Geosynthetics, J. Kuwano & J. Koseki (eds)

Geosynthetic barriers for environmental protection at landfills

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Keywords: geomembrane, GCL, durability, coastal landfill, liner, cover, barrier

ABSTRACT: Field evidence of the effectiveness of composite liner systems constructed using a rigorous construction quality assurance program clearly demonstrates well designed and constructed composite liner systems leak at such a slow rate that the impact of this leakage on human health and the environment is negligible. However, analysis indicates that the service life of the primary geomembrane may be on the order of 100 to 200 years may be expected for many landfills. The field data also shows that GCLs are clearly as effective as, if not superior to, compacted low permeability soil as the underlying component of a composite liner system. Field data also indicates that GCLs can be effective in capping systems despite concerns over durability. The service life of a GCL used in a cap or liner may be expected to be in the order of hundreds to thousands of years from a hydraulic conductivity perspective. The durability of the fibers may limit the service life of a reinforced GCL to less than a hundred years from a shear strength perspective in some cases. The most significant advance with respect to the use of geosynthetic barrier systems since 2002 has been the increasing implementation of these systems in developing countries. The development of geomembrane-based barrier systems for coastal landfills is an emerging technology employing geosynthetic barrier systems. Numerical analyses of landfill barrier performance have helped identified key factors in their effectiveness. Slope stability and interface shear strength continue to be a major issue for the design of geosynthetic barrier systems. Local stability of geosynthetic barriers is also an important concern for steep-sided landfills.

1 INTRODUCTION

By the beginning of the 21st Century, geosynthetic barrier layers, including geomembranes and geosynthetic clay liners (GCLs), were firmly established in industrialized countries as the primary means of preventing migration into the environment of liquids and gases from both municipal solid waste (MSW) and hazardous waste landfills. Geosynthetic barriers are also widely used to prevent infiltration through landfill covers. Due to their technical and cost effectiveness, use of this technology has now spread to less industrialized countries. Over the past decade, substantial information has accumulated on both the effectiveness and durability of geosynthetic barrier systems. This information clearly demonstrates the short term effectiveness of properly designed and constructed geosynthetic barriers and indicates that such barriers may be expected to maintain their integrity for time periods on the order of decades and in most circumstances for hundreds of years with a high degree of confidence. However, field data on barrier performance is limited to twenty to thirty years for geosynthetic barriers, requiring extrapolation of field and laboratory data when projecting performance over longer periods. Extrapolations suggest that under some circumstances geomembranes and GCLs may significantly degrade over time periods of less than 100 years.

In industrialized countries, composite liner systems composed of a geomembrane overlying a low permeability soil layer have become the standard of practice for disposal of wastes on land, either as a single composite liner or in double liner systems. The geomembrane provides the primary barrier to advective transport of liquids and gases from the waste. The underlying low permeability soil layer provides a redundant barrier to mitigate advective transport through defects in the geomembrane. With increasing frequency, GCLs are being used instead of compacted low permeability

soil layers in liner systems. Composite barrier layers are generally overlain by a high permeability liquid collection layer (Figure 1) to limit the head on the geomembrane layer, further minimizing advective transport across the liner.

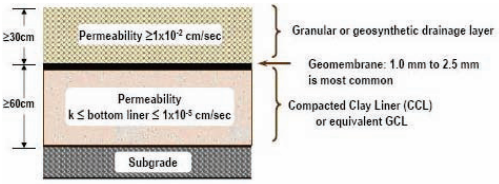


Figure 1. US EPA single composite liner system

In situations where advective transport must be held to an absolute minimum, double liners that employ two barrier layers with an intervening liquid collection layer (sometimes referred to as the leak detection layer) are employed. Double liners may employ either single layer barrier layers or composite barrier layers. Generally, double liners are required for hazardous waste facilities, while either single composite liners or double liners may be employed at MSW facilities.

Geosynthetic barrier systems are also being used with increasing frequency in landfill cover applications. In particular, GCLs are being used instead of compacted low permeability soil in final covers, particularly in Europe. Composite caps employing a geomembrane overlying compacted low permeability soil or a GCL are generally used when infiltration to the waste mass must be held to an absolute minimum, e.g. in a hazardous waste landfill.

Due at least in part to the maturing of these technologies and to accumulating information on their effectiveness, over the past decade less industrialized countries have begun to adopt geosynthetic barrier systems for environmental protection at landfills, either on a regulatory basis or on a voluntary basis. Regulations in less industrialized societies are often less stringent than in industrialized nations due to socio-economic constraints. However, the use of geosynthetic barriers in liner and cover systems is rapidly becoming standard practice for both hazardous waste facilities and large municipal waste facilities in many developing nations.

Coastal landfills, where waste is disposed of in shallow near-shore confined disposal areas, are being considered with increasing frequency in coastal areas where there is little available inland space or where suitable inland disposal sites are a long distance from the waste sources. Coastal landfills have been constructed in recent years in Japan, China, and Korea. Japan has a fairly extensive history of MSW coastal landfill disposal as well as post-closure reclamation of these facilities for beneficial use (e.g., Shimizu 1997, Katsumi and Kamon 2002, Kamon et al. 2005). Japanese regulations require the construction of an engineered barrier for lateral containment at coastal landfill sites and these barrier systems generally employ geosynthetic barrier elements.

As geosynthetic barrier systems become more common and as records of the performance of these systems accumulate, the evidence of their effectiveness as barriers to liquid and gas transport grows. Field data of barrier effectiveness includes indirect data from groundwater and vadose zone monitoring adjacent to landfills with geosynthetic barriers and direct data from detailed studies of leakage rates into double liner systems, post-construction/pre-waste placement electrical leak detection surveys, field test sections, and tests on exhumed geosynthetic barrier elements. Groundwater and vadose zone monitoring data collected in general surveys of the performance of landfills with geosynthetic barriers indicate excellent performance with respect to liquid containment. However, problems with respect to gas containment have been identified at some modern landfills that employ geosynthetic barrier systems, attributable to design details that allowed gas to move around the edge of the barrier system and into the subsurface. Field data on leakage rates into double liner systems have confirm analytical models of the effectiveness of composite barriers and also indicate the superiority of GCLs compared to low permeability soil liners for double liner systems. Post-construction electrical leak detection surveys demonstrate the effectiveness of modern quality assurance techniques in limiting construction defects. Test sections and tests on exhumed geomembranes and GCLs have both demonstrated their durability and, particularly with respect to GCLs, identified limitations.

Engineering concerns with respect to the integrity and durability of geosynthetic barriers include short term mechanical durability and long term durability due to chemical reactions, durability when subject to environmental stresses, and overall barrier system stability. Short term mechanical durability considerations for geomembranes include puncture due to overburden stresses and tensile tearing due to waste settlement. Long term geomembrane durability concerns are generally associated with anti-oxidant depletion. Durability concerns with GCLs include cyclic drying and wetting (i.e. desiccation), cyclic freezing and thawing, and chemical compatibility (e.g. cation exchange and compatibility with non-aqueous liquids). For coastal landfills, integrity concerns include cyclic forces due to wave, tidal, and sometimes tsunami loading,

stresses due to settlement of compressible foundation materials, the challenging installation environment, and overall stability.

Overall and local stability are significant concerns for geosynthetic barrier systems. The primary factors influencing overall stability of a geosynthetic barrier system (beside system geometry and external loads) are the interface shear strength and/or the internal shear strength of a GCL. There is still significant debate about the appropriate interface shear strength (peak or residual) and the associated factor of safety to use in stability analyses. The durability of the reinforcing fibers is also a concern for reinforced GCLs. Local stability concerns may arise due to stress imposed on side slope liner systems by waste mass deformations.

2 THE USE OF GEOSYNTHETIC BARRIERS IN LANDFILLS

2.1 *Liner Systems*

2.1.1 *Industrialized countries*

By the turn of the 21st century, the use of geomembrane – low permeability soil composite barrier layers in base liner systems for solid and hazardous waste landfills was a well established practice in industrialized countries. Furthermore, many countries allowed for the use of GCLs as a replacement for a compacted low permeability soil layer in composite liner systems, either explicitly or implicitly through regulations allowing "engineered alternatives." Bouazza et al. (2002) summarized "typical" regulatory requirements for base liner systems from 17 predominantly industrialized jurisdictions in North America, Europe, the Asia-Pacific region, and Africa. Only two of these jurisdictions, South Africa and the province of British Colombia in Canada, did not require a geomembrane barrier for their MSW landfills, although the state of New South Wales in Australia only required a geomembrane to be used "in areas of significant threat to the environment."

A 2004 survey of regulations for MSW landfills in the European Union, the states of New South Wales and Victoria in Australia, Brazil, and South Africa conducted for the California Integrated Waste Management Board (CIWMB) by GeoSyntec Consultants (GeoSyntec, 2004) arrived at findings similar to Bouazza et al. (2002): only South Africa did not require composite liner systems with geomembranes for MSW landfills. (While the European Union (EU) directives do not explicitly call for geomembranes, most EU countries require them when implementing these directives.)

Current Japanese regulations require that the base of a MSW landfill include at least 5 m of low permeability soil with a saturated hydraulic conductivity no greater than $1x10^{-5}$ cm/s or one of the following: (1) two geomembranes which sandwich a nonwoven fabric or other cushion material, (2) a geomembrane underlain by an asphalt-concrete layer at least 50 mm in thickness with a saturated hydraulic conductivity no greater than $1x10^{-7}$ cm/s, or (3) a geomembrane underlain by a clay liner at least 0.5 m in thickness with a hydraulic conductivity $\leq 1x10^{-6}$ cm/s. However, concerns about the effectiveness of these regulations, particularly with respect to the thickness and saturated hydraulic conductivity of the low permeability soil, have been raised (Katsumi et al., 2001).

While there have been no major changes in the waste containment regulations for most industrialized countries in recent years, there has been a trend towards greater acceptance of GCLs as an alternative to low permeability soil layers. However, an attenuation layer may have to be included beneath the GCL to provide adequate diffusion resistance. While GCLs are used primarily in capping systems in Europe, Bouazza et al. (2002) identified Switzerland as explicitly allowing the use of GCLs in a composite liner. Other countries, including the United States and Canada, regularly allow the use of GCLs in composite liner systems, either alone or in combination with an underlying diffusion attenuation layer.

Some jurisdictions require an engineering demonstration of "equivalent protection to the environment" in order to replace a prescribed compacted low permeability soil layer with a GCL. However, in recent years, requirements for a formal demonstration of "equivalence" of GCLs have been relaxed GCLs are often accepted as an alternative to low permeability soil layers either *de facto* or on the basis of citations of demonstrations in the literature such as those provided by Bouazza et al. (2002) and Lake et al. (2004). Another factor contributing to increased acceptance of GCLs is field studies of liner and cover system performance. These studies show that GCLs are more effective with respect to advective flow resistance than composite liners employing "equivalent" compacted low permeability soil layers and that properly designed GCL cover systems can maintain their effectiveness even under relatively severe environmental stress.

As a result of this increased regulatory acceptance, geosynthetic clay liner systems are being used with increasing frequency as an alternative to low permeability soil layers in both MSW and hazardous waste landfills. One manifestation of this increased regulatory acceptance is revised guidance from the United States Environmental Protection Agency (US EPA) which now allows alternative base liners that employ a GCL to be used in landfills that recirculate leachate and in bioreactor landfills, a practice that was not

allowed under the original "Subtitle D" regulations requiring composite liner systems for MSW landfills. GCLs have also become popular as an alternative to low permeability soil layers for composite side slope liner systems due to the ease of construction of the GCL compared to compacted low permeability soil layers.

2.1.2 *Developing countries*

Perhaps the most significant advance in the use of geosynthetic barriers for environmental protection since the turn of the century is the increased use of geomembranes and composite liners in base liner systems for landfills in developing countries. Bouazza et al. (2002) reported that the design of geosynthetic lining systems for landfills in developing countries was "still in its infancy." However, the use of geosynthetic barriers at landfills in developing countries is growing rapidly. Ashford et al. (2000) reported on 10 of 12 landfills constructed in Thailand between 1996 and 2000. Seven of these landfills employed a composite liner system and an eighth landfill employed a single geomembrane liner. Only two of the ten landfills reported on did not employ a geomembrane barrier layer.

In South Africa, landfill standards are governed by the "Minimum Requirements for Waste Disposal by Landfill" (Department of Water Affairs & Forestry, 1998), which classifies landfills according to waste type (general or hazardous), size, and potential for leachate generation. Development of these requirements was guided by the precept that, since the majority of South Africa has a negative climatic water balance, properly designed and operated landfills should not generate significant leachate. These "Minimum Requirements" have since been adopted and modified for use in neighboring countries in Southern Africa, including Botswana, Namibia, Mozambique and Swaziland. For general waste (MSW) landfills, only compacted low permeability soil liners are required for base liner systems. For leachate generating landfills, a leakage detection layer is required beneath the primary low permeability soil liner, and a thinner secondary low permeability soil liner beneath the leakage detection layer is required. Where clay of suitable quality is not available, a GCL can be used to replace the low permeability soil, provided that it is "sandwiched" between soil layers to provide support and confining pressure.

The use of GCLs as replacement liners for low permeability soil liners in Southern Africa is rapidly gaining popularity, mainly because of the scarcity of good clay, and because the cost of GCLs has virtually halved since their introduction over 10 years ago. A good example of this is the TSB Komati sugar mill landfill in South Africa. The first phase of the landfill was constructed in 1994 using a bentonite modified soil liner. In 1999, the landfill was extended by using a GCL instead of the bentonite modified soil, with far simpler construction methods and better quality control.

For hazardous waste landfills, the South Africa minimum requirements call for a double geomembrane – low permeability soil composite liner systems, including a leakage detection layer. Currently there are five commercial hazardous waste landfills in South Africa, as well as a number of industry owned hazardous waste landfills used exclusively for their own disposal requirements. All of the hazardous landfills have double composite liner systems, generally comprised of a 1.5mm or 2mm HDPE primary geomembrane overlying 600mm of compacted clay, with a 150mm leakage detection layer of stone or sand, and a 300mm compacted clay secondary liner. One cell at the largest hazardous landfill in South Africa was lined with a flexible polypropylene (fPP) geomembrane in 1997. However due to concerns about the integrity and long-term performance of this type of liner, fPP has subsequently been disallowed for use in landfill base liner systems in South Africa.

