

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

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John BOWDERS, *University of Missouri-Columbia, USA*
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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.

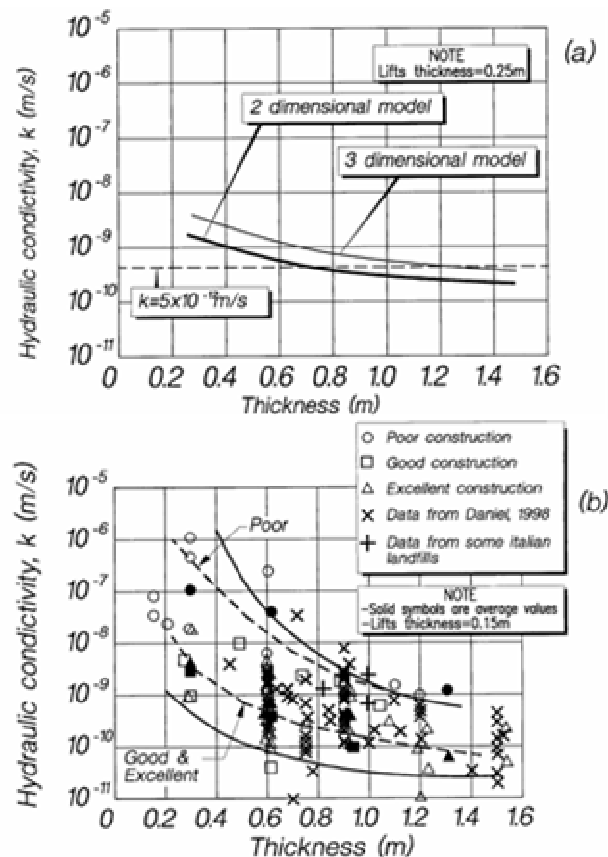


Figure 1. Hydraulic conductivity of compacted soil liners versus thickness: (a) theoretical assessment, (b) experimental trend from field data (Jessberger et al., 1993; Benson & Daniel, 1994a, b)

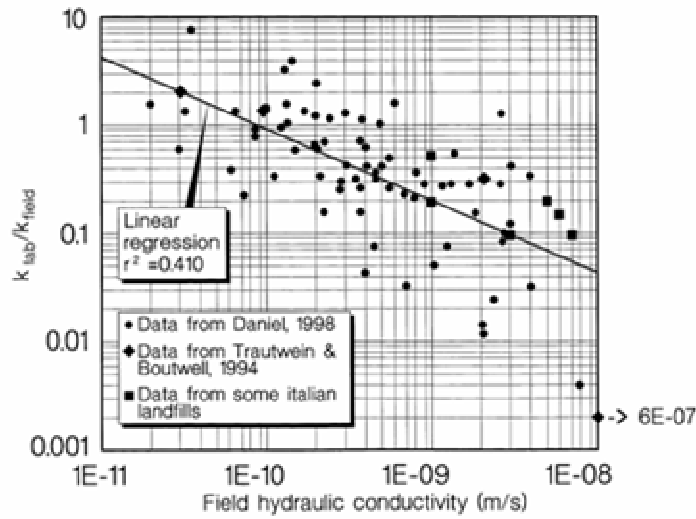


Figure 2. Ratio of laboratory (k_{lab}) to field (k_{field}) HC vs field HC (Manassero et al., 2000)

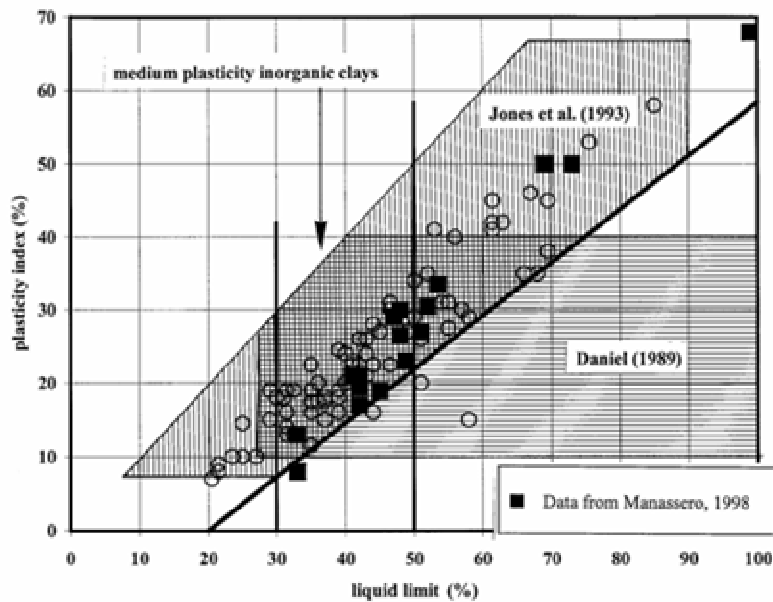


Figure 3. Index properties of some compacted clays (Manassero et al., 2000)

Figure 4 shows field HC versus the percentage of water content-dry density ($w-\gamma_d$) 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.

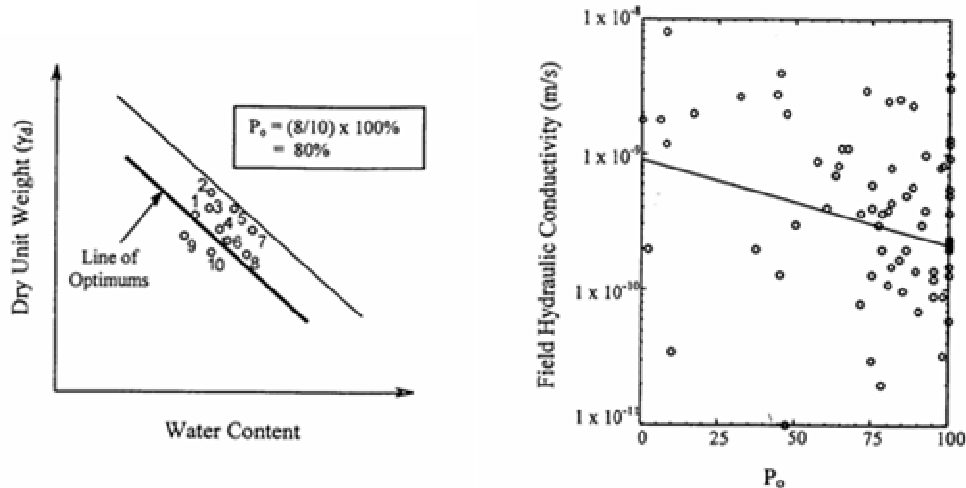


Figure 4. Relationship between field HC and percentage (P_o) of tests results laying above the line of optima (Daniel, 1998)

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 & Fratolocchi, 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.

Heibroek (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 than 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:

$$K_d = K_{oc} \cdot f_{oc} \quad (1)$$

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):

$$\frac{\partial S}{\partial t} = \alpha(S_\infty - S) \quad (2)$$

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 $s/w \geq 3/1$ 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.

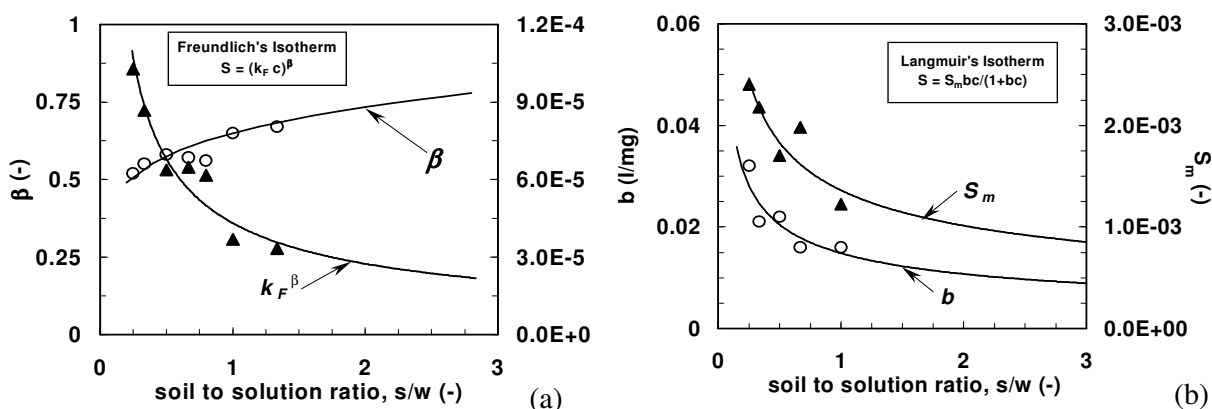


Figure 5. Influence of soil to solution ratio on the parameters of (a) Freundlich's and (b) Langmuir's sorption isotherm (Manassero et al., 1998)

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.

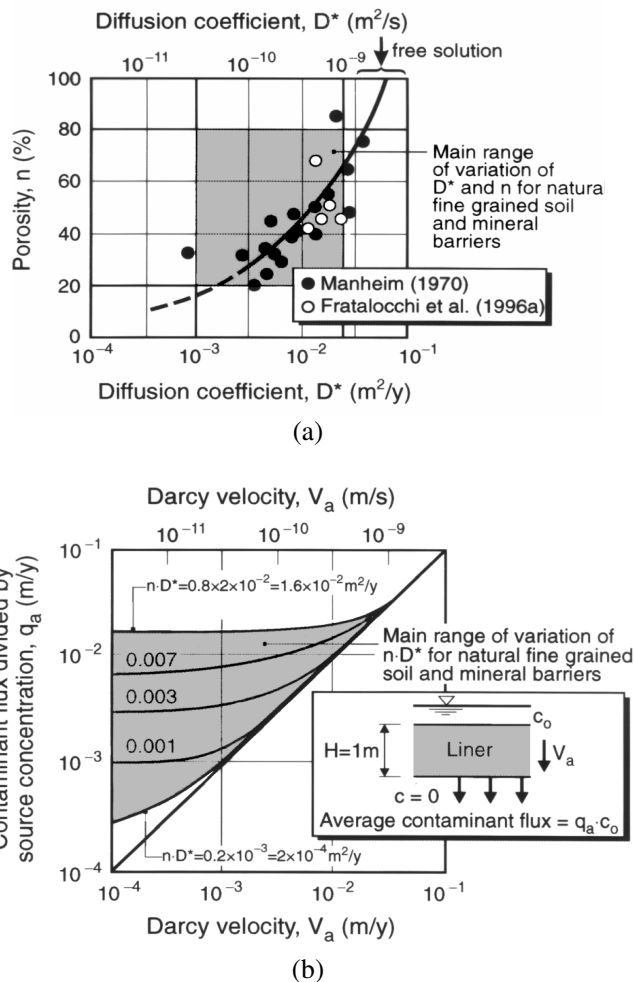


Figure 6. (a) Range of diffusion coefficients for soils and mineral barriers; (b) relative importance of diffusion and advection through mineral barriers (from Manheim, 1970; Rowe, 1988; Jessberger, 1995)

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.

