the requirements applied to a long-term disposal facility. This approach requires carrying out chemical separation of very radioactive materials, going well beyond present reprocessing activities. Whatever the result of this research, there will always remain significant quantities of waste to be disposed of in some other fashion.

Geological disposal was proposed nearly 50 years ago (NAS, 1957) as the most suitable technical solution to ensure satisfactory environmental protection and it is still widely recognized as the only viable approach for an ultimate solution that avoids the need of ensuring a permanent safe managed storage (NEA, 1999; NRC, 2001). Naturally, geological disposal requires the strongest geotechnical, input.

7.3.2 Fundamentals of deep geological disposal

The aim of geological disposal of radioactive waste is to remove it from human environment and to ensure that any radionuclide release rates remain below prescribed limits (Chapman and Mc Kinley, 1987). As indicated above, some countries are also considering deep geological disposal of all waste types, including LLW, because they consider the additional cost involved is compensated by the perceived enhanced safety of deep geological disposal.

Two types of deep geological disposal have been considered for nuclear waste, mined repositories or deep boreholes drilled from the surface. The mined repository will be considered here. It is by far the possibility most likely to be implemented although some countries have undertaken some research on the borehole alternative. In any case, the phenomena that occur and that must be examined are similar for the two cases.

Figure 2 shows a typical scheme for an underground mined repository. It involves the sinking of deep shafts down to a depth of several hundred meters. The depth will, of course, be controlled by local geological conditions. The shafts provide access to a network of horizontal drifts that constitute the main repository area. Part of those drifts will be access tunnels and part will be devoted to nuclear waste disposal. As Figure 3 shows, many options are contemplated regarding canister emplacement. A possibility is to place them in the horizontal drift itself but other options put them inside boreholes drilled from the main access tunnels. The boreholes may be vertical or horizontal.

To provide more detail, Figure 4 shows a scheme of a disposal drift in the present Spanish reference system for disposal in granite. A concrete plug separates the disposal area from the access tunnel. The space between canisters and the host rock is normally filled by a suitable material to constitute an engineered barrier. The material most usually considered is compacted swelling clay, normally some kind of bentonite on its own or mixed with other materials like sand. However, cement-based materials (special concretes) and crushed salt (for repositories in salt rock) are also being considered for some specific applications.

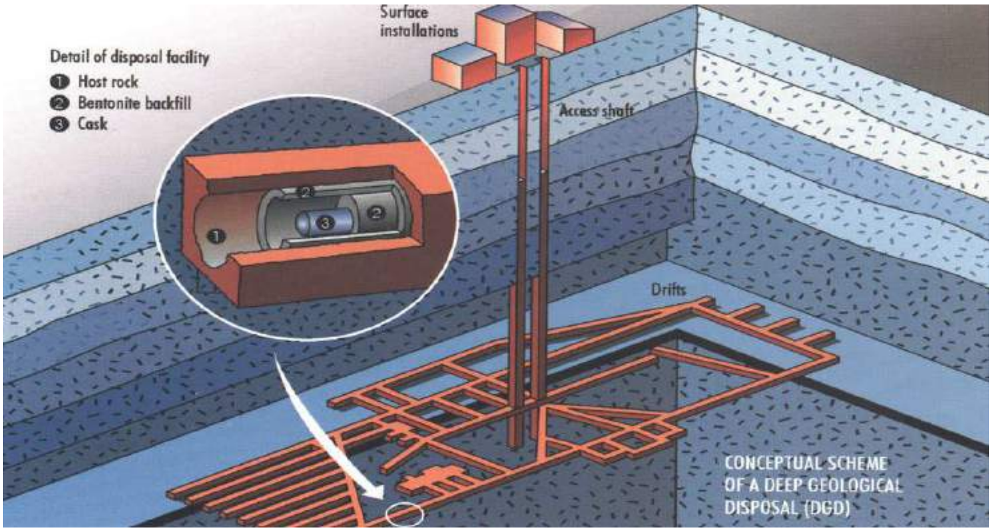


Figure 2. Conceptual scheme of a deep geological repository for nuclear waste

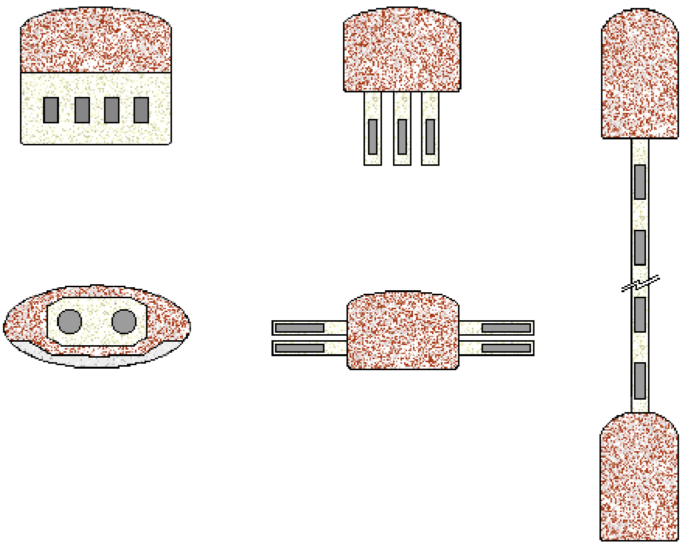


Figure 3. Options for emplacement of canisters in a deep underground repository

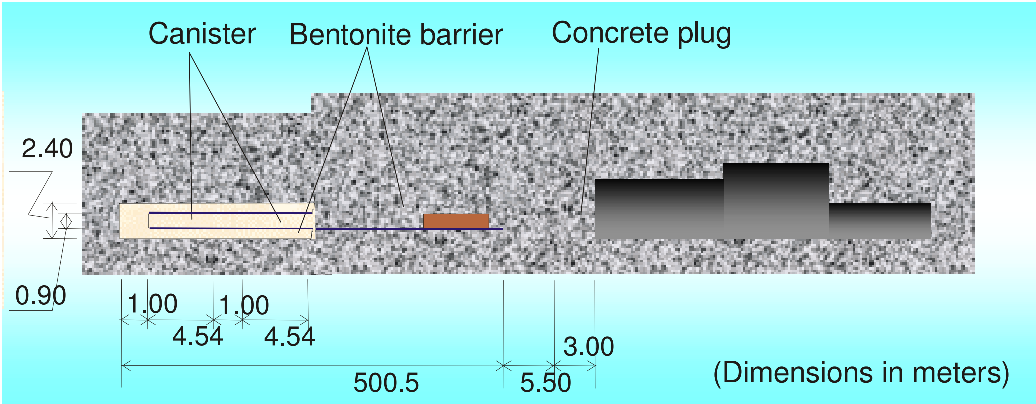


Figure 4. Longitudinal section of a disposal drift in the Spanish reference concept for a deep geological repository in granite

7.3.3 Multi-barrier concept

All designs for deep geological disposal of HLW adopt the multi-barrier concept to achieve the required degree of waste isolation. There are two main components of the multi-barrier system:

- the engineered barrier system (EBS) comprising the solid waste matrix, the container and the backfill. Sometimes the EBS name is reserved exclusively for the backfill surrounding the waste
- the natural barrier, i.e. the host rock and groundwater system surrounding the repository.

Each one of the elements in the multi-barrier scheme will contribute to the safety of the overall disposal system. Originally it was thought that each barrier should be designed in such a way to provide sufficient isolation on its own, so that a simultaneous failure of all barriers would be required for significant radioactive releases to occur. In fact, this is too restrictive and, in cases involving long-lived wastes, possibly impossible to achieve. It is more realistic to consider all the barriers acting together in a single repository system.

Figure 5 shows a scheme of the multi-barrier system for a deep repository of HLW waste using a bentonite barrier as backfill. Ideally, each barrier should provide favourable conditions for isolation and controlled release of radionuclides. Thus, in the case of vitrified HLW waste, the glass should exhibit low corrosion rates and a high resistance to radioactive action. It should also ensure a homogeneous distribution of radionuclides. If the waste is composed of spent fuel rods, the UO2 should have a low solution rate and high stability in front of radioactive and thermal effects. The next barrier is the metal waste container. It should guarantee isolation for a few thousand years and, also, the corrosion products should provide a favourable chemical environment for the waste. In some designs, very long-lived canisters (often copper or titanium based) are envisaged that could prolong the isolation period to tens of thousands of years.

The bentonite barrier fulfils several important functions. In the first instance, a very low hydraulic conductivity restricts water penetration and retards significantly solute transport due to its low diffusion coefficient and to additional sorption effects. It should also provide a favourable chemical environment and be able to self-heal if subjected to physical perturbation such as cracking and fissuring events.

Regarding the geological barrier, it is convenient to distinguish between the host rock itself, where the repository is located, from the rest of the geosphere. The host rock should guarantee a favourable geochemistry, that there is only a limited supply of water, and should provide, in addition, mechanical and geological stability. The geosphere, in turn, should ideally ensure long flow times to the biosphere, additional retardation and dilution of radioactive material (via sorption and matrix diffusion) and long term stability of hydrogeological conditions. It should be remembered that any delay in radionuclide transport reduces the problem because of radioactive decay.

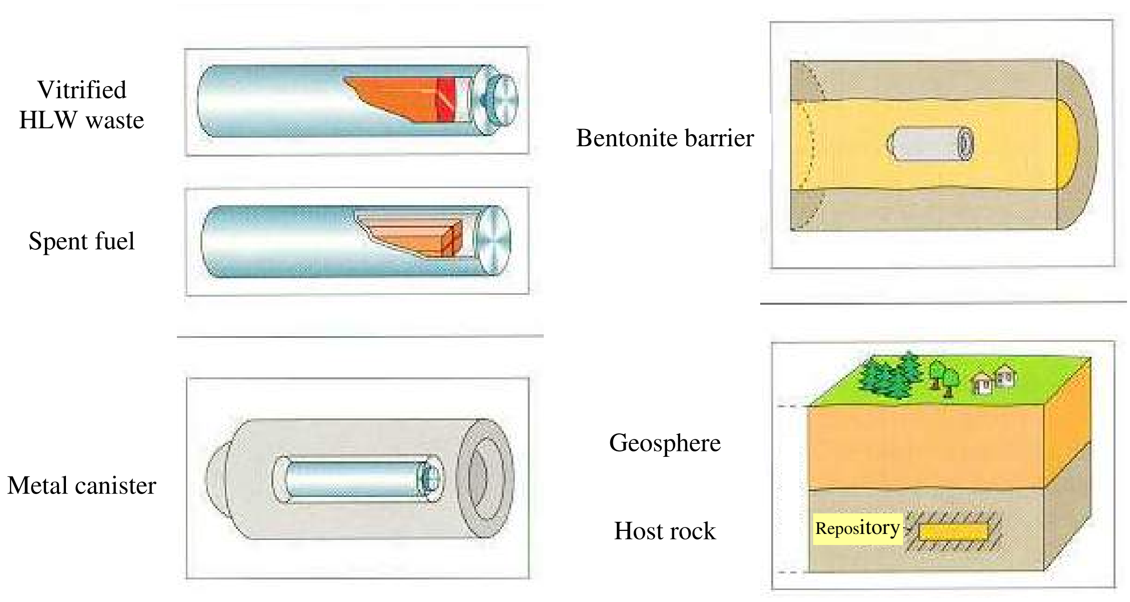


Figure 5. Multi-barrier concept for a deep geological repository for HLW

7.3.4 Host rocks

The safety of a nuclear waste repository is closely related to the characteristics of the host rock where it is built. An important feature of the geological barrier is that, while the engineered barrier can be designed and manufactured to specifications, no action is possible (except in a very limited sense) on the geological barrier; so selection and characterization are of paramount importance. The main functional requirements for the geological barrier are:

- to protect the various engineered barriers, ensuring stable hydraulic, mechanical and geochemical conditions
- to achieve a small, slow and stable water flow in the vicinity of the repository
- to retard as much as possible the migration of the radionuclides
- to permit the construction and operation of the repository
- to protect the repository against human intrusion

To achieve those requirements, the geological host formations should ideally exhibit:

- sufficient thickness, depth and extension to isolate the repository from potentially disruptive natural processes or undesired human activities
- tectonic stability (absence of active faults) and low seismic activity
- small structural complexity
- lithological homogeneity
- low permeability and hydraulic gradient
- adequate retention properties for radionuclides

The range of rock types that meet those conditions is very wide. The most extended ones are crystalline rocks, argillaceous rocks (plastic clays and indurate mudrocks) and rock salt. All of them are currently being investigated in this context.

