GEOENVIRONMENTAL APPLICATIONS OF GEOSYNTHETICS

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Abstract: The Keynote Lecture focuses on the use of geosynthetics for geoenvironmental applications that are linked with landfills, mining applications and ponds for environmental protection. The Keynote Lecture gives insight into the different applications and the corresponding design theories, regulations and standardization when appropriate, from all over the world. Special attention is given to Europe. The questions of geopipes and geomembranes under high loads are addressed, as in landfill and mining applications high loads can be encountered. The topic of geomembrane protection is then discussed, mainly though large laboratory and field tests. The question of the geomembrane protection in case of geomembrane wrinkles is also discussed. Quality assurance and leak detection are then addressed as those points are crucial for the use of geosynthetics. Geosynthetic barrier field performance including use of GCLs in landfill caps, bottom liners temperature and GCL shrinkage are presented. A detailed review of the modelling of advective flow through composite liners based on analytical solutions, empirical equations and numerical modelling for the quantification of leakage both for liquid and gas flow is then given. Migration and attenuation through GCLs is then discussed prior to a discussion of equivalence of composite liners based on elements presented in the previous sections. Modelling for soil liners and composite liners is presented.

Keywords: landfill, mining, hydraulic properties, diffusion, chemical compatibility, field performance.

INTRODUCTION

In recent years there have been many advances in the understanding of issues related to the use of geosynthetics such as geomembranes and geosynthetic clay liners (GCLs) as contaminant barriers. As a consequence there has also been a significant increase in geoenvironmental applications (Rowe 2007). These applications range from the traditional use of geosynthetics in landfill base liners or drainage layers, landfill capping systems, liquid waste ponds, containment of past spills of hydrocarbons, secondary containment around fuel tanks, containment for fluid or gas in mining applications, for example.

The objective of this Keynote Lecture is to focus on the use of geosynthetics in landfills, mining applications and liquid waste ponds.

The disposal of waste materials is a matter of increasing public concern. The major component of the solid waste disposal system in almost every country is the landfill. During the last few years, in some countries, legal, economical and educational efforts have led to a significant reduction in the generation of waste. In some cases the design of landfills facilities is delayed or can be given up completely due to the decreasing amount of refuse. In spite of this development, there is still, and will be, a great demand for solid waste landfills in most parts of the world. The design and construction of landfills remains a major challenge to civil engineers (Zanzinger & Gartung 2002). As there is a close relationship between the sealing and dewatering elements of the basal liner and of the cover barrier (Zanzinger & Gartung 2002) both aspects will be presented in the first section of this Keynote Lecture dedicated to an overview of the use of geosynthetics for geoenvironmental applications.

Mines, unlike other facilities such as dedicated landfill sites, usually have a much larger foot print, therefore environmental protection measures have to be practical, robust, sustainable and environmentally sound (Renken *et al.* 2005*a*). Consequently the utilization of geosynthetic materials for the design and construction of liner systems in mining applications has increased over the last two decades as the performance of geosynthetic materials in the mining environment becomes better understood. A description of the use of geosynthetics for mining applications will also be given.

Liquid containment represents the historical application for geomembranes, or of what can be considered as the historical ancestor of geomembranes (Koerner 2005, Montjoie *et al.* 1992). In liquid waste ponds the leachate head or liquid effluent can be even greater than in landfills, leading to potential larger flow rates in ponds. Consequently lining and drainage of ponds will be discussed in the following including lined ponds for mining applications.

The paper will then address eight issues of importance to the use of geosynthetics in geoenvironmental applications. The first two issues, design of plastic pipes under high loads and design of geomembranes under high loads, arise from the need in mining applications to consider the presence of high stress levels up to 4MPa. Design methods for geopipes with deep burial depths are not well defined nor understood. Understanding the performance and limitations of geopipes in deep burial applications is important as pipes are being designed and installed in harsh environments. The state of the art in this area will be presented based on modelling, laboratory experiments and field investigations.

The selection and design of the geomembrane liner for applications with high loads requires a thorough understanding of the interaction between the liner system components and the type of applied load (normal and shear loads). When the interaction of the liner system components is considered in the liner system design, geomembrane liners have been shown to perform well under very high loading conditions. Insight will be given regarding testing procedures to evaluate the adequation of the lining system, foundation settlement and internal settlement.

Puncture protection of geomembranes and GCLs will then be discussed. The focus of the dedicated section will be on large scale tests, performed either in the laboratory or in the field. Attention will be paid to the case geomembrane exhibit wrinkles.

Then geosynthetics quality assurance will be discussed. Quality Assurance (QA) and Quality Control (QC) in the manufacturing, design, and installation of geosynthetic materials plays an important role in the overall performance of engineered structures which utilize geosynthetics. Information on damage to geomembranes will be presented in order to emphasize the need for QC. Then elements regarding construction quality assurance plan, geosynthetic installation and leak detection will be supplied.

While discussing the field performance of liners, certainly the most discussed issue in the past two decades was leakage through liners involving a geomembrane. A wide number of authors have published data on this topic synthesised among others by Rowe (1998), Bouazza *et al.* (2002), Rowe (2005) and Kavazanjian *et al.* (2006). Consequently the issue on hydraulic performance of composite liners will be briefly discussed herein. Rather, a synthesis of topics of growing interest like the behaviour of GCLs in covers, liner temperature and GCL shrinkage will be discussed in the section dedicated to geosynthetic barrier field performance data.

A synthesis including recent advances on the existing methods available for the prediction of liquid and gas flow rates through composite liners due to the existence of defects in the geomembrane will then be presented, including analytical solutions, empirical equations and numerical modelling in the section dedicated to modelling of flow through composite liners.

As any assessment of long-term environmental impacts from landfills requires contaminant transport modelling (Rowe 2005), it is not sufficient to only consider advective transport in liners. Diffusion and sorption through GCLs will thus be discussed in the section dedicated to migration and attenuation through GCLs. Furthermore, as for large hydraulic heads or increases in hydraulic conductivity of a GCL advection can become an important transport mechanism, hydraulic conductivity of GCLs to leachate and mining solutions will also be presented. Finally a brief overview of oxygen diffusion through GCLs will be given.

At last, a synthesis of existing approaches to equivalency for lining systems will be presented that are being used either to quantify flow through soil liners or composite liners. An illustration of results that can be obtained for the European landfill bottom liners will be given.

OVERVIEW OF THE USE OF GEOSYNTHETICS FOR GEOENVIRONMENTAL APPLICATIONS

Introduction

The disposal of waste materials is a matter of increasing public concern. The major component of the solid waste disposal system in almost every country is the landfill. During the last three decades, the practice of landfilling has evolved into fully engineered facilities subject to stringent regulations in order to protect the environment. To limit contaminant migration to levels that will result in negligible impact on environment several types of lining systems can be used for waste containment. Those lining systems are associated to drainage systems in order to reduce the hydraulic head of leachate on top of the liner. An emphasis will be given in the following of this section on the European (EU) regulation as regards to landfilling. Other regulations will also be presented. An update of the situation in developing countries will be given in regard to the use of geosynthetics in landfilling practices. The nature of the geomembrane and GCL that can be used for lining in landfills will also be discussed.

Mines, unlike other facilities such as dedicated landfill sites, usually have a much larger footprint, typically hundreds of hectares. Therefore environmental protection measures have to be practical, robust, sustainable and environmentally sound (Renken *et al.* 2005*a*). Consequently the utilization of geosynthetic materials for the design and construction of liner systems in mining applications has increased over the last two decades as the performance of geosynthetic materials in the mining environment becomes better understood. Geosynthetic materials are now commonly used in the design and construction of lining systems for heap leach pads (HLP), tailings storage facilities (TSFs), waste rock/overburden storage facilities, and lined ponds and channels. Geosynthetic liner systems have also been employed in process plant areas and within the mine site to provide environmental containment of industrial solutions, chemicals, and petroleum products. The design approach for geosynthetics in mining applications generally follows that used for municipal and hazardous landfills, with several distinct differences. Designs utilising geosynthetic materials in mining applications need to consider the presence of high stress levels (up to 3.5MPa), steep terrain construction techniques, dynamic loading from heavy haul trucks, and high hydraulic heads (up to 100 metres). Much has been written about the use of geosynthetics in landfill applications (Bouazza *et al.* 2002, Daniel & Koerner 1995, 2007, Kavazanjian *et al.* 2006, Koerner 2005, Koerner & Wilson-Fahmy 1994, Qian *et al.* 2001, Rowe 1998, 2005), but very little has been written about the performance of geosynthetics in mining applications. Discussions on the general use of geosynthetics in mining applications are presented in Breitenbach (1995), Breitenbach & Smith (2006), Kaczmarek *et al.* (1999), Lupo & Mandziak (2001), Lupo & Morrison (2005, 2007), Renken *et al.* (2005*a*,*b*, 2007), Thiel & Smith (2004), van Zyl *et al.* (1988) and van Zyl (1997). A description of the use of geosynthetics and design approaches used for mining applications are presented in the following sections. No deep investigation in regulation will be performed as the regulations governing the use of geosynthetics in mining applications vary considerably from country to country.

Liquid containment represents the historical application for geomembranes. As noticed by Lopes & Gomes-Coelho (2007) regarding leachate ponds the same care should be taken for lining and protection aspects of those ponds as for the landfill itself. The reason for that is that the leachate head or liquid effluent in ponds can be even greater than in landfills, leading to potential larger flow rates in ponds. Consequently lining and drainage of ponds will be discussed in the following, including lined ponds for mining applications.

Lining systems in landfills

Regulations for bottom liners

The design of a landfill liner system can be made either on a prescriptive basis or on a performance basis (Manassero *et al.* 1998). In the first case the requirements for a minimum lining system profile are specified through regulations, while in the second approach it must take into account numerous parameters such as transport parameters and service life of the mineral barriers, drainage layers, geosynthetics, and also the main characteristics of the waste in order to evaluate the leachate quality and production over the landfill activity and after closure (Manassero *et al.* 2000).

Both approaches present advantages and disadvantages. As regards prescriptive design, the main benefits can be summarised as follows (Estrin & Rowe 1995): (1) it minimises the effort of approval for the regulator by providing a process which basically allows a check list comparison to be made between the proposed design and the prescriptive design requirements; (2) it makes it easy for proponents since the regulator can easily determine if the proponent's application complies with the prescriptive specifications; and (3) it ensures a minimum environmental protection. However, it might be either insufficient to assure minimisation of environmental impacts at long term, or overly conservative (Rowe *et al.* 1995).

Concerning the performance design, the main benefits of this approach includes (Estrin & Rowe 1995): (1) allowing landfill designer to bring updated engineering concepts in designing to achieve these performance standards, which promotes both theoretical and practical research investigation and the application of evolving technology in the field; (2) need of a detailed evaluation of the proposed design prior to approval; and (3) the lining systems can be adapted to the specific characteristics of the waste and the considered site. The drawbacks of this approach can be listed as follows (Manassero *et al.* 1998): (1) the reliability of the design model must be validated; (2) the reliability of each input parameter for modelling the behaviour of landfill linings performance and the time and space variability of the contaminant targets must be checked; and (3) evaluation of some projects can be very difficult.

Most of the regulations all around the world follow the prescriptive design. Performance design has been used in some countries such as Canada and USA.

At present, landfills have composite liners that can be single or double (see Figure 1). The second one includes a secondary drainage (also termed in literature as leakage detection layer or leakage detection system) placed between the primary and secondary liners. It can be formed either by granular soils or by geonets. The aim of this layer is to control the leachate that goes through the primary liner system.

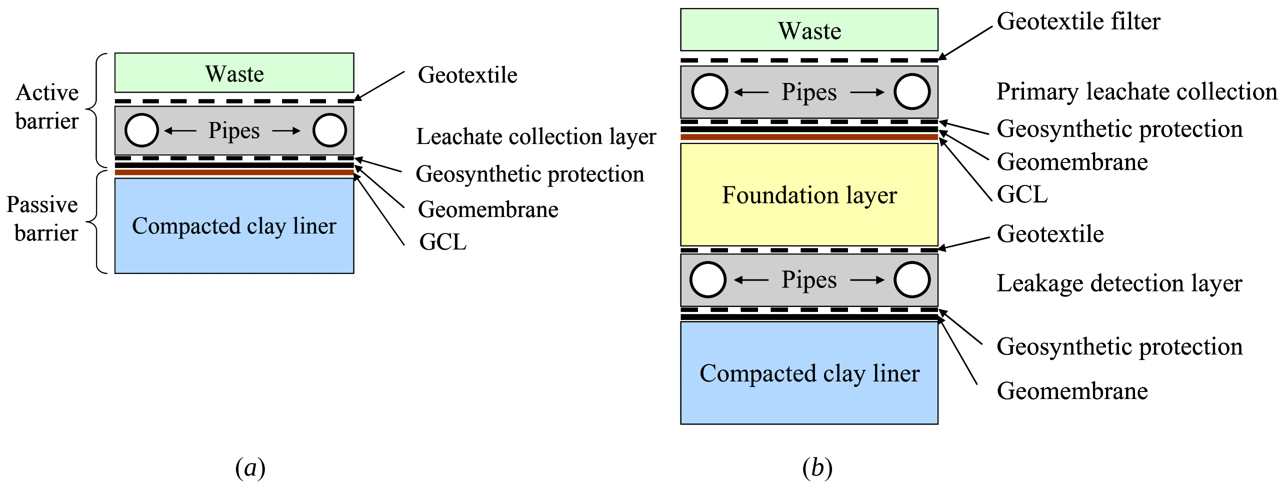


Figure 1. Examples of landfill composite liner systems: (*a*) single; and (*b*) double (adapted from Rowe (2005) and Barroso & Lopes (2007))

Double composite liners are mainly used in hazardous landfills at USA. According to Koerner (2000), 24% of municipal solid waste (MSW) landfills in USA and 14% of landfills worldwide have been designed with double lining systems.

The European legislation on waste disposal (OJEC 1999) states in annex 1 of directive 1999/31/CE that a landfill must be situated and designed so as to meet the necessary conditions for preventing pollution of the soil, groundwater or surface water and ensuring efficient collection of leachate. Protection for soil, groundwater and surface water is to be achieved by the combination of a geological barrier and a bottom liner during the operational/active phase, and by the combination of a geological barrier and a top liner during the passive phase/post-closure.

The landfill base and sides shall consist of a mineral layer which satisfies hydraulic conductivity, k, and thickness requirements with a combined effect in terms of protection of soil, groundwater and surface water at least equivalent to the ones resulting from the following requirements:

- Landfill for hazardous waste (HW): $k \leq 10^{-9}$m/s; thickness≥5m;
- Landfill for non-hazardous waste: $k \leq 10^{-9}$m/s; thickness≥1m; and
- Landfill for inert waste: $k \leq 10^{-7}$m/s; thickness≥1m.

Where the geological barrier does not naturally meet the above conditions it can be completed artificially and reinforced by other means giving equivalent protection. An artificially established geological barrier should be no less than 0.5m thick. The Directive does not, however, explicitly state that this thickness of 0.5m must necessarily have a hydraulic conductivity lower than 10^{-9} m/s, thus leaving scope for interpretation by member states (Guyonnet *et al.* 2007).

In France a clear distinction between what is referred to as the "active barrier" and the "passive barrier" which corresponds to the geological barrier in the sense of 1999/31/EC is made (Guyonnet *et al.* 2007). French legislation (Code permanent environment et nuisances 1997, JORF 2002*a*,*b*, JORF 2006) regarding landfilling of non-hazardous waste according to the European regulation prescribes a passive barrier that has functions of longer-term protection and attenuation of the residual pollutant flux emitted from the landfill. As shown in Table 1, various materials can be considered in order to address these functions, including GCLs.

Table 1. Distinction between "active" and "passive" bottom landfill barriers in French legislation (From Guyonnet *et al.* 2007)

	Function	**Materials**
Active barrier	Drainage	Granular, synthetic
	Active sealing	High density polyethylene geomembrane
Passive barrier	Passive sealing	Natural clays, treated soils, GCLs
	Attenuation	Natural geological environment

Based on a survey conducted by GeoSyntec Consultants (GeoSyntec 2004) and the review by Bouazza *et al.* (2002), Kavazanjian *et al.* (2006) indicate that by the turn of the 21st century the use of geomembrane-low permeability soil composite barrier layers in base liner systems for solid and hazardous waste landfills was a well established practice in industrialized countries. Furthermore, many countries allow for the use of GCLs as a replacement for a compacted low permeability soil layer in composite liner systems, either explicitly or implicitly through regulations allowing "engineered alternatives" with a trend towards greater acceptance of GCLs. In France, in case an equivalent geological barrier would be used, the thickness of 0.5m is interpreted as referring to a mineral layer of low (not exceeding 10^{-9} m/s) hydraulic conductivity. The same requirement applies in Spain (Martínez Santamaría 2007). Following, GCLs are used as a reinforcement of a low hydraulic conductivity layer but never as a substitution of this layer according to MEDD (2002) (see Figure 2). In Portugal also GCLs always are associated to a low hydraulic conductivity mineral layer (Lopes & Gomes Coelho 2007).

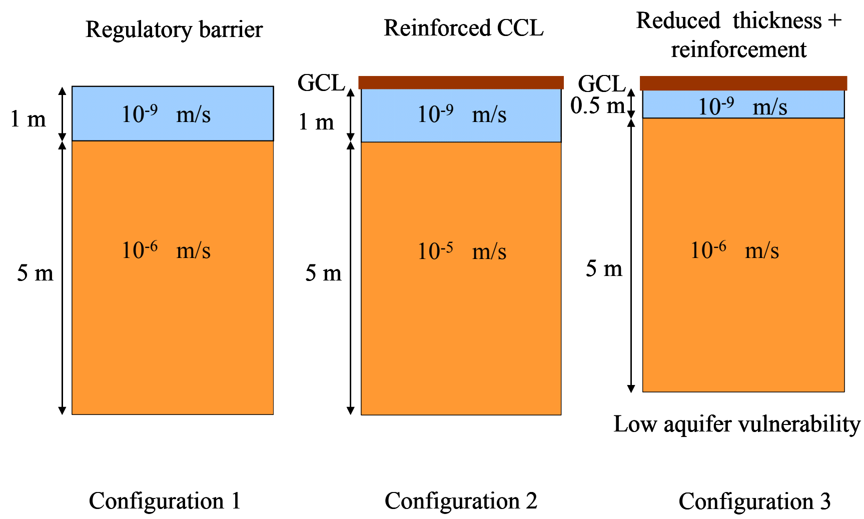


Figure 2. Schematic illustration of recommendations regarding GCL use in geological barriers in France (MEDD 2002)

According to Kavazanjian *et al.* (2006) one manifestation of this increase regulatory acceptance of the use of GCLs is revised guidance from the United States Environmental Protection Agency (US EPA) which now allows alternative base liners that employ a GCL to be used in landfills bioreactor landfills. Geomembrane-GCL composite liners are of quasi-systematic use in new landfills in Portugal (Lopes 2006).

However an attenuation layer (AL) may have to be included beneath the GCL to provide adequate diffusion resistance. This is the case in France where a 5m thick attenuation soil layer with a hydraulic conductivity lower than 10^{-6}m/s is required under the geological barrier.

Based on Bouazza *et al.* (2002) and GeoSyntec (2004), among the jurisdictions they studied, only South Africa and the province of British Columbia in Canada did not require composite liner systems with geomembranes for MSW landfills. The state of New South Wales in Australia only required a geomembrane to be used “in areas of significant threat to the environment”.

The regulation for barrier systems in Japan was dramatically revised in 1998 (Imaizumi *et al.* 2006). Four different alternatives are allowed:

- A 5m thick low permeability cohesive soil with a hydraulic conductivity lower than 10^{-7}m/s;
- A geomembrane underlain by an asphalt-concrete layer at least 50mm in thickness with a saturated hydraulic conductivity no greater than 10^{-9}m/s;
- A geomembrane underlain by a clay liner at least 0.5m thick with a hydraulic conductivity lower than 10^{-8}m/s; or
- Two geomembranes which sandwich a nonwoven fabric or other cushion material.

Despite strong regulations, the situation is not necessarily ideal. For example, in Greece, the investigation of many cases revealed that the most common method of handling solid wastes is uncontrolled disposal in different parts of the country, in contravention of both national and EU legislation. The environment is thus deteriorating steadily (Bosdogianni 2007). In Cyprus, in 2005, among the 7 official landfill sites, the requirements of the directive 99/31/EC were not fulfilled. Furthermore, 113 unofficial and uncontrolled disposal areas in operation were identified (Lolos *et al.* 2007).

Developing countries

According to Kavazanjian *et al.* (2006) perhaps the most significant advance in the use of geosynthetics for environmental protection since the turn of the century is the increased use of geomembrane and composite liners in base liner systems for landfills in developing countries. Indeed the use of geosynthetics barriers at landfills in developing countries is growing rapidly. Nevertheless the situation is variable from one part of the world to the other and from country to country.

In Eastern Europe there is a growing awareness of the need for improvement of waste disposal sites. According to Wilson *et al.* (2005) none of the dumpsites in the Kaliningrad Oblast of Russia can be considered as a sanitary landfill and they are not built according to Russian standards for landfills. The objective is to implement similar high standards of environmental protection to neighbouring countries from the EU. In most of the towns in Republic of Serbia, municipal deposits are unregulated and there are no necessary facilities and pollution prevention measures. Change at the national Waste management Serbia also includes a program of harmonization with the EU (Stevanovic-Carapina *et al.* 2005). The situation will also improve in Romania where out of the existing 265 landfills only 16 comply with the EU regulation, while the remaining 249 are to be closed. Another 30 landfills will be built in the next 10 years. The bottom liner has to include a geomembrane with a thickness greater than 1mm according to Feodorov (2005). This author also mentions the use of a 2.5mm thick high density polyethylene (HDPE) geomembrane for lining the bottom of a HW landfill in Romania in 2005. In Bosnia and Herzegovina, the only way of management of MSW and HW is disposal at small municipal landfills which do not meet the criteria necessary for sanitary landfills.

In central Asia (Kazakhstan, Kyrgyzstan, Uzbekistan, Tajikistan and Turkmenistan) the situation of waste management is characterized by poor landfill practices although most of the industrial and mining activities are located in the area of high population density where the main groundwater resources and the majority of landfill are located (Ritskowski *et al.* 2007).

In South America, many countries like Brazil, