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Geosynthetics in waste containment facilities: recent advances

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ABSTRACT: Geosynthetics are widely used in waste containment facilities, as these facilities have used literally all types of geosynthetics in all identified functions (e.g., filtration, reinforcement, etc.). The inclusion of geosynthetic components is likely to expand as manufacturers develop new and improved materials and as engineers/designers develop new analysis routines for new applications. This paper focuses on specific advances involving the use of geosynthetics in the different components of waste containment facilities. In particular, this paper addresses recent advances involving the use of geosynthetics in bottom liner systems (e.g. geosynthetic clay liners, service life of geomembranes), cover systems (e.g. reinforced cover systems, exposed geomembrane covers), side slope lining systems (e.g., interface stability, steep sided walls), liquid collection systems (e.g., determination of the maximum liquid thickness, design of double slope layers), cut-off wall systems (e.g. interlocks and geomembrane performance) and remediation work (e.g. prefabricated vertical drains for methane extraction and soil flushing). Recent case histories are also provided to document the implementation of these advances in engineering practice.

1 INTRODUCTION

The protection of groundwater and surface water is now a major consideration in the design of waste containment facilities in many countries. Geosynthetics play an important role in this protective task because of their versatility, cost-effectiveness, ease of installation, and good chracterization of their mechanical and hydraulic properties. Furthermore they can offer a technical advantage in relation to traditional liner systems or other containment systems. The use of geomembranes as the primary water proofing element at the Contrada Sabetta Dam, Italy (Cazzuffi 1987) and to keep an upstream clay seepage control liner from dessicating in the Mission Dam (today Terzaghi Dam), Canada (Terzaghi & Lacroix 1964)) in the late 1950's represent applications that have been the precursors of today's usage of geosynthetics in containment systems. Both applications predated the use of conventional geosynthetics by some 20 years. Geosynthetics systems are nowadays an accepted and well established component of the landfill industry (since at least early 1980's). Containment systems for landfills typically include both geosynthetics and earthen material components, (e.g., compacted clays for liners, granular media for drainage layers, and various soils for protective and vegetative layers).

The objective of this paper is to provide a review of recent advances on the use of geosynthetics in waste containment facilities. Emphasis is on the advances that have taken place in the period 1998-2002 (i.e. since the Sixth International Conference on Geosynthetics). The state of the art on the use of geosynthetics in waste containment facilities previous to this period has been documented by various important sources, which have set the path for the growth of geosynthetics in this field (e.g. Giroud & Cazzuffi 1989; Koerner 1990; Cancelli & Cazzuffi 1994; Gourc 1994; Rowe et al. 1995; Bonaparte 1995; Gartung 1996; Daniel & Bowders 1996; Manassero et al. 1996, 1998; Rowe 1998). This paper also builds on more recent reviews on the use of geosynthetics in waste containment facilities (e.g. Zornberg & Christopher 1999; Manassero et al. 2000; Rowe 2001). The reader is referred to these sources for further information on factors influencing the selection of different types of geosynthetics and factors to be considered in construction.

Focus of this paper is not on recent advances in geosynthetic materials, but on specific advances involving the use of these materials in the different components of waste containment facilities. Accordingly, following an overview of landfill facilities and their regulations, this paper addresses recent advances in bottom liner systems (e.g. hydraulic conductivity and chemical compatibility of GCLs, service life of geomembranes), cover systems (e.g. reinforced cover systems, exposed geomembrane covers, gas migration), side slope lining systems (e.g., interface and internal stability through GCLs, steep sided walls), liquid collection systems (e.g., determination of the maximum liquid thickness, design of double slope layers), cut-off wall systems (e.g. interlocks and geomembrane performance) and soil and groundwater remediation (e.g. soil vapour/gas extraction and soil flushing systems enhanced with prefabricated vertical drains; geotextiles for permeable reactive barriers). Recent case histories are also provided to document the implementation of recent advances in engineering practice.

2 LANDFILLS: AN OVERVIEW

2.1 *Historical perspective*

According to the Concise Oxford Dictionary, a landfill is defined as follows:

Landfill, *n*.

1- Waste material etc. used to landscape or reclaim areas of ground.
2- The process of disposing of rubbish in this way.
3- An area filled in by this process.

The third definition is the operable one used for the purposes of this paper. Landfills, in various forms, have been used for many years. The first recorded regulations to control municipal waste were implemented during the Minoan civilization, which flourished in Crete (Greece) from 3000 to 1000 B.C.E. Solid wastes from the capital, Knossos, were placed in large pits and covered with layers of earth at intervals (Tammemagi 1999). This basic method of landfilling has remained relatively unchanged right up to the present day. Landfill design evolved as a series of responses to problems. Only when a problem was identified or reached a sufficient level of concern were corrective steps taken. These improvements were invariably driven by regulatory requirements. In Athens (Greece), by 500 B.C.E. it was required that garbage be disposed of at least 1.5 kilometres from the city walls. Each household was responsible for collecting its own waste and taking it to the disposal site. The first garbage collection service was established in the Roman Empire. People tossed their garbage into the streets, and it was shovelled into a horse drawn wagon by appointed garbageman who then took the garbage to an open pit, often centrally located in the community. The semi-organised system of garbage collection lasted only as long as the Roman Empire. As industrialisation of nations occurred, many containment facilities were constructed to retain various types of raw materials and/or waste products. Most of these containment facilities were not designed and almost none were lined to prevent leakage of wastes into the surrounding environment. Until the late 1970s there was little engineering input into landfilling practice and little consideration given to the impact of landfilled wastes on land and groundwater. By the end of the 1970's, the problems in managing landfill sites had arisen from the contamination of soil and groundwater (with, for example, heavy metals, arsenic, pesticides, halogenated organic compounds and solvents) and the potential risks to exposed populations. From the 1970's through the 1990s landfill design philosophy moved towards the objective of containment and isolation of wastes. This has resulted in a major upsurge in the development of engineered waste disposal systems, which included extensive use of geosynthetics. In the United States and Europe, the evolution of municipal landfill design philosophy since the 1970's has been relatively simple and has involved three significant phases through the 1990s and is entering a fourth phase as we enter the 21st century. These phases of municipal landfill development are summarized in Table 1. In Australia this evolutionary process has followed the same steps with the exception that the development of policy, regulation and guidance for landfill design was given more attention only in the mid-1990s (Bouazza & Parker 1997). The focus in this decade is anticipated to be on mechanical and biological waste treatment, either in ground or prior to deposition, including increased use of leachate recirculation and bioreactor technology, as owners, regulators, and engineers become more familiar with these concepts and their benefits with respect to decreasing long term costs and liabilities. While waste reduction and reuse efforts may diminish the per capita quantity of waste generated in industrialized nations, there is no doubt that landfills will remain an important method of waste disposal for the foreseeable future due to their simplicity and cost-effectiveness. In this respect, geosynthetics will certainly continue to play a key role in landfill design, construction and operation. In less developed countries, this evolutionary process is taking place at a much slower pace since their priorities are on providing housing, education and health to their population.

Table 1. Summary of municipal landfill evolution (modified from Bouazza & Kavazanjian 2001).

Date	Development	Problems	Improvements
1970s	Sanitary landfills	Health/nuisance, i.e odour, fires, litter	Daily cover, better compaction, engineered approach to containment
Late 1980s-early 1990s	Engineered landfills, recycling	Ground and groundwater contamination	Engineered liners, covers, leachate and gas collection systems, increasing regulation, financial assurance
Late 1980's, 1990s	Improved siting and containment, waste diversion and re-use	Stability, gas migration	Incorporation of technical, socio-political factors into siting process, development of new lining materials, new cover concepts, increased post-closure use
2000s	Improved waste treatment	?	Increasing emphasis on mechanical and biological waste pre-treatment, leachate recirculation and bioreactors, "smart landfills"

2.2 *Landfill components*

There are various design philosophies and landfill management approaches in use today (Rowe et al. 1995). One (passive) is to provide a cover system as impermeable as possible and as soon as possible after the landfill has ceased operating, so as to minimize the generation of leachate (waste liquid). This approach has the benefits of minimizing both the amount of leachate that must be collected and treated, and the mounding of leachate within the landfill. It also has the disadvantage of extending the contaminating lifespan. With low infiltration, it may take decades to centuries before the field capacity of the waste is reached and full leachate generation to occur. An alternative philosophy (active) is to allow as much infiltration as would practically occur. This would bring quickly the landfill to field capacity and allow the removal of a large proportion of contaminants (by the leachate collection system) during the period when the leachate collection system is most effective and is being carefully monitored (e.g. during landfill construction and, say, 30 years after closure). The disadvantages of this approach are two-fold: Firstly, larger volumes of leachate must be treated; this has economic consequences for the operator. Secondly, if the leachate collection system fails, a high infiltration will result in significant leachate mounding. Geosynthetics play an important role in either case and contribute, in both design approaches, to minimize contaminant migration into the surrounding environment to levels that will result in negligible impact.

The liner components of confinement systems used in modern waste disposal facilities are illustrated in Figure 1. Geosynthetics and related products have found wide application in the design and construction of these facilities and also in remediation projects as will be discussed later in the paper. This application has been triggered by the economical and technical advantages that geosynthetics can offer in relation to more traditional materials.

Referring to the three liner components (i.e. bottom, side and cover liners) of a containment system as shown in Figure 1, it is possible to summarize their main functions as follows (see also Manassero et al. 2000):

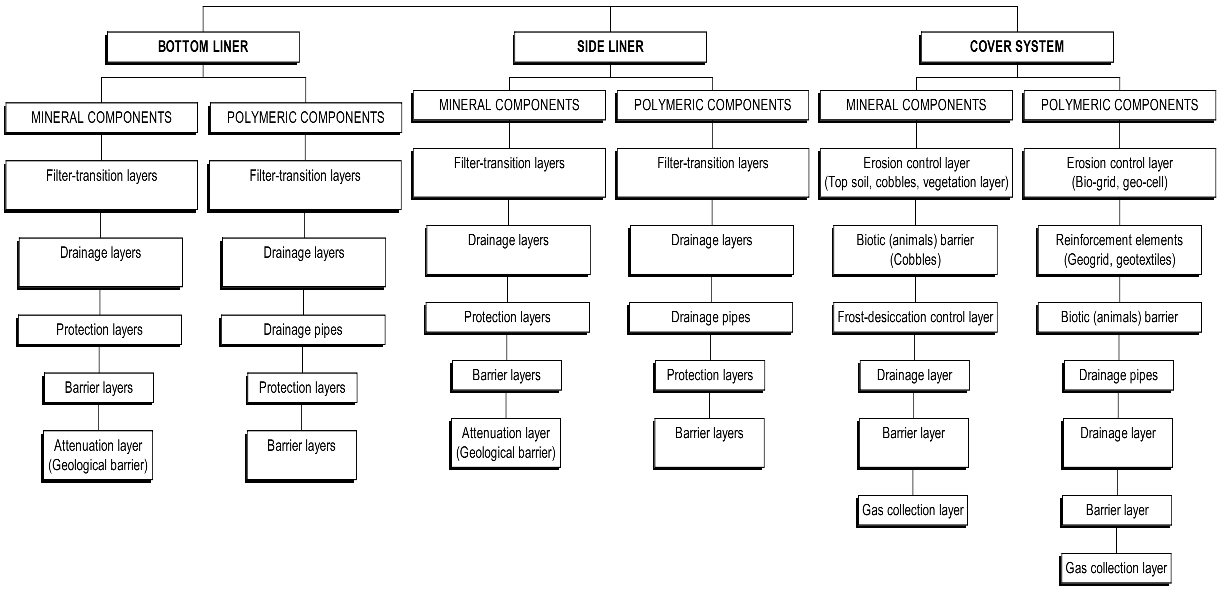


Figure 1. Liner components of solid waste containment systems (from Manassero et al., 2000).

- The bottom liner must reduce as much as possible the advective and diffusive contaminant migration toward the underlying vadose zone and/or aquifer. The performance of the bottom barriers is fundamentally governed by the following parameters: (1) field hydraulic permittivity and diffusivity, compatibility, sorption capacity and service life. On the other hand the performance of filters and drainage layers are governed by the capacity to avoid clogging, which in turn is influenced by the type of waste and landfill management. Direct field observations have shown that clogging in the liquid collection and removal system (LCRS) is reduced by increasing the seepage velocity of the leachate (Rowe 1998). Recent advances involving the use of geosynthetics in bottom liner systems are discussed in Section 4.
- The side slope liner has the same function as the bottom liner. However, its drainage component is less demanding than for the bottom liner due to the generally high hydraulic gradients along the side slopes. On the other hand, design of the side lining may be governed by stability considerations and by the need of controlling biogas migration into the vadose zone. Recent advances involving the use of geosynthetics in side slope liner systems are discussed in Section 5.
- The cover system has numerous functions: it must control water and gas movement and it should minimize odors, disease vectors and other nuisances. Cover systems are also used to meet erosion, aesthetic, and other post-closure development criteria. In spite of the numerous functions of cover systems, their design criteria are often less stringent than those used in the design of the other two liner components because they can be easily repaired and monitoring of their performances is simpler. Accordingly, many advances are expected in the near future regarding the design of cover systems. Recent advances involving the use of geosynthetics in cover systems are discussed in Section 7.

In addition to the three liner system components, geosynthetics have gained significant use in two additional components in waste containment systems, namely, the liquid collection systems and cut-off wall systems:

- The liquid collection systems are used for liquid collection in association with cover liners, for leachate collection layers in association with bottom liners, and as leakage detection and collection layers in the case of double liners. Gas collection systems have also been designed using geosynthetics. Recent advances involving the use of geosynthetics in liquid collection systems are presented in Section 6.
- Cut-off wall systems are being designed increasingly making use of geosynthetics. This is particularly the case for closure projects of old sites that have been constructed without stringent bottom liner systems or for hazardous waste containment. The advantages of these systems have been fully recognized, the trend is to design them as highly engineered structures where aspects like chemical compatibility, diffusion, defects, etc. are taken into account to evaluate their global performance. Recent advances involving the use of geosynthetics in cut-off walls are presented in Section 8.

Application of existing geosynthetics materials to new applications, e.g., prefabricated vertical drain remediation, systems is a good indicator of their immense potential in remediation work, this aspect is dicussed in section 9.

2.3 *Geosynthetics in landfills*

There are numerous types of geosynthetics, which can be used in waste containment applications and each has a specific function. Functions can include:

- Separation: the material is placed between two dissimilar materials so that the integrity and functioning of both materials can be maintained or improved,
- Reinforcement: the material provides tensile strength in materials or systems that lacks sufficient tensile capacity,
- Filtration: the material allows flow across its plane while retaining the fine particles on its upstream side,
- Drainage: the material transmits flow within the plane of their structure,
- Hydraulic/Gas Barrier: the material is relatively impervious and its sole function is to contain liquids or gasses, and

- Protection: the material provides a cushion above (or below) geomembranes in order to prevent damage by punctures during placement of overlying materials.

The individual types of geosynthetics are given in Table 2. In some cases, a geosynthetic may serve multiple functions (e.g., a geocomposite layer that serves as a drainage means and a protection layer for an underlying geomembrane).

Table 2. Types and functions of various geosynthetics. ✔ main function; ★ secondary function

Geosynthetic types	Function					
	Separation	Drainage	Filtration	Reinforcement	Hydraulique/ gas barrier	Protection
Non woven geotextile	✔	★	✔		✔[1]	✔
Woven geotextile	✔		★	✔		
Geogrids				✔		
Geomembranes					✔	
Geocells	✔			✔		
Geosynthetic clay liners					✔	★
Geocomposites	★	✔	★	★	✔	✔
Geonet		✔				
Geopipe		✔				

1=asphalt-saturated geotextiles

Landfills employ geosynthetics to varying degrees depending on the designer and the applicable regulatory requirements. In this respect *Geogrids* can be used to reinforce slopes beneath the waste as well as for veneer reinforcement of the cover soils above geomembranes (Zornberg et al. 2001). A growing area for geosynthetic reinforcement materials is in vertical and horizontal expansions of landfills (Stulgis et al. 1995). Here the geogrids or high strength geotextiles are used as support systems for geomembranes placed above them in resisting differential settlement of the underlying waste. Reinforcing is also used in liner sections located above potential subsidence zones (Gabr et al. 1994). *Geonets* are unitized sets of parallel ribs positioned in layers such that liquid can be transmitted within their open spaces. Their primary function is in-plane drainage. There are basically two designs on the market, biplanar and tri-planar geonets. The tri-planar geonets are a more recent development, which resist vertical compression under load and allow larger in-plane flows (Banks & Zhao 1997). Because of their open structure, geonets must be protected from becoming clogged by soil or adjacent material. In all cases, geonets are used with geotextiles or geomembranes on one or both of their planar surfaces. *Geomembranes* are relatively impermeable sheets of polymeric formulations used as a barrier to liquids and/or vapors. The most common types of geomembranes are high density polyethylene (HDPE), very flexible polyethylene (VFPE), polyvinyl chloride (PVC), and reinforced chlorosulfonated polyethylene (CSPE-R), although there are other types available (Koerner 1991). Polypropylene (PP) is an example of a relatively new use of a polymer in geomembranes (Matichard et al. 1996, Bouazza 1998, Comer et al. 1998). In the early uses of geomembranes for waste containment applications, there was concern about chemical compatibility with waste liquids and leachates, and about the service life of geomembranes. Now, it is widely accepted that the long-term durability of geomembranes is not a major concern (Hsuan & Koerner 1998; Rowe & Sangam 2001) as they are compatible with most chemicals (Tisinger et al. 1991). A properly designed geomembrane has the potential of hundreds of years of service lifetime, but installation must be accomplished according to the best possible quality management principles. Construction quality issues are viewed as the principal limitations of the performance of geomembranes. *Geocomposites* represent a subset of geosynthetics whereby two or more individual materials are utilized together. They are often laminated and/or bonded to one another in the manufacturing facility and are shipped to the project as a completed unit. The type of geocomposite most commonly used in landfills is a geotextile/geonet composite (Banks & Zhao 1997). The geotextile serves as both a separator and a filter, and the geonet or built-up core serves as a drain. There can be geotextiles on both the top and bottom of the drainage core and they may be different from one another. For example, the lower geotextile may be a thick needle-punched nonwoven geotextile used as a protection material for the underlying geomembrane, while the top geotextile may be a thinner nonwoven heat-bonded or woven product. *Geosynthetic clay liners* (GCLs) represent a composite material consisting of bentonite and geosynthetics. The geosynthetics are either geotextiles or a geomembrane. With geotextile-encased bentonite, the bentonite is contained by geotextiles on both sides. The geotextiles are bonded with an adhesive, needle-punching, or stitch-bonding. For the geomembrane-supported GCL, the bentonite is bonded to the GM using a water-soluble adhesive. There are numerous styles of each type of product currently available. Due to the flexibility of production and rapid innovation, different types of GCLs are also available with variation in their performances. Of the various types of geosynthetics used for containment of waste, GCLs are one of the newest and their use is rapidly expanding. *Geopipes* are commonly used in landfill applications. A geopipe system is used in the sand or aggregate leachate collection layer to facilitate collection and rapid drainage of the leachate to a sump and removal system. Geopipes are also used in sidewall risers and manholes for removing leachate. Facilities that operate wet cells (i.e. with leachate recirculation) employ geopipe to transport and redistribute leachate back into the waste fill (Reinhart & Townsend 1998). The pipes may be made of PVC or HDPE. The latter can be solid wall or corrugated. *Geotextiles* are common components in landfills, they are used for filtration purpose or as cushion to protect the geomembrane from puncture. Geotextiles are also used occasionally to reinforce the waste mass in order to increase its global stability (Gisbert et al. 1996). *Geocells* are three-dimensional, expandable panels made from HDPE or polyester strips. When expanded during installation, the interconnected strips form the walls of a flexible, three-dimensional cellular structure into which specified infill materials are placed and compacted. This creates a system that holds the infill material in place and prevents mass movements by providing tensile reinforcement. Cellular confinement systems improve the structural and functional behavior of soil infill materials. Geocell applications include protection and stabilization of steep slope surfaces and reinforcement of subbase of bottom liners.

The multiple uses of geosynthetics in the design of modern municipal solid waste landfills is a good illustration of an application in which the different geosynthetics can be and have been used to perform all the functions discussed previously. Virtually all the different types of geosynthetics discussed previously have been used in the design of both base and cover liner systems of landfill facilities. Figure 2 illustrates the extensive multiple uses of geosynthetics in both the cover and the base liner systems of a modern landfill facility (Zornberg & Christopher 1999). The base liner system illustrated in Figure 2 is a double composite liner system. Double composite liner systems are used in some instances for containment of municipal solid waste and are frequently used for landfills designed to contain hazardous waste. The base liner system shown in the figure includes a geomembrane/GCL composite as the primary liner system and a geomembrane/compacted clay liner composite as the secondary system. The leak detection system, located between the primary and secondary liners, is a geotextile/geonet composite. The leachate collection system overlying the primary liner on the bottom of the liner system consists of gravel with a network of perforated pipes. A geotextile protection layer beneath the gravel provides a cushion to protect the primary geomembrane from puncture by stones in the overlying gravel. The leachate collection system overlying the primary liner on the side slopes of the liner system is a geocomposite sheet drain (geotextile/geonet composite) merging into the gravel on the base. A geotextile filter covers the entire footprint of the landfill and prevents clogging of the leachate collection and removal system. The groundwater level may be controlled at the bottom of the landfill by gradient control drains built using geotextile filters. Moreover, the foundation soil below the bottom of the landfill may be stabilized as shown in the figure using randomly distributed fiber reinforcements, while the steep side soil slopes beneath the liner could also be reinforced using geogrids. Different types of geosynthetics (e.g. geogrids, geotextiles, fibers) could have been selected for stabilization of the foundation soils.

The cover system of the landfill illustrated in Figure 2 contains a composite geomembrane/GCL barrier layer. The drainage layer overlying the geomembrane is a geocomposite sheet drain (composite geotextile/geonet). In addition, the soil cover system may include geogrid, geotextile, or geocell reinforcements below the infiltration barrier system. This layer of reinforcements may be used to minimize the strains that could be induced in the barrier layers by differential settlements of the refuse or by a future vertical expansion of the landfill. In addition, the cover system could include a geogrid or geotextile reinforcement above the infiltration barrier to provide stability to the vegetative cover soil. Fiber reinforcement may also be used for stabilization of the steep portion of the vegetative cover soil. A geocomposite erosion control system above the vegetative cover soil is indicated in the figure and provides protection against sheet and gully erosion. Figure 2 also illustrates the use of geosynthetics within the waste mass, which are used to facilitate waste placement during landfilling. Specifically, the figure illustrates the use of geotextiles as daily cover layers and of geocomposites within the waste mass for collection of gas and leachate. Geosynthetics can also be used as part of the groundwater and leachate collection well system. The use of geotextiles as filters in groundwater and leachate extraction wells is illustrated in the figure. Finally, the figure shows the use of an HDPE vertical barrier system and a geocomposite interceptor drain along the perimeter of the landfill facility. Although not all of the components shown in Figure 2 would normally be needed at any one landfill facility, the figure illustrates the many geosynthetic applications that can be considered in landfill design.

3 GEOSYNTHETICS AND REGULATIONS

Over the past two decades considerable attention has been focused on the management of wastes in the environment. It has become necessary to design and construct safe waste disposal facilities or landfills, which employ the best of the available technologies. Much of this focus on safe waste containment was prompted in most countries by the introduction of more stringent regulations. Nowadays not only must new waste containment facilities meet stringent government requirements, often involving composite liner systems (geomembrane/compacted clay liner or geomembarne/geosynthetic clay liner) and often used in combination with cover systems to form the containment for a landfill, but many existing facilities must either be cleaned up and closed or retrofitted with pollution-reduction/prevention systems and monitored to ensure that current legal requirements are met.

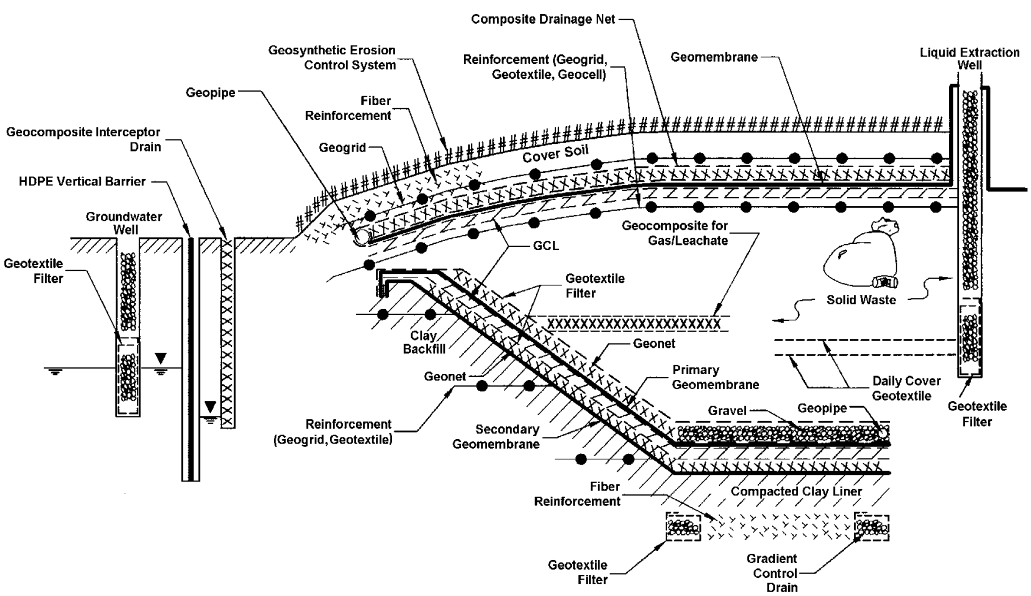


Figure 2. Multiple uses of geosynthetics in landfill design (from Zornberg & Christopher 1999).

3.1 *MSW landfills*

Modern municipal solid waste (MSW) facilities are typically designed with a barrier system intended to limit contaminant migration to levels that will result in negligible impact. The system includes a leachate collection system (LCS), which is intended to: (a) control the leachate head acting on the underlying liner, and (b) collect and remove leachate. The leachate collection system typically incorporates a geotextile filter, a granular layer or geonet, and perforated collection pipes. The liner may range from a thick natural clay deposit to engineered liner systems involving one or more geomembrane (GM) and/or compacted clay liner (CCL) or geosynthetic clay liner (GCL). The purpose of a composite liner is to combine the advantages of two materials, such as geomembranes and clays, each having different hydraulic, physical and endurance properties.

MSW landfills typically are designed with a single composite liner. In most cases, they do not include a leakeage detection system. A system of groundwater monitoring wells is typically used to monitor for possible leakage from these facilities. However, in some cases a double liner with a geonet leakeage detection system, typical of hazardous landfills (see section 3.2) has been used for MSW landfills. For example, at least 8 states in the United States require double lined facilities for non-hazardous wastes (Koerner 1997). Currently, 24% of MSW landfills in the United States, and 14% of landfills worldwide, have been designed using double liner systems (Koerner, 2000). Figures 2 and 3 show a double composite liner system at the bottom liner of the landfill. Regulations in many countries have established both prescriptive designs (Table 3) and performance-based designs (see Rowe 1997 and Manassero et al. 2000 for a discussion of the differences in these approaches).

In developing countries the design of lining systems is still in its infancy. Limited data is avalaible in the literature. However, Ashford et al. (2000) have collected a comprehensive list of liners designed in Thailand. The liners were part of landfills currently in operation, being contructed, or scheduled for construction. In particular, the lining system at one of these landfills in Thailand (the Pathum Thani site) consisted of compacted clay placed over a geomembrane. It seems in this case that the excessive stress induced by the compaction of the clay has not been taken into account in the design. However, it should be recognized that, until only a few years ago, open dumping was the standard practice. Giroud et al. (1995) queried whether a developing country should adopt or adapt landfill regulations and designs from countries with stringent environmental regulations. Their answer was that waiting to construct a landfill with a state of the art system in a developing country is likely to result in more pollution than accepting lower standards and immediately constructing landfills with liners built using local materials. Bouazza (1998) reported that using local materials can achieve an acceptable performance and present a viable economical alternative. However, one has to be cautious about the long term behaviour of the different liner components. More importantly, Rowe (1997) pointed out that countries without regulations should be cautious about adopting the existing system of regulations from another country. He stressed that careful technical consideration should be given to the conditions (e.g. economic, hydrogeologic, climatic, the skills of available workforce, and the potential to ensure good construction quality control and assurance for different systems) in a given country before deciding on the appropriateness of a given design.

3.2 *Hazardous landfills*

Hazardous wastes can originate from a wide range of industrial, agricultural, commercial, and household activities. They represent a very high risk potential to the environment and population health. Most industrialised nations are currently confronted with very acute hazardous contamination problems. In Western Europe, it is estimated that 70% of hazardous waste is still deposited in landfills (WHO 1995). The legacy of the former Soviet Union also leaves a myriad of soil and groundwater contamination issues to be dealt with. In the Russian Federation, it is estimated that 75 million tonnes of hazardous waste were produced in 1990. Only 18% of the waste is treated or recycled; most is deposited or stored on site or in sites not designed for storing hazardous waste, including those for domestic waste (WHO 1995).

To deal with this reality, landfilling of hazardous waste is typically under strict regulations. Hazardous waste landfills in industrialised countries that follow today's restrictive regulations involve highly engineered storage/disposal facilities. The liner system for this type of landfills is generally different from a liner for a MSW landfill and also varies from country to country. Double liner systems are required in the US (Subtitle C regulations), that is, 100% of new hazardous landfills are required to have double liner systems (Koerner 2000). Figures 2 and 3 illustrate a double composite bottom liner system. It generally includes a filter zone, which separates the waste from a free draining zone (primary leachate collection and removal system, PLCRS), which lies over the primary geomembrane barrier (primary liner). A leakage detection system (secondary leachate collection and removal system, SLCRS). Beneath the SLCRS is the secondary liner, which typically includes a composite liner made of a geomembrane with a compacted clay layer below it. The new European recommendation (OJEC 1999) prescribe that the bottom lining system for hazardous waste must consist of at least 5 m thick compacted clay liner with $k \leq 10^{-9}$ m/s. It is apparent that European countries rely more on mineral liners than on geosynthetics and related products. It should also be pointed out that the European recommendation is not adopted by all European countries.

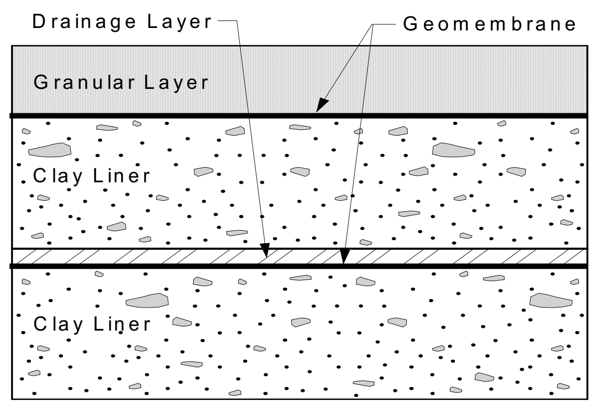


Figure 3 : Cross section of double liner systems.

3.3 *Cover systems*

Most regulations generally prescribe a cover system for the waste after closure. Figure 4 illustrates a typical design, which reflects widely used standards in landfills. It relies heavily on the use of a composite liner (GM + CCL or GM +GCL). This design is aimed at limiting percolation of water into the underlying waste, allowing minimization of the transport of contaminants from the landfill to the groundwater.

The significant recent interest in alternative cover systems, especially in arid areas, and the emergence of concepts such as leachate recirculation and bioreactor landfills will certainly lead to changes in regulations to accommodate these advances in landfill technology.

Table 3. Typical liner requirements in various parts of the world (modified from Manassero et al. 2000 and Rowe & Bouazza 2000).

Country	Bottom liner	Waste type
Australia-New Zealand		
NSW state	0.9 m compacted clay* **	Municipal solid waste
	0.9 m compacted clay* + GM (to be used in areas of significant threat to the env.) **	Municipal solid or hazardous waste
Victoria State	0.6 m compacted clay + GM	Municipal solid waste
New Zealand	0.6 m low permeability soils or compacted clay* + GM	Municipal solid waste
Europe		
EEC Directive	1 m clay* ($k=10^{-9}$ m/s) + artificial sealing layer	Municipal solid waste
	5 m clay* ($k=10^{-9}$ m/s) + artificial sealing layer	Hazardous waste
Austria	5 m ($k=10^{-7}$ m/s) or 3m ($k=10^{-8}$ m/s natural subsoil or equivalent artificial mineral barrier + 0.75 m mineral liner ($k=10^{-9}$ m/s) + GM (2.5 mm thick) plus protective geotextile (1200 g/m^2)	Municipal solid waste
	0.5 m mineral liner ($k=10^{-9}$ m/s)	Inert waste
France	5 m in-situ clay* + GM	Industrial, hazardous, Municipal solid, commercial waste
Germany	0.5 m compacted clay*	Inert waste
	Natural (geological) barrier or compacted soil subgrade + 0.75 m mineral liner* ($k=10^{-10}$ m/s) + GM (2.5 mm thick) plus protective layer	Municipal solid waste
	3 m compacted soil subgrade + 1.5 m mineral liner* + GM (2.5 mm thick) plus protective layer	Hazardous waste
Sweden	Compacted clay or GCL or HDPE geomembrane or LDPE geomembrane (thickness > 1 mm)	Municipal solid waste
Switzerland	0.5 m compacted clay + Geomembrane or asphalt layer (thickness > 0.07 m and porosity <3% + geomembrane, subsoil thickness 10 m	Municipal solid waste
	0.8 m compacted clay	Inert waste
United Kingdom	1.5 m compacted clay* + GM (thickness > 2mm)	Municipal solid waste
Africa		
South Africa	Compacted clay (thickness ns) + leak detection + GT + compacted clay (thickness ns)	Municipal solid waste (leachate generating landfills)
	Compacted clay (thickness ns) + dessication protection layer	Municipal solid waste (dry areas)
Asia		
Japan	Low permeability soil or GCL, sand-cement-bentonite mix, asphalt layer, double geomembrane	Municipal solid waste (fly ash)
North America		
USA (EPA)	Composite liner: 0.6 m compacted clay* + GM	Municipal solid waste
	Double liner : 0.9 m compacted clay* + GM, and GM separated by blanket drain	Hazardous waste
Canada		
British Columbia	2 m compacted clay* (natural attenuation landfill), 1 m compacted clay* (engineered landfill)	Municipal solid waste
Ontario	Double liner: 0.75 m compacted clay* + GM, 1 m soil subgrade**	Municipal solid waste – larger landfills
	0.75 m compacted clay* + GM, 3 m soil subgrade**	Municipal solid waste – smaller landfills
Quebec	0.6 m compacted clay* + double GM separated by a leak detection layer, 0.6 m subsoil	Municipal solid waste

*minimum thickness

ns= non specified

** These are not prescribed requirements, but designguidelines. It is possible to demonstrate that natural K at a site is sufficiently low so that an engineered liner is not required.

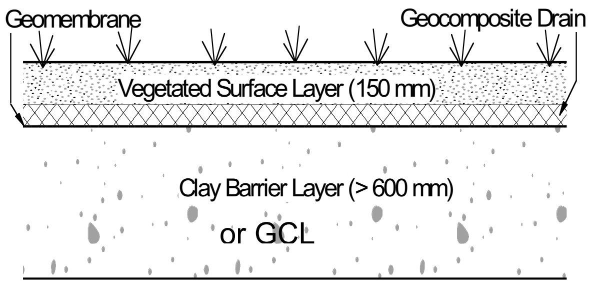


Figure 4. Composite final cover

These will present the geosynthetics industry with more challenges to face in upcoming years. Regulations dealing with final covers for hazardous waste landfills require that final covers be designed and constructed to provide long term minimization of migration of liquids through the landfill and function with minimal maintenance. Geomembranes are an essential component of the composite liner systems use in these types of landfills.

