WASTE CONTAINMENT: STRATEGIES & PERFORMANCE

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ABSTRACT

This paper reviews strategies for lining and covering waste containment systems, as well as systems used to remove leachate. Emphasis is placed on factors influencing field performance. Field performance data indicate that compacted clay liners and composite liners are performing as intended, and that their performance can be predicted with standard calculation methods. Resistive covers employing a soil barrier with low saturated hydraulic conductivity as the primary impedance to flow have been shown to perform poorly due to the effects of desiccation and frost action. Covers with a composite barrier layer or water balance covers have a much better performance record, and can effectively eliminate percolation into waste. The performance of leachate collections systems ranges from poor to excellent depending on the design and materials of construction. Successful leachate collection systems are constructed with coarse uniformly graded gravel with little fines and are overlain by a non-woven geotextile filter.

1 INTRODUCTION

Engineering of waste containment facilities such as solid and hazardous waste landfills, liquid waste and process water impoundments, and mine waste repositories began with the introduction of clay liners and clay caps. The overall objective was to limit discharge of toxic contaminants to groundwater. This objective has not changed over the years. However, liner and cover design has undergone great change, with new materials being introduced and new design strategies being adopted. These changes have occurred as a result of many factors, most important being the desire to further reduce the discharge of contaminants to groundwater while employing technology that is both cost-effective and practical.

This paper is a review of strategies for waste containment, with emphasis on the performance of these systems. The paper is divided into three main topics: liners, leachate collection systems, and covers. All three elements have a critical role in the proper functioning of a waste containment system. Two other issues of importance, gas control and oxygen transport, are not discussed in this paper. Readers are referred to McBean et al. (1995) for a discussion of gas control issues and Kim (2000) for a discussion of oxygen transport, the latter being pertinent for mine waste facilities.

2 LINERS

Lining systems can vary significantly in complexity (Fig. 1). The simplest lining systems consist of a compacted clay liner, geosynthetic clay liner (GCL), or geomembrane liner overlain by a granular collection layer (Fig. 1a). A more sophisticated and effective lining system incorporates a composite liner (Fig. 1b) comprised of a geomembrane placed directly on top of a clay liner (or other type of soil liner). Composite liners are now the most common liners used for municipal solid waste (MSW) landfills in North America, Europe, Australia, New Zealand and many Asian countries (Holzlohner et al. 1995, Kortegast et al. 1997, Benson 2000, Katsumi et al. 2001).

Regulations for hazardous waste landfill often require composite liners as well. Redundant lining systems incorporating one or more liners are also used (Fig. 1c), primarily for sites where risks are greater [e.g., hazardous waste landfills or corrective action management units (CAMUs) at remediation sites].

2.1 PERFORMANCE-BASED DESIGN

In most cases, regulatory considerations provide the basis for selecting the type of liner for a particular waste. This approach, which ignores performance of the lining system and the risk to the surrounding environment, can result in over-design at some sites and under-design at other sites. In the United States, for example, unlined radioactive waste facilities are still being constructed, whereas some relatively innocuous industrial wastes are being placed in facilities with double composite liners.

A better strategy, referred to as performance-based design, is to select a lining system that is consistent with the waste type, hydrogeological setting, and the intended use of groundwater resources. This strategy involves selecting a liner that discharges contaminants at a sufficiently low rate so that the risks imposed on the surrounding environment are acceptably low. Systems more complex and redundant than those required by regulations may be selected using this approach. Alternatively, a performance-based design may show that a less stringent system is necessary.

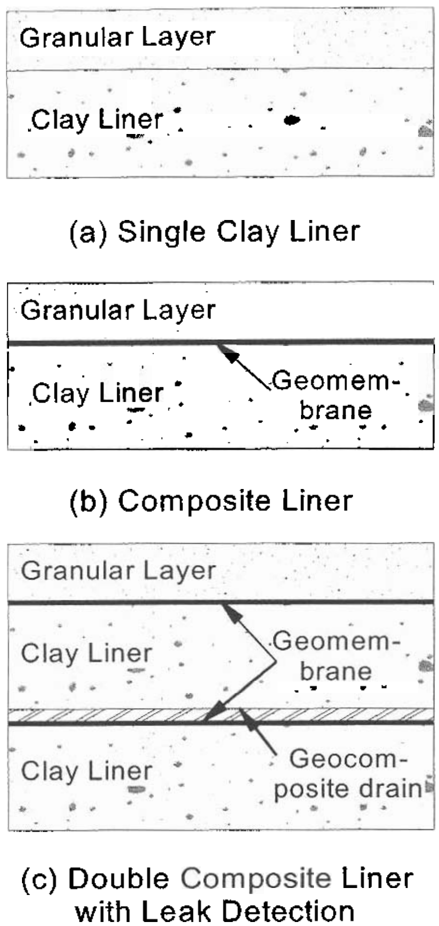

Fig. 1: Compacted clay liner with granular leachate collection layer (a), composite liner with granular leachate collection layer (b), and double composite liner with upper granular leachate collection layer and lower geo-composite leak detection layer (c).

The waste containment facility recently constructed at the Rocky Mountain Arsenal (RMA) in Commerce City, Colorado, USA provides an example of a lining system selected using a performance-based design method. Wastes from pesticide and chemical warfare agents manufactured at RMA are being disposed in an "enhanced landfill cell" that is lined with a triple composite liner. Two leak detection layers were installed, one each between adjacent composite liners. This level of redundancy is required to ensure that mass is discharged at a rate low enough to yield an acceptable risk to the surrounding environment. If the liner had been selected using a conventional regulatory approach, US regulations would only have required a double liner consisting of a granular liquid collection system, a geomembrane. a leak detection layer, and an underlying composite liner (e.g., Fig. 1c without the upper compacted clay liner).

Performance-based design requires contaminant transport analyses. Preliminary design is conducted using approximate analytical solutions to the governing differential equation for solute transport (e.g., Foose et al. 2001). Final design normally requires numerical modeling. A detailed description of this approach is beyond the scope of this paper, but is covered by Rowe (1998), Foose et al. (1999, 2002), and Katsumi et al. (2001). The remainder of this section deals with those factors that affect the performance of the lining system.

2.2 COMPACTED CLAY LINERS

2.2.1 Chemical compatibility

When clay liners were first used, little was known about how they would interact with landfill leachate. Concern regarding chemical compatibility led to extensive research on clay-waste interactions. Good reviews of this research can be found in Mitchell and Madsen (1987), Shackelford (1994), and Rowe et al. (1995). The results of this research show that aqueous inorganic solutions generally do not have a detrimental impact on the hydraulic conductivity of compacted clays unless the electrolyte concentration is very strong (on the order of 1 M or more) or the pH is very low (< 2) or very high (>13).

Similarly, organic solutes are only important when they are concentrated. For immiscible liquids (e.g., chlorinated solvents such as trichloroethylene), long-term tests with dilute solutions (aqueous solutions at concentrations below the solubility limit) have shown that there is essentially no impact on the hydraulic conductivity of clay (e.g., Kim et al. 2001). Similarly, miscible organic compounds only impact hydraulic conductivity when they constitute at least 50% of the solution (Bowders and Daniel 1987, Fernandez and Quigley 1988). However, if immiscible liquids are present as a separate phase or miscible organic compounds are in abundance, very high hydraulic conductivities can be achieved (Anderson et al. 1982, Fernandez and Quigley 1985, 1988, Foreman and Daniel 1986, Bowders and Daniel 1987). An example from Fernandez and Quigley (1988) is shown in Fig. 2 for Sarnia clay permeated with aqueous mixtures having different proportions of ethanol (dielectric constant = 32) and leachate from a MSW landfill. The hydraulic conductivity of the clay remains very low (6×10^{-9} cm/s) until the solution contains at least 60% ethanol. As the proportion of ethanol is increased beyond 60%, the hydraulic conductivity rises dramatically, reaching as high as 6×10^{-7} cm/s.

The key outcome of the research effort on clay-waste interactions is that chemical compatibility generally is not an issue for facilities containing municipal solid waste, non-hazardous industrial solid wastes, and stabilized hazardous solid wastes provided that these facilities do not accept liquid wastes. Excluding liquid wastes precludes the presence of concentrated miscible organic liquids or immiscible organic liquids in a separate liquid phase. Clay liners in facilities that accept liquid wastes or those containing strong industrial wastes (e.g., some petroleum or mining wastes) may be prone to compatibility problems. In cases where compatibility may be an issue, compatibility tests should be conducted. Unfortunately, no standard test method for compatibility testing exists, although good guidance on testing

methods can be found in Shackelford (1994) and Daniel (1994). For cases where incompatibilities exist, stabilization techniques can be employed as suggested by Broderick and Daniel (1990).

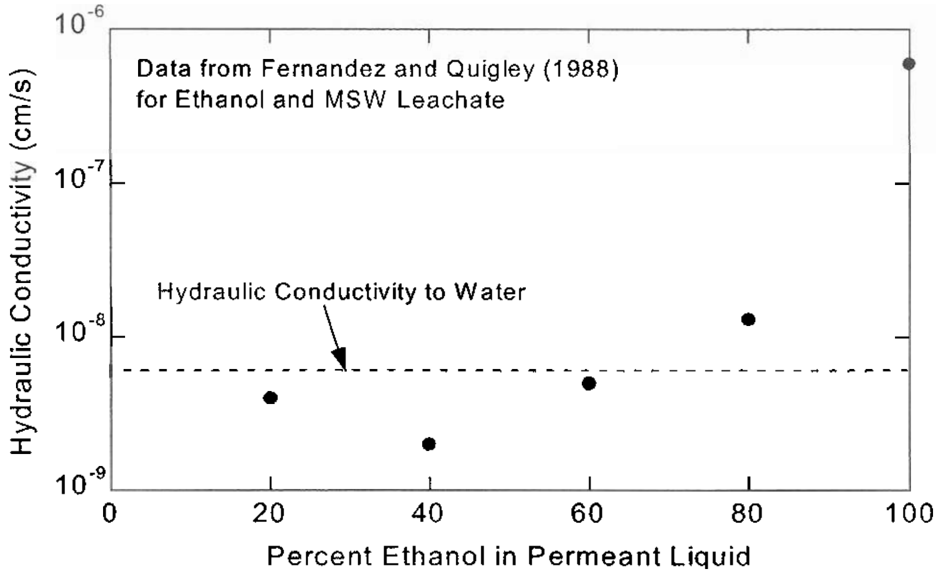

Fig. 2: Hydraulic conductivity of Sarnia clay permeated with solutions comprised of varying amounts of landfill leachate and ethanol (adapted from Fernandez and Quigley 1988).

The general absence of compatibility problems is evident in the good field performance exhibited by clay liners. The only failures are isolated incidents related to strong industrial wastes, and these have not been reported in the literature. An example characteristic of good performance is the Keele Valley Landfill in Ontario, Canada, which is a clay-lined landfill receiving municipal solid waste. Large lysimeters under the landfill have continuously shown that the clay liners at Keele Valley have retained their low hydraulic conductivity since 1984 (Lahti et al. 1987, Reades et al. 1990). Similarly, clay liners have been required in the State of Wisconsin, USA since 1978. Each landfill cell is required to have at least one large lysimeter for monitoring the quantity and quality of liquid discharged from the liner (Gordon et al. 1989, 1990). Currently 23 landfills in Wisconsin contain lysimeters, and none of the lysimeters have shown excessive leakage rates indicative of chemical incompatibility. Hydraulic conductivities computed from the lysimeter data indicate that clay liners in Wisconsin typically have hydraulic conductivities between 10^{-8} and 10^{-7} cm/s.