For the hazardous waste facility constructed in the desert at Walvis Bay, Namibia, in 2001, since no clay was available, the base liner consisted of a 1.5mm HDPE geomembrane and GCL composite primary liner, a geonet leakage detection layer, and a 1mm linear low density polyethylene (LLDPE) geomembrane secondary liner. Because the leachate was expected to contain high concentrations of salts and the GCL was "sandwiched" between two geomembranes without access to moisture, partial pre-hydration of the GCL with potable water was carried out prior to placement of the HDPE primary liner. A similar base liner design was used in 2003 at the Mavoco hazardous waste facility near Maputo in Mozambique. However the secondary liner consisted of a 1mm HDPE geomembrane rather than a LLDPE geomembrane. Once again, because the in-situ soil was not sufficiently clayey, a GCL was used instead of compacted low permeability soil in the composite primary liner and the GCL was partially pre-hydrated

From 2000 to 2002, four landfills with composite liner systems were constructed in Chad and Cameroon as part of a World Bank – financed pipeline construction project (GeoSyntec, 2003). The landfills were "state-of-the- practice" facilities located at pump station sites. One hazardous waste landfill cell and one non-hazardous waste landfill cell were constructed at Kome, Chad. At Belabo, Cameroon, a non-hazardous waste cell was constructed. At both sites, local labor was used extensively for installation of geosynthetic materials. In another World Bank project in Ghana from 2000 to 2005, three new sanitary landfills were constructed in the cities of Kumasi, Sekondi-

Takoradi, and Tamale. The Kumasi landfill has a geomembrane – compacted low permeability soil liner. However, the other two landfills did not employ geomembrane barriers.

In Asian and Pacific countries the use of geosynthetics in landfill barrier systems has increased markedly in the last 15 years and the use of geomembrane liner systems is becoming commonplace. In Japan, Taiwan, China, Hong Kong, Australia and New Zealand liner system design and construction is similar to other industrialized countries. Landfill designers in Japan, New Zealand, and Taiwan can face significant seismic design challenges, with free field peak ground accelerations for seismic design in excess of 0.5g in some areas.

The sophisticated landfills developed in Hong Kong in the 1990s (SENT, WENT, NENT) make extensive use of geosynthetics, including primary geomembranes and GCLs as barriers, geonets as leachate collection and drainage layers, purpose-made reinforcement, and geotextile protection layers. One innovative use of geosynthetics as part of the barrier system in Hong Kong is as part of a steep rock slope liner system in which a single-sided textured geomembrane draped over the rock slope relies on an "engineered" slip plane created by placing a geotextile on top of the smooth top surface of the geomembrane to minimize tensile stresses in the geomembrane and thus maintain its integrity.

The degree of sophistication of geosynthetic barrier systems in Asian countries is often a reflection not only of local physical conditions but also of market factors. Table 1 presents a collection of liner projects constructed in the Asia/Pacific region over the past 15 years that employed GCLs, illustrating the range in sophistication of liner systems employed in the region.

As shown in Table 1, the range of designs being applied in Asia include landfills with just a GCL liner as the low permeability advective transport barrier to landfills with geomembrane/GCL composite barriers in single and, for hazardous waste, double liner configurations. In many cases the more sophisticated systems have been promulgated by private sector project proponents and exceed regulatory minimum standards. GCLs may be particularly suitable for use in developing countries due to the ability to install them properly using unskilled labor and a minimum amount of equipment.

It is clear that in many developing (and developed) Asian countries rapid changes are occurring in landfill design practice due to increasing regulatory pressure coupled with the sheer growth in waste volumes and, in some cases, other factors such as non-differentiation of waste types within the waste stream. An inevitable outcome of the change from traditional methods of waste disposal in developing countries (e.g., open dumping) to engineered landfill facilities employing geosynthetics in barrier systems is an increase in the cost of developing landfill airspace. In some cases, where waste service provision historically has been on a "least cost" basis, a significant period of time is required for market adjustment to the cost of such improvements and landfill design and construction standards will move gradually towards the standards used in industrialized countries. For example, Malaysia is undergoing a rapid upgrading of landfill facilities and its waste infrastructure in general. In recent years new landfills being developed in Malaysia have utilized geomembrane liner systems, sometimes enhanced with GCLs to create a composite liner in critical areas – typically under and around key leachate drainage components. Siting constraints associated with new facilities, particularly in densely populated countries such as the Philippines and Vietnam, mean that sites are often chosen based upon non-technical criteria such as land availability and ease of access. This in turn means that the use of geosynthetic barrier components will be increasingly required to provide environmental protection at sites that are inherently less suited technically for waste disposal than might otherwise have been the case.

Table1. Examples of Asia / Pacific landfills employing geosynthetic barriers in the liner system

Location	Date	Waste	Barrier Layer(s)
Newcastle, Australia	1997	MSW	GM/GCL
Brisbane, Australia	1997	MSW	GM/GCL
Beijing, China	2003	MSW	GM/GCL
Guangdong, China	2003	MSW	GM/GCL
Hong Kong (WENT)	1993	MSW	GM/GCL
Hong Kong (SENT)	1994	MSW	GM/GCL
Hong Kong (NENT)	1995	MSW	GM/GCL
Ibarraj, Japan	1998	MSW ash	GM/GCL/GM
Takasaki, Japan	2000	MSW ash	GM/GCL
Jeonrabuk-Do, Korea	1995	Hazardous	GM/GCL
Pohang, Korea	1998	MSW	GM/GCL
Malacca, Maylasia	1996	MSW	GCL
Kuching, Maylasia	1999	Leachate	GCL
Blenheim, New Zealand	1998	MSW	GCL
Kate Valley, New Zeal.	2003	MSW	GM/GCL/GM
Kaoshiung, Taiwan	1998	MSW ash	GM/GCL
Kang San, Taiwan	1998	Hazardous	GM/GCL Dbl.
SinFu, Taiwan	2002	MSW	GM/GCL
Ratchaburi, Thailand	1996	Hazardous	GM/GCL Dbl
Rayong, Thailand	2002	Industrial	GM/GCL

GM = geomembrane, GCL = geosynthetic clay liner

2.2 *Cover Systems*

In general, the use of geosynthetic barriers in landfill final cover systems has lagged behind their use in base liner systems. None of the countries reviewed in the 2004 CIWMB study (GeoSyntec, 2004) require geosynthetic barrier systems for MSW landfill final covers. GCLs are being considered with increasing frequency for use as the barrier layer in caps, particularly in Europe. However, in recent years in the United States geosynthetic barrier elements in final cover systems may have actually lost ground to soil barriers with respect to their use in MSW landfill covers due to increased acceptance of evapotranspirative final cover systems in arid and semi-arid climates (ACAP, 2001; Kavazanjian, 2001), performance concerns with geosynthetic barrier layers in covers, and a change in philosophy of MSW containment (from waste entombment to allowing the waste to breathe).

Performance concerns with geosynthetic cover systems include degradation of GCL hydraulic conductivity due to chemical compatibility effects, cyclic drying and wetting and freeze-thaw, degradation of the shear strength of reinforced GCLs due to fiber degradation, and lateral gas migration and groundwater impacts induced by geosynthetic covers. The gas migration and groundwater impact concerns associated with geosynthetic covers is, ironically, due to their excellent gas containment characteristics: gas that used to diffuse to the atmosphere through the top of the landfill is forced laterally and downward (towards groundwater). Concern over the effectiveness of geosynthetic covers is also associated with the general change in philosophy in some jurisdictions from use of gas- and liquid-tight cover systems for MSW landfills to cover systems that allow the waste to "breathe". This change in philosophy is probably one of the major reasons that geosynthetic barriers are not used as widely in cover systems as they are in liner systems for MSW landfills.

Geosynthetic barriers are still the preferred barrier in final covers for hazardous waste landfills and for capping of uncontrolled hazardous waste dumps and other sites where control of infiltration is paramount. As for landfill liner systems, a composite barrier composed of a geomembrane overlying a low permeability soil layer is generally recognized as the preferred cover system when infiltration must be minimized. Figure 2 illustrates the composite cover system mandated by the US EPA under the Resource Conservation and Recovery Act (RCRA) for final covers at hazardous waste sites and also employed for capping of uncontrolled waste dumps. GCLs are frequently used instead of the clay layer shown in Figure 2 as the low permeability soil layer in these composite cover systems.

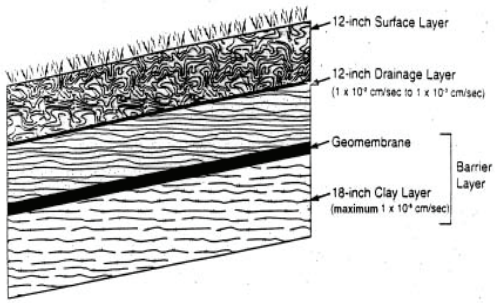


Figure 2. US EPA composite cover system (EPA, 1999)

While the use of composite geomembrane/low permeability soil or geomembrane/GCL caps on MSW landfills is still relatively rare, single-layer geomembrane and GCL caps are used with some regularity at MSW facilities, particularly in wetter climates where evapotranspirative covers are not an accepted alternative. In some cases, older-style "dumps" are being capped with geomembranes to enhance LFG generation and capture as a basis for project development as part of compliance with the Kyoto protocol. Furthermore, exposed geomembranes, discussed in detail by Bouazza et al. (2002), are being used with increasing frequency for projects where a service life of less than 20 years is acceptable and aesthetics of the cover is not a major concern (Reinhart et al., 2002; Thiel et al., 2003).

GCLs are being used with increasing frequency as the barrier layer in MSW landfill covers in the USA and Europe as questions concerning durability (discussed subsequently) have become addressed. GCLs are often preferred over geomembranes or compacted clay barriers due to the ease of installation and performance concerns with the other types of barriers. Kavazanjian and Dobrowolski (2003) describes a case in which a GCL was selected as the preferred final cover barrier after a clay barrier was rejected due to concerns over desiccation, an exposed geomembrane cover was rejected on aesthetic grounds and due to concerns over induced lateral gas migration, and an evapotranspirative cover was considered acceptable on an infiltration control basis. GCL covers are also being employed in the Asia / Pacific region. Table 2 identifies several recent GCL cover projects in the Asia / Pacific region.

Table 2. Selected GCL Cover Projects in the Asia / Pacific Region

Location	Date	Waste
Melbourne, Australia	1997	Contaminated soil
Miyagi, Japan	2002	MSW
Lorong Halus, Singapore	1995	MSW
Kaoshuing, Taiwan	1998	MSW

In South Africa, recent proposed changes to the "Minimum Requirements" for landfill cover systems require the mandatory use of a GCL in the landfill cover for all large MSW leachate generating sites, as well as at hazardous waste landfills and at disposal sites with inadequate base liners. Although there is significant opposition to these proposed changes on account of cost, an old industrial waste site near Johannesburg was rehabilitated in 2004 by capping with a GCL sandwiched between two layers of sand. The authorities insisted on the use of the GCL at this site because the landfill was unlined and because there was uncertainty over the extent of historical hazardous waste disposal at the site.

2.3 *Coastal Landfills*

Waste disposal in coastal landfills is emerging as an important topic in countries which have little available inland space. For instance, in Japan, coastal landfills and the associated containment systems are important considerations, particularly for metropolitan areas such as Tokyo and Osaka, due to limitations on inland space available for waste disposal. In addition, subsequent beneficial use of closed coastal landfills is an important engineering consideration, since these coastal landfills are constructed close to metropolitan areas and can provide new land for development. Coastal landfills may be preferable to inland landfills for several reasons, including (1) the large area required for an inland landfill may not be available or may be prohibitively expensive, (2) the groundwater beneath a typical coastal site may not be suitable for beneficial uses, (3) many coastal sites are isolated, with few inhabitants nearby (or fewer than inland sites), and (4) environmental concerns related to coastal landfills may be less controversial compared to inland landfills. Disadvantages of coastal landfills include difficulties in monitoring and the possibility for bio-accumulation of toxic substances. Therefore, containment systems for coastal landfills are equally important to those for inland landfills.

The Japan Ministry of Transport (transformed into the Ministry of Land, Infrastructure, and Transport in January 2001), which regulates issues related to ports, harbors, and coastal development, has published guidelines for containment systems in coastal waste landfill sites. Coastal areas often have thick layers of cohesive sediments that satisfy the requirements for base containment systems. Thus, design, construction, and performance of the lateral barriers are often the critical issues for a coastal landfill containment system. At coastal landfill sites when the native subgrade materials do not meet regulatory standards for waste containment, geomembranes may also be used as bottom barriers. Typical lateral barrier systems used in coastal landfill sites are shown in Figure 3. Japanese regulations for these lateral containment systems require a composite barrier system composed of two geomembranes, or alternative barrier materials, with some sort of stabilized backfill material placed in between (WAVE, 2000). The gravity caisson quay wall system shown in Figure 3(a) and the rubble mound quay wall systems shown in Figure 3(c) both employ two geomembranes separated by stabilized backfill materials that serve as a cushion. The backfill may also function as part of the barrier system if it has low hydraulic conductivity and sufficient ductility to resist service loads.

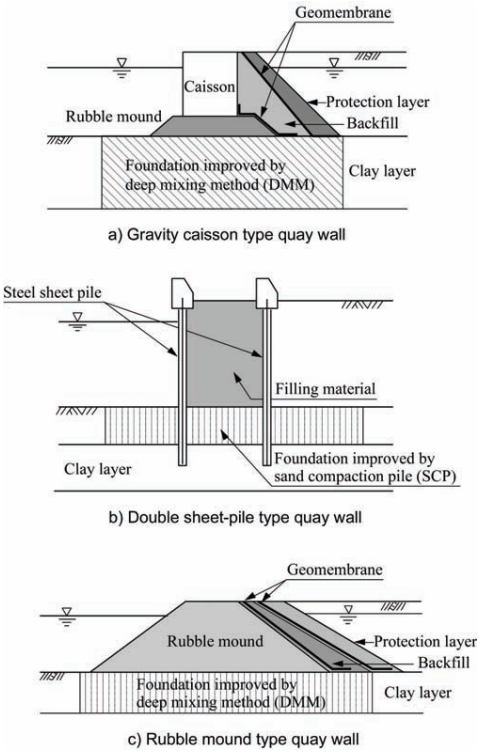


Figure 3. Side barrier systems for Japanese coastal landfills

3 GEOSYNTHETIC BARRIER FIELD PERFORMANCE DATA

One of the most important studies on the field performance of geosynthetic barrier systems is a study completed for the US EPA in 2002 (EPA, 2002). This study provided data from monitoring of collection rates from the liquid collection layer beneath the primary liner from 187 individual cells at 54 landfills in the United States for periods of up to 10 years. Among the important findings drawn by

the investigators from this study were that 1) high quality construction quality assurance dramatically reduces the leakage rate through geosynthetic liner systems; and 2) groundwater contamination from leakage through a composite liner had not been detected at any of the landfills in the study. Analysis of the data from the study also provides important information about the effectiveness of GCLs as an alternative to compacted soil for the low permeability soil component of a composite liner system. Table 3 compares the liquid collection rates for geomembrane – compacted clay liner (GM-CCL) composite systems overlain by sand liquid collection layers to the rates for geomembrane-GCL (GM/GCL) composite systems overlain by sand liquid collection layers. Data on average, minimum, and maximum flow rates is provided for the initial, active, and post-closure periods. Collection rates in all three categories for all three periods for the GM-GCL systems were generally equal to or less than those for the GM-CCL systems and the rates are dramatically lower (typically by an order of magnitude) during the active life and post-closure period for the GM-GCL systems compared to the GM-CCL systems. Similar data was reported for GM-GCL systems compared to GM-CCL systems when a geonet liquid collection layer was used.