It is worth noting that the design of a landfill bottom liner and cover systems is generally based on either a prescriptive standard or a performance standard. Most of the present regulations around the world belong to the prescriptive design standard type, only recently some countries have introduced performance standards as an alternative to the requirements for a minimum liner or cover system profile The introduction of performance design and its increasing acceptance by the geoenvironmental community has led to a redefinition of landfill design criteria. The implementation of performance-based approaches requires that the design engineer takes into account numerous aspects such as: transport parameters and service life of the mineral barriers, drainage layers, geosynthetics, and the main features of the waste in order to be able to estimate the leachate quality and production over the landfill activity and post closure period. Information on the advantages of prescriptive design standards and performance design standards are discussed in details in Estrin and Rowe (1995) and Manassero et al. (1998).

4 GEOSYNTHETICS IN BOTTOM LINER SYSTEMS

The primary objective of a bottom liner, or a barrier layer, is to prevent or reduce the migration of potentially harmful chemicals, or contaminants, into the surrounding environment. With respect to this objective, several different types of liners are used for the containment of waste (Manassero et al. 2000; Rowe 2001; Benson 2001) and they can vary significantly in complexity. The simplest bottom lining systems consist of a compacted clay liner, geosynthetic clay liner, or a geomembrane liner overlain by a granular collection layer. A more sophisticated and effective lining system incorporates a composite liner comprised of a geomembrane placed directly on top of a clay liner or other type of soil liner. Interest in reducing leakage rates below those achieved with clay liners has resulted in the use of composite liners consisting of a compacted clay liner overlain by a geomembrane. In some cases, a geosynthetic clay liner is used in lieu of the compacted clay liner. Geomembranes are infrequently used alone because they inevitably contain defects, and these defects can result in large leakage rates (Giroud & Bonaparte 1989a, b; Katsumi et al. 2001). The key considerations in the design of bottom liners include (Rowe 1999): (1) potential for advective transport (sometimes referred to as leakage); (2) potential for diffusive transport; (3) potential for natural attenuation (e.g., sorption, biodegradation, and dilution); (4) service life of the liner (i.e. how long can it be relied upon to control advective transport to the design level); (5) potential geotechnical problems (e.g. stability problems; differential settlement).

This section will discuss some of the advances made in the recent years in the design of bottom liner systems. Specifically, the section will focus on geosynthetic clay liners and service life of geomembranes.

4.1 *Geosynthetic clay liners*

Over the past decade, design engineers and environmental agencies have shown a growing interest in the use of geosynthetic clay liners (GCLs) in conjunction with a compacted clay liner or as a replacement to a compacted clay liner. This stems from the fact that they often have very low hydraulic conductivity to water ($k_w < 10^{-10}$ m/s) and relatively low cost. The main advantages of the GCL are their limited thickness, good compliance with differential settlements of underlying soil or waste, easy installation and low cost. On the other hand, the limited thickness of this barrier can produce: (1) vulnerability to mechanical accidents, (2) limited sorption capacity, and (3) an expected significant increase of diffusive transport if an underlying attenuation mineral layer is not provided. Moreover, when hydrated with some types of leachates instead of pure water, bentonite will show a minor swelling that will result in reduced efficiency of the hydraulic barrier. Advantages and disadvantages of GCLs are summarized in Table 4. As the use of the GCLs broadens, they are being investigated intensively, especially in regard to their hydraulic and diffusion characteristics, chemical compatibility, mechanical behaviour, durability and gas migration (e.g. Bouazza et al. 1996; Petrov et al. 1997a, b; Fox et al. 1998; Daniel et al. 1998; Lake & Rowe 2000; Shackelford et al. 2000; Mazzieri & Pasqualini 2000; Vangpaisal & Bouazza 2001; Vasko et al. 2001; McCartney et al. 2002).

4.1.1 *Hydraulic conductivity, chemical compatibility and diffusion*

The hydraulic performance of GCLs depends in most cases on the hydraulic conductivity of the bentonite. The only exceptions are GCLs containing a geomembrane where the geomembrane is seamed during construction (e.g., with a cap strip). In general, laboratory hydraulic conductivities to water of different types of geotextile supported GCLs vary approximately between 2 x 10^{-12} m/s and 2 x 10^{-10}m/s, depending on applied confining stress (Fig. 5). Petrov et al. (1997a), attributed the reduction in GCL hydraulic conductivity to lower bulk void ratios resulting from higher confining stresses. More importantly, they showed that there is a strong correlation between the bulk void ratio and the hydraulic conductivity, k, for a given permeant.

Table 4. Advantages and disadvantages of GCLs (from Bouazza 2002).

Advantages	Disadvantages
•≥ Rapid installation	•≥ Low shear strength of hydrated bentonite (for unreinforced GCLs)
•≥ less skilled labour	•≥ GCLs can be punctured during or after installation
•≥ Low cost	•≥ Possible loss of bentonite during placement
•≥ Very low hydraulic conductivity to water if properly installed.	•≥ Low moisture bentonite permeable to gas.
•≥ Can withstand large differential settlement	•≥ Potential strength problems at interfaces with other materials
•≥ Excellent self healing characteristics	•≥ Smaller leachate attenuation capacity
•≥ Not dependent on availability of local soils	•≥ Possible post-peak shear strength loss
•≥ Easy to repair	•≥ Possible higher long term flux due to a reduction in bentonite thickness under an applied normal stress
•≥ Resistance to the effects of freeze/thaw cycles.	
•≥ More airspace resulting from the smaller thickness.	•≥ Possible increase of hydraulic conductivity due to compatibility problems with contaminant if not pre-hydrated with compatible water source.
•≥ Field hydraulic conductivity testing not required.	
•≥ Hydrated GCL is an effective gas barrier	
•≥ Reduce overburden stress on compressible substratum (MSW)	•≥ Higher diffusive flux of contaminant in comparison with compacted clay liners.
	•≥ Prone to ion exchange (for GCLs with sodium bentonite)
	•≥ Prone to dessication if not properly covered. (at least 0.6 m of soil).

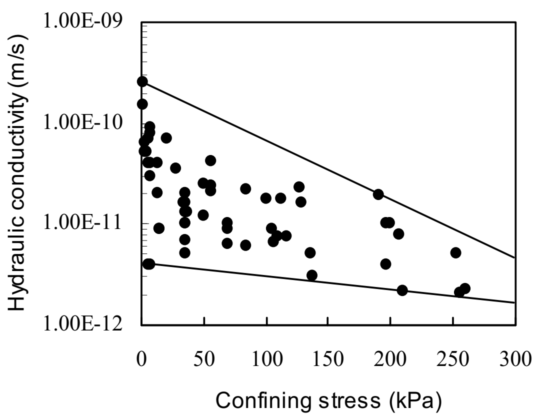


Figure 5. Variation of hydraulic conductivity versus confining stress (from Bouazza 2002).

GCLs often are used to contain liquids other than water. In this case, the evaluation of hydraulic conductivity of GCLs when acted upon by chemical solutions is of a paramount importance. Hydraulic conductivity to the actual permeant liquid is usually assessed via a "compatibility test" where the specimen is permeated with the liquid to be contained or a liquid simulating the anticipated liquid. GCL compatibility with various permeants has been studied by a number of researchers and evaluated for numerous projects (Shan & Daniel 1991; Rad et al. 1994; Ruhl & Daniel 1997; Petrov et al. 1997a, b; Petrov & Rowe 1997; Rowe 1998; Mazzieri et al. 2000; Jo et al. 2001). It should be noted that all studies have concentrated on the GCL short term behaviour. The GCL features, which influence their hydraulic conductivity with liquids other than water; are: aggregate size, content of montmorillonite, thickness of adsorbed layer, prehydration and void ratio of the mineral component. On the other hand, the main factors related to the permeant that influence the hydraulic conductivity are: concentration of monovalent and divalent cations. When performing these tests, it is important to monitor the chemical composition in permeant influent and effluent and that sufficient pore volumes of the permeant has passed through the sample to ensure that chemical equilibrium has been reached. Furthermore, it is recommended that the height of the GCL be constant before terminating these types of tests. A detailed summary of issues related to GCL short term chemical compatibility is provided by Rowe (1998).

A method that has been suggested to prevent alterations in hydraulic conductivity of GCLs is prehydration (Daniel et al. 1993) i.e., when the GCL is hydrated with water. Daniel et al. (1993) found that when partially saturated GCLs were permeated with concentrated hydrocarbons, the hydraulic conductivity was high for a low initial water content. At water content of 100 % the hydraulic conductivity for the hydrocarbons was close to that of water. They concluded that GCLs would not be affected by chemical permeant liquids, provided that the prehydration water content exceeds 100%.

Vasko et al. (2001) evaluated how prehydration water content affected the hydraulic conductivity of GCLs permeated with divalent salt solutions. Their results are shown in Figure 6, prehydration water content was found to not have any apparent effects on hydraulic conductivity for the intermediate and weaker solutions. For the stronger solutions, lower hydraulic conductivity was obtained with higher prehydration water content. The hydraulic conductivity decreased from $1x10^{-6}$ m/s to $3x10^{-9}$ m/s as the prehydration water content increased from 9% to 150% and then remained constant as the prehydration water content increased.

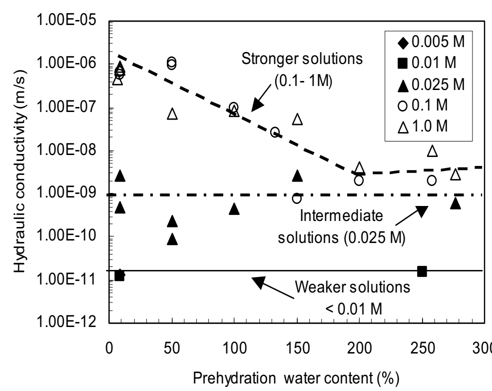


Figure 6. Hydraulic conductivity versus prehydration water content for unconfined GCL samples (modified from Vasko et al. 2001)

Vasko et al. (2001) indicated that the benefits accrued by hydration with water followed by permeation with a non wetting organic liquid (as obtained by Daniel et al. 1993) are not obtained when the permeant liquid is a wetting aqueous solution. This difference was attributed to the different hydration mechanisms involved when the GCL is in contact with a wetting and non- wetting liquids. Another possibility was that the tests conducted by Daniel et al. (1993) were terminated before equilibrium was established.

Long-term chemical compatibility of GCLs is another important issue that has received recent attention (Shackelford et al. 2000; Benson 2001). GCLs have low hydraulic conductivity to tap water because they contain in most cases sodium bentonite. The presence of sodium in the exchange complex permits osmotic swelling of the bentonite, which drastically reduces the size of the pores and the volume of the pore space that is actively involved in flow. However, if sodium is exchanged for cations with higher valence (e.g., Ca^{++} or Mg^{++}, which are common in leachates), osmotic swelling does not occur and the bentonite becomes orders of magnitude more permeable (Shackelford et al. 2000; Jo et al. 2001). The hydraulic conductivities reported by Jo et al. (2001) for a GCL permeated with different single species salt solutions (0.1 M solutions with divalent or trivalent cations) were approximately 10^{-7} to 10^{-6} m/s. Whereas, the hydraulic conductivity to deionised (Dl) water was approximately 2 x 10^{-11}m/s. Benson (2001) pointed out that at low concentrations, the exchange process occurs very slowly due to mass transfer limitation that occur between the bulk pore water and the interlayer water between the montmorillonite layers. The laboratory hydraulic tests on GCL using dilute $CaCl_2$ solutions reported by Benson (2001) show that after nearly a year of permeation, equilibrium has not been established and the hydraulic conductivity continues to increase gradually (Fig. 7). This raises the possibility that much longer times will be required for equilibrium to occur in the field.

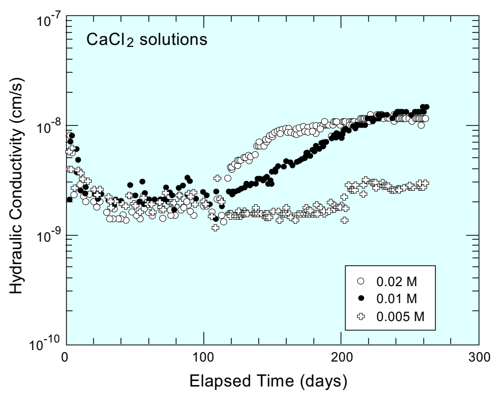
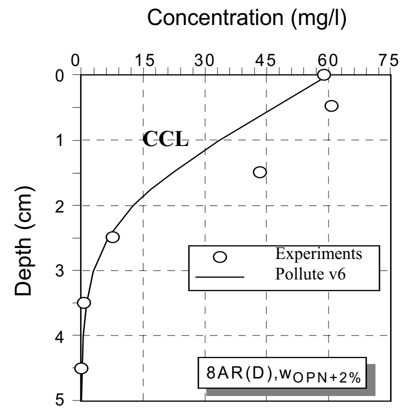


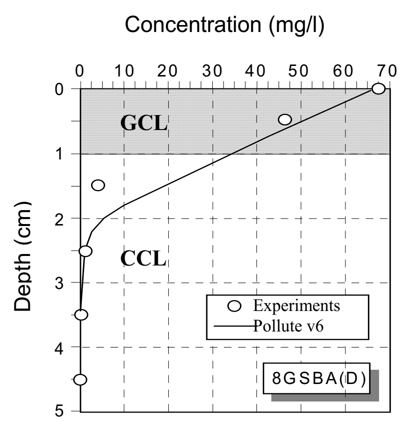
Figure 7. Hydraulic conductivity of a GCL to dilute $CaCl_2$ (from Benson 2001)

Diffusion is a chemical process involving contaminant migration from areas of higher concentration to areas of lower concentration even when there is no flow of water. The diffusive behaviour of inorganic contaminants through a GCL has been reported recently by Rowe (1998) and Lake & Rowe (2000). Their main findings can be summarized as follows: (1) void ratio and related confining stress have a strong influence on diffusion coefficient; (2) the diffusion coefficients due to the modification of the micro-structure of the sensitive mineral component (in particular sodium bentonite); GCL manufacture process was found to not significantly affect the diffusion coefficient. However, it is worth noting that Lake & Rowe (1999) indicated that a slight reduction in diffusion coefficient could be achieved when using a thermal locked GCL compared to a non-thermal locked GCL. This is simply due to the increased bonding of the fibres to the carrier geotextile in the thermal locked GCL, which reduces the swelling thus the bulk void ratio.

Roque (2001) conducted a series of diffusion tests of inorganic contaminants on a needle punched GCL placed directly on top of a compacted clay liner (CCL) to simulate the case when a GCL is used as an augmentation to a CCL. Some of his results are shown in Figure 8. The effective diffusion coefficients of potassium, zinc and cadmium were found to be 1.5 times, 2.8 times and 2.2 times, respectively, smaller than those obtained from the diffusion tests carried out on the compacted clay liner. This means that these inorganic contaminants were adsorbed to varying degrees onto the bentonite mineral surface. These results suggest that the use of a GCL can lead to a reduction in the depth of diffusive migration of these chemicals. Under these conditions, the use of the GCL in the bottom liner can contribute to the reduction in the CCL thickness.



(a)



(b)

Figure 8. Variation of zinc concentration with depth; (a) compact clay, (b) GCL + compacted clay: Experimental results and analytical solutions (modified from Roque 2001)

4.1.2 *Punctures, bentonite thinning, internal erosion*

GCLs are susceptible to accidental punctures, which may occur during handling and installation. In this respect, their hydraulic performance can be compromised depending on the level of damage incurred. It has been shown that small penetrations or defects can be effectively sealed by the sodium bentonite in the GCL, with a minor increase in the hydraulic conductivity of the damaged specimen compared to intact specimens (Shan & Daniel 1991; Bouazza et al. 1996; Mazzieri & Pasqualini 2000). Furthermore, the healing kinetics of open holes up to 30 mm diameter show that only a short time (15 days) is necessary to heal the defect (Didier et al., 2000b). More importantly, Didier et al. (2000b) found that the stability of the self healing area depended on the hydraulic head, it was observed that failure of the self healed area occurred when the hydraulic head was higher than 1 m (under a 10 kPa confinement). Although it is established that the self healing capacity of sodium bentonite GCLs is high, experimental evidence published recently show this capacity can be impeded if the self healing process is coupled with ion exchange (Lin & Benson 2000; Mazzieri & Pasqualini 2000).

A number of case histories related to GCL in-situ defects have been reported in the literature. Mazzieri & Pasqualini (1997) reported on a case where an adhesive bonded GCL was punctured by plant roots, resulting in an increase on the hydraulic conductivity. However, Daniel (2000) pointed out that the source of high hydraulic conductivity was likely to be the root itself, not the seal between the bentonite and the perimeter of the root. This was further confirmed by Didier et al. (2000b) who showed that a very good seal can be obtained around objects inserted in GCLs. Peggs & Olsta (1998) describe a case study where a GCL was severely punctured by the subgrade stones and compromised its hydraulic performance, but this was more a design issue rather than a performance issue.

The hydraulic performance of geotextile supported GCLs depends also on the distribution of bentonite mass/area within the material. Once hydrated, the bentonite has a very low shear strength. Consequently, it is possible that stress concentrations and permanent structural loads cause the bentonite to squeeze laterally, leading to a local reduction in thickness. The reduction in thickness would in turn cause a higher liquid flux at these locations (Koerner & Narejo 1995; Fox et al. 1996). To avoid local bentonite displacement, and consequent possible impact on the hydraulic performance of a GCL, a cover soil of suitable thickness and particle size should be placed over a GCL before it hydrates and before it is subjected to concentrated surface loads. The presence of coarse grained material, such as gravel, overlying a GCL can also cause bentonite migration due to stress concentration. However, it was found that the effect on hydraulic conductivity is insignificant even at high confining stress (Fox et al. 1998b, 2000). Another potential source of stress concentration is the presence of wrinkles in an overlying geomembrane, these may create a void or area of reduced stress into which bentonite in an underlying GCL could migrate (Stark 1998). The choice of subgrade is another important consideration for the installation of GCLs. Like the cover soil, the subgrade on which the GCL is installed should have suitable particle size. Daniel (2000) discusses steps to be taken to minimize bentonite thinnning in GCLs

The process of internal erosion involves the movement of fine particles due to the presence of a high hydraulic gradient (typical in fluid containment facilities). Stam (2000) reported a case where abnormal leakage was observed in a GCL lined lake. Excavation of the installation revealed areas of "patchy" bentonite piping through the lightweight nonwoven geotextile of the GCL into the coarse sand subgrade to a depth of 15-20 cm. Orsini & Rowe (2001) and Rowe & Orsini (2002) presented recently results of an investigation into the performance of GCLs with a 6 mm pea gravel subgrade. Their work showed that at high gradients GCLs with conventional woven or non woven carrier geotextiles are more susceptible to internal erosion than GCLs having a scrim reinforced carrier geotextile. In this respect designs invoving GCLs over a gravel subgrade should be subject to careful scrutiny. Another scenario that can be considered is a geotextile supported GCL overlying a leachate collection layer (coarse grained material or geonet). The possible accumulation of bentonite fines in the drainage layer may have a detrimental effect on the hydraulic transmissivity of the drainage layer and lead eventually to the failure of the leachate collection system. Giroud & Soderman (2000) provide a detailed analysis of the mechanisms and consequences of bentonite migration from a GCL. They proposed a criterion for acceptable bentonite migration. The criterion sets the limit for acceptable bentonite migration, into a geonet drainage layer, at 10 g/m^2. At this limit the drainage layer is not significantly affected. Another way of avoiding bentonite loss from geotextile supported GCL is to use an additional geotextile filter between the GCL and the drainage layer (Estornell & Daniel 1992).

4.1.3 *Equivalency geosynthetic clay liners-compacted clay liners*

The performance design trend imposes the quantitative evaluation of the equivalence of alternative liners and traditional liners. Therefore, in order to quantify the equivalence between GCLs and CCLs, the following main features and parameters of GCLs should be evaluated (Rowe 2001): (1) the hydraulic conductivity of GCLs permeated with non-standard liquids, (2) the effect of holes on GCL hydraulic conductivity, and (3) the diffusion and sorption parameters of GCLs. Rowe (1998), Shackelford et al. (2000) and Lake & Rowe (2000) provide insight regarding equivalence demonstration. The comparison of GCL versus CCL in terms of actual performance is one of the most relevant topics for the engineers involved in landfill design, construction, management and regulation. Moreover, when comparison between different products must be carried out, it is important to keep in mind that it is not possible to generalize about "equivalency" of liner systems since what is "equivalent" depends on what is being compared and how it is being compared (Rowe 1998). The performances of liner systems are also related to the contaminant amount, concentration and decay parameters, the aquifer characteristics and its distance from the bottom of the landfill, the efficiency of capping and drainage systems. A qualitative comparison of GCLs and CCLs, provided by different authors referring to different criteria is given in Table 5. The performance of a GCL, for most criteria, should be either equivalent to or exceed that of a CCL.

Nevertheless, a tentative procedure to compare the performance of different kinds of double layer barriers such as those sketched in Figure 9 can be estimated considering steady state conditions of contaminant flux and taking into account advection and diffusion phenomena (Manassero et al. 2000).

Table 5. Potential equivalency between geosynthetic clay liners and compacted clay liners (from Manassero et al. 2000)

Category	Criterion for Evaluation	Equivalency of GCL to CCL			
		GCL Probably Superior	GCL Probably Equivalent	GCL Probably Inferior	Site or Product Dependent
Construction Issues	Ease of Placement	X			
	Material Availability	X			
	Puncture Resistance			X	
	Quality Assurance	X			
	Speed of Construction	X			X
	Subgrade Condition	X			
	Water Requirements	X			X
Contaminant Transport Issues	*Attenuation Capacity*			X[(1)]	X
	Gas Permeability				X
	Solute Flux and Breakthrough Time	X[(2)]		X	
Hydraulic Issues	*Compatibility*	X[(2)]		X	
	Consolidation Water	X			
	Steady Flux of Water				
	Water Breakthrough Time		X		
Physical/ Mechanical Issues	Bearing Capacity				X
	Erosion				X
	Freeze-Thaw	X			
	Settlement-Total		X		
	Settlement-Differential	X			
	Slope Stability				X
	Wet-Dry	X			

(1) Based only on total exchange capacity, TEC
(2) Only for GCLs with a geomembrane

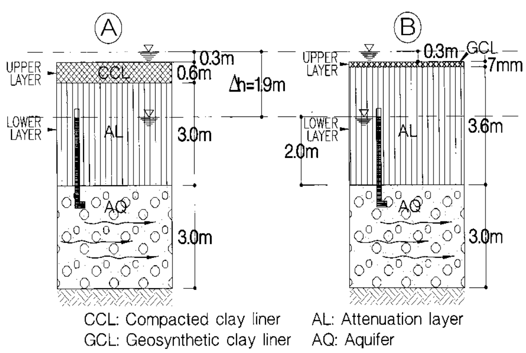


Figure 9. Cross section of a CCL based liner and a GCL based liner

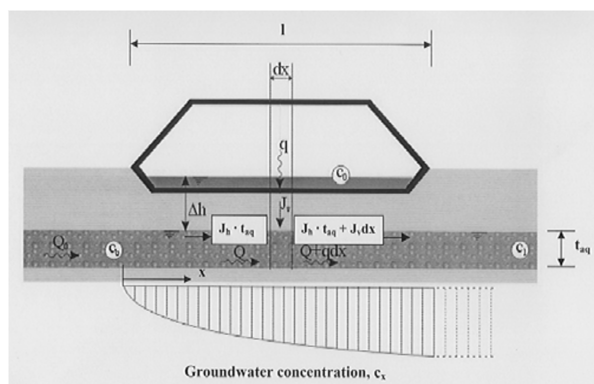


Figure 10. Schematic for the evaluation of pollutant mass balance in steady state conditions.

Considering a typical landfill containing persistent pollutants and underlain by a flushing aquifer (Fig. 10), the steady state condition is, in general, the most critical, at least, in terms of amount of contaminant flux. On the basis of the aforementioned assumptions, the current steady state contaminant vertical flux, J_v, per unit area of the bottom barrier, can be represented by the following equation (Manassero et al. 2000):

$$J_v = q\frac{c_0\ e^{\frac{q}{\Lambda}} - c_x}{e^{\frac{q}{\Lambda}} - 1} \quad (1)$$

where:
c_0 :contaminant source concentration (in the leachate);
c_x: contaminant concentration in the aquifer at a horizontal distance x from the upstream side of the landfill
$q = \Gamma \Delta h$: Darcy seepage velocity through the barrier;
Δh : total hydraulic head loss across the barrier;

$$\Gamma = \left[\frac{1}{\sum_{i=1}^{j}\frac{L_i}{k_i}}\right] \quad (2)$$

with:
Γ≥≥equivalent hydraulic permittivity of a layered barrier;
k_i = hydraulic conductivities of the i-th layer and;
L_i = thickness of the i-th layer;
j = number of layers;
≥ ≥

$$\Lambda = \left[\frac{1}{\sum_{i=1}^{j}\frac{L_i}{n_i D_i}}\right] \quad (3)$$

with:
Λ≥≥= equivalent diffusivity of a layered barrier≥
n_i = porosity of i-th layer and,
D_i = diffusion-dispersion coefficient of the i-th layer
q/Λ=P≥≥Peclet number
≥

$$R_c = \frac{c_x - c_b}{c_0 - c_b} \quad (4)$$

with:
Rc = relative concentration of the pollutant in the aquifer at distance x from the upstream boundary of the landfill
c_x = aquifer concentration at distance x from the upstream boundary of the landfill
c_b = background concentration in the aquifer upstream the landfill.

Considering the boundary conditions at the landfill bottom barrier as described by Rowe & Booker (1985) (i.e. flushing or mixing aquifer) the evaluation of R_c versus the distance from the landfill upstream side can be carried out by solving the following differential equation resulting from the pollutant mass balance illustrated in Figure 11:

$$\left(\frac{Q_0 + q\cdot x}{q}\right)\left(\frac{dc}{dx}\right) = \frac{e^P(c_0 - c_x)}{e^P - 1} \quad (5)$$

where:
$Q_0 = q_{h0}\cdot t_{aq}$ = volumetric flow of the aquifer (upstream of the landfill) per unit horizontal width perpendicular to seepage direction;
q_{h0} = horizontal Darcy seepage velocity of groundwater upstream the landfill;
t_{aq} = thickness of the aquifer.

The solution of this differential equation under the appropriate boundary conditions can be expressed as follows:

$$R_c = \frac{c_x - c_b}{c_0 - c_b} = 1 - \left(1 + \frac{q\cdot x}{Q_0}\right)^{\left(\frac{e^P}{1-e^P}\right)}$$

or

$$\frac{c_0 - c_x}{c_0 - c_b} = \left(1 + \frac{q\cdot x}{Q_0}\right)^{\left(\frac{e^P}{1-e^P}\right)} \quad (6)$$

Equation 6 can be represented in a simplier form by the plots of Figures 11 and 12. The contaminant mass flux in the aquifer below the barrier at a distance x from the upstream landfill side can be evaluated by the integration of equation 1 along the horizontal direction (x axis):

$$\frac{J_h}{c_0\cdot q_{h0}} = \left(1 + \frac{q\cdot x}{Q_0}\right) - \left(1 - \frac{c_b}{c_0}\right)\cdot\left(1 + \frac{q\cdot x}{Q_0}\right)^{\frac{1}{1-e^P}} \quad (7)$$

A representation in dimensionless units of equation 6 is shown in Figure 13.

In the case the landfill total length (l) referred to the aquifer thickness is lower than l/t_{aq}=100 the operative thickness of the aquifer, $t_{aq}(eq)$, must be evaluated on the basis of the following equation (USEPA 1996):

$$t_{aq}(eq) = \sqrt{2\alpha_z l} + t_{aq}\cdot\left[1 - e^{\frac{q\cdot l}{Q_0}}\right] \quad (8)$$

where α_z is the aquifer dispersivity; and verifying that $t_{aq}(eq) \leq t_{aq}$.
Moreover the value of Q_0 to be used with equations 7 and 8 must be reduced by the factor $t_{aq}(eq)/t_{aq}$.

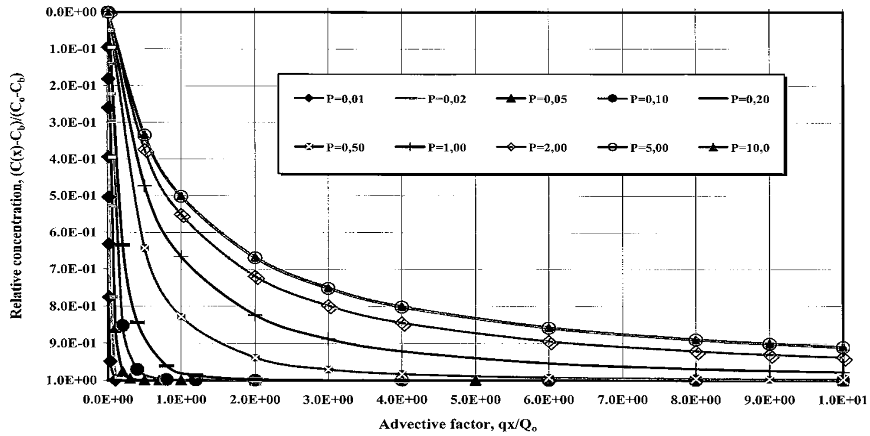


Figure 11. Steady state pollutant concentration in the aquifers below landfills (from Manassero et al. 2000).

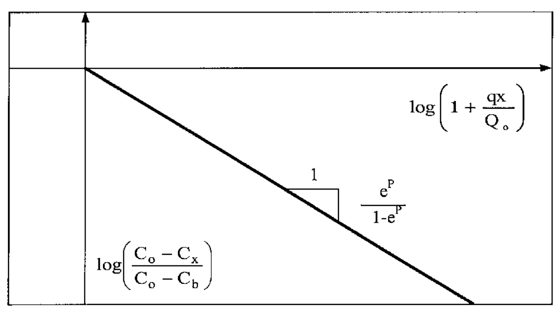


Figure 12. Steady state concentration in the aquifers below landfills vs advective factor of the aquifer and Peclet number of the barrier (from Manassero et al. 2000).

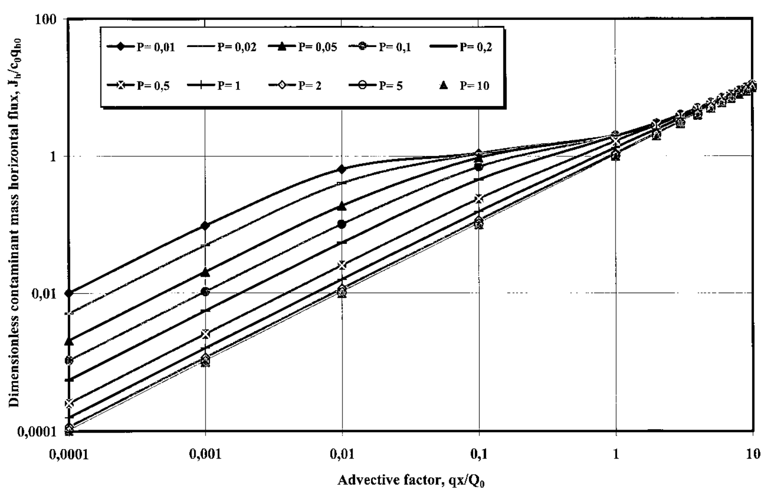
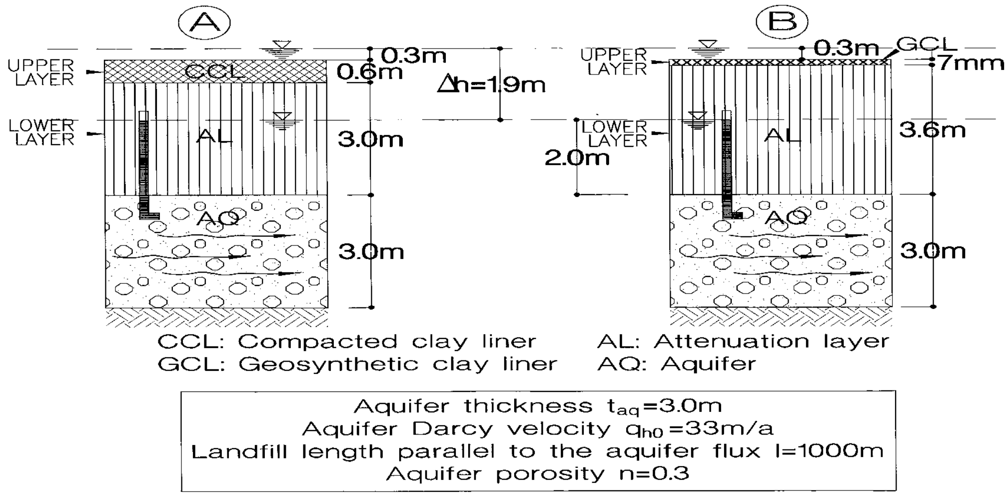


Figure 13. Evaluation of contaminant mass horizontal flux (J_h) in an aquifer with $c_b = 0$ (from Manassero et al. 2000).

Given the limitations previously listed and described, the parameters Γ (permittivity) and Λ (diffusivity) allow an easy comparison, referring to the steady state conditions, among double layer barriers such as CCL plus an attenuation layer (AL) and GCL plus an attenuation layer. The results of the comparison between these two types of liners are given in Figure 14. The input parameters of the two couple of layers have been selected among the current and representative values given by Rowe (1998).



Subscript (b) for upper layer (barrier) parameters

Subscript (a) for lower layer (attenuation) parameters

COMPACTED CLAY LAYER	GEOSYNTHETIC CLAY LAYER
$K_b = 1*10^{-9}$ (m/s)	$K_b = 2*10^{-10}$ (m/s)
$L_b = 0.6$ (m)	$L_b = 0.007$ (m)
$n_b = 0.4$ (-)	$n_b = 0.7$ (-)
$D_b = 6.35*10^{-10}$ (m^2/s)	$D_b = 1.59*10^{-10}$ (m^2/s)
$n_bD_b = 2.54*10^{-10}$ (m^2)	$n_bD_b = 1.1*10^{-10}$ (m^2)
ATTENUATION LAYER	**ATTENUATION LAYER**
$K_a = 10^{-7}$ (m/s)	$K_a = 10^{-7}$ (m/s)
$L_a = 3$ (m)	$L_a = 3.6$ (m)
$n_a = 0.3$ (-)	$n_a = 0.3$ (-)
$D_a = 9.5*10^{-10}$ (m^2/s)	$D_a = 9.5*10^{-10}$ (m^2/s)
$R_a = 1$ (-)	$R_a = 1$ (-)
$n_aD_a = 2.85*10^{-10}$ (m^2/s)	$n_aD_a = 2.85*10^{-10}$ (m^2/s)
$\Gamma = 1.59*10^{-9}$ (s^{-1})	$\Gamma = 1.40*10^{-8}$ (s^{-1})
$\Lambda = 7.76*10^{-11}$ (m/s)	$\Lambda = 7.89*10^{-11}$ (m/s)
$q = 3.02*10^{-9}$ (m/s)	$q = 2.66*10^{-8}$ (m/s)
$Q_0 = q_{h0}*t_{aq} = 3.17*10^{-6}$ (m^2/s)	$Q_0 = V*s = 3.17*10^{-6}$ (m^2/s)
$R_c = 0.661$ (-) @ x = l	$R_c = 0.945$ (-) @ x = l
$J_h/c_0q_{h0} = 1.951$ (-)	$J_h/c_0q_{h0} = 17.34$ (-)

Figure 14. Comparison of steady state transport performances of two liners using GCL and CCL (modified from Manassero et al. 2000).

Looking at the results in terms of R_c and $J_h/(c_0q_{h0})$, the following comments can be made:

- a good CCL plus AL leads to a better long-term performance than a GCL plus AL of the same total thickness.
- The higher contaminant concentration and flux shown by the GCL is mainly due to the parameters Γ_{GCL} and Γ_{CCL}, i.e. due to the higher hydraulic conductivity and then to the advective transport, whereas the two diffusive contributions are fully comparable.
- The advective transport is largely the prevailing contribution to the contaminant migration referring to the barriers considered in the example. Therefore, in this case, a further reduction of diffusion coefficient of both CCL and GCL is ineffective.

In summary, it seems that the permittivity (k/L) is still the critical issue for the GCL, looking in particular at compatibility problems with leachates. On the other hand the diffusive transport is reduced by the contribution of the AL and by the good performance of GCL in itself from this point of view (Tables 7 and 8). In the following part of this paper, it will be shown that the reduction of advective pollutant transport by a geomembrane placed on the top of these mineral barriers can significantly change the conclusions of the comparison shown in the previous example.

Table 7. Chloride diffusion characteristics of some GCLs (Rowe 1998)

GCL	Applied Stress σ_v' (kPa)	Hydrated Thickness t_{GCL} (mm)	Effective Diffusion Coefficient D_e (m^2/s)	Porosity n (-)
1	20	11.1	$3.0x10^{-10}$	0.80
	65	9.1	$2.0x10^{-10}$	0.77
	100	7.1	$1.5x10^{-10}$	0.71
	350	5.6	$0.4x10^{-10}$	0.51
2	25	9.1	$2.5x10^{-10}$	0.77
	140	7.1	$1.6x10^{-10}$	0.68
	280	5.6	$0.7x10^{-10}$	0.64
3	29	11.1	$2.9x10^{-10}$	0.83
	100	7.1	$1.3x10^{-10}$	0.74

Finally, in order to obtain a preliminary idea about the performances of the barriers shown in Figure 14 under transient conditions (i.e. finite mass of contaminant and decay phenomena). It is possible, in a first approximation, to simply compare the following parameters taking into account that the lower value leads to a better performance in terms of both advective and diffusive transport:

$$\frac{\Gamma}{\Omega} \quad ; \quad \frac{\Lambda}{\Omega} \qquad (9)$$

where:

$$\Omega = \frac{\sum R_i L_i}{\sum L_i} \qquad R_i = \frac{\rho_i K_{di}}{n_i}$$

with: R_i = retardation factors of the i-th layer;
≥ ρ_I = dry unit weight of the i-th layer;
K_{di} = distribution coefficients of the i-th layer.

Referring to Table 8, and considering both steady state and transient conditions, it can be observed that GCL barriers can perform better in presence of heavy metals than CCLs (see also Fig. 14). It is worth stressing that the simplified equivalency criteria can give reliable indications only on a case by case basis and referring to specific conditions related to both time and space domains. For more details about GCL parameters and comparison between CCL and GCL see Rowe (1998).

4.2 *Composite Barriers*

Composite liners are now commonly used as standard liner systems. They consist of a geomembrane (usually high density polyethylene, HDPE) overlying a mineral barrier (usually a CCL or, in some cases, a GCL). The advantages of composite liners in terms of advective transport, are apparent especially for poor quality mineral barrier ($k > 10^{-9}$ m/s). There are a large number of factors controlling their performance. In the case of a composite liner, the geomembrane provides the primary resistance to advective contaminant flow and diffusion of some contaminants. The clay component of the composite liner (CCL or GCL) serves to reduce leakage through any holes or defects in the geomembrane and also provides some attenuation of contaminants that can diffuse through an intact geomembrane (see Rowe 1998).

It is possible to compare the performance of composite liners using CCL and GCL via the procedure suggested in Section 4.1. The average Darcy seepage rate (q) through the composite liners must be substituted into equations (6) and (7) as can be seen in Figure 15. The diffusion coefficient of a HDPE geomembrane (GM) can be evaluated from Table 9. As can be observed, the CCL and GCL composite barriers illustrated in Figure 15 are practically equivalent, with the contaminant migration being largely governed by diffusion. Therefore, the importance of the geomembrane is evident in reducing the advective migration of contaminants when typical thickness and related hydraulic conductivity of mineral barriers are taken into account. In this case the geomembrane hides the higher permittivity of the GCL in comparison with the CCL this is the main reason why the overall performance of the two types of composite liners are almost fully equivalent under the given assumptions. On the other hand the pure diffusion coefficient of the geomembrane is in general some order of magnitude lower than that of the mineral layers. However, because the geomembrane is generally very thin, its contribution to the reduction of the diffusive flux is limited, in particular for some organic compounds (see Sangam & Rowe 2001, for more details).

Table 8. Diffusion and linear sorption coefficients of natural and treated clay used in GCLs (Lo 1992; from Rowe 1998)

Liner Material	Effective Diffusion Coefficient D_e (m^2/s)			K_d value (mL/g)			
	Chloride	Lead	1.2 DCB	Lead (pH-7)	1,2 DCB	1,2,4 TCB	1,2,4,5 TECB
CL	$2.4x10^{-10}$	$5.9x10^{-10}$	$9.8x10^{-11}$	6000	1.4	2.2	10
Organo-Clay	$4.9x10^{-10}$	$9.0x10^{-10}$	$1.5x10^{-10}$	140	609	1320	4500
HA-A1OH-Clay	$3.6x10^{-10}$	$7.6x10^{-10}$	$1.2x10^{-10}$	417	20	38	254

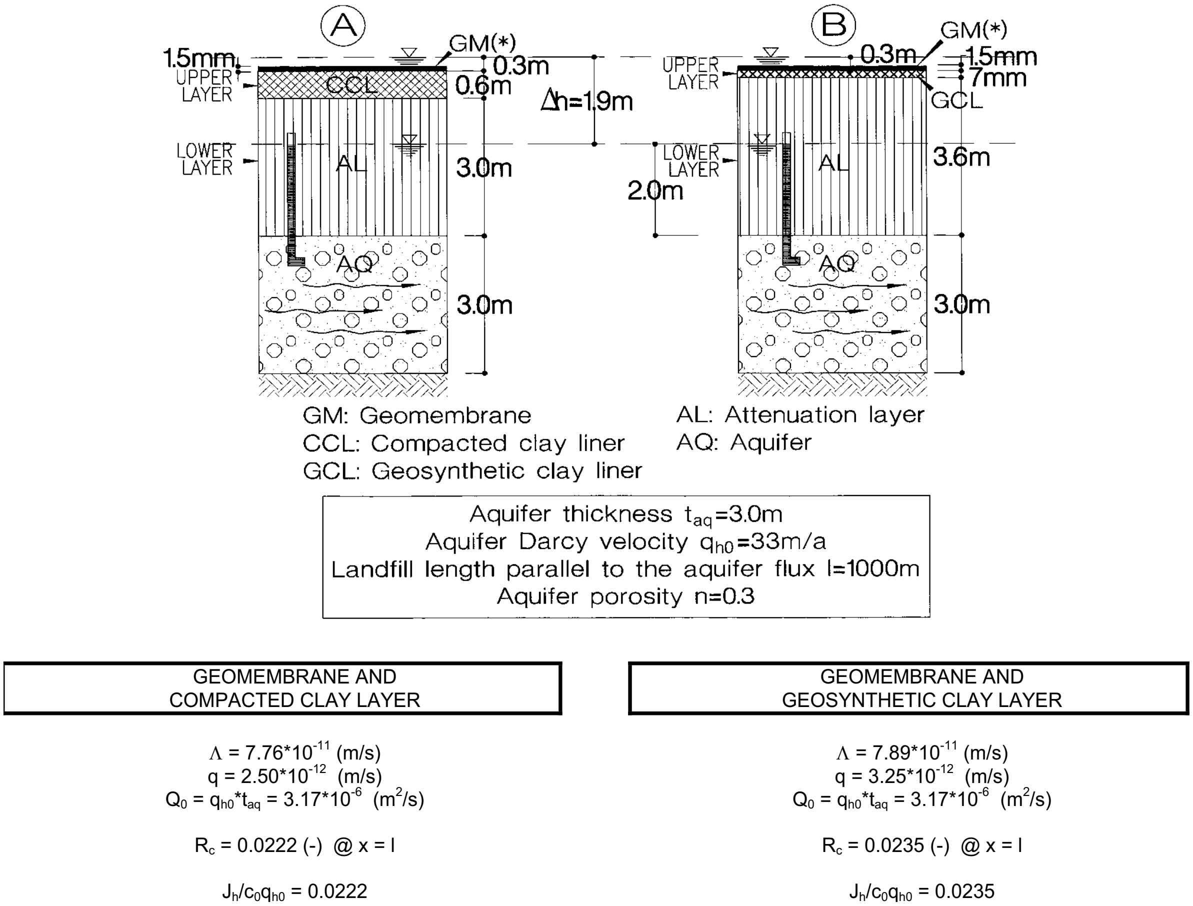


Figure 15. Comparison of steady state transport performances of two composite liners using GCL and CCL (see also Figure 14) (modified from Manassero et al. 2000)

Table 9. Approximate values of diffusive permeability (P_g) of HDPE geomembranes (Rowe 1999)

Permeant (aqueous)	P_g (m^2/s)	P_g (m^2/a)
Toluene	$0.3\text{-}1x10^{-10}$	$1\text{-}3x10^{-3}$
Trichloroethylene	$3\text{-}9x10^{-11}$	$1\text{-}3x10^{-3}$
Benzene	$\sim 2x10^{-12}$	$\sim 7x10^{-5}$
Dichlomethane	$\sim 2x10^{-12}$	$\sim 7x10^{-5}$
Methyl ethyl ketone	$3\text{-}8x10^{-13}$	$1\text{-}3x10^{-5}$
Acetic acid	$< 4x10^{-15}$	$< 1x10^{-7}$
Choloride	$< 5x10^{-15}$	$< 2x10^{-7}$
Water	$< 1x10^{-15}$	$< 3x10^{-8}$

On the basis of the above observations it becomes fundamental to know the service life of geomembranes in order to optimise the landfill liner design. Rowe & Sangam (2002) indicated that the service lives of HDPE geomembranes are essentially controlled by the antioxidants in the material and the service temperature. Sangam (2001) examined the service life of HDPE geomembranes under various exposure conditions scenarios that the geomembrane may be subjected to when used as bottom liners for MSW landfills. These estimates were based on: 1) antioxidant depletion rates inferred for the accelerated tests and 2) the induction time reported by Viebke et al. (1994) for an unstabilised HDPE, and 3) an assumed degradation time of 25 years. It was estimated that, provided that the landfill is well maintained such that the temperature is not higher than 15°C, the primary geomembrane would last at least 200 years whereas for the conditions where the temperature is at 33°C, the service life is estimated to be about 70 years. These service lives have been predicted assuming, in general, good working conditions in a well managed landfill and in particular: good design and construction practice and negligible tensile stress concentration in the geomembrane.

Given the above indications, it can be fully acceptable to design a landfill liner with a certain confidence on the performance of geomembranes in medium and long term (i.e. 50 to 350 years). Moreover this conclusion can be also strengthened by the fact that in many cases, after landfill closure (assuming that a low permeability capping system has been used) and at the end of the service life of the leachate collection system, the seepage velocity through the basal lining system, and therefore, the advective transport of the pollutants toward the underlying aquifer, will be mainly governed by the capping system and by the annual precipitation and climate conditions of the considered region (i.e. hydrological balance of the landfill).

4.2.1 *Service life of geomembranes: Some recent issues*

4.2.1.1 Puncture protection

Most modern landfills will typically incorporate a primary leachate collection system (LCS) consisting of a granular soil layer and a network of perforated pipes, on top of the geomembrane liner. The LCS is intended to control the leachate head on any underlying liner and reduce the potential for advective migration of contaminants. With the recognition of problems associated with chemical and biological clogging of the leachate collection system (e.g., Brune et al. 1991; Rowe et al. 1995; Rowe 1998; Fleming et al. 1999; Rowe & VanGulck 2001), and concerns over the long term efficiency of sand leachate drainage layers have led to the adoption of a coarse drainage gravel to replace the sand to minimize the effects of clogging. A major concern associated with the use of coarse gravel is the effect that it may have on the geomembrane, particularly at the high overburden pressures that may be expected in large landfills. This has resulted in the need to introduce a protection layer between the gravel and geomembrane in order to ensure its long-term integrity. In this respect, the protection layer (e.g., geotextiles, or geomats, or sand filled cushions) is seen as a mean of preventing damage to the geomembrane and also minimizing the concentrated stresses and strains induced in the geomembrane. In either case, an acceptable limit state must be established for geomembrane strain. Presently, two approaches, based on different design philosophy, are used to evaluate the performance of a proposed protection layer.

The first approach seeks to prevent short-term puncture of geomembranes; the second approach seeks to ensure the long term performance of the geomembrane (Tognon et al. 2000). The first design philosophy seeks to prevent local elongation of geomembranes past the yield point, thus allowing deformations while preventing puncture of the geomembrane. There is no upper limit given for the local strain. Wilson-Fahmy et al. (1996), Narejo et al. (1996) and Koerner et al. (1996) provide a basis for protection layer design consistent with this philosophy. The design method focuses on the selection of a nonwoven needle-punched geotextile protection layer with sufficient mass per unit area to provide an adequate global factor of safety against geomembrane yield. This approach is widely used in the United States and governs in most of the cases the acceptability of the protection layer (e.g. Richardson, 1996, Reddy et al., 1996, Reddy & Saichek, 1998a, b; Richardson & Johnson 1998). Along the same philosophy, Badu-Tweneboah et al. (1998) presented another approach for evaluating the effectiveness of geomembrane liner protection. The approach is based on the use of multi-axial tension tests (ASTM D5617) performed on geomembrane specimens after exposure to anticipated field conditions. A criterion based on the geomembrane mode of failure in the multi-axial tension test is used to determine if a certain type of level of mechanical damage is acceptable, this means that for the damage to be acceptable the tensile strain characteristics of the geomembrane must not be significantly affected.

The second design philosophy seeks to limit the development of local strains within the geomembrane, due to a combination of pressures transmitted through the drainage layer, subgrade settlement and waste down-drag, over a long term. In this case the aim of the protection is to avoid the likelihood of environmental stress cracking over time, which can be detrimental to the integrity of the ling system (Seeger & Muller 1996). A 0.25% local strain was set as the limiting value for local deformation (i.e. deformations due to drainage layer impingement) in Germany (DGGT 1997) and the U.K (Environment Agency 1998). This value was arrived at by taking the maximum total allowable strain to be 6%, based on results from HDPE gas line pipe testing studies and applying a factor of safety of 2 to give a total permissible strain of 3% arising from the combined effects of differential settlement, waste down-drag, and drainage layer impingement. A committee chose the arbitrary value of 0.25% based on these considerations (Gallagher et al. 1999; Zanzinger 1999).

Several geosynthetic products are commonly used as protection layer for geomembranes. The specifications can vary from country to country depending on the adopted design philosophy. In the United States, non-woven geotextiles with relatively low mass per unit weight are often used (Reddy et al. 1998; Richardson 1996; Narejo et al. 1996) simply, as mentioned earlier, because the concern is focused on preventing mechanical damage of the geomembrane. Whereas in Europe, in Germany in particular, non woven geotextiles, with larger mass per unit area ($>2000 g/m^2$), are generally used but only in small sized landfills (i.e. in light load situation) (Seeger & Muller 1996). For larger landfills (i.e. heavy load situation), alternative materials such as sand filled geotextiles mats, cushioning geosynthetic products without mineral filling are used (Zanzinger 1999; Gartung 2000).

In order to assess the suitability and ability of a proposed protection layer to meet any performance criteria, a range of tests is available and it is usually linked to the design philosophy put in place. The tests may take the form of index, quasi-performance, performance or field tests. This paper will concentrate only on the performance or field tests since they have been widely used in recent years. Performance tests attempt to mimic site conditions as closely as possible through the use of site-specific materials and representative testing conditions or under operating conditions. The results of such tests are considered applicable to the selection of field protection layers. The most common performance test is that specified under the German technical regulations (BAM 1994) where the test is conducted in a 300mm-diameter pressure container. Gallagher et al. (1999) present a detailed discussion of this laboratory test. In addition several investigations utilising large-scale testing have been undertaken to assess the relative merits of various protection layers. These include field studies on the effects of construction and MSW loading {Reddy et al 1996; Richardson, 1996; Richardson & Johnson 1998; Reddy & Saichek 1998) and large-scale laboratory testing (Zanzinger and Gartung 1998; Zanzinger 1999; Tognon et al. 2000). The results from field studies concurred on the fact that using a non woven geotextile, with low mass per unit area, was sufficient to protect the geomembrane from mechanical damage (i.e. there is no upper limit given for the local strain).

Recent large scale tests have shed some more light on the issue of geomembrane protection. Zanzinger (1999) investigated a range of geosynthetic and composite geosynthetic-mineral protection layer materials (see Table 10). The large scale investigation took place in a 5.4 m by 4.4m by 2.0 m test cell. The loading stress, which was increased over 13 phases within 9 months, created an average vertical stresse of 800 kPa acting on the protection layer and a smooth HDPE geomembrane. In the final phase of the test, a loading stress of up to 1,000 kPa was used over a period of two weeks. A summary of his results is presented in Table 10. The large scale results were also compared to laboratory tests conducted at a stress of 1,350 kPa, applied for 100 hours. No material shown in Table 10 met the performance criterion of $\varepsilon < 0.25\%$ for peak strain, however they were deemed suitable on the basis that the average strain was below 0.25%. The best performance was noted for the rubber-filled material. A much higher peak strain was obtained from the large scale tests (for all tested materials) than from the laboratory tests (quasi performance tests). This difference was attributed to the longer period of stress application and also to variations in subgrade properties (compacted clay in large scale tests and elastic layer in laboratory test).

Tognon et al. (2000) conducted large-scale testing on a variety of protection layer materials (see list in Table 11). The tests were conducted in a 2m by 2m by 1.6m cell, under loads varying between 250 and 900 kPa for a relatively short duration of time (200-720 minutes). Their results are presented in Table 11; similar to the observations made earlier none of the protective material tested met the <0.25% strain requirement. The best performance was observed for the sand-filled geocushion materials and for the rubber geomat with a polyester grid. Tognon et al. (2000) noted the influence of the tensile strength on the performance of some of the products and recommended that consideration be given to tensile properties in addition to the mass per unit area considerations typically utilised in protection layer selection. The poorest performance was noted for the geotextile protection layers selected on the basis of the design formulas presented by Narejo *et al.* (1996). Although no punctures were observed, the short term peak strain of 13% (for GT2) is very close to the tensile yield strain reported by the manufacturer.

Understandably, questions have been raised regarding the ability of protection layers designed in accordance with the Narejo et al. (1996) method to prevent unacceptable impacts on geomembranes. Tognon et al. (2000) suggested that an alternative approach to selecting nonwoven geotextile protection layers based on mass per unit area is worthy of consideration. Such a method would ideally consider the role tensile strength plays in puncture prevention, as this reinforcing mechanism has been identified as being potentially important. Interestingly, Jones et al. (2000) came to a similar conclusion that is the selection of the geotextile protective layer based only on its unit weight can be inappropriate. Their investigation showed that factors such as fibre type and quality, and manufacturing method had a controlling influence on the protective performance.

In summary, the method of selecting protection layers based on performance testing seems to be the most applicable. However, the test results reported by Zanzinger (1999) and Tognon et al. (2000) clearly indicate that even the most robust protection layer materials currently in use are not capable of meeting the <0.25% peak strain requirement. This raises the question of credibility and accuracy of the 0.25% strain criterion. Further investigation in its suitability and accurateness is certainly warranted. Finally, all the recent investigations point to the fact that typical geotextile protection layers are not adequate for controlling the local strains in the geomembrane.

Table 10. Protection Layer Performance Data (Modified from Zanzinger 1999)

Protection Layer	Description	Mass per Unit Area (g/m^2)	Thickness at 2 kPa (mm)	Large-Scale Test		Performance Test	
				Average Strain (%)	Peak Strain, ε (%)	Average Strain (%)	Peak Strain, ε (%)
Geocomposite 1 (Rubber geomat)	GT/rubber	6,650	12.2	0.08	0.41	0.04	0.21
Geocomposite 2	GT-GN-GT	2,030	12.9	0.08	0.66	0.02	0.07
Geocontainer 1 (sand filled cushion)	GT (W) connected with spacer threads, filled on site with sand	70,000	50	0.11	0.75	0.34	0.99
Geocontainer 2 (sand filled cushion)	GT (W) connected with spacer threads, filled on site with sand	56,000	40	0.05	0.46	0.02*	0.19*
GT1	NW, NP	4,270	26.8	0.09	0.58	0.06	0.17
GT2	NW, NP	2,140	13.4	0.10	0.92	0.09	0.26
Geocomposite 3 (Sand geomat)	GT, sand geomat, GT (W, NW)	48,000	26	0.07	0.79	0.07	0.62

Note: GT= Geotextile, GN= geonet, NW= Non woven, NP=needle punched, * sample taken from a landfill construction site, tested at σ_v = 1350 kPa, T = 40°C and t = 1000 hours

Table 11. Protection Layer Performance Data (Modified from Tognon et al. 2000)

Protection Layer	Description	σ_{max} (kPa)	M_A (g/m^2)	H (mm)	Typical Indentation		Maximum Indentation	
					Peak Strain[a] (%)	Peak Strain[b] (%)	Peak Strain[a] (%)	Peak Stra (%)
GT1	NW, NP	250	435	2	1.9	4.0	3.8	8.0
GT2 (two layers)	NW, NP	900	1200	4	1.0	4.8	10	13.0
Sand filled geocushion	GT (NW, NP), sand fill and polymer webbing, GT (NW, NP)	650	70,000	25	0.5	0.52	0.29	0.8
Sand filled geocushion	GT (NW, NP), sand fill and polymer webbing, GT (NW, NP)	900	70,000	25	0.19	0.52	1.4	0.9
Rubber geomat1	GT (NW,NP),Rubber matrix, GT (NW, NP)	600	6,000	10	1.0	3.6	3.6	7.5
Rubber geomat 2	GT (NW, NP), Rubber matrix, Polyester grid	600	6,000	10	0.12	1.1	0.16	1.2

Note: M_A =total mass per unit area, σ_{max}= maximum applied pressure, H= thickness at 2 kPa
(a) estimated from arch elongation, (b) estimated using combined membrane and bending actions

4.2.1.2 *Geomembrane defects and leakage*

Defects in the geomembrane (tears, cuts, etc.) results generally from construction activities and sometimes they can also occur due to poor manufacturing. Other features such as wrinkles may also increase the rate of leakage through such defects. The high coefficient of thermal expansion of HDPE and fluctuations in ambient temperature during installation are well recognized as major factors in the formation of wrinkles (Rowe 1998). Table 12 gives some examples of types of holes or defects that a geomembrane can suffer and possible causes.

The geomembrane component of a composite liner is essentially impervious to liquid flow when devoid of holes or defects. Water can still move through the geomembrane by diffusion, but the water transmission rates are very low. Effective hydraulic conductivities corresponding to water diffusion are on the order of 10^{-14} m/s to 10^{-15} m/s for most geomembranes (Giroud & Bonaparte 1989). However, defects in the geomembrane can occur even with carefully controlled manufacture and damages can be found even in sites where a strict construction quality program has been put in place. For example Rollin et al. (1999) indicated that despite the implementation of good CQA programs on 10 sites surveyed in France and Canada, leaks were detected during the installation phase. The recent papers on leak detection surveys are indicating that most of the damages are occurring after installation of the geomembrane. The survey conducted in 1996 by Nosko et al. (1996) on detection of localized defects in geomembranes indicated that: (1) 25% of the defects occurred during installation (20% inadequate seams and 5% mechanical damage); (2) 73% of the defects were due to mechanical damage caused during placement of the overlying soil; and (3) 2% of the defects occurred during the post construction phase. McQuade & Needham (1999) presented the results of an extensive survey of 111 leak location surveys conducted on landfills and other geomembrane-lined facilities in the U.K. It was found that 48% of these sites (53 sites) had no defects. It was noted that relatively few defects were detected in seams, indicating improved welding methods, testing and CQA of field seams over the past decade. The bulk of the defects were caused during placement of cover materials. A more recent survey by Nosko & Touze-Foltz (2000) summarised the results of electrical damage detection systems installed at more than 300 sites and covering more than 3.250.000 m^2. This survey showed that the majority of the damages (71%) were caused by stones (Fig. 16), followed by heavy equipment (16%). Interestingly, most of the failures (78%) were found to be located in the flat areas of the liner (bottom liner, Fig. 17), only 9% were found at the corners and edges of the landfills.

It is interesting to note from the reported surveys that the bulk of the defects were related to mechanical damage caused by the placement of soil on top of the geomembrane. In this respect the recommendations made by Giroud (2000) should be taken into account to minimize geomembrane installation and post-installation defects.

Table 12. Typical defects and possible causes (from McQuade & Needham 1999).

Stage	Type of defects	Possible cause/comment
Manufacture	Pinholes, excessive thickness changes, poor stress crack resistance	Unusual now for procedures with good quality control. Poor resin
Delivery	Scuffing, cuts, brittle cracks, tears, punctures	Unloading with unsuitable plant or lifting equipment. Impact. Poorly prepared storage areas
Placement	Scratches, cuts, holes, tears,	Dragging sheet along ground, trimming of panels, rough subgrade, use of equipment on top of sheet without protection layer, wind damage, large wrinkles, folds, damage by lifting bars
Welding	Cuts, overheating, scoring, poor adhesion, crimping	Careless edge trimming, welding speed or temperatures incorrect, excessive grinding, dirt or damp in weld area, excessive roller pressure
Cover placement	Tears, cuts and scratches, holes, stress in membrane	Action of earthmoving plant, insufficient cover during placement, careless probing of cover depth, contraction of sheet due to ambient temperature reduction
Post-installation	Holes, tearing, slits, cracks	Puncture from drainage materials, puncture by items of deposited waste, opening of partial depth cuts, pulling apart of poor quality welds, downdrag stresses caused by settling waste, differential settlement in the base

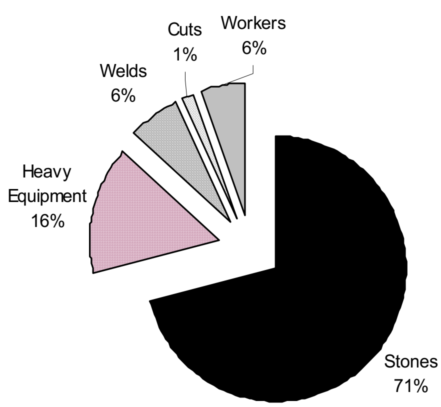


Figure 16. Cause of damage in geomembrane liners (modified from Nosko & Touze-Foltz 2000)

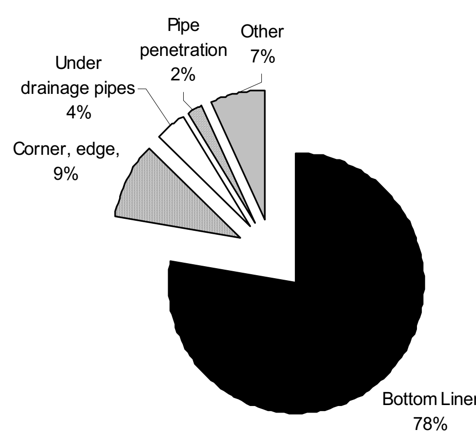


Figure 17. Location of damage in geomembrane liners (modified from Nosko & Touze-Foltz 2000)

Another interesting aspect related to geomembrane defects is the leak density per liner area (i.e., number of leaks per hectare or m^2). The results of the survey presented by McQuade & Needham (1999) indicated a large range in frequency (from 0 to 120 holes per hectare). The average frequency, for the 111 surveyed sites, was 4.2 holes per hectare, although the median of 0.7 holes per hectare was considered more representative of the standard achievable with competent installation and thorough CQA. There was incomplete data relating the number of holes to the presence and rigor of quality control and assurance procedures. For sites where it was known that a thorough program of CQA was implemented, the hole frequency ranged from 0 to 5.7 per hectare, with an average of 0.8 and a median of zero. McQuade & Needham (1999) rightly concluded that the application of a thorough and stringent quality assurance program substantially reduces the frequency of holes in a geomembrane liner. A similar conclusion has been reached by Rollin et al. (1999) from their work on French and Canadian landfills. The density of leaks variation per area of liner surveyed is plotted in Figure 18 based on the data reported by Laine & Darilek (1993), Colucci & Lavagnolo (1995), McQuade & Needham (1999) and Rollin et al. (1999). It is interesting to note that the density of leaks tends to decrease as the surveyed area increases. However, no final conclusion can be drawn on this issue due to the uncertainty linked to the varying conditions encountered in different sites. Colucci & Lavagnolo (1995) pointed out that larger installations tend to have better construction quality program whereas smaller sites have proportionally more complex features to deal with.

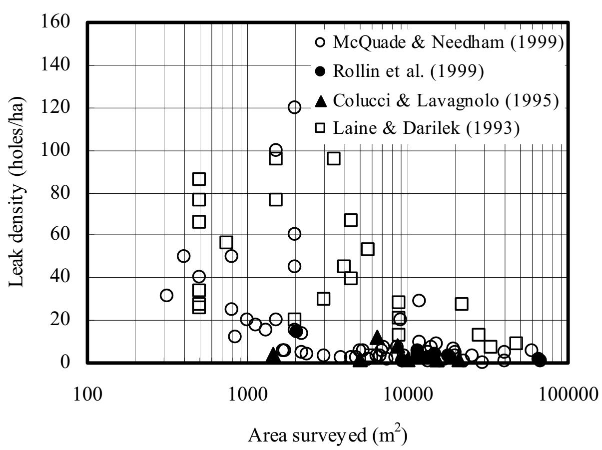


Figure 18. Variation of leak density versus area surveyed.

Many excellent attempts have been carried out in order to predict the rate of leakage through composite liners by calculations based on the fundamental parameters that govern the problem. Excellent summaries of the current methods for holes in direct contact with the underlying mineral liner or GCL may be found in Rowe (1998), Touze-Foltz et al. (1999), Foose et al. (2001), and Giroud & Bonaparte (2001). Another solution proposed by Rowe (1998) takes into account the presence of holes when in contact with a wrinkle in the geomembrane. Touze-Foltz et al. (1999, 2001) and Rowe & Booker (2000) extended this solution to take into account the non uniform transmissivity and small transmissivity, respectively, at the interface GM/CCL or GCL, and the interaction of two wrinkles. The reader is referred to these sources for further information.

4.2.1.3 *Leakage rate from field studies*

For many MSW landfills, a single composite liner is employed. There is no leak detection system. In these cases, one depends on the leachate collection system and the single composite to function properly. It is common to have a system of groundwater monitoring wells to monitor for possible leakage from these facilities. However, in some cases a double liner with a leak detection system (geonet or sand) between the composite liners is proposed for MSW landfills on the basis that the geonet or sand provides a rapid detection system for leaks in the primary composite liner and afford the operator time for a response before contaminants escape the landfill and enter the subsurface. For this reason, at least 8 states in the United States require double lined facilities for non-hazardous wastes (Koerner 1997). A certain number of studies on leakage rates have been made on landfills with secondary leachate collection systems (leak detection systems or LDS) by measuring the flow in these systems. An important aspect, which needs to be taken into account when considering data relating to these flow rates is the source of fluid (Gross et al. 1990). Besides leakage from the composite liner, fluid may enter the leak detection system as (a) infiltration during construction of the system, (b) water from consolidation of the clay liner or other mineral layer, and (c) groundwater infiltration from outside of the landfill. Leakage rates must thus be estimated taking these additional sources of fluid into account. Some of these studies have been summarized by Rowe (1998) (Tables 13 and 14). Referring to these tables, it is necessary to observe that in the case of a CCL or GCL most of the leakage collected by the secondary leachate collection and removal system is attributed, by the authors of the papers quoted in the references, to the consolidation water and not to leakage through the geomembrane. Another study conducted by Tedder (1997) on field performance of active double lined landfills in Florida, USA, reached the same conclusions. Tedder compiled leachate flow from 24 double lined cells. The lining included an HDPE geomembrane (GM) or an HDPE GM + GCL as primary liner and HDPE GM + soil or HDPE GM + GCL as secondary liner. The LDS consisted either of a sand layer or a geonet. The LDS flow rates were reported to be related to construction water expelled from the LDS during increasing overburden pressures and accidental or deliberate discharges of stormwater directly in the LDS. There was no evidence of a chronic leakage problem to the LDS in these cells as would be expected if significant flaws existed in the primary liner. Notwithstanding the above, Rowe (1998) and Smith (2000) pointed out that given the thinness of some of the liners and the fact that consolidation of the liner increases the rate of contaminant transport across the liner there is the potential for organics that readily diffuse through the geomembrane and experience relatively little retardation in clay (such as DCM) to enter into the SLCS rather quickly. This latter aspect warrants further field investigation.

Beech et al. (1998) provide an evaluation of a double liner performance at a MSW landfill in New York, USA. This landfill comprised two cells refereed to as the western cell and the eastern cell both cells have the same lining system. The leakage requirement or action leakage rate (ALR) for both cells was set at 180 lphd by the local authority. Analysis of the daily liquid removal rates from the secondary leachate collection system during pre-operation monitoring showed flows ranging from 50 to 1600 Iphd (Fig. 19). For the western cell, most the detected liquid was attributed to construction and consolidation water. However, in the eastern cell the majority of the liquid was found to come from a 3mm diameter hole in the geomembrane primary liner along the side slope of the cell. This defect occurred despite the fact that a good construction quality assurance (CQA) program was in place during liner construction.

Table 13. Average flow rates in PLCRS and SLCRS for landfills with composite liners involving GM and CCL (in lphd). (Othnam et al. 1996, from Rowe 1998)

Primary Liner				SLCRS		PLCRS		SLCRS		
GM		Clay				Flow Rates		Flow Rates		
Type	Thickness (mm)	Type	Thickness (mm)	Material	Thickness (mm)	Average (lphd)	Peak (lphd)	Average (lphd)	Peak (lphd)	Period
CSPE	0.9	CCL	600	Sand	450	1120	2076	113	260	41-93
HDPE	2.0	CCL	450	Sand	300	4400	5790	59	152	35-54
HDPE	1.5	CCL	900	GN	5	1142	3985	167	275	42-66
HDPE	2.0	CCL	450	GN	5	53	170	1.5	10	34-58
HDPE	1.5	CCL	900	GN	5	1144	1371	60	102	30-37

PLCRS: Primary leachate collection and removal system
SLCRS: Secondary leachate collection and removal system

Table 14. Mean and standard deviations of flow in PLCRS and SLCRS for 6 landfill cells with a GCL as part of a composite primary liner (in lphd) (Bonaparte et al. 1996, from Rowe, 1998)

		Average Flows				Peak Flows			
		PLCRS		SLCRS		PLCRS		SLCRS	
	Cells	Mean	sd	Mean	sd	Mean	sd	Mean	sd
Initial Period	25/26	5.350	3.968	36.6	68.5	14.964	11.342	141.8	259.9
Active Operation	18/19	276	165	0.7	1.1	752	590	7.7	13.7
Post-Closure	4	124	-	0.2	-	266	-	2.3	-

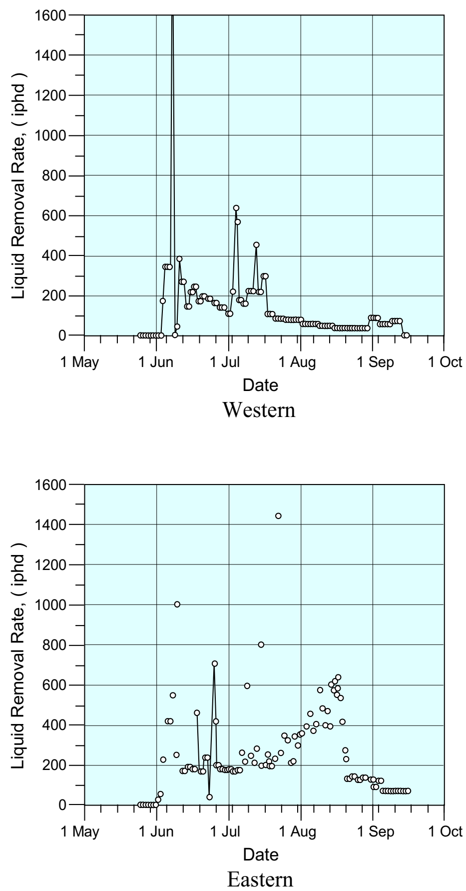


Figure 19. Daily liquid removal from a MSW landfill in New York, USA (from Beech et al. 1998).

A recent study sponsored by the USEPA reviewed the performance of upper liners at sites containing double liner systems containing leak detection systems (Bonaparte et al. 2000, from Qian et al. 2002). The reported data were from as survey conducted on 287 single or multiple cells of 90 double lined landfills, which had up to 10 years of service performance. Composite liners were composed of geomembrane, geomembrane/compacted clay liner and geomembrane/geosynthetic clay liner with either sand or geonet leak detection layer materials. The results of this investigation are presented in Table 15. Bonaparte & Gross (1990) indicated that landfills with a proper quality program had top liner leakage rates less than 500 lphd and typically less than 200 lphd. In this respect the average flow rates, reported in Table 15, over the different life cycle stages indicate that composite liners can achieve a relatively good performance (provided that a proper quality control has been put in place). It can also be seen from Table 15 that geomembrane/geosynthetic clay liner (GM-GCL) systems generally outperform both the geomembrane (GM) and the geomembrane/geosynthetic clay liner (GM-CCL) systems. However, in the light of the work reported by Benson (2001) on long term chemical compatibility of GCLs (see section 4.1.1) the above observation may not be indicative of the long term behaviour of composite liners containing GCLs, because most of the GCLs in the USEPA's field study were in service for less than 5 years. In addition, one can also conclude that the long term performance of the different barrier systems is governed by the water balance of the top layers of the landfill since the flow rates were found to decrease with time or life cycle stages. In summary, the above cases highlight the importance of secondary leachate collection system monitoring during the different life cycle stages of a landfill to make sure that design requirements for leakage prevention are met.

Table 15. Flow rates from the Leachate Detection Systems (LDS's) of Modern Double-Lined Landfills (all flow rates are given in litres/hectare/day) (Modified from Qian et al., 2002, * *based on results from Bonaparte et al. 2000*)

Liner and LDS Type	Type I (GM-Sand)			Type II (GM-GN)			Type III (GM/CCL-Sand)		
Life Cycle Stage	1	2	3	1	2	3	1	2	3
Average Flow	380	170	64	90	100	ND	210	140	64
Minimum Flow	7.6	0.00	0.2	4.8	1.4	ND	1.2	22	0.00
Maximum Flow	2140	1480	240	370	360	ND	1180	660	270
# of "points"	30	32	8	7	11	ND	31	41	15
# of landfills	11	11	4	4	6	ND	11	11	4
Liner and LDS Type	Type IV (GM/CCL-GN)			Type V (GM/GCL-Sand)			Type VI (GM/GCL-GN)		
Life Cycle Stage	1	2	3	1	2	3	1	2	3
Average Flow	170	83	65	130	22	0.3	6.5	2.6	ND
Minimum Flow	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	ND
Maximum Flow	690	500	130	970	280	0.9	34	9.0	ND
# of "points"	21	27	12	19	19	4	6	4	ND
# of landfills	6	9	3	3	3	1	1	2	ND

Notes: Life cycle stages: Stage 1= Initial life; Stage 2: Active life; Stage 3= Post-closure;; "points"= number of measuring points i.e., outlets of single or multiple cells. GM=geomembrane; GN=geonet; CCL= compacted clay liner; GCL=geosynthetic clay liner; ND= no data

5 GEOSYNTHETICS IN SIDE SLOPE LINER SYSTEMS

A waste containment facility side slope liner system must not only provide a sound hydraulic/gas barrier but must also be structurally stable during all phases of a project (i.e. during construction, operation, and closure). In this respect, stability evaluation is a critical consideration for side slope design. Landfill waste containment systems are often composed of several layers of geosynthetics and natural soils. One of the most important problems associated with the use of geosynthetics for landfill linings is their stability when placed on slopes. This aspect is exacerbated by the fact that an increased number of landfills are designed with a small footprint and with moderate to steep slopes to increase their capacity. In this case, slope stability becomes a major issue for containment facilities where composite liners are used, despite all the progress that has been made in the past decade after the massive slide that occurred during filling of the Kettleman Hills hazardous waste landfill in southern California, USA (Mitchell et al. 1990, Seed et al. 1990, Byrne et al. 1992).

5.1 *Interface stability*

The interfaces between the different material layers composing a multi-layered lining system often represent potential slip surfaces that need to be considered in slope stability analyses. The Kettleman Hills failure was a salutary remainder to our profession on the importance of a proper evaluation of the interface strength of the different components of a multilayered system. Since the Kettleman failure new efforts have been made to gain more knowledge of shear resistance of the different interfaces present in liner systems. As a result, a more extensive data base is now avalaible (Bemben & Schulze 1993; Stark & Poeppel 1994; Vaid & Rinne 1995; Pasqualini et al. 1995, 1996; Shallenberger & Filtz 1996; Sharma et al. 1997; Bouazza 1998; Snow et al. 1998; Blumel & Stoewahse 1998; Jones & Dixon, 1998, 2000; Ellithy & Gabr 2000; Wasti & Ozduzgun 2001; Gourc et al. 2001; Stoewahse et al. 2002; Dixon et al. 2002; McCartney et al. 2002). In this respect, very significant progress has been made in understanding and measuring soil/geosynthetic or geosynthetic/geosynthetic interface strengths under different operating conditions. There are several devices currently in use to test the shear strength of the different interfaces present in liner systems including: 1) the large scale direct shear box; 2) the conventional direct shear box; 3) the torsional or ring shear device; 4) the tilt table and 5) the cylindrical shear device.

Table 16 summarises the advantages and disadvantages of these devices.

Various ranges of interface strengths between geosynthetics and mineral liners or geosynthetics collected from available literature are given in Table 17. The wide range of variations observed in Table 17 is due to the variability of the geosynthetics materials, testing conditions, testing protocols and testing equipment as reported by some of the above authors. It is important to stress the fact that published values of interface friction cannot be used for design of a specific project, without at least careful review of test materials, test conditions and test methods. It is of a paramount importance to determine the interface strength on a site specific basis for design purposes.

One of the major concerns with the use of geosynthetics in side slopes is their behaviour when subjected to shear forces. Their stability is controlled by the shear strengths mobilized at the interface between various soils and geosynthetics and sometimes within the geosynthetics themselves. The geosynthetics interfaces generally exhibit strain softening behaviour. This means when these interfaces are sheared, the peak shear strength is mobilized within a small amount of displacement (typically few millimeters) and then the strength decreases to a residual strength at significantly larger displacement. With this type of behaviour there is always a question of whether peak or residual shear strength should be used in the analysis, this uncertainty represents a real dilemma for the design engineer. The forensic work of Mitchell et al. (1990) related to the Kettleman Hills failure investigation pointed out the importance of assessing the residual interface resistance of the different liner components.

Their test results are particularly instructive examples of the values and variability of the geosynthetics interface shear resistance. A number of other technical references highlighted the importance of residual strength resistance and its implication on design (Stark & Poeppel 1994; Bouazza 1998; Jones & Dixon 2000; Filtz et al. 2001; Gilbert 2001; Thiel 2001). The consensus seems to point towards the use of residual strength with a safety factor ≥ 1. Gilbert (2001) stressed the fact that in this case peak and residual strengths are needed for all components in the containment system. The peak strengths are needed to identify the location of slippage, while the residual strengths are needed to establish the residual strength for the system. A similar approach has been proposed by Leshchinsky (2001) for the design of geosynthetics reinforced soils. The readers are advised to consult the above papers to gain a better insight on this particular topic.

Table 16. Summary of advantages and disadvantages associated with test devices for measuring interface shear strength (from Gilbert et al. 1995; Shallenberger & Filtz 1996; Lalarakotoson et al. 1991; Jones & Dixon 2000; Gourc et al. 2001; Marr, 2001; McCartney et al. 2002).

Test device	Advantages	Disadvantages
Large-scale direct shear box	Industry standard Large scale Large displacement Minimal boundary effects Expedient specimen preparation	Machine friction Load eccentricity Limited continuous displacement Limited normal stresses Expensive
Large displacement shear box	Large area of interface Capable of detecting end effects Determination of residual strength with a linear displacement device	Influence of end effects Availability
Conventional direct shear box	Experience with soil Inexpensive Large normal stress Expedient specimen preparation	Small geosynthetic experience base Machine friction Load eccentricity Small scale Limited displacement Boundary effects
Ring shear device	Unlimited continuous displacement	Machine friction Mechanism of shearing not comparable to that exhibited in the field Small scale Expensive No lateral restraint for migration of plastic soils
Tilt table	Minimal machine effects Minimal boundary effects Ability to monitor tensile forces Low normal stress Inexpensive	Limited continuous displacement Limited normal stresses No post peak behaviour
Cylindrical shear	Unlimited continuous displacement Better controlled confinement during shearing Larger sample size with less edge effects Area of shear plane remains constant Constant direction of shear displacement	Availability Experience with dry materials only No restraint for migration of plastic soils

Table 17. Ranges for strength parameters of different interfaces in landfill liner systems (from Manassero et al. 1997).

GEOSYNTHETIC - SOIL INTERFACE

Geomembrane (HDPE) - Sand	ϕ = 15° to 28°
Geomembrane (HDPE) - Clay	ϕ = 5° to 29°
Geotextile – Sand	ϕ = 22° to 44°
Geosynthetic clay liner - Sand	ϕ = 20° to 25°
Geosynthetic clay liner - Clay	ϕ = 14° to 16°
Textured HDPE – Compacted clay	ϕ = 7° to 35° c' = 20 to 30 kPa
Textured HDPE - Pea gravel	ϕ = 20° to 25°
Textured HDPE – Sand	ϕ = 30° to 45°
Geotextile – Clay	ϕ = 15° to 33°

GEOSYNTHETIC - GEOSYNTHETIC INTERFACE

Geonet – Geomembrane (HDPE)	ϕ = 6° to 10°
Geomembrane (HDPE) – Geotextile	ϕ = 8° to 18°
Geotextile – Geonet	ϕ = 10° to 27°
Geosynthetic clay liner - Textured HDPE	ϕ = 15° to 25°
Geosynthetic clay liner - Geomembrane (HDPE)	ϕ = 8° to 16°
Geosynthetic clay liner - Geosynthetic clay liner	ϕ = 8° to 25° c' = 8 to 30 kPa
Textured HDPE – Geonet	ϕ = 10° to 25°
Textured HDPE – Geotextile	ϕ = 14° to 52°

5.2 Geosynthetic clay liners and slope stability

The potential use of GCLs on slopes as part of composite liners may subject them to a complex, long-term state of stress. The primary design concern when GCLs are placed in contact with other geosynthetics or soils on a slope is the interface friction, which must be sufficiently high to transmit shear stresses that may be generated during the lifetime of the facility. Another concern is the possible internal failure of the GCL (within the bentonite or at the interface between the bentonite and geosynthetics in the GCL). The need for a more careful design of lining systems has been stressed by the recent failures generated by slip surfaces along liner interfaces (Byrne et al. 1992; Stark et al. 1998). Much effort has been devoted in the past decade to improve the understanding of the different factors affecting the shear resistance of the different interfaces present in liner systems. As a result, very significant progress has been made in understanding and measuring GCL internal strength and GCL–soil/geomembrane interface strengths.

A comprehensive review concerning GCL internal and interface shear strength testing, as well as an analysis of a large database of direct shear tests on internal and interface GCL shear strength has recently been completed (McCartney et al. 2002). This study compares the relative strengths and weaknesses of different GCL and geomembrane types, focusing on the effects of different conditioning and testing procedures on GCL shear strength (i.e. hydration, consolidation, rate of shearing, normal stress during different stages of testing). Figure 20a shows a set of 320 test results for the internal shear strength of different reinforced and unreinforced GCLs tested under a wide range of conditioning procedures but similar test procedures. All of the tests were conducted by a single laboratory with test procedures consistent with ASTM D6243. Similarly, Figure 20b shows the large-displacement (50-75 mm) shear strength of 187 of the GCLs referred to in Figure 20a. There is significantly less variation in the large displacement shear strength, although the shear strength is still slightly greater than the residual shear strength of unreinforced sodium bentonite. Similar trends in peak and large displacement shear strengths were observed in this study for the GCL-geomembrane interface. The variation in shear strength with changing GCL type and conditioning procedures implies the importance of conducting site and product specific laboratory testing for internal and interface GCL shear strength.

McCartney et al. (2002) propose that the conditioning and testing procedures affect the swelling behavior of the GCL, resulting in variable material properties and either positive or negative excess pore water pressures generated during shearing. Variability associated with the swelling of the GCL is ultimately related to the variability in the internal or interface shear strength. These results are generally consistent with laboratory results conducted in several other studies (Daniel & Shan, 1991, Stark & Eid, 1996, Gilbert et al. 1996, Eid & Stark 1997, Fox et al. 1998a) on the internal strength of unreinforced and reinforced (stitch bonded and needle punched) GCLs. Peak shear strengths for the unreinforced GCL products were found to be similar and comparable to those for sodium bentonite (i.e. very low shear strength), which makes them prone to instability. Because of this, unreinforced GCLs are usually not recommended for slopes steeper than 10H:1V (Frobel 1996; Richardson 1997). On the other hand, reinforced GCLs have greater internal peak strength due to the presence of fiber reinforcements. The behavior of reinforced GCLs has been shown to depend on the resistance against pullout and/or tensile rupture of the fibers reinforcements and the shear strength of the bentonite (at large displacements once the fibers have failed). The peak shear strength of different types of reinforced GCLs (needle-punched, thermal bonded, stitch-bonded) may differ significantly (McCartney et al. 2002). It is worth noting that despite the fact that internal failure of reinforced GCLs could possibly occur in the laboratory, there are no known cases of slope failures that can be attributed to internal shear failure of reinforced GCLs.

Laboratory interface shear tests are routinely conducted to evaluate interface friction between GCLs and soils or geosynthetics under operating conditions. As a result, a more extensive database is now available (Garcin et al. 1995; Bressi et al. 1995; Feki et al. 1997; Gilbert et al. 1996; Von Maubeuge & Eberle 1998; Eid et al. 1999; Triplett & Fox 2001; McCartney et al. 2002). The major finding worth noting is the possible reduction in frictional resistance between a geomembrane and a GCL due to extrusion of bentonite through woven geotextiles and nonwoven geotextiles with a mass of unit area less than 220 g/m^2 into the adjacent geomembrane interface.

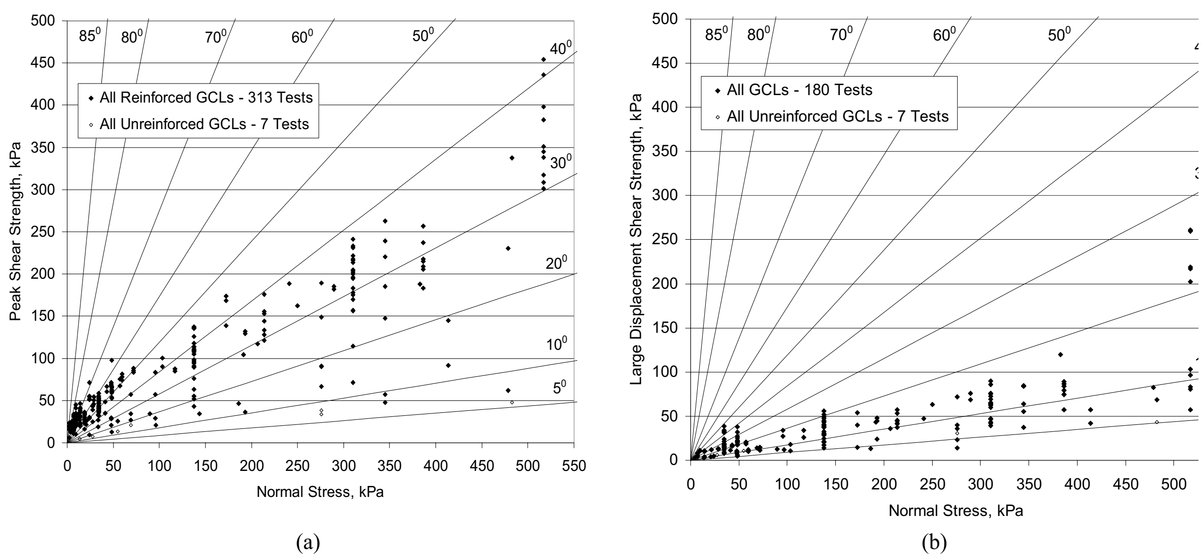


Figure 20. Reinforced and unreinforced GCLs, (a) Peak shear strength (b) Large-displacement shear strength (McCartney et al. 2002)

McCartney et al. (2002) observed that different reinforced GCLs would experience different interface shear strengths, implying that sodium bentonite extrusion from the GCL is related to the internal fiber reinforcements in addition to the conditioning procedures.

Despite the observed difference between internal and interface GCL shear strength, variability may still imply that a prescribed approach to laboratory testing may not be acceptable. McCartney et al. (2002) identified that the variability of both internal and interface GCL shear strengths is a key issue in laboratory testing. Interpretation of this variability is necessary to correctly quantify the shear strength of an interface. For this reason, the use of basic probability principles and reliability based design is necessary to assess the variabilty. Figure 21 shows probability density functions for the peak shear strength of a needle-punched GCL for 19 tests with the same test conditions and procedures. Variability in the internal shear strength is related to the internal fiber reinforcement characteristics as well as changes related to the swelling of the GCL. It is interesting that the interface shear strength has been observed to be only slightly less variable than the internal GCL shear strength. There are many possible factors that may affect bentonite extrusion from the GCL during hydration, as well as variable frictional connections between textured geomembranes and the woven geotextile of the GCL. It should be noted that variability in the results increases significantly with increasing normal stress.

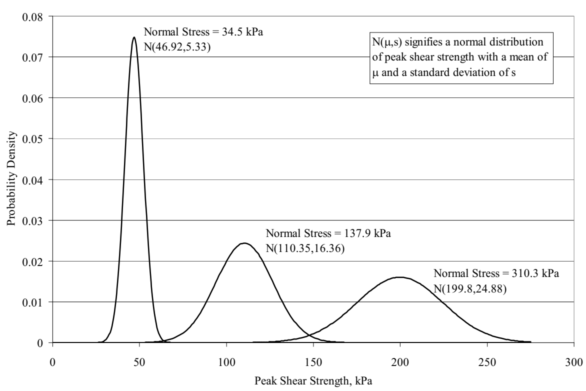


Figure 21. Probability density functions for the internal peak shear strength of a needle-punched GCL, developed from 19 test series of three different normal stresses (McCartney et al. 2002)

As mentioned earlier, no full scale field failures related to the internal shear strength of reinforced GCLs have been reported. This implies that field testing may be required to truly determine the critical interface in a layered system. Tanays et al. (1994), Feki et al. (1997) and Daniel et al. (1998) reported the findings from full scale field tests of the internal and interface shear strength behaviour of unreinforced and reinforced GCLs configured with other liner components (geomembranes, geotextiles, and soils). Tanays et al. (1994) and Feki et al. (1997) presented results on a experimental cell where a stitch bonded GCL was installed on slopes inclined at 2H:1V and 1H:1V respectively. Displacements within the GCL were found to be very low on the 2H:1V slope and remained unchanged during the period of observation (500 days). One day after its installation on the 1H:1V slope, the GCL reached an average strain of 5.5% with extension occurring at the top of the slope. Further displacements decreased with time of observation (3 months). It was assumed that partial failure of the GCL occurred at the measuring points due to excessive strain (>2%). Significant information concerning interface behaviour has been garnered from Daniel et al. (1998). It was reported that all geosynthetic configurations on test slopes inclined at 3H:1V performed satisfactorily. Three slides have occurred on steeper slopes (2H:1V). One slide occurred internally in an unreinforced GCL (a geomembrane backed GCL) because of sodium bentonite hydration. Two slides occurred at the interface between a reinforced GCL and a geomembrane 20 and 50 days after construction. The slides were due to reduction in the interface strength caused by bentonite extrusion through a woven geotextile. Stark et al. (1998) presented a case study describing a slope failure involving an unreinforced GCL in a landfill liner system, which is discussed in detail in section 5.4

5.3 *Steep sided walls*

Lining systems for steep to almost vertical slopes are also receiving significant attention, which stems from the fact that landfills are designed to accommodate two factors: land saving (smaller landfill foot print) and increase of landfill capacity. To achieve these goals, the inclination of side slopes is generally increased to improve the ratio between the volumetric capacity and the print of the landfill. Different types of sidewall lining systems have recentrly been proposed that are able to achieve the same safety level as the bottom composite liners while allowing construction on slope angle up to around 70^o. This is partly due to the fact that old quarries, especially those formerly used to mine sand and gravel are still favourite spots for landfills. These slopes are usually important sources of groundwater recharge. The conventional composite liner consisting of compacted clay and geomembrane for deep steep sided landfills is usually avoided due to stability concerns. The work by Hertweck and Amann (1997) has indicated that clay barriers could be placed on up to 1:2 steep slopes for relatively small heights; however greater heights and steeper slopes would lead to slope failure within the clay. If constructed in short lifts ahead of the main refuse placement overall stability could still pose problems since the low density and high compressibility of the refuse would result in large deformations occurring before the passive resistance could be mobilized fully.

The use of GCLs, composite liners where the compacted clay liner component is strengthened by the use of cement and geocomposites has been implemented to enhance stability (Manassero et al., 2000). On the other hand, the drainage and filter layers are not a problem for steep slope, whereas the protection layer can be very important in particular when geomembranes (GM) and GCLs are employed. In some cases it can be a good practice to use some types of wastes as a protection layer such as tires or big sacks containing waste in a powder form.

Different types of sidewall lining systems have been recently proposed. Figure 22a shows a mineral layer being constructed with natural clay in horizontal lifts achieving the final slope profile by means of a finishing excavation. The natural clay was mixed on site before compaction with 2% to 10% by weight of cement in order to achieve the strength that assures the stability of the slope. The improved contact between the geomembrane of the geocomposite and the mineral filler. The second type of liner for steep sides (Figure 22b) consists of composite geotextile-geomembrane bags filled with plastic concrete or cement-bentonite (CB) self-hardening slurries. The main advantages of this technique is the reduction of discontinuities between different phases of casting operations. On the other hand, this kind of liner is generally expensive. The third type of steep slope liner (Figure 22c) is comprised of a geosynthetic clay liner either combined with geomembranes to form a composite liner or on their own to form a single liner. This can offer a very cost effective solution due to the easy emplacement and low cost involved.

When GCLs are placed on very steep slopes, the fiber reinforcement structure is challenged by the gravitational weight of the overlying materials, typically soil and/or solid waste. Applied stresses imposed on the upper geotextile must be

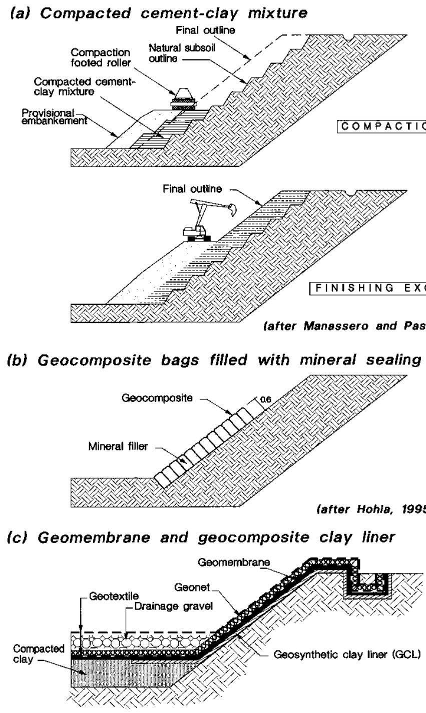


Figure 22. Alternative liners for steep side slopes (after Manassero et al. 2000).

transferred to the lower geotextile via the fibers penetrating through the bentonite layer. Thus, the importance of the fiber reinforcement structure is readily apparent. The fibers are forced into tension by the imposed shear stresses, which must be sustained for the time required by the site-specific conditions. This could be very short if a passive wedge is placed against the slope, or very long if the slope is to be left unsupported. Recent work by Zanzinger & Alexiew (2000) and Koerner et al. (2001) has shown that the fiber structures maintained its full integrity when short term creep tests have been conducted on needle punched and stitch bonded GCLs over 1000 hours to simulate loads typically involved in steep sided slopes, furthermore deformations were found to be within reasonable limits. However, both research teams concluded that long-term behaviour was difficult to quantify on the basis of short term tests (1000 hours). Nevertheless it was suggested that extrapolation procedures can be used to predict the long-term creep behaviour but this warrants further investigation. Another aspect that has received recent attention in the context of steep slopes, is the long-term degradation of fibres of reinforced GCLs when subjected to long term shear stresses. Hsuan (2002) proposed a testing procedure, the so-called incubation method, where the fibres are first subjected to liquid and temperature effects, then followed by peel strength tests. This testing approach is still at the proposal stage but there is no doubt that it will form part of the future activity in GCL research as the need of information on long term durability becomes more acute. This later aspect is reinforced by the recent decision of the German certification authority for building products to temporarily certify GCLs for landfill applications with restriction to limited slopes (3H:1V) (Thies et al. 2002). The certification was given on the condition to assess and proof the long-term shear strength.

The hydration and swelling process involved in GCLs installed in very steep slopes are other factors, which still remain unknown. It is very important to design GCL based containment using sound engineering principles wherein aspects like hydration and swelling process and hydraulic durability are taken into account to evaluate the overall performance under the typical conditions encountered in very steep sided walls. Unfortunately, our knowledge of these aspects is still poor, so the debate is largely uninformed or based on conjecture. These are areas where research is urgently needed, as hydraulic durability is a fundamental parameter for a material and essential for the assessment of global performance.

5.4 *Case histories*

In light of recent waste slope failures reported by Byrne et al. (1992), Mitchell (1996), Milanov et al. (1997), Pardo de Santayana & Veiga Pinto (1998), Rowe (1998), Schmucker & Hendron (1999), Koerner & Soong (2000) stability of municipal solid waste landfills has started to receive more attention than in the past. In this respect, assessing the stability of waste fills has become a very important aspect of waste containment system analysis and design. As with any stability study the selection of the most probable mode of failure and the accurate assessment of the necessary physical and mechanical properties and hydraulic conditions of the waste and the foundation soils are the most critical aspects. However, stability analyses for MSW landfills are more complex than those for classical earth structures as a result of the difficulties involved in evaluating the physical and mechanical properties of the waste, the interface interactions, and the variation of these parameters with depth. In addition, the variation of the waste properties with time may need to be considered in the analysis. As part of the stability analysis, the shape of the potential failure surface must be evaluated. For instance, failure surfaces passing through the waste are generally observed to be circular. However, if stability along one of the interfaces (waste/liner, liner/foundation soil, etc.) is the most critical, the analysis may need to be performed considering a non-circular failure surface passing along the interface with the lowest strength (Bouazza & Donald, 1999, Bouazza & Wojnarowicz, 1999, Brandl, 1999). This section presents a number of case studies involving landfill side slope instability reported in recent literature.

Case History 1: Mahoning landfill, Youngstown, Ohio, USA

Originally the location of a strip mining operation, the 80 ha site was transformed into a landfill for MSW following the cessation of mining operations (Stark et al. 1998). The solid waste was used to fill between the high walls and ponds left by the strip mining. This practice was stopped in 1976 and the site fell dormant for a period 10 years. A new owner took over in 1992 with a requirement to install a liner and collection system and change of the waste placement practice. A plan view and cell layout is shown in Figure 23. Cells 1A and 1B were constructed and filled before cell 2, which was lower in elevation. Furthermore, in preparation for cell 2, a small toe excavation was required for connection of the leachate removal pipes at the boundary between cell 1 and cell 2. Cell 1 was lined with 0.9 m compacted clay liner (compacted wet of optimum) and a geomembrane backed GCL with the bentonite component facing the clay. The initial signs of failure were cracks, 25 to 75 mm wide and 3 to 12 m long, observed at the crest of the slope in cell 1. The cracks exhibited no vertical offset and were characteristics of horizontal separation (i.e indication of translation). Observations at the excavated toe indicated that the geomembrane was wrinkled into a 1.2 m area, which further indicated that the waste mass in cell 1B was moving towards cell 2. The failures surface was found to be within the bentonite component of the unreinforced geomembrane backed GCL. Removal of the waste showed that this failure extended 47 m from the toe back into the waste mass, broke through the geomembrane and moved upward through the waste at approximately 60^{o} angle to the horizontal as shown in Figure 24.

The major lesson from this case is that an unreinforced geomembrane backed GCLs should not be placed in contact with CCLs compacted wet of optimum; due to the generally low shear strength of the hydrated component.

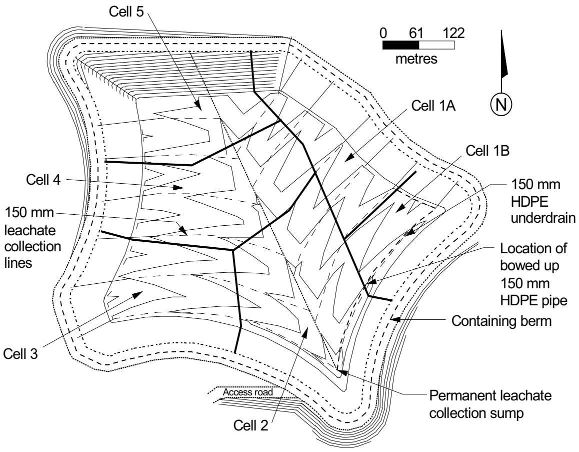


Figure 23. Plan view and cell layout (modified from Stark et al. 1998)

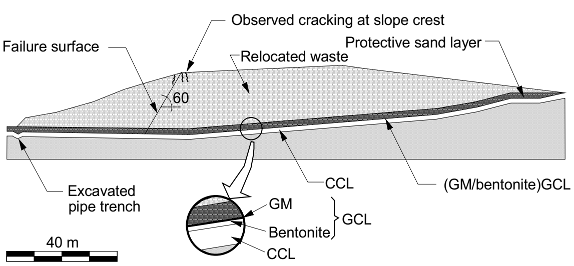


Figure 24. Critical cross section with estimated failure surfaces and details of liner system (modified from Stark et al. 1998).

Case History 2: Bulbul Drive landfill, South Africa

The Bulbul drive landfill was licensed to accept hazardous waste both in liquid and solid form, which is co-disposed with municipal solid waste (Phase2). It is situated in a valley with a longitudinal floor slope of 10% and sideslopes of 36% (Brink et al. 1999). Landfilling started at the top of the site with stability berms being provided across the valley at the toe of the landfill to ensure stability. Each phase of landfilling was placed against the previous phase in a diagonally layered configuration as shown in Figure 25. A compacted clay liner is located beneath both phases 1 and 1A, whereas a composite liner was installed beneath phase 1B. The composite liner consisted of a compacted clay liner and 1.5 mm flexible polypropylene geomembrane and was used on the valley floor. The sideslope liner consisted of the polypropylene geomembrane protected by a non woven geotextile. Liquid waste was deposited into trenches excavated into the upper surface of the landfill and along the interface between Phase 1A and phase 1B.

On 8 September 1997 the landfill failed and discharged 160,000 m^3 of waste into the valley. The failure took place very rapidly as a single translational slide. The post failure investigation showed that the failure surface was at the interface between phases 1A and 1B and then transitioned to the lining system beneath phase 1B. The investigation concluded that the shear strength at the interface between phases 1A and 1B played a critical role in the failure. Liquid injected at the interface between the two sections on a regular basis also contributed to an excessive pore pressure and weakening of the interface. The major lesson learned from this case is that the shear strength on the interface between two phases at a landfill can be significantly lower than that of the waste body itself.

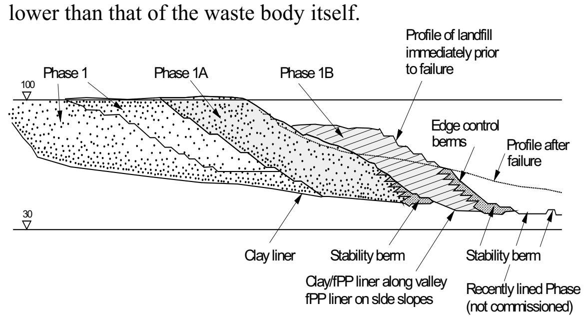


Figure 25. Section through the landfill at time of failure (from Brink et al. 1999).

Case History 3: BIFFA landfill, Brussels, Belgium

The BIFFA waste disposal facility for waste with high content of organic material was excavated 35 m deep in sandy soil (De Meerleer et al. 2000). The slopes were very steep (1V:2H to 1V:1.5H) and had lengths of up to 70 m. The side liner consisted of 0.6 m compacted clay, an HDPE with spikes underneath and a smooth upper surface, a composite reinforcement geotextile which had a dual function:a temporary protection function of the GM during the construction activity and a long term protection to fulfill the zero tensile stress in GM design put in place. In this latter case, the interface friction between the geomembrane and the lower part of the geotextile was expected to form a sliding surface. A sand layer mixed with cement was placed on top to act as a protective layer to the GM.

During installation (approximately three weeks after the beginning of construction), one long section of geotextile failed at the crest of the slope by breaking at the top and sliding down to the base level, together with its overlying sand layer. The geotextile was 5 m wide and was loaded with 300 mm of stabilised sand over the distance of 70 m on the 1.5V:2H slope. The geotextile moved on top of the underlying GM without causing damage. The post failure investigation showed that the sliding of the geotextile was caused by the cable of a winch situated at the top of the slope. The winch and the cable were used to support bulldozers working on the steep slope. The friction of the cable against the geotextile at the edge of the slope has led to local damage in the geotextile and initiated its rupture. A positive aspect of this failure is that the concept of zero tensile stress on the geomembrane was proven to be successful, since it has not suffered any damage. The major lesson learned from this case is that construction equipment tools should not come in contact with geosynthetics.

Case History 4: MidWest landfill, (location unknown), USA

A massive slide occurred at a municipal solid waste landfill in the upper Midwestern US in April 2000 (Benson 2001). Plans for the new cell called for construction in a series of phases, with a new phase being constructed annually. The first phase consisted of a 400 m long and 20 m deep excavation in glacial clay till (Fig. 26), with the base grade being approximately 17 m below the ground water table. The base of this phase was to be 80 m wide, and the side slope was to be at 3:1 (length of side slope = 60 m). To reduce construction costs, however, the base was constructed half as wide (40 m) as originally planned. No other changes to the geometry were made. A composite liner system was constructed consisting of a compacted clay liner 1.2-m thick overlain by a 1.5-mm-thick smooth HDPE geomembrane, a non-woven geotextile cushion (550 g/m^2), and a

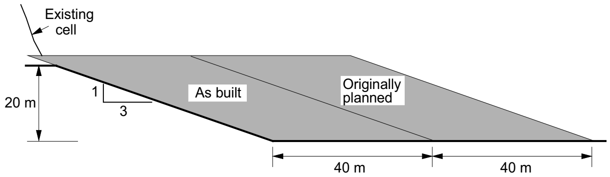


Figure 26 Cross section of initial phase of cell where slide occurred (from Benson 2001).

600 mm layer of leachate collection stone (nominal diameter = 50 mm).

A smooth geomembrane was used in place of the textured geomembrane proposed in the design to reduce material costs. Filling proceeded in lifts 3 m thick and continued for 6.5 months at which time the surface of the waste was approximately 3 m above the surrounding ground surface. At this point, a crack formed in the waste at the top of the slope near the center of the cell (Fig. 27). Within minutes, the crack propagated along the extent of the cell. This crack was followed immediately by translation of 142,000 Mg of waste, leaving a 15-m wide crevice along the extent of the cell. No injuries occurred, but the cell was closed and waste being delivered to the facility was diverted, resulting in significant loss of revenue.

The slide was analyzed in two dimensions because the breadth of the slide area was large compared to the ends (Fig. 27). Spencer's method and a two-block analytical solution were employed using effective stresses, no cohesion, and the interface friction angles measured as part of the post failure investigation. Four cross-sections along the length of the slide were considered. Both Spencer's method and the analytical solution yielded essentially the same results. The factor of safety ranged from 1.10 - 1.21 (peak) or 0.87 - 0.96 (large-displacement). Veneer analyses conducted with the peak and large-displacement friction angles also showed that the geosynthetics should have failed in tension, which was later confirmed in test pits excavated along the top of the slope.

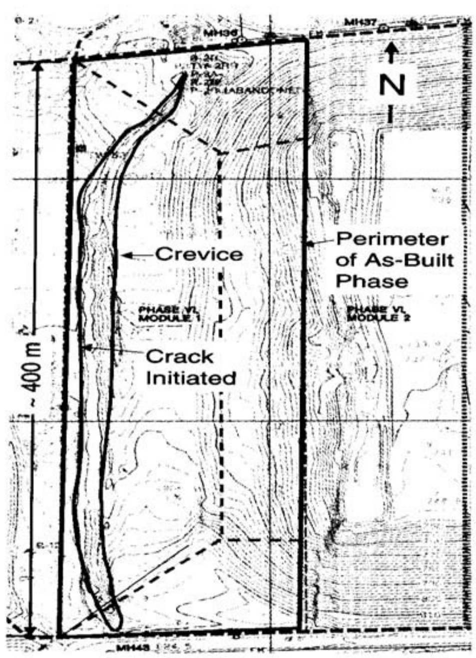


Fig. 27. Topographic map of phase where slide occurred showing as-built outline of phase and location of crevice after slide (from Benson, 2001).

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

There are two key lessons to be learned from this failure. First, careful testing and analysis before construction would have shown the deficiencies in the as-built design prior to construction, and would have permitted the analysis of alternative scenarios where some cost savings may have been accrued and stability could have been ensured. Unfortunately, no interface shear testing or additional analysis was conducted, because the construction engineer argued, "shear testing of the various base liner components would serve no useful purpose." Second, strategies to accrue cost savings must be analyzed carefully to ensure that the construction savings being accrued through design changes will not have unforeseen negative impacts. Indeed, the savings accrued at this site during construction (US\$ 10/m^2 = ?Euro/m^2) was very small compared to the cost of remediating the failure (US \$250/m^2= ?Euro/m^2) one year later.

5.5 *Comments*

Giroud (2000) pointed out the importance of learning from past failures of structures incorporating geosynthetics in order to minimize any risk of failures in the future. In this respect, there is no doubt that all the forensic investigations related to the Kettleman Hills landfill failure has contributed greatly to the education of our profession on the importance of proper assessment of geosynthetics interfaces and to the development of our knowledge in the design of lining systems for side slopes. However, the above cases and the cases reported by Rowe (1998) and Koerner & Soong (2000) make the recommendations made by Rowe (1999) and Giroud (2000) to minimize the likelihood of failure even more important to follow. Some of these recommendations, related to geosynthetics, are given in the following:

- Geosynthetics must be treated like other construction materials, potential and limitations of geosynthetics must be known of the designers and the contractors.
- Properties of geosynthetics must be measured and not estimated, because geosynthtetics that seem identical often have different properties.
- Tests used to measure geosynthetic properties must be carefully selected because tests that are not representative of the field situation give results that are incorrect.
- Design engineers must consider all potential failure mechanisms
- Good CQC/CQA to ensure that the barrier system is installed as designed
- Avoid co-disposal of liquid waste (or increasing the approved amount of liquid waste) without fully assessing the potential impact on stability.
- Contingency plans must be in place in the event of changed conditions occurring during construction (e.g. excessive rain,etc.)
- Design engineers must write specifications that are complete and precise and that address problems that might occur during construction

In general, landfills are heavily regulated by federal and/or state laws. Requirements are usually spelled out regarding types and conditions of acceptable waste materials, methods for waste placement and compaction, lining system design and construction, leachate collection system, and monitoring both during and after active operation. Consequently, the attention of the design engineers has mainly focused on the design of pollution reduction/prevention systems and monitoring to ensure that current legal requirements for non-pollution are met. In this respect, very significant progress has been made in understanding the behaviour and performance of liners, covers, and leachate and gas collection removal systems under different operating conditions. However, one should not lose sight of the importance of stability of landfills. In this respect, the failures reported by numerous authors cited in this paper are a salutary remainder to the importance of a proper evaluation of the stability of waste repositories. However, it is also important to point out that many landfills have been successfully constructed without stability problems, which is also a good indicator of the awareness of our profession regarding stability issues.

6 GEOSYNTHETICS IN LIQUID COLLECTION SYSTEMS

6.1 *General considerations*

Calculating the thickness of liquid in a liquid collection layer is an important design step because one of the design criteria for a liquid collection layer is that the maximum thickness of the liquid collection layer must be less than an allowable thickness. The term "thickness" is used instead of the more familiar term "depth", because thickness (measured perpendicular to the liquid collection layer slope), and not depth (measured vertically), is actually used in design

The thickness of liquid in a liquid collection layer depends on the rate of liquid supply. A typical case of liquid supply is that of liquid impinging onto the liquid collection layer. Two examples of liquid collection layers with such a type of liquid supply can be found in landfills (Fig. 28): (i) the drainage layer of the cover system (Fig. 28a), where the liquid that impinges onto the liquid collection layer is the precipitation water that has percolated through the soil layer overlying the drainage layer; and (ii) the leachate collection layer (Fig. 28b), where the liquid that impinges onto the leachate collection layer is the leachate that has percolated through the waste and through the protective soil layer overlying the leachate collection layer. The terminology "liquid impingement rate" is often used in the case of landfills to designate the rate of liquid supply.

Equations are available (Giroud et al. 2000a) to calculate the maximum thickness of liquid in a liquid collection layer that meets the following conditions:

- the liquid supply rate is uniform (i.e. it is the same over the entire area of the liquid collection layer) and is constant (i.e. it is the same during a period of time that is long enough that steady-state flow conditions can be reached);
- the liquid collection layer is underlain by a geomembrane liner without defects and, therefore, liquid losses are negligible;
- the slope of the liquid collection layer is uniform (a situation referred to herein as "single slope") ; and
- there is a drain at the toe of the slope that promptly removes the liquid.

The last two conditions are not met in cases where the liquid collection layer comprises two sections on different slopes, with no drain removing the liquid at the connection between the two sections; in those cases, the only drain is at the toe of the downstream section. A detailed study of liquid flow in a liquid collection layer located on a single slope with a perfect drain at the toe is presented by Giroud et al. (2000a).

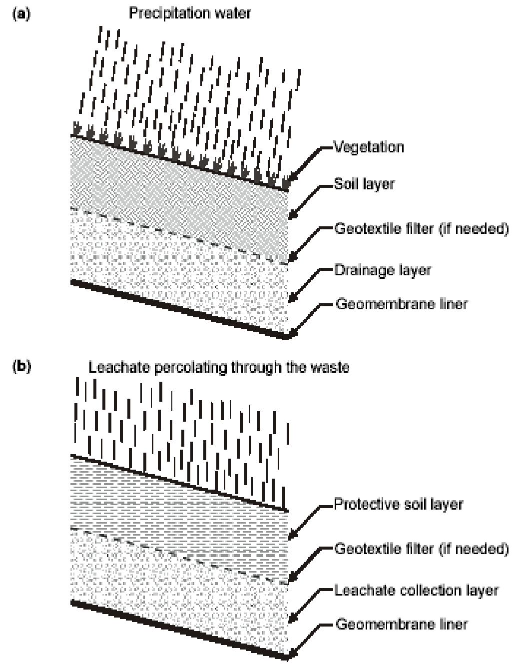


Figure 28. Examples of liquid collection layers subjected to a uniform supply of liquid in a landfill: (a) drainage layer in a cover system; (b) leachate collection layer (Giroud et al. 2000b).

6.2 *Shape of the liquid surface and maximum liquid thickness*

The shape of the liquid surface in the liquid collection layer in the case where there is a perfect drain at the toe of the liquid collection layer is shown in Figure 29. The shape of the liquid surface depends on a dimensionless parameter, λ, called "characteristic parameter", and defined as follows:

$$\lambda = \frac{q_h}{k \ \tan^2\beta} \tag{10}$$

where: q_h = liquid impingement rate (i.e. rate of liquid supply per unit horizontal area); k = hydraulic conductivity of the liquid collection material in the direction of the flow; and β = slope angle of the liquid collection layer with the horizontal.

The maximum liquid thickness must be estimated for atwo reasons: (1) the liquid thickness is typically limited by regulations (e.g. the Resource Conservation and Recovery Act in the US limits requires a maximum liquid thickness of 0.3 m), and (2) good design requires that the liquid thickness be less than the thickness of the lateral drain (to avoid confined flow). Regardless of the shape of the liquid surface, the maximum liquid thickness, t_{max} , in the liquid collection layer is given by the following equation, known as the modified Giroud's equation (Giroud et al. 2000a):

$$t_{max} = j\,\frac{\sqrt{\tan^2\beta \ + \ 4\,q_h/k} \ - \tan\beta}{2\cos\beta}\,L = j\,\frac{\sqrt{1+4\lambda}-1}{2}\,\frac{\tan\beta}{\cos\beta}\,L \tag{11}$$

where: L = horizontal projection of the length of the liquid collection layer in the direction of the flow; and j is a dimensionless parameter, called "modifying factor", and defined as follows:

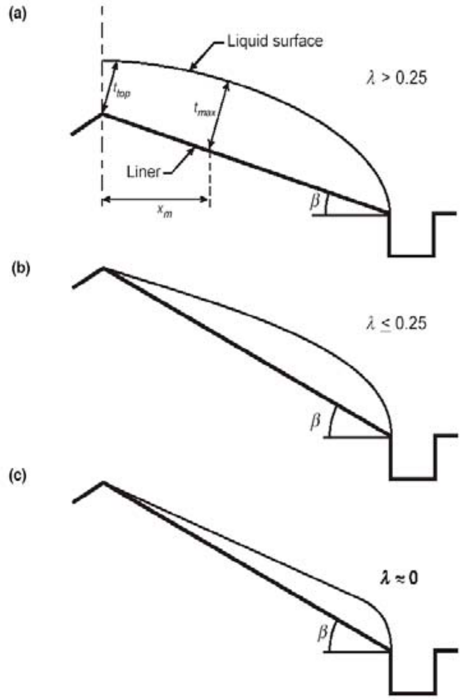


Figure 29. Shape of the liquid surface in a liquid collection layer as a function of the dimensionless characteristic parameter, λ: (a) λ > 0.25; (b) λ ≤ 0.25; (c) λ very small (Giroud et al. 2000a)

$$j = 1 - 0.12\exp\left\{-\left[\log\left(8\lambda/5\right)^{5/8}\right]^2\right\} \tag{12}$$

Numerical values of the modifying factor, j, range between 0.88 and 1.00. Therefore, a conservative approximation of Equation 11 is the following equation, which is known as the original Giroud's equation:

$$t_{max} = \frac{\sqrt{\tan^2\beta + 4q_h/k} - \tan\beta}{2\cos\beta}L = \frac{\sqrt{1+4\lambda}-1}{2}\frac{\tan\beta}{\cos\beta}L \tag{13}$$

An exact solution to the problem was first published by McEnroe (1993). However, this solution is very tedious to use and is subject to significant errors resulting from the number of significant digits used during calculations.

When λ is very small (e.g. $\lambda < 0.01$), which occurs in many practical situations, Equations 11 and 13 are equivalent to the following approximate equation (Giroud et al. 2000a):

$$t_{max} \approx t_{lim} = \frac{q_h}{k\sin\beta}L = \frac{q_h}{k\tan^2\beta}\frac{\tan\beta}{\cos\beta}L = \lambda\frac{\tan\beta}{\cos\beta}L \tag{14}$$

where, t_{lim} is the maximum liquid thickness in the limit case where q_h is small and β and k are large (Giroud et al. 2000a).

It should be noted that:

$$j\frac{\sqrt{1+4\lambda}-1}{2} < \frac{\sqrt{1+4\lambda}-1}{2} < \frac{\sqrt{1+4\lambda+4\lambda^2}-1}{2} = \frac{\left(1+2\lambda\right)-1}{2} = \lambda \tag{15}$$

Therefore, regardless of the value of λ, Equation 14 provides a conservative value of the maximum liquid thickness and a very good approximation for drainage layers involving geonets.

Equation 14 is simpler than Equation 13, which in turn is simpler than Equation 11. A detailed discussion of the approximation made when Equations 13 or 14 are used is presented by Giroud et al. (2000a). In conclusion, Equation 14 provides an acceptable approximation of t_{max} if the liquid thickness is less than one tenth of the height of the liquid collection layer (i.e. the difference in elevation between the top and the toe of the liquid collection layer slope). As a result, from a practical standpoint, Equation 14 is always valid in the case of geosynthetic liquid collection layers, and rarely valid in the case of granular liquid collection layers located on a slope that is not steep.

A parametric study of typical parameter values indicates that λ is rather small (i.e. less than 0.1) in all typical cases, except in the case of a liquid collection layer with a relatively low hydraulic conductivity (sand) placed on a slope that is not steep (2%) and that is subjected to a high liquid impingement rate (0.1 m/day). Furthermore, in the case of geosynthetic liquid collection layers, λ is very small because the maximum liquid thickness is very small compared to the length of the liquid collection layer. Indeed, Equation 14 shows that, if t_{max}/L is very small, λ is also very small. The shape of the liquid surface is then illustrated in Figure 29c. The thickness at the top is zero and the maximum liquid thickness (which occurs at the toe) is small. Therefore, in the case of a geosynthetic liquid collection layer, the slope of the liquid surface is quasi parallel to the slope of the liquid collection layer and, as a result, the hydraulic gradient is equal to the classical value for flow parallel to a slope, $\sin\beta$. In contrast, in the case of a granular liquid collection layer, the slope of the liquid surface (Figs. 29a and 29b) increases from the top to the toe of the liquid collection layer. As a result, the hydraulic gradient increases from the top to the toe of the liquid collection layer, where it is significantly greater than $\sin\beta$.

6.3 *Equivalency of geosynthetic to granular lateral drains*

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

Equivalency between two lateral drainage systems must take into consideration the service flow gradients and maximum liquid thickness. Giroud et al. (2000c) have shown that, to be equivalent to a natural drainage layer, the minimum transmissivity of the geocomposite must be greater than the tranmissivity of the natural drainage layer. The minimum transmissivity of the geonet is obtained by multiplying the transmissivity of the natural drainage layer by an equivalency factor, E. For natural drainage layers having maximum flow depths of 30 cm, E can be approximated as follows:

$$E = \frac{1}{0.88}\left[1 + \frac{t_{prescribed}}{0.88\,L}\frac{\cos\beta}{\tan\beta}\right] \tag{16}$$

where $t_{prescribed}$ is the maximum liquid thickness prescribed by regulations. The equivalency defined by Equation 16 is based on equal unconfined flow volumes in natural and geocomposite drainage systems. However, the very low heads associated with unconfined flow in a geocomposite lateral drain will result in a significantly reduced head acting on the underlying liner system, and therefore in a reduced potential leakage.

6.4 *Double slopes*

Two examples of liquid collection layers that comprise two sections with different slopes are presented in Figure 30 for a landfill cover system and a landfill leachate collection system. The two sections of a liquid collection layer are designated as the upstream section and the downstream section.

When a liquid collection layer comprises two sections, different liquid collection materials may be used in the two sections; for example, a geonet may be used on the steep slope and gravel may be used on the other slope. However, there are many applications where the same material is used in both sections; for example, a geonet may be used as the liquid collection layer in the various slopes of a landfill cover.

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

The maximum liquid thickness in the downstream section of a two-slope liquid collection layer can be calculated using equations that account for both the *liquid impinging onto the downstream section* and the *liquid impinging onto*, and *flowing from*, the *upstream section*. The maximum liquid thickness in the upstream section of a two-slope liquid collection layer can be calculated using equations that depend on the material used in the upstream section and in the downstream section. In some cases, a transition zone is needed between the upstream and downstream sections.

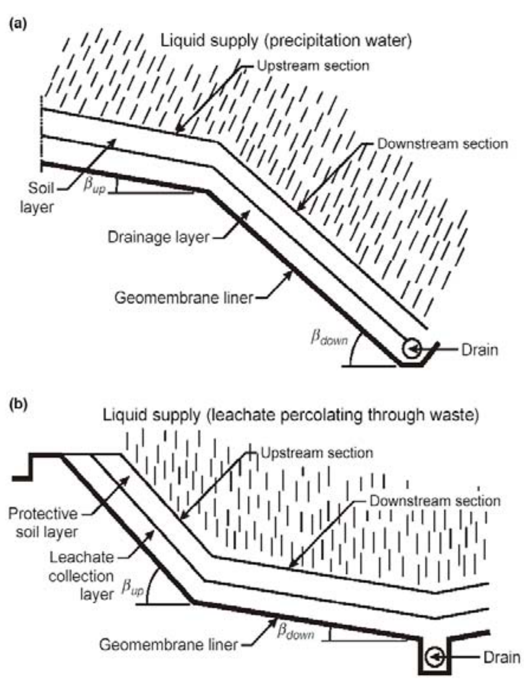


Figure 30. Examples of liquid collection layers located on two different slopes with no drain at the connection between the two slopes: (a) drainage layer in landfill cover system; (b) leachate collection layer in a landfill (Giroud et al. 2000b).

7 GEOSYNTHETICS IN COVER SYSTEMS

Waste containment facilities must have a final cover system designed to minimize infiltration and erosion and control ingress or egress of gases (e.g., egress of decomposition gases from municipal solid waste or ingress of oxygen into sulphidic mining wastes). The cover system must have an erosion control layer underlain by an infiltration control layer. In order to maximize the capacity of the landfill, major advances have recently taken place regarding the use of geosynthetic reinforcements to allow significantly steeper and higher final covers. An evaluation of different approaches for reinforcement of steep covers is presented in Section 7.1. In addition, significant advances have taken place regarding the design of alternative cover systems. Alternative cover systems should achieve the same or lower infiltration as regulatory-defined prescriptive covers. Examples of alternative cover systems include evapotranspirative covers, capillary barriers, and exposed geomembrane covers. While the infiltration analysis of evapotranspirative covers and capillary barriers does not involve the use of geosynthetics for infiltration control, these systems facilitate the use of geosynthetic reinforcements as illustrated in Case History 6 described in section 7.1.6. Significantly new concepts have been developed for the design of exposed geomembrane covers as explained in Section 7.2. Gas permeability and diffusion of some elements of the cover systems will be also considered in section 7.5.

7.1 *Reinforced cover systems*

7.1.1 *General considerations*

The design of veneer slopes (e.g. steep cover systems for waste containment facilities) poses significant challenges to designers. The use of uniaxial reinforcements placed along the slope (under the veneer and above a typically strong mass of soil or solid waste) and anchored on the top of the slope has been a common design approach. However, this alternative may not be feasible for steep, long veneer slopes. As the veneer slope rests on top of a comparatively stronger mass solid waste, alternative approaches can be considered. This includes use of uniaxial reinforcements placed horizontally (rather than along the slope) and anchored into the underlying mass. A second alternative includes the use of fiber-reinforced soil. A review of analyses for veneers reinforced using horizontally placed inclusions is presented in this section.

This section presents an analytical framework for quantification of the reinforcement requirements for reinforced veneers where reinforcements are placed horizontally and embedded into a comparatively strong underlying mass. Emphasis in this evaluation is placed on the assessment of an infinite slope confirguration. This allows direct comparison of the different reinforcement alternatives.

Design criteria for reinforced soil structure have been the focus of significant debate (Zornberg & Leshchinsky 2001). Although different definitions for the factor of safety have been reported for the design of reinforced soil slopes, the definition used in this study is relative to the shear strength of the soil:

$$FS = \frac{\textit{Available soil shear strength}}{\textit{Soil shear stress required for equilibrium}} \tag{17}$$

This definition is consistent with conventional limit equilibrium analysis, for which extensive experience has evolved for the analysis of unreinforced slopes. Current design practices for reinforced soil slopes often consider approaches that decouple the soil reinforcement interaction and do not strictly consider the factor of safety defined by Equation 17. Such analyses neglect the influence of reinforcement forces on the soil stresses along the potential failure surface and may result in factors of safety significantly different than those calculated

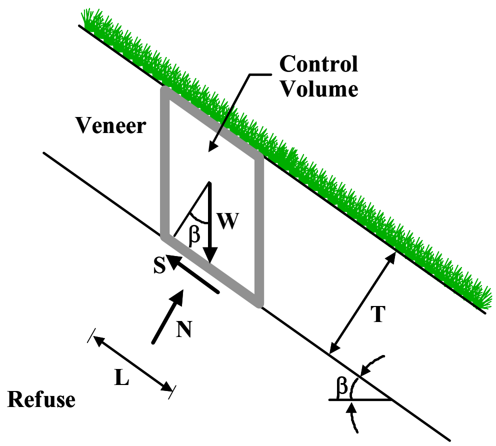


Figure 31. Unreinforced veneer

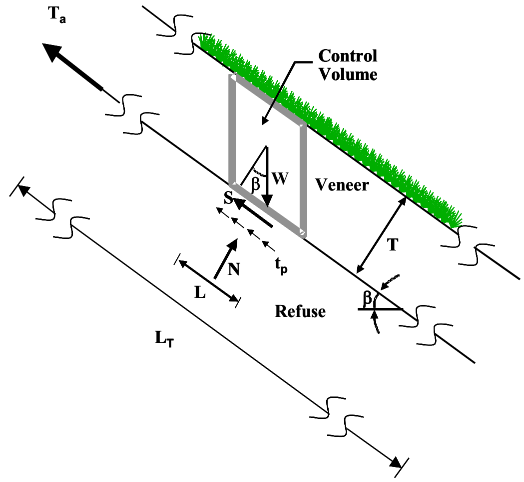


Figure 32. Veneer reinforced with uniaxial geosynthetic parallel to the slope

using more rigorous approaches. Considering the normal and shear forces acting in a control volume along the veneer slope (or infinite slope), and assuming a Mohr-Coulomb shear strength envelope, Equation 17 can be expressed as:

$$FS = \frac{c + (N/L)\tan\phi}{S/L} \tag{18}$$

where N = normal force acting on the control volume; S = shear force acting on the control volume; L = length of the control volume; c = soil cohesion; and ϕ = soil friction angle.

Equations 17 and 18 are valid for both unreinforced and reinforced systems. In the case of an unreinforced veneer (Fig. 31), the shear and normal forces required for equilibrium of a control volume can be defined as a function of the weight of this control volume. That is:

$$S = W \sin\beta \tag{19}$$

$$N = W \cos\beta \tag{20}$$

$$W = \gamma\, L\, T \tag{21}$$

where W = weight of the control volume; β = slope inclination; T = veneer thickness; and γ = soil total unit weight.

From Equations 18, 19, 20, and 21, the classic expression for the factor of safety FS_u of an unreinforced veneer can be obtained:

$$FS_u = \frac{c}{\gamma\, T \sin\beta} + \frac{\tan\phi}{\tan\beta} \tag{22}$$

7.1.2 *Covers reinforced with uniaxial geosynthetics parallel to the slope*

Figure 32 shows a schematic representation of a cover system reinforced using uniaxial geosynthetics placed parallel to the slope. An infinite slope case is considered. In the case, the shear force needed for equilibrium of the control volume is smaller that the one in the unreinforced case. In this case, the shear force is defined by:

$$S = W \sin\beta - t_p\, L \tag{23}$$

where t_p = distributed reinforcement tensile stress of the reinforcement parallel to the slope. When the geosynthetic reinforcements are placed parallel to the slope, the distributed reinforcement tensile stress is a function of the allowable reinforcement tensile strength (T_a) and the total slope length (L_T), as follows:

$$t_p = \frac{T_a}{L_T} \tag{24}$$

From equations 18, 20, 21 and 23, the factor of safety for the parallel-reinforcement case, $FS_{r,p}$, can be estimated as:

$$FS_{r,p} = \frac{\dfrac{c}{\gamma\, T \sin\beta} + \dfrac{\tan\phi}{\tan\beta}}{1 - \dfrac{t_p}{\gamma\, T \sin\beta}} \tag{25}$$

The equation above can be simplified by defining the normalized distributed reinforcement tensile stress t_p^* (dimensionless), as follows:

$$t_p^{\,*} = \frac{t_p}{\gamma\, T} \tag{26}$$

Using Equations 22 and 26 into Equation 25:

$$FS_{r,p} = \frac{FS_u}{1 - t_p^{\,n}\dfrac{1}{\sin\beta}} \tag{27}$$

Equation 27 provides a convenient expression for stability evaluation of reinforced veneer slopes. It should be noted that if the distributed reinforcement tensile stress t equals zero (i.e. in the case of unreinforced veneers), Equation 27 leads to $FS_{r,p} = FS_u$.

Even though the focus in this paper is on infinite slope analysis, typical design is performed using two-wedge finite slope analysis. Figure 33 shows the geometry considered in the methodologies proposed by Giroud et al. (1995) and Koerner & Soong (1998). Some differences between these approaches in the adopted geometry are shown in the figure. More importantly, these approaches differ in the definition of the factor of safety.

Giroud et al. (1995a) do not include a factor of safety at the horizontal failure surface (AB) and define the factor of safety as the ratio between the resisting and the driving forces acting on the active wedge as projected on the slope direction. The factor of safety in this solution is the sum of five separate terms, which facilitates identification of the different contributions to the stability of the slope. Table 18 presents the contribution of different parameters to the factor of safety. Giroud et al (1995b) analyze stability analysis considering seepage forces. The analysis presented by Koerner & Soong (1998) is consistent with the generic definition of factor of safety stated by Equation 17. Using the proposed method, the factor of safety is obtained by solving a quadratic equation. Koerner & Soong (1998) also provide analytical framework to address cases involving construction equipment, seepage forces, seismic forces, and the stabilizing effects of toe berms, tapered slopes and slope reinforcements. Thiel & Stewart (1993) and Punyamurthula & Hawk (1998) provide additional information regarding stability analysis of steep cover systems. Case History 5 (see section 7.1.6) describes the use of geogrid reinforcement in a cover system. Additional case histories are described by Baltz et al. (1995) and Martin & Simac (1995).

Table 18. Effect of different terms in the factor of safety estimated using Giroud et al. (1995a) methodology (adapted from Giroud et al. 1995a)

Slope	Infinite slope		Additional terms for finite slope		
Mechanism	Interface shear		Toe buttressing		Geosynthetic
Parameter	Interface friction	Interface adhesion	Soil internal friction	Soil cohesion	Geosynthetic tension
Symbol	δ	a	ϕ	c	T_a
Factor of safety	$\frac{\tan\delta}{\tan\beta}$	$+\frac{a}{\gamma \; T\sin\beta}$	$+\frac{T}{H_T}\frac{\tan\phi/(2\sin\beta\cos^2\beta)}{1-\tan\beta\tan\phi}$	$+\frac{c}{\gamma \; H_T}\frac{1/(\sin\beta\cos\beta)}{1-\tan\beta\tan\phi}$	$+\frac{T_a}{\gamma \; H_T \; T}$
ϕ↗	↔	↔	↗	↗	↔
β↗	↘	↘	↘	↘	↔
h↗	↔	↔	↘	↘	↘
γ↗	↔	↘	↔	↘	↘
t↗	↔	↘	↗	↔	↘

Notes: H_T : Total height of the slope; T_a: Allowable tensile strength of geosynthetic reinforcement
Influence on FS: ↗ increasing; ↔ no influence; ↘ decreasing; ? either increasing or decreasing

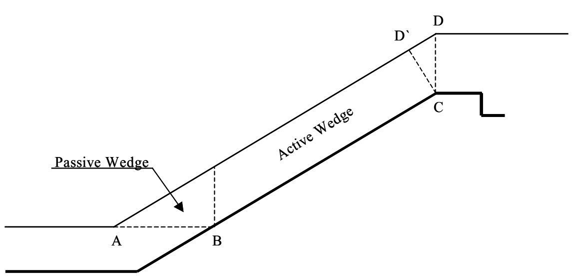


Figure 33: Schematic representation of the geometry of a cover for two-wedge finite slope analysis
Notes: ABC = slip surface
CD = top of the cover soil as defined in the analysis by Koerner and Soong (1998)
CD` = top of the cover soil as defined in the analysis by Giroud et al. (1995a)

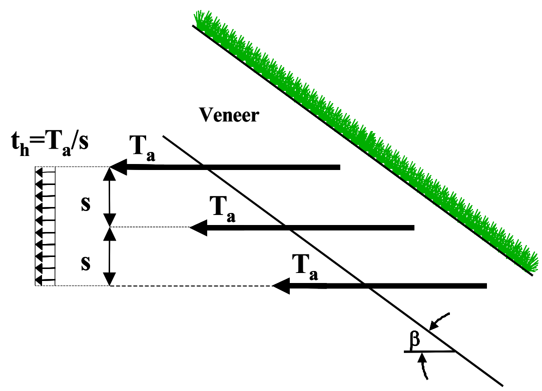


Figure 34. Veneer reinforced with horizontal uniaxial geosynthetics

7.1.3 *Covers reinforced with horizontal uniaxial geosynthetics*

Figure 34 illustrates a cover (veneer) reinforced using horizontal uniaxial geosynthetics. Also in this case, the shear and normal forces acting on the control volume are defined not only as a function of the weight of the control volume, but also as a function of the tensile forces that develop within the reinforcements. For the purpose of the analyses presented herein, the reinforcement tensile forces are represented by a distributed reinforcement tensile stress t_h, which corresponds to a uniformly distributed tensile force per unit height. For a given slope with layers of reinforcement t_h can be expressed by:

$$t_h = \frac{T_a}{s} \quad (28)$$

where T_a = allowable tensile strength of the reinforcement and s = vertical spacing between the layers.

In this case, the shear and normal forces needed for equilibrium of a control volume are defined by:

$$S = W\sin\beta - t_h \; L\sin\beta\cos\beta \quad (29)$$

$$N = W\cos\beta + t_h \; L\sin^2\beta \quad (30)$$

From Equations 18, 21, 28, 29, and 30 the following expression can be obtained for the factor of safety $FS_{r,h}$ of a veneer reinforced with horizontal uniaxial geosynthetics:

$$FS_{r,h} = \frac{\dfrac{c}{\gamma\ T \sin\beta} + \dfrac{\tan\phi}{\tan\beta} + \dfrac{t_h}{\gamma\ T} \sin\beta \tan\phi}{1 - \dfrac{t_h}{\gamma\ T} \cos\beta} \quad (31)$$

The equation above can be simplified by defining the normalized distributed reinforcement tensile stress t_h* (dimensionless), as follows:

$$t_h^* = \frac{t_h}{\gamma\ T} \cos\beta \quad (32)$$

Using Equations 22 and 32 into Equation 31 leads to:

$$FS_{r,h} = \frac{FS_u + t_h^* \tan\beta \tan\phi}{1 - t_h^*} \quad (33)$$

Equation 33 provides a convenient expression for stability evaluation of reinforced veneer slopes. It should be noted that if the distributed reinforcement tensile stress t_h equals zero (i.e. in the case of unreinforced veneers), Equation 33 leads to $FS_r = FS_u$

Additional aspects that should be accounted for in the design of reinforced veneer slopes include the evaluation of the pullout resistance (i.e. embedment length into the underlying mass), assessment of the factor of safety for surfaces that get partially into the underlying mass, evaluation of reinforcement vertical spacing, and analysis of seismic stability of the reinforced veneer. Case History 6 (see section 7.1.6) illustrates the implementation of a horizontally-reinforced cover system.

7.1.4 *Covers reinforced with randomly distributed fibers*

A promising potential alternative for stabilization of steep landfill covers involves the use of fiber-reinforcement. Advantages of fiber-reinforcement over planar reinforcement in the stabilization of landfill covers are:

- Fiber-reinforcement is particularly suitable for stabilization of veneer slopes, as it provides additional shear strength under low confining pressures. A small increase of shear strength under low confinement has a significant impact in the stability of shallow slopes.
- Randomly distributed fibers helps maintaining strength isotropy and do not induce potential planes of weakness that can develope when using planar reinforcement elements.
- No anchorage is needed into solid waste as in the case of reinforcement with horizontal geosynthetics or at the crest of the slope as in the case of reinforcement parallel to the landfill slope.
- In addition to stabilizing the cover slopes, fiber reinforcement has the potential of mitigating the potential for crack development, providing erosion control, and facilitating the establishment of vegetation.

Relevant contributions have been made towards the understanding of the behavior of fibers. A soil mass reinforced with discrete, randomly distributed fibers is similar to a traditional reinforced soil system in its engineering properties but mimics admixture stabilization in the method of its preparation (Gray & Al-Refeai 1986; Bouazza & Amokrane 1995). Potential advantages of fiber-reinforced solutions over the use of other slope stabilization technologies have been identified, for example, for slope repairs in transportation infrastructure projects (Gregory & Chill 1998) and for the use of recycled and waste products such as shredded tires in soil reinforcement (Foose et al. 1996). Micro-reinforcement techniques for soils also include the use of "Texol", which consists of monofilament fibers injected randomly into sand (Leflaive 1985) and the use of randomly distributed polymeric mesh elements (McGown et al. 1985; Morel & Gourc 1997). The use of fiber-reinforced clay backfill to mitigate the development of tension cracks was evaluated by several investigators (e.g. Al Wahab & El-Kedrah 1995; Maher & Ho 1994). Several composite models have been proposed in the literature to explain the behavior of randomly distributed fibers within a soil mass. The proposed models have been based on mechanistic approaches (Maher & Gray 1990), on energy dissipation approaches (Michalowski & Zhao 1996), and on statistics-based approaches (Ranjar et al. 1996).

Fiber-reinforced soil has often been characterized as a single homogenized material, which has required laboratory characterization of composite fiber-reinforced soil specimen. The need for laboratory characterization has been a major drawback in the implementation of fiber-reinforcement in soil stabilization projects. To overcome this difficulty, a discrete approach that characterizes the fiber-reinforced soil as a two-component (fibers and soil) material was recently developed (Zornberg & Li 2002). The main features of this approach are:

- The reinforced mass is characterized by the mechanical properties of individual fibers and of the soil matrix rather than by the mechanical properties of the fiber-reinforced composite material
- A critical confining pressure at which the governing mode of failure changes from fiber pullout to fiber breakage can be defined using the individual fiber and soil matrix properties.
- The fiber-induced distributed tension is a function of fiber content, fiber aspect ratio, and interface shear strength of individual fibers if the governing mode of failure is by fiber pullout.
- The fiber-induced distributed tension is a function of fiber content and ultimate tensile strength of individual fibers if the governing mode of failure is by fiber breakage.
- The discrete framework can be implemented into an infinite slope limit equilibrium framework. Convenient expressions can be obtained to estimate directly the required fiber content to achieve a target factor of safety.

The design methodology for fiber-reinforced soil structures using a discrete approach is consistent with current design guidelines for the use of continuous planar reinforcements and with the actual soil improvement mechanisms. Consequently, fiber-reinforced cover systems are expected to become an economical and technically superior alternative for reinforcement of landfill covers.

Figure 35 shows a schematic view of a fiber-reinforced infinite slope. The behavior of the fiber-reinforced soil mass depends on whether the failure mode is governed by pullout or breakage of the fibers. The governing failure mode of the fiber-reinforced soil mass depends on the confinement. A critical normal stress, $\sigma_{n,crit}$, can be defined for comparison with the normal stress σ_n at the base of the veneer. If $\sigma_n < \sigma_{n,crit}$, the dominant mode of failure is the fibers pullout. This is the case for cover system applications. In this case, the fiber-induced distributed tension t_f is defined by (Zornberg & Li 2002):

$$t_f = \eta\ \chi\ c_{i,c}\ c + \eta\ \chi\ c_{i,\phi} \tan\phi\ \sigma_n \quad (34)$$

where $c_{i,c}$ and $c_{i,\phi}$ are the interaction coefficients for the cohesive and frictional components of the interface shear strength; η = aspect ratio (length/diameter) of the individual fibers, and χ = volumetric fiber content.