2.2.2 Field hydraulic conductivity

A more significant issue with clay liners is how to successfully build a clay liner that has the desired low hydraulic conductivity at field scale. Daniel (1984) and Day and Daniel (1985) indicated that the hydraulic conductivity of compacted clay liners operative at field scale could be orders of magnitude higher than the hydraulic conductivity measured on small specimens collected during construction for quality control testing in the laboratory. These studies indicated that macropores, which control flow at field scale, are not adequately represented in small specimens collected in sampling tubes, and suggested that clay liners may be much more permeable than believed. The reports by Daniel (1984) and Day and Daniel (1985) were very controversial (11 discussions of Day and Daniel were published in the July 1987 issue of the ASCE *Journal of Geotechnical Engineering*), and remain controversial today. Nevertheless, their publication led to great scrutiny regarding the operative field hydraulic conductivity of clay liners and to elucidation of the key factors controlling field hydraulic conductivity.

Research has shown that the key factor influencing the field hydraulic conductivity of clay liners is the compaction condition relative to the line of optimums (Reades et al. 1990, Benson and Boutwell 1992, Benson et al. 1999, Benson and Boutwell 2000). Clay liners compacted wet of the line of optimums typically are comprised of microscale pores and are devoid of interconnected macropores (Fig. 3). As a result, the field hydraulic conductivity (K_F) of clay liners compacted wet of the line of optimums is typically low and comparable to the hydraulic conductivity measured on small laboratory-scale (diameter = 76 mm) specimens (K_L). In contrast, clay liners compacted dry of the line of optimums typically contain numerous macropores that are not captured in small laboratory-scale specimens (Benson and Daniel 1990, Shackelford and Javed 1991). As a result, clay liners compacted dry of the line of optimums generally have high field hydraulic conductivity, which is not reflected in the small specimens used for laboratory testing.

Benson and Boutwell (1992, 2000) analyzed data from 51 sites throughout North America and determined that the ratio K_F/K_L could be related to the percentage of the compaction data (i.e., measurements of water content and dry unit weight made during construction) falling wet of the line of optimums (P_o). This relationship is shown in Fig. 4. For $P_o > 85\%$, $K_F \sim K_L$.

A similar analysis on data from 85 sites by Benson et al. (1999) showed that requiring a high P_o is likely to yield low K_F. In particular, Benson et al. (1999) found no sites with $K_F > 10^{-7}$ cm/s, the common permissible maximum hydraulic conductivity, when P_o was greater than 90%.

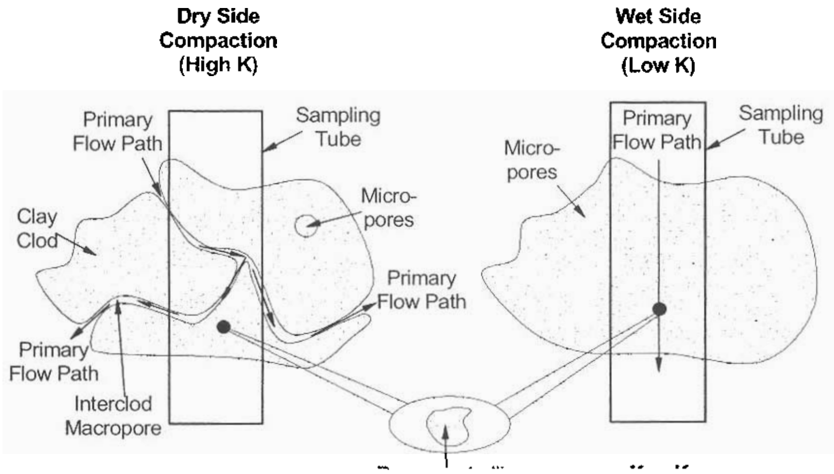

Fig. 3. Schematic diagram showing size of pores conducting flow relative to the size of a typical thin-wall sampling tube (adapted from Benson and Boutwell 2000).

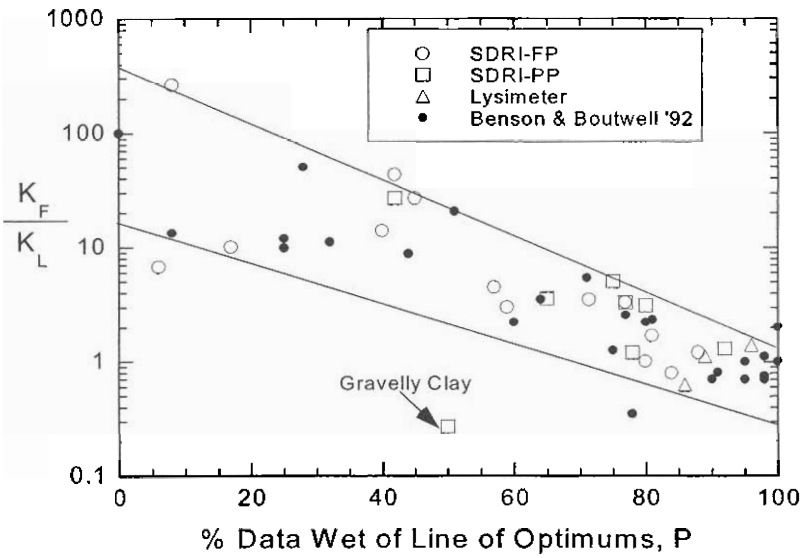

Fig. 4. Ratio K_F/K_L vs. percentage of compaction data falling wet of the line of optimums (P_o). Graph adapted from Benson and Boutwell (2000).

The importance of compaction wet of the line of optimums has resulted in new strategies for preparing compaction specifications for clay liners. Modern specifications are based on the results of a series of hydraulic conductivity tests conducted on specimens prepared at different combinations of water content and dry unit weight (e.g., following the methods described in Daniel and Benson 1990 and Benson 1994). The range of water contents and dry unit weights that yields suitably low hydraulic conductivity is identified as an acceptable zone for compaction control, as illustrated by the shaded zone in Fig. 5. This zone typically corresponds to a band wet of the line of optimums, and is truncated based on other design considerations such as required interface shear strength, ductility, or trafficability.

2.3 COMPOSITE LINERS

Interest in reducing leakage rates below those achieved with clay liners has resulted in the use of composite liners consisting of a compacted clay liner overlain by a geomembrane. In some cases, a geosynthetic clay liner (GCL) is used in lieu of the compacted clay liner (see subsequent discussion). Geomembranes are infrequently used alone because they inevitably contain defects, and these defects can result in large leakage rates (Giroud and Bonaparte 1989, Katsumi et al. 2001).

Due to its high resistance to chemical deterioration and relatively low cost, most geomembranes are manufactured from high-density polyethylene (HDPE), although linear low-density polyethylene (LLDPE), polypropylene (PP), or polyvinyl chloride (PVC) geomembranes are used when chemical compatibility is not considered problematic or if other design issues require a geomembrane with different mechanical properties than those associated with HDPE.

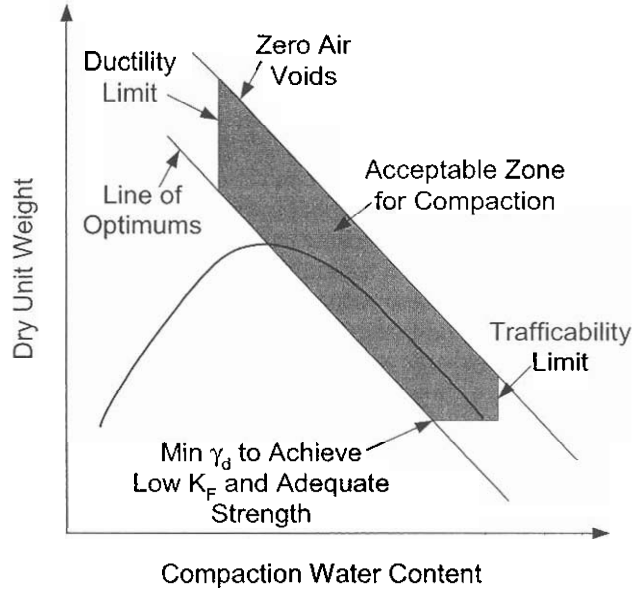

Fig. 5. Modern compaction specification that ensures compaction wet of the line of optimums (adapted from Benson et al. 1999a).

2.3.1 LEAKAGE RATE PREDICTIONS

An illustration of the lower leakage rates associated with composite liners is shown in Fig. 6. Leakage rates for composite liners were computed with Giroud's empirical equation (Giroud 1997):

$$Q = \xi\left[1 + 0.1\left(\frac{D_l}{L}\right)^{0.95}\right] a^{0.1} D_l^{0.9} K^{0.74} \qquad (1)$$

where Q is the leakage rate per hole (m^3/s), D_l is the depth of leachate on the liner (m), K is the saturated hydraulic conductivity of the soil component of the composite liner (m/s), a is the area of the defect in the geomembrane (assumed to be 10^{-4} m^2 based on recommendations in Giroud and Bonaparte 1989), L is the thickness of the composite liner (assumed to be 0.9 m), and □ is the interface contact factor (set at 0.21 for good contact per Giroud 1997). Leakage rates for clay liners were computed with Darcy's Law using a unit hydraulic gradient. For the composite liners, the geomembrane was assumed to contain 2 or 12 holes/ha, corresponding to very good or modest construction quality control (Giroud and Bonaparte 1989). Comparison of the leakage rates indicates that introduction of a geomembrane reduces the leakage rate by an order of magnitude, or more. This reduction in leakage essentially eliminates advective transport, rendering diffusion the only significant transport mechanism (Foose et al. 1999, 2002).

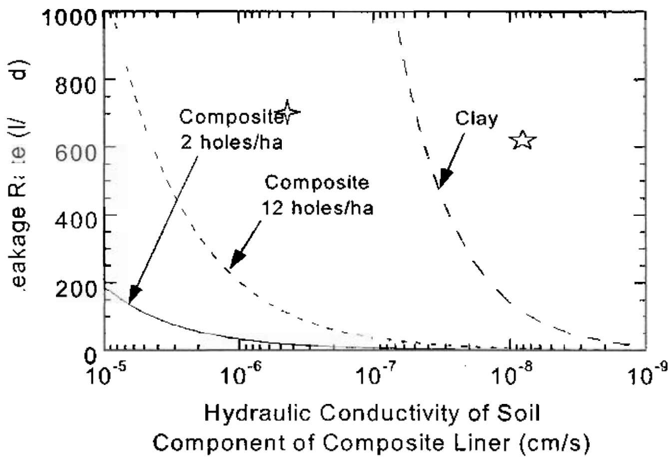

Fig. 6. Leakage rates for composite and compacted clay liners. GM = geomembrane.

A recent study sponsored by the United States Environmental Protection Agency (USEPA) reviewed the performance of the upper composite liner at sites containing double liner systems with two composite liners (Bonaparte et al. 2001). Data from 172 landfill cells were reviewed. The average leakage rates reported by Bonaparte et al. (2001) are shown in Fig. 6 for composite liners constructed with clay liners (cross) and GCLs (star). Good agreement exists between the average leakage rates recorded in the field and those predicted using Giroud's equation, which suggests that the calculation methods currently used in practice yield reliable predictions. A similar conclusion was drawn by Foose et al. (2001) in a theoretical study of methods to predict leakage rates.