INPUT DATA

mineral layer reference permeability : $K_0 = 10^{-11}$ m/s	aquifer Darcy velocity : $V = 4.76 \cdot 10^{-3}$ m/s (plus the contribution from the landfill)
mineral layer hydraulic gradient : $i = 1.3$ -	aquifer thickness : $h = 3.0$ m
mineral layer porosity : $n = 0.5$ -	aquifer porosity : $n = 0.3$ -
mineral layer thickness : $L = 1$ m	landfill length : $l = 100$ m
mineral layer pure diffusion : $D^* = 4.0 \cdot 10^{-10}$ m ² /s for $v \leq 4.0 \cdot 10^{-3}$ m/s	boundary conditions : $c = \text{constant @ the top and mixing aquifer @ the bottom}$
mineral layer diffusion-dispersion : $D = 0.01 v$ m ² /s for $v > 4.0 \cdot 10^{-3}$ m/s	

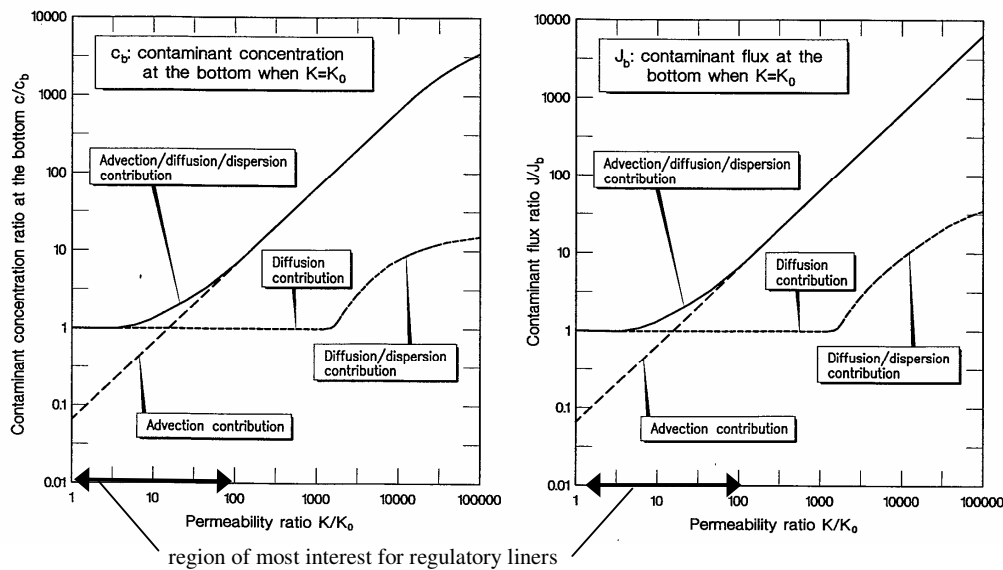


Figure 7. Contribution of advection/diffusion/dispersion to contaminant migration in terms of concentration and flux (Manassero, 1997)

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/m² 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.

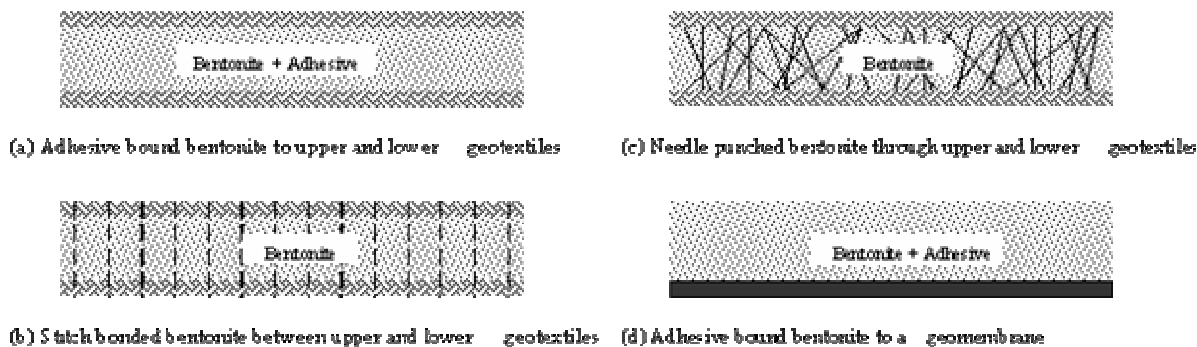


Figure 8. Cross section of geosynthetic clay liners (Daniel & Koerner, 1995)

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	<i>Ease of Placement</i>	X			
	<i>Material Availability</i>	X			
	<i>Puncture Resistance</i>			X	
	<i>Quality Assurance</i>	X			
	<i>Speed of Construction</i>	X			
	<i>Subgrade Condition</i>	X			X
Contaminant Transport Issues	<i>Water Requirements</i>	X			X
	<i>Weather Constraints</i>				
	<i>Attenuation Capacity</i>				
	<i>Gas Permeability</i>			X ⁽¹⁾	X
Hydraulic Issues	<i>Solute Flux and Breakthrough Time</i>	X ⁽²⁾		X	X
	<i>Compatibility</i>	X ⁽²⁾		X	
	<i>Consolidation Water</i>	X			
	<i>Steady Flux of Water</i>		X		
Physical/Mechanical Issues	<i>Water Breakthrough Time</i>				X
	<i>Bearing Capacity</i>				X
	<i>Erosion</i>				X
	<i>Freeze-Thaw</i>	X			
	<i>Settlement-Total</i>	X	X		
	<i>Settlement-Differential</i>	X			
	<i>Slope Stability</i>				X
	<i>Wet-Dry</i>	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:

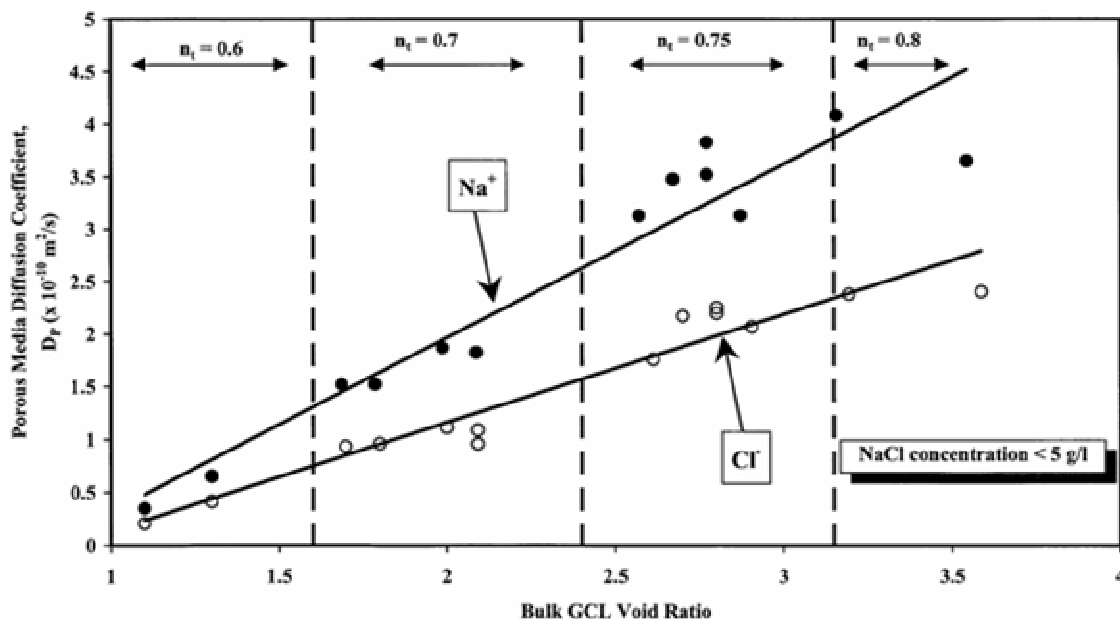


Figure 9. Typical values of total porosity (n_t) and porous media diffusion coefficient of sodium and chloride vs bulk GCL void ratio (Rowe, 1998a)

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 ($k_b/k_a; n_b D_b/n_a D_a$).

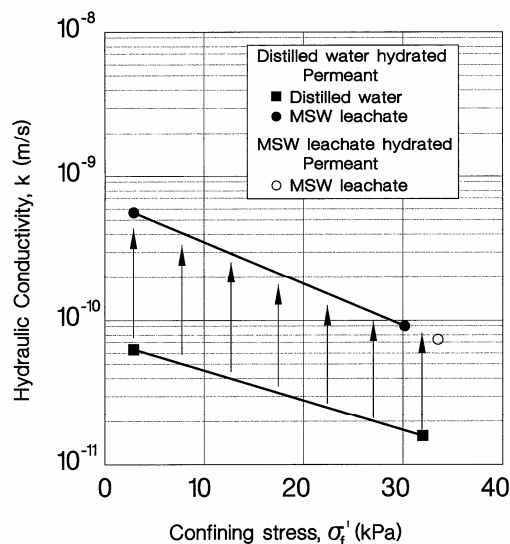


Figure 10. Hydraulic conductivity of a GCL vs static confining stress for permeation of synthetic MSW leachate (modified from Petrov & Rowe, 1997)

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).

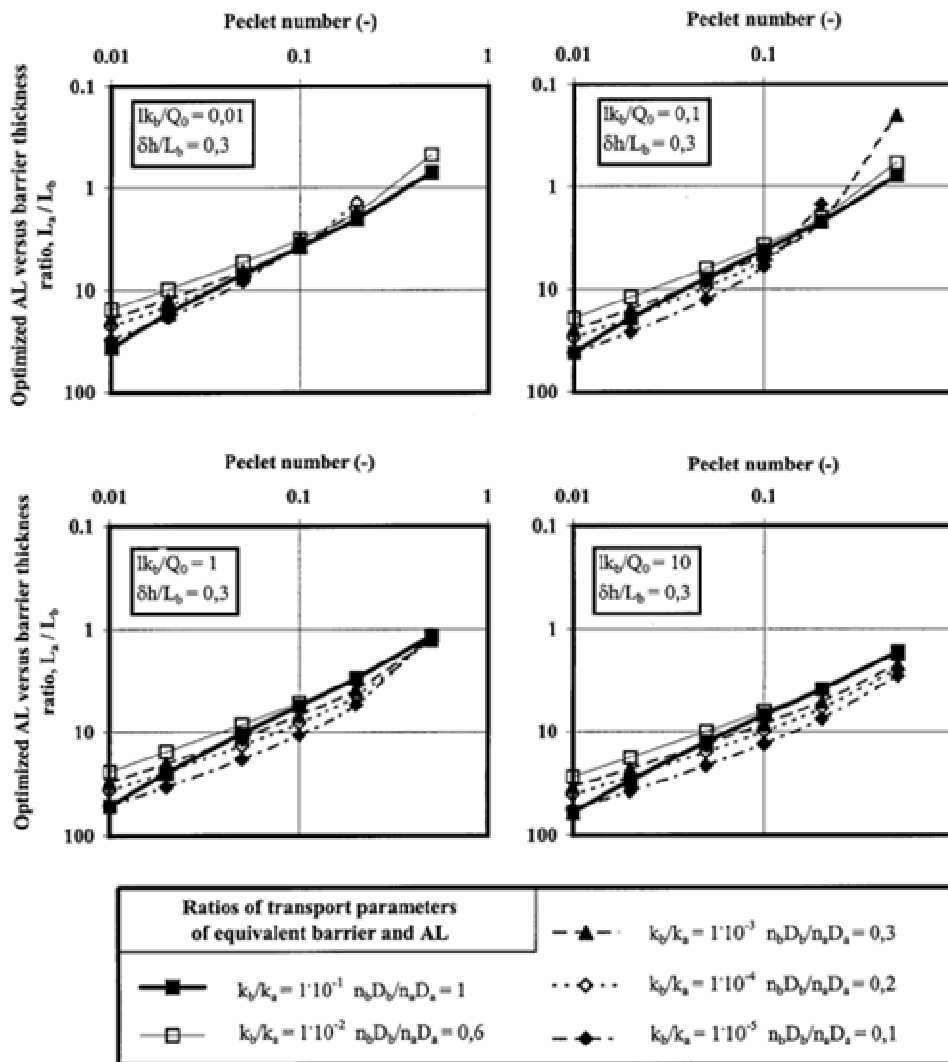


Figure 11. Optimized thickness of AL referring to steady state conditions (Manassero et al., 2000)

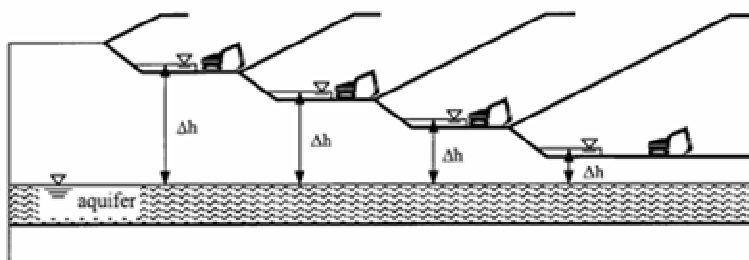


Figure 12. Increase of landfill capacity vs decrease of attenuation layers thickness (Manassero et al., 2000)

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				SLCRS		PLCRS		SLCRS		
GM		Clay				Flow Rates		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)

	Cells	Average Flows				Peak Flows			
		PLCRS		SLCRS		PLCRS		SLCRS	
		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.

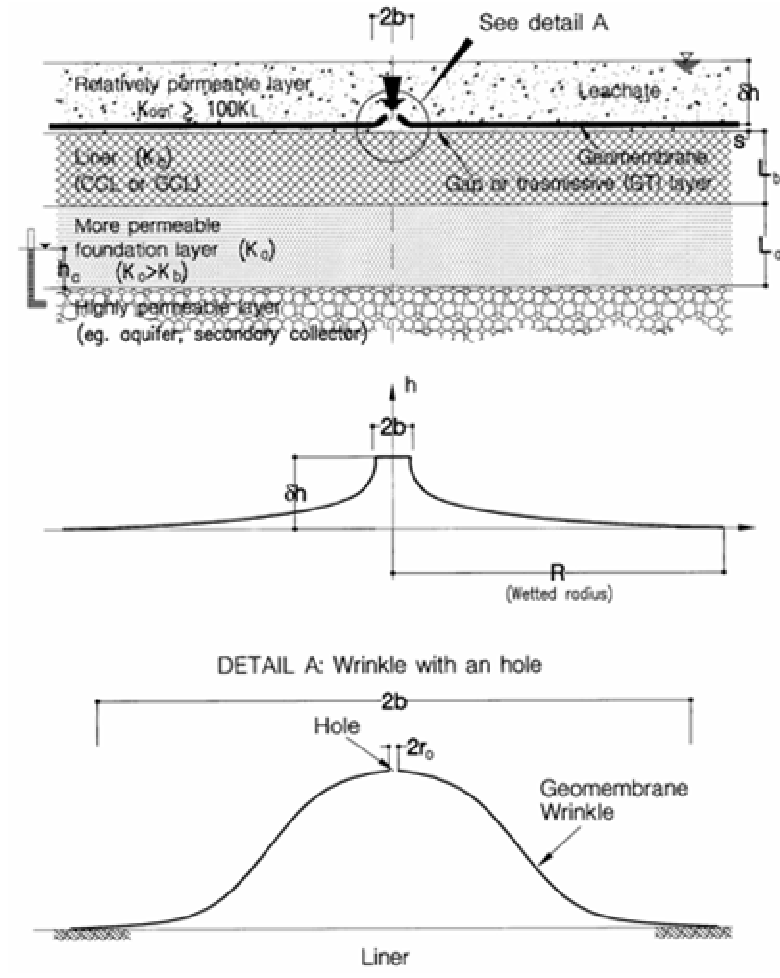


Figure 13. Schematic showing a hole of radius r_0 in a wrinkle of a geomembrane and the underlying strata together with the head distribution between the geomembrane and clay liner (Rowe, 1998a)

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 ($r_0=0.001\text{m}$) in a wrinkle: $L=3\text{m}$, $2b=0.2\text{m}$, $h_a=0$, $q_0=1*10^{-9}\text{ m/s}$ (Rowe, 1998a)

Liner H K_L θ	h_w (m)	Q (m^3/s)	Leakage* (lphd)
CCL	0.03	$3.0*10^{-9}$	0.6
0.6 m	0.3	$2.4*10^{-8}$	5.2
$1*10^{-9}\text{ m/s}$	3	$2.4*10^{-7}$	51
$1*10^{-7}\text{ m/s}$	30	$2.4*10^{-6}$	510
CCL	0.03	$1.6*10^{-9}$	0.3
0.6 m	0.3	$1.0*10^{-8}$	2.2
$1*10^{-9}\text{ m}^2/\text{s}$	3	$9.7*10^{-8}$	21
$1.6*10^{-8}\text{ m}^2/\text{s}$	30	$9.6*10^{-7}$	210
GCL	0.03	$7.8*10^{-9}$	1.7
0.01m+	0.3	$1.3*10^{-8}$	2.9
0.59m AL	3	$6.9*10^{-8}$	15
$2*10^{-10}\text{ m/s}$	30	$6.2*10^{-7}$	130
$1*10^{-10}\text{ m}^2/\text{s}$			

* 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	$4.4*10^{-12}$	0.09	(a)
Toluene			
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° (e.g. the primary liner of a municipal waste landfill) and 350 years at a temperature of 12°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).

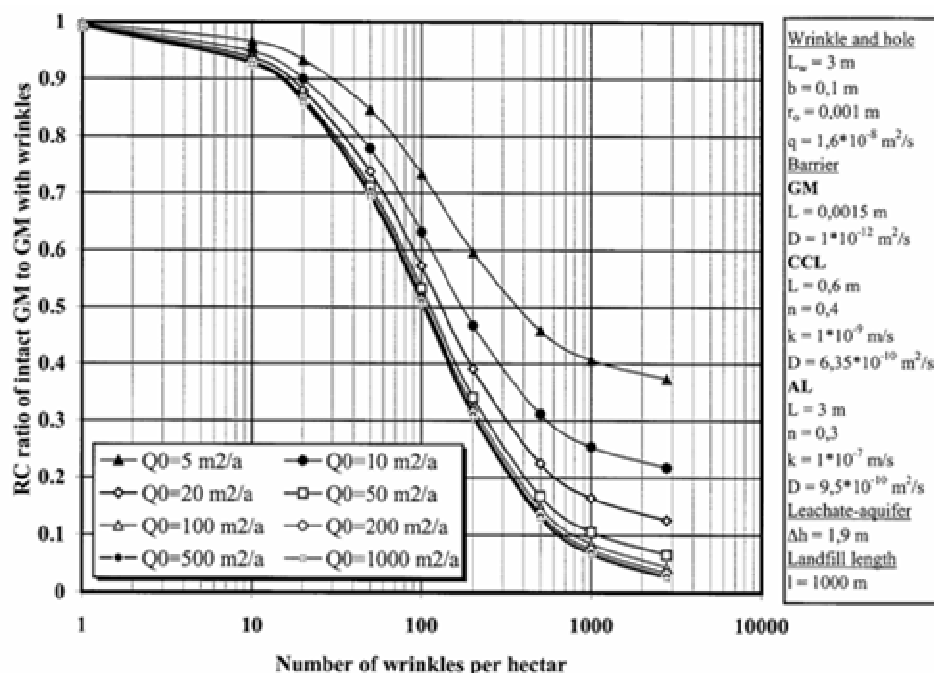


Figure 14. Efficiency of GM+CCL+AL barrier systems vs the number of wrinkles (with holes) in the GM

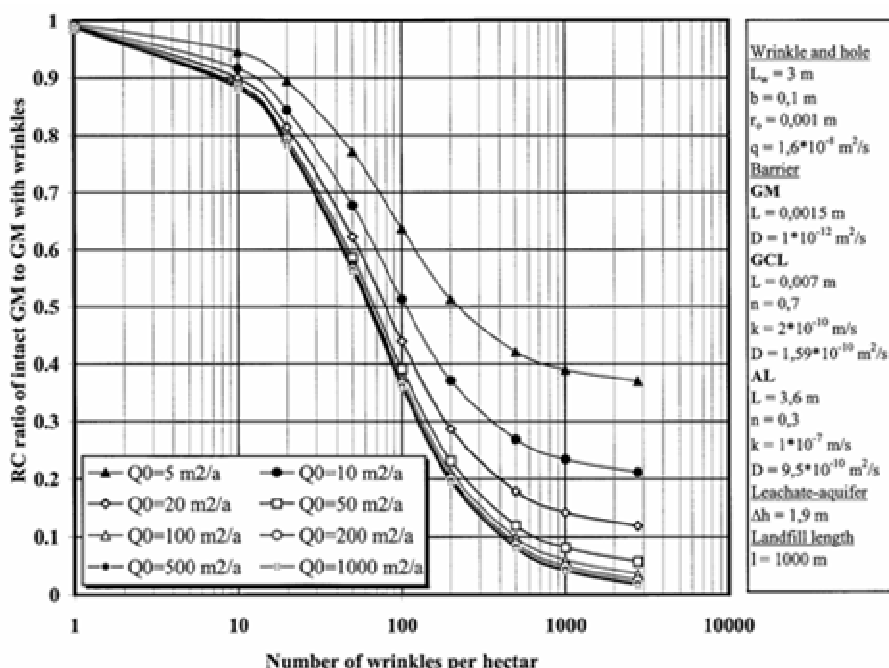


Figure 15. Efficiency of GM+GCL+AL barrier systems vs the number of wrinkles (with holes) in the GM

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°.

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.

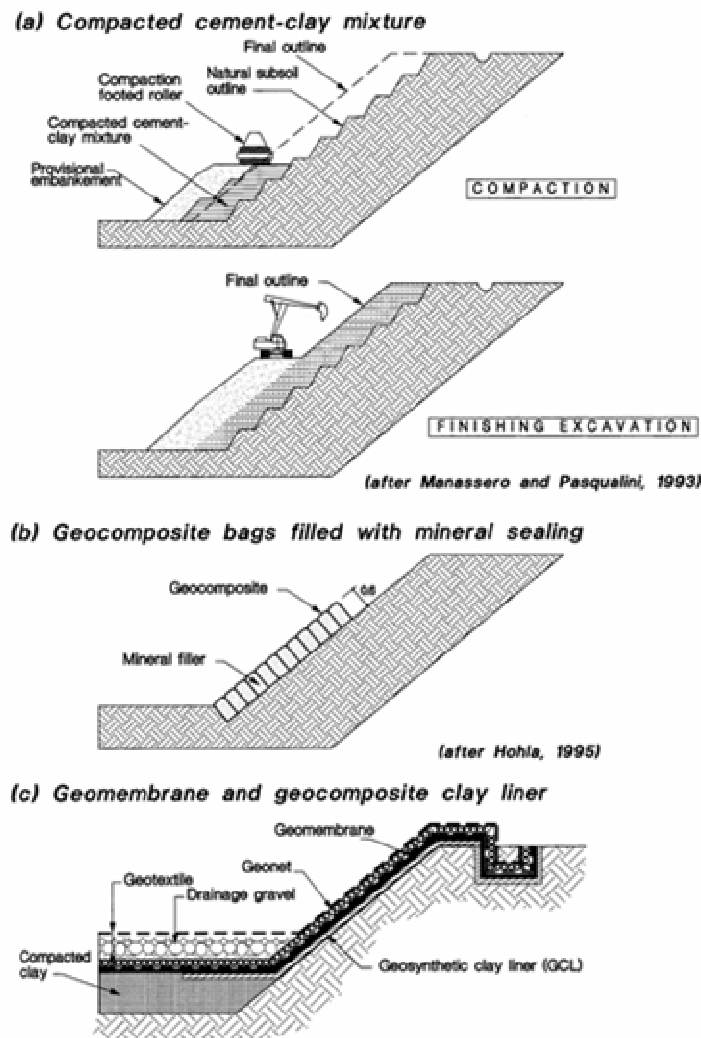


Figure 16. Alternative liners for steep side slopes (Manassero & Pasqualini, 1993; Hohla, 1995)

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)

<u>GEOSYNTHETIC - SOIL INTERFACE</u>	
Geomembrane (HDPE) - Sand	$\phi = 15^\circ$ to 28°
Geomembrane (HDPE) - Clay	$\phi = 5^\circ$ to 29°
Geotextile - Sand	$\phi = 22^\circ$ to 44°
Geosynthetic clay liner - Sand	$\phi = 20^\circ$ to 25°
Geosynthetic clay liner - Clay	$\phi = 14^\circ$ to 16°
Textured HDPE - Compacted clay	$\phi = 7^\circ$ to 35° $c' = 20$ to 30 kPa
Textured HDPE - Pea gravel	$\phi = 20^\circ$ to 25°
Textured HDPE - Sand	$\phi = 30^\circ$ to 45°
Geotextile - Clay	$\phi = 15^\circ$ to 33°
<u>GEOSYNTHETIC - GEOSYNTHETIC INTERFACE</u>	
Geonet - Geomembrane (HDPE)	$\phi = 6^\circ$ to 10°
Geomembrane (HDPE) - Geotextile	$\phi = 8^\circ$ to 18°
Geotextile - Geonet	$\phi = 10^\circ$ to 27°
Geosynthetic clay liner - Textured HDPE	$\phi = 15^\circ$ to 25°
Geosynthetic clay liner - Geomembrane (HDPE)	$\phi = 8^\circ$ to 16°
Geosynthetic clay liner - Geosynthetic clay liner	$\phi = 8^\circ$ to 25° $c' = 8$ to 30 kPa
Textured HDPE - Geonet	$\phi = 10^\circ$ to 25°
Textured HDPE - Geotextile	$\phi = 14^\circ$ 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.