Similarly, if $\sigma_n > \sigma_{n,crit}$, the dominant mode of failure is fiber breakage. Even though this is not generally the governing mode of failure for cover slopes the solution for this case is presented for completeness. The fiber-induced distributed tension t_f is defined by:

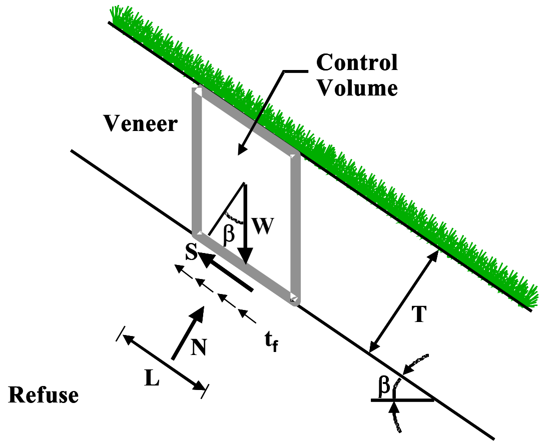


Figure 35. Veneer reinforced with randomly distributed fibers

$$t_t = \sigma_{f,ult} \quad \chi \tag{35}$$

where $\sigma_{f,ult}$ = ultimate tensile strength of the individual fiber.

In a fiber-reinforced veneer, the shear force needed for equilibrium of the control volume equals:

$$S = W \sin\beta - \alpha\, t_f\, L \tag{36}$$

were α is an empirical coefficient that accounts for preferential orientation of fibers. For the case of randomly distributed fibers considered herein α equals one.

Using Equations 21, 22, and 20 into Equation 34 leads to the factor of safety for a fiber-reinforced veneer, $FS_{r,f}$:

$$FS_{r,f} = \frac{\dfrac{c}{\gamma\, T \sin\beta} + \dfrac{\tan\phi}{\tan\beta}}{1 - \dfrac{\alpha\, t_f}{\gamma\, T \sin\beta}} \tag{37}$$

Defining the normalized distributed reinforcement tensile stress t_f^* (dimensionless) of a fiber-reinforced slope as follows:

$$t_f^* = \frac{t_f}{\gamma\, T} \tag{38}$$

Considering Equations 22 and 37 into Equation 34:

$$FS_{r,f} = \frac{FS_u}{1 - \alpha\, t_f^* \dfrac{1}{\sin\beta}} \tag{39}$$

7.1.5 *Comparison among different approaches for cover stability*

The summary presented in this section provides a consistent framework for comparison of different reinforcement approaches for cover systems. They were all developed considering a consistent definition for the factor of safety (Equation 17). Solutions are presented for the case of unreinforced, slope-parallel, horizontally-reinforced and fiber-reinforced veneers. Table 19 summarizes the expressions for the factor of safety in each case and the influence of the parameters governing the stability of the cover. As expected, additional reinforcement always leads to a higher factor of safety while increasing slope inclination would typically lead to decreasing stability. It is worth noting that increasing soil friction angle leads to increasing stability, when compared to the unreinforced case, only for the case of fiber reinforced slopes. It should also be noted that increasing total height of the slope (or increasing total length) does not affect detrimentally the efficiency of horizontally placed reinforcements and of fiber reinforcement.

The use of reinforced soil structures has also been extensively used for stabilization of waste cover systems. The design of these systems does not differ from the design of these systems for other applications such as transportation infrastructure. It should be noted, however, that the reinforced soil structures may be founded on highly compressible waste material, as illustrated in the Case History 7 presented in section 7.1.6. Additional projects involving use of reinforced soil structures to stabilize cover systems are presented by Cargill & Olen (1998).

Table 19. Effect of different terms in the factor of safety of cover systems using different reinforcement approaches

	Definition of Factor of Safety			Influence on the factor of safety compared to FS_u			
				t^*↗	β↗	ϕ↗	L_T or H_T
Unreinforced veneer	$FS_u = \dfrac{c}{\gamma\ T\ \sin\beta} + \dfrac{\tan\phi}{\tan\beta}$						
Reinforcement parallel to slope	$FS_{r,p} = \dfrac{FS_u}{1 - t_p^n \dfrac{1}{\sin\beta}}$	with	$t_p^* = \dfrac{t_p}{\gamma\ T}$	↗	↘	⇔	↘
Horizontal reinforcement	$FS_{r,h} = \dfrac{FS_u + t_h^n \sin\beta\ \tan\phi}{1 - t_h^n \cos\beta}$	with	$t_h^* = \dfrac{t_h}{\gamma\ T}$	↗	?	↗	⇔
Fiber-reinforcement	$FS_{r,f} = \dfrac{FS_u}{1 - \alpha\ t_f^n \dfrac{1}{\sin\beta}}$	with	$t_f^* = \dfrac{t_f}{\gamma\ T}$	↗	↘	↗	⇔

Notes: t_p = distributed tensile stress per unit length of a cover with reinforcement parallel to the slope (Equation 24)
t_h = distributed tensile stress per unit height of a cover with horizontal reinforcement (Equation 28)
t_f = distributed tensile stress per unit length of a cover with fiber-reinforcement (Equation 34)
Influence on FS: ↗ increasing; ⇔ no influence; ↘ decreasing; ? either increasing or decreasing

7.1.6 *Case histories*

Case History 5: McColl Superfund site, Fullerton, California, US

This project is a good example of a site where multiple systems of soil reinforcement were used for stabilization of the final cover system. The soil reinforcement systems included the use of geogrid reinforcements, geocell systems, and reinforced buttress structures (Collins et al. 1998; Hendricker et al. 1998).

The site involves twelve pits containing petroleum sludges and oil-based drilling muds. The sludges were generated by the production of high-octane aviation fuel and were placed in the pits between 1942 and 1946. Between 1952 and 1964, the site was used for disposal of oil-based drilling muds. These wastes and their reaction products and byproducts are found as liquid, gas and solid phases within the pits. At the time of deposition, essentially all of the waste materials were mobile. Over time, much of the waste had hardened. The drilling muds are a thixotropic semi-solid sludge, which can behave as a very viscous fluid.

Key considerations for the selection of the final remedy were to: (i) provide a cover system that includes a barrier layer and a gas collection and treatment system over the pits to minimize infiltration of water and release of hazardous or malodorous gas emissions; (ii) provide a subsurface vertical barrier around the pits to minimize outward lateral migratio of mobile waste or waste byproducts and inward lateral migration of subsurface liquid; and (iii) provide slope stability improvements ofr unstable slopes at the site.

The geogrid reinforcement for the cover system over the more stable pits was constructed with two layers of uniaxial reinforcement placed orthogonal to one another. Connections at the end of each geogrid roll were provided by Bodkin joints. Adjacent geogrid panels did not have any permanent mechanical connections. This was found to be somewhat problematic, as additional care was required during placement of the overlying gas collection sand to minimize geogrid separation. After the connections were made, the geogrid was covered with sand and then pull taut using a backhoe to pull on the end of the geogrid. Details of the cover system involving geogrid reinforcement are shown in Figure 36.

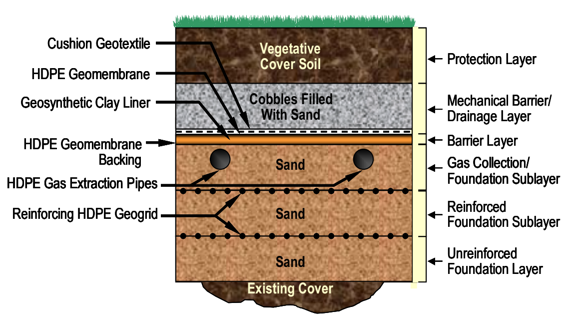


Figure 36. Cover system reinforced using uniaxial geogrids

A geocell reinforcement layer was constructed over the pits containing high percentages of drilling muds. While the construction of this reinforcement layer proceeded at a slower pace than the geogrid reinforcement, it did provide an immediate platform to support load. As the bearing capacity of the underlying drilling mud was quite low, the geocell provided load distribution, increasing the overall bearing capacity of the cover system. Details of the cover system involving geogrid reinforcement are shown in Figure 37.

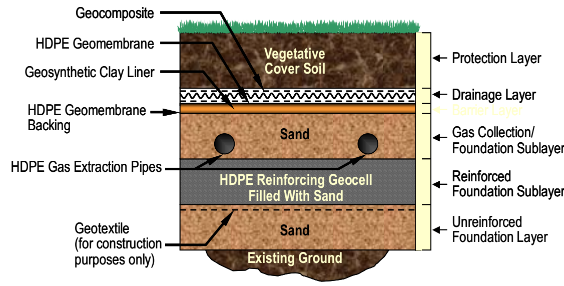


Figure 37. Cover system reinforced using geocells

A total of three reinforced earth structures were constructed at the site. One of the structures was necessary to provide a working pad of rconstruction of the subsurface vertical barrier. This reinforced earth structure had to support the excavator with a gross operating weight of 1,100 kN that was used to dig the soil-bentonite cutoff wall. Another reinforced earth structure at the site had to span a portion of completed cutoff wall. Due to concerns that the stress of the reinforced earth structure on the underlying soil-bentonite cutoff wall would lead to excessive deformation of the wall due to consolidation of the cutoff wall backfill, a flexible wall fascia was selected. As shown in Figure 38, a soldier pile wall was constructed to provide stability of the system during construction. The use of geosynthetic alternatives in this project was more suitable and cost effective than their conventional counterparts.

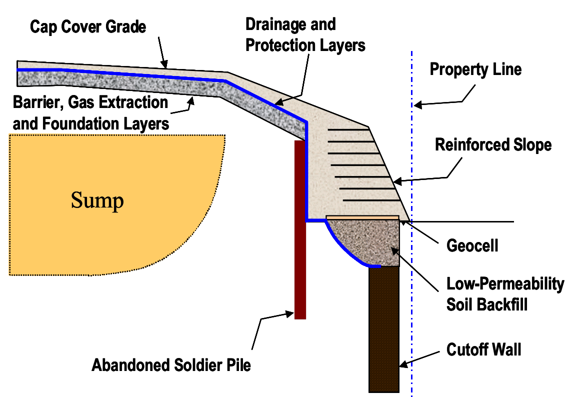


Figure 38. Buttressing reinforced slope at McColl Superfund site

Case History 6: North Slopes at OII Superfund site, Monterey Park, California, US

A cover reinforced using horizontally placed geogrids was constructed as part of the final closure of the Operating Industries, Inc. (OII) landfill. This case history highlights the final closure of a hazardous waste landfill where the severe site constraints were overcome by designing and constructing an alternative final cover incorporating horizontal geosynthetic veneer reinforcement (Zornberg et al. 2001).

The 60-hectare south parcel of the OII landfill was operated from 1948 to 1984, receiving approximately 30-million cubic meters of municipal, industrial, liquid and hazardous wastes. In 1986, the landfill was placed on the National Priorities List of Superfund sites. Beginning in 1996, the design of a final cover system consisting of an alternative evapotranspirative soil cover was initiated, and subsequent construction was carried out from 1997 to 2000. The refuse prism, which occupies an area of about 50 hectares, rises approximately 35 m to 65 m above the surrounding terrain. Slopes of varying steepness surround a relatively flat top deck of about 15 hectares.

The final cover design criteria mandated by the U.S. Environmental Protection Agency (EPA) primarily deal with the percolation performance of the cover, static and seismic stability of the steep sideslopes of the landfill, and erosion control. Stability criteria required a static factor of safety of 1.5, and acceptable permanent seismically induced deformations less than 150 mm under the maximum credible earthquake. The basis of the seismic stability criteria is that some limited deformation or damage may result from the design earthquake, and that interim and permanent repairs would be implemented within a defined period.

One of the most challenging design and construction features of the project was related to the north slope of the landfill. The north slope is located immediately adjacent to the heavily traveled Pomona freeway (over a distance of about 1400 meters), rises up to 65 meters above the freeway, and consists of slope segments as steep as 1.5:1 (H:V) and up to 30 m high separated by narrow benches. The toe of the North Slope and the edge of refuse extends all the way up to the freeway. The pre-existing cover on the North Slope consisted of varying thickness (a few centimeters to several meters) of non-engineered fill. The cover included several areas of sloughing instability, chronic cracking and high levels of gas emissions. The slope was too steep to accommodate any kind of a layered final cover system, particularly a cover incorporating geosynthetic components (geomembranes or GCL). Because of the height of the slope and lack of space at the toe, it was not feasible to flatten the slope by pushing out the toe, removing refuse at the top, or constructing a retaining / buttress structure at the toe of slope.

After evaluating various alternatives, an evapotranspirative cover incorporating geogrid reinforcement for veneer stability was selected as the appropriate cover for the North Slope. The evapotranspirative cover had additional advantages over traditional layered cover systems, including superior long-term percolation performance in arid climates, ability to accommodate long-term settlements, good constructability, and ease of long-term operations and maintenance. The selected cover system included the following components, from the top down: 1) vegetation to promote evapotranspiration and provide erosion protection; 2) a 1.2 m – thick evapotranspirative soil layer to provide moisture retention, minimize downward migration of moisture, and provide a viable zone for root growth; and 3) a foundation layer consisting of soil and refuse of variable thickness to provide a firm foundation for the soil cover system.

Stability analyses showed that for most available monocover materials, compacted to practically achievable levels of relative compaction on a 1.5:1 slope (90% of modified Proctor or 95% of Standard Proctor), the minimum static and seismic stability criteria were not met. Veneer geogrid reinforcement with horizontally placed geogrids was then selected as the most appropriate and cost-effective method for stabilizing the North Slope cover. The analytical framework discussed in Section 2 was used in the design. Figure 39 shows the typical veneer reinforcement detail selected based on the shear strength of the soils used in construction.

The veneer reinforcement consisted of polypropylene uniaxial geogrids, installed at 1.5-meter vertical intervals for slopes steeper than 1.8:1, and at 3-meter vertical intervals for slopes between 2:1 and 1.8:1. The geogrid panels are embedded a minimum of 0.75 meters into the exposed refuse slope face from which the pre-existing cover had been stripped. The geogrid

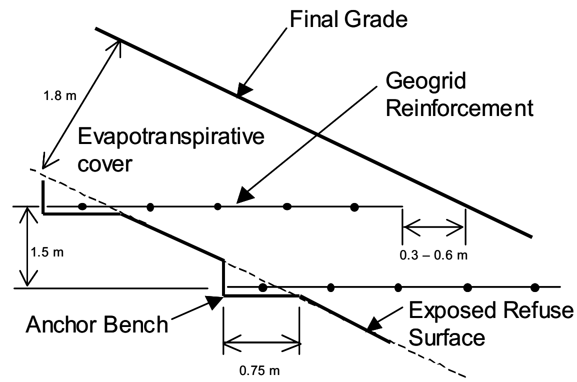


Figure 39. Typical reinforcement detail for horizontal reinforcement anchored into solid waste (from Zornberg et al. 2001)

panels were curtailed approximately 0.3 to 0.6 meters away from the finished surface of the slope cover. This was done to permit surface construction, operation and maintenance activities on the slope face without the risk of exposing or snagging the geogrid.

Construction of the North Slope was accomplished in 12 months. Approximately 500,000 cubic meters of soil and 170,000 square meters of geogrid were placed. Total area of geogrid placement exceeded 9.3 hectares. The maximum height of reinforced portion of the landfill slopes was 55 m (the maximum height of the total landfill slope was 65 m).

Case History 7: Toe Buttress at OII Superfund site, Monterey Park, California, US

In addition to the project described in Case 6, a geogrid-reinforced toe buttress was constructed in 1987 ath the OII Superfund site in order to enhance the stability of the southeastern slopes of the OII Landfill Superfund site (Zornberg and Kavazanjian, 2001). The toe buttress is immediately adjacent to a residential development. The waste slopes behind the toe buttress are up to 37 m high with intermediate slopes between benches up to 18 m high and as steep as 1.3H:1V.

The approximately 4.6 m high, 460 m long toe buttress was built using sandy gravel as backfill material. The front of the structure was founded on concrete piers. However, as the back of the toe buttress was founded on waste, the structure has been subjected to more than 0.6 m of differential settlements since the end of its construction. In response to concerns regarding the internal stability of the reinforced soil structure, finite element analyses were performed to evaluate the long-term integrity of the geogrid reinforcements under static and seismic loads. The analyses considered 40 years of settlement followed by the design earthquake. The finite element modeling evaluated the strains induced in the geogrid reinforcement considering both material and geometric nonlinearity. The analyses were performed in three sequential phases: (i) toe buttress construction, modeled by sequentially activating soil and bar elements in the reinforced soil zone; (ii) gradual increase in differential settlements, simulated by imposing incremental displacements at the base of the reinforced soil mass; and (iii) earthquake loading, modeled by applying horizontal body forces representing the maximum average acceleration estimated in a finite element site response analysis.

A total of 2.0 m of differential settlement was imposed on the base of the finite element mesh to simulate long-term differential settlement. The maximum strain in the geogrid reinforcements calculated after this long-term static loading is less than 3.0 percent, well below the allowable static strain of 10 percent. The calculated maximum geogrid strain induced by construction, long-term differential settlement, and earthquake loading is approximately 8.5 percent, well below the allowable strain of 20 percent established for rapid loading. The results of this study indicate that the integrity of the geogrid-reinforced toe buttress should be maintained even when subjected to large differential settlements and severe earthquake loads.

7.2 *Exposed geomembrane cover systems*

Exposed geomembrane covers have been recently analyzed, designed, and constructed to provide temporary and final closure to waste containment facilities. Significant cost savings may result from elimination of topsoil, cover soil, drainage, and vegetation components in typical cover systems. Additional advantages include reduced annual operation and maintenance requirements, increased landfill volume, easier access to landfilled materials for future reclamation, and reduced post-construction settlements. In addition, if the landfill slopes are steep, the use of exposed geomembrane covers may provide solution to erosion concerns and to stability problems associated with comparatively low interface shear strength of typical cover components. Disadvantages associated with the use of exposed geomembrane covers include increased vulnerability to environmental damage, increased volume and velocity of stormwater runoff, limited regulatory approval, and aesthetics concerns. However, exposed geomembrane covers have been particularly applicable to sites where the design life of the cover is relatively short, when future removal of the cover system may be required, when the landfill sideslopes are steep, when cover soil materials are prohibitively expensive, or when the landfill is expected to be expanded vertically in the future. In particular, the current trends towards the use of "leachate recirculation" or bioreactor landfills makes the use of exposed geomembrane covers a good choice during the period of accelerated settlement of the waste. Key aspects in the design of exposed geomembrane covers are assessment of the geomembrane stresses induced by wind uplift and of the anchorage against wind action (Giroud et al. 1995; Zornberg & Giroud, 1997; Gleason et al. 2001).

7.3 *Geomembrane stresses induced by wind uplift*

The resistance to wind uplift of an exposed geomembrane cover is a governing factor in its design. Wind uplift of the geomembrane is a function of the mechanical properties of the geomembrane, the landfill slope geometry, and the design wind velocity. Procedures for the analysis of geomembrane wind uplift have been developed by Giroud et al. (1995) and Zornberg & Giroud (1997). Additional guidelines are provided by Wayne & Koerner (1988). A number of exposed geomembrane covers have been designed and constructed using these procedures (Gleason et al. 2001), as discussed in Section 7.4 .

Wind uplift design considerations involve assessment of the maximum wind velocity that an exposed geomembrane can withstand without being uplifted, of the required thickness of a protective layer that would prevent the geomembrane from being uplifted, of the tension and strain induced in the geomembrane by wind loads, and of the geometry of the uplifted geomembrane. The fundamental relationship of the geomembrane uplift problem is the "uplift tension-strain relationship" defined by (Zornberg & Giroud 1997):

$$\varepsilon_w = \frac{2T}{S_e L}\sin^{-1}\left[\frac{S_e L}{2T}\right] - 1 \tag{40}$$

where: ε_w = geomembrane strain component induced by wind uplift; T = total geomembrane tension; S_e = effective wind-induced suction; and L = length of geomembrane subjected to suction. Figure 40 shows a schematic representation of an uplifted geomembrane. It should be noted that the uplift tension-strain relationship (Equation 40) relates the strain induced only by the wind (ε_w) with the total tension in the geomembrane (T) induced also by other sources like temperature or gravity. In other words, Equation 1 is not a relationship between the wind-induced strain (ε_w) and the wind-induced tension (T_w).

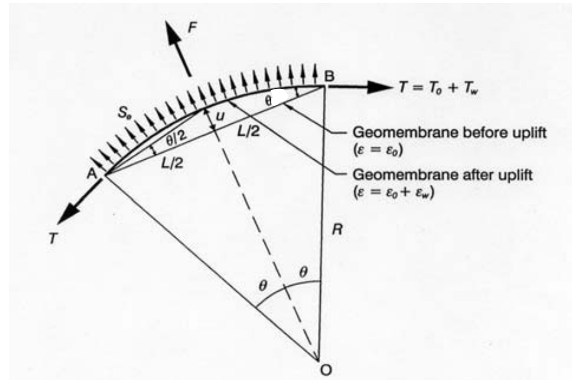


Figure 40. Schematic representation of an uplifted geomembrane (Zornberg & Giroud 1997).

The wind uplift pressure, S_e can be estimated for a given wind velocity as (Giroud et al., 1995; Dedrick, 1975):

$$S_e = 0.6465\,V^2 \tag{41}$$

Two solutions are available for tension in a geomembrane due to wind uplift: one for the simpler condition of a linear stiffness for the geomembrane, and a second solution for a nonlinear stiffness. If the geomembrane tension-strain curve, or a portion of it, can be assumed to be linear, ε_w can be estimated using the geomembrane tensile stiffness J, the initial tension T_0, the effective suction S_e , and the geomembrane length L by solving the following equation (Zornberg and Giroud, 1997):

$$\frac{S_e L}{2(T_0 + J\varepsilon_w)} = \sin\left[\frac{S_e L}{2(T_0 + J\varepsilon_w)}(1+\varepsilon_w)\right] \tag{42}$$

The expression above may be solved by trial and error in order to determine ε_w. After determining the wind-induced strain component, ε_w, the tension component induced by wind, T_w , can also be estimated using the geomembrane tensile stiffness J .

7.3.1 *Anchorage against wind action*

Alternative means have been proposed to provide anchorage to the expoced geomembrane cover to resist uplift forces. A method for designing anchor benches and trenches used to secure geomembranes exposed to wind action was presented by Giroud et al. (1999). Figure 41 shows a typical anchor bench. Three potential failure mechanisms are identified: (i) sliding of the anchor bench or trench in the downslope direction; (ii) sliding of the anchor bench or trench in the uplope direction; and (iii) uplifting of the anchor bench or trench. It is shown that the first mechanism is the most likely and that the third mechanism is the least likely. Criteria are provided by Giroud et al. (1999) to determine what is the potential failure mechanism in each specific situation. This is defined by the geometry of the slope on which the geomembrane is resting and the geomembrane tensions induced by wind action. It is also shown that a simple method, consisting of only checking the resistance of anchor benches and trenches against uplifting is unconservative as lateral sliding is more likely to occur than uplifting.

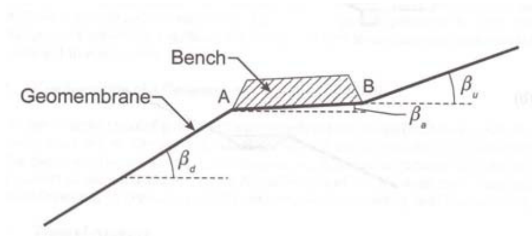


Figure 41. Configuration of an anchor bench to prevent wind uplift in an exposed geomembrane cover (Giroud et al. 1999).

7.4 *Case histories*

A number of exposed geomembrane covers have been recently designed using the aforementioned procedures for wind uplift analysis. Four of the recently constructed exposed geomembrane covers in the US are listed below (Gleason et al. 1998, Gleason et al. 2001; Zornberg et al. 1997). A fifth case history is detailed next. At each landfill, the design and operations criteria for the exposed geomembrane cover, as well as the rationale for constructing the exposed geomembrane cover were significantly different. The sites are:

- ≥ Crossroads Landfill, Norridgewock, Maine: an exposed geomembrane cover was designed and installed over a 2-ha landfill that had reached its allowable interim grades based on site subsurface stability. With time, the subsurface strata of clay beneath the landfill will consolidate and gain shear strength, thus allowing for additional waste placement.
- ≥ Naples Landfill, Naples, Florida: an exposed geomembrane cover was designed to provide a temporary cover for a 9-ha landfill for two purposes: (i) the exposed geomembrane cover was constructed a year prior to the planned construction of a typical final cover system in order to control odors associated with landfill gas; and (ii) on two of these slopes, the exposed geomembrane cover was installed over areas that will be overfilled in the near future.
- ≥ Sabine Parish Landfill, Many, Louisiana: an exposed geomembrane cover was designed and installed over a 6-ha landfill that had severe erosion because of long steep sideslopes that could not be reasonably closed using conventional closure system technology.
- ≥ A feasibility evaluation of the use of an exposed geomembrane cover was conducted for the OII Superfund landfill (see Case History 6). The main reason for having considered an exposed geomembrane cover at this site was the difficulty in demonstrating adequate slope stability, under static and seismic conditions, in the case of conventional covers where geosynthetics are overlain by soil layers. Although an evapotranspirative cover system was finally adopted at the site, an exposed geomembrane cover was also considered because it would have been stable under both static and seismic conditions.

<u>Case History 8: Delaware Solid Waste Authority (DSWA), Sussex County, Delaware, US</u>

An exposed geomembrane cover was designed and installed over a 17-ha landfill to provide a long-term cover system (i.e. 10 to 20 years) over waste that may be reclaimed at a later date (Gleason et al. 1998). Several geomembranes were considered for the design of the exposed cover system. Calculations for the selected geomembrane involved determination of resistance to wind uplift. A reinforced geomembrane with a linear stress-strain curve characterized by a tensile stiffness, J = 165 kN/m and a strain at break of 27% was selected for the design. The geomembrane anchors on the cover system were designed to include a swale that conveys storm-water runoff from the landfill in a nonerosive manner.

Figure 42 shows the exposed geomembrane cover placed over Cells 1 and 2 at the DSWA's southern facility. This cover was placed over 5% to 4H:1V slopes. The exposed geomembrane cover will be removed to allow potential mining of the in-place waste and placement of additional waste into the cells. A 0.9 mm green polypropylene geomembrane with a polyester scrim reinforced was used. In this application, the interface friction required of the geomembrane is defined by the swale anchorage structure.

Figure 42. Exposed geomembrane cover over Cells 1 and 2 of the DSWA's southern facility.

7.5 *Geosynthetic clay liners in covers: gas migration issues*

In recent years, design engineers and environmental agencies have shown a growing interest in the use of geosynthetic clay liners (GCLs) as an alternative to soil barriers as part of cover systems. GCLs were found to be very effective as hydraulic barriers in cover systems, easy to install, and could withstand distortion and distress while maintaining their low hydraulic conductivity (Bouazza 2002). As part of the evaluation process, the hydraulic properties of GCLs were considered to be the prime factors. However their performance as gas barriers has recently come under a growing scrutiny. With major environmental concern regarding gas emission, control of landfill gas is becoming an important issue for the protection of public health and safety. In other situations, covers may have to prevent oxygen from the atmosphere to come in contact with reactive materials, such as sulphidic tailings that could otherwise generate acid (e.g., Yanful, 1993; Cabral et al. 2000)

7.5.1 *Advective flow*

In advective flow, the gas moves in response to a gradient in total pressure. To equalize pressure, a mass of gas travels from a region of higher pressure to a lower one. In the context of landfills, the primary driving force for gas migration, especially through cover systems, is pressure differentials due to natural fluctuations in atmospheric pressure (barometric pumping). Falling pressures tend to draw gas out of the landfill, increasing the gas concentration near the surface layers. Conversely, high or increasing barometric pressure tends to force atmospheric air into the landfill, diluting the near surface soil-gas and driving gas deeper into the landfill (this is a possible explanation on how VOCs can find their way into groundwater). A change in the leachate/water table position or on temperature can also cause pressure differences and lead to gas migration. Recent studies have shown that the gas permeability (or permittivity) of GCLs may vary depending on the manufacturing process, moisture content and the overburden pressure during the hydration process (Didier et al. 2000a; Bouazza & Vangpaisal 2000; Vangpaisal & Bouazza 2001; Bouazza et al. 2002). Typical permittivity results versus volumetric water content for needle punched GCLs are shown on Figures 43 and 44. Commercially available GCLs consisting of essentially dry bentonite [powder in GCL1 and GCL2 (with bentonite impregnated into the cover non-woven geotextile layer), and granular in GCL3] sandwiched between polypropylene geotextile layers were used in the investigations, reported by the above authors. It can be observed that the variation of permittivity follows the same trend for both hydration conditions (i.e., confined under 20 kPa and 0 kPa). The permittivity increases with decreasing volumetric water content. Interestingly, at the same volumetric water content, GCL1 and 2 tend to have lower permittivity than GCL3. This is because of the nature of the bentonite in GCL3 (with bentonite in granular form). The hydrated granular bentonite is stiffer, clearly visible as soft grain particularly at the lower level of volumetric water content, than the hydrated powdered bentonite. This indicates the presence of larger inter-granular pore spaces, which provides preferential gas flow paths. The difference in permittivity of GCL3 to GCL 1 and 2 is lower at higher volumetric water content due to the reduction of inter-granular pore spaces when bentonite forms a gel and becomes softer. It is also observed from Figure 43 and 44 that the impregnated bentonite in the non-woven geotextile (GCL2) also contributes to the lower gas permeability, particularly in case of free swell hydration.

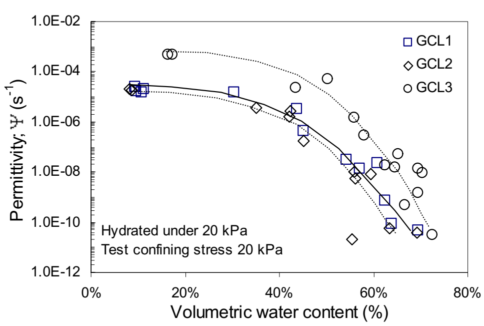


Figure 43. Variation of gas permittivity with volumetric water content for confined hydration (from Vangpaisal & Bouazza 2001).

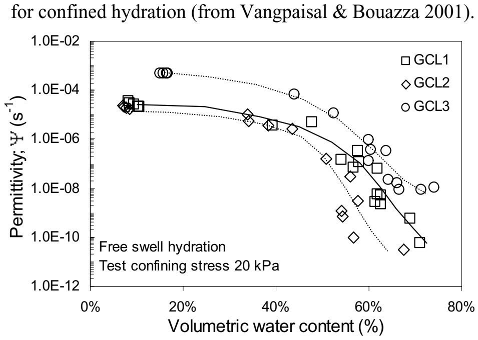


Figure 44. Variation of gas permittivity with volumetric water content for free swell hydration (from Vangpaisal & Bouazza, 2001).

Vangpaisal (2002) used a finite element model to simulate gas advective fluxes from the base of a multilayered cover containing a GCL. The cover system was assumed to consist of 5 layers (Fig. 45). The layers from top to bottom are (1) a topsoil layer, (2) a protective layer, (3) a drainage layer, (4) a GCL, and (5) a foundation layer. The total thickness of the cover was 1200 mm.

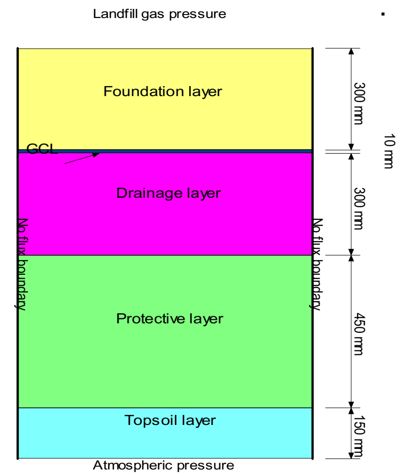


Figure 45. Domain simulated using finite element model

Results of his simulations are shown in Figures 46. Three conditions of the soils above the barrier layer (GCL) were simulated: wet soils (80% saturation), moist soils (60% saturation) and dry soils (40% saturation). The barrier layer (GCL) was assumed to have moisture contents of 120%, 90% and 70% respectively, representing wet, moist and dry conditions likely to exist at different landfill sites. The underlying soil was assumed to have a saturation degree of 85% in all simulations. It can be seen that gas flux is highly dependent on the condition of the cover system and differential gas pressure across it. Higher fluxes are obtained when soils above the barrier layer are drier because drier soils have higher gas permeability. Gas flux also significantly increases as the differential gas pressure increases particularly in the dry condition. For example at a differential pressure P= 10 kPa, gas flux through the cover in the dry condition is approximately 3000 times higher than in the wet condition, and 24 times higher than in the moist condition. A cover system incorporating a GCL as the sole barrier layer may not effectively impede gas migration, particularly in dry climatic conditions. However, in the moist and wet conditions, where the GCL tends to achieve high degree of saturation, the cover system becomes more effective in mitigating gas migration.

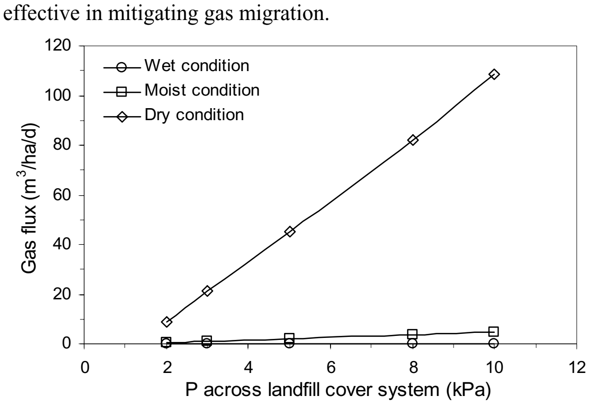


Figure 46. Gas flux through landfill cover systems at ranges of differential gas pressure

The finite element predictions of gas flux through a GCL cover for degree of saturation of GCL ranging from 40% to 80%, are presented in Figures 47a and b. Gas flux is clearly controlled by the degree of saturation of the GCL in the cover system and the condition of the soil above it. Gas flux decreases significantly as the degree of saturation of the GCL increases. The significant effect of the GCL degree of saturation on gas flux, particularly for the cover in dry condition, confirms that the GCL has an important role in cover systems in mitigating gas flux. At a degree of saturation of GCL of 40 % (MC = 50 %) and for the same differential gas pressure, the gas flux through the cover in a dry condition is approximately 8 times higher than the gas flux through the cover in a wet condition. However, the difference in gas flux from the two conditions is lower at the higher range of the GCL degree of saturation. The gas flux becomes equivalent at a degree of saturation of more than 70 % (MC = 100 %). At this level of saturation, the GCL effectively mitigates gas flow, and hence, changes in the condition of the layer components above the GCL have no effect on the effectiveness of the cover system to control gas migration.

Therefore, it is important that the GCL in landfill covers achieves moisture content of at least 100 % to effectively minimise gas migration. The GCL must also be protected from moisture reduction in order to maintain its long-term effectiveness. However, other considerations including the internal and interface shear strength must also be evaluated.

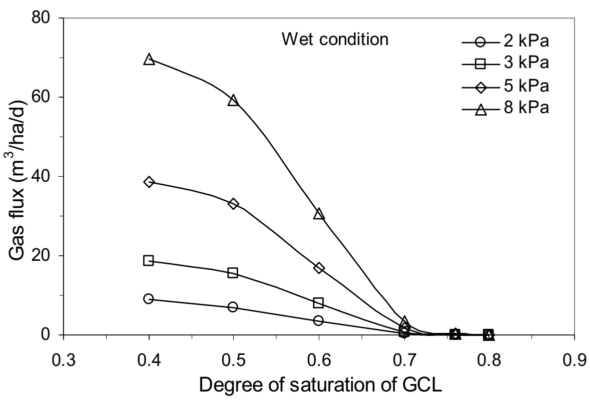


Figure 47a. Variation of gas flux through landfill cover at wet condition.