2.3.2 INTERFACE STABILITY

Slope stability continues to be a major issue for containment facilities where composite liners are used, even more than a decade after the massive slide that occurred during filling of the Kettleman Hills hazardous waste landfill in southern California, USA (Mitchell et al. 1990, Seed et al. 1990, Stark and Poeppel 1994). This is particularly true in the MSW industry, where strategies to limit annual capital expenditures are now common. Deeper cells with a smaller footprint are being constructed to reduce the lined area per volume of waste, and smooth geomembranes are being used in lieu of textured geomembranes to reduce material costs.

The negative implications of these strategies become apparent in the context of a massive slide that occurred at a municipal solid waste landfill in the upper midwestern US in April 2000. Plans for the new cell called for construction in a series of phases, with a new phase being constructed annually. The first phase consisted of a 400 m long and 20 m deep excavation in glacial clay till (Fig. 7), with the base grade being approximately 17 m below the ground water table. The base of this phase was to be 80 m wide, and the side slope was to be at 3:1 (length of side slope = 60 m). To reduce construction costs, however, the base was constructed half as wide (40 m) as originally planned. No other changes to the geometry were made. A composite liner system was constructed consisting of a compacted clay liner 1.2-m thick overlain by a 1.5-mm-thick smooth HDPE geomembrane, a non-woven geotextile cushion (550 g/m^2), and a 600 mm layer of leachate collection stone (nominal diameter = 50 mm). Smooth geomembrane was used in place of the textured geomembrane proposed in the design to reduce material costs.

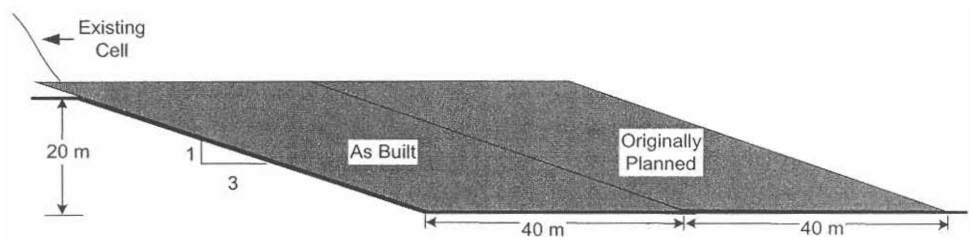

Fig. 7: Cross-section of initial phase of cell where slide occurred.

Filling proceeded in lifts 3 m thick and continued for 6.5 mos, at which time the surface of the waste was approximately 3 m above the surrounding ground surface. At this point, a crack formed in the waste at the top of the slope near the center of the cell (Fig. 8). Within minutes, the crack propagated along the extent of the cell. This crack was followed immediately by translation of 142,00 Mg of waste, that left a 15-m wide crevice along the extent of the cell. No injuries occurred, but the cell was closed and waste being delivered to the facility was diverted, resulting in significant loss of revenue.

A forensic investigation began two days after the slide. Samples of the clay liner, geosynthetics, and the leachate collection stone were obtained, and a suite of interface shear tests were conducted using a large-scale direct shear machine and methods described in ASTM D 5321. One condition of particular importance was the presence of pore water pressures induced by groundwater at the interface between the clay liner and geomembrane. This condition required that an effective stress analysis be conducted. However, the schedule of the forensic investigation did not permit drained direct shear testing of the clay-geomembrane interface, which preliminary testing showed would require at least 10 days per test. Consequently, undrained tests were conducted with a series of pore water pressure tips located along the length of the specimen and close to the clay-geomembrane interface.

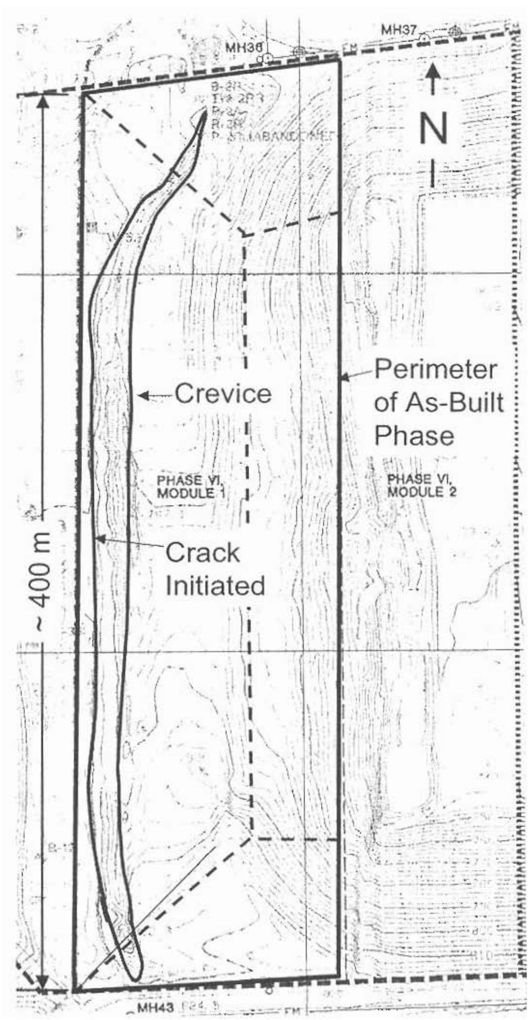

Fig. 8. Topographic map of phase where slide occurred showing as-built

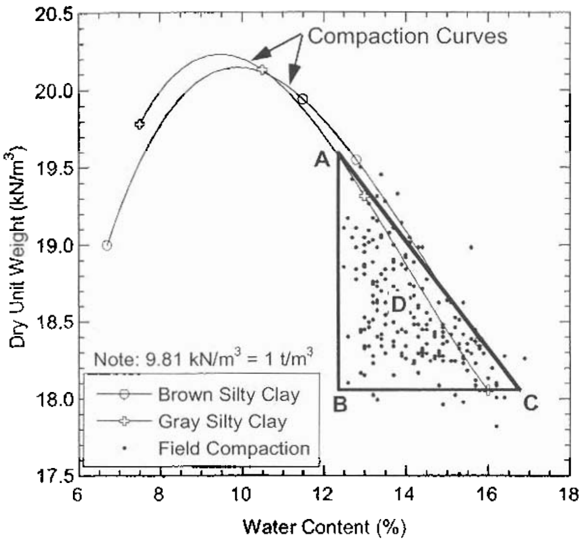

Fig. 9. Compaction data and locations of test conditions A-D for direct shear tests on clay-geomembrane interface.

Tests were conducted at four combinations of water content and dry unit weight (A, B, C, and D) (Fig. 9). Three of these combinations (A-C) bracket the range of water contents and dry unit weights measured during construction of the clay liner. The fourth combination (D) corresponds to the average water content and dry unit weight at the time of construction.

Failure envelopes from the direct shear tests are shown in Fig. 10 in terms of total and effective stresses. As anticipated, different failure envelopes were obtained for the different compaction conditions when the data were interpreted in terms of total stresses (Fig. 10a). The highest strengths were obtained at Point A (lowest water content and highest dry unit weight) and the lowest strengths at Point C (highest water content and lowest dry unit weight). Intermediate strengths were obtained for the average condition.

Failure envelopes interpreted in terms of effective stresses are shown in Fig. 10b for peak and large-displacement (60 mm) conditions. When interpreted in terms of effective stresses, the data converge to form two envelopes corresponding to peak and large-displacement conditions. The effective stress envelopes have no cohesion and friction angles for peak and large-displacement conditions of 15° and 12°.

The slide was analyzed in two dimensions because the breadth of the slide area was large compared to the ends (Fig. 8). Spencer's method (as implemented in WinSTABL, www.uwgeosoft.org) and a two-block analytical solution were employed using effective stresses, no cohesion, and the aforementioned interface friction angles. Four cross-sections along the length of the slide were considered. Both Spencer's method and the analytical solution yielded essentially the same results. The factor of safety ranged from 1.10 – 1.21 (peak) or 0.87 – 0.96 (large-displacement). Veneer analyses conducted with the peak and large-displacement friction angles also showed that the geosynthetics should have failed in tension, which was later confirmed in test pits excavated along the top of the slope.

Additional analyses were conducted using the large-displacement interface friction angles to assess stability of the original design, i.e., an 80 m wide base and textured geomembrane. If a wider (80 m) base was constructed using smooth geomembrane, the factor of safety would have been 1.25 based on large-displacement strengths. If textured geomembrane had been used with the narrow (40 m) base, the factor of safety would have been 1.8. A factor of safety of 3.1 would have existed had a wider base and textured geomembrane been used. That is, failure probably would not have occurred had any of these other conditions been realized.

There are two key lessons to be learned from this failure. First, careful testing and analysis before construction would have shown the deficiencies in the as-built design prior to construction, and would have permitted the analysis of alternative scenarios where some cost savings may have been accrued and stability could have been ensured. Unfortunately, no interface shear testing or additional analysis was conducted, because the construction engineer argued, "shear testing of the various base liner components would serve no useful purpose." Second, strategies to accrue cost savings must be analyzed carefully to ensure that the construction savings being accrued through design changes will not have unforseen negative impacts. Indeed, the savings accrued at this site during construction (≈US \$10/m²) was very small compared to the cost of remediating the failure (≈US \$250/m²) one year later.

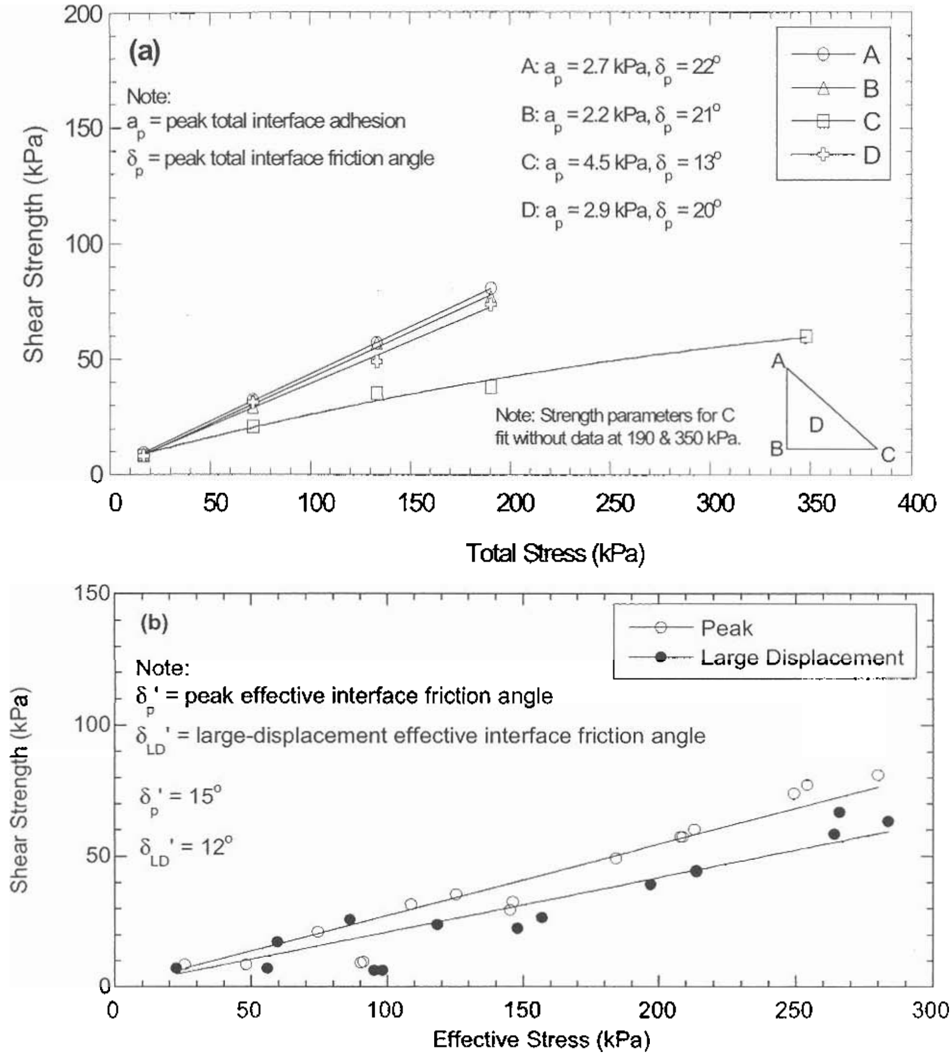

Fig. 10: Total (a) and effective stress (b) failure envelopes for the clay-geomembrane interface

2.4 GEOSYNTHETIC CLAY LINERS

Geosynthetic clay liners (GCLs) are thin prefabricated clay liners consisting of sodium bentonite (~7.5 kg/m^2) encased between two geotextiles or glued to a geomembrane (Fig. 11). Their low saturated hydraulic conductivity to tap water, perceived resistance to damage caused by frost and desiccation, lower installation cost, and rapid deployment make GCLs appear as an attractive alternative to clay liners. In fact, the data reported in USEPA's review of operating facilities (Fig. 6) suggests that leakage rates associated with composite lined facilities employing GCLs are approximately ten times lower, on average, than those with compacted clay liners (Bonaparte et al. 2001).