Table 3. Liquid collection rates for double liner systems (liters per hectare per day) (from EPA, 2002)

Stage	Initial	Active	Post-Closure
	Sand/Geomembrane/GCL		
Average Flow	132.5	22.5	0.3
Minimum Flow	0.0	0.0	0.0
Maximum Flow	984.2	283.9	0.9
	Sand/Geomembrane/Compacted Clay		
Average Flow	113.6	141.9	64.4
Minimum Flow	1.2	22.7	0.0
Maximum Flow	1192.4	671.9	274.4

In December 2003, the California Integrated Waste Management Board released a summary of the environmental performance of 223 MSW landfill in California for the period from 1998-2001 (GeoSyntec, 2003). The 223 landfills included 16 landfills with base liners employing geomembrane barriers over the entire waste footprint and another 70 landfills with footprints partially-lined with geomembrane. Six of the 16 landfills that were completely lined with geomembrane barrier systems were reported to be in "corrective action," indicating that monitoring systems detected a release of contaminants from the waste mass. In five of these six cases, the release was clearly related to landfill gas, while in the sixth case the cause of the release was undetermined but was suspected to be related to "construction issues".

Kavazanjian and Corcoran (2002) discuss the occurrence of landfill gas impacts at MSW landfills with geomembrane liner systems. Improper attention to detail with respect to termination of the side slope leachate collection layer can create a pathway for landfill gas to bypass the liner system and infiltrate the subsurface, as illustrated in Figure 4. There are a number of means by which this situation can be mitigated, including placing the leachate collection system under vacuum, placing gas collection trenches in the waste adjacent to the side slope (taking care not to draw too much oxygen onto the system), and/or modifying the termination detail for the side slope leachate collection layer.

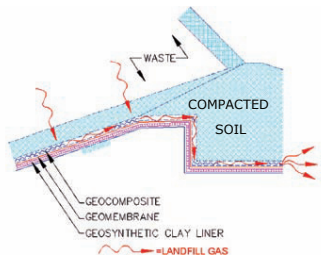


Figure 4. Landfill gas migration pathway around geosynthetic liner system (Kavazanjian and Corcoran, 2002)

Another important source of field information on geomembrane integrity is electrical leak detection surveys. These surveys are generally conducted after placement of the leachate collection gravel on top of the primary geomembrane and can be a powerful addition to CQA programs. There are a variety of factors that can affect the results of an electrical leak detection survey, but if properly performed they are capable of detecting all but the very smallest defects in a geomembrane liner (Hruby and Barrie, 2003). Forget et al. (2005) summarize 10 years of leak detection surveys on geomembranes. These investigators found a leak density of approximately 0.5 leaks/hectare for covered geomembranes installed under strict a construction quality assurance (CQA) program compared to a leak density of 16 leaks per hectare in the absence of a CQA program. Hruby and Barrie (2003) report an average defect rate of 11 defects per hectare from surveys at 276 sites with total lined area exceeding 3,000,000 m^2, but provide no information on CQA at these sites.

The leak density of 0.5 leaks/hectare reported by Forget et al. (2005) for "strict" CQA is well below the leak density typically assumed for evaluation of the effectiveness of geomembrane liner systems. For instance, the US EPA HELP program (Schroeder et

al., 1994) assumes a default leak density of 2.5 leaks per hectare for "excellent" installation quality and from 2.5 to 10 leaks/hectare for "good" installation quality. Dwyer (1998) intentionally introduced 8 defects into his 0.13 hectare geomembrane test section (over 60 defects per hectare) to simulate "typical" field conditions in his comparative study of geosynthetic and soil covers. The difference between the leak density of 0.5 leaks/hectare reported by Forget et al. (2005) from field leak detection surveys and the leak density of approximately 60 leaks/hectare employed by Dwyer (1998) is particularly significant as the Dwyer study is often cited as a basis for concluding that soil covers can be as effective as geosynthetic covers in limiting percolation through final covers. Other important findings from the Forget et al. (2005) study were that there was no correlation between the number of leaks and the geomembrane thickness and that a CQA program reduced the number of tears and faulty seams by a factor of approximately 6 and the number of punctures by approximately 21 for a 1 mm geomembrane.

Industrialized countries routinely require comprehensive CQA programs for geosynthetic barrier construction. CQA requirements are also being adopted in developing countries, though there remains some opposition to mandating them due to cost considerations. For instance, proposed changes to the South African "Minimum Requirements" for hazardous waste landfill construction mandate a CQA program for the lining system implemented by a suitably qualified independent third party.

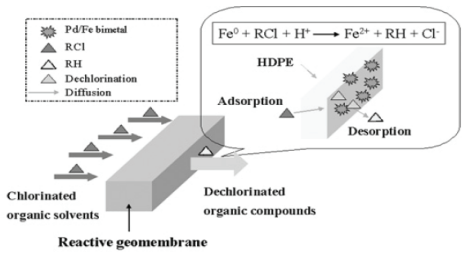


Figure 5. Reactive Geomembrane (Shin et al., 2005)

4 DIFFUSIVE FLUX

4.1 *Geomembranes*

As discussed by Bouazza et al. (2002), the primary function of a geomembrane barrier is to control advective flux of liquid and gas. Due to their thin profile, geomembrane barriers are generally not relied upon for control of diffusive flux of chemical species across the barrier. This is particularly true for organic compounds because of the relatively high value of HDPE's diffusion coefficient for organic compounds. However, Shin et al. (2005) report on the use of a "reactive" geomembrane to reduce the diffusive flux of chlorinated organic solvents, as illustrated in Figure 5.

Shin et al. (2005) coated the reactive geomembrane with a Palladium / Iron (Pd/Fe) compound known to be an effective for degradation (dechlorination) of chlorinated organic compounds such as TCE and PCE (Shin et al., 2004). These investigators report that when tested in a "double-compartment apparatus," the Pd/Fe coated geomembrane effectively halted diffusive flux of a 100 mg/L TCE solution for a period of at least 40 days and that the concentration of TCE in the loading chamber was "completely degraded." In the control experiment with an untreated geomembrane, a concentration of 100 mg/l TCE in the loading chamber resulted in a concentration of 10 mg/l of TCE in the collection chamber after 30 days. Similar results were reported for PCE. However, no details are given on the type or thickness of geomembrane or on the volumes of the loading and control chambers in the double-compartment apparatus.

Sangam and Rowe (2005) looked at the influence of surface fluorination on diffusion of organic contaminants through an HDPE membrane. They found that in laboratory tests a 4 μm fluorinated layer on the surface of a 1.5 mm HDPE geomembrane had little effect on the partition parameter for most organic contaminants but decreased the diffusion coefficient by a factor of 1.4 to 1.5 compared to an untreated geomembrane. The net result of fluorination on diffusive flux of organic contaminants was roughly equated to placing an additional 0.4 to 0.9 m of clay beneath the geomembrane. They also note that increasing the thickness of the fluorinated layer would likely improve the performance of the treated geomembrane and that field trials are required to evaluate the effect of installation and other field conditions on the effectiveness of a fluorinated geomembrane on reducing the diffusion of organic contaminants.

The use of forced air to reduce the concentration of volatile organic chemicals above the geomembrane has also been proposed as means of minimizing diffusion across geomembrane barriers. Figure 6 shows a proposed diffusion control system for a double liner system that employs a geomembrane-GCL composite system as the primary liner and a single geomembrane as the secondary liner (Aquatan, 2004). Forced air is circulated through the leak detection layer between the primary and secondary liner to minimize the concentration gradient, and thereby minimize chemical diffusion, of volatile organic compounds

across the secondary liner. An additional feature of this system as proposed by the developer is the use of "moist" (saturated) air in the forced air circulation system to maintain hydration of the GCL.

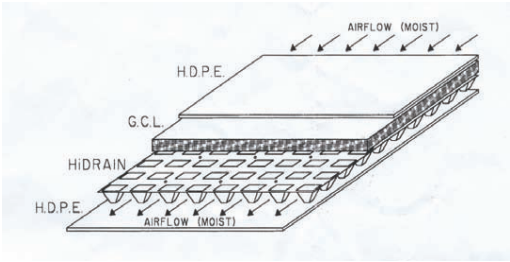


Figure 6. Diffusion control for a double liner system (Aquatan, 2004)

4.2 *GCLs*

GCLs are not generally relied upon as barriers to diffusive flux. As discussed by Rowe (2005), a compacted soil "attenuation layer" can be placed beneath a GCL employed in a landfill liner system to provide an engineered barrier to diffusive flux. However, in many cases natural geologic features may be relied upon to provide adequate diffusive flux control if they provide sufficient separation between the liner and the nearest "sensitive receptor". If diffusive flux is a concern in a GCL cover, a compacted soil foundation layer can serve as the attenuation layer.

5 GEOSYNTHETIC BARRIER DURABILITY

5.1 *Geomembrane Aging*

The impact of aging on the service life of a high density polyethylene (HDPE) geomembrane is an important consideration in design of waste containment systems. Bouazza et al. (2002) note the dependence of the service life of HDPE on temperature. Summarizing the work of Rowe and Sangam (2002), they state that at 15°C, a primary geomembrane should last about 200 years in a typical landfill liner (i.e. buried) application while at 33°C the service life is on the order of 70 years. Tarnowski at al. (2005) report on the service life of exposed and buried HDPE geomembranes estimated from testing on a) four different exposed geomembranes after from 20 to 31 years of service life, and b) 2.5 mm HDPE geomembrane specimens stored in water for 6 years and in air for 13 years, both at 80°C. The exposed geomembrane with the longest exposure time was from an impoundment constructed in 1974 in Galing, Germany, for a zinc and lead production process sludge with a pH between 2 and 4 and a water content of 50-60%. Table 4 shows results from tests conducted on specimens recovered from the exposed geomembrane from this impoundment (Galing I) in 1977, 1994, and 2005.

Table 4. Physical properties of the Galing I exposed geomembrane (Tarnowski, et al., 2005)

Properties	Test method	Unit	Results		
Year			1977	1994	2005
Density	DIN 53479-A	g/cm^3	0.962	0.962	0.968
MFR 190/5	ISO 1133	g/10 min	0.23	0.37	0.29
Stress at yield	DIN EN ISO 527-3				
MD	PK-5 (100	N/mm^2		27.3	28.4
CMD	mm/min)	N/mm^2	25.1	27.6	27.0
Elongation at yield	DIN EN ISO 527-3				
MD	PK-5 (100	%		9.4	10.1
CMD	mm/min)	%	12	8.8	9.3
Stress at break	DIN EN ISO 527-3				
MD	PK-5 (100	N/mm^2		6.3	6
CMD	mm/min)	N/mm^2		9.7	9
Elongation at break	DIN EN ISO 527-3				
MD	PK-5 (100	%	143	165	137
CMD	mm/min)	%		75	35
OIT at	DIN EN				
210°C	728	min		2.7	
200°C		min		6.4	
190°C		min		17.4	5

The primary factor affecting the degradation of HDPE is depletion of anti-oxidant compounds in the polymer. Anti-oxidant content is often quantified by the oxidation induction time (OIT) of the polymer. While the results of the tests in Table 4 indicate that, except for elongation at break, the relevant mechanical properties of the geomembrane had not changed significantly, there had been a significant reduction OIT, with the OIT value at 190°C in 2005, after 30 years of exposure, equal to only 5 minutes (versus 17.4 minutes in 1994). Additional testing on this geomembrane in 2005 indicated that no antioxidants remained in the upper one-third of the geomembrane. Similar testing on a second sludge impoundment built in Galing in 1984 (Galing II) showed similar results: no significant change in mechanical properties of interest but significant depletion of antioxidants, as shown in Figure 7. Based upon tests conducted on specimens recovered from the exposed geomembranes from the four

projects, Tarnowski et al. (2005) concluded that, for exposed 2.5 mm geomembranes "service life expectancy of more than 50 years can be forecasted for high-quality HDPE- geomembranes."

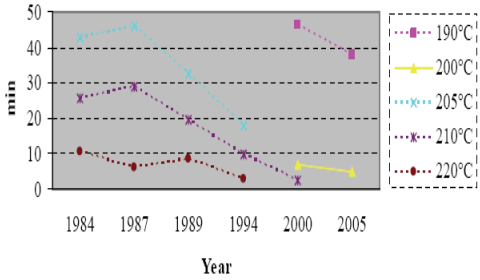


Figure 7. OIT values (DIN EN 728) for the exposed geomembrane at Galing II (Tarnowski et al., 2005)

Table 5. Estimated times for the three stages of HDPE geomembrane degradation (after Rowe, 2005)

Temp. C°	Stage 1 yrs Simulated	Stage 2 yrs Adjusted	Stage 3 yrs Adjusted	Service Life yrs Adjusted
10	280	30	1380	1690
20	115	10	440	565
20	50	4	150	205
25	35	2	90	130
30	25	1	55	80
50	10	0.6	20	35
60	6	0.3	9	15

Based upon the rate of ageing of the geomembranes stored in water, Tarnowski et al. (2005) state that the calculated service life of a buried 2.5 mm HDPE geomembrane at 20°C is conservatively estimated at 300 years, may well be in excess of 1000 years if the high activation energy of highly molecular HDPE stabilizers is taken into account, and may even be as great as 5000 years and conclude that "service life expectancy of properly produced and installed HDPE geomembranes with a "sufficient" thickness of at least 2.0 mm in buried applications is more than 500 years." However, Rowe (2005) notes that HDPE service life (in particular, anti-oxidant depletion) depends significantly on temperature and that, due to the presence of transition metals in leachate, geomembranes should age faster in leachate than in water. Rowe provided estimates of the service life of buried geomembranes exposed to leachate to be on the order of 200 years in a 20°C environment and as little as 15 years in a 60°C environment. Rowe estimated the duration of the three stages of geomembrane degradation: antioxidant depletion (Stage 1), induction (Stage 2), and degradation (Stage 3) as a function of temperature. Rowe's estimates are shown in Table 5, along with his estimate of the total service life. The adjusted Stage 2 and Stage 3 times in Table 5 account for accelerated degradation due to exposure to leachate.

The estimates in Table 5 are based upon a constant temperature. Rowe also presents estimates based upon idealized temperature time histories for a landfill geomembrane, resulting in antioxidant depletion times (Stage 1 of degradation) ranging from 150 years for a landfill with an ambient liner temperature of 10°C and a peak liner temperature of 35°C to less than 20 years for a landfill with an ambient liner temperature of 20°C and a peak liner temperature of 45°C. Considering that a typical liner service temperature for a "dry" landfill is approximately 35°C (Koerner and Koerner, 2006), Rowe concludes it appears that "the service life for HDPE GM in MSW landfill is likely about 160 years for a primary liner at 35°C and greater than 600 years for a secondary GM provided it is at a temperature of less than 20°C (this will be achieved only if there is an adequate thickness of soil between the primary and secondary systems to provide an adequate thermal barrier)."

5.2 *Geosynthetic Clay Liner Service Life*

5.2.1 *Degradation Mechanisms*

Bouazza et al. (2002) discuss potential degradation mechanisms for GCLs in landfill applications, including chemical degradation due to cation exchange and other chemical compatibility effects, punctures, bentonite thinning, and internal erosion. Hsuan and Koerner (2002) point out in a needlepunch reinforced GCL, degradation of the fibers leading to shear strength degradation must also be considered (degradation of GCL shear strength is addressed subsequently, in Section 5.3).

5.2.2 *Chemical Degradation*

Degradation of the performance of GCLs due to chemical compatibility effects continues to be an area of active research. Research on chemical compatibility of GCLs can be divided into three categories: (1) experiments on the effect of cation exchange and of chemical solutions and waste leachate on long-term saturated hydraulic conductivity and other chemical transport parameters, (2) understanding of the osmotic behavior of GCLs, and (3) development of modified bentonite for use in GCLs. Investigations on the effect of cation exchange and inorganic solutions on the saturated hydraulic conductivity of GCLs have included Jo et al. (2001), Egloffstein (2002), Kolstad et al. (2004a), Jo et al. (2004, 2005), Lee and Shackelford (2005a), Lee et al. (2005), Katsumi and Fukagawa (2005), and Katsumi et al. (2006b). Investigations of the impact of organic chemical solutions (including leachate) on GCL saturated

hydraulic conductivity include Lake and Rowe (2004, 2005), Rowe et al. (2005a, 2005b) Gaidi and Alimi-Ichola (2002), Guyonnet et al. (2005), Shan and Lai (2002,) Thiel and Criley (2005), and Katsumi and Fukagawa (2005).

Jo et al. (2001) investigated the effects of the concentration and valance of cations in the permeant on hydraulic conductivity and found that the saturated hydraulic conductivity was correlated to the volumetric swell of the GCL bentonite. While the investigation conducted by Jo et al. (2001) was limited to single species, Kolstad et al. (2004a) and Katsumi and Fukagawa (2005) investigated the saturated hydraulic conductivity of GCLs permeated with multi-species inorganic chemical solutions as well as with the waste leachate. These investigators found that the saturated hydraulic conductivity of the GCL was related to both the swell volume of the bentonite and the electrical conductivity of permeant. Figure 8, from Katsumi and Fukagawa (2005) shows the relationship between the saturated hydraulic conductivity of a GCL and the swell volume of the bentonite.

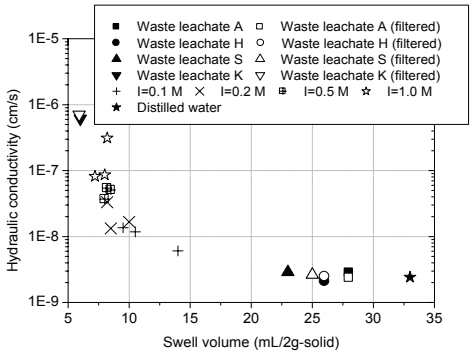


Figure 8. Swell volume of bentonite versus the hydraulic conductivity of a GCL permeated with waste leachate and NaCl-$CaCl_2$ solutions (Katsumi and Fukagawa 2005)

Egloffstein (2002) studied the influence of electrolytic concentration, ion exchange, and ion exchange with simultaneous partial desiccation on the saturated hydraulic conductivity of GCLs. Figure 9 shows the evolution of the saturated hydraulic conductivity of GCLs in tests performed by Egloffstein. Series (a) shows in the self-sealing behavior of a "medium-heavy" sodium bentonite GCL (a sodium-bentonite GCL with 4000 to 5000 g/m^2) after ion exchange and partial desiccation from tests conducted at two different normal stresses upon exhumed (excavated) samples. Series (b) shows medium-heavy and heavy (8000 g/m^2) sodium GCL behavior after "radical" ion exchange due to initial hydration with a 0.3 mol $CaCl_2$ salt solution. Series (c) shows the behavior of a heavy calcium GCL initially hydrated with de-ionized water and then subject to percolation with the 0.3 mol $CaCl_2$ solution. Series (d) shows the behavior of medium-heavy and heavy GCLs initially hydrated with water and then subject to "soft" ion exchange due to exposure in situ to seepage of low electrolytic content. These tests show an impact of both pre-hydration with water and bentonite content on the saturated hydraulic conductivity of the GCL.

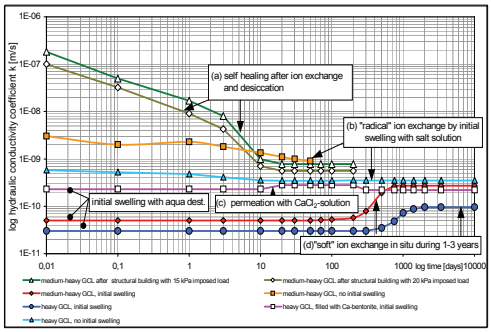


Figure 9. Evolution of GCL hydraulic conductivity with time for a hydraulic gradient 30 to 45 (Egloffstein, 2002).

Bentonite quality and microstructure are also important factors influencing the chemical compatibility of GCLs. Lee and Shackelford (2005a) and Lee et al. (2005) investigated the effect of bentonite quality on the hydraulic behavior of GCLs permeated with chemical solutions. A microstructure study conducted by Guyonnet et al. (2005) found that "gel" phase formation in hydrated bentonite is dependent on the type of permeant and sequence of permeation. While no gel phase was observed in GCL samples hydrated with a 0.1 mol/L $CaCl_2$ solutions, a gel phase appeared when the GCL was hydrated with waste leachate after the GCL samples were prehydrated with water. Importantly, they also found that gel formation had a significant effect in that it decreased the saturated hydraulic conductivity.

Various GCL test programs have investigated effects of prehydration, confining pressure, sorption, and other factors on chemical compatibility. Transport parameters, including sorption effects, for GCLs exposed to volatile organic compounds (VOCs) were reported by Lake and Rowe (2004, 2005) and Rowe et al. (2004, 2005). Studies focusing on the prehydration of GCLs were conducted by Katsumi et al. (2004), Lee and Shackelford (2005b), and Thiel and Criley (2005). Kolstad et al. (2004b) and Katsumi et al. (2006a) conducted hydraulic conductivity tests on a GCL that is prehydrated and calendared during

manufacturing and found that it is resistant to degradation from permeation with $CaCl_2$ solutions. The effect of confining pressures on GCL chemical compatibility was examined by Katsumi and Fukagawa (2005) and Thiel and Criley (2005). Katsumi and Fukagawa (2005) conducted hydraulic conductivity tests using 2.0 mol/L NaCl and 0.25 mol/L $CaCl_2$ solutions and found that the effects of the overburden pressure on the hydraulic conductivity permeated with NaCl and $CaCl_2$ solutions were significant and depended on the type of chemicals. Thiel and Criley (2005) tested prehydrated GCLs permeated with waste leachate and pure water and found that the hydraulic conductivity of the GCL appeared to be independent of the fluid chemistry at effective stresses greater than 400 to 500 kPa.

Shackelford et al. (2000) pointed out several problems that can occur in GCL chemical compatibility testing due to incomplete understanding of clay-chemical interaction during permeation. In particular, they pointed out the importance of continuing hydraulic conductivity tests long enough to achieve chemical equilibrium and proposed termination criteria for GCL hydraulic conductivity chemical compatibility tests. Hydraulic conductivity test duration may be particularly important for prehydrated GCLs permeated with dilute chemical solutions. Based on the termination criteria proposed by Shackelford et al. (2000), 2-5 years may be required to reach equilibrium for prehydrated GCLs subject to dilute chemical solutions. The long duration required for equilibrium of GCLs permeated with dilute solutions has been demonstrated in hydraulic conductivity tests conducted on non-prehydrated GCLs (Jo et al. 2004, Jo et al. 2005, Katsumi et al. 2006b) and on prehydrated GCLs (Lee and Shackelford 2005b).

Osmotic behavior of GCLs has been postulated to result in beneficial effects with respect to solute transport. Osmotic behavior results in liquid movement from a solution of lower chemical concentration (higher water activity) to one of higher concentration (lower water activity) when these solutions are separated by a membrane barrier. Transport parameters for the osmotic behavior of GCLs have been reported by Malusis and Shackelford (2002, 2004), Manassero and Dominijanni (2003), and Shackelford (2005).

The use of chemically-resistant modified bentonite GCLs is an area of active research and development. Several chemically modified organobentonites that are resistant to chemical compatibility effects have been developed (e.g., Onikata et al. 1996, Lorenzetti et al. 2005). The effect of these modifications has been attributed to the activation of osmotic swelling and an increase in sorption capacity. Lorenzetti et al. (2005) conducted an experimental study to obtain the transport parameters of benzene for benzyltriethylammonium-bentonite (BTEA-bentonite) and hexadecyltrimethylammonium-bentonite (HDTMA-bentonite). They found that while the organobentonites exhibited a greater hydraulic conductivity than natural bentonites, there was a decrease in benzene flux for BTEA-bentonite compared to natural bentonite. Another new development is multi-swellable bentonite (MSB), developed by Onikata et al. (1996 and 1999). MSB is bentonite mixed with propylene carbonate (PC) to activate the osmotic swelling capacity. The PC occupies the inter-layer of the smectite (montmorillonite) and has a strong attraction to water molecules. This results in strong swelling behavior even when the permeant contains polyvalent cations or a high concentration of monovalent cations. MSB should therefore perform better as a barrier against chemical solutions than natural bentonite. Furthermore, MSB is expected to have good long-term stability based upon the results of hydraulic conductivity tests conducted over a period of 3 years using 0.1-1.0 mol/L NaCl solutions by Katsumi et al. (2001) and Katsumi et al. (2006a). These tests suggest that MSB has a strong potential for use in GCLs in landfill bottom and side barriers. However, the long-term stability of chemically-modified GCLs must be demonstrated before they can reliably be used in practice.

5.2.3 *Physical Degradation*

Additional hydraulic conductivity degradation concerns for GCLs include degradation induced by cyclic freeze/thaw and desiccation (drying/wetting) conditions. Shan and Daniel (1991) demonstrated that the hydraulic conductivity of an unreinforced sodium GCL was unaffected by 3 cycles of wetting and drying in the laboratory. Lin and Benson (2000) showed that the hydraulic conductivity of a needlepunch reinforced sodium GCL was unaffected by 5 cycles of wetting with deionized water under a normal load of 17.5 kPa followed by air drying but that when wetted with a 0.0125-mol solution of $CaCl_2$ the saturated hydraulic conductivity degraded by at least two orders of magnitude after 5 such cycles when tap water was used and 7 such cycles when deionized water was used. Sporer & Gartung (2002b) carried out a series of wet-dry cycles on calcium bentonite GCLs. The capability of the calcium GCL to absorb water declined after only one wet-dry cycle, suggesting that even a single desiccation event could have an extremely pronounced negative effect on the hydraulic conductivity of a calcium bentonite GCL.

Kraus et al. (1997) found little change in the hydraulic conductivity of unreinforced and reinforced sodium GCLs after 20 cycles of freezing and thawing in the laboratory, as shown in Figure 10, suggesting that even if a GCL is not protected by

sufficient soil cover it is not subject to degradation by freeze/thaw cycles.

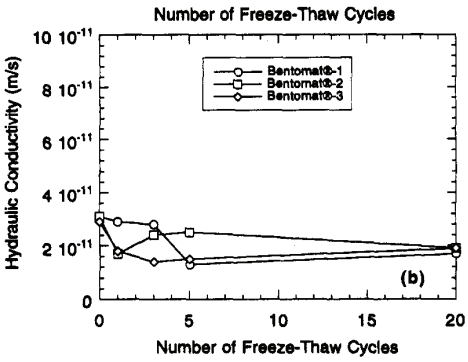


Figure 10. Influence of freeze-thaw on saturated hydraulic conductivity of GCLs (Kraus et al., 1997)

Della Porta et al. (2005) looked at the influence of both wetting and drying and freezing and thawing on the behavior of both bentonite and a needlepunch reinforced GCL. When subjected to a drying temperature of 105°C, the index flux of the GCL degraded (increased) with each cycle of wetting and drying, ultimately degrading by a factor of approximately 4 after 8 cycles. However, when the drying temperature was 60°C, there was no change in the properties of the GCL through four cycles of wetting and drying (after four cycles at 105°C, the index flux of the GCL increased by a factor of 2). The properties of the bentonite were initially unaffected by wetting and drying at 105°C, but after the fifth cycle the swelling capacity decreased slightly and the index flux increased correspondingly. In the freeze/thaw tests, essentially no change was observed after 8 cycles of freezing and thawing for a cycle time of 24 hours. However, when the cycle time was increased to 72 hours, the index flux of the GCL increased by a factor of 4 after four cycles of freeze / thaw.

Melchior (2002) evaluated GCLs exhumed from a landfill cover test site in Hamburg-Georgswerder. In 1994, two lysimeters with an area of 100 m² each were integrated into the landfill cover to measure the leakage through needlepunched and stitch-bonded GCLs. These GCLs contained only 3500 g/m² sodium bentonite, each, and were overlain by only 30 cm of topsoil (sandy loam) and a 15 cm drainage layer (gravel). In 1995, 1996, 1998 and 1999 samples of the GCLs were recovered and evaluated visually and by chemical, physical and mineralogical laboratory tests. The samples were x-rayed and specimens were prepared for thin section and electron microscopic imaging. Figure 11 shows desiccation cracking of one of the GCLs as revealed by x-ray imaging. The results of the laboratory tests showed that under the shallow soil cover of the test section desiccation, cation exchange, plant root penetration and shrinkage increased the saturated hydraulic conductivity of GCLs significantly. Furthermore, re-wetting and swelling of the bentonite in the laboratory did not restore the saturated hydraulic conductivity of the GCLs.