Crystalline rocks have the important advantaged of exhibiting low permeability (if the fracture frequency is low), high chemical stability and low economic value. In addition, they are high strength rocks; therefore excavation should be relatively straightforward with only a limited amount of support required. It must be said, however, that sometimes these rocks are located in tectonic zones and that the material is brittle, so they sustain fractures that constitute the main pathways for water flow and radionuclide transport. Characterizing the fracture network and its interconnectivity may be a very tough challenge. Excavation may cause additional fractures that have no capacity for self-healing. This type of rock is being considered by many countries in Europe (i.e. Czech Republic, Finland, Sweden, France, Spain, Sweden, Switzerland, Ukraine) and elsewhere (i.e. Canada, China, India).

Argillaceous rocks may be plastic clays or indurated mudrocks. As a matter of fact, their behaviour concerning possible radionuclide transport is quite different. Thus, plastic clays may tend to self heal discontinuities that may appear during construction (Barnichon and Volckaert, 2002) or at other stages of the repository whereas indurate mudrocks may sustain fractures that remain open for long periods of time. In any case, argillaceous rocks have a very low permeability and, in addition, significant retardation properties for solute transport. An advantage is that argillaceous rocks have no economic value. Often their strength is not high; therefore significant amounts of support may be required during excavation and construction. It is expected that fractures will be rare in plastic clays as they have significant self-healing properties. Indurated clays are more likely to undergo fracturing during excavation. Their capacity for self-healing is uncertain; it probably depends on the amount of swelling minerals that they contain and on their degree of cementation. Argillaceous rocks are also more sensitive to chemical changes of various types. An important one is oxidation occurring during the ventilation period of the repository. Argillaceous rocks are being investigated in Belarus, Belgium, France, Hungary, Spain and Switzerland.

Finally, salt rock has also a very low permeability and, because of its very high creep rate, the material is largely self-healing. Salt rock has some economic value but it is not high. A repository built in salt rock would be vulnerable to fresh water entry and openings may require some support. Countries exploring this possibility are: Belarus, Denmark, Germany, Netherlands, and Ukraine.

There are other types of rock that have also been considered at some point. The most important case is unsaturated tuff, the rock at the proposed US HLW repository in Yucca Mountain, Nevada. The lists of countries considering the various host rock types has been taken from Whiterspoon (1996) modified with more recent information. It should be stated, however, that this is an area of public policy where priorities and objectives are prone to abrupt changes; therefore that information should be taken as provisional and subject to modification.

7.4 CONSTRUCTION OF A DEEP GEOLOGICAL REPOSITORY

7.4.1 Site investigation and construction

The site investigation for a repository will try to confirm the characteristics required for a suitable geological host formation listed in 7.3.4. Although there will probably be larger latitude in the choice of location than in an ordinary engineering project, final selection criteria will certainly include strong political and social considerations as well as technical input.

Site investigation carried out from the surface down to depths of several hundred meters has strong similarities to investigations carried out for mining and resource location. Here, however, the main emphasis is on the detailed characterization of the rock structure and its permeability. Mechanical properties, though significant, have a lower priority. Ground investigation will continue throughout the construction stages where advantage will be taken of the possibility of direct access to the rock at depth, so a close and detailed geological characterization should be incorporated in the construction planning. During construction, reconnaissance methods that are not intrusive will be strongly favoured to avoid the possibility of opening new preferential pathways for radionuclide migration by site investigation activities. In this context, geophysical techniques such as georadar, seismic tomography, electrical conductivity probes and others are being pursued with some success.

Generally, the construction of a repository will involve the sinking of a number of access shafts, the excavation of a network of horizontal access tunnels and the drilling of vertical boreholes or excavation of additional horizontal drifts for waste emplacement. Although the considerable depth of the repository is a significant issue, the underground construction work required is well within the capabilities of conventional civil and geotechnical engineering. However, the design and construction of a HLW repository will exhibit a number of specific features not usually encountered in other conventional projects.

For instance, the excavation method to be used may be controlled by the need to minimise the disturbance and fracturing that occurs around the excavation. There are a number of ways that can be used to try to minimize those effects: i) the use of appropriate excavation techniques such as TBM or road header rather than blasting, ii) the early emplacement of rock support systems, iii) effectively repressurising the excavation by use of expansive backfill or by mechanical means.

Some construction procedures can also be affected by the final objective for the repository. For instance, some types of reinforcement materials might be proscribed so that they do not eventually interfere with the geochemistry of the site or because they liberate too much gas when subject to long-term corrosion. Stress relief elements in ground support components have been advocated to accommodate the thermally induced deformation of the rock on hot repository areas (Figure 6).

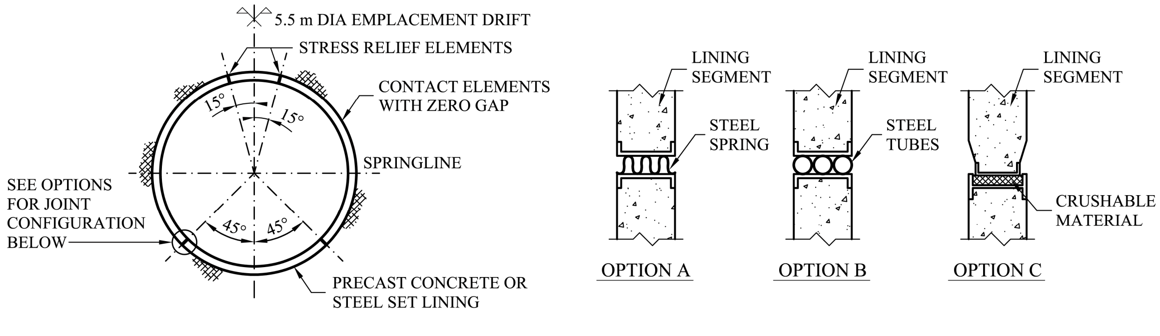


Figure 6. Lining systems incorporating stress relief elements (Sun et al. 1999)

Desaturation of the rock may be an issue as it may change the structure of the host rock, lead to cracking and a permeability increase. It may be due to conventional ventilation effects during construction but, also, to the effect of high suctions in the backfill materials of engineered barriers and seals. The need for control of ventilation during excavation and construction must therefore be recognized. Ventilation may also give rise to chemical changes in some rocks due to oxidation. As an example of this type of phenomena, Figure 7 shows the area where a number of gypsum spots have been observed during the excavation of niches in Opalinus clay in the Mont Terri underground laboratory (Bossart et al., 2002). The highest numbers are found in the first 70 cm. This finding is best interpreted as an oxidation phenomenon in an interconnected fracture network in the first 70 cm of the tunnel wall. The fractures in this zone are connected to the tunnel and are filled with air, supporting chemical oxidation reactions, namely oxidation of pyrite. At larger distances, from 70 cm to 2 m, the unloading fractures are only partially interconnected and possibly partially saturated with porewater and only very limited oxidation reactions are possible.

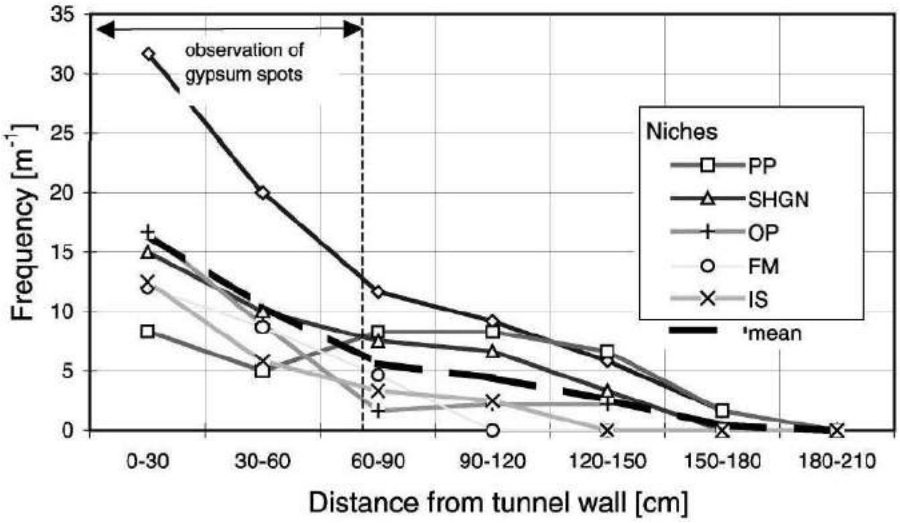


Figure 7. Fracture frequencies from the side walls of the newly excavated niches. The area where most gypsum spots have been observed is indicated (Bossart et al., 2002).

Prior to closure, it will be necessary to seal and backfill access tunnels and shafts so that they do not become preferential radionuclide migration pathways. The main purposes of seals and backfill are to provide: i) low permeability plugs ii) long term mechanical support for the underground openings, iii) additional locations for radionuclide sorption, and iv) some protection against human intrusion. Figure 8 shows a typical scheme for a seal. It can be observed that the seal is keyed into the rock in order to intercept (at least partially) potential flow through the zone affected by the excavation. The issues concerning seal behaviour are similar to those arising in the design of engineered barriers, with the important difference that no high temperatures are expected. Bentonite seals are at present the preferred option because of their high sealing capacity but other materials (e.g. concrete, bitumen, pulverised fuel ash) are also considered. An optimal solution from the cost point of view is to try to incorporate excavation debris in the seal design.

Until relatively recently, the generally accepted view was that closure of a deep repository, after sealing is finished, should be regarded as a final act without further operations on site. No rational purpose could be assigned to any long term monitoring of the site. However, in recent times, the issue of retrievability has become very topical. The idea is to allow for a certain period (long in terms of human activities) during which the radioactive waste could be recovered if a decision to do so were adopted. In some senses, this is an attractive proposition because it combines the advantages of a long-term intermediate storage with the much higher inherent safety of a deep geological repository. It also adds flexibility to the process with opportunities for review, taking account of both unforeseen technical advances and public interest matters (NEA, 2001). Achieving a deep nuclear waste storage where the waste remains retrievable for a long period, however, poses serious challenges to the conception, design, construction, operation and long term monitoring of the repository. Some of the geotechnical considerations associated with the retrievability option are discussed in Tanious et al. (1987).

Another issue that has also emerged fairly recently is the possibility of performance confirmation. The purpose is to confirm that the behaviour of the repository, at the beginning of its lifetime, corresponds closely to what had been predicted in advance. A good correspondence of observations with predictions would enhance the feeling that a good understanding of the problem has been reached and that closure of the repository could be undertaken with more confidence. A period of performance confirmation would set strict requirements for the quality and precision of modelling and would imply a period of strong monitoring of the repository. This in contrast with the classical approach where closure would be achieved as soon as possible and the design would not consider any maintenance of monitoring afterwards.

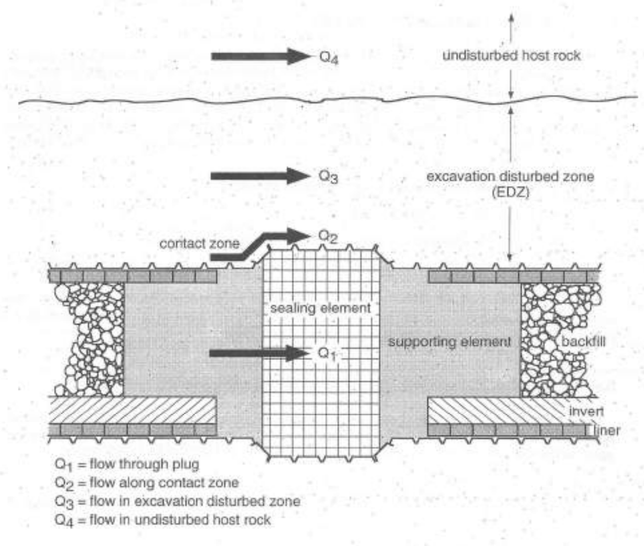


Figure 8. Scheme of a generic conceptual design of a seal of a horizontal access drift.

7.4.2 Excavation damaged zone and excavated disturbed zone

An important consideration during the construction of an underground repository for nuclear waste is the final state of the rock affected by the excavation. This is again an area in which the specific features of nuclear waste disposal are evident. In conventional underground excavations for civil engineering purposes, the state of the rock immediately adjacent to the tunnel surface is not a major concern apart from its possible role in local instabilities. In contrast, the presence of a fractured zone close to underground openings may provide a preferential path for radionuclide migration and is, therefore, a matter of obvious relevance.

In an effort for clarifying the issues involved, in the recent Luxembourg Conference on this topic (European Commission, 2004), it was proposed to distinguish between the excavation disturbed zone (EdZ) and the excavation damaged zone (EDZ). The EdZ comprises all the rock affected in some way by the excavation whereas in the EDZ the modifications of the hydromechanical and geochemical properties are sufficiently important to potentially affect the repository functionality. It is difficult to propose precise general definitions valid for all rock types and it has proved preferable to limit the definition to each specific rock type, as indicated in Table 1.

Table1. Proposed definitions of EDZ and EdZ for the main four rock types (Tsang et al., 2005).

	Crystalline rock	Rock Salt	Indurated clay	Plastic clay
Excavation disturbed zone (EdZ)	Region where only reversible (recoverable) elastic deformation has occurred.	Region with change of stress relative to initial state.	Region where only reversible processes (elastic strain, pore pressure changes, etc.) take place; not relevant to creation of preferential pathways for radionuclide migration.	Zone with significant modification of state (pore pressures, stresses, etc.); no negative effects on safety.
	Note: It is theoretically impossible to define the outer limits of the EdZ	Note: Outer boundary not clearly delineated; effects of EdZ at great depths not likely to be important over operation period, but may be significant for long-term performance.		
Excavation damaged zone (EDZ)	Region of irreversible deformation with fracture propagation and/or development of new fractures.	Region of considerable property change by microfracturing; significantly changed hydraulic properties.	Micro-cracked zone with damage and failure, and with weakly connected micro-cracks. A zone in which permeability increases by several orders of magnitude, owing to newly formed connected porosity—may become an issue in safety assessment.	An evolving zone with geomechanical and geochemical modifications of state and material properties, which might have a negative effect on operational and long-term safety.
	Note: Strong transient behaviour; depends on construction methods, as well as stress redistribution.	Note: Extent and quality of EDZ may change over time, depending on stress–strain conditions.	Note: EDZ is not the same as plastic or yielded zone.	

During construction, the EDZ is influenced by the excavation method, stress changes due to the formation of the opening and by the emplacement of rock support. There is already a fair amount of data regarding the possible extent and characteristics of the EDZ in different rock types gathered from intense observations during the excavation of underground laboratories. For instance, in crystalline rock, excavation method appears to have a dominant effect. Observations obtained in the ZEDEX project (Figure 9) performed in hard crystalline rock at Äspö research laboratory (Emsley et al., 1997) showed that the EDZ had a thickness of 0.3-0.8 m when the drift was excavated by blasting. In contrast the thickness of the EDZ reduced to only 0.03 m when the excavation was achieved using a TBM. No significant self-sealing of fractures is expected in this rock type.

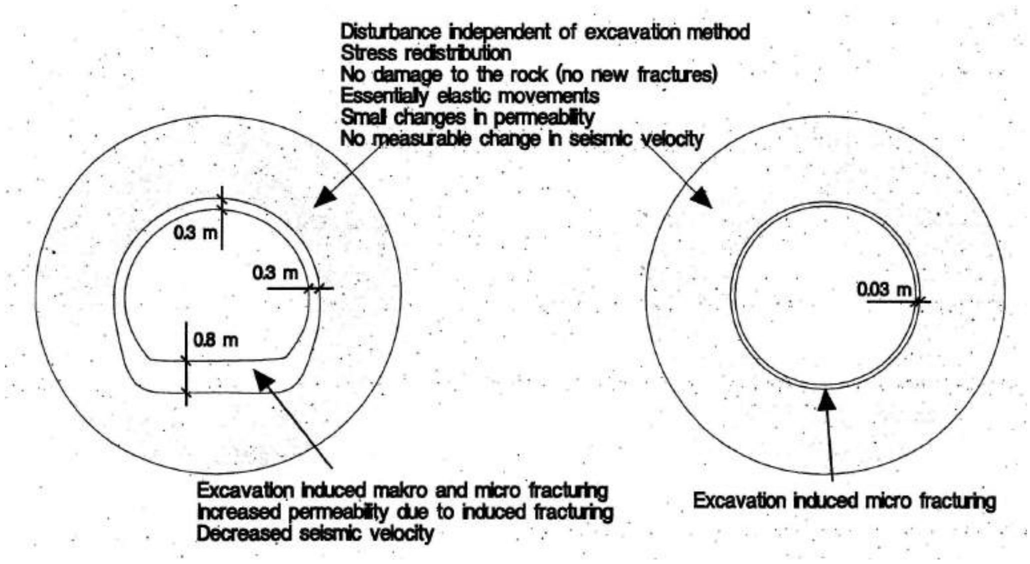


Figure 9. Features of the EDZ in granite observed n the ZEDEX project (Emsley et al., 1997)

A recent example of EDZ characterization has been described in Bossart et al. (2002) in relation with the excavation of a new gallery in the Mont Terri Rock Laboratory using blasting, pneumatic hammering and road header techniques. The rock is Opalinus clay, an indurated mudrock with a very strong bedding anisotropy. To measure the extent and hydraulic properties of the rock adjacent to the tunnel opening, pneumatic tests involving nitrogen injection or air extraction were used. The tests are performed at 10 cm intervals using single or, more often, double packer systems (Figure 10). The measured permeability distributions are shown in Figure 11, where it can be observed that variations ranging between two and four orders of magnitude are obtained. Quite high permeability zones are observed within the 10-20 cm from the rock/shotcrete interface. The EDZ zone is thicker at the top of the gallery compared with that at the lateral walls. Hydraulic testing was completed by hydraulic cross-hole testing that basically confirmed the above results. The investigation also included the analysis of resin-injected overcores that allow the direct observation of fracture orientations and frequencies.

As a result of this work, it was possible to build a conceptual model of the EDZ made up of two zones: an inner zone and an outer zone. The inner zone is up to 1 m thick and consists of an interconnected and air filled fracture network caused mainly by unloading with quite high permeabilities and transmissivities. The outer zone is limited to a zone of 2 m around the tunnel and consists of mostly saturated and isolated fractures, which exhibit lower transmissivities. In any case, detailed examination of the EDZ has generally revealed a highly heterogeneous system. The extent of the EDZ tends to be somewhat smaller in the section excavated with road header techniques but it is always substantial as it is mainly controlled by unloading and rock structure.

Further investigations tried to determine the self-healing capacity of the EDZ in Opalinus clay (Meier et al., 2002). Two sites were examined; the first one contained a high transmissive single fracture at a depth of about 70 cm from the tunnel surface and the second one involved a fracture network contained in the first 80 cm of rock. After measuring their initial transmissivities, the fractures were saturated injecting water through a single packer system. Additional water injections were performed to ensure saturated conditions during the one year experimental period. The variations of transmissivities with time are shown in Figure 12. It is clear that self healing occurs but, at the end of the experimental period, initial conditions (i.e. before excavation) have not been recovered and the continuation of the decreasing transmissivity trend is difficult to predict.

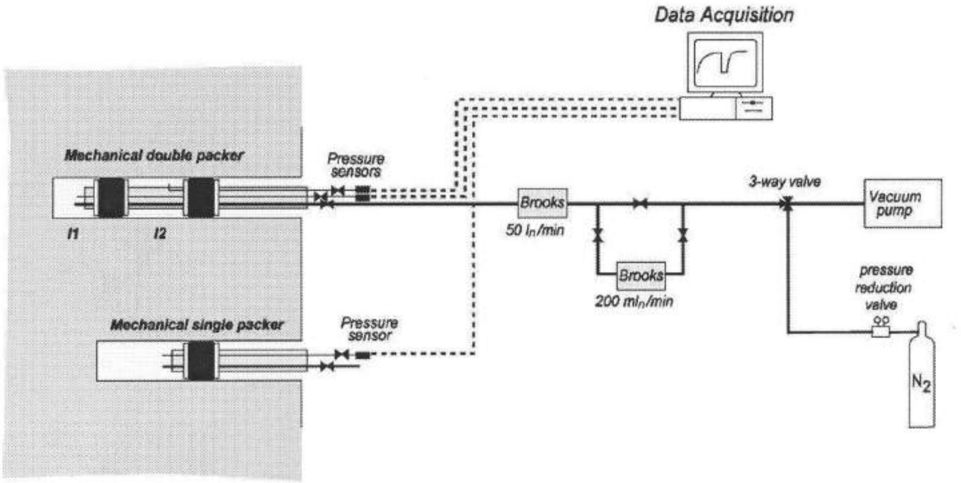


Figure 10. Permeability determination in the EDZ. Pneumatic testing procedure for nitrogen injection and air extraction (Brossart et al., 2002).