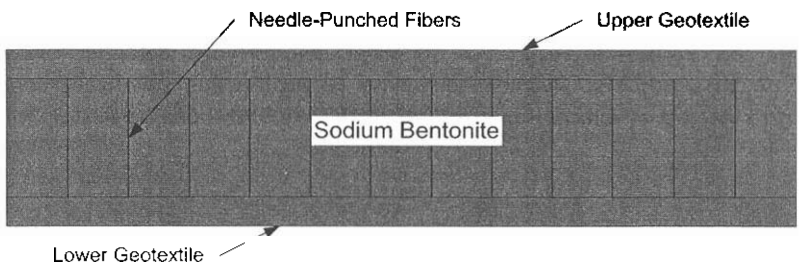

Fig. 11: Schematic of a common GCL with two carrier geotextiles that are needle-punched together to retain the bentonite and provide greater internal shear strength.

While GCLs appear attractive, they are not a panacea and must be used only where engineering analyses indicate they are appropriate. In liner applications, there are three issues of particular importance: puncture potential, bentonite thinning, and long-term chemical incompatibility. Stability is also an issue, but is overcome using reinforced GCLs (Daniel et al. 1998, Stark et al. 1998).

2.4.1 *Puncture and bentonite thinning*

Problems associated with puncture and bentonite thinning are of concern because GCLs are thin (≈10 mm), and thus are prone to a catastrophic breach. Puncture potential can be avoided by careful attention to construction and filling activities. To minimize the likelihood for puncture, subgrades need to be properly cleared of debris and surface irregularities prior to deployment of the GCL. The first layer of waste should also be limited to particulate media, paper, and small debris, and should not contain large linear objects such as bars, rods, pipes, or wire. Cushion layers can also be installed to protect the GCL.

Bentonite thinning is more problematic and arises from two different mechanisms: localized stress concentrations (e.g., as might be caused by gravel particles adjacent to a GCL) and broad gradients in stress that cause gradual migration of the soft hydrated bentonite to regions of lower stress (Stark 1998, Fox et al. 2000). Like puncture, localized stress concentrations can be dealt with using earthen or geosynthetic cushion layers (Fox et al. 1998) and by ensuring that appurtenances (e.g., pipes, etc.) do not apply concentrated stresses on the liner. Migration of bentonite caused by stress gradients is more difficult to overcome, and has received little attention in the literature (Stark 1998). Most reports of bentonite migration are anecdotal, and arise from field observations made when GCLs are excavated during activities associated with lateral expansions. For example, at one solid waste landfill in the midwestern US, portions of an in-service GCL were found to be practically devoid of bentonite when it was excavated during an expansion.

2.4.2 *Long-term chemical compatibility*

Long-term chemical compatibility of GCLs is another very important issue that has received inadequate attention in the literature (Shackelford et al. 2000). GCLs have low hydraulic conductivity to tap water because they contain sodium bentonite. The presence of sodium in the exchange complex permits osmotic swelling of the bentonite, which drastically reduces the size of the pores and the volume of the pore space that is actively involved in flow. However, if sodium is exchanged for cations with higher valence (e.g., Ca^{2+} or Mg^{2+}, which are common in leachates), osmotic swelling does not occur and the bentonite becomes orders of magnitude more permeable (Shackelford et al. 2000, Jo et al. 2001). For example, hydraulic conductivities reported by Jo et al. (2001) for a GCL permeated with different single species salt solutions are shown in Fig. 12. The hydraulic conductivity of the bentonite to deionized (DI) water is approximately 2×10^{-9} cm/s, whereas the hydraulic conductivity to 0.1 M solutions with divalent or trivalent cations is approximately 10^{-5} to 10^{-4} cm/s.

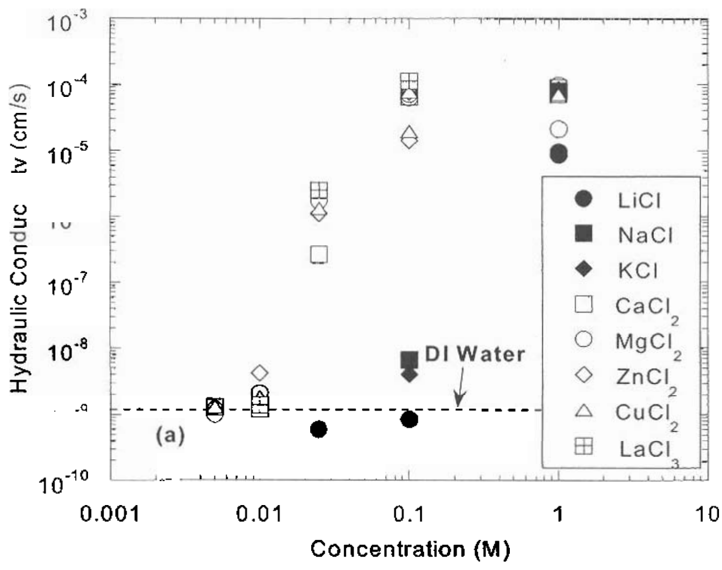

Fig. 12. Hydraulic conductivity of a GCL to single species salt solutions having different cation valence and concentration (adapted from Jo et al. 2001).

At low concentrations, the exchange process occurs very slowly due to mass transfer limitations that occur between the bulk pore water and the interlayer water between the montmorillonite layers. Even in accelerated laboratory tests, complete exchange may require permeation for a year or more. For example, the data in Fig. 13 are from on-going long-term tests being conducted on a GCL using dilute $CaCl_2$ solutions as part of a collaborative project between the University of Wisconsin-Madison and Colorado State University. Even after nearly a year of permeation, equilibrium has not been established and the hydraulic conductivity continues to increase gradually. Much longer times, perhaps on the order of decades, will be required for equilibrium to occur in the field. Thus, the low leakage rates reported in USEPA's field study of composite liners containing GCLs (Bonaparte et al. 2001) may not be indicative of long-term behavior, because most of the composite liners with GCLs in the field study were in service for less than 5 years.

Kolstad (2000) studied the effects of long-term permeation with liquids containing multiple inorganic species to assess how actual leachates may affect the hydraulic conductivity of GCLs in the long term. Composition of the solutions was characterized by ionic strength and the "ratio of monovalent to divalent cations," or RMD, which is defined as:

$$\mathrm{RMD} = \left[\frac{N_M}{N_D^{1/2}}\right]$$

where N_M is the total normality of monovalent cations in the permeant solution and N_D is the total normality of divalent cations in the permeant solution. RMD is similar to the sodium adsorption ratio (SAR). However, RMD refers to the composition of the permeant liquid, whereas SAR refers to the composition of the pore water in equilibrium with the soil solids.

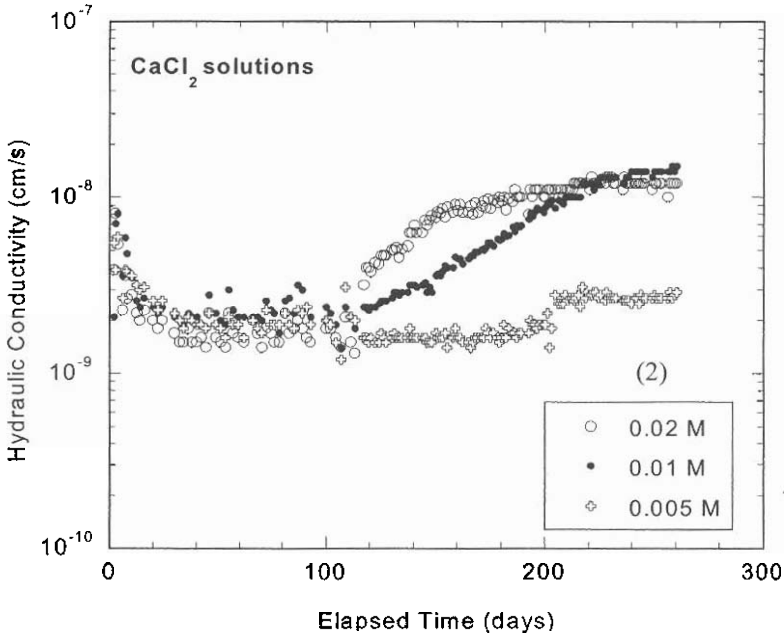

Fig. 13. Hydraulic conductivity of a GCL to dilute $CaCl_2$ solutions during on-going long-term chemical compatibility tests.

Kolstad estimated the long-term hydraulic conductivity of in-service GCLs using results of his chemical compatibility tests and the composition of a variety of leachates reported in the literature. Five different types of facilities were considered: municipal solid waste (MSW) landfills, US hazardous waste landfills, US construction and demolition (C&D) waste landfills, mine waste facilities, fly ash disposal facilities, and paper sludge landfills.

The long-term hydraulic conductivities that Kolstad estimated are shown in Fig. 14, which is a graph of RMD vs. ionic strength for the various leachates Kolstad compiled from the literature. The graph is overlain with contours of hydraulic conductivity obtained from his long-term multi-species hydraulic conductivity tests. Hydraulic conductivities are estimated for a particular leachate by interpolating between the contours.

Based on the data shown in Fig. 14, hydraulic conductivities less than 10^{-8} cm/s may be realized, even under long-term conditions. However, in contrast to current thinking, GCLs used for sites with stronger leachates (e.g., ionic strength > 0.1 M) may become much more permeable after extended service. This possibility illustrates the need for a long-term chemical compatibility assessment whenever GCLs are to be used in lining systems.