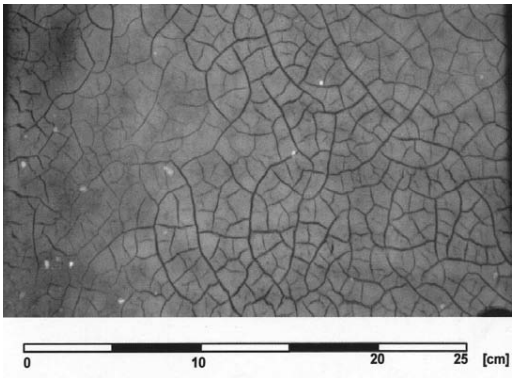


Figure 11. X-ray image showing desiccation cracks in the bottom layer of a GCL overlap from the test section at Hamburg-Georgswerder (Melchior, 2002)

The degradation of the GCLs was attributed to both desiccation and root penetration. The desiccation was the result of both the upward gradient of water induced by suction in the overlying soil and direct water removal through plant roots (i.e. transpiration). Ion exchange in the bentonite from sodium to calcium (confirmed by the laboratory tests), which leads to a loss of swelling capacity and to changes in the micro structure of the bentonite, was suggested as a primary reason the bentonite did not "self-heal" upon re-swelling. Melchior (2002) concluded that the use of GCLs in capping systems could be problematical if they were not adequately protected against desiccation.

The GCLs in the Hamburg-Georgswerder test section evaluated by Melchior had relatively shallow soil cover. Mansour (2001) showed that the saturated hydraulic conductivity of a needle-punch reinforced 4500 g/m² sodium GCL buried under 0.66 m of vegetated cover soil was unaffected after five years of exposure in a relatively extreme semi-arid climate at a site in Bakersfield, California, USA. The Bakersfield site had an annual precipitation (PPT) of less than 150 mm each year and annual potential evapotranspiration (PET) in excess of 1500 mm each year, as shown in Figure 12, and summertime temperatures routinely over 40°C.

Blümel et al. (2002) performed lysimeter field tests on GCLs in a humid climate. GCLs filled with

4500 g/m² to 5500 g/m² of sodium bentonite, covered by a layer of silty sand of about one meter in thickness and exposed to humid climate conditions in northern Germany, showed percolation rates of only 3 to 5 mm/year after three years of exposure. The percolation was less than 0.5 % of the annual precipitation. Observations over the three-year period showed no significant evidence that the reduction of water content in the bentonite of the GCL during the summer caused desiccation cracking or other notable defects in the GCL. Mineralogical testing on exhumed GCL samples showed that the transition from sodium to calcium bentonite due to cation exchange was essentially completed during the three year observation period but that this ion exchange processes did not cause any significant increase in permeation.

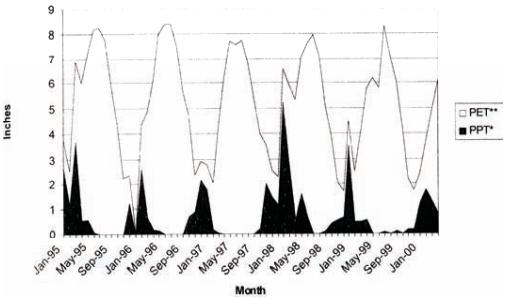


Figure 12. Climate conditions at the Bakersfield, California GCL test site (Mansour, 2001)

Henken-Mellies et al. (2002) studied the behavior of a calcium bentonite GCL in a landfill cover system 60 km southwest of Nuremberg, Germany. A 520 m² lysimeter collected and measured the relevant water fluxes. The water balance measured during a 3-year monitoring period showed that evaporation accounted for almost 70% of the water flux, 30% of the water was drained off laterally in the drainage geocomposite on top of the GCL, and only 0.5% of the precipitation seeped through the GCL (Figure 13). In-situ-measurements of moisture content of the bentonite within the GCL showed seasonal changes, indicating the 1 m thick recultivation (vegetation) layer did not completely shield the GCL from desiccation stresses. However, despite these internal moisture content changes, the GCL proved to be an effective sealing layer within the landfill cover system during the 3-years observation period.

Sporer (2002) reported on sodium bentonite GCLs excavated from the cover systems at 8 landfills. The height of cover soil on top of the GCLs was between 0.5 and 1.0 m. The covers were 3 to 6 years old and all of the GCLs had been converted by cation exchange into calcium bentonite GCLs. The swell volume amounted to between 7 to 10 ml and the water absorption capacity of the GCLs lay between 200 and 250%. GCL permittivity measured in fixed wall cells varied from $5 \cdot 10^{-8}$ s^{-1} and $1 \cdot 10^{-7}$ s^{-1}. X-ray imaging showed no evidence of tears or cracking in the GCLs.

Heerten and Maubeuge (1997) reported on excavation of GCLs at three landfills. The GCLs were subject to between 0.6 and 1 m of cover and had also undergone cation exchange. The results of triaxial and fixed wall tests gave permittivities between 10^{-9} s^{-1} and 10^{-8} s^{-1}. Heerten (2004) described excavation of a GCL after 5 years in service at a landfill at a very location (mean yearly total precipitation 500 mm/a) in the autumn 2003, after an extremely hot summer. The recultivation layer measured 140 cm. Cation exchange was also complete in this GCL and the permittivity was measured as $1.2 \cdot 10^{-8}$ s^{-1}. No desiccation was observed despite the hot summer. The moisture content of the bentonite was approximately 125%.

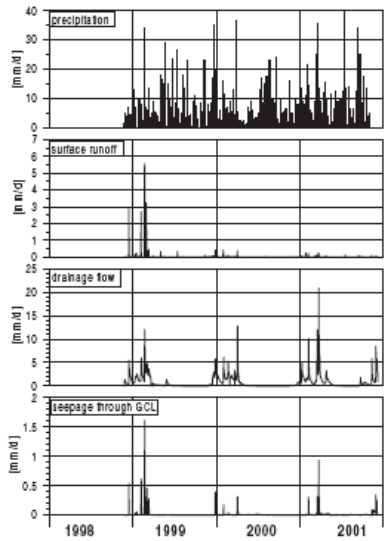


Figure 13. Water balance measurements from Nuremberg GCL test section (Henken-Mellies et al. 2002).

Sivakumar Babu et al. (2002) investigated the onset of cracking in the bentonite of selected GCLs. Suction pressures in bentonitic materials used in the GCLs were evaluated. Large variations in suction pressures were induced under simulated field conditions. The suctions present in an initially dry GCL reduced to very low levels quite rapidly when hydrated, which facilitates self-healing of the GCL. Low levels of suction were retained under simulated field conditions as long as the soil layers adjoining

the GCL were also in a low suction range. However, high suctions were induced if the GCL was placed next to zones of soils having high suctions or if the GCL was placed next to materials such as gravel or dry sands that cannot retain suctions.

Sporer & Gartung (2002a) conducted laboratory testing to quantitatively assess protective measures against GCL desiccation. The laboratory testing program considered the major factors affecting the desiccation process: temperature, temperature gradient, and soil properties. The testing program showed that temperature has a significant influence on desiccation. At higher temperatures in all test set-ups the water content loss was generally more pronounced. At lower temperatures, and thus with lower temperature gradients, moisture movement towards the top of the GCL predominates over moisture movement downwards while at higher temperatures and temperature gradients the opposite was the case. These test results demonstrated that laboratory tests on desiccation behavior need to take into account both upward and downward moisture under the combined influence of water, water vapor, and temperature.

Double layer GCL-systems tested by Sporer and Gartung (2002) showed a completely different desiccation behavior of the upper and lower GCL. Under the applied boundary conditions the upper GCL apparently protected the lower GCL against desiccation. Temperature also played a decisive role here. At 20°C, without a temperature gradient, no difference in the desiccation behavior of the upper and lower GCL was observed. However, differences between the upper and lower GCLs became more obvious with increasing temperature. Water movement downwards dominated over water movement upwards at increasing temperatures, leading to the conclusion that the upper GCL layer did not prevent water movement out of the lower GCL but that the loss in water content of the lower GCL was balanced by water movement of the upper GCL downwards.

Southen & Rowe (2002) examined the desiccation behavior of composite basal landfill lining systems under thermal gradients. Testing under simulated field conditions indicated that the risk of landfill temperature-induced desiccation adversely affecting the performance of a composite liner system containing a GCL was negligible. Rowe (2005) states that the service life of a GCL used in a composite liner system should be on the order of thousands of years provided that the design hydraulic conductivity is based upon considerations of chemical effects such as bentonite-leachate compatibility, groundwater and subgrade soil chemistry, and applied stress, there is no significant loss or movement (thinning) of the bentonite in the GCL during placement or in service, the seams of the GCL do not separate, and the GCL does not desiccate.

A recently reported phenomenon that has caused concern about the performance of GCLs is separation of the lapped seams of GCLs beneath geomembranes that are exposed without soil cover for an extended period of time. Separation of lapped seams beneath exposed geomembranes has been reported in the USA for GCLs placed upon relatively steep (1.5H:1V to 2H:1V) side slopes in at least 5 instances (Koerner and Koerner, 2005). In all cases the GCL was installed with a state-of-the-practice CQA program using overlaps between 150 and 300 mm at the seams and the separation occurred prior to placement of waste against the side slope. The observed separation between panels was typically between 0 and 300 mm from 2 months to 5 years after placement. Figure 14 shows the observed GCL seam separation at one California landfill.

Figure 14. GCL seam separation at a California landfill (Koerner and Koerner, 2005)

Postulated GCL seam separation mechanisms include necking of the GCL panel under downward gravity loads and shrinkage of GCLs manufactured at moisture contents greater than the in situ equilibrium moisture content (Thiel and Richardson, 2005). The shrinkage mechanism is supported by observations of free water accumulating between the GCL and geomembrane at the toe of the steep side slope composite liners installed in climates with large diurnal temperature extremes (personal communications with J.P. Giroud, independent consultant and Jeff Dobrowolski, GeoSyntec Consultants). However, the shrinkage mechanism does not appear to be able to predict the magnitude of the observed separation in at least some of the cases.

To mitigate the potential for seam separation, Koerner and Koerner offer the following recommendations: 1) backfill the geomembrane/GCL composite liner in a timely fashion and consider protecting the GCL with insulation when this is not possible; 2) use geotextiles to manufacture GCLs which minimize transverse contraction when they are longitudinally stressed (i.e., do not use double nonwoven GCLs with no internal scrim); 3) increase the minimum overlap to 250-450 mm; and 4) control the manufacturing moisture content to as low a value as possible (others have suggested it be limited to less than 20%). The use of a geomembrane with a white (or other pale-colored) surface to reduce diurnal temperature variation and reduce the tendency for moisture re-distribution beneath the geomembrane has also been suggested as a seam separation mitigation measure.

5.3 *Slope Stability (Interface Shear Strength)*

5.3.1 *Peak versus residual strength*

Because interfaces generally have a lower in-plane shear resistance than the shear resistance of the materials on either side of the interface, geosynthetic barriers often provide a plane of weakness along which overall barrier instability can occur. Overall barrier stability is therefore typically governed by the in-plane shear resistance of the barrier, the geometry of the containment system, and the loads on the system. In plane shear resistance includes both interface shear strength between barrier system elements and the internal shear strength of GCLs, if present in the system.

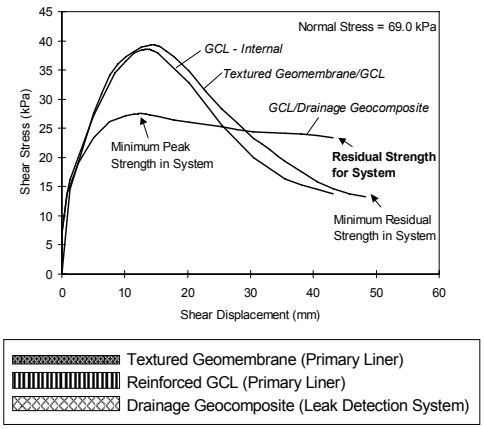


Figure 15. Liner system interface shear strengths

Generally, limit equilibrium analyses are used to evaluate overall stability for landfill systems. As many geosynthetic interfaces exhibit a post-peak strength decrease during shear (Figure 15), one of the key considerations when conducting an overall stability analysis is what shear strength to use in the analysis. Analyses may use peak shear strength, post-peak shear strength, or some combination of peak and post-peak strengths for the various interfaces in the containment system. The selection of the appropriate interface shear strength is a non-trivial question, as regulations often specify a minimum factor of safety for failure surfaces that engage the barrier system (i.e. for overall barrier stability) and whether or not a particular system satisfies that criteria may depend upon which shear strength is used.

Many regulations which specify a minimum factor of safety for overall barrier stability require a factor of safety of 1.5 for "permanent", or long term, conditions, consistent with geotechnical practice for slopes and embankments. Some regulations allow for a reduced value, typically on the order of 1.1 to 1.3, for interim or temporary conditions. Regulations will also sometimes specify what shear strength to use. For instance, the states of New York (NYS DEC, 2005) and Ohio (Ohio EPA, 2004) in the USA require a factor of safety of 1.5 for "permanent" conditions and allow the use of the peak strength for the base liner system while requiring use of the "large deformation shear strength," the post-peak shear strength at the limiting deformation in an interface direct shear test conducted using a 300 mm shear box, for the side slope liner system. The state of California, on the other hand, requires the use of the large deformation shear strength for all interfaces and requires a factor of safety of 1.5 for permanent conditions (DWR, 2003).

The use of a post-peak shear strength for side slope liner systems would appear to be a reasonable and prudent measure considering the large deformations side slope liner systems are usually subjected to due to waste placement and post-placement waste settlement. However, the rationale for using a post-peak strength for the base liner system is less clear and most engineers seem to agree that if the post-peak strength is used for all interfaces, requiring a factor of safety of 1.5 is excessive and unnecessary. For instance, Stark and Poeppel (1994) recommend checking for a factor of safety of 1.5 using peak shear strengths for the base liner system and residual shear strength to the side slope liner system but also suggest checking for a factor of safety of 1 with residual shear strength assigned to all liner surfaces.

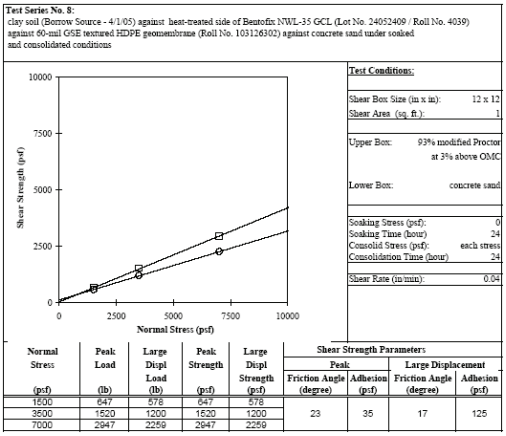
For barrier systems employing a GCL, the "in-plane" shear strength of a hydrated GCL is typically employed. Bouazza et al. (2002) summarize typical values for the shear strength of various liner system geosynthetic interfaces and for hydrated GCLs and the hydrated GCL in-plane shear strength is generally among the lowest, if not the lowest, of these shear strengths. Even when the GCL is reinforced, the post-peak shear strength of a hydrated GCL in a landfill liner system as reported by Bouazza et al. is often described by a friction angle on the order of 8 to 10 degrees for overburden pressures. This is particularly true at higher overburden pressures (i.e. at overburden pressures representative of base liner conditions), where the interface plane of weakness often makes a transition from the geotextile/GCL interface to an internal GCL failure plane.