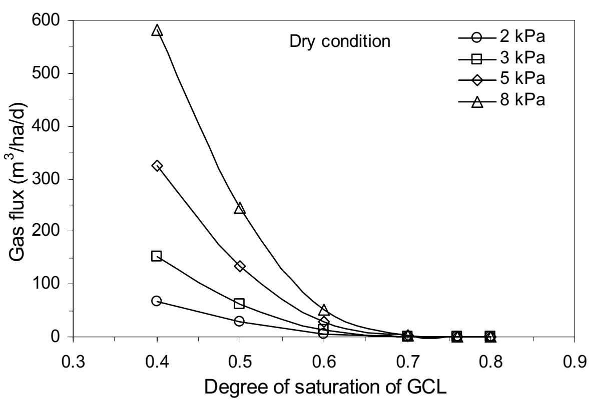


Figure 47b. Variation of gas flux through landfill cover at dry condition.

7.5.2 *Diffusive flow*

Gas movement by diffusion occurs due to molecular interactions. When a gas is more concentrated in one region of a mixture more than another, gas will diffuse into the less concentrated region. Thus molecules move in response to a partial pressure gradient or concentration gradient of the gas. Some applications of final covers require special considerations beyond those used for municipal solid waste facilities or capping of contaminated soils. These applications include facilities for mine wastes (e.g., sulphidic mine rock and tailings or uranium mill tailings). Covers for reactive mine waste must preclude ingress of water or oxygen to minimize oxidation of the underlying waste. Traditional principles can be used to design covers that will limit percolation. However, limiting oxygen transport requires special considerations.

Oxygen transport can be limited by including a barrier layer that impedes oxygen diffusion. Soil layers with high degree of saturation can limit oxygen diffusion since the liquid-phase diffusion coefficient for oxygen is orders of magnitude lower than the gas-phase diffusion coefficient. In particular, oxygen diffusion decreases as the degree of saturation increases (Yanful 1993). Aubertin et al. (2000) and Rahman et al. (2002) illustrated also the importance of moisture content variation on the gas diffusion of GCLs.

The diffusion of gases, such as oxygen, will occur mainly through the air filled pores of the soil with only a small amount occrring in the dissolved phase through the water filled pores. Aubertin et al. (2000) indicated that in a soil with volumetric air content θ_a (θ_a = n (1-Sr), n = porosity and Sr=degree of saturation), and volumetric content, θ_w (θ_w = nSr), the porous media diffusion coefficient, D_{pg}, can be expressed as:

$$D_{pg} = \theta_a D_\theta + \theta_w H D_R \qquad (43)$$

where D_θ and D_R are the diffusion coefficients for the air and water phases in the porous media, H is the modified Henry's law coefficient (H=0.03 for oxygen in an air water system at room temperature). The gaseous diffusion coefficient in soil D_0 is often related to the diffusion of gas through air, D_a. Many studies have been conducted to examine the variation in the ratio D_θ/D_a with the degree of saturation. Rowe (2001) summarized these results and formulated an empirical relationship for gas diffusion through soils. This relationship is given by:

$$D_\theta / D_a = \exp\left[-1.03\exp(0.017Sr)^{1.64}\right] \qquad (44)$$

where the oxygen diffusion through air, D_a, equals 2.06 10^{-5} m^2/s at 25^{o}C. Figure 48 presents the results of the model proposed by Rowe (2001) along with experimental results collected in various studies. The upper most and lower most values form the compiled results were selected to draw the upper and lower boundaries and these two curves are also presented in Figure 48. Recent laboratory test results obtained by Rahman and Bouazza (2002) are presented in Figure 48 for comparison purposes. The results in the figure show that the laboratory obtained values are significantly lower than those predicted by the model. Most values fall close to the lower boundary. The reasons for such disparities could be due to the fact that the model was developed for uniform porous materials (soils). If different layers are encountered, it is usual to consider equivalent properties. GCL is a composite material and there are uncertainties in distribution of bentonite and homogeneity in moisture distribution even in properly hydrated and cured samples. It is also difficult to consider the equivalent properties of GCL. In any case, a general trend similar to soils has been observed, the ratio $D_\theta//D_a$ was found to decrease significantly as the degree of saturation increased. Results obtained by Aubertin et al (2000) and Soltani (1997) are also presented in Figure 48. Soltani (1997) tested three different types of materials at different degree of saturation. It could be seen that the results vary significantly at similar degree of saturation, although they fall within the band drawn from the data sets of Rowe (2001).

This is probably due to variability in effective air porosity at high degree of saturation and product characteristics. Results by Aubertin et al (2000) seem to agree with the results obtained by Rahman & Bouazza (2002) on the same type of GCLs.

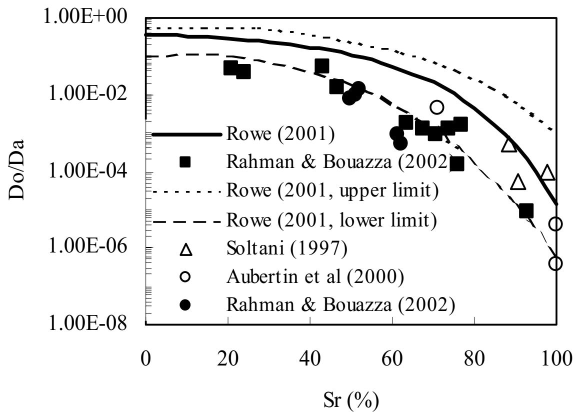


Figure 48. Diffusion coefficient as a function of saturation , model and laborarataory results (from Rahman & Bouazza 2002).

8 GEOSYNTHETICS IN CUT-OFF WALL SYSTEMS

8.1 *Overview*

The construction of cut-off walls, especially in geotechnical and hydraulic engineering, has been a traditional field for civil engineers for decades. In geoenvironmental engineering, they are increasingly used to encapsulate contaminated ground or contaminated sources (abandoned landfill, special industrial plants etc.) and waste containment facilities. The principle of encapsulation or in-situ confinement is to embed the cut-off walls in an artificial base or in a natural low permeable or aquitard stratum (Fig. 49).

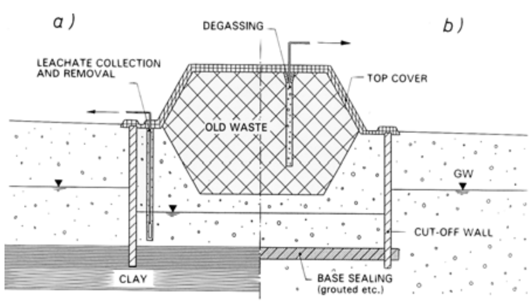


Figure 49. Encapsulation of a waste deposit and groundwater lowering within the cut-off walls (from Brandl 1994).
a) embedment of the cut-off walls in a natural in a natural low-permeable stratum
b) embedment of the cut-off walls in a natural in a grouted base sealing

There are several bjectives of in-situ confinement of waste containment and/or contaminated sites. The first and obvious objective is that of arresting any further migration of the contaminants. Accomplishing this objective will help in establishing spatial boundaries for future remediation efforts and will protect groundwater. Another objective of in-situ confinement is to take advantage of existing technologies, which afford us the opportunity to act immediately to contain the contaminated area. In addition, confinement, although it does not remediate the affected area, does afford the opportunity to return to the site at a latter time with a technology for remediating the contaminated soil and groundwater. The cut-off barrier can be placed upgradient from a waste site to prevent groundwater flow into the contaminated region, downgradient to prohibit flow of contaminant or leachate offsite, or around a contamination source to contain contaminants and inhibit the inflow of uncontaminated groundwater. Cut-off walls can be used as part of a full containment system (i.e., macro-encapsulation), on their own to prevent lateral migration of contaminant, or in conjunction with a cover system. The macro-encapsulation reduces the amount of uncontaminated upgradient groundwater entering the site, thus reducing the volume of leachate generated. When used in conjunction with some active means of reducing the hydraulic head within the barrier, the hydraulic gradient can be maintained in an inward direction, further preventing leachate escape.

The general design aspects of a vertical cut-off barrier are a function of various parameters, including the shape, extension and pollutants of the contaminated site, the geometry of the vertical wall as well as the required function(s) the cut-off wall has to fulfill (Brandl & Adam 2000, and Manassero et al. 2000). More importantly, the design and construction of cut off walls have to distinguish:

- If the cut off walls are temporary or permanent measures.
- If the cut off walls are keyed into an aquitard or only into a lower permeable stratum.
- If the cut off walls should only act as a hydraulic barrier or also as barrier against diffusion of contaminants.
- If the cut off walls have only a barrier effect or also a statical function.

Consequently there are several construction systems available to address the above range of requirements. Table 20 gives an overview of the currently used cut off wall technologies and their capacity. In the area of encapsulation, diaphragm walls and vibrating beam slurry walls ("thin diaphragm walls") seems to predominate (Brandl & Adam 2000).

Composite cut-off wall systems comprising soil or cement bentonite slurry with geomembrane insertion and single geomembrane walls are the two main types of cut-off walls using geosynthetics. The geomembrane cut off wall is mainly installed in soft or loose ground, whereas the composite cut off wall can be installed in all type of ground foundations. The main function of the geomembarne, usually an HDPE, used in cut-off walls is to build a low permeable barrier against contaminant transport and control possible gas migration. The installation methods of these types of barriers have been described in details by Koerner & Guglielmetti (1995), these methods are summarized in Table 21 and briefly discussed in the following section.

Table 20. Overview of methods for cut-off wall construction. Approximate values for common width d (m) and currently maximum wall depth t_{max} (m) (from Brandl & Adam 2000).

TECHNOLOGY	CUT-OFF SYSTEM	GROUND PLAN (schematical)	DIMENSIONS d (m)	t_{max} (m)
permeability reduction of in-situ soil	compaction wall		0,4-1,0 1)	10-20
	grouting wall		1,0-2,5	20-80
	soil freezing wall		≥ 0,7	50-100
	jet grouting wall		0,4-2,5	30-70
			≥0,15-0,3 2) (lamella)	20-30
	soil mix wall		0,8-1,5	30-60
	cut-mix-grout wall		≥ 0,7	10
soil displacement methods	geomembrane wall		≥ 0,005 (0,002) 6)	20-40
	sheet pile wall		≈ 0,02	20-30
	vibrating beam -slurry wall "thin (diaphragm) wall"		≥0,05-0,2 3)	10-35
	earth concrete driven-sheet pile wall		≥ 0,4	15-25
excavation methods	secant bored pile wall		0,4-1,5	20-40
	diaphragm wall (with hydrofraise)		0,4-1,6 4)	100-170
	diaphragm wall (with grab)		0,4-1,0	40-70
	diaphragm wall with incorporated liner(s)		0,6-1,0 (0,4-1,6) 5)	20-50

1) vibro-compaction, vibro-flotation (vibro-displacement, vibro-replacement); 2) total width of lozenge-shaped jet grouting walls: ≥ 0.5m; 3) near the flanges of the vibrating beam significantly wider; 4) up to 3.0 m in special cases; 5) in special cases; 6) in the case of twin walls

8.2 *Installation methods*

There are different installation methods currently available for the construction of geomembrane and composite cut off walls. Koerner & Guglielmetti (1995), Privett et al. (1996), and Daniel & Koerner (2000) provide a detailed overview of these methods, only a brief presentation will be given in the following paragraph, the reader is refereed to the above papers for further details regarding installation procedures. Koerner & Guglielmetti (1995) described three common methods for installing single geomembrane cut off walls, these are: 1) trenching method; 2) vibration insertion; 3) segmented trench box. Figure 50 shows the installation method based on the use of a trenching machine. A large bucket trencher or a disc cutter is used to excavate an unsupported trench in the ground a geomembrane is unrolled or inserted (see Fig. 51) in a self-supporting trench, which is afterwards backfilled with sand, native soil or drainage material or a combination, e.g. low permeable soil on one side and drainage gravel on the other.

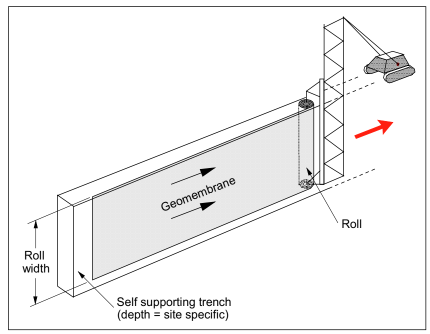


Figure 50. Trenching machine method (from Koerner & Guglielmetti 1995)

Table 21. Installation methods for geomembranes (modified from Koerner & Guglielmetti 1995).

Method or Technique	Geomembrane Configuration	Trench Support	Typical Trench Width (mm)	Typical Trench Depth (m)	Typical Backfill	Advantages	Disadvantages
Trenching machine	Continuous	None	300-600	1.5-15	Sand or native soil	• No Seams • Rapid installation • No slurry	• Depth limitations • Soil type limitation • Trench stability necessary
Vibrated insertion plate	Panels	None	100-150	1.5-20	Native Soil	• Rapid installation • Narrow trench • No material spoils	• Soil type limitation • possible panel stressing • Bottom key is a concern
Slurry supported	Panels	Slurry	600-900	50	SB, SC, CB, SCB, sand or native soil	• No stress on panels • Conventional method • Choice of backfill	• Requires slurry • Buoyancy concerns • Slow process
Segment trench box	Panels or continuous	None	900-1200	3.0-9.0	Sand or native soil	• Can weld seams • Visual inspection • No stress on panels • No slurry	• Depth limitations • Slow incremental process
Vibrating beam	Panels	Slurry	150-220	50	SB, SC, CB, SCB slurry	• Narrow trench • No material spoils • No stress on panels • Usually CB slurry	• Requires slurry • Slow incremental process • Soil type limitation

SB= soil-bentonite, SC=soil-cement, CB=cement-bentonite, SCB= soil-cement-bentonit

Figure 51. Geomembrane insertion in a dry trench.

The geomembrane can also be unrolled at the ground surface (Fig. 52), but the sheet must be distorted into an L configuration in order to reach its final position

Figure 52. Geomembrane installation for shallow cut-off walls.

In the vibrated insertion plate method, a steel insertion plate is used to support a geomembrane panel and to insert the geomembrane panel into native soil or backfill within a trench (Fig. 53). The geomembrane is pinned to the insertion plate at the bottom by means of dowels. A vibratory hammer is used to insert the plate and geomembrane to the desired depth. Once the desired depth is reached the insertion plate is withdrawn, leaving the geomembrane in the trench. This technique is limited to soft or loose ground.

The segmented trench box method uses a modified steel trench box to support the sides of the trench immediately following excavation (Fig. 54). The trench box is advanced along the length of the trench, and geomembrane panels are inserted in the gap between respective halves of the trench box.

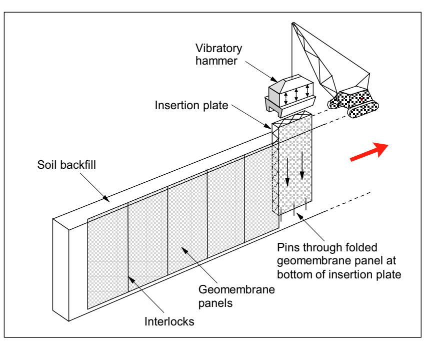


Figure 53. Vibrated insertion plate method (from Koerner & Guglielmetti 1995)

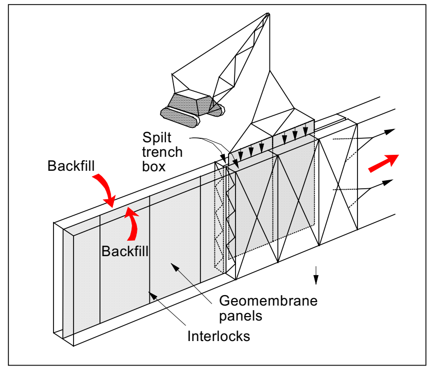


Figure 54. Segmented trench box method (from Koerner & Guglielmetti 1995)

To install a composite cut-off wall two methods are utilized: the vibrating beam method and the slurry supported method. Both methods use a bentonite slurry, either to support the trench (slurry supported method) or to create a thin slurry wall (vibrating beam method). The slurry-supported method is the most frequently used whereas the vibrating beam method is not very common.

In the slurry supported method (Fig. 55) soil is dug out under the protection of a self-hardening slurry (soil-bentonite or cement bentonite). The geomembrane panels are then inserted directly into the fresh slurry suspension using a steel frame for support.

The vibrating beam method combines the vibrating beam slurry wall technique ("thin diaphragm wall") with the vibrated insertion plate method (Fig. 56). First a thin wall of slurry is created as a beam is vibrated into the ground. After a section of slurry filled "trench" has been constructed, geomembrane panels are inserted in a manner similar to the vibrated insertion method.

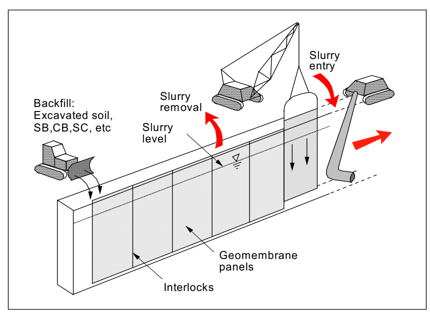


Figure 55. Slurry supported installation method (from Koerner & Guglielmetti 1995)

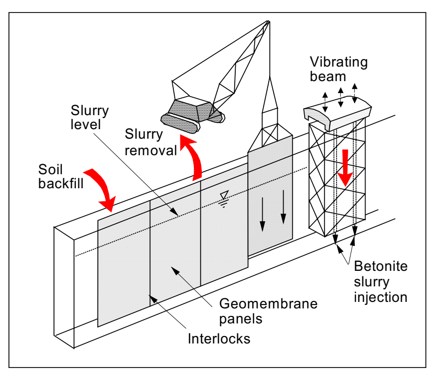


Figure 56. Vibrated beam method (from Koerner & Guglielmetti 1995)

A composite cut-off wall, as mentioned earlier, is a combination of soil-bentonite (SB) or cement bentonite (CB) and geomembrane walls. They increase the effectiveness of vertical barriers by combining the advantages of two systems: the diaphragm wall technique and the single liner (geomembrane) method. The attributes of the geomembrane and the SB or CB components are complimentary resulting in a composite wall that should be more effective than SB/CB or geomembrane walls in the same way that a composite liner is more effective than its soil and geomembrane components (Manassero & Viola 1992). Furthermore, a composite cut-off wall can offer an extra margin of safety against potential contamination (Koerner & Guglielmetti 1995; Brandl 1998; Manassero 1999).

8.3 *Hydraulic Performance of geomembrane and composite cut off walls*

8.3.1 *Interlocks performanc*e

The main concerns about the use of geomembranes in cut off walls are related to the joints, both between each geomembrane sheet and between the geomembrane and the base of the excavation. The base seal has been attempted by mounting the geomembrane on an installation frame and hammering the whole assembly a predetermined distance into the base layer. It is required to undertake this operation with some care to avoid damaging the geomembrane and the interlocks. An alternative technique is to over-excavate into an aquiclude to achieve an extended flow path past the toe of the geomembrane (Jefferis, 1993). The interlocking system of the geomembrane panels is one of the key elements governing the hydraulic performance of the cut-off wall. It is obvious that the objective of the interlocking connections is to perform as good as the geomembrane panels. To reach this goal the focus should be placed on proper installation and especially on testing the integrity of the joints. Presently, there are several types of geomembrane interlocking systems (Fig. 57).

Welding or grouting leads to a significant higher barrier effect than a mere plug-in connection. Consequently, homogeneous extrusion welding is used increasingly, but also interlocks with hydrophilic seals. This method uses primarily neoprene based rubber material as gasket. In contact with water the sealing material can swell up to 16 times its original diameter, thus creating a high sealing pressure. Additionally the lock can be grouted. For this technology sealing materials, which swell after coming into contact with hydrocarbons, are still evolving (Brandl, 1998).

Thomas & Koerner (1996) pointed out that settlement or lateral deformation might stress the connections during the installation process, and it was important to evaluate the strength and the permeation resistance of the interlocks. In this respect, testing the liquid tightness of interlocks in the field is very difficult but feasible. Cortilever (1999) described a field permeability test conducted on the joint systems used in the composite cut off walls which encapsulated the Schoteroog landfill in Harleem, the Netherlands. A special monitoring tube was welded on numerous joints to measure the hydraulic conductivity of the interlocks Figure 58. The water table within the wall was lowered one meter below the water level outside the wall creating an inward hydraulic flow. Measuring the water table within the tube over a certain period and comparing it to the ground water table outside the wall allowed the calculation of the flow through the interlock. The calculated average leakage through the joints was found to be 3.21 cm^3/day per meter lock at 1 m water head. During the summer (May to September) of 1996, 17.742 m^3 of leakage where discharged inside the landfill, whereas only 6.6 m^3 passed through the joints during the same period. For this project the amount of leakage passing through the joints was deemed to be negligible.

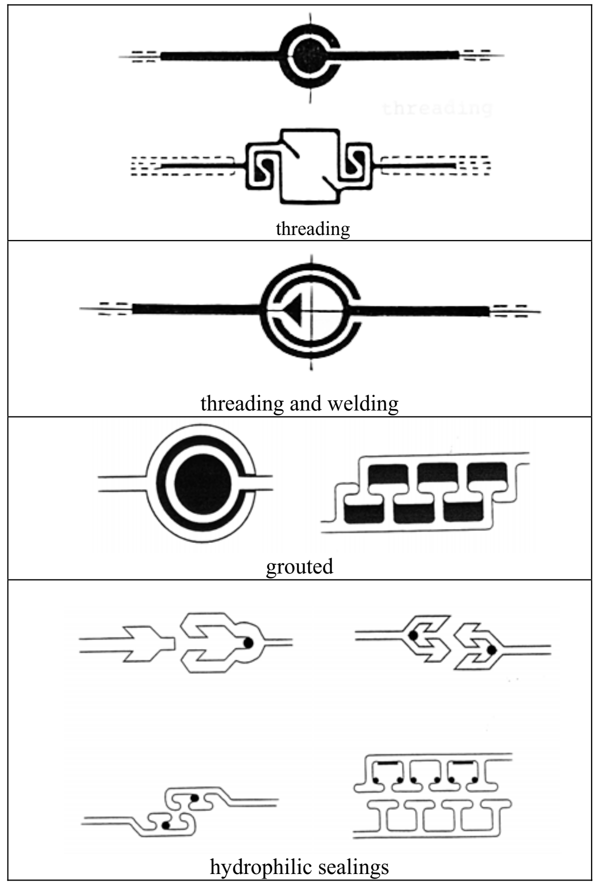


Figure 57. Locks of vertical geomembrane liners (from, Brandl, 1990, Manassero &Pasqualini 1992; Koerner & Guglielmetti 1995; and Thomas & Koerner 1996)

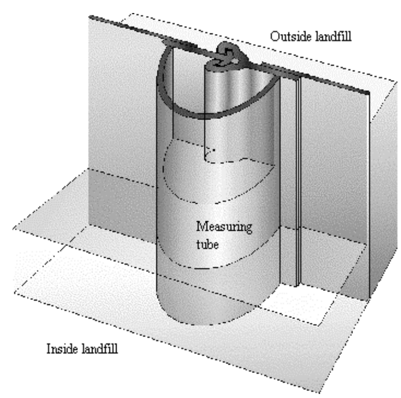


Figure 58. Schoteroog landfill: monitoring tube (Cortilever 1999)

Laboratory tests are also available to carry out at least a preliminary investigation. One of this test is shown in Figure 59, a preliminary assessment of the interlock performance can be made using equation 45 adapted by Manassero & Pasqualini (1992) from Giroud & Bonaparte (1989).

$$q = \frac{\pi\, k\,(\Delta H)}{\ln(2S/b)} \tag{45}$$

where k is the hydraulic conductivity of the slurry material (soil-bentonite or cement-bentonite), ΔH the difference between outside (the wall) and inside (the wall) piezometric levels, S the cut-off wall thickness, b the width of the equivalent “open slot” of the joint and q the flow per unit length of the slot.

By substituting current hydraulic conductivity values of the mineral component of the wall and the joint equivalent slot width (0.01 to 0. 1 cm) into equation 45 it is possible to observe the following results:

•≥ One order of magnitude of change in slot width modifies global hydraulic conductivity by a maximum of 15%.

•≥ The flow through the composite wall is directly related to the hydraulic conductivity of the mineral component of the wall.

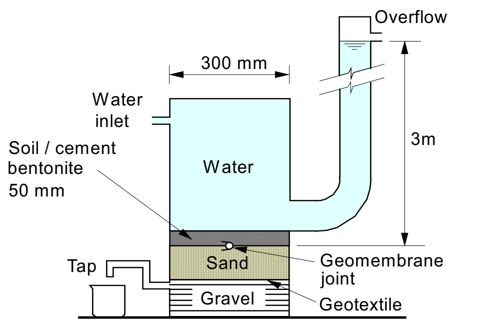


Figure 59. Laboratory set up for the assessment of geomembrane joint efficiency (modified from Cortlever 1988)

•≥ With current wall thickness, joint equivalent slot width and spacing, the geomembrane installation reduces global hydraulic conductivity by almost one order of magnitude.

•≥ Two or three orders of magnitude reduction in hydraulic conductivity might be obtained with the more sophisticated joint systems, which are currently available.

It is important to note that the test results are intended to act only as a general guidance on the effect of interlocks within the whole system. They are not representative of the real scenario because there are a number of assumptions, which have been made in the calculations.

Another laboratory test setup to measure the flow through an interlock is shown in Figure 60. In this particular test, pressures up to 700 kPa have lead only to a small amount of seepage passing through the hydrophilic gasket interlock.

There is no doubt that the joint sealing technology has dramatically improved over the past few years. However, there still remains concern about the chemical compatibility of the sealing material used in the joints or interlocks. Apart from the data reported by Esnault (1992), no other information is available in the literature on this particular aspect. Figure 61 shows the variation of the expansion of a hydrotite lock against time for samples immersed in water, water and cement and salted solutions of different strengths. The sealing material was found to swell up to 16 times its original volume when in contact with water, whereas the swell was only about 2 to 3 times its original volume when the sealing material was in contact with a solution containing 30% of salt.

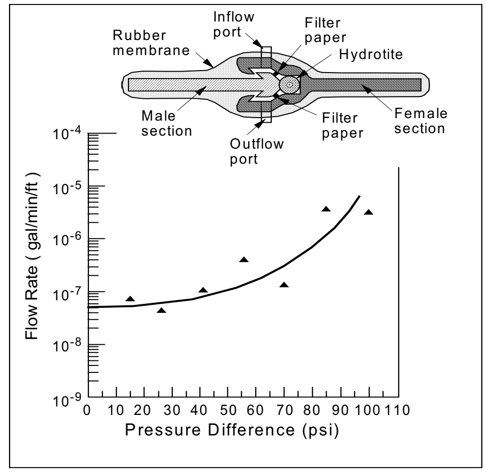


Figure 60 Hydraulic gasket interlock test (modified from Gundle Lining Systems, Inc 1993, after Koerner & Guglielmetti 1995).

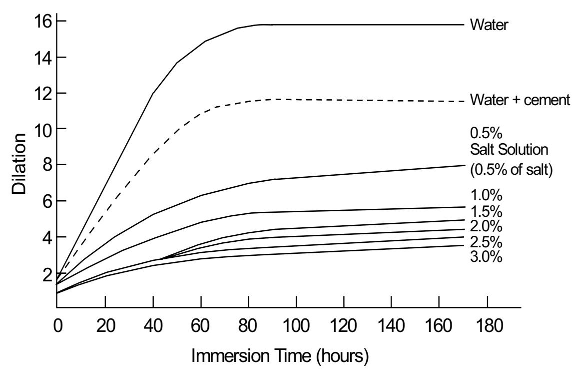


Figure 61. Expansion of hydrotite lock in water and in some mineral solutions (modified from Esnault, 1992)

The performance of such sealing materials can also be exacerbated by the installation procedure (in composite walls) since they are usually formed beneath the slurry with no control about their integrity. Koerner & Guglielmetti (1995) describe several methods to test the continuity of the different connection types. The installation of a contact element to the bottom and conductive wires up to the top of each side of the connection is an appropriate method to test the integrity of the interlocks. Measuring the resistance in the open versus the closed circuit verifies the continuity of the interlock at the intended depth. For grouted interlocks a visual way to inspect the joints is to observe the flow of grout coming out of the adjacent channel or outer pipe, respectively. Volumetric checks to provide a mass balance could be developed.

8.3.2 *Leak detection techniq*ues

Post-construction monitoring is necessary to ensure the proper function of the cut-off wall and to detect occurring leaks. There are various methods, including geophysical, electrochemical, and mechanical methods. All these techniques are under different stages of research and development, however none have been used with unconditional success and some are still at the experimental stage (Koerner 1998). This section of the paper is not intended to go into details of the available techniques but rather to draw the readers attention to the importance of leak detection. Inyang (1995) distinguishes between

•≥ barrier integrity monitoring, where changes within the barrier are monitored,

•≥ barrier permeation monitoring, where developments within the components of a containment system are measured, and
•≥ external monitoring, which is based on measurements outside the containment system.

A network of downgradient monitoring wells is the most often used method to provide leak detections. The number of wells to be installed is a very difficult question to answer, especially for geomembrane cut-off walls. The leakage would probably be from one or only a few sources. Consequently a system of widespread monitoring wells would perhaps fail because the plume is likely to concentrate on some narrow pathways (Koerner & Guglielmetti 1995). However, Koerner (1998) points out that if there are enough wells and the same pollutant can be measured in different wells, concentration gradient can be drawn and might lead to the detection of the leaks position in the wall.

For critical applications, however, a different strategy for leak detection should be considered when using geomembranes as vertical walls. Brandl (1998) presented a case study where a cellular cut-off wall was installed to deal with the possible leakage problems related to the use of conventional geomembrane cut off walls. The central Vienna (Austria) waste deposit, an area of approximately 60 ha, was surrounded by two parallel walls being connected by cross walls at certain longitudinal intervals to form a ring of consecutive cells around the deposit (Fig. 62). In each chamber the water was reduced to a level lower than outside but higher than inside the waste deposit. In combination with installed piezometer and wells in each cell the detection of leaks and an easy repair of the wall before pollutants may spread outside, is possible. Furthermore an inward seepage pressure, with counter effects towards an eventual diffusion through the wall, is created.

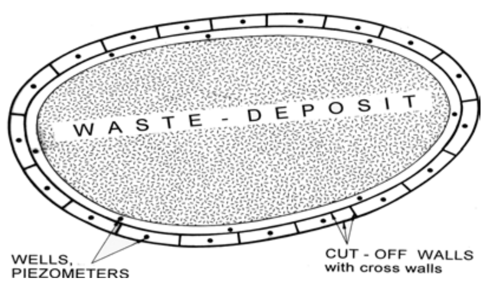


Figure 62. “Vienna cut-off double wall system” with a cellular screen a around waste deposit site.

Another vertical barrier, using the same principal – two parallel walls with a central monitoring and leak detection core - as the cellular cut-off wall is shown in Figure 63. This system, two parallel HDPE geomembranes with a central core (sand, grout mass, geosynthetics etc.) serving as monitoring and leak detection system, is used to encapsulate areas with high contaminate potential (Brandl, 1998). The technology is also available with a geonet leak detection layer between two geomembrane sheets or with a geonet between two geomembrane sheets backfilled with soil-bentonite, soil-cement, cement-bentonite or soil-cement bentonite. With sand as leak detection drainage layer, the geomembrane would be placed on each side of the excavated trench. With a geonet as the drainage layer , the composite system is placed against one side of the trench and the remainder backfilled as desired. For such a double liner system placed against the side of the trench facing the waste and the opposite side backfilled with a low hydraulic conductivity soil, cement or grouted material, one has the vertical equivalent of hazardous waste liner system type (Koerner & Guglielmetti, 1995).

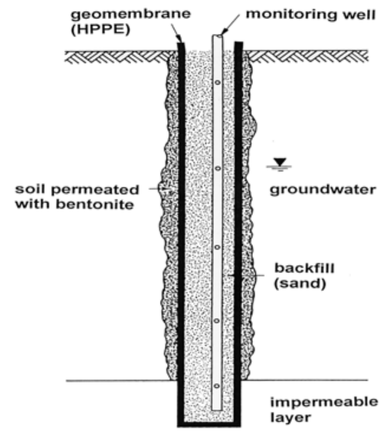


Figure 63. Geosynthetic – double wall system with monitoring and leak-detection system. (from Brandl, 1998)

8.4 *Groundwater flow*

There are three possible pathways for flow or contaminant transport past geomembrane or composite cut off walls: 1- through defects in the geomembrane or poor sealing of the interlocks of the geomembrane panels, 2- beneath the cut off wall, 3- diffusion through the cut off wall (Fig. 64).

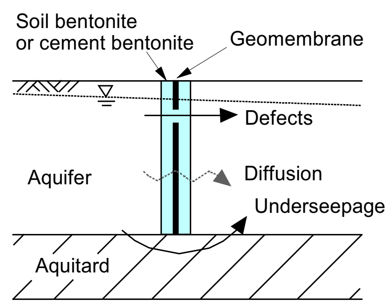


Figure 64. Possible pathways for flow past cut off walls (modified from Foose & Vonderemebse 2001).

Field data are rarely available to assess the above scenarios, numerical modeling and bench scale testing are the tools mostly used to investigate the possible behaviour of cut off walls in the field and identify the worst case scenarios. It is obvious that to create a cut-off wall without or just with a small area of defects represents one of the key issues for its function as a vertical barrier. Among various factors that influence the flow through the cut-off wall, defects of joints have the worst impact on transport rates. Brandl & Adam (2000) have pointed out that for a leak of only 1% of the screen area makes already a significant discharge of seepage water possible (Fig. 65). The barrier efficiency is only 10 to 20 % (depending on theoretical hydraulic assumptions), and it drops further if there are more leaks which in total exhibit the same area as one large leak (Fig. 66).

Manassero et al. (1995) proposed an analytical model for assessing the efficiency of composite walls. The model considers flow through the mineral backfill and through joints in the

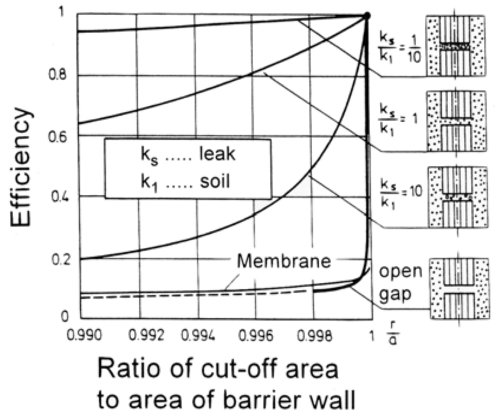


Figure 65. Barrier effect of cut-off walls with a leak (joint). Comparison of geomembrane and thick wall, influence of partial clogging of joints (after Brauns, 1978).

Example for a soil with effective grain size d_{w1} = 0,2mm.

a = axial spacing of cut off panels (= axial distance of joints)

r = effective length of cut-off panels

k_s = hydraulic conductivity of leak filling (due to clogging)

k_1 = hydraulic conductivity of surrounding soil

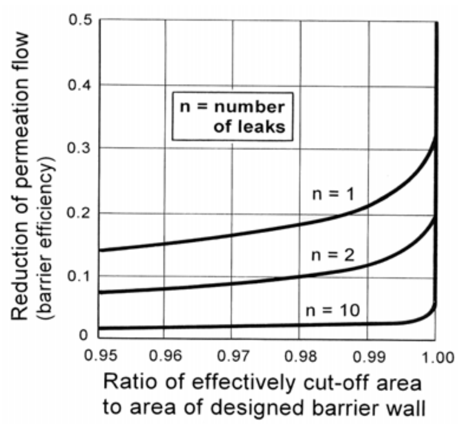


Figure 66. Barrier effect of cut-off walls with one or more leaks of the same total (cumulative) area.

geomembrane component of the wall. Parametric analyses showed that the spacing and the geometry of the joints had only a marginal impact on flow rates past the composite wall. However, the effective hydraulic conductivity of the joints and the hydraulic conductivity of the mineral backfill were found to have a strong influence on the flow rate. Manassero et al. (1995) showed that a composite wall containing a geomembrane, without defects and a joint sealing material with hydraulic conductivity 10,000 times lower that the mineral component of the wall, has a flow rate approximately 10,000 times lower than the flow rate past a simple mineral cut off wall.

Tachavises & Benson (1997a, 1997b) investigated the hydraulic importance of defects in vertical cut-off walls using a three-dimensional numerical groundwater flow model. They compared flow rates through soil-bentonite (SB), geomembrane (GM) and composite geomembrane-soil (CGS) cut-off walls. The focus was put on the influence of the location, size, hydraulic conductivity and penetration (meaning whether the defect is as thick as the wall or just influences part of the wall thickness) of the leak area on the flow through the barrier wall. All materials (geomembranes and joints included) were modeled as a porous medium, thus having different hydraulic conductivities.

Figure 67 shows the importance of a good key to reduce the flow rate. Hanging walls with a gap between the bottom of the wall and the aquitard layer were found to decrease the hydraulic effectiveness of the system, whereas, if there was good key flow rates were reduced significantly. Furthermore it was also shown that defective joints had a dramatic impact on flow rates past geomembrane walls. The joints where modeled as poor, semi-pervious and perfect. Although semi-pervious joints had a less dramatic impact on flow rates past the wall, they rendered a geomembrane wall far less effective. Variation of the joints width and position did not change significantly the results, similar findings were reported by Manassero et al. (1995).

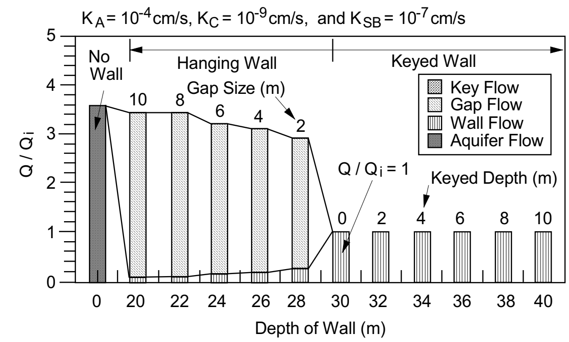


Figure 67. Normalized flow rates past a SB wall as a function of wall depth (from Tachavises & Benson 1997b)

Q_i = flow rate through intact wall in intimate contact

K_A = hydraulic conductivity of the aquifer

K_C = hydraulic conductivity of the aquitard

K_{SB} = hydraulic conductivity of the backfill

Composite cut-off walls are far less sensitive to leaks than single geomembranes. However, if high permeable "windows" in the soil bentonite wall and the defects in the geomembrane are lined up, then only a small area of leaks renders the wall practically ineffective (similar results as for soil-bentonite walls). In the case where the defects in the geomembrane and the high permeable windows are not lined up the flow rates past composite cut-off walls lies between those past intact geomembrane and intact soil-bentonite walls (Fig. 68). Figure 68 also shows that poor shells can reduce the effectiveness of such measures. This fact can also be seen in Figure 65, where clogging of leaks, caused by a good shell, led to a significant improvement of the barrier effect. Consequently, Brandl & Adam (2000) suggested that single geomembrane walls should be used for secondary purposes, in the case of low contaminant potential and for temporary measures. For the permanent containment of polluted areas with an excessively high contaminant potential slurry trench walls or geosynthetic twin-walls, as well as systems with leak detection and leakage removal are recommended.

Lee and Benson (2000) conducted an experimental study to evaluate factors affecting flow rates past geomembrane and composite geomembrane-soil (CGS) vertical cut-off walls. Intact composite cut-off walls were found to have flow rates as much as 100 times lower than comparable geomembrane walls. Flow rates increased when the GM walls contained defective joints. The flow rate for a sealed GM wall in direct contact with the aquitard was found to be 34 times higher than that of a sealed and keyed GM wall. This dramatic increase in flow rate is believed to be caused by underseepage. For all wall types, an effective key was required to achieve low flow rate past the wall

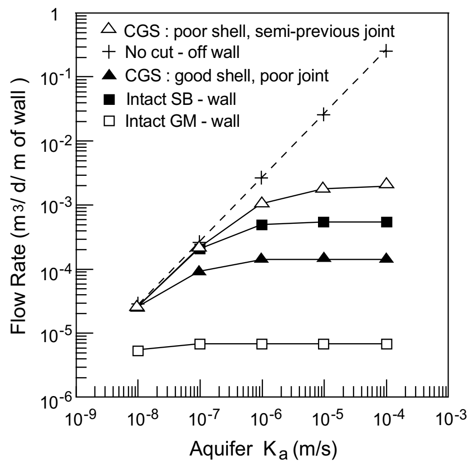


Figure 68. Flow rates trough soil bentonite, geomembrane and composite soil bentonite cut-off walls (modifed from Tachavises & Benson 1997a)

The influence of the joint seals on the effectiveness of GM cutoff walls is shown in Figure 69 where flow rates for keyed GM walls having various types of joint defects are plotted along with flow rates for the aquifer without a wall as a function of the average hydraulic gradient across the aquifer. Flow rates for the keyed GM wall with intact seal ranged from 9.2 to 34.8 L/day-m^2, with higher flow rates occurring at higher hydraulic gradients. Flow past the keyed GM wall is approximately 100 times lower than flow through the aquifer alone. Because the GM wall itself is essentially impervious, flow past the GM wall was mainly beneath the wall. Higher flow rates are obtained when a portion of the joint is unsealed because flow passes through the unsealed portion of the joint in addition to flow through the aquitard. When the joint is completely unsealed, flow past the wall is comparable to flow through the aquifer when no wall exists. These results are consistent with those from the modeling studies presented by Manassero et al. (1995) and Tachavises & Benson (1997).

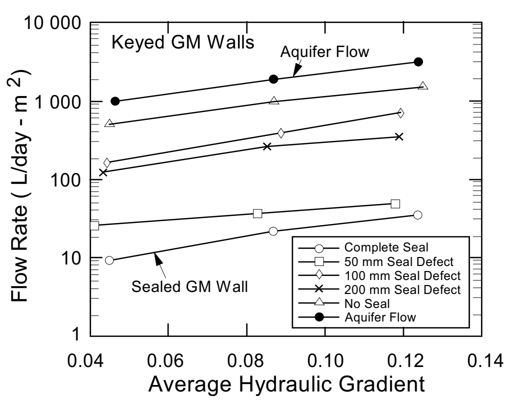


Figure 69. Flow rates for keyed walls (from Lee and Benson, 2000)

To further illustrate the importance of the joint seal, the quantity of flow passing through the joint was plotted in Figure 70 against the defective joint fraction. For a wall with a completely sealed joint, essentially no flow passes through the joint because the hydraulic conductivity of a sealed joint is extremely low and 100% passes through the aquitard. For walls containing seal defects, the unsealed portion of the joint conducts 35-98% of flow, with joints having larger unsealed portions typically conducting a greater percentage of flow

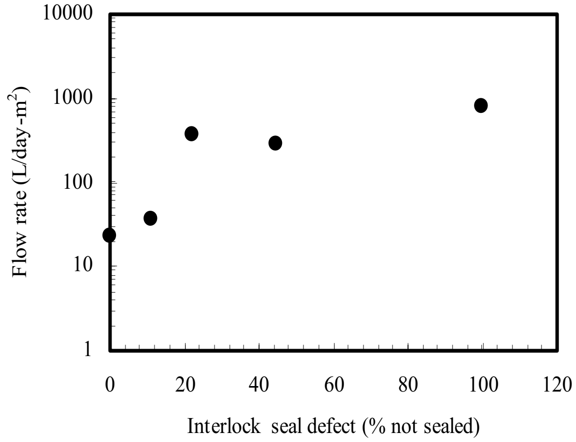


Figure 70. Flow rates and fraction of flow through sealsdefects in geomembrane walls (hydraulic gradient = 0.086). (Data source, Lee & Benson 2000).

8.5. Contaminant transport

Manassero et al. (2000) indicated that the performance of composite cut off walls, with regards to contaminant transport, are a function of the following factors : (1) thickness of the composite wall, (2) hydraulic conductivity, diffusion and sorption parameters of the mineral component of the wall, (3) diffusion coefficient of the geomembrane, (4) distance and equivalent width of joints, (5) length of the joint path, (6) hydraulic conductivity and diffusion parameters of sealing material filling the joints, and (7) geomembrane thickness. Most of the above factors have been investigated in details by various authors in relation to studies related to conventional cut-off walls, composite cut-off walls and lining systems. The exception, as discussed earlier, being factor number 6.

A comparison between a conventional cut-off wall and a composite cut-off wall in terms of advective transport has been presented by Manassero et al. (2000). It was found that significant reduction of advective flow could be obtained with the composite cut off wall. However, Manassero et al. (2000) have also shown that for some cases a geomembrane in a composite cut-off wall could have unfavorable effects on the contaminant transport. An inward hydraulic gradient and the presence geomembrane as liner in a composite cut-off wall were found to possibly lead to higher long-term external pollutant concentration in comparison to conventional slurry walls. This is due to both pure diffusion through the geomembrane and unfavourable advection and diffusion combination around the joint system. Similar conclusions have been obtained by Rowe et al. (1995) with regard to landfill liners analyses in transient conditions and referring to the same type of transport scenario. All these aspects must be carefully considered in the composite barriers design in order to optimise any dewatering system and barrier efficiency. As far as pathways for advective transport are concerned, the work reported by Foose & Vonderemebse (2001) indicated that the scenario was similar to groundwater flow (i.e. pathways will be through defects in the joints between geomembrane panels and seepage under the wall). Foose & Vonderemebse (2001) work also indicated that even for hydraulically effective composite cut-off walls, organic contaminant could diffuse through the wall. Figure 71 shows the variation of the ratio of mass flow rate of a VOC solute via diffusion to the total mass flow rate past the wall as a function of the defective joint fraction. The pathways for transport of a VOC solute are through defects in the joints, seepage beneath the wall, and diffusion through the entire face of the wall. For well constructed composite walls in direct contact with an aquitard having a hydraulic conductivity of 1 x 10^{-7} m/s, 15% to 20% of the contaminant transport is due to diffusion through the wall. Of the remaining contaminant flow, 80% is beneath the wall. If the hydraulic conductivity of the aquitard is 1 x 10^{-10} m/s, less flow occurs beneath the wall and more than 60% of the contaminant transported past the wall is due to diffusion through the face of the wall. If the wall is perfectly constructed and the hydraulic conductivity of the aquitard is 1 x 10^{-10} m/s more than 95% of the mass transport past the wall is due to diffusion. This indicates that diffusion of VOCs can be a significant pathway for contaminant transport and should not be ignored in the evaluation of the performance of composite cut-off walls in contact with organic contaminants.

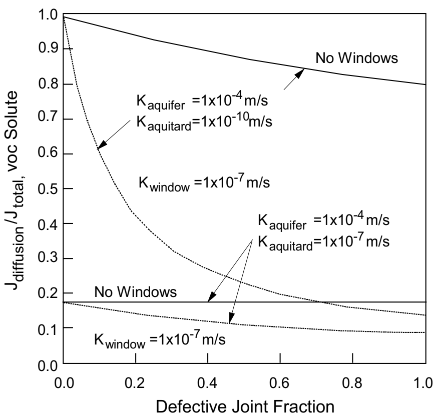


Figure 71. Mass transport of VOC solutes (modified from Foose and Vonderemebse, 2001).

It is worth noting that for special applications where hazardous constituents are migrating by diffusion, geocomposites can be used instead of single geomembranes. Such geocomposite is shown in Figure 72, it consists of two geomembranes with a drainage core as a leachate collection and removal system. With this lining system leak detection is possible. Furthermore, hazardous constituents, migrating by diffusion can be removed by the drainage system and the monitoring pipe (Brandl, 1994). Another alternative is the insertion of sandwich-like composite panels into the slurry trench wall (diaphragm wall). In this case a metal foil (usually aluminum) is placed between two HDPE-geomembranes. This method is theoretically promising if seepage or groundwater is heavily contaminated by chlorinated hydrocarbons, and also protects against certain vapor migration. Comprehensive practical experience does not yet exist (Brandl and Adam, 2000).

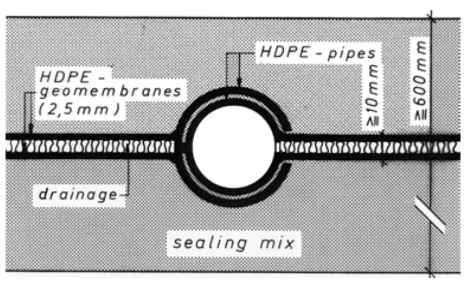


Figure 72.Composite cut-off wall with integrated geomembrane. Central geomembrane lining system with two geomembranes and a drainage core as a leachate collection and removal system (from

8.6. *Case Histories*

Case histories of successful application of composite cut-off wall as part of remediation strategies for contaminated areas have been reported by various authors (Barker et al. 1997, Bocchino & Burson, 1997, Burke et al. 1997). In all applications composite cut-off walls were considered to be the most feasible and cost effective solution among various alternatives. In most cases, the installation of high safety containment barriers was always accompanied by a special testing program prior, during and after construction.

Case History 9: Organic chemical plant, Liguria, Italy

The shutting down of a chemical plant due to the serious pollution of an adjacent river required a fast and safe remediation strategy to reopen the site as soon as possible (Manassero & Viola 1992). Site investigations had shown that the pollution of the river was caused by a groundwater flow from the contaminated subsoil under the chemical plant towards the river (Fig. 73). The geological profile of this site was nearly ideal for the construction of a cut-off wall as a barrier to protect the river from future pollution. A high permeable polluted layer of sand and gravel was underlaid by a low permeable, unpolluted layer of marl bedrock at depths between 3 –12 m. The final design emphasized on a precise constructed, high safety barrier (Fig. 74). Therefore, a 120 cm thick cement bentonite slurry wall was constructed prior the actual composite barrier. The outer wall was built to the base of the permeable sand and gravel layer with the aim to remove large boulders and to allow a precise installation of the inner wall. Afterwards the final barrier was constructed as a 60 cm composite cut-off wall reaching 2 m into the marl bedrock. Additional safety was added to the system by lowering the water table inside the contaminated area, thus creating an inward hydraulic gradient. The whole design and construction process was accompanied by a testing program including chemical compatibility testing of the cement bentonite slurry mixture, quality control of the slurry during construction and in-situ permeability tests inside trial panels, which were constructed close to and under the same conditions as the actual cut-off wall but without a geomembrane core. Data collected from the groundwater monitoring wells during the first time after finishing the construction showed a decrease of the pollutant concentration outside the vertical barrier of about one order of magnitude per year, thus proving the effectiveness of this chosen solution.

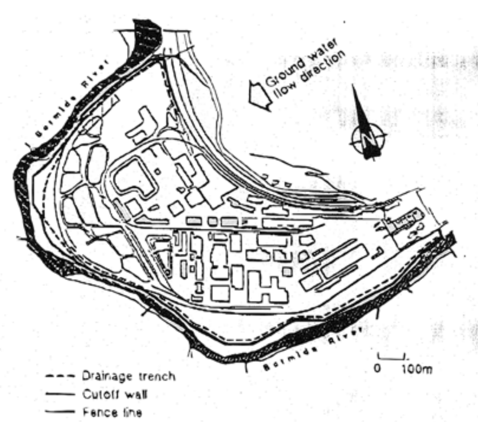


Figure 73. Plan view of the alignment of the slurry wall (adapted from Manassero & Viola 1992)

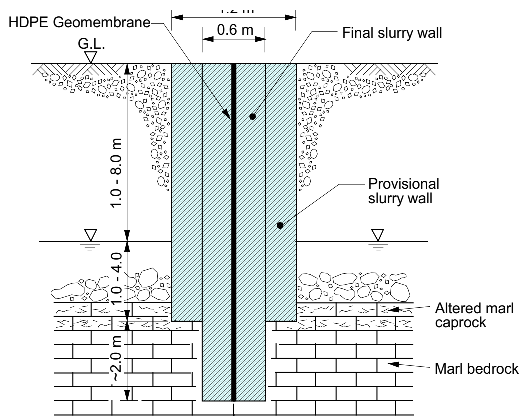


Figure 74. Schematic cross section trough the composite cut-off wall (from Manassero & Viola 1992).

<u>Case History 10: Manufactured gas plant, York, Pennsylvania, USA</u>

A second case history is presented herein to illustrate the use of HDPE panels as part of a remediation plan for a site contaminated with coal tar (Burson et al., 1997; Zornberg and Christopher, 1999). The site was a defunct manufactured gas plant located in York, Pennsylvania. The site is surrounded by commercial and residential areas and a creek (Codorus Creek) borders the site for a distance of approximately 305 meters. During years of operation and the subsequent closure of the manufactured gas plant, some process residuals migrated to subsurface soils and groundwater. Over time, the presence of coal tar-like material in the form of dense non-aqueous phase liquid (DNAPL), was observed seeping from the bank of the Codorus Creek. DNAPL was also noted in some monitoring wells on site.

Several remediation scenarios were evaluated with the purpose of intercepting the tar-like material migrating through the soil and into groundwater, encountered approximately 5.0 m below ground surface. A system consisting of a combination of soil improvement by jet grouting, a vertical barrier using HDPE panels, and a network of recovery wells was finally selected.

The use of vertical HDPE panels and trenchless technology allowed placement of the barrier as close as 3 m from the bank of the Codorus Creek, which was considered not to be feasible with a conventional slurry wall technology. The HDPE barrier system selected for this project was a 2 mm thick geomembrane, which allowed for the vibratory, trenchless installation. Sealing of the interlocks was achieved with a chloroprene-based, hydrophilic seal (see Fig. 57). HDPE panels were keyed into soil improved by jet grouting, as discussed below. The panels were installed using a conventional vibratory pile driving equipment, without a trench, thus reducing the amount of contaminated spoils to be disposed (Fig. 75).

In order to complete closure of the contaminated material, jet grouting was used to provide a seal to control DNAPL migration between the bottom of the HDPE panels and the irregular bedrock contact. Jet grouting consists of the high pressure injection of a cement and bentonite slurry, horizontally into the soil strata in order to improve its mechanical and hydraulic properties. The containment wall was approximately 290 m in length. The soils along the alignment of the barrier system consisted of granular fills, with large amounts of cinder material. Also mixed into the fill were varying amounts of rubble and debris. These highly permeable soils were underlain by the competent bedrock. Holes were pre-drilled down to bedrock, and the jet grouting improvement was done by injecting the grout horizontally from the competent rock up to an elevation approximately 6 meters below the ground surface.

A groundwater recovery system was implemented once the barrier was completed. Since its installation in the fall of 1995, the HDPE panel jet grout barrier system has performed as intended.

Figure 75. Installation of HDPE barrier wall utilizing conventional

9 GEOSYNTHETICS USE IN GROUNDWATER AND SOIL REMEDIATION

9.1 *Permeable reactive barriers*

Commonly, barrier walls are tight cut-off walls forming a passive containment. However, for in-situ groundwater remediation, cut off walls can be designed as permeable reactive walls (Fig. 76). The contaminant plume is then flowing through a straight or curved wall or can be directed to a gate. Groundwater cleaning in the reactive wall or gate is performed site-specifically, whereby physical, chemical and/or microbiological measures are possible. Several systems contain exchangeable geosynthetic filter panels, but also geotextiles to encapsulate special (granular) reactive material. A barrier wall may also consist of an alternating sequence of cut-off wall elements and reactive walls (Brandl & Adam 2000). At the present time the major limitation of this new treatment method is the relatively narrow range of contaminants that can be treated and the complications introduced by mixed contaminants (Jefferis et al. 1997).

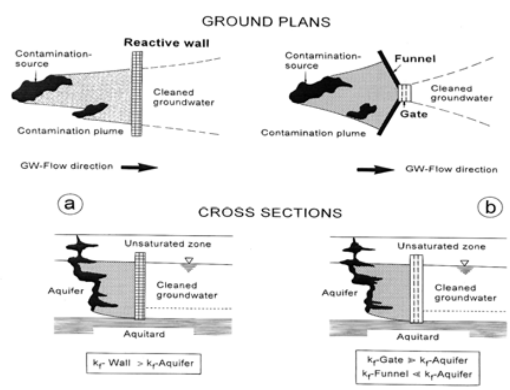


Figure 76. In-situ groundwater cleaning with permeable reactive (a) walls or “funnel and gate” system (b). (from Brandl & Adam 2000)

9.2 *Use of Prefabricated vertical drains for soil remediation and methane extraction.*

Prefabricated vertical drains (PVDs) are used extensively for dewatering of low permeability soils. In dewatering applications, the drains function to decrease the length of the flow path for water to escape from within the fine-grained soil and thus shorten the length of time needed for dewatering the soil and accelerates in this way its consolidation. The design of prefabricated vertical drains varies according to a specific application. Key parameters usually addressed in the design involving liquid movement include: Equivalent diameter of the drain which dictates the size of the inflow surface; and discharge capacity of the drain.

Recent work has adopted PVDs for in situ remediation of contaminated soils below and above the groundwater table. Below the gound water level, the technology is an enhancement of the soil flushing technique (Quaranta, et al. 1997). PVDs may also be used above ground water level to enhance soil vapor or methane extraction (Collazos et al. 2001).

9.2.1 *Enhanced Soil Flushing Using PVDs*

Soil flushing is a treatment technology that removes contamination located below ground water level with the aid of flushing solutions. The technology uses injection and extraction wells that are strategically located within the contaminated zone. The soil flushing equipment injects the flushing solution under a positive pressure into the injection wells. The solution permeates the contaminated soil picking up the contaminants on its path toward the extraction wells, which are used to collect the contaminated pore water (Quaranta et. al. 1997). Soil flushing becomes less efficient in fine-grained soil. The extremely small pores in fine-grained soil limit the rate which flushing agents can be delivered into or extracted from the soil.

A soil flushing technique based on the use of PVDs has been developed and employed successfully for removing contaminants from below the groundwater table in fine-grained soil (Gabr et al. 1996, 1999; Welker et al. 1998; Quaranta et al. 2000). In this case, the system relies on the flow capacity of the geocomposite drainage systems (strip drains). It involves installation of strip drains on a grid pattern similar to the process used for accelerating consolidation of soft soil deposits and it is used in the same way as pump and treat well systems are used in coarse-grained soils

The soil flushing technology is operated in a manner in which alternating rows of drains are either operated as injection wells or extraction wells. The extraction is accomplished by pulling a vacuum on drains. Liquids containing the contaminant are removed from the subsurface via the extraction drains and clean liquid is returned through the injection wells. The technology can be used in low hydraulic conductivity soils (10^{-5} to 10^{-10} m/s) where conventional pump and treat wells become ineffective, it is known as the Well Injection Depth Extraction (WIDE) system. The system has been field demonstrated and shown effective with removal of trichloroethylene and BTEX compounds along with uranium and technicium 99 from soils with hydraulic conductivities in the 10^{-8} to 10^{-9} m/s range.

9.2.2 *Vapor or gas extraction using PVDs*

Soil vapor extraction, known as SVE, is an in situ treatment technology that uses vacuum blowers and extraction and vent wells to reduce concentrations of volatile organic compounds (VOCs) and some semi-volatile organic compounds (SVOCs) from unsaturated soil (vadose zone). The extracted vapors are typically treated at the surface and released to the atmosphere or injected back into the subsurface. The factors that control the performance of a soil vapor extraction are: the chemical composition of the contaminants, vapor flow rates through the unsaturated zone, and the flow path of carrier vapors relative to the location of the contaminants. Prefabricated vertical drains can be used to place "wells" at close spacings thus decreasing the travel time for air to pass through the soil and increasing the opportunity for interception of the contaminant vapors. The many vents or extraction points afforded by the drains provides more options for better control of the flow regime (Fig. 77).

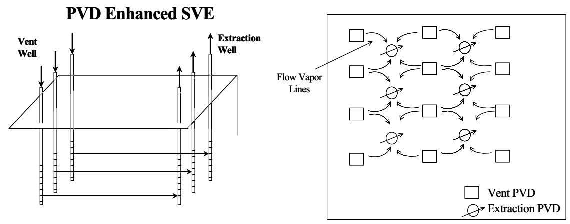


Figure 77. (a) Vent and extraction PVDs (b) Flow path direction

In a large scale laboratory testing, Collazos (2000) and Collazos et al. (2001) have found that assumptions regarding the equivalent drain diameter typically used with prefabricated vertical drains in soil dewatering projects had little impact on the resulting air permeability calculated from the drawdown curves. A field demonstration of the PV drain enhanced SVE technology has also been performed at a site in Columbia, Missouri, USA (Collazos et al. 2002). The site is an active municipal solid waste landfill at which methane gas is migrating laterally though the subsurface away from the facility. A plan view of the situation is shown in Figure 78. The study has been conducted around gas monitoring probe #9. Methane (CH_4) levels in the monitoring probe have been recorded at concentrations as high as 45 percent methane, the concentration of methane was found to vary during the course of the year (Fig. 79).

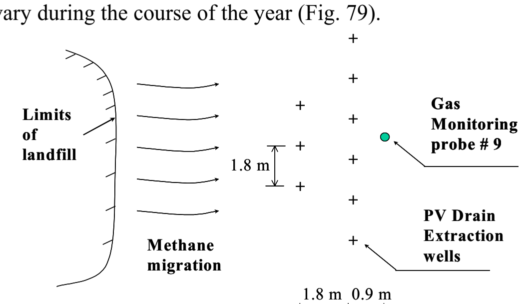


Figure 78. Plan view of the field demonstration site for PV drain enhanced soil vapor extraction system for interception and capture of methane gas migration in subsurface (from Collazos et al. 2002).

The objective of the study was to intercept the migrating methane, capture it and recover it thereby reducing the concentrations in monitoring probe number 9 and arresting the lateral migration. The situation presented a good opportunity to test and evaluate the performance of PV drains to extract gases from the subsurface.

Nine PV drains were installed in two rows (Figure 78 and 80) between the landfill and probe #9. The PV drains were equally spaced at a distance of 1.80 m. The depth of the PV drains ranged from 5.5 to 6.0 m. The objective of the PV drain SVE system was to capture the migrating methane gas. The methane concentration in gas monitoring probe number 9 was continuously measured as an array of different PV drains was operated as extraction wells.

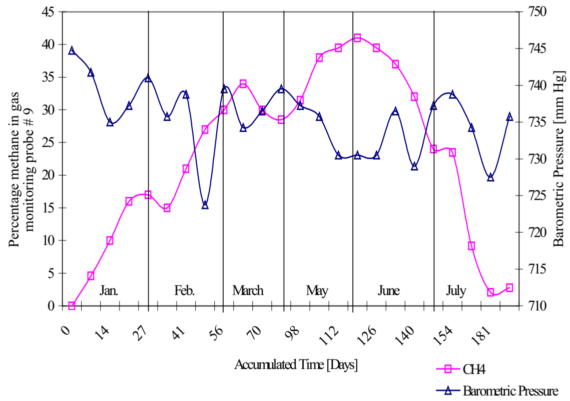


Figure 79. Contamination of methane gas in monitoring probe # 9 over a period of seven months. The barometric pressure is also shown during this period (from Collazos et al. 2002).

Figure 80. PV drains at the Columbia landfill site.

The results are shown in Figure 81. The percentage of methane gas in the monitoring probe decreased for all combinations of PV drain extraction wells. Initially all nine PV drains were open for extraction. Then successively, the extraction field was decreased by one PV drain until only one PV drain was operating. In all cases, the methane level in the monitoring probe decreased. The decrease was rapid at first, when all PV drains were extracting, and decreased with time. The decreasing rate at which the concentration in probe number 9 was lowered can be attributed to the low gas permeability of the soils. The rate at which the methane migrated through the soils could not maintain steady state with the extraction rate. The break in the data was a two hour period when no extraction was taking place. Even after this delay, the methane concentration in the monitoring well did not recover. Measurements on the gas probe seven days after the last extractions showed that the methane concentration was only 4.3 percent. The gas migration rate is very slow; however, the data in Figure 80 show that we are entering an historical period of low methane concentrations for the site. Additional testing is being conducted in order to determine the in situ intrinsic permeability of the soils and to ascertain the radius of influence for the individual PV drain wells.

Both the WIDE system (for below the groundwater table) and the PVD enhanced-SVE system (for above the groundwater table) offer advantages to existing in situ remediation technologies. Multiple drains (wells) provide redundancy in the case of drain failure, increase the likelihood of intercepting the contaminant, can be operated in a variety of schemes in order to ensure contaminant plume capture and decrease the potential for contaminant excursions from the treatment volume. The WIDE system has been successfully field demonstrated and is fully operable. Evaluation of the operational parameters for the PVD enhanced-SVE system is still on-going. These developing technologies make full utilization of the benefits of geosynthetics drainage systems. They are chemically compatible (Logan 1998) with most contaminants, economical to use, and provide the necessary flow capacities for the remediation systems

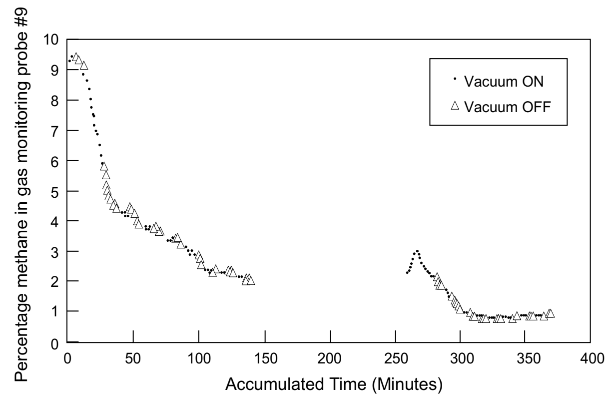


Figure 81. Percentage of methane in gas monitoring probe # 9 during soil vapor extraction using the PV drain system (from Collazos et al. 2002)

10 CONCLUSIONS

Geosynthetics are actively used in waste containment facilities and will continue to be significant components for many of the multiple systems in a landfill facility. The inclusion of geosynthetic components is firmly entrenched in designs and is likely to expand as manufacturers develop new and improved materials and engineers/designers develop new analysis routines. The geosynthetics industry is certainly well equipped to tackle new design challenges, the innovative use of geosynthetics in landfills reported in recent years is a testimony of its readiness and willingness to respond to such challenges.

This paper centered on recent advances in the use of geosynthetics in bottom liners, sidewall liners, cover systems, as well as in vertical cutoff walls and remediation work. The salient conclusions that can be made are as follow:

The contribution of a geomembrane is significant in decreasing the overall hydraulic conductivity of composite barriers, in limiting the diffusive transport of some types of pollutants, and in delaying direct contact between the mineral liner and the leachate, reducing thereby potential compatibility problems.

Based on the geomembranes service life, it is acceptable to design a landfill liner with confidence on their performance in medium and long term (i.e. 50 to 350 years). This conclusion is also strengthened by the fact that following landfill closure (assuming that a low permeability capping system has been used) and towards the end of the service life of the leachate collection system, the seepage velocity through the basal lining system (and therefore, the advective transport of the pollutants) will be mainly governed by the cover system and by climate conditions (i.e. hydrological balance of the landfill)

The method of selecting protection layers based on performance testing seems to be applicable. However, recent research work clearly indicates that even robust protection layer materials currently in use are not capable of meeting the <0.25% peak strain requirement. This has raised the question of credibility and accuracy of the 0.25% strain criterion. Further investigation in its suitability and accurateness is warranted. Finally, recent investigations point to the fact that typical geotextile protection layers are not adequate for controlling the local strains in the geomembrane.

The bulk of the geomembrane defects reported by various authors have been related to mechanical damage caused by the placement of soil on top of the geomembrane. There is strong need to minimize geomembrane installation and post-installation defects.

A number of studies on leakage rates have been carried out on landfills with secondary leachate collection systems (leak detection systems or LDS) by measuring the flow in these systems. These studies have highlighted the importance of secondary leachate collection system monitoring during the different life cycle stages of a landfill.

Data available suggests that GCLs have very low hydraulic conductivity to water and can maintain their hydraulic integrity over the long term. Critical aspects about the service life of the GCL as far as hydraulic integrity is concerned are related to long-term chemical compatibility problems, penetration, localized loss of bentonite, bentonite thinning, piping phenomena and ion exchange. Finally, geosynthetic clay liners, integrated with an attenuation layer are considered as a possible alternative to compacted clay liners in composite liners. However, careful comparison must be carried out between the two alternatives on a case by case basis. The actual boundary conditions, the different pollutant transport phenomena, the contaminant lifespan and the active service life of the barrier materials and other landfill components must be taken into account.

When designing GCL-lined slopes it is essential to recognize the differences between different types of GCLs and, consequently differences in interface and internal shear strengths. Significant databases on internal and interface GCL shear strength values have been recently compiled, which provide good understanding of the probabilistic distributions of the peak and large displacement strength values. These results are suitable for future reliability based stability analyses.

Calculating the thickness of liquid in a liquid collection layer is an important design step because one of the design criteria for a liquid collection layer is that the maximum thickness of the liquid collection layer must be less than an allowable thickness. Simple equations have been developed to calculate the maximum thickness of liquid in a liquid collection layer. Such equations are suitable to define transmissivity requirements of liquid collection layers in single and double slopes.

Major advances have recently taken place regarding the use of geosynthetic reinforcements to allow significantly steeper and higher final cover systems. Solutions are presented for the case of unreinforced, slope-parallel, horizontally-reinforced and fiber-reinforced veneers. As expected, additional reinforcement always leads to a higher factor of safety while increasing slope inclination would typically lead to decreasing stability. It is worth noting that increasing soil friction angle leads to increasing stability, when compared to the unreinforced case, only for the case of fiber reinforced slopes. It should also be noted that increasing total height of the slope (or increasing total length) does not affect detrimentally the efficiency of horizontally placed reinforcements and of fiber reinforcement.

The use of reinforced soil structures has also been extensively used for stabilization of waste cover systems. The design of these systems does not differ from the design of these systems for other applications such as transportation infrastructure. It should be noted, however, that the response of these reinforced soil structures has been adequate even when founded on highly compressible waste material.

Exposed geomembrane covers have been recently analyzed, designed, and constructed to provide temporary and final closure to waste containment facilities. Key aspects in the design of exposed geomembrane covers are assessment of the geomembrane stresses induced by wind uplift and of the anchorage against wind action. Procedures for the analysis of geomembrane wind uplift and methods for designing anchor benches and trenches used to secure geomembranes exposed to wind action have also been developed. The use of exposed geomembrane covers is particularly suitable in sites with steep landfill slopes and in landfills where leachate recirculation is considered.

Although GCLs are usually installed to limit water infiltration they may also serve an important role in covers as gas barrier. The gas permitivitty of needle punched GCLs was found to vary depending on the manufacturing process, volumetric water content and the overburden pressure during the hydration phase The diffusive transport was found dependent on moisture content variation, which reinforces the fact that the GCLs should remain fully saturated in order to mitigate gas migration due to diffusion.

Cut-off walls have gained popularity as barriers to contaminants. However, the role of diffusion of contaminant should be carefully evaluated. It was shown that the use of geomembranes could be detrimental for certain boundary conditions (inward hydraulic flow) due to both pure diffusion through the geomembrane and particular combinations of seepage and diffusion through the joint system. Furthermore, recent work indicates that diffusion of organic contaminant can be a significant pathway for contaminant transport and should not be ignored when evaluating the performance of composite cut-off walls in contact with organic contaminants.

Joint sealing technology has dramatically improved over the past few years. However, there still remains concern about the chemical compatibility of the sealing material used in the joints or interlocks.

Geomembrane cut-off walls are very sensitive to leaks. A leak area of only 1% reduces the barrier effect of geomembrane cut-off walls up to 80 - 90%. On the other hand, composite cut-off walls, which perform well even with geomembrane defects, are an adequate measure to increase the effectiveness of a vertical barrier. Therefore, for high risk applications, geomembranes should be used as liners in slurry-trench walls or as geosynthetic twin-walls. Applications with leak detection and removal systems should be preferred.

Geosynthetics, in particular, prefabricated vertical drains can successfully provide alternatives to conventional remediation systems.

Recent case histories, presented throughout the different topics covered in this paper, document the implementation of recent advances the use of geosynthetics in landfill engineering practice.

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