2.5 ISSUES FOR DESIGNERS

Effective lining systems can be constructed with a variety of materials and layering arrangements. When traditional materials of construction are used, chemical compatibility is not normally an issue, unless unusual wastes are being contained that produce strong (ionic strength > 0.01 M for GCLs, > 1 M for compacted clays), very acidic (pH < 2), or very basic (pH > 12) leachates. Wastes containing concentrated organic liquids may also be problematic. In such circumstances, long-term compatibility testing should be

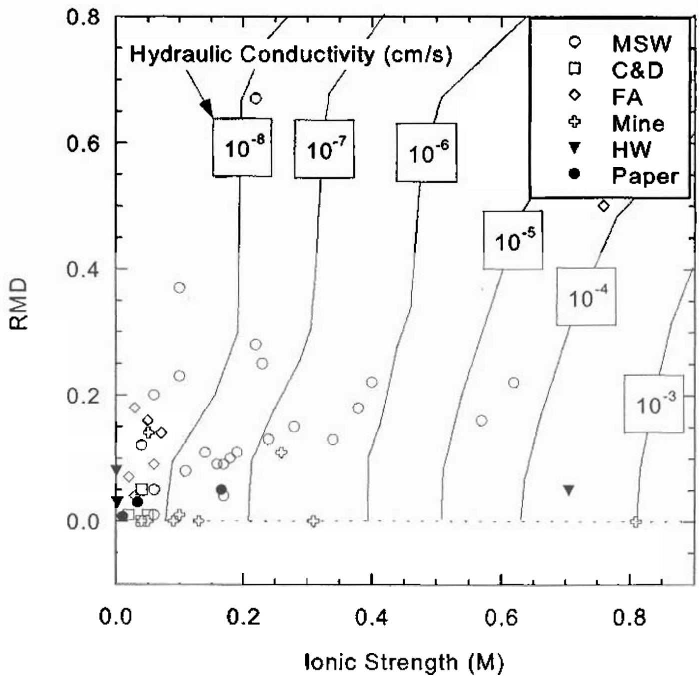

Fig. 14. Anticipated hydraulic conductivities of GCLs after long-term permeation with various leachates types (adapted from Kolstad 2000).

conducted to ensure that the materials being selected are acceptable

The appropriate configuration of the liner should be determined using a performance-based approach. Leakage rates used in a performance-based analysis can be reliably computed using conventional methods (e.g., Darcy's law and Giroud's equation). Construction specifications (e.g., compaction criteria) should also be developed that ensure that the field-scale performance of a facility is consistent with the performance anticipated based on design computations made using laboratory test data as input.

Ensuring stability of the facility is equally important as selecting a liner configuration that meets transport criteria. Reliable stability analyses can be made using conventional analytical methods and slope stability programs. Interface strengths corresponding to large-displacement conditions measured in large-scale direct shear machines appear to provide reliable estimates of the mobilized shear strength at field scale, and can be used with confidence for stability calculations.

As with other engineered facilities, design changes made during construction must be analyzed to ensure that the performance of the modified facility is acceptable and that the facility is stable. When re-analyses are not made, catastrophic failures can occur, resulting in large unanticipated costs.

3 LEACHATE COLLECTION SYSTEMS

The purpose of a leachate collection system (LCS) is to remove leachate collecting on top of the liner so that the head on the liner is maintained at an acceptably low level. A secondary purpose is to collect leachate for recirculation, i.e. as in a bioreactor landfill. Hydraulic design of leachate collection systems is reasonably straightforward. Numerous publications describe the methods for selecting the drainage medium, pipe spacing, pipe size, and pipe perforations needed to achieve a desired maximum leachate depth. Giroud et al. (2001) provide an excellent discussion of how to compute the depth of leachate for a given LCS design. A good discussion of pipe deflections is in Paruvakat (1993). Oweis and Khera (1998) and McBean et al. (1995) describe how to select pipe sizes to ensure adequate flow capacity.

A more important issue that has received less attention is the potential for clogging of the LCS, and how clogging is affected by the materials used in construction. Field exhumations conducted in Europe, Canada, and the United States have shown that complete failure of leachate collection systems can occur under certain circumstances (Brune et al. 1994, Koerner et al. 1994, Fleming et al. 1999). Clogging of LCSs is generally caused by accumulation of both inorganic and organic mass in the pore space. Although the exact geochemical processes responsible for deposition of the clogging material are not well understood, the general framework is believed to be bacterially facilitated. Bacteria present in the leachate colonize the granular medium, providing the foundation for accumulation of clogging materials. As leachate flows through the pores, the leachate and the bacteria present on the surface of the particles interact, and organic compounds in the leachate decompose. Byproducts of the decomposition process alter the geochemistry of the leachate, resulting in precipitation of calcite, calcium carbonate, and iron sulfide (Brune et al. 1994, Rowe et al. 2000). Inorganic solids normally precipitate on the surface of the bacteria, forming a conglomerate of organic and inorganic clogging material referred to as "incrustation" (Brune et al. 1994).

Formation of clogging agents appears inevitable unless major changes are made in the landfilling process, most of which are not economically feasible. Accordingly, the designer's challenge is to design a LCS that will not clog throughout the design life of the facility, even though clogging materials will accumulate in the LCS. Field and laboratory studies indicate that there are four primary factors that should be considered when designing a LCS: (i) particle size distribution of the granular collection medium, (ii) the filter above the granular medium, (iii) rate of mass loading, and (iv) maintenance (Koerner and Koerner 1990, Brune et al. 1994, Koerner et al. 1994, Fleming et al. 1999, Rowe et al. 2000). Mineralogy of the granular medium is also believed to be an important factor affecting clogging, although scientific evidence regarding the importance of mineralogy is conflicting (e.g., see Pasky et al. 1998, Bennett et al. 2000).

3.1 PARTICLE SIZE DISTRIBUTION

For a given leachate, properties of the granular medium having the greatest influence on clogging potential are the median particle size and the distribution of particle sizes. These factors are particularly influential because they control the size of the pore space through which the leachate flows. Granular media with smaller median particle size and broader pore size distribution typically have smaller pores that are more readily clogged. This effect has been clearly shown in laboratory column studies simulating leachate collection systems conducted by Brune et al. (1994) and Pasky et al. (1998). Leachate from actual MSW landfills was used in both studies.

The influence of median particle size is evident in the data reported by Pasky et al. (1998), as shown in Fig. 15 for three columns containing three sizes of uniformly graded gravel (5-10, 10-20, and 20-40 mm). Clogging of the gravel is reported in terms of a reduction in "drainable" porosity that occurred as the columns were continuously subjected to leachate flow. Drainable porosity is defined as the volume of void space per unit volume of LCS stone that will drain under unit gradient conditions during a relatively short (< 1 hr) period of time (Rowe et al. 2000).

A reduction in drainable porosity occurs in all three columns throughout the duration of the experiment (approximately two years). The magnitude of the reduction in drainable porosity strongly depends on particle size. The reduction was negligible (<2%) for the coarsest gravel (20-40 mm), and dramatic (~12%) for the finer gravel. Also, by the end of the study, the drainable porosity stabilized in the coarsest gravel, but continued to decrease at an appreciable rate (~0.2% per month) in the finer gravel. Similar findings are reported in Brune et al. (1994), who state "at the end of the experiment, the permeability of the coarse drainage material, especially gravel of grain size 16-32 mm, was hardly affected, while that of finer drainage material, e.g., gravel of grain size 8-16 mm, showed a significant reduction in pore volume."

The appreciable degree of clogging in the gravel-sized LCS media reported by both Brune et al. (1994) and Pasky et al. (1998) indicates that LCSs constructed with sand will perform poorly. If sand had been used in their laboratory tests, the rate and total amount of reduction in drainable almost certainly would have been very large. This should not be surprising. Sand filters have been used for years to process drinking water as a means to remove solids and viruses. Because the sand is as a filter, sand filters accumulate clogging agents and regularly require back-flushing to maintain their perviousness (Clark et al. 1997). Flowing leachate through sand in a LCS will exacerbate these clogging mechanisms because leachates almost always contain more biological material and suspended solids than un-processed drinking waters. Accordingly, if drinking waters will clog sand filters, then sands used to carry leachate in a landfill will almost certainly become clogged.

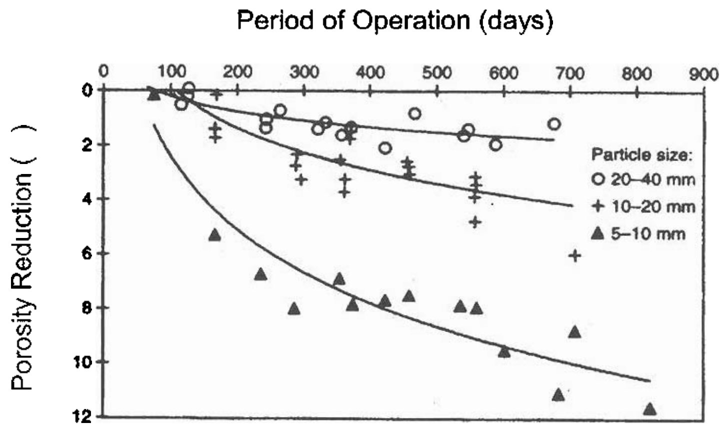

Fig. 15: Reduction in drainable porosity as a function of time (adapted from Pasky et al. 1998).

The distribution of particle sizes can be equally as important as the median particle size, as illustrated by data from the column tests conducted by Brune et al. (1994). The columns in their study containing broadly graded gravels (1-32 mm) "suffered almost a complete loss of permeability" and had "hardly any [visible] pores in the upper section." Fines in the granular medium can also contribute to clogging. In a field exhumation of a LCS in Ontario, Canada, Fleming et al. (1999) found that fines from the granular medium formed part of the incrustation material clogging the LCS.

Generation of fines generally is more problematic with crushed rock and with limestone or dolostone gravels. Angular surfaces on crushed rock tend to break off during handling, contributing to fines in the LCS. The amount of fines increases when crushed rock is handled more often prior to placement. Gravels prepared by crushing carbonate rock (limestone and dolostone) also tend to have more fines than gravels prepared from hard rock because carbonate stone is more readily abraded during handling.

3.2 FILTERS AND RATE OF LOADING

LCS media can be protected using a properly designed geotextile filter. The utility of geotextile filters is evident in the column tests conducted by Brune et al. (1994). They found that gravel in columns with a non-woven HDPE geotextile filter between the LCS medium and the waste "remained loose and covered with only a light film," whereas "gravel in the columns without [a] geotextile...was covered with a thicker film...and clumped together." Fleming et al. (1999) also found that clogging of gravel at the Ontario landfill they studied was greatly reduced in a section where the LCS was protected by an overlying geotextile. The underlying gravel was reported to be "relatively clean" and free of fines, which apparently were filtered by the geotextile.

As with granular media, geotextiles tend to accumulate clogging materials over time (Brune et al. 1994, Koerner and Koerner 1994). However, column studies conducted by Koerner et al. (1994) suggest that clogging materials tend to

form a filter zone on the surface of the geotextile, and that the hydraulic conductivity of the geotextile stabilizes over time (Fig. 16). At equilibrium, the hydraulic conductivity of the geotextile may be reduced dramatically (several orders of magnitude) relative to its initial value, but the geotextile generally will remain more permeable than the overlying waste and not impede flow into the LCS. ASTM D 1987 provides a method to assess the degree of clogging and the equilibrium hydraulic conductivity of the geotextile.

Location of a geotextile filter is the most important factor affecting its success because the location affects the rate of mass loading (mass of clogging material per unit area of geotextile per unit time). Geotextiles used as pipe wraps or trench wraps almost always become too impermeable for the flow to be carried because the area of geotextile conducting flow (i.e., the area adjacent to the holes or perforations in the pipe or the gross area surrounding a trench) is much smaller than the areal extent of the leachate collection system from which the leachate emanates. For example, in three of four US field sites exhumed by Koerner et al. (1994), the geotextile pipe wrap had effectively become impervious near the holes in the collection pipe. A much more effective location for a geotextile filter is to place it over the entire surface of the leachate collection system. When the geotextile is placed in this manner, the mass-loading rate is lower and the geotextile is more likely to accommodate clogging materials without becoming impervious. Koerner et al. (1994) describe a design method for geotextile filters that accounts for the location of the geotextile and the rate of mass loading.

The importance of mass loading also applies to other geometric characteristics of the leachate collection system. LCSs that are designed with more closely spaced pipes and shorter pipe runs are less likely to clog because the mass loading near the pipe is lower (Fleming et al. 1999, Rowe et al. 2000).

3.3 MAINTENANCE

Maintenance is a critical aspect of LCS systems that is often ignored. LCS pipes should be jetted at least annually, and then inspected with a downhole camera for evidence of clogging. Piezometers should also be incorporated into the LCS to monitor head levels throughout the landfill. Transducers used to monitor head level should be selected that have the resolution necessary to measure the relatively low heads desired in LCSs.

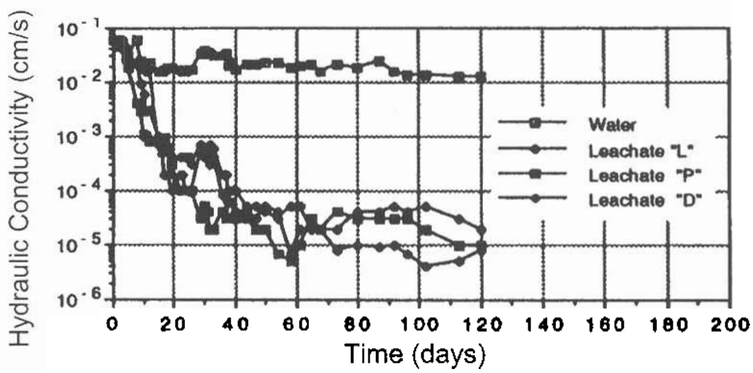

Fig. 16: Reduction in hydraulic conductivity of a non-woven geotextile when permeated with MSW leachate (adapted from Koerner et al. 1994).

3.4 RECOMMENDATIONS FOR DESIGNERS

Models are currently in development that can be used to assess how various design variables influence the rate and magnitude of LCS clogging and to select materials and geometries that will remain sufficiently permeable to effectively drain a landfill throughout its design life (e.g., see Rowe and Van Gulck 2001, Cooke et al. 2001). In the interim period, the designer is provided with the following recommendations in lieu of a more rigorous design approach.

- Select LCS media that have larger particle sizes and are uniformly graded. Ensure that the median particle size is at least 50 mm and the coefficient of uniformity is less than 1.5.
- If practical, use rounded river gravels rather than crushed rock to limit the amount of fines generated during handling. Minimize the number of times the stone is moved between the source and placement in the landfill cell. Ensure the *in-place* (i.e., as placed in the landfill cell) percent fines is no more than 4%.
- Place the LCS stone over the entire base of the cell and place a non-woven geotextile filter between the LCS medium and the overlying waste. Avoid heat set geotextiles and place a soil buffer between the geotextile and the waste (see Koerner and Koerner 1990).
- Reduce mass loading by reducing pipe spacings and limiting pipe runs. Use pipe spacings less than 30 m (60 m in sawtooth designs) and limit pipe runs to less than 400 m to make clean outs practical.
- Ensure that LCS pipes are jetted and inspected at least annually, and preferably semi-annually.
- Avoid carbonate rich gravels such as limestone and dolostone and limit the carbonate content to 10%. Although this recommendation has less supporting field data than the other recommendations, qualitative reports suggest that fewer clogging problems occur in LCSs constructed with non-carbonaceous stone.

4 COVERS

In contrast to lining systems, covers are exposed to a much different environment where stresses are low, unsaturated conditions persist, and interaction with the atmosphere occurs continuously. Covers used today can be grouped into two categories, resistive covers and water balance covers that reflect the methodology used to limit the ingress of precipitation. Resistive covers employ a conventional hydraulic barrier (compacted clay layer, geosynthetic layer, or composite clay-geosynthetic barrier) that limits flow by its imperviousness. Water balance covers limit ingress of water by storing water that infiltrates and then removing this water through evaporation and transpiration by plants.

4.1 RESISTIVE COVERS

Most resistive covers employ a similar design that consists of several layers, each with a unique function. A schematic of a resistive cover with a composite barrier layer is shown in Fig. 17. The lower most layer is the hydraulic barrier, which may consist of a compacted clay layer or GCL, a geomembrane, or a composite barrier comprised of a clay barrier or a GCL overlain by a geomembrane. Covers that employ a geomembrane generally include a drainage layer above the geomembrane to prevent pore water pressures from building and causing instability (Bonaparte et al. 1996). A vegetated protective layer is placed on top of the drainage layer. Composition and thickness of the protective layer depend on the degree of protection that is required. Thicker layers are used when the barrier layer must be protected from environmental stresses such as freezing or desiccation. A cobble sub-layer may also be used in the lower portion of the protective layer to prevent intrusion by biota.

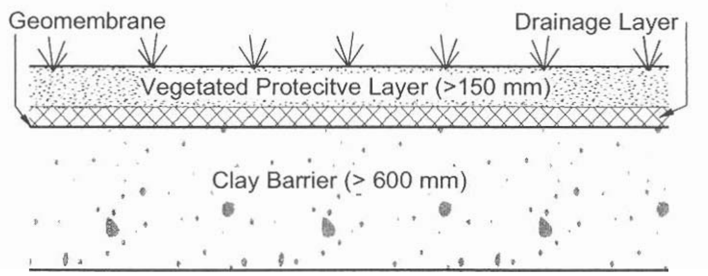

Fig. 17: Schematic of a conventional resistive cover that includes a drainage layer and a composite

Because effectiveness of the barrier layer generally controls the performance of a resistive cover, Section 4.1 focuses on factors affecting the barrier layer. Giroud et al. (2001) provide a comprehensive discussion of drainage layer design. A review of drainage layer design is in Benson (2000).

4.1.1 Resistive covers with compacted clay as the barrier layer

Resistive covers with a compacted clay layer as the hydraulic barrier are still commonly constructed today, particularly for closure of old unlined waste sites and older clay-lined landfills. This practice continues despite the potential for severe damage of clay barriers by frost and desiccation (Corser and Cranston 1991, Benson and Othman 1993, Othman et al. 1994, Benson and Khire 1995, Albrecht 1996, Khire et al. 1997, Albrecht and Benson 2001). Essentially all compacted clay barrier layers will be damaged by either freezing, desiccation, or both, unless the clay is protected by a geomembrane to prevent desiccation and a thick enough protective layer to prevent freezing of the clay.

Field data are shown in Fig. 18 that illustrate the damage incurred by a clay barrier exposed to desiccation. The data were collected from an instrumented cover located in southwestern Georgia, USA. This location in Georgia has a warm and humid climate with an average rainfall of 1280 mm and no snow. The cover consists of a compacted clay barrier 460 mm-thick overlain by a vegetated protective layer 150 mm thick, which is the conventional cover design used in Georgia for closure of un-lined waste sites. Lean clay was placed in three lifts each 150 mm thick. Local topsoil seeded with Bermuda grass was used for the protective layer. The compacted clay layer had an as-built saturated hydraulic conductivity of 5×10^{-8} cm/s. The cover is instrumented with a large (10 m x 20 m) lysimeter for collecting percolation, a surface runoff collection system, and a series of probes to measure the distribution of water content and matric potential within the cover (Bolen et al. 2001). A description of the lysimeter is in Benson et al. (2001).

For the first nine months after construction (March–November 2000), the percolation rate remained low ($\approx$50 mm/yr). September and October 2000 were unseasonably dry, and desiccation of the cover occurred as shown by the nearly monotonic drop in soil water storage during this period. After this period, the percolation rate increased by about an order of magnitude ($\approx$500 mm/yr) even though the rate of precipitation was not appreciably different after October 2000 than it was prior to September 2000. The percolation record after October 2000 also is irregular. Abrupt increases in percolation follow precipitation events, which is a characteristic of preferential flow. An examination of the cover conducted in response to this change in behavior showed that desiccation cracks had formed, and that these cracks are most likely responsible for the preferential flow that is occurring.

The data shown in Fig. 18 illustrate that clay barriers that undergo desiccation will be damaged severely, even in a short period of time. Admittedly, the cover in this study had a very thin protective layer. A thicker protective layer probably would have reduced the amount of desiccation that occurred, and prolonged the useful life of the clay barrier. However, thick (> 1 m) protective layers have been found to be inadequate for protecting clay barriers from desiccation, even in humid climates. Albrecht (1996) describes the hydrological performance of a resistive cover with a clay barrier layer overlain by 1 m of protective cover soil at a humid site in central Wisconsin, USA. Percolation from this cover increased by nearly an order of magnitude as a result of desiccation, and the saturated hydraulic conductivity of the clay barrier layer increased by a factor of 1000. Similarly, Melchior (1997) describes a resistive cover with a compacted clay barrier layer and 1 m of overlying soil that was severely cracked by desiccation in the humid climate of Hamburg, Germany. Percolation from the cover in Hamburg increased by an order of magnitude as a result of the damage caused by desiccation.

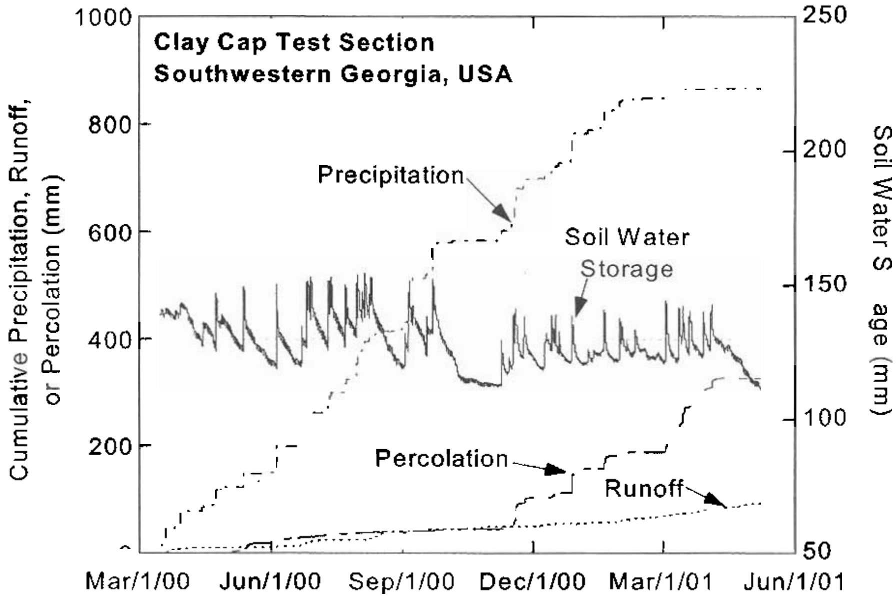

Fig. 18: Water balance of an instrumented resistive cover with a compacted clay barrier layer in southwestern Georgia, USA.

Frost can have an equally damaging effect on clay barrier layers (Benson and Othman 1993, Othman et al. 1994). An example of the damage that can occur is illustrated in Fig. 19, which shows the hydraulic conductivity of a compacted clay as function of the number of freeze-thaw cycles. The laboratory tests were conducted on specimens compacted in the laboratory and tested following methods described in ASTM D 6035. The field data are from a compacted clay barrier layer constructed in southern Michigan, USA, with the same clay used in the laboratory tests. Within three to five cycles, the hydraulic conductivity increased by two orders of magnitude (Benson et al. 1995). Vertical shrinkage cracks and horizontal cracks formed by ice lenses were responsible for this large increase in hydraulic conductivity.