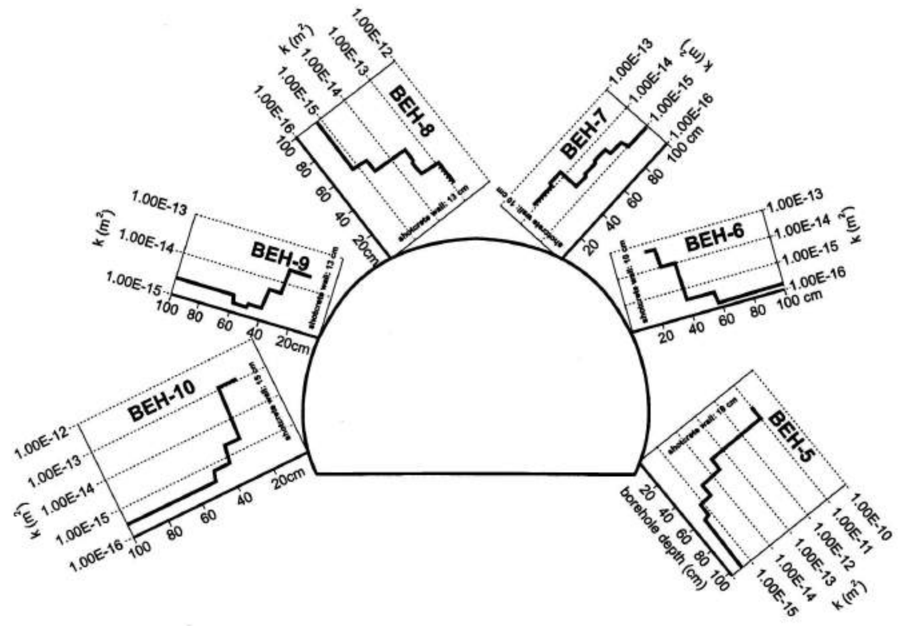


Figure 11. Permeability distribution at different locations around a gallery excavated in Opalinus clay (Brossart et al., 2002).

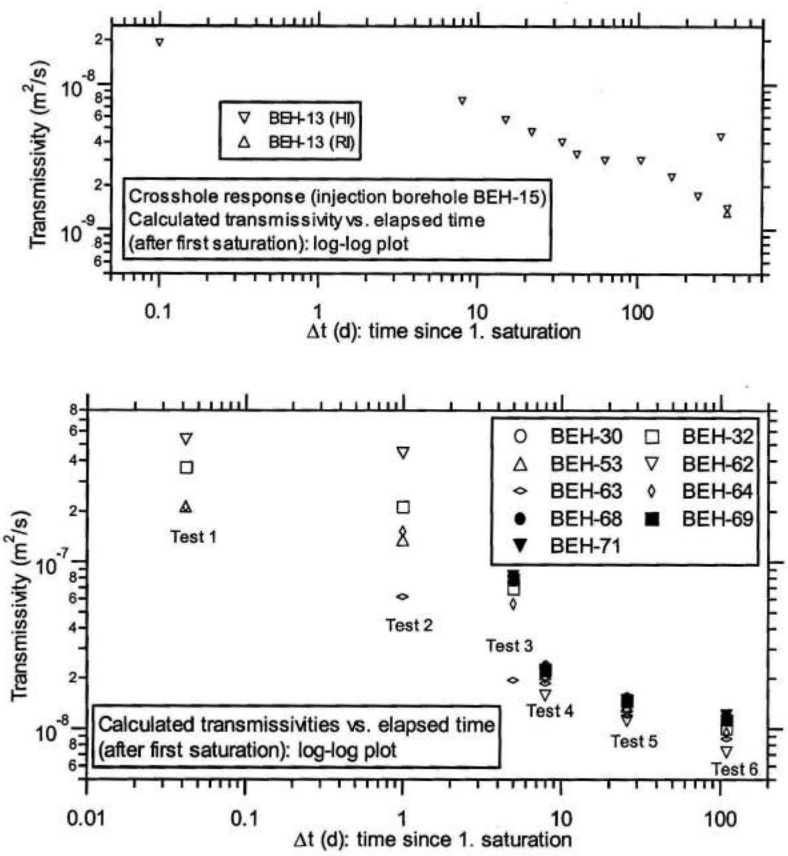


Figure 12. Variation of transmissivities with time in the EDZ set up excavatingn Opalinus clay. The upper Figure concerns a single fracture and the lower Figure refers to a fracture network (Meier et al., 2002).

Contrary to some expectations, fractures have also been observed in excavations performed at depth in plastic lays. Figure 13 sows the pattern of fractures found during the excavation of a tunnel 230 m deep in Boom clay in the Mol underground laboratory. There is no information on whether permeability is significantly affected but the presence of those fractures and the question of self-healing with time remains an open issue.

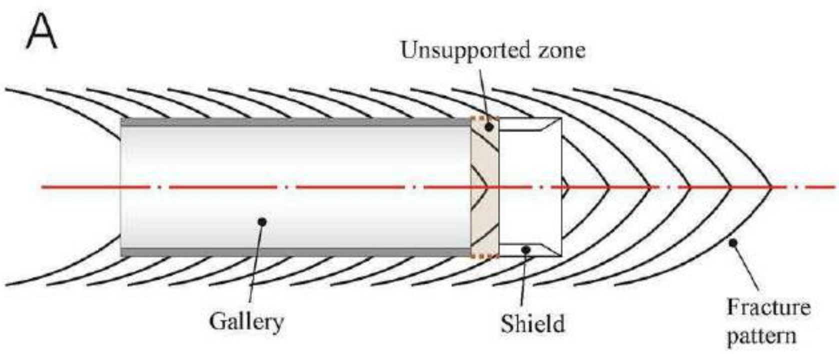


Figure 13. Fracture pattern observed in the excavation of a tunnel in Boom plastic clay (Bernier, 2004)

The EDZ in salt rock has also different features. Several investigations have been carried out in the Asse salt mine showing that there is generally an increase of permeability of several orders of magnitude close to the tunnel walls (Wieczorec and Zimmer, 1998). The main reason underlying this increase in permeability is the dilatancy of the rock salt when subjected to high shear stresses in the region close to the drift opening. Typical thicknesses of the EDZ are in the order of 1 to 1.5 m. Because of the high ductility of the rock salt, it would be expected that there will be a tendency for the EDZ to self-heal and reduce its permeability with time. Results so far have not proved conclusive in this respect, although it seems that self-healing occurs preferentially in backfilled or supported openings and not so much in open drifts.

From the information available, it is clear that the presence of the EDZ is an important issue with regard to the design and construction of the repository. However some uncertainties remain regarding matters such as the influence of rock type, rock structure, method of excavation and support, capacity and time required for self-healing. A tentative consensus has been put forward in NEA (2002b) summarised in Table 2. The low relative importance attributed to the EDZ in plastic clays and rock salt relies on their presumed self-healing capacity, a feature that has yet to be reliably confirmed in the field.

Table 2. A summary of the general level of significance and understanding of different aspects of the EDZ in various rock types (NEA, 2002b).

Rock type	Difficulty of characterisation	Level of understanding	Difficulty of modelling	Relative importance of EDZ
Crystalline	high	high	high	medium
Rock salt	high	high	high	low
Plastic clays	high	high	high	low
Indurated clays	medium	medium	medium	high but it should tend to medium

7.5 PERFORMANCE AND SAFETY ASSESSMENT

7.5.1 Quantitative safety demonstration

A safety assessment of a deep geological repository involves a systematic analysis of the behaviour of the repository to check that it meets the legally established safety criteria throughout its whole history. A synthesis document based on the analyses of 10 recently conducted Safety or Performance Assessments has been put forward by NEA (1997). In that document, it is suggested that the term Safety Assessment should be applied to the whole repository system whereas Performance Assessment should be used when only a part of the repository is considered. However, there is not a commonly agreed usage of those two terms yet

A safety or performance assessment requires the consideration of all processes and phenomena ha may affect the performance of the repository in a significant manner. This involves combining experimental and field data with scientific understanding and qualitative observations to construct models of the possible future behaviour of the disposal system (Figure 14). This safety evaluation is a complex process because of the variety of materials and components of a repository and to the large number of interacting processes that may play a role. In addition, the performance assessment must consider extremely long periods of time in order to encompass the interval during which the radionuclides are potentially dangerous.

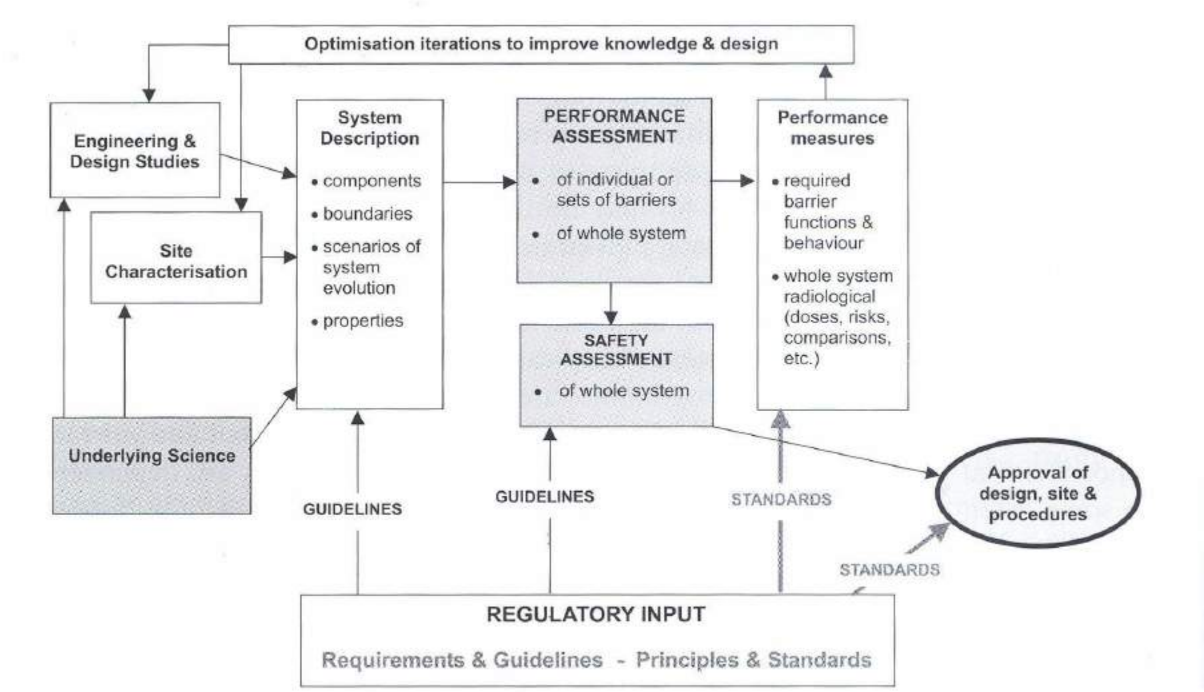


Figure 14. Interaction scheme of safety and performance assessment with technical information and regulatory requirements (Chapman and Mc Combie, 2003)

It should be understood, however, that a safety or performance assessment does not require the comprehensive description of all processes and interactions that can take place in a repository system. Only conditions that may or may be thought to impinge on safety require description. Simplified models and assumptions can be made provided they are shown to be conservative. Simplifications, however, should be made from sound bases that usually require a thorough examination of the problem; well beyond what is required by the strict safety case.

A typical performance assessment exercise contains the following steps: i) definition of reference safety criteria, ii) description of the repository design, iii) definition of scenarios covering the possible future evolutions of the system, iv) analysis of the behaviour of the various system barriers, v) output computation vi) examination of the outcome including sensitivity analysis and uncertainty assessment.

The definition of scenarios involves the identification of all factors likely to influence significantly the long term evolution of the system. Factors include properties (e.g. rock permeability), processes (e.g. gas generation by canister corrosion) and events (e.g. failure to seal the repository). The ensemble of selected factors and their interactions constitute the “reference system” and its evolution constitutes the “reference scenario”. External factors originating outside the system (e.g. climate change, human intrusion) must also be considered. Combination of external factors with the reference system leads to other scenarios that have to be evaluated as well. Geotechnical input is, of course, crucial in the process of identification of the various factors and their likely significance.

The next step is the analysis of the behaviour of the various barriers constituting the deep geological disposal concept. In this phase, the contribution of Geotechnical Engineering is pervasive. To carry out this task in an effective way, it is convenient to identify a number of subsystems that are analyzed separately. An example of a first-level subsystem classification is depicted in Figure 15. It can be observed that the output of a subsystem constitutes the input of the next one. A conceptual model is built for each subsystem that includes the most relevant processes, the main parameters and the interaction between phenomena. The subsystem is then quantitatively analysed by means of appropriate numerical models. Finally the result of each subsystem is integrated in the description of the overall behaviour of the entire system.

This division between different subsystems must be made considering what are the phenomena and time scales relevant to each particular component. Often there are large differences between the processes that operate in the various subsystems. A useful conceptual distinction refers to the division between near field and far field. In a rough way, the near field may be defined as the part of the disposal system that is directly affected by the presence of the waste. It usually includes the canister, the buffer or barrier and the adjacent rock. The far field extends from the boundary of the near field (not a precise location) to the region near the surface that may interact with the biosphere. In this respect, the potential contribution of geotechnical engineering is especially strong in the analyses affecting the near field. Of course, it is always possible to divide each subsystem into other items in a hierarchical manner. However, there are clear limit to this process of subdivision set by the degree of coupling and interaction of different phenomena.

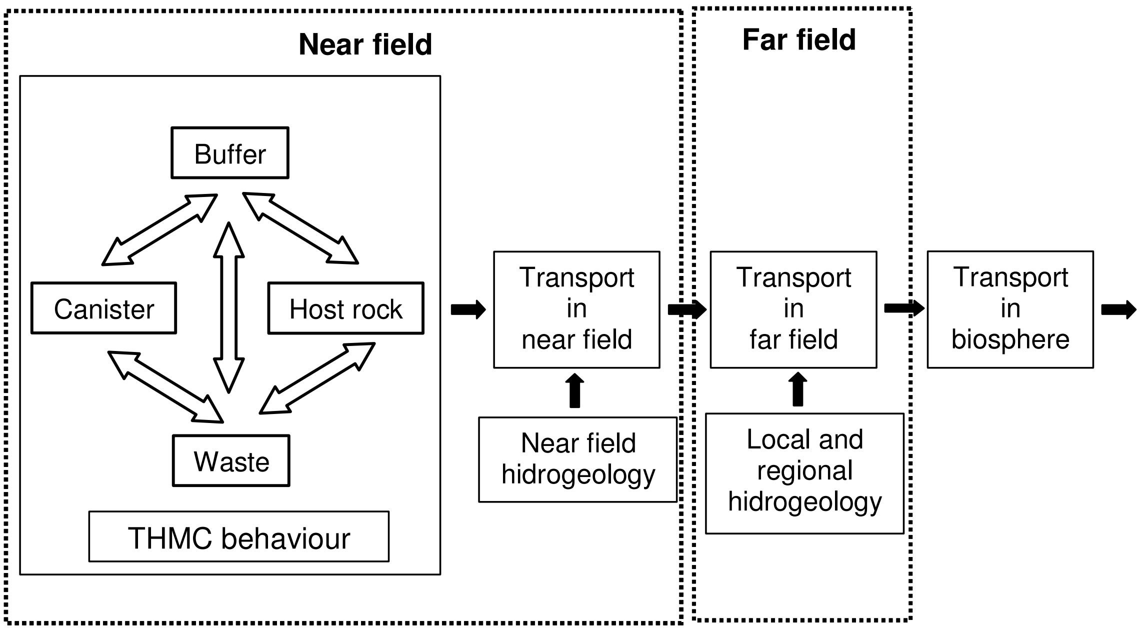


Figure 15. Example of performance assessment division by subsystems

7.5.2 Processes in the engineered barrier and near field

The near field is an area of complex phenomena and interactions. Here attention is focussed on the description of the behaviour of the backfill barrier surrounding the canister and of the immediate adjacent host rock. Although the near field also includes the canisters and waste matrices, they are the concern of materials science. To make the discussion more specific, it is assumed that the backfill is bentonite-based, the most common case.

Considering the thermo-hydro-mechanical (THM) behaviour of the bentonite barrier first, it is important to realize that the barrier material, compacted clay, is initially in an unsaturated state, so there will be a solid phase, a liquid phase and a gas phase to take into account. The main actions that affect the bentonite barrier (at least in the short term) is the heating arising from the canisters and the hydration from the surrounding rock. Figure 16 shows, in a schematic way, some of the thermo-hydraulic processes occurring in the barrier and immediate adjacent rock. At the inner boundary, the barrier receives a very strong heat flux from the canister. The dominant heat transfer mechanism is conduction that occurs through the three phases of the material. A temperature gradient will therefore develop in the near field and heat dissipation will be basically controlled by the thermal conductivity of the barrier and host rock. Maximum temperatures envisaged in repository design can be quite high. Some designs limit the maximum temperature to 100º C but other concepts may allow temperatures as high as 200º C.

In the inner zone of the barrier, the heat supplied by the heater results in a temperature increase and in strong water evaporation that induces drying of the bentonite. Degree of saturation and water pressure will reduce significantly in this region. Vapour arising from bentonite drying will diffuse outwards until finding a cooler region where vapour will condense, causing a local increase in water saturation. Vapour diffusion is a significant mechanism of water transfer mechanism and, to a much lesser extent, of heat transport.

Due to low water pressures existing initially in the unsaturated material that constitutes the backfill, hydration will take place with water moving from the host rock to the barrier. The distribution of water potential is also perturbed by the phenomena of bentonite drying and vapour transport as described above. Hydration will eventually lead to saturation of the barrier, but saturation times can often be very long due to the low permeability of the bentonite and/or host rock.

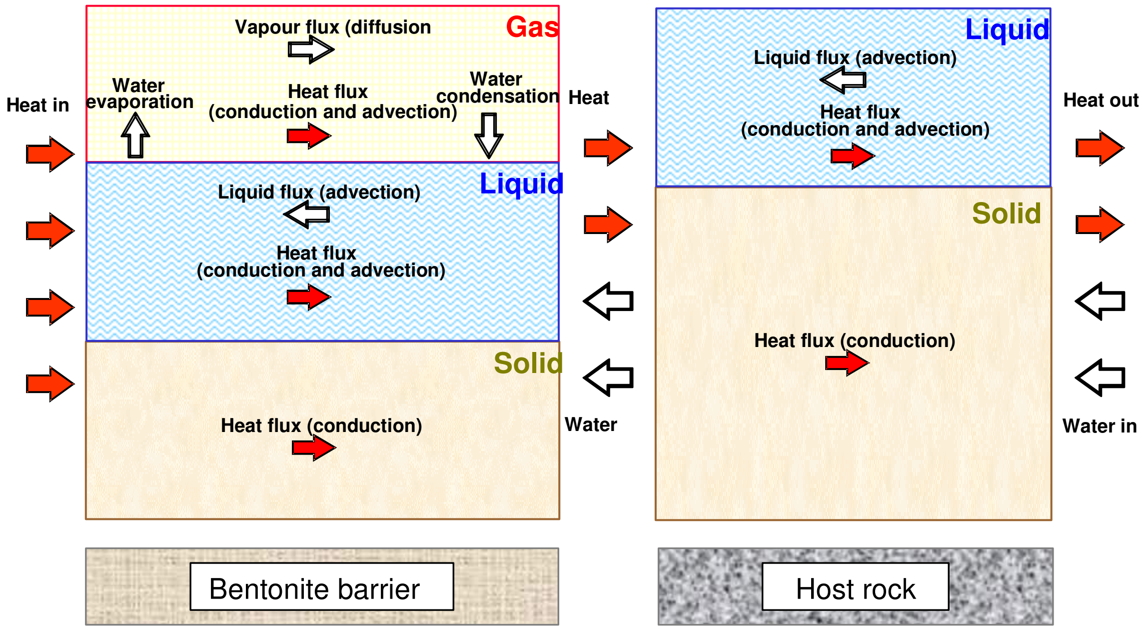


Figure 16. Scheme of thermo-hydraulic processes in the near field

In addition to the thermo-hydraulic behaviour, there are also important mechanical phenomena occurring. Drying of the bentonite will cause shrinking of the material whereas hydration will produce swelling that may be quite strong in bentonite barriers. Because the barrier is largely confined between canister and rock, the main result of hydration is the development of swelling pressures, in a process quite akin to a swelling pressure test. The magnitude of the stresses developed is critically dependent on the emplacement density of the bentonite and may reach values of several MPa.

The crucial feature of the THM behaviour described is that all those phenomena are strongly coupled, interacting with each other in a complex manner. As an example, Figure 17 shows the phenomenon of vapour transport. Evaporation and condensation depend on the value of suction (hydraulic variable) and temperature (thermal variable). Transport itself is a mixture of advection and diffusion that is influenced by temperature (thermal), degree of saturation (hydraulic) and porosity (mechanical). In fact, vapour transport can not be considered on its own, but as a branch of a cycle closed by the movement in opposite direction of liquid water. The flow of liquid water also depends on temperature (water viscosity), degree of saturation (relative permeability) and porosity (intrinsic permeability). Another example of interaction is the main heat transfer mechanism, heat conduction. This is basically controlled by thermal conductivity that, in turn, it depends on degree of saturation (hydraulic effect) and porosity (mechanical effect). There are many other cases that could also be described. When the barrier and rock involve saline material the additional effects of dissolution and precipitation are also very significant (Olivella et al., 1996a; Gens and Olivella, 2000a).

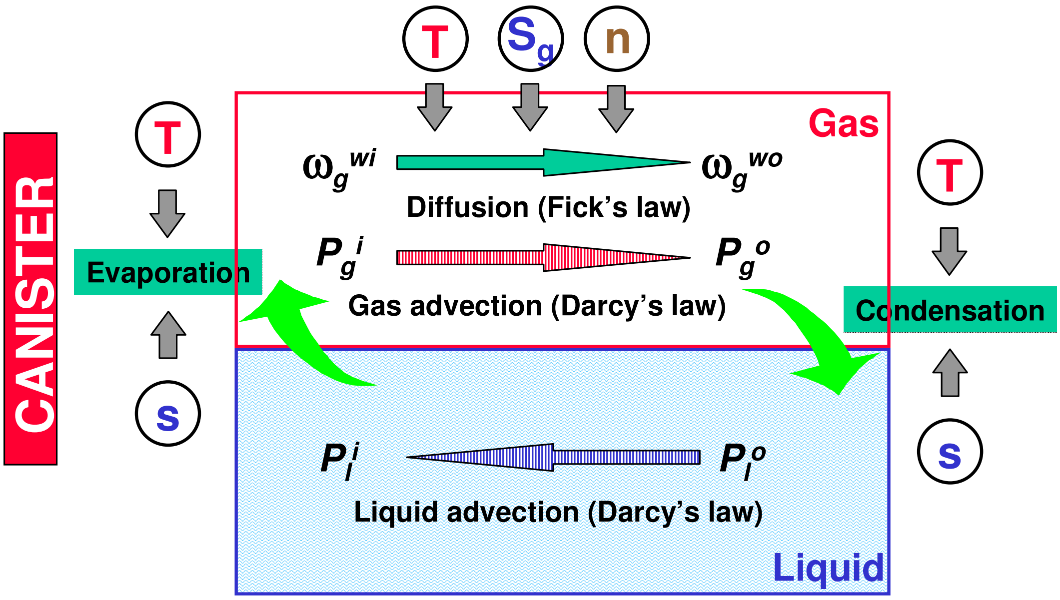


Figure 17. Coupled phenomena in vapour transport

The complexity of THM behaviour increases further when the interaction of the bentonite barrier with the host rock is taken into account. In Figure 16, the rock is depicted as saturated, but, in fact, the zone close to the barrier may desaturate due to the effect of strong suctions in the backfill. The desaturation of a zone around the engineered barrier has a large effect on important behaviour features such as the rate and duration of the process of barrier saturation (Gens et al., 1998).

The analysis and prediction of the performance of nuclear waste repositories require adequate numerical modelling. From the geotechnical point of view, the most characteristic advance has been the development of formulations and computer codes capable of performing coupled thermo-hydro-mechanical analyses (Gawin et al., 1995; Olivella et al., 1994; 1996b; Thomas and He, 1995). They involve the simultaneous solution of the equations of i) water mass balance, ii) air mass balance, iii) energy balance, and iv) equilibrium (momentum balance). The formulations must be based on sound physical principles, as applications to very long-term predictions, well beyond any empirical experience, are required. The formulations and associated computer codes must also account for all the relevant phenomena discussed above and their interactions. This is achieved by the development and selection of adequate constitutive laws. The capacity of this type of formulation to reproduce the real THM behaviour of a repository has been demonstrated (Gens and Olivella, 2000b; Gens et al, 2002a).

The overall behaviour of the barrier and adjacent rock provides the background environment for the radionuclide transport in the near field. The main features of this transport are strongly influenced by the geochemistry characteristics of the medium. Geochemistry is initially controlled by the chemical characteristics of the bentonite and water used in compaction. This geochemistry is later modified by the groundwater penetrating the backfill, by the waste form itself, and, potentially, by radiolysis.

When the waste form finally breaks down, radionuclides are dissolved or mobilized as particles and may start to migrate. The breakdown rate of the waste matrix and its solubility control the rate of release of the radionuclides. Two main mechanisms of migration exist: advection and diffusion. Because of the very low hydraulic conductivity of bentonite, advection is likely to be negligible. Migration rates are also influenced by geochemical effects such as retardation due to sorption, precipitation and other processes.

To account for these phenomena, it is necessary to evaluate the geochemistry evolution throughout the lifetime of the repository. A first approach is to perform chemical thermodynamic calculations. If chemical equilibrium can be assumed (a reasonable hypothesis for the vast majority of chemical processes considered), the main difficulty lies in the scarcity of data for some reactions and species. Further developments in this area have included recently the coupling of geochemistry models to transport equations and, finally, to THM formulations (Guimaraes et al., 1999; Gens et al., 2002b; Thomas et al., 2002). The eventual outcome of near field study must be the prediction of radionuclide releases with time. They constitute the source term for the far field analyses.

7.5.3 Processes in the natural (geological) barrier

The features of the far field that are relevant to the isolation problem are those controlling the variation with time of the concentration of the various radionuclides reaching the biosphere. Basically they are:

- Path-length and water velocity of radionuclide migration trough the rock
- Physical and chemical interaction between rock and transported radionuclides.

In this area, it is difficult to make general statements; many characteristics are bound to be rock-type dependent and site specific.

Radionuclides may be in solution in the groundwater, in particulate form (colloids) or attached to other suspended material. Similarly to the near field, the main transport mechanisms will be advection and diffusion. Diffusion will only be significant when water flow is very slow.

In any case, migration will be controlled by the hydrogeological features of the geological medium. The most important properties in this respect are hydraulic conductivity, available porosity and piezometric gradients. However, one of the most important facts is the possible presence of geological features that may provide preferential flow paths. In many natural systems the existence of those features totally controls the rates of radionuclide migration. The hydrogeological characterisation of the relevant geological body is thus an important challenge for any siting decision.

Once the water flow characteristics are established, radionuclide migration is further modified by physical dispersion and retardation. The magnitude of physical dispersion varies substantially depending on the type of rock and on whether flow occurs mainly along fissures or through the rock matrix. Predominant fracture flow tends to result in low dispersion.

Hydrodynamic dispersion is further modified by matrix diffusion and physico-chemical retardation. The first phenomenon refers to the diffusion of solutes from the liquid circulating in the water flow features towards the interior of the rock matrix. This obviously will delay the arrival of the radionuclides to the biosphere. Only when concentrations in the flowing water eventually reduce, the mechanism will reverse. Physico-chemical retardation may be due to a variety of causes resulting from the interaction between rock surfaces and radionuclides as illustrated in Figure 18.

7.5.4 Safety assessment result

The output of the performance or safety assessment exercise may be expressed in different ways, depending on the selected performance measure (Chapman and Mc Combie, 2003). The most usual performance measures are the effective individual dose or the risk, in which the dose is factored by the probability of its occurrence and the probability that a given dose will lead to a health effect. A typical output example, in terms of the computed individual dose, is presented in Figure 19 for the reference scenario of the Spanish reference concept for high level nuclear waste (Astudillo, 2001). It can be noted that not only the total dose is given but also the individual contributions of each radionuclide. Two immediate observations can be made: i) computations are extended to extremely long times, 10^6 years in this case, and ii) the maximum doses obtained are very low compared with the typical natural radiation figures given in the Introduction and are also well below the specified limit value of 10^{-4} Sv/year. Naturally, the computed doses increase when considering the effect of external factors. Figure 15 is also useful to indicate that the central phenomenon to be examined is the transport of the radionuclides to the biosphere and that the analyses of the large variety of phenomena involved should be performed with this final aim in mind. Naturally, complexity and uncertainty in many areas prevent exact predictions. This difficulty, however, may be overcome (at least partially) by the use of conservative hypothesis and by the consideration of a sufficiently wide range of possible scenarios. Often, probabilistic approaches are incorporated in safety evaluation procedures. The complexity of the system and relevant processes implies that probabilistic assessments are normally based on Monte Carlo methods.

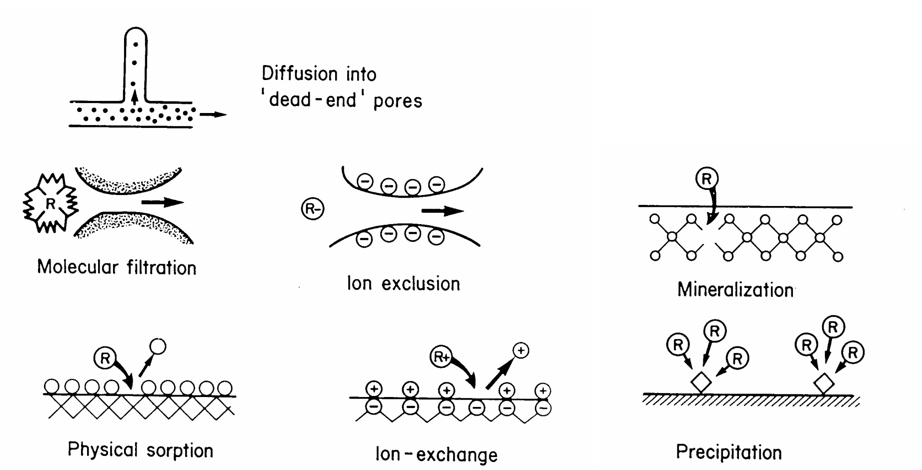


Figure 18. Schematic representation of possible retardation mechanisms (Chapman and Mc Kinley, 1987)

The final performance assessment requires detailed data of the specific design of the repository and its geological setting. Although some site-specific repository analyses have been performed (Vieno et al., 1996), as, generally, the various countries have not made yet a decision on a final location for the repository, most performance assessments are carried out at present using generic design and generic rock properties (although often based on specific geological frameworks). These generic safety evaluations are very useful to identify possible design improvements and the areas in which more research is required. The design can then be optimized to improve safety. Safety evaluation is therefore an iterative process that continuously integrates the increase of knowledge on the behaviour of the various system components. Geotechnical input is an essential part of this continuing task.

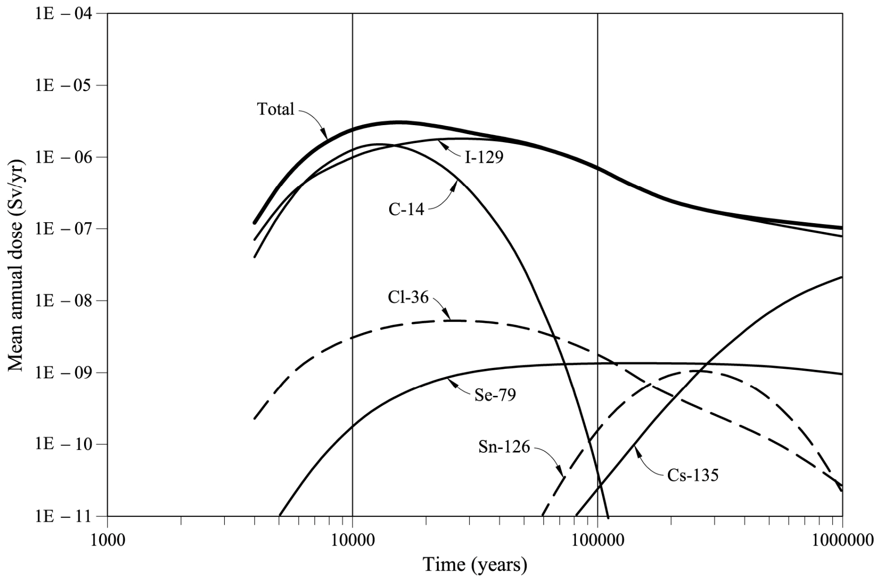


Figure 19. Mean annual individual dose computed for the reference scenario of the Spanish reference concept for high level nuclear waste (Astudillo, 2001)

7.5 DISPOSAL OF LOW AND INTERMEDIATE LEVEL NUCLEAR WASTE

The issues associated with the disposal of low level (LLW) and intermediate level (ILW) nuclear waste are quite different from those concerning the disposal of high level nuclear waste (HLW). LLW has a low activity caused by short lived radionuclides. ILW may contain some longer lived radionuclides but their activity is still modest. In any case they are basically not heat emitting. As indicated above, LLW and ILW arise from a variety of sources, the volume of ILW and LLW is orders of magnitude larger than that of HLW. Fortunately the requirements for disposal are much easier to meet.

The most characteristic method of ILW and LLW disposal is placing it either in shallow trenches or directly on the (suitable conditioned) ground. The waste is subsequently protected with an adequately designed cover. In this respect, disposal of ILW and LLW has many points in common with the usual disposal strategies of more conventional hazardous waste.

Shallow ILW and LLW waste disposal also employs a multi-barrier concept. Both compacted and uncompacted LLW are often placed in simple steel drums. ILW generally receives a more careful treatment being encapsulated in cement, resin or polymers before being placed in steel or concrete containers. The containers are then placed in the storage units and, generally, mortar is injected to fill the space between the individual containers. The barrier made up by the primary containers and the storage unit is sometimes called the physico-chemical barrier. The storage units are then stacked on storage cells made of concrete that constitutes the engineered barrier. Finally, the storage cells are covered by a cap made of a number of soil layers. The cap and the underlying and surrounding ground are the geological barrier. A conventional multilayered cap is generally envisaged with topsoil at the surface, a drainage made of rockfill and sand layers in the middle and a layer of compacted clay adjacent to the storage cell.

Often the waste is placed above the water table taking into consideration possible seasonal fluctuations with especial attention to the possibility of extreme events of water table rise. The desired water flow is via surface infiltration, downward past the trench, through the unsaturated zone and into the water table. Possibilities for contamination are thereby minimised. When disposing the waste above the water table, it is better if the host ground is not too impermeable to avoid ponding in the trenches in case the cover cap fails. The underlying ground should be able to drain percolating waters away from the trench. Drainage should be provided to collect any water that percolates from the ground into the waste emplacement. The multi-barrier philosophy with waste being placed above the water table is the approach adopted in the modern disposal facilities of Aube in France and El Cabril in Spain (Figure 20).

It is also possible to place the waste below the water table if the host ground has a very low permeability so that advection will be negligible and transport will occur via diffusion. The intermediate case, placing the waste where seasonal movements take place, is to be avoided, as it would lead to sequential flooding and drying of the trenches leading probably to its rapid degradation.

LLW and ILW are likely to produce much larger volumes of gas than HLW. The main sources are biogenic gas from organic matter and gases (mainly hydrogen) arising from the corrosion of metals in the waste and metal containers. For some waste types, therefore, it may be necessary to install vents in trench caps or use backfills with high gas permeability. In any case, post closure maintenance during the early life of the disposal facility may well be required.

Figure 20a. El Cabril disposal facility for medium and low level waste

Figure 20b. Experimental cover at Aube disposal facility for medium and low level waste

From the geotechnical point of view the main issues to address are the hydrogeology of the site and a good design of the cap and any associated drainage system. It is necessary to achieve a good knowledge and understanding of the local hydrogeology to ensure that favourable conditions will prevail not only at present but in the foreseeable future as well. The design of the cap and the waste drainage system is likely to require multiphase flow modelling that includes not only the ground but the various barrier components. Mechanical aspects are generally limited to ensure the overall stability of the site, admissible settlements and adequate bearing capacity. Generally those conditions are easy to achieve. Naturally, thermal problems are absent in this case.

Some countries (Sweden, Germany, Switzerland) have chosen deep disposal for ILW and even (at least partially) LLW. They consider that the additional cost involved is compensated by the higher degree of safety of a disposal at depth. The issues associated with this decision are therefore similar to those already discussed in the context of HLW disposal, except for the absence of heat emission and transport. Sometimes the disposal facility is excavated relatively near the surface whereas, on other occasions, quite deep mine galleries are used for placing the waste. As mentioned earlier, gas production may be significantly higher in this type of waste and a suitable solution for gas evacuation is harder to achieve in a deep repository. When a backfill is used around the waste, cement-based materials are preferred to bentonite-based ones. If the repository is in rock salt, crushed salt is again used as backfill.

7.6 REMEDIATION OF URANIUM MINING SITES

The nuclear fuel cycle always starts at uranium mining sites and mills. In some respects, the environmental aspects of a uranium mine are the same as those from metalliferous mining but the radioactivity associated with the uranium ore requires some special management. The uranium itself has a very low level of radioactivity but uranium minerals are always associated with more radioactive elements such as radium and radon in the ore. Virtually all the radioactive material from the associated minerals ends up in the tailings dam. Process water from the mills also contains radium and other metals and must be treated accordingly. Currently about 36,000 tonnes of uranium are extracted annually from mines worldwide, the main producers are Canada and Australia. Significant amounts of uranium are also produced in Europe, especially in Russia and the Czech Republic.

The present worldwide production of uranium mill tailings exceeds 20 million tonnes annually. It is therefore necessary that environmental and health risks from these materials are reduced to acceptable levels (Matthews, 1986). Such large volumes preclude sophisticated disposal options and, generally, the tailings must be treated in place. It should be said that many of the worst problems requiring solution arise from older mining activities when regulations were not as strict as nowadays.

In principle the treatment for uranium tailings has many features in common with the solutions used for other tailings containing hazardous waste. A tailings remedial project must achieve:

- reduction and control of water infiltration and prevent the risk of dispersing polluting materials
- ensure an adequate long term stability of the slopes
- avoid significant erosion.
- minimize gamma radiation out put and radon emissions

The latter must be especially considered when designing cover systems (Almeida et al., 2002). A time between 20 and 1000 years is usually specified for the remedial measures to remain effective. The International Atomic Energy Agency has published a useful summary of current practices for managing and confining this type of tailings (IAEA, 1992). The geotechnical analysis of the problem must include:

- short term and long term stability analysis of the tailings heap slopes with a conservative estimate of pore pressure distribution,

- multiphase flow analysis of infiltration into and round the tailings,
- transport analysis of contaminants into the underlying aquifers,
- estimation of radon releases.

An example of the remedial measures adopted refers to the rehabilitation of the Andújar Uranium Mill (FUA) work in Southern Spain. The Andújar Mill processed uranium ore from a variety of sources in Spain from 1959 to 1981. At the time, no special precautions were taken to manage the resulting tailings. From 1987, a sustained effort has been made to analyze the problem, and to design and to implement remedial measures. Simultaneously, the existing mill facilities were also dismantled. Computations showed that it was necessary to reduce the height of the tailings and simultaneously reduce the slopes to around 12º to ensure long term stability. Figure 21 shows the solution adopted where it can be seen that the tailings taken from the top and the available demolition rubble were incorporated into the design. Some bulky dismantling products were also encapsulated in the finished dyke.

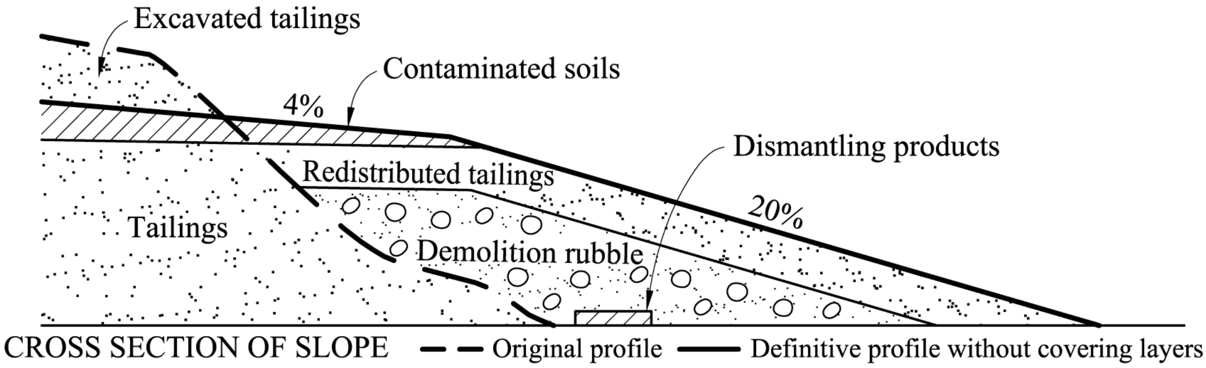


Figure 21. Cross section of the tailings dyke before and after remediation works. Final cover is not shown.

The cover finally selected, shown in Figure 22, had a quite complex structure. The most important components are: a 30 cm layer of rockfill, grain size 100-300 mm, for erosion protection, a 50 cm mantle, suitable for plant development using soil from nearby areas, a biointrusion barrier made up of a 30 cm layer of rockfill, 50-100 mm particle size, a 25 cm layer of gravel providing drainage and an impervious 60 cm layer of compacted clay acting as barrier for infiltration and radon emission.

Often, there has been significant aquifer contamination during the time in which tailings were sitting without protection. This was the case in the Andújar Mill. On other occasions, aquifer contamination arises from the method that has been used to extract the uranium mineral from the ground. At present, about 19% of total uranium mining is performed by in situ leaching. In situ leaching has the advantage that it involves less exposure for workers, no large tailing piles are generated, and, often, the cost is less. However, it is sometimes difficult, or even impossible, to restore the original conditions in the leaching zone and here is the risk of the leaching fluid going beyond the uranium zone and contaminate adjacent groundwater bodies. In that case, remedial measures are required. Kazda et al. (1997) have presented an interesting case in Northern Bohemia (Czech Republic) where deep mining was combined with leaching technology to extract the uranium. As a result around 200 million cubic meter of groundwater became contaminated. Pump and treat remediation method was selected as the most suitable one. It is envisaged that the remediation process will last for several decades during which the concentration of pollutants in the aquifer will gradually decrease.

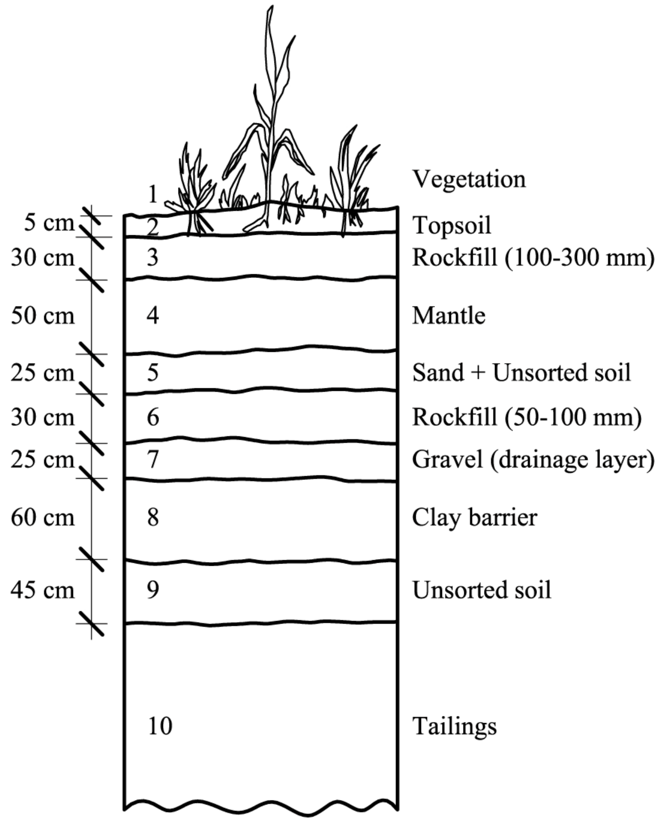


Figure 22. Scheme of the selected cover for the tailings dyke of the Andújar Uranium Mill

7.7 FINAL REMARKS

In some respects the issues associated with nuclear waste are very similar to those concerning more conventional toxic and dangerous waste. However radioactivity involves a series of additional conditions that have been discussed in this chapter. Nuclear waste is best considered in the context of the entire nuclear cycle, from uranium mining, through power generation, potential fuel reprocessing and radioactive waste disposal. The environmental problems and potential solutions are quite different in the case of uranium tailings, low and intermediate level waste and high level waste. They have been treated separately in this chapter.