The governing post-peak shear strength in a liner or cover system is not the lowest post-peak shear strength from among all of the in the system but rather the post-peak shear strength of the with the lowest peak strength, as illustrated in Figure 15. Thus the GCL post-peak shear strength may not be the governing shear strength of the system. Furthermore, a new generation of reinforced GCLs with thermal-locked fibers has shown significantly higher post-peak shear strengths in laboratory testing than previous GCLs, with the failure occurring at the geotextile / geomembrane interface (rather than internally) over a wide range of overburden pressure, even when the GCL is hydrated under low overburden pressures prior to testing. Figure 16 shows the results of recent interface shear tests conducted on a thermal-locked reinforced GCL proposed for use on a California landfill that was hydrated (inundated with water) under "free swell" (zero overburden) conditions for 24 hours prior to testing. These test results indicate the GCL post-peak in-plane shear strength may be characterized by a friction angle of 17 degrees and an adhesion of 6 kPa at an overburden pressure of 350 kPa (7000 psf) for this product.

Due to wide variability reported for some thermal locked products, a rigorous conformance testing program is recommended if a GCL is required to provide such a high post-peak strength in order to maintain stability

5.3.2 *GCL strength degradation*

Zanzinger & Alexiew (2002) performed long-term internal shear testing on 300 mm × 300 mm GCL specimens. These investigators applied shear stress ratios of up to 90% of the short-term internal shear strength to reinforced GCLs for up to 5000 hours without shear failure, indicating good long term durability for GCL reinforcement. However, as noted previously, the potential for GCL fiber



Normal Stress (psf)	Peak Load (lb)	Large Displ Load (lb)	Peak Strength (psf)	Large Displ Strength (psf)	Shear Strength Parameters: Peak Friction Angle (degree)	Peak Adhesion (psf)	Large Displacement Friction Angle (degree)	Large Displacement Adhesion (psf)
1500	647	578	647	578	23	35	17	125
3500	1520	1200	1520	1200				
7000	2947	2259	2947	2259				

Figure 16. In-plane shear strength of a thermal-locked GCL

degradation over time must be considered. Hsuan and Koerner (2002) described the durability and lifetime of polymer fibers with respect to reinforced GCLs. They described the physical and chemical degradation processes for polypropylene and polyethylene fibers that are used in needle punched and stitch bonded reinforced GCLs. The influence of material structure, polymeric orientation, and ambient environmental conditions on the degradation were evaluated. Possible performance and index test methods which can be used for monitoring the polymeric degradation of reinforcing fibers were suggested. They concluded that when GCLs are subjected to long-term shear stresses, fiber durability is obviously important, particularly with respect to sloping surfaces and canyon-type landfill liners. Factors involved in fiber durability are stress level, environmental conditions (e.g., oxygen level), required lifetime (e.g., the half life) and polymer formulation details. The key to the polymer formulation is the manufacturing process for the fibers and the type and amount of antioxidants.

Thomas (2002) also noted that, since the strength of GCLs is largely due to the strength of the reinforcing fibers, it is important to understand the long-term behavior of the GCL reinforcement. He addressed the question of long-term oxidative stability of a polypropylene textile made from fibers used to reinforce a commercial needle-punched GCL. Testing in forced-air ovens at four temperatures for times up to 400 days suggested that the reinforcing materials would retain 50% of their strength for 30 years when exposed to air at 20°C. However, the amount of fresh air circulated in the oven dramatically affected the oxidation rate. This may be very important since aging is often performed in gravity convection ovens. At lower oxygen concentration (e.g. at 8% oxygen) a service lifetime approaching 100 years for the reinforcing

fibers is likely in buried applications (Salman et al., 1998).

Thies et al. (2002) found that ageing, or degradation of the polymer, and stress cracking, or slow disentanglement of fibers, may compromise the strength of a reinforced GCL. Therefore, the long-term shear behavior was investigated by accelerated (elevated temperature) aging tests on four types of needlepunched GCLs. The carrier geotextiles were made of high-density polyethylene (GCL 1 – HDPE-GCLs) and polypropylene (GCL 2 – PP GCLs). Two different manufacturing methods, denoted Method A and Method B, were used in each case. Accordingly, the samples were named GCL 1A and GCL 1B and GCL 2A and GCL 2B. Method A resulted in an enhancement of the anchorage of the reinforcing fibers in the carrier geotextile compared to Method B. Testing showed that the failure mechanisms of the reinforcing fibers have to be taken into account. The failure of GCL samples made by Method A was accompanied by breaking of the reinforcing fibers from their anchoring within the carrier geotextile. The reinforcing fiber bundles of GCL samples made by Method B were disentangled and pulled out of the carrier geotextile at failure. Figure 17 shows the Arrhenius plot of the failure times versus temperature for the GCLs tested by Theis et al. GCL 1A failed much faster than GCL 1B under all testing conditions. Failure modes may vary depending on manufacturing process. The different mechanisms of failure caused different time of failure.

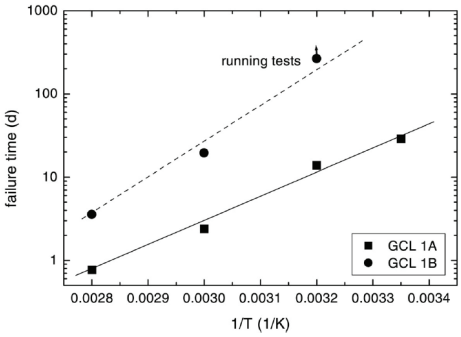


Figure 17. Arrhenius plot for shear-creep tests of needlepunched HDPE-GCLs with different fiber anchorage (Thies et al., 2002)

The results of the Thies et al. accelerated aging tests were also compared with data from short-term peel tests (Müller et al., 2004). The comparison showed that short-term peel test data were not relevant for the assessment of long-term shear strength (Table 6). The GCL with the better peel strength surprisingly had a shorter mean time to failure. The PP-GCLs (GCL 2) had much higher failure times than the HDPE-GCLs (GCL 1). The investigation clearly indicated that long-term shear tests are necessary for an assessment of the lifetime of GCLs, as short term peel testing alone may lead to fateful misjudgments Giroud et al. (2004) suggested encapsulating a GCL with geomembranes on both sides to inhibit hydration as a means of enhancing the shear strength of a liner or cover system that employs a GCL. The encapsulation system was developed for use with GCLs that employ carrier geomembranes but could also be employed with a geotextile-encased GCL and two independent geomembranes. Figure 18 illustrates encapsulation of a carrier geomembrane GCL (GM-GCL), with a welded geomembrane. The GM-GCL seams are simply lapped and the carrier geomembrane is placed against the subgrade.

Table 6. Peel strength versus time to failure in long-term testing. (All data expressed as relative values, i.e. multiples of the smallest value in each data group) (Müller et al., 2004)

Product Number	Relative peel strength in a peel test	Relative time to failure in a long-term shear test
GCL 1A	3.6	1
GCL 1B	3.3	4.4
GCL 2A	2	24
GCL 2B	1	>270

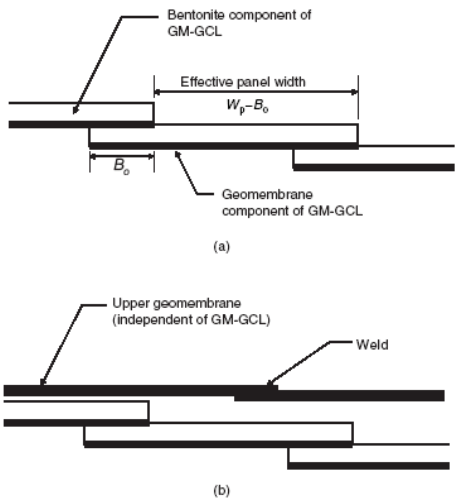


Figure 18. Schematic representation of an encapsulated GCL with a) panels with lapped seams and b) panels covered by a welded geomembrane (Giroud et al., 2004)

Hydration of an encapsulated GM-GCL occurs due to seepage from the subgrade and leachate collection system through defects in the geomembranes and from the subgrade through the lapped seams of the GM-GCL. The shear strength of the encapsulated GCL is a prorated shear strength based upon the percentage of the area of the encapsulated GCL that is hydrated. For typical defect frequencies, leachate heads, and subgrade soil suctions, the percentage of area hydrated through defects in the geomembranes was shown to be negligible based upon work by Giroud and Daniel (2004).

The percentage of hydration through the lapped seams was shown to depend primarily upon subgrade suction (or pore pressure) and overburden pressure. For a typical cover (low overburden pressure) application using lapped seams, the time to 50 percent hydrated area was on the order of 100 to 250 years while for a typical liner (high overburden pressure) system using lapped seams an with an unsaturated subgrade it was on the order of 5000 to 10000 years. However, this calculation assumes that the geomembrane maintains its integrity for this period of time. The shear strength of an unreinforced GCL with 50 percent hydrated area was calculated to be characterized by a post-peak friction angle on the order of 27 degrees for an overburden pressure of 10 kPa, representative of a cover system, and 15 degrees for an overburden pressure of 500 kPa, representative of a liner system, compared to friction angles of 8 and 4 degrees, respectively, for an unreinforced hydrated GCL. The encapsulated system was also estimated to reduce leakage through the composite liner system by several orders of magnitude based upon typical defect frequencies and leachate heads and the assumption that the geomembrane maintains its integrity and that defects in the upper geomembrane would not be aligned with defects in the lower geomembrane.

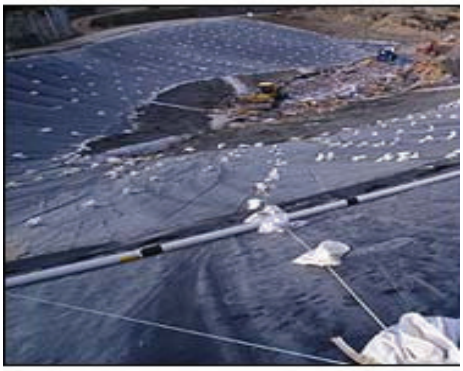


Figure 19. Kate Valley Landfill, Canterbury New Zealand. Liner system with encapsulated GCL

GCL encapsulation was employed in construction of the 36.9 hectare Kate Valley landfill located at Waipara, North Canterbury, New Zealand in 2005. The design of this facility was complicated by a small base and relatively steep side slopes (illustrated in Figure 19), stringent seismic design requirements with a relatively large design peak ground acceleration, and no suitable low permeability borrow soil within the project area.

Due to the lack of suitable low permeability soil at Kate Valley, a GCL was used in both the base and side slope liner systems. To provide a residual shear strength sufficient to satisfy seismic performance requirements for a 0.39 g "Design Basis Earthquake" and a 0.70 g "Maximum Credible Earthquake", the encapsulated system was employed. The in-plane shear strength of the liner system was based upon the weighted shear strength approach of Giroud et al. (2004) and a design life on the order of 150-300 years for the geomembrane. This design life was chosen based upon the assumption that the "semi-pervious" evapotranspirative cover that would be employed after closure would allow the waste to stabilize over this period of time. This design life resulted in a 20% hydrated area and a pro-rated residual shear strength for the GCL represented by a cohesion of 50 kPa and a friction angle of 15° for the liner system. Design criteria also called for a static factor of safety greater than 1.0 using the residual shear strength of fully hydrated GCL. The fully hydrated residual strength of the GCL was characterized by a cohesion of 5 kPa and a friction angle of 5°. At least six California landfills have employed GCL encapsulation in the past 5 years (GSE 2001).

5.4 *Local Stability Analysis*

5.4.1 *Landfill settlement and deformation*

While current engineering design practice generally considers only the limit state approach for assessing the stability of landfill structures, stability failure can be defined in terms of two states:

- *Ultimate limit state* where there is a complete loss of stability or function (e.g. waste slope failure), i.e. global stability; and
- *Serviceability limit state* such that the function of a structure is impaired (e.g. stressing of a landfill liner leading to increased permeability), i.e. local stability.

Limit equilibrium analysis cannot be used to assess local stability of the lining system and waste body on the side slope. Limit equilibrium methods can only be used to predict stability conditions that involve complete failure of the waste body through the formation of a continuous shear surface. Even if mobilized shear strengths for the geosynthetic

interfaces from a numerical analysis are used (as discussed subsequently), the limit equilibrium analysis does not give a reliable indication of local instability. However, numerical analyses have demonstrated that, as discussed in Section 5.3, using a large displacement shear strength on the side slope and peak shear strength on the base is a valid approach for assessment of global stability.

Landfill design should consider both stability of the lining system and waste mass and also integrity of the lining components subject to post-waste placement deformations. For landfill side slopes, waste deformations play an important role in the magnitude and distribution of the stresses in lining components. The high compressibility of waste and degradation processes can result in large settlements within the waste body and hence significant displacements adjacent to the lining system. This can cause relative displacement between individual elements (e.g. geomembrane and geotextile protection layers), leading to overstressing and loss of function of the barrier system. While conventional limit equilibrium stability analyses employ singular values of interface shear strength (e.g. either peak, residual or factored values, as discussed in Section 5.3), in reality the mobilized shear strength may vary along the interface due to relative displacements.

Lining systems placed on shallow slopes (i.e. greater than 30°) using traditional construction techniques are generally stable prior to waste placement (i.e. the geosynthetics and mineral layers placed on the slopes are stable), although in the longer term waste settlement can result in overstressing. It is desirable provide an interface above the primary liner (e.g. geomembrane) that has a lower peak shear strength than the interface on the lower surface of the barrier layer, as discussed in Section 2.1.2 for the Hong Kong landfill. This ensures that this primary barrier component is not placed in tension by down drag forces generated by waste settlement. However, even if tensile stresses in the barrier layer are minimized, there is still the potential for geotextile protection and drainage composite components above the geomembrane to be carried down with the waste. This can cause tensile failure and loss of function of these geosynthetics components (Figure 20). Loss of geotextile protection can lead to puncture and leakage through the barrier layer and a discontinuous drainage layer may result in increased leachate and gas pressures acting on the barrier.