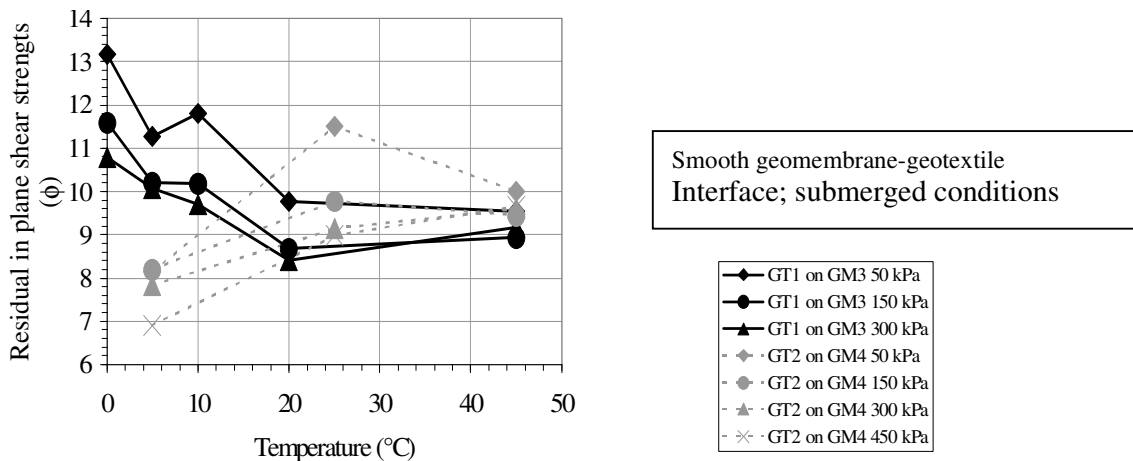


Figure 17. Influence of temperature on the residual equivalent friction angle at the interface between different geotextiles and geomembranes (Pasqualini et al., 2002).

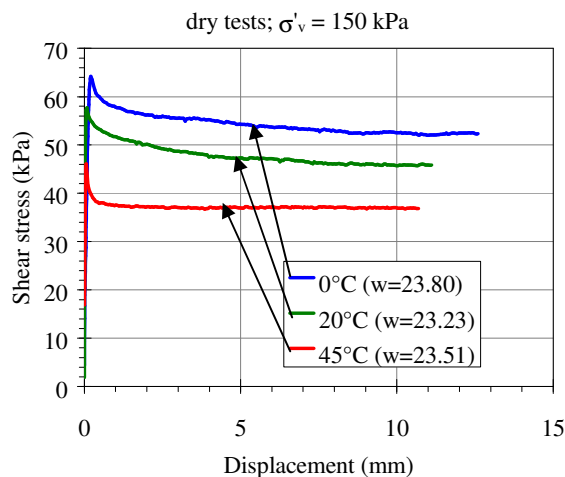


Figure 18. Influence of temperature on the interface shear strength at the interface between a compacted clay and a smooth geomembrane (Stella, 1998).

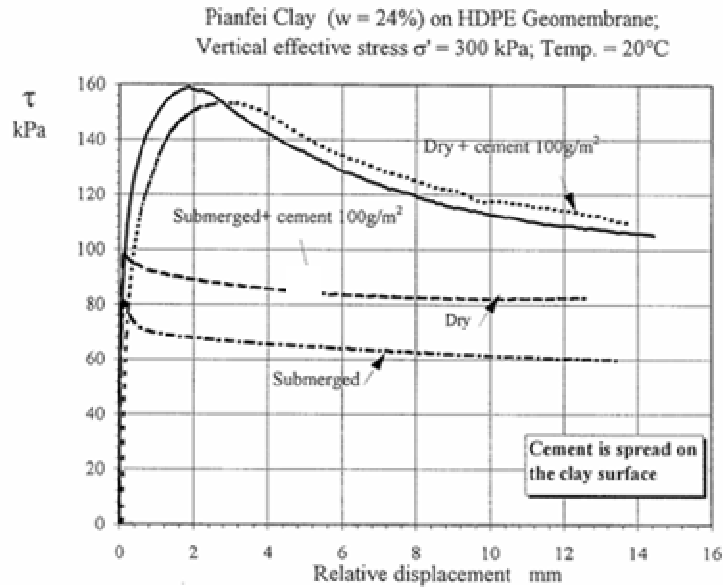


Figure 19: Direct shear tests results on compacted clay-geomembrane interfaces (Pasqualini & Stella, 1994)

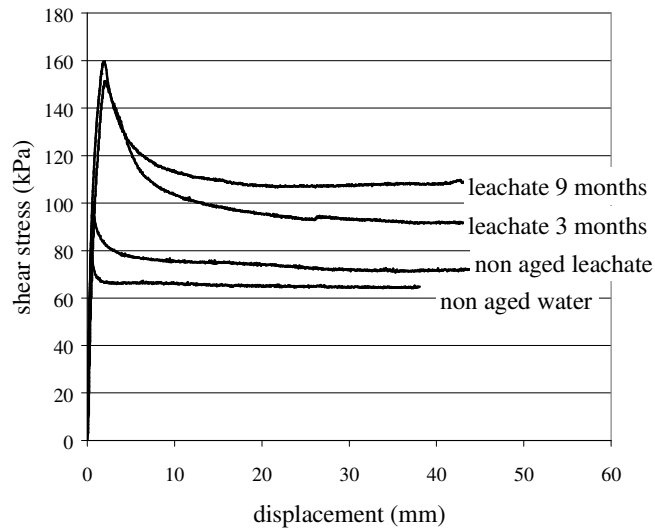


Figure 20. Influence of aging in leachate on the shear strength at smooth geomembrane-geotextile interface (Pasqualini et al., 2002).

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.

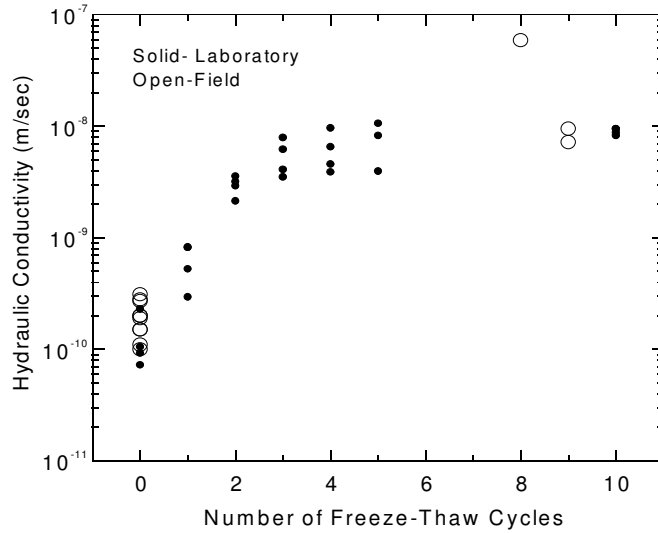


Figure 21. Hydraulic conductivity of compacted clay vs number of freeze-thaw cycles (Benson et al., 1995).

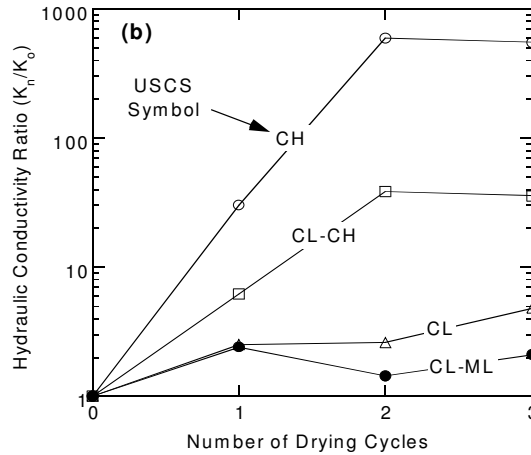


Figure 22. Hydraulic conductivity ratio (hydraulic conductivity after a specified number of wet-dry cycles, K_n , divided by the initial hydraulic conductivity, K_0) vs. number of wet-dry cycles (Albrecht, 1996).

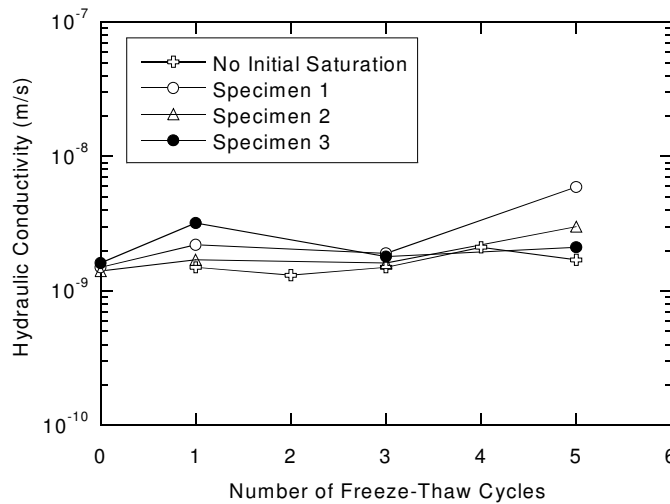


Figure 23. Hydraulic conductivity vs freeze-thaw cycling for a soil-bentonite mixture (Kraus et al., 1997)

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° (Figure 24); at 6° 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).

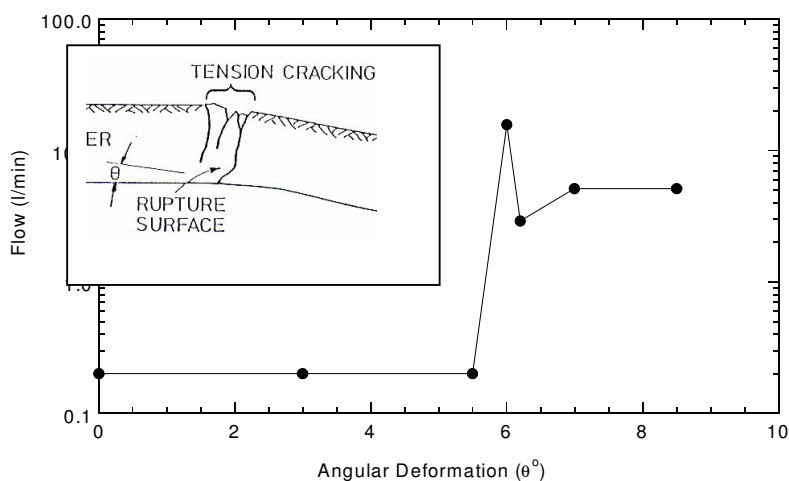


Figure 24. Flow rates measured at various angular deformations (Jessberger and Stone, 1991).

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 to 150 mm/a in humid climates (10-20% of precipitation) and 30 mm/a in semi-arid climates (~12% of precipitation).

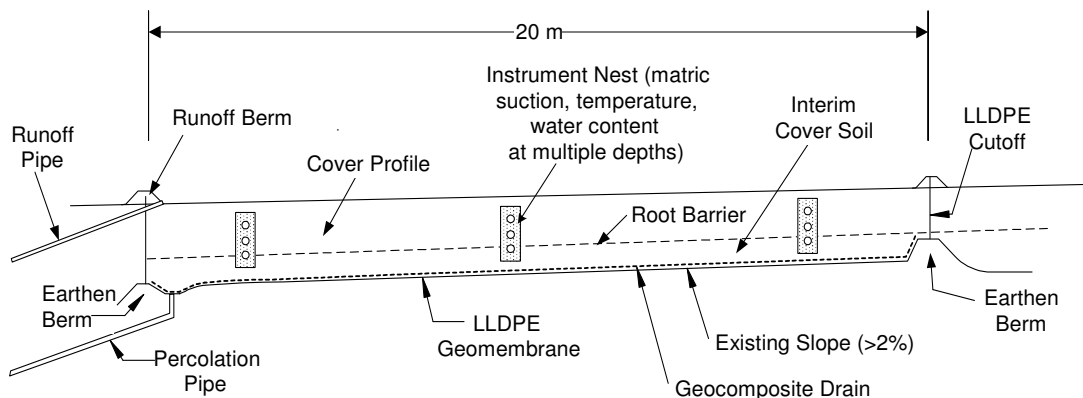


Figure 25. Cross-section of a typical test section used in USEPA's Alternative Cover Assessment Program (Benson et al., 1999)

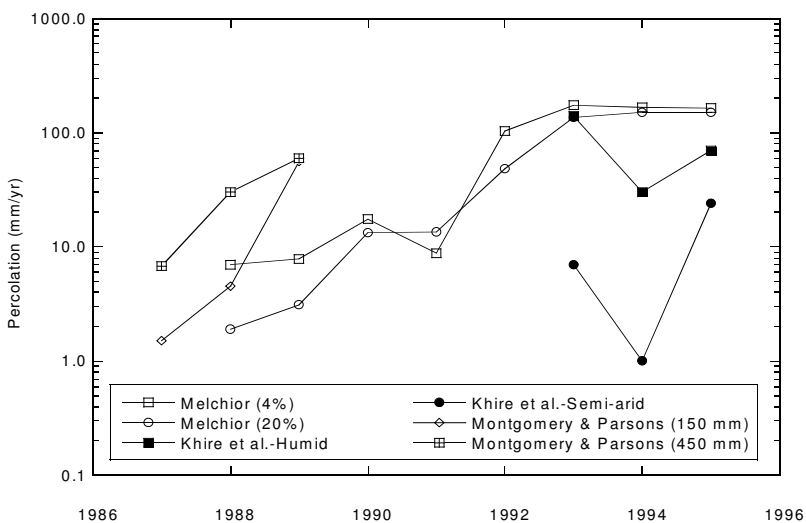


Figure 26. Annual percolation from test sections simulating compacted clay covers (data from Montgomery and Parsons, 1990; Melchior, 1997; Khire et al., 1997).

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.

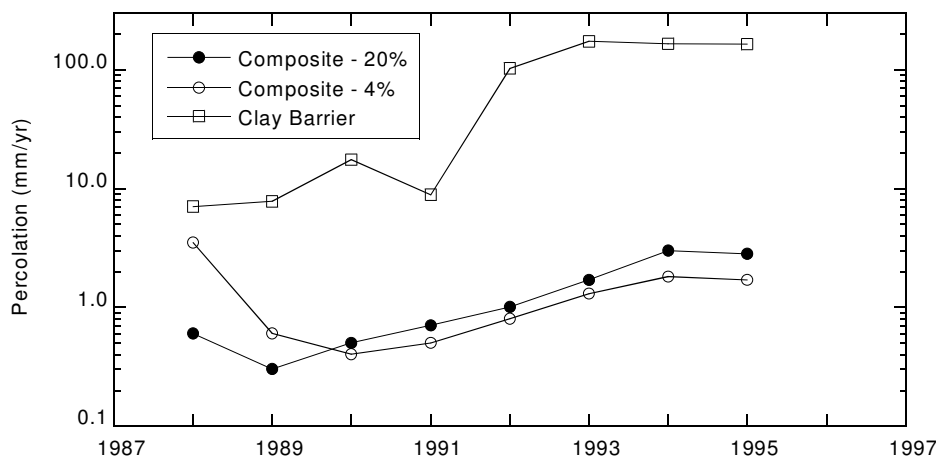


Figure 27. Percolation data from Melchior (1997) for test sections representing composite and compacted clay covers.

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.

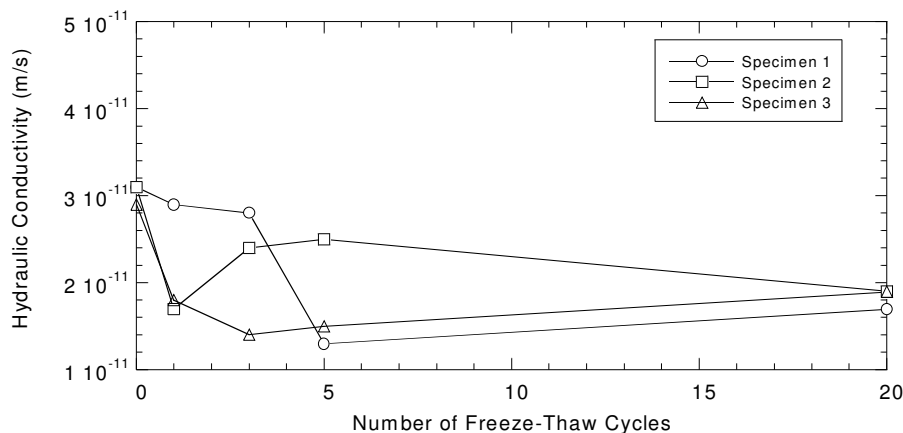


Figure 28. Hydraulic conductivity of GCLs vs freeze-thaw cycling (lab tests; Kraus et al. 1997).

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 ($\sim 10^{-10}$ 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.

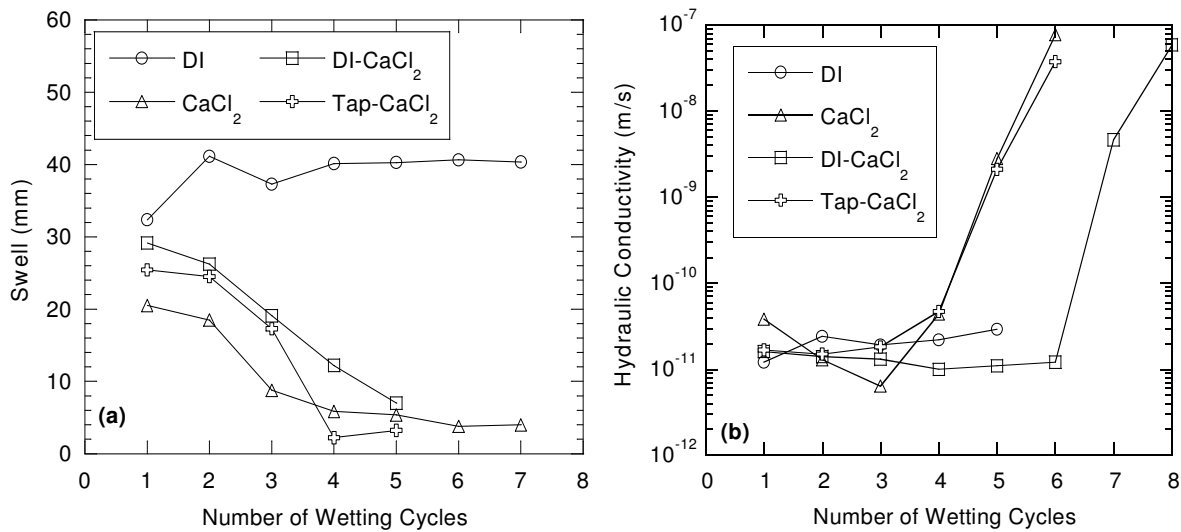


Figure 29. Swell (a) and hydraulic conductivity (b) of GCL subjected to wet-dry cycling using DI water and 0.0125 M CaCl₂ solution (adapted from Lin and Benson 2000).

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 3×10^{-9} to 7×10^{-9} m/s. A specimen of the exhumed GCL was tested and found to have very similar hydraulic conductivity (2×10^{-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).

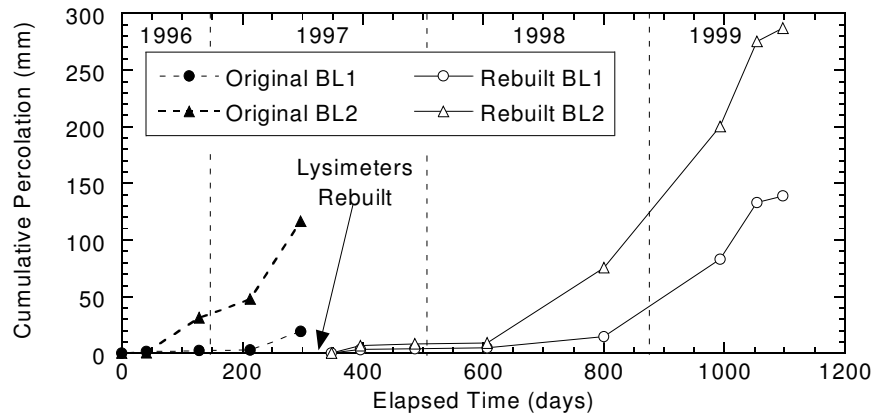


Figure 30. Percolation rates recorded by two lysimeters beneath the GCL cover over a fly ash landfill in Wisconsin, US (Manassero et al., 2000).

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:

$$Q = -k \frac{K_r}{\mu} A \frac{dP}{dx} \quad (1)$$

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:

$$Q = -\frac{K}{\rho g} A \frac{dP}{dx} \quad (2)$$

where g is the acceleration due to gravity and K is given by:

$$K = \frac{\rho g}{\mu} k K_r \quad (3)$$

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:

$$\frac{\rho_0 T_0}{P_0} = \frac{\rho T}{P} \quad (4)$$

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:

$$\frac{d}{dx}(\rho Q) = 0 \quad (5)$$

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:

$$\frac{d^2}{dx^2}(P^2) = 0 \quad (6)$$

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:

$$P^2 = P_1^2 + \left(\frac{P_2^2 - P_1^2}{L} \right) x \quad (7)$$

From equation 2 and 7 the volumetric flow of gas at distance x can be obtained from the following equation:

$$Q_x = -\frac{K A}{2\rho g L} \frac{(P_2^2 - P_1^2)}{\sqrt{P_1^2 + \frac{(P_2^2 - P_1^2)}{L} x}} \quad (8)$$

Considering the volumetric flow of gas at a distance L , equation 8 becomes:

$$Q_L = -\left(\frac{K}{\rho g} \right) A \frac{(P_2^2 - P_1^2)}{2LP_2} \quad (9)$$

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).

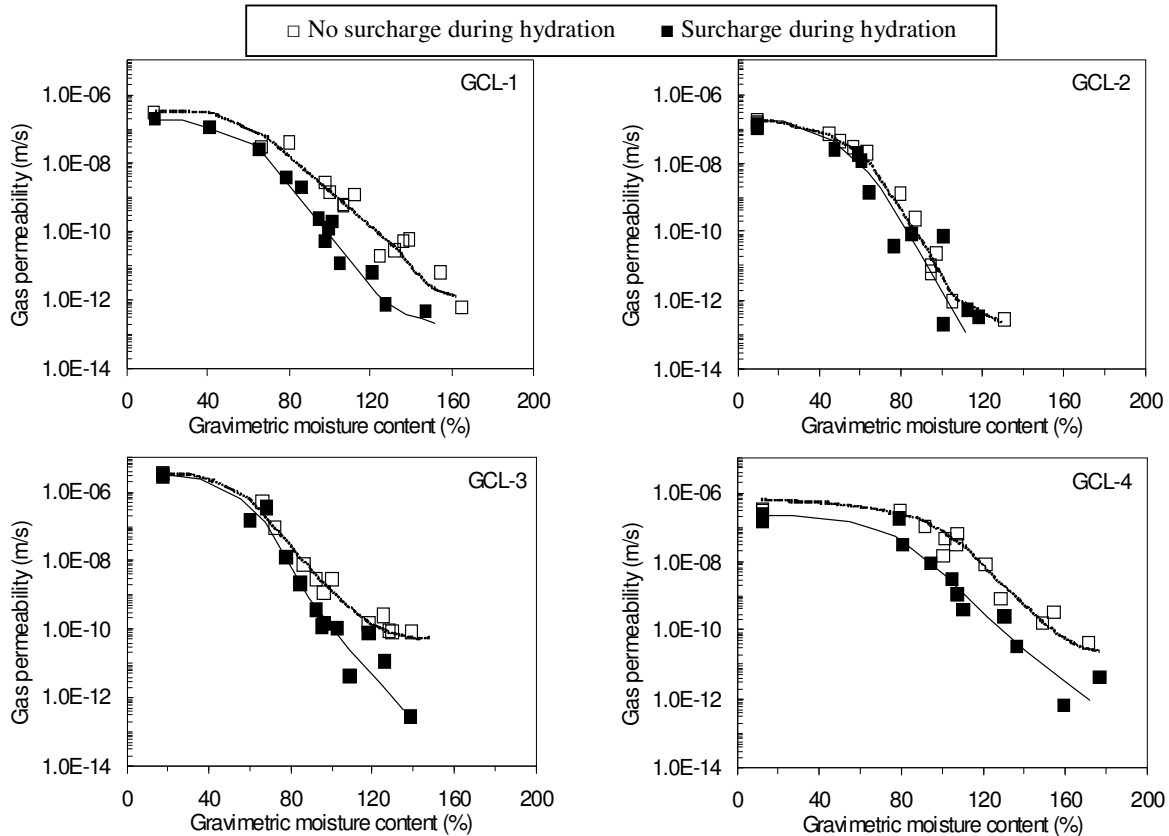


Figure 31. Relation between gas permeability and gravimetric moisture content for 4 different GCLs.

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.

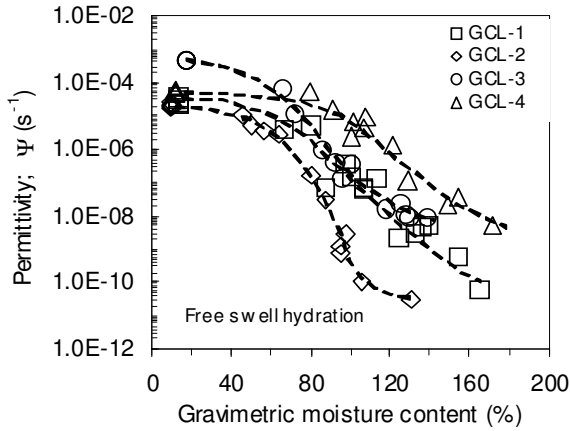


Figure 32 Variations of gas permittivity with gravimetric moisture content for GCLs under confined hydration. (from Vangpaisal & Bouazza, 2004)

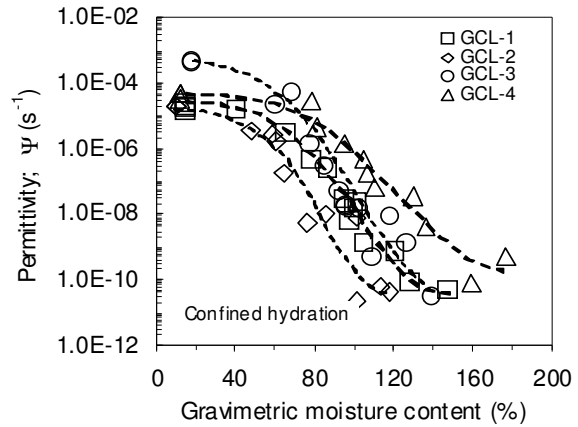


Figure 33 Variations of gas permittivity with gravimetric moisture content for GCLs under free swell hydration. (from Vangpaisal & Bouazza, 2004)

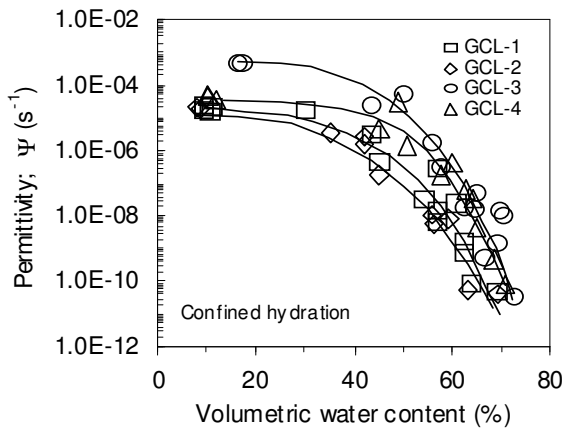


Figure 34 Variations of gas permittivity with volumetric water content for GCLs under confined hydration. (from Vangpaisal & Bouazza, 2004)

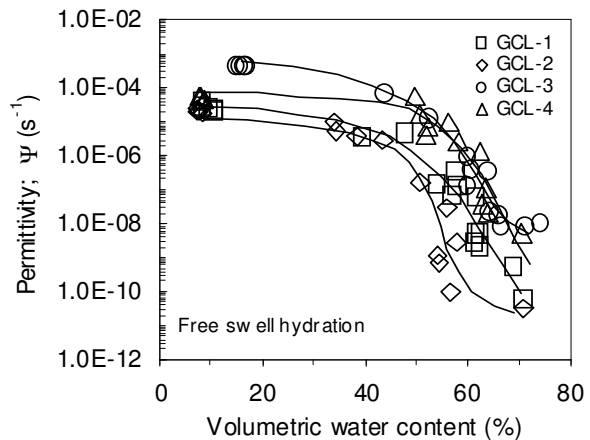


Figure 35 Variations of gas permittivity with volumetric water content for GCLs under free swell hydration. (from Vangpaisal & Bouazza, 2004)

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:

$$J_g = -n_e D_e \frac{\partial c}{\partial z} \quad (10)$$

where, J_g is the mass diffusive flux of a gas [M/L²T], n_e is the soil porosity available for solute diffusion (*i.e.* effective porosity), c is the gas concentration in the gaseous phase [M/L³], D_e is the effective diffusion coefficient of gas [L²/T], z is a distance (thickness, or height, etc.) [L], $\partial c/\partial z$ is the concentration gradient [M/L⁴]. 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.*,

$$\frac{\partial J}{\partial z} = n_e \frac{\partial c}{\partial t} \quad (11)$$

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:

$$\frac{\partial c}{\partial t} = D_e \frac{\partial^2 c}{\partial z^2} \quad (12)$$

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.

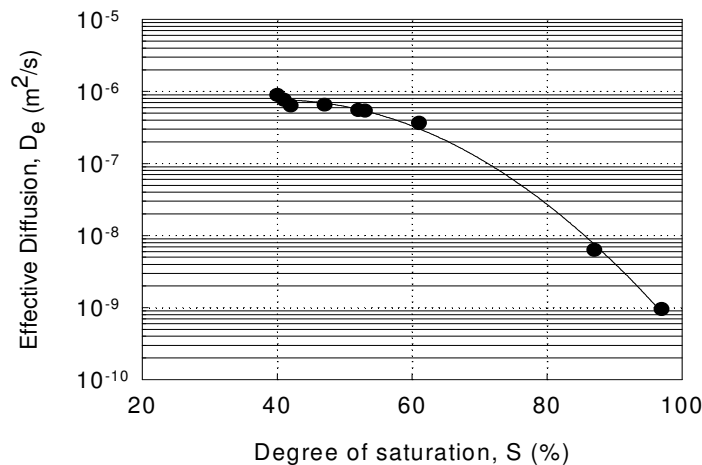


Figure 36. Effective diffusion coefficient of oxygen vs degree of saturation (Bouazza and Rahman, 2004)

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 ($S \geq 90\%$) 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.

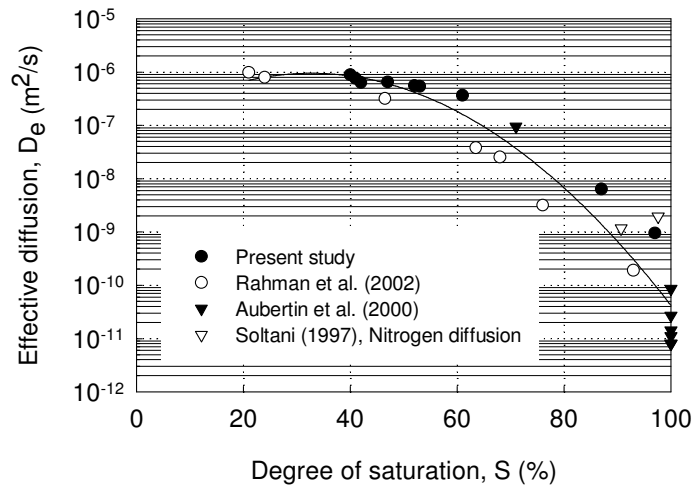


Figure 37. Effective diffusion coefficient of oxygen vs degree of saturation in different GCLs (Bouazza and Rahman, 2004)

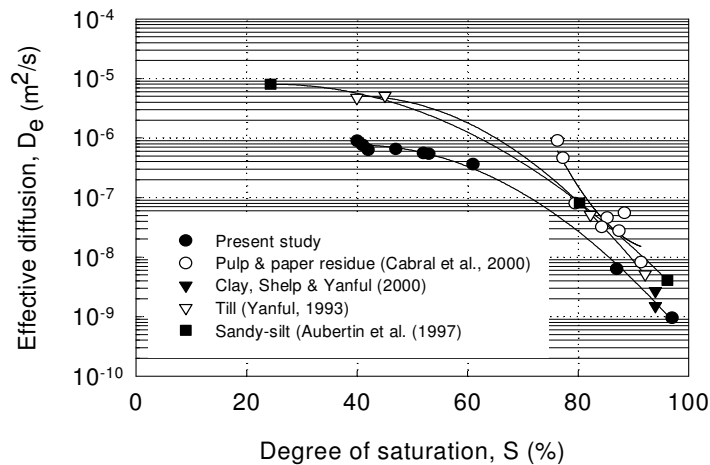


Figure 38. Effective diffusion coefficient of oxygen vs degree of saturation: comparison between study and soils (Bouazza and Rahman, 2004)

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° (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 (needed 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).

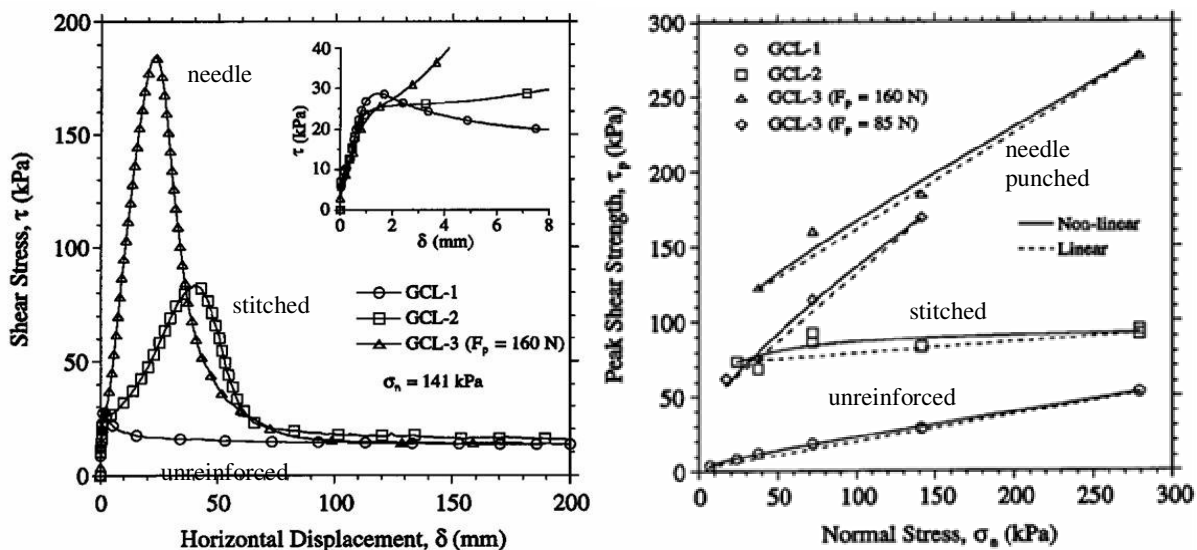


Figure 39. Shear strength of different types of GCLs (Fox et al., 1998)

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).

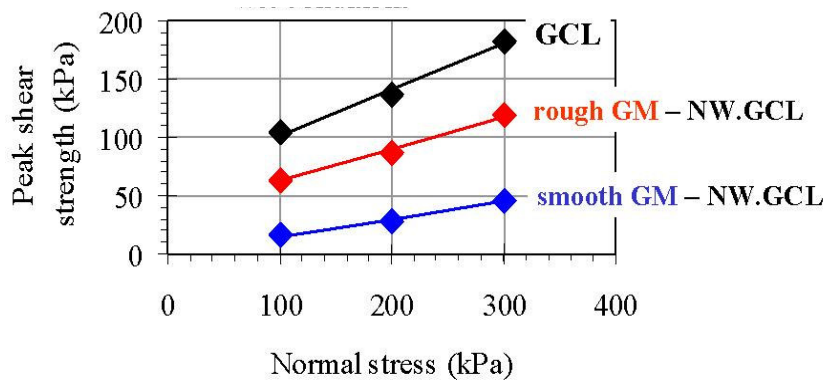


Figure 40. Comparison between peak shear strength of a needle-punched GCL and the shear strength at the interface with a smooth and a rough geomembrane.

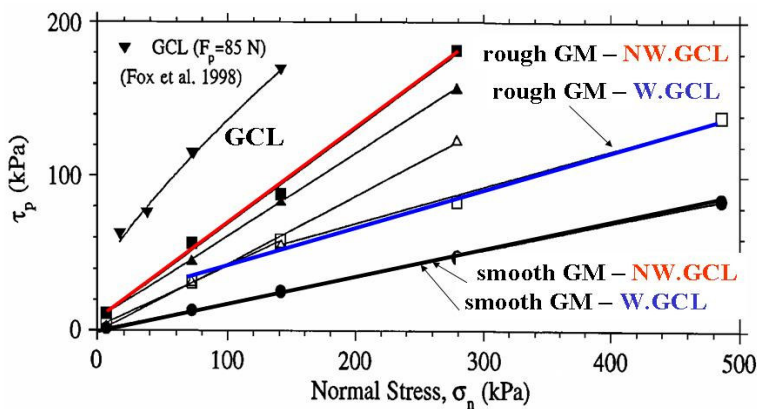


Figure 41. Influence of the type of GM and GT on the shear resistance at the interface between GM and GCL (Triplett & Fox, 2001)

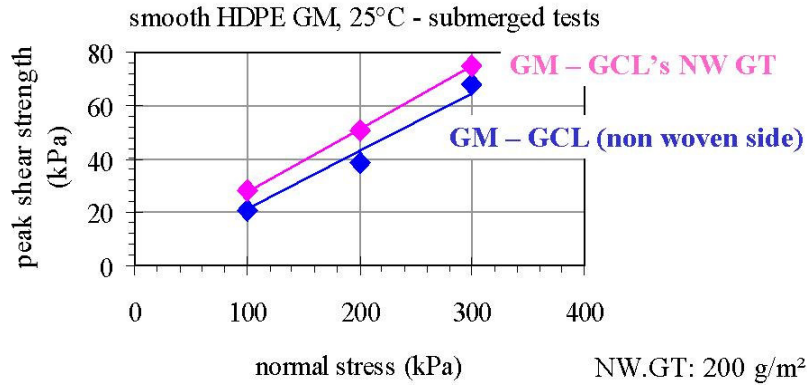


Figure 42. Influence of the bentonite at the interface between geomembrane and GCL. NW.GT: 200 g/m²

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 τ_{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.

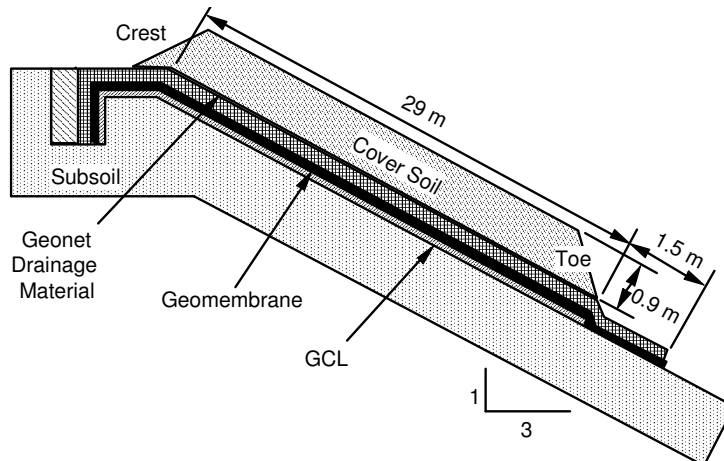


Figure 43. Test section used to assess stability in field study conducted by Daniel et al. (1998).

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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 ≤ 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 coarser 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).

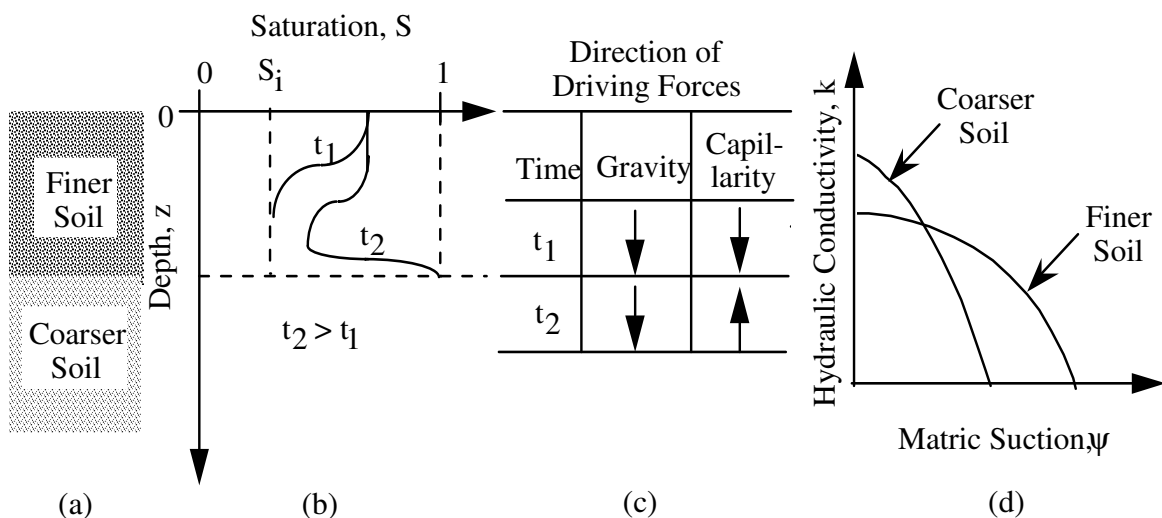


Figure 1. Conceptual layering in a capillary barrier cover: (a) cross-section; (b) wetting fronts; (c) driving forces; and (d) unsaturated hydraulic conductivity relationships (Shackelford and Nelson, 1996).

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

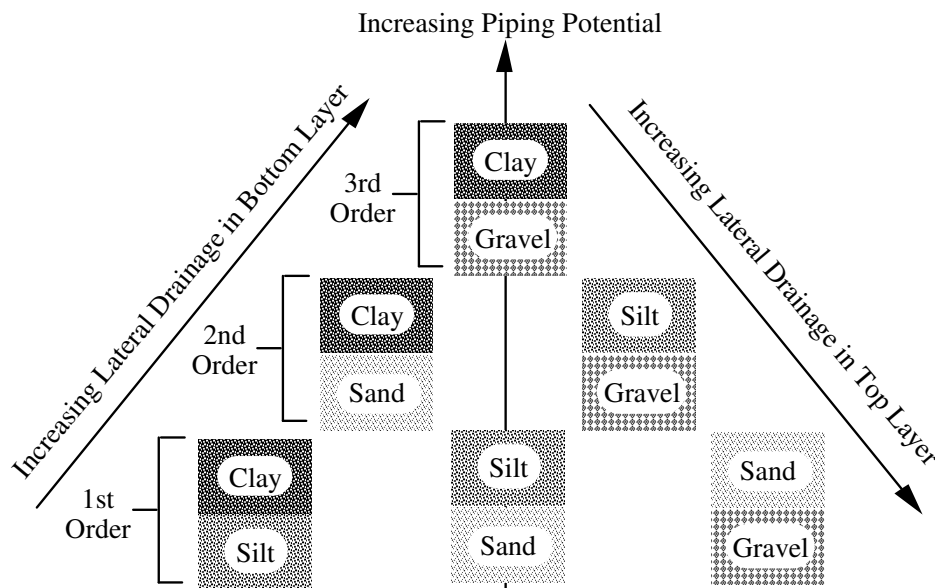


Figure 2. The capillary barrier pyramid (Shackelford and Nelson, 1996)

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.

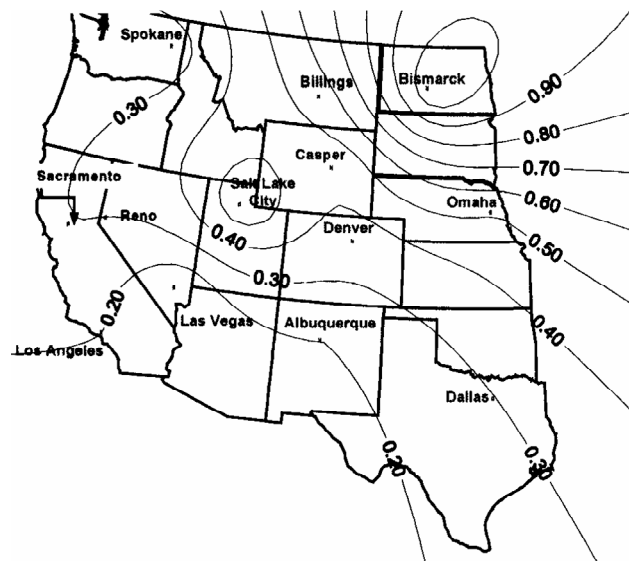


Figure 3. Contour map of the western US showing required thickness (m) of silt loam monolithic cover required to meet a target percolation rate of 10 mm/yr (adapted from Winkler, 1999).

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 1×10^{-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):

$$\epsilon_w = \frac{2T}{S_e L} \sin^{-1} \left[\frac{S_e L}{2T} \right] - 1 \quad (1)$$

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).

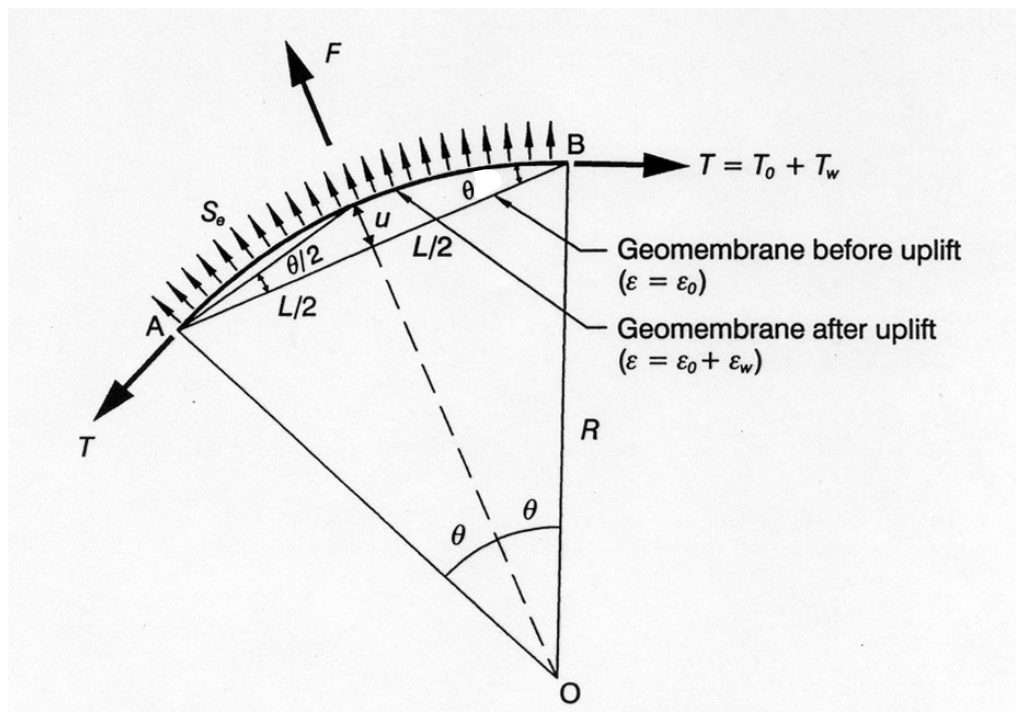


Figure 4. Schematic representation of an uplifted geomembrane (Zornberg and Giroud, 1997).

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.