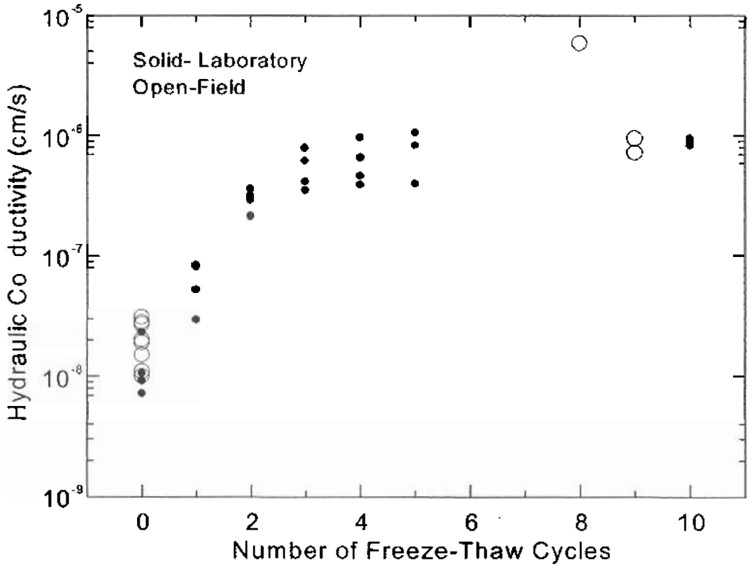

Fig. 19: Hydraulic conductivity of compacted clay barrier as a function of number of freeze-thaw cycles (adapted from Benson et al. 1995).

4.1.2 Resistive covers with a GCL as the barrier layer

GCLs are often perceived as being more resistant to damage caused by freezing and desiccation than compacted clays. Field studies have confirmed that GCLs are resistant to frost damage. Kraus et al. (1997) report on a comprehensive field and laboratory study where GCLs were subjected to freezing and thawing under conditions characteristic of those in resistive covers. They found that the hydraulic conductivity was essentially unchanged even after 20 freeze-thaw cycles. The hydraulic conductivity did not change because the cracks formed by ice lenses closed when the GCLs thawed. Similar closure of cracks does not occur in compacted clays because compacted clays are stiff compared to the soft bentonite in hydrated GCLs.

In contrast to freeze-thaw cycling, desiccation appears to have a detrimental impact on the hydraulic conductivity of GCLs. Studies by Melchior (1997), James et al. (1997), and Lin and Benson (2000) indicate that exchange of divalent cations present in natural pore waters for sodium in the exchange complex of 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 fail in the long term unless cation exchange can be prevented.

An example of this phenomenon occurred in a cover over a fly ash landfill in southwestern Wisconsin, USA. A cross-section of the cover is shown in Fig. 20. Failure of the GCL was detected because the agency regulating the site required the owner to install two lysimeters (BL1 and BL2) beneath the GCL to monitor the percolation rate.

Percolation collected in the two lysimeters is shown in Fig. 21. Excessive percolation was first noticed during the spring thaws of 1997. The GCL was exhumed in mid-1997 and inspected to determine a cause for the excessive leakage rates. The inspection suggested that thinning of the GCL due to pressure applied by gravel in the lysimeter was the cause for the failure. The exchange complex of the bentonite was not examined. A laver of sand was added to the lysimeter above the gravel as a cushion, a new GCL was placed, and the surface layers were replaced

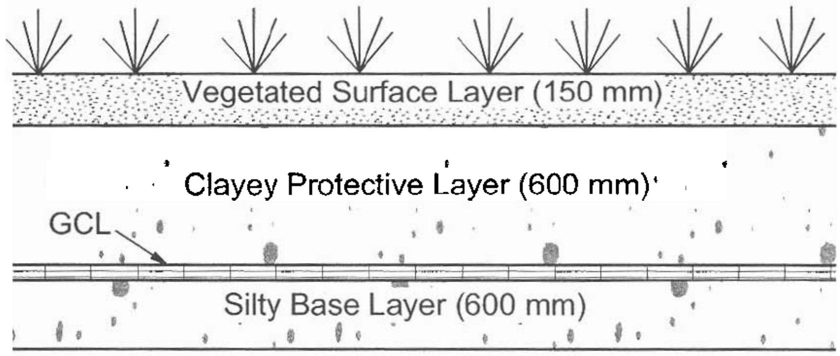

Fig. 20: Schematic of cover with a GCL for fly ash landfill in southwestern Wisconsin, USA.

Percolation monitoring continued after the lysimeters were re-built in 1997. Approximately 15 months after re-construction, the leakage rate became excessive again (Fig. 21) and the GCL was exhumed. Inspection of the GCL revealed that the bentonite was dry and cracked, as shown in Fig. 22. Analysis of the exchange complex 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.

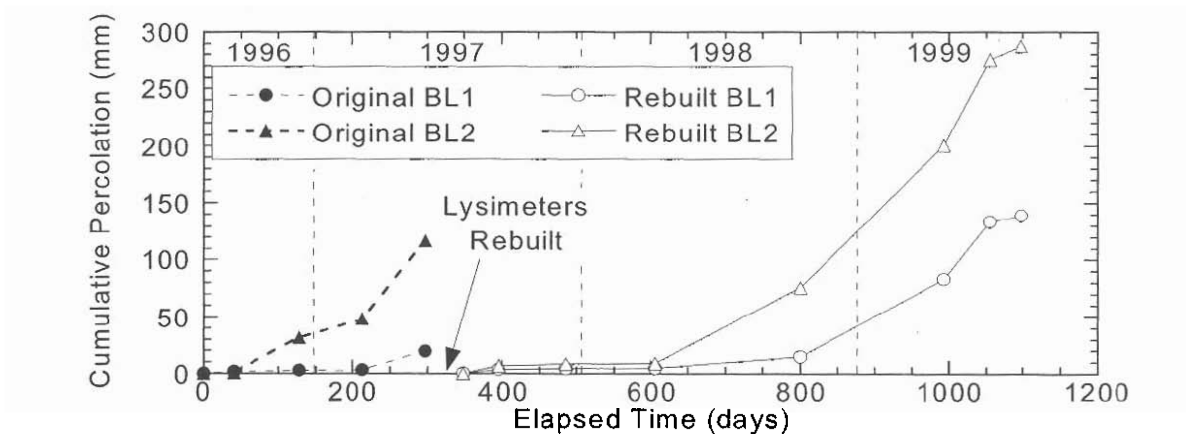

Fig. 21: Percolation rates recorded by two lysimeters beneath the GCL cover over a fly ash landfill in Wisconsin, USA. The cover was constructed in mid-1996

The in-service hydraulic conductivity of the GCL was estimated from the leakage rate measured in 1999 and found to range between $3x10^{-7}$ to $7x10^{-7}$ cm/s. A specimen of the exhumed GCL was tested and found to have very similar hydraulic conductivity ($2x10^{-7}$ cm/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).

4.1.3 Resistive covers with composite barrier layers

In contrast to the data for clay barrier layers, field data suggest that resistive covers with a composite barrier layer are very effective at minimizing percolation and are not prone to damage by desiccation (Corser and Cranston 1991, Melchior 1997, Dwyer 1998, 2001). The superior performance realized with a composite barrier is due to the dual functions provided by the geomembrane: (i) minimizing ingress of infiltrating water and (ii) mitigating the upward flow of water from the underlying compacted clay layer. In stark contrast to the observations cited in Sec. 4.1.1, exhumations of composite barrier layers as much as eight years after construction have shown none of the signs of weathering that are commonly observed in compacted clay barrier layers (Melchior 1997). Rather, clay layers typically have the same appearance as they had immediately after construction (Corser and Cranston 1991).

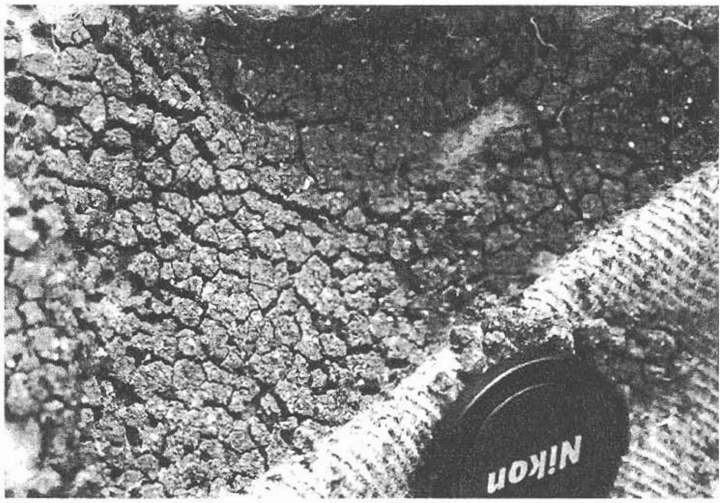

Fig. 22. Dry and cracked bentonite in GCL over the fly ash landfill in Wisconsin, USA.

The superior field performance of resistive covers with composite barrier layers is illustrated in Table 1, which summarizes percolation rates for resistive covers with compacted clay and composite barrier layers for several studies. The data are segregated into humid and semi-arid sites. In general, percolation rates for resistive covers with composite barriers are less than 10 mm/yr, and more commonly are on the order of 1-3 mm/yr in humid climates and 0.1-1 mm/yr in semi-arid climates. In contrast, percolation rates for resistive covers with clay barriers are typically about 100 mm/yr in humid climates and 10-30 mm/yr in semi-arid climates.

Table 1: Summary of Percolation Rates for Resistive Covers.

Source	Barrier Layer	Percolation Rate (mm/yr)	
		Humid Climates	Semi-Arid Climates
ACAP[a]	Clay	10-500	-
ALCD[b]	Clay	-	5
Melchior (1997)	Clay	150	-
Khire et al. (1997)	Clay	50-150	10-30
Montgomery and Parsons (1990)	Clay		
ACAP	Composite	0.6-8	0-1[c]
ALCD	Composite	-	0.1-2[c]
Melchior (1997)	Composite	0.5-3	

[a]ACAP is the Alternative Cover Assessment Program sponsored by USEPA (see Bolen et al. 2001); [b]ALCD is the Alternative Landfill Cover Demonstration conducted by Sandia National Laboratory under the sponsorship of the US Department of Energy (see Dwyer et al. 2001); [c]data include composite barrier layers containing GCLs.

4.2 WATER BALANCE COVERS

Lower costs, simplified maintenance, and the absence of geosynthetic layers have resulted in significant interest in water balance covers as engineered alternatives to conventional resistive covers (Benson et al. 2001, Bolen et al. 2001). Water balance covers are earthen covers employing water storage principles that are designed to perform equally as well as their resistive counterparts. They are designed to store water during periods of elevated precipitation and limited evapotranspiration and to release the stored water to the atmosphere during drier periods with higher evapotranspiration. When the total water storage (S) exceeds the storage capacity of the cover (S_c), percolation occurs.

Accordingly, design of water balance covers consists of adjusting the storage capacity until the percolation rate is acceptable. The storage capacity is adjusted by varying the thickness of a layer of finer textured soil used to store water (Stormont and Morris 1998, Khire et al. 2000).

A variety of designs are being considered for water balance covers, but most can be classified as monolithic barriers or capillary barriers. Monolithic barriers consist of a thick vegetated layer of finer textured soil that has high water storage capacity. The cover is made sufficiently thick so that the percolation rate remains below a target value. Capillary barriers employ a contrast in texture (e.g., silt layer overlying a sand layer) to form a capillary break that impedes downward flow until the finer soil becomes nearly saturated. A break exists at the textural interface because, at the same matric potential, coarse soils typically have lower unsaturated hydraulic conductivity than finer soils (Stormont and Anderson 1999, Khire et al. 1999). Design methods for monolithic covers are described in Benson (1999) and Manassero et al. (2000). Stormont and Morris (1998) and Khire et al. (2000) describe design methods for capillary barriers.