Jones & Dixon (2005) used numerical modeling to investigate the potential for overstressing of a geotextile protection layer above a textured geomembrane in response to degradation-controlled waste settlement (i.e. the long-term performance of the protection geotextile was assessed). Overall stability was assessed using conventional limit equilibrium analyses and numerical modeling using the FLAC finite difference program (HCItasca, 2006) was employed to investigate the mechanism of local slippage of side slope lining materials. Strain softening behavior of the geosynthetic interfaces was incorporated in the numerical model. These analyses demonstrated that even if the overall stability of the waste mass and liner system is acceptable, it is still possible to have local integrity failure of the geotextile, as illustrated in Figure 20a.

Landfill geometries investigated by Jones and Dixon (2005) included slope angles from 18° to 45° and slope heights 30 to 60 metres. Limit equilibrium analysis for each geometry gave a factor of safety against global stability failure greater than unity. This suggests that the geometries analysed will not suffer a "global" stability failure that would result in large scale deformation of the waste body. FLAC analyses of the same side slope geometries were used to assess the influence of waste settlement on mobilised shear strengths along the interface and relative displacement between the geomembrane and geotextile. The FLAC analyses demonstrated that waste settlement can result in significant relative displacement of lining components on the side slope. These large displacements (in excess of 1000mm) of the geotextile layer above the geomembrane can be considered a local failure that could lead to loss of protection and hence integrity of the geomembrane. Even though limit equilibrium analysis indicated overall stability, the numerical modelling of waste/barrier interaction showed that large displacements can still occur.

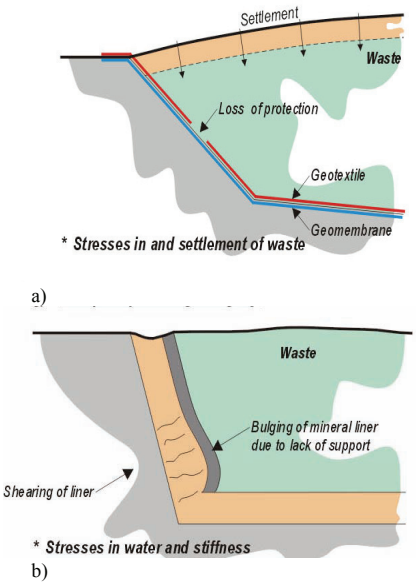


Figure 20. Mechanisms of local side slope integrity failure a) shallow slope and b) steep slope (after Dixon and Jones 2005)

Unfortunately there are no post waste placement field measurements of geosynthetic performance on shallow side slopes to validate the mechanism of behaviour modelled by Jones and Dixon (2005). However, displacements in the order of hundreds of millimeters have been measured in field trials by Villard *et al.* (2000) for similar interfaces when mineral drainage layers were placed on a side slope, so it is reasonable to suggest that geotextile displacements on the order of meters could result due to waste placement and settlement.

5.4.2 *Design implications for steep side slopes*

Quarries and canyons typically have steep (i.e. up to vertical), and often benched, side slopes. Use of quarries and canyons for development of landfill facilities introduces a number of issues related to the design, construction and performance of side slope lining systems. Geosynthetics are employed in many designs due to their effectiveness, light weight, ease of installation and quality controlled engineering properties. Two design approaches are used for steep side slopes:

- self-supporting systems, which can be constructed, and are stable, to the full height of the side slope prior to waste placement; and
- waste supported systems that are constructed in phases, with waste placed to support a lift of the lining system prior to construction of the next lift.

Self supported systems are the most common because they generally take up less air space and are cheaper to construct. However, field monitoring of stresses and deformations of a mineral (i.e. compacted clay) 70° side slope waste supported system (Dixon *et al.* 2004) has shown that lateral waste support for the liner during construction is low and variable. Large deformations of the barrier resulted in integrity failure (i.e. loss of function) with active gas venting behind the barrier (Figure 21). Although this case history relates to a mineral barrier system it demonstrated that MSW placed using standard layer thickness and compaction practice may not provide adequate lateral support to a steep slope system.

As with shallow side slopes, downdrag of lining components in quarry and canyon landfills due to waste settlement can lead to loss of integrity. It is good design practice to provide a preferential slip plane located between the geomembrane and waste to ensure that this primary barrier is not overstressed. This allows relative displacements to occur at a predefined interface, with all layers above this interface designed to cope with the large downward displacements that will occur. For

Figure 21. Large tension crack at the rear of a mineral liner placed on a 70° slope caused by liner deforming towards the waste body on the right of the photograph (Dixon *et al.* 2004)

example, single-sided textured geomembranes can be used with the textured side against the sub-grade and smooth face upwards. The designed preferential slip plane is therefore located between the upper surface of geomembrane and overlying protection geotextile. Overlaps of the geotextile are provided so that a continuous protection layer is in place following waste settlement. Fowmes *et al.* (2006) detail a case history of a geomembrane lined steep side slope where a preferential slip plane was not introduced, resulting in the geomembrane becoming overstressed such that tearing occurred during waste settlement.

Common steep slope systems in the UK use a reinforced soil system to support the geomembrane primary liner, with geotextile protection and geocomposite drainage layers between the barrier and waste (Figure 22). The reinforced soil structure usually has polystyrene facing units to provide a flat contact surface for the geomembrane. Many steep side slopes have benches, and this introduces a design challenge to ensure that geotextile and drainage layers will be continuous as they slip past the bench corner. This design problem can be solved with appropriate construction details. For instance, the reinforced soil can be used to infill the benches to produces a continuous gradient slope that can be lined as shown in Figure 22, providing a continuous plane along which layers overlying the slip plane can slide.

Fowmes *et al.* (2005) used FLAC to model the geometry and materials shown in Figure 22 to investigate performance of the lining components in response to waste settlement. The model incorporated strain softening interfaces between the

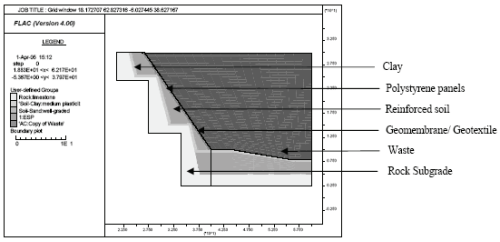


Figure 22. Lining system for benched steep slope with reinforced soil used to infill benches and provide support for geomembrane (after Fowmes *et al.* 2005)

multiple geosynthetic layers and staged construction of the lining and waste (i.e. a bench was constructed followed by waste placement against it before construction of the next bench). Modeling of staged construction is critical to assessment of liner performance due to the initial low waste lateral stresses and stiffness (Dixon *et al.* 2004). As an example of outputs from the analysis, Figure 23 shows stresses calculated in the geomembrane for one of the lifts.

To date, the numerical models used to assess the response of a liner system to landfill settlement are relatively simplistic in the way that the waste is represented. Research is in progress to develop constitutive models that better represent observed mechanical behavior of MSW, including degradation (e.g. Krase and Dinkler, 2005), and to obtain appropriate engineering parameters for use in the models (e.g. Dixon *et al.* 2006). In addition, although use of numerical modeling is becoming an integral part of landfill design, there is a lack of field data that can be used to validate the models. There is an urgent need to instrument lining systems to measure construction and post waste placement behavior. Improved understanding of waste/barrier interaction could lead to development of innovative designs for geosynthetic steep slope lining systems.

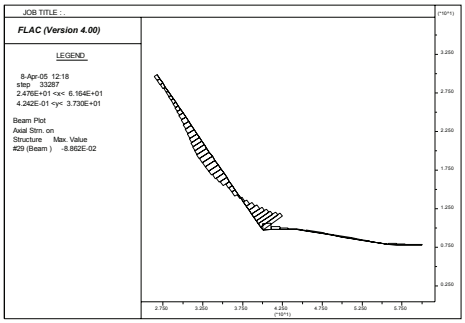


Figure 23. Axial strains in the geomembrane (maximum tensile value 4.47%)

6 COASTAL LANDFILLS

6.1 *Engineering Challenges*

Significant engineering challenges associated with coastal landfill barrier construction may be summarized as follows:

1. The facilities are located in exposed coastal sites where they are subjected to potentially damaging environmental conditions. In particular, lateral barrier systems must be able to maintain their integrity when subjected to the forces of waves, tides, and sometimes tsunamis.
2. Coastal landfills are usually located on top of relatively thick natural low permeability soil deposits. These low permeability deposits, while advantageous from the viewpoint of containment, are typically also very compressible and weak. Therefore, significant settlement is likely to occur and global and local stability must also be taken into account.
3. Due to the potential for large differential settlements associated with the large total settlement, the barrier systems must be ductile (flexible) enough to maintain their integrity when subject to differential settlement loads. The strains induced in geomembranes by the anticipated settlements must be carefully analyzed to determine whether the geomembrane will remain serviceable. The stabilized soil material used as backfill between the geomembranes must also have sufficient ductility to continue to function as a protective layer and as a barrier layer if it is designed to perform that function as well (Watabe et al. 2000, Kotake et al. 2005).

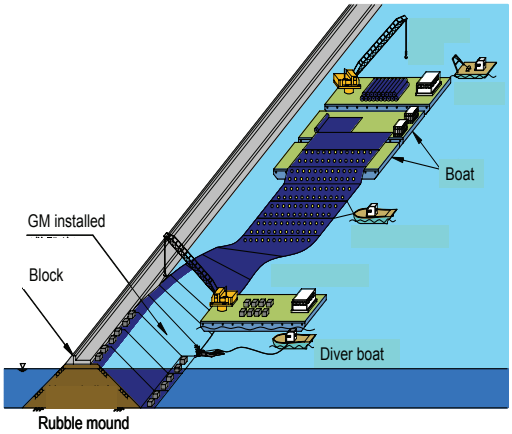


Figure 24. Geomembrane installation for a coastal landfill

4. As the construction works are conducted over water, installation of the geomembranes is complicated. A typical installation process, as shown in Figure 24, consists of seaming the geomembrane panels on land or on a ship at the site, dropping the seamed geomembrane into the water, and then sinking the geomembrane to bear on the subgrade.

The caisson seawall or rubble mound retaining dikes for these coastal landfills are constructed before the geomembranes are installed. Because they are often founded upon soft clay, ground improvement is often required to provide sufficient bearing capacity to support these structures. The configuration of the geomembrane barrier may depend upon the ground improvement method. For instance, if sand compaction piles are used (a common technique in marine construction), they should be expected to increase the hydraulic conductivity of the foundation layer and therefore the geomembrane should be lengthened to contact enough of the original clay layer beyond the ground improvement zone to maintain the integrity of the containment system, e.g. it should extend at least 5 m beyond the last row of sand compaction piles to meet the regulatory minimum requirements for containment. On the other hand, if the deep mixing method is employed to stabilize the subgrade, the subgrade may be expected to maintain the low hydraulic conductivity of the clay layer and no special considerations are required with respect to tie-in between the geomembrane and the subgrade.

In gravity caisson or rubble mound type structures (Figures 3(a) and 3(c)), the geomembranes can communicate with the open sea through rubble layers and are therefore subjected to hydraulic pressure directly from the sea. As the hydraulic pressure from the open sea fluctuates due to wave and tidal actions, the geomembranes are subjected to cyclic loading and uplift. To protect the geomembranes against wave uplift forces, protective layers are placed on top of the geomembranes to weight it down. However, due to the unbalanced load on the sloping geomembrane, until the waste is placed against the protective layer to buttress it, veneer stability of this system is an issue (Kotake et al. 2004).

Seismic stability of coastal landfills is also an important design issue. The Hyogoken-Nanbu (Kobe) earthquake of January 17, 1995 caused significant damage to many coastal structures; notably significant displacement of some gravity caissons occurred due to the strong ground motions and liquefaction of backfill. In centrifuge testing specifically conducted to investigate the stability of a coastal landfill side barrier systems against seismic motions, a 1/15th scale model test on a caisson type side barrier system with a PVC geomembrane was subjected to the ground motion record from Port Island, Kobe city. The results of this test indicated that the geomembrane strains approached a maximum value 11.6% at the top of the slope and that the strain-time profiles of geomembrane had the same time phase as displacement of the caisson (Kano et al. 2005). These results suggest that placing non-liquefiable MSW close to the side barrier may be advantageous to the seismic stability.

6.2 *Factors Influencing Containment Performance*

The containment performance of coastal landfill barriers is an important engineering issue. Performance is governed by a variety of factors including the integrity of the geomembrane, the filling materials, and the seabed (foundation) deposits. Kamon and Inui (2002) and Katsumi and Kamon (2002) presented a two-dimensional advection-dispersion numerical analysis to evaluate the leakage of contaminants from the waste to the outer sea for the caisson type of quay wall systems illustrated in Figure 25.

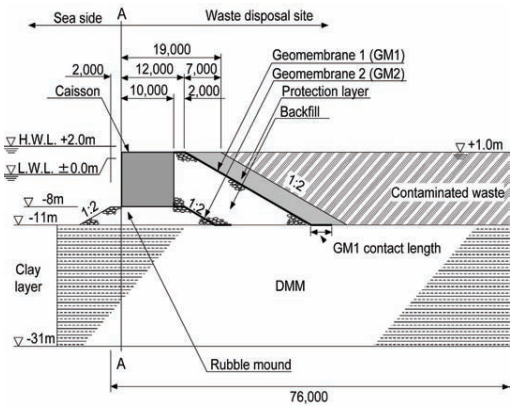
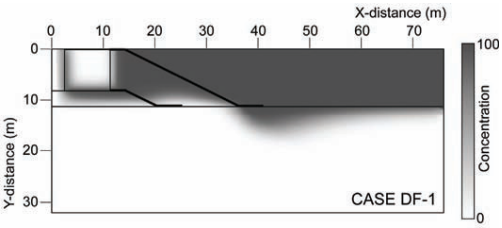


Figure 25. Cross Section of cassion quay wall barrier system for Kamon and Inui (2002) advection-dispersion modeling

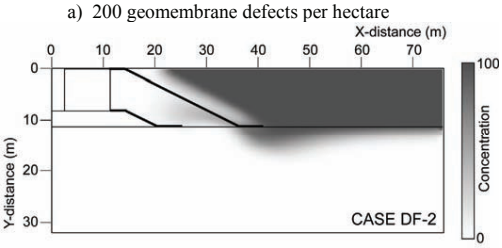
The finite element advection-dispersion chemical transport model "DTransu–2D EL" (Nishigaki et al. 1995) was used to analyze the concentration and the mass flux of the constituents released through the barrier. In these analyses it was assumed that the waste layer has a water head 100 cm higher than that of the outer sea. It was assumed that joints of the caissons were sealed and the equivalent hydraulic conductivity for the caisson was $1x10^{-9}$ cm/s for 2-D analysis. The hydraulic conductivity of the clay layer (improved by the deep mixing method) was assumed to be $1x10^{-5}$ cm/s, which satisfies the regulatory requirement for a natural clay bottom liner. The backfill material between the two

geomembranes is sand having a saturated hydraulic conductivity of $1x10^{-3}$ cm/s. The geomembranes were modelled as uniform layers with an equivalent saturated hydraulic conductivity based upon an assumed number of defects, as proposed by Kamon et al. (2002). The equivalent saturated hydraulic conductivity for the geomembrane was based on previous research (Giroud et al. 1994, Katsumi et al. 2001) as well as two-dimensional axi-symmetric numerical analyses. The assumed number of defects, and hence equivalent saturated hydraulic conductivity, were varied to see their influence on contaminant transport.