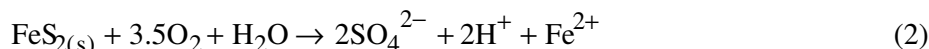
3.2.3 Geochemical Covers

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):



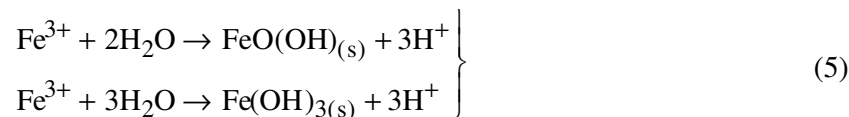
The result of this chemical reaction is the production of a low pH solution (e.g., $pH < 6$) containing relatively high concentrations of potentially toxic heavy metals associated with the tailings, such as Fe^{2+} . In the presence of sulfate (SO_4^{2-}) resulting from dissolution of pyritic tailings (Eq. 1), calcite ($CaCO_{3(s)}$) dissolves resulting in gypsum ($CaSO_4 \cdot 2H_2O_{(s)}$) precipitation as follows:



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):



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):



Thus, the existence of $\text{CaCO}_{3(\text{s})}$ (Eq. 3) results in an increase in pH and the consequent precipitation of $\text{FeO}(\text{OH})_{(\text{s})}$ and/or $\text{Fe}(\text{OH})_{3(\text{s})}$.

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 ($\text{CaO}_{(\text{s})}$) and crushed limestone ($\text{CaCO}_{3(\text{s})}$) 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 (1.5×10^{-6} M HCl, pH = 5.80) were 1.4×10^{-5} m/s and 1.3×10^{-6} m/s. The measured hydraulic conductivity values for various surface amended columns ranged from 1.6×10^{-7} m/s to 5.7×10^{-7} m/s as the result of the formation of hardpan layers with hydraulic conductivity values ranging from 2.8×10^{-9} m/s and 7.8×10^{-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 materials 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 kg 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 °C to 70 °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/m³) 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/m³ and 4.5 kN/m³, 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):

$$k = 10^{(0.022OC - 8.03)} \quad (6)$$

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:

$$C_c = 0.009(w_o) \quad (7)$$

and

$$C_c = 0.027(OC) \quad (8)$$

Landva and LaRoche (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·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 dodecyltrimethylammonium (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 perchloroethylene) 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).

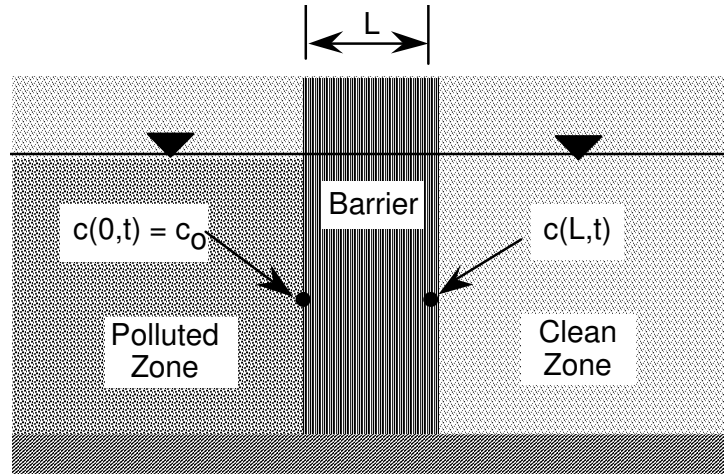


Figure 5. Schematic cross-section of passive containment barrier (Shackelford, 1999)

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 ≤ 10 percent of a constant source concentration, c_0 , assuming a representative value for the effective diffusion coefficient of the contaminant, D^* , as $5 \times 10^{-10} \text{ m}^2/\text{s}$ (see Shackelford, 1999).

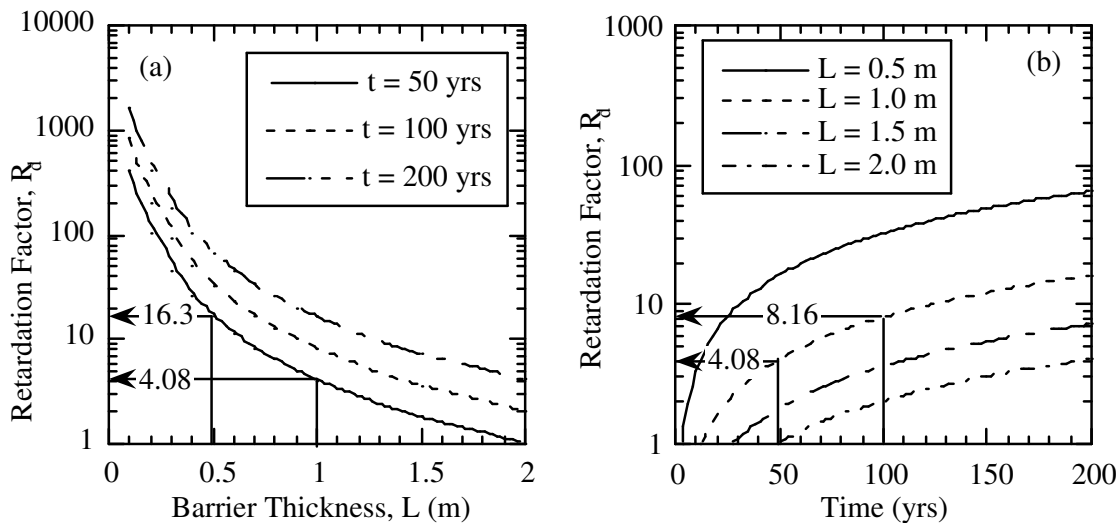


Figure 6. Retardation factor (a) as a function of thickness for a given containment time, and (b) as a function of containment time for a given thickness for the case of pure diffusion and a concentration at $x = L$ equal to 10 % of that relative to that at $x = 0$ (i.e., $C_L/C_0 = 0.1$) (Shackelford, 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 2.5×10^{-3} to 2×10^{-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 $\sim 10^{-7}$ and 10^{-8} m/s to $\sim 10^{-10}$ m/s after treatment with the biofilm-producing bacterium *Beijerinckia indicica*. In addition, they show that the hydraulic conductivity of $\sim 10^{-10}$ 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.

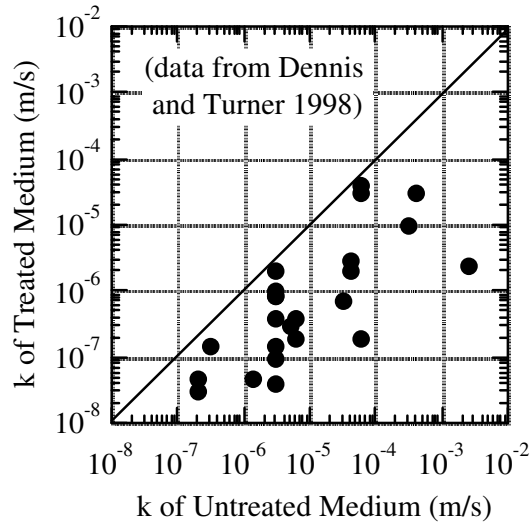


Figure 7. Hydraulic conductivity (k) of porous media before and after treatment with bacteria (Shackelford, 1999; Shackelford and Jefferis, 2000).

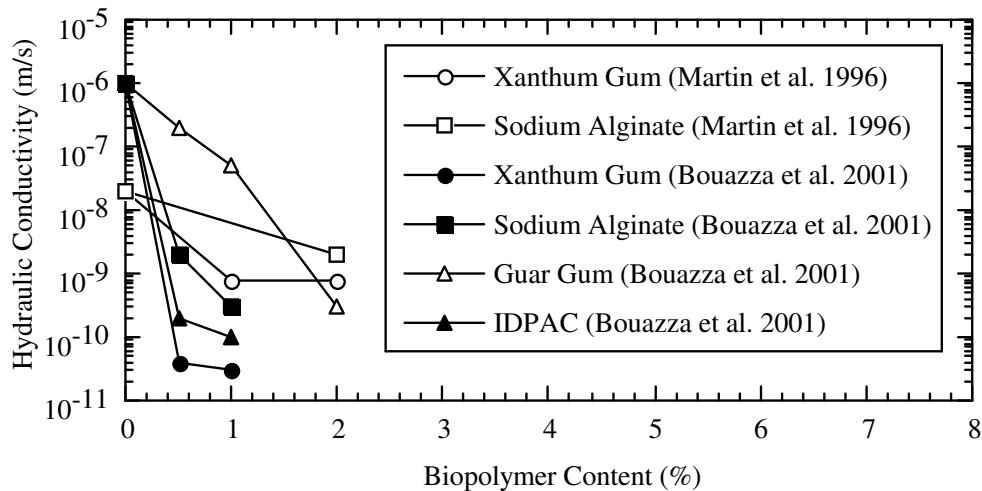


Figure 8. Hydraulic conductivity of soil biopolymer mixtures (Bouazza et al., 2001)

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)		Material added
	Initial	Final	
Berea sandstone	1.3×10^{-6}	5×10^{-8}	<i>Bacillus subtilis</i> (dead)
Sandstone cores	3×10^{-6}	9×10^{-7}	<i>Klebsiella pneumoniae</i> UMB
Sandstone cores	3×10^{-6}	4×10^{-8}	<i>Klebsiella pneumoniae</i> UMB (resuscitated)
Clayey silt	5×10^{-8}	8×10^{-10}	<i>Xanthan Gum</i> (1%) at 30% moisture content
Clayey silt	10^{-6}	1×10^{-8}	<i>Xanthan Gum</i> (1%) at 13.7% moisture content
Clayey silt	10^{-8}	5×10^{-10}	<i>Xanthan Gum</i> (1%) at 20% moisture content
Clayey sand	10^{-6}	3×10^{-10}	<i>Guar Gum</i> (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}	<i>Xanthan Gum</i> (1%)
Silty sand	10^{-7} to 10^{-8}	1×10^{-10}	<i>Beijerinckia indica</i>

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 1.3×10^{-11} to 1.2×10^{-12} m/s and field-measured conductivities ranging from 1.1×10^{-9} to 1.9×10^{-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 1.8×10^{-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 1×10^{-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 1×10^{-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 $\geq 22 \text{ kN/m}^3$. 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: 3×10^{-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 – 2×10^{-8} m/s -Lab: 8 % asphalt cement $<10^{-12}$ m/s -Field cores: 1×10^{-8} to 9×10^{-10} m/s	-Poor construction quality control
Western Processing Company, Kent Washington, 1987	-Cover for waste site -6 % asphalt cement -Hi-way paving mix	-3×10^{-3} to 1×10^{-4} m/s cores from cover	-12 to 17 % air voids -Insufficient compaction
Landfill Cover, Oregon, 1990	-Cover and roadway	-Field test: $<1 \times 10^{-9}$ 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: 1.3×10^{-11} to 1.2×10^{-12} m/s - Field SRIs: 1.1×10^{-9} to 1.9×10^{-11} m/s FAA: 1.8×10^{-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: 1.0×10^{-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	$<1 \times 10^{-9}$ 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 1×10^{-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 SiO₂-protective coat on the surface, which stops further corrosion. The resistance against alkaline solutions depends essentially on the temperature. Hot alkaline solutions attack the [SiO₄]-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. $\leq 10^{-9}$ 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).

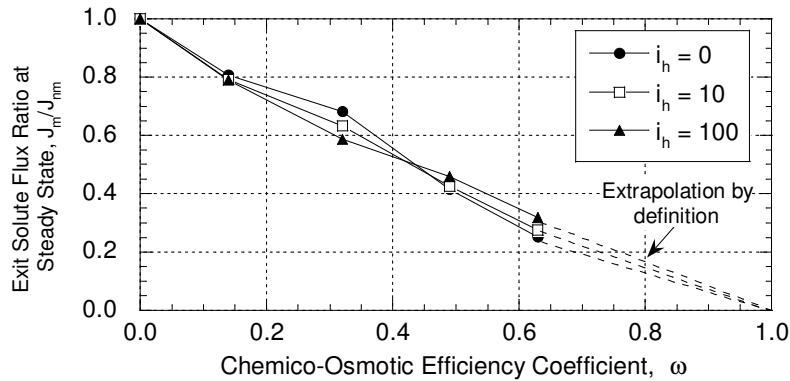


Figure 9. Effect of membrane behavior on solute mass flux at steady state through a 1-m-thick clay barrier (i_h = hydraulic gradient; J_m = membrane steady-state solute flux; J_{nm} = non-membrane steady-state solute flux) (results from Malusis 2001).

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 $k \sim 8.5 \times 10^{-10}$ 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 polymer 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.

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