Although there are a number of novel features in the problems associated with nuclear waste management, the general pattern of topics (time evolution, generalised behaviour of multiphase materials, coupled approaches, scale effects) is very characteristic of the classical geotechnical approach understood in a generalized way. For this reason, Geotechnical Engineering, working in an interdisciplinary context, is ideally positioned to provide effective responses to the demanding challenges set by this field of application.

ABBREVIATIONS AND ACRONYMS

ADS: Accelerator driven system
ANDRA: Agence nationale pour la gestion des déchets radioactifs
EBS: Engineered barrier system
ENRESA: Empresa Navioanl de Residuos S.A.
EDZ: Excavation damaged zone
EdZ: Excavation disturbed zone
HLW: High level waste
IAEA: International Atomic Energy Agency
ILW: Intermediate level waste
LL-ILW: Long lived intermediate level waste
MLW: Medium level waste
MOX: Mixed uranium-plutonium oxide fuel
NAS: National Academy of Sciences
NEA: OECD Nuclear Energy Agency
NRC: National Research Council
OECD: Organisation for Economic Cooperation and Development
UNEP: United Nations Environment Programme
URL: Underground Research Laboratory
SKB: Svensk Kärnbränslehantering AB (Swedish Nuclear Fuel and Waste Management Co.)

REFERENCES

Almeida, M. and Leoni, G. (2002). Final covers for uranium mill tailings impoundments. *Environmental Geotechnics* (de Mello, L.G. and Almeida, M., eds), A.A. Balkema, Lisse, 2, 923-931.

American Nuclear Society (1998). *High level radioactive waste management*, Las Vegas, Nevada. AMS: La Grange Park, Il.

Astudillo, J. (2001). *El almacenamiento geológico profundo de los residuos radiactivos de alta actividad.* ENRESA, Madrid, 200 pp.

Barnichon, J.-D. and Volckaert, G. (2002). Hydro-mechanical coupling in the Boom Clay (Mol URL, Belgium): Recent in-situ observations, predictions and perspectives. *Hydromechanical and Thermohydromechanical Behaviour of Deep Argillaceous Rock* (Hoteit, N. et al. eds), A:A. Balkema, Lisse, 285-294.

Bernier, F. (2004). Private comunication.

BNWL (1974). *High level radioactive waste Management Alternatives.* BNWL-1900. Richland, Washington, Pacific Northwest Laboratotries.

Bossart, P., Meier, P.M., Moeri, A., Trick, T. and Mayor, J.-C. (2002). Geological and hydraulic characterisation of the excavation disturbed zone in the Opalinus clay of the Mont Terri Rock Laboratory. *Engineering Geology*, 66, 19-38.

Canadian Nuclear Society, (1996). *Proc. Int. Conf. on Deep Geological Disposal of Radioactive Waste.* Winnipeg, Manitoba.

Chapman, N.A. and Mc Combie, C. (2003). *Principles and standards for the disposal of long-lived radioactive wastes.* Pergamon, Amsterdam.

Chapman, N.A. and Mc Kinley, I.G. (1987). *The geological disposal of nuclear waste.* John Wiley: Chichester.

Emsley, S., Olsson, O., Stenberg, L., Alheid, H.-J., and Fall, S. (1997). *ZEDEX – A study of damage and disturbance from tunnel excavation by blasting and tunnel boring*. SKB Technical report 97-30.

European Commission (2004). I*mpact of the Excavation Disturbed or Damaged Zone (EDZ) on the Performance of Radioactive Waste Geological Repositories,* Proceedings of a European Commission CLUSTER Conference and Workshop. Luxembourg (C. Davies and F. Bernier, eds.), EUR 21028 EN

Gawin, D., Baggio, P. and Schhrefler, B.A. 1995. Coupled heat, water and gas flow in deformable porous media. *Int. J. Num. Meth. Fluids*, 20: 967-987.

Gens, A., Garcia-Molina, A.J., Olivella, S., Alonso, E.E. and Huertas, F. (1998). Analysis of Full Scale In-situ Heating Test Simulating Repository Conditions. *Int. Journal for Numerical and Analytical Methods in Geomechanics*, 22: 515-548.

Gens, A., Guimaraes, L. do N., Garcia-Molina, A. and Alonso, E.E. (2002a). Factors controlling rock-clay buffer interaction on a radioactive waste repository. *Engineering Geology,* 64, 297-308.

Gens, A., Guimaraes, L. do N. and Olivella, S. (2002b). Coupled chemomechanical analysis for saturated and unsaturated soils. *Environmental Geomechanics* (Vulliet, L., Laloui, L. nad Schrefler, B., eds.), EPFL Press, Lausanne, 109-123.

Gens, A. and Olivella, S. (2000a). Coupled thermo-hydro-mechanical analysis of saline materials. Application to radioactive waste containment. *European Congress on Computational Methods in Applied Sciences and Engineering*, Barcelona, ECCOMAS (published in CD ROM).

Gens, A. and Olivella, S. (2000b). Non isothermal multiphase flow in deformable porous media. Coupled formulation and application to nuclear waste disposal. Developments in Theoretical Geomechanics – *The John Booker Memorial Symposium* (D.W. Smith and J.P. Carter, eds), A.A. Balkema, Rotterdam, 619-640.

Guimaraes, L. N., Gens, A. and Olivella, S. (1999). THM and reactive coupling in unsaturated porous media. *7th Int. Symp. Numerical Models in Geomechanics*, Graz, Austria. 303-308.

IAEA (1992). *Current practices foe the Management and Confinement of Uranium Mill Tailings.* International Atomic Energy Agency Technical Report Series 335, Viena, Austria.

Kazda, I., Novak, J., Vanicek, I., Vanicek, M. (1997). Environmental impact of uranium mining in Northern Bohemia. *Geoenvironmental Engineering, Contaminated Ground: Fate of Pollutants and Remediation* (R.N. Yong and H.R. Thomas, eds.), Thomas Telford, London

Materials Research Society, (2001). *Scientific Basis for Nuclear Waste Management XXV*, Boston, US, Symposium Proceedings 713, Warrendale, Pa.

Matthews, M.L. (1986). UMTRA Project: Implementation of design. *Geotechnical and Geohydrogeological Aspects of Waste Management*, A.A. Balkema, Rotterdam, 15-21.

Meier, P.M., Blümling, P., Trick, T and Vlackaert, G. (2002). Self-healing of fractures within the EDZ at the Mont Terri rock laboratory: Results after one year of experimental work. *Hydromechanical and Thermohydromechanical Behaviour of Deep Argillaceous Rock* (Hoteit, N. et al. eds), A:A. Balkema, Lisse, 267-274.

Mc Combie, C., Pentz, D.L., Kurzeme, M. and Miller, I. (2000). Deep geological repositories: a safe and secure solution to disposal of nuclear wastes, *GeoEng2000, An International Conference on Geotechnical and Geological Engineering*, Melbourne. Technomic Publishing, Lancaster Pa, 1, 686-691.

NAS (1957). *The disposal of Radioactive Waste on Land.* Publication 519, Washington D.C., National Academy Press.

NEA- Nuclear Energy Agency (1997). *Lessons learnt from Ten Performance Assessment Studies*. OECD, Paris

NEA- Nuclear Energy Agency (1999). *Confidence in the long.term safety of deep geological repositories: its development and communication*. OECD, Paris

NEA- Nuclear Energy Agency (2001). *Considering reversibility and retrievability in geological disposal of radioactive waste*. OECD, Paris, 39 pp.

NEA- Nuclear Energy Agency (2002a). *Nuclear Energy and the Kyoto Protocol*. OECD, Paris, 52 pp.

NEA- Nuclear Energy Agency (2002b). *Characterization and representation of the Excavation Disturbed or Damaged Zone (EDZ)*. OECD, Paris, 83 pp.

NRC (2001). *Disposition of High-Level Waste and Spent Nuclear Fuel.* National Research Council D.C., National Academy Press.

Olivella, S., Carrera, J., Gens A. and Alonso, E.E. (1994). Non-isothermal Multiphase Flow of Brine and Gas through Saline media. *Transport in Porous Media*, 15: 271-293.

Olivella, S., Carrera, J., Gens A. and Alonso, E.E. (1996a). Porosity variations in saline media caused by temperature gradients coupled to multiphase flow and dissolution/precipitation. *Transport in Porous Media*, 25: 1-25.

Olivella, S., Gens, A., Carrera, J. and Alonso, E.E. (1996b). Numerical Formulation for a Simulator (CODE_BRIGHT) for the Coupled Analysis of Saline Media. *Engineering Computations,* 13: 87-112.

Sun, Y., Nolting III, R.M., Cogan, J. (1999). Expected performance of stress relief elements for ground support at the Yucca Mountain Repository. *Geo-Engineering for Underground Facilities*, Geotechnical Special Publication 90 (G. Fernandez and R.A. Bauer, eds), ASCE, 829-840.

Tanious, N.M., Nataraja, M.S. and Daemen, J.J.K. (1987). Retrievability of high-level nuclear waste from geologic repositories – Regulatory and rock mechanics/design considerations. *Rock Mechanics: Proceedings of the 28th US Symposium*, A.A. Balkema, Rotterdam, 1025- 1033.

Thomas. H.R., Cleall, P.J. and Hashm, A.A. (2002). Thermal/hydraulic/chemical/mechanical (THCM) behaviour of partly saturated soil. *Computer Methods and Advances in Geomechanics* (Desai et al., eds), A.A. Balkema, Rotterdam, 1, 756-763.

Thomas, H.R. and He, Y. (1995). An analysis of coupled heat, moisture and air transfer in a deformable unsaturated soil, *Géotechnique,* 45: 667-689.

Tsang, C-F-, Bernier, F., Davies, C. (2005). Geohydromechanical processes in the Excavation Damaged Zone in crystalline rock, rock salt, and indurated and plastic clays – in the context of radioactive waste disposal. *International Journal of Rock Mechanics and Mining Sciences*, 42, 109-125.

UNEP (1985). *Radiation, doses, effects, risks*. United Nations Environmental Programme, Nairobi.

Vieno, T., Nordman, H., Salo, J.-P., Vira, J. (1996). Site-specific analysis of repository performance-four years to go to site selection. *Proc. Int. Conf. on Deep Geological Disposal of Radioactive Waste*. Winnipeg, Manitoba. Canadian Nuclear Society, 8-139: 8-148.

Whiterspoon, P. (ed.), (1996). *Geological problems for radioactive waste isolation: second world-wide review*. Report LBNL-38915, UC-814, University of California, USA.

Wieczorek, K. and Zimmer, U. (1998). *Untersuchungen zur Auflockerungszone um Hohlräume im Steinsalzgebirge*. Abschlußbericht, Gesellschaft für Anlagen- und Reaktorsicherheit (GRS) mbH, GRS-A-2651, Köln.