An example of the results from the finite element advection-dispersion analysis is shown in Figure 26. The profile in Figure 26(a) is based upon an assumed geomembrane defect frequency of 200/hectare, while the profile in Figure 26(b) is based upon 2.5 defects per hectares in each geomembrane. These assumptions correspond to an equivalent saturated hydraulic conductivity of $1x10^{-7}$ cm/s or $1.3x10^{-9}$ cm/s, respectively.



a) 200 geomembrane defects per hectare

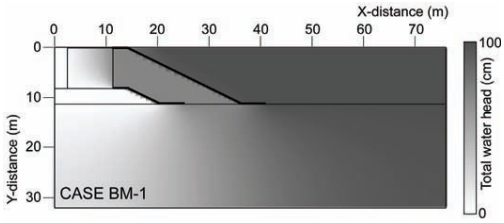


b) 2.5 geomembrane defects per hectare

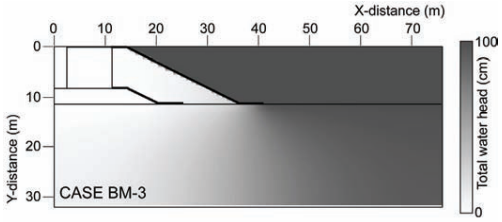
Figure 26. Concentration profiles for 2-D advection-dispersion analysis by Kamon and Inui (2002) for Backfill with $1x10^{-3}$ cm/s saturated hydraulic conductivity

For the case with 200 geomembrane defects per hectare, illustrated in Figure 26(a), after 50 years constituents from the reclaimed waste layer reached open sea. In contrast, for the case of 2.5 geomembrane defects per hectare, illustrated in Figure 26(b), constituent concentrations remained almost zero in the open sea as well as in rubble mound and backfill, even after 50 years. These analyses illustrate the importance of integrity of the geomembranes and, hence, CQA, to the performance of coastal landfill barrier systems.

Figures 27 illustrates the effect of the hydraulic conductivity of the backfill material sandwiched between two geomembrane with a high frequency of defects (200 defects/ha) in them. The results in Figure 27 correspond to the results in Figure 26(a) except that the hydraulic conductivity of the backfill material in Figure 27(a) is $1x10^{-1}$ cm/s, two orders of magnitude higher than in Figure 26(a) while the hydraulic conductivity of the backfill material in Figure 27(b) is $1x10^{-4}$ cm/s, one order of magnitude lower than in Figure 26(a). Figure 28 shows the flow rate and the mass flux for periods of 50 and 100 years across line A-A line in Figure 25 for backfill saturated hydraulic conductivity from $1x10^{-1}$ cm/s to $1x10^{-5}$ cm/s. Figure 28 clearly shows the influence of the saturated hydraulic conductivity of the backfill material between the two geomembranes on the flux through the barrier system. These analyses demonstrate that, even with the two geomembranes required by the Japanese code, good quality geomembrane construction and backfilling between the geomembranes with a low hydraulic conductivity material can result in a significant improvement in containment system performance.



a) Backfill between geomembranes with $1x10^{-1}$ cm/s saturated hydraulic conductivity



b) Backfill between Geomembranes with $1x10^{-4}$ cm/s Saturated Hydraulic Conductivity

Figure 27. Advection-dispersion results of Kamon and Inui (2002) for varying saturated hydraulic conductivity of geomembrane backfill

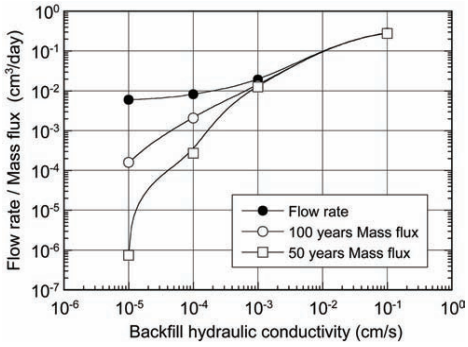


Figure 28. Influence of saturated hydraulic conductivity of geomembrane backfill on mass flux as calculated by Kamon and Inui (2002)

Based on their numerical analysis, Kamon and Inui (2002) proposed requiring that foundation soil beneath caisson quay wall systems for coastal landfills have a saturated hydraulic conductivity lower than $1x10^{-4}$ cm/s and that the primary (upper) geomembrane be placed with a contact length longer than 5 m on the bottom clay layer. They also proposed using a self-sealing material for the geomembrane if gravelly or sandy filling material is used between the GMs. Having a filling layer between the geomembranes with a low saturated hydraulic conductivity, optional under the current regulatory requirements, was strongly recommended.

6.3 *Case Histories*

The first use of geomembranes in a coastal landfill barrier system was the containment facility constructed in Tachibana bay, Tokushima prefecture, Japan. Tsuchida et al. (2000) and Watabe et al. (2000) have reported on construction of this landfill. A rubble mound type of quay wall system (Figure 3(c)) was employed for this project. As shown in Figure 29, the landfill was constructed on a thick clayey soil layer to provide a natural barrier at the base. The deformation of the geomembranes due to consolidation settlement and shear displacement of the seabed soil associated with both containment system construction and waste placement was a critical engineering issue, as was durability of the geomembranes against the cyclic force of waves. Tsuchida et al. (2000) conducted a numerical analysis to calculate the tensile stress in the geomembrane and found that using cement-treated dredged soil between the geomembranes was an effective way to minimize their deformation. Therefore, soil dredged from the bottom of the landfill site was treated with cement and used as the backfill between the two geomembranes. This approach both increased the waste capacity of the system and reduced the costs associated with transporting an alternative backfill soil from offsite.

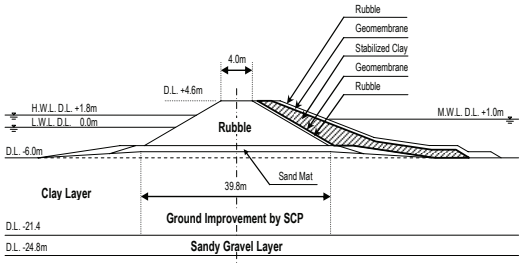


Figure 29. Cross section of coastal landfill at Tachibana bay, Tokushima prefecture, Japan (Tsuchida et al. 2000)

A second example of a Japanese coastal landfill is the landfill located in Kawanoe City, Ehime prefecture. This landfill was constructed in a place where the sea bed was a permeable sandy layer. Therefore, the entire area of the disposal site was covered with double geomembranes (Kotake et al. 2006). For lateral barrier systems, caisson or rubble type seawalls were constructed with the two geomembrane system. For the bottom liner, two geomembranes were also installed and were connected continuously with the geomembranes for the lateral barrier. Kotake et al. (2004, 2006) describes the evaluation of the impacts of wave forces on the geomembranes and the stability of the multi-layered configurations employed in the side barrier system.

7 SUMMARY AND CONCLUSIONS

Composite geosynthetic barrier systems consisting of a geomembrane overlying either a GCL or a compacted low permeability soil layer have been generally recognized as the most cost effective means of controlling the advective transport of gasses and liquids for almost 2 decades. Almost every industrialized country requires the use of composite liners in engineered landfills, as either a single or double composite liner system for MSW or in a double liner system for hazardous waste. Composite cover systems are also widely used in industrialized countries for hazardous waste landfills and at waste sites where minimization of infiltration is paramount. Field evidence of the effectiveness of composite liner systems constructed using a rigorous construction quality assurance program continues to grow. This evidence clearly demonstrates that while it may be true that "all liners leak," well designed and constructed composite liner systems leak at such

a slow rate that the impact of this leakage on human health and the environment is negligible. The field data also shows that GCLs are clearly as effective as, if not superior to, compacted low permeability soil as the underlying component of a composite liner system with respect to advective flow resistance.

The most significant advance with respect to the use of geosynthetic barrier systems since the last International Geosynthetics Conference in 2002 has been the increasing implementation of these systems in developing countries. Once considered too exotic and too expensive for implementation in developing countries, geosynthetic barrier systems are being implemented with increasing frequency for new landfill construction in the developing world. Technological advances in geosynthetic barrier systems include the development of strategies for enhancing their ability to control diffusive flux. Reactive coatings and coatings that improve the diffusion resistance of geomembranes have been demonstrated to be effective in a laboratory setting, though field evidence of the effectiveness of these approaches to diffusive flux control has yet to be provided. The use of forced air in the leak detection layer of a double liner has also been proposed as a means of controlling the diffusive flux of volatile organic contaminants across a geomembrane barrier.

The development of geomembrane-based barrier systems for coastal landfills is another emerging technology. Coastal landfills offer several advantages over traditional land-based facilities in densely populated coastal regions. Barrier systems that employ a double geomembrane barrier for lateral containment are currently required for coastal landfills in Japan. Numerical analyses of coastal landfill barrier performance have identified key factors in their effectiveness, including the integrity of the geomembranes and the nature of the backfill material used between geomembranes in a double geomembrane barrier system.

Field and laboratory testing on exposed HDPE geomembrane indicates that a service life on the order of 50 years may be expected for these systems. Analysis of laboratory data indicates that the service life of a buried geomembrane in a landfill application depends to a large extent on the service temperature(s) of the geomembrane. For a typical "dry" landfill with service temperatures that approach a maximum of around 35°C for the primary liner, a service life on the order of 100 to 200 years may be expected. For the geomembrane of a secondary liner with an estimated service temperature of around 20°C, the estimated service life of the geomembrane increases to around 600 years, while in a "wet" landfill with service temperatures for the primary liner of the order of 45°C the geomembrane service life may be as low as 50 years. A GCL used in a liner application may be expected to have a service life on the order of thousands of years from a hydraulic conductivity perspective provided chemical compatibility between the bentonite and the leachate and subgrade soils is taken into account. Contact with chemicals in leachate and other solutions can lower the swelling of a GCL and thereby lead to a higher permeability. A new generation of chemically-modified GCLs shows promise in reducing the susceptibility of GCLs to degradation from permeation with organic and inorganic solutions. Prehydration may also be effective at reducing the susceptibility to chemically-induced degradation, particularly at higher confining stresses. However, the duration of the laboratory tests used to demonstrate these effects must be taken in to account, as chemical modification and prehydration may just delay the onset of degradation.

Degradation due to desiccation (wetting and drying) is of particular concern for GCLs used in cover applications. Desiccated GCLs not only leak water but also landfill gas. Therefore GCLs demand special care to cope with the risk of desiccation. The water balance of the topsoil is of primary importance in preventing desiccation. Sufficient soil cover should be placed so that the GCL is below the "active" zone, the zone in which significant drying can occur. The material properties of the topsoil, the placement of the soil, and the thickness of the topsoil layers as well as the choice and care of the vegetation should be considered on a site specific basis. GCLs used in cover applications may be susceptible to degradation due to freezing and thawing if the thickness of the cover soil is less than the depth of frost penetration, particularly if the GCL is subject to sustained periods of freezing.

Field evidence shows that transformation of sodium bentonite to calcium bentonite due to cation exchange regularly occurs in GCLs used in cover systems. This transformation typically takes place over a period of several months to few years. Cation exchange causes an increase in hydraulic conductivity of the GCL and reduces the ability of the GCL to self-heal. The saturated hydraulic conductivity of a GCL transformed from sodium bentonite to calcium bentonite by cation exchange may increase by approximately one-half to one order of magnitude if the cation exchange occurs in situ without desiccation. If cation exchange and desiccation occurs under unsaturated conditions (e.g. arid climate, low thickness of cover soil, low water storage capacity of cover soil) an increase of the hydraulic conductivity of the GCL of 2 to 3 orders of magnitude can occur. However, if sufficient normal stress exists (>15 to 20 kN/m^2 or ≈ 0.75 to 1 m cover soil), self-sealing will occurs upon access to water. After ion exchange, desiccation, and "self-sealing," GCLs may have a higher hydraulic conductivity than new sodium GCLs (≈ 1 order of

magnitude higher) but the effectiveness against inflitration will still be substantial. If insufficient normal stress exists (for example, in the test plots at Hamburg-Georgswerder with only 45 cm of cover soil), self-sealing of calcium bentonite GCLs may not occur. Cracks in bentonite resulting from desiccation which do not self-heal after re-wetting may be attributed not only to cation exchange but also to irreversible changes in the structure of the bentonite.

Slope stability and interface shear strength continue to be a major issue for the design of geosynthetic barrier systems. While some regulatory agencies are mandating use of a post-peak shear strength for all interfaces in conjunction with a static factor of safety of 1.5 for global stability assessment, most engineers seem to agree that the post-peak shear strength may not be appropriate for all interfaces, e.g. for interfaces in the base liner system, and that if post-peak shear strengths are used for all interfaces a factor of safety less than 1.5, e.g. on the order of 1.1 to 1.3, is sufficient for most landfill applications. The use of a hydrated shear strength for geosynthetic clay liners is generally accepted as appropriate unless the GCL is protected from hydration. Thermal-locked reinforced GCLs appear to offer relatively high post-peak shear strengths even after hydration at low overburden pressure. However, degradation of the reinforcing fibers may occur over periods from 50 to several hundred years, depending on service conditions and fiber composition. Encapsulation with two geomembranes has also been employed as a means of increasing in-plane GCL shear strength, by minimizing the hydration of the bentonite. Encapsulation has the added benefit of increasing the advective flow resistance of the liner system. However, considering the very low advective flow rates of a properly designed and constructed composite liner it is not clear that encapsulation is necessary from an environmental protection perspective.

Local instability of geosynthetic barrier systems caused by landfill deformations associated with waste degradation and settlement should be considered in design, particularly in steep-walled landfills. Large settlements associated with waste compression and degradation can put large downdrag forces on lining system components. Local stability assessment may require the use of some type of finite element or finite difference program for evaluating liner system stresses due to landfill displacements.

Coastal landfills represent a relatively new application of geosynthetic barrier systems in landfill engineering. Geomembranes are routinely used for lateral barriers in these facilities and geosynthetic bottom barriers may also be used if the facility is sited upon permeable ground. Numerical analyses demonstrate the importance of good construction, including construction quality assurance, in providing a high quality geomembrane barrier and a high degree of environmental protection.

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