An example of the field performance of a water balance cover in a humid climate is shown in Fig. 23. The data are from the same site in southwestern Georgia, USA that was described in Sec. 4.1.1. At this site, the two covers are being evaluated in a side-by-side setting. The water balance cover at the site is a monolithic design with a 1.3-m-thick layer of sandy clay mixed with compost (75% soil -25% compost) that is vegetated with hybrid poplar trees on 3 m centers (Bolen et al. 2001). The developer of this monolithic cover refers to it as an "ECap." An understory of Bermuda grass exists beneath the poplar trees. The sandy clay in the ECap was placed more loosely than the compacted clay used in the adjacent resistive cover, and had a saturated field hydraulic conductivity of 5×10^{-7} cm/s when constructed.

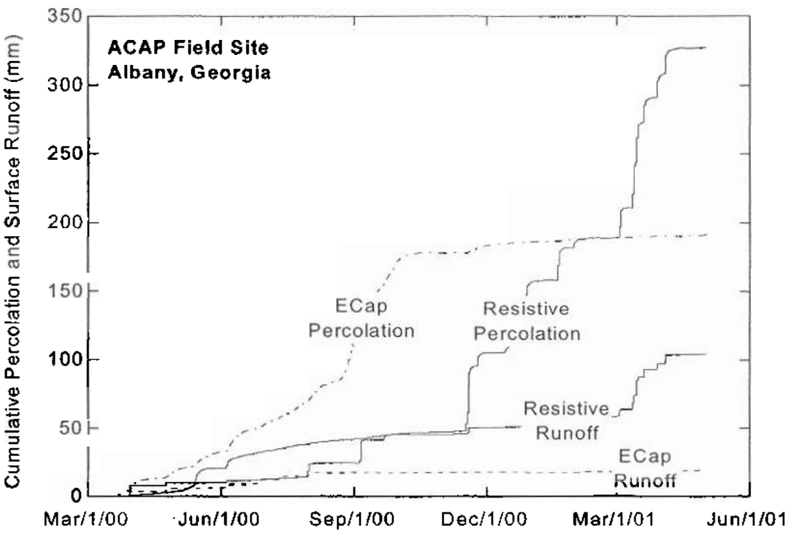

Fig. 23. Cumulative percolation and runoff from resistive and water balance (ECap) covers in southwestern, Georgia, USA.

Initially percolation from the ECap was greater than that from the resistive cover (Fig. 23) because the ECap had higher saturated hydraulic conductivity when the two covers were constructed. However, by the end of November 2000, percolation from the resistive cover was occurring at a substantially higher rate than that from the ECap. Percolation from the ECap occurred at a lower rate for two reasons. First, the soil used for the ECap was compacted less densely, and thus was less sensitive to damage during the period of desiccation that occurred during September-October 2000. As a result, the ECap probably contains fewer desiccation cracks than the resistive cover. Second, poplar tree growth began in earnest towards the end of 2000, resulting in a large amount of water being removed from the ECap by the trees. The grasses on the resistive cover extract far less water than the trees on the ECap, making more water available to percolate downward.

Water balance covers can be even more effective at limiting percolation in semi-arid and arid climates. Vegetation in these climates is very effective in scavenging water from the soil, leaving the soil with an empty reservoir for soil water storage at the end of each growing season. Six water balance covers in semi-arid climates are currently being evaluated by the United States Environmental Protection Agency's (USEPA) Alternative Cover Assessment Program (ACAP). Five of these covers employ a monolithic design; the other is a capillary barrier. Percolation rates for these covers are summarized in Table 2. The percolation rates are very low, ranging between 0-1.4 mm/yr. These percolation rates are comparable to percolation rates for resistive covers with composite barriers located in similar climates (Table 1).

In some cases, a water balance cover may be needed that transmits practically no percolation. One approach to design such a cover is to use the "dry barrier" concept (Ankeny et al. 1997). A dry barrier is a capillary barrier constructed with a layer of finer textured soil overlying a cobble layer. Air is routed through the coarse layer using passive ventilation techniques to remove moisture that reaches the interface between the fine and coarse layers (Albrecht 2001,

Albrecht and Benson 2002). The methodology is effective because atmospheric air generally has far lower humidity than soil gas, and can absorb moisture from the soil.

Table 2: Percolation Rates for ACAP's Water Balance Covers in Semi-arid Locations

Site	Cover Type	Layering	Percolation Rate (mm/yr)
Sacramento, California	monolithic	1.2 m sandy clay	0
Sacramento, California	monolithic	2.8 m sandy clay	1.4
Polson, Montana	capillary barrier	0.6 m silt over 0.6 sand	0.3
Helena, Montana	monolithic	1.2 m sandy clay	0
Finley Buttes, Oregon	monolithic	1.2 m sandy silt	0
Finley Buttes, Oregon	monolithic	1.8 m sandy silt	0
Altamont, California	monolithic	1.1 m crushed clayston e	0

An example of water removal rates that can be achieved with a dry barrier is shown in Fig. 24. The data are from a dry barrier test section 10 m wide by 20 m long that is located in eastern Oregon, USA. The test section contains a lysimeter similar to that described in Section 4.1.1. Very fine silty sand was used for the surface layer (0.6 m thick) and river cobble having a nominal diameter of 150 mm was used for the coarse layer (0.3 m thick). Atmospheric air was routed through the barrier using stacks made from PVC pipe that were attached to diffusers in the coarse layer. During the monitoring period, approximately 1.2 mm of water was removed by the passive ventilation system, which corresponds to an annual rate of 2.4 mm/yr. No percolation was recorded during the monitoring period. The water removal rate for this test section is higher than the percolation rate currently being recorded at the ACAP field sites, indicating that covers employing the passive dry barrier technology may be capable of eliminating percolation into waste at semi-arid sites.

4.3 RECOMMENDATIONS FOR DESIGNERS

The following recommendations are made for engineers designing covers for waste containment systems.

- Uncovered clay barriers and GCLs in resistive covers undergo extensive weathering as a result of freezing and desiccation. Accordingly, compacted clay barrier layers or GCLs should not be used in resistive covers without an overlying geomembrane and adequate frost protection.
- Resistive covers with composite barriers employing compacted clay layers or GCLS are very effective at limiting percolation rates. When conducting hydrologic evaluations for resistive covers with composite barriers, percolation rates between 1 and 3 mm/yr should be used for humid sites. For semi-arid sites, percolation rates between 0.1 and 1 mm/yr should be used.
- Water balance covers employing trees or deep rooted grasses or shrubs can be effective alternatives at humid sites where the conventional cover relies on a resistive design with a compacted clay barrier.
- At semi-arid sites, water balance covers can perform as well as resistive covers with composite barrier layers. For water balance analyses, percolation rates between 0.5 and 1.5 mm/yr should be used for water balance covers at semi-arid sites.

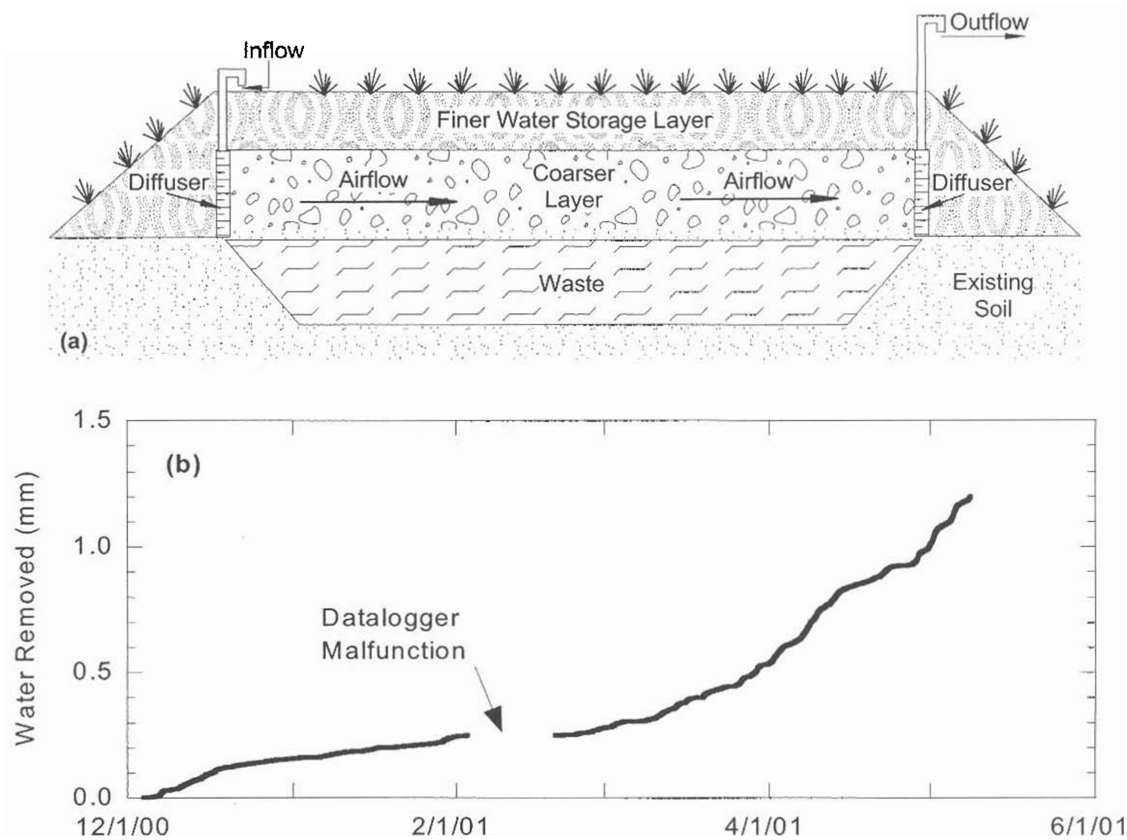

Fig. 24: Schematic of dry barrier test section (a) and water removed from test section during monitoring period (b) (adapted from Albrecht 2001 and Albrecht and Benson 2002).

5 SUMMARY

This paper has reviewed strategies for lining and covering waste containment systems, as well as systems used to remove leachate. Emphasis has been placed on factors influencing the field performance of these systems.

The discussion of liners included compacted clay liners, geosynthetic clay liners, geomembrane liners, and composite liners. Issues such as chemical compatibility, scaling from laboratory to field, and stability issues were covered. Field performance data reviewed in the paper indicate that compacted clay liners and composite liners are performing as intended, and that their performance can be predicted with standard calculation methods. Failures of lining systems typically occur when oversights are made during design or construction. Many failures can be eliminated by carefully preparing construction specifications and by carefully scrutinizing changes made to designs prior to construction.

Factors affecting the performance of leachate collection systems were reviewed in the context of laboratory column experiments and observations made at sites where leachate collection systems have failed. These studies show that leachate collection systems are most likely to be successful when constructed with coarse uniformly graded gravel with little fines that is overlain by a non-woven geotextile filter. Smaller pipe spacings and more regular maintenance will also enhance the performance of leachate collection systems.

The discussion of covers focused primarily on environmental factors that affect hydrologic performance in the context of laboratory data and field performance data. Resistive covers employing a soil barrier with low saturated hydraulic conductivity (compacted clay or GCL) as the primary impedance to flow have been shown to perform poorly due to the effects of desiccation and frost action. Covers with a composite barrier layer or water balance covers have a better performance record, and can effectively eliminate percolation into waste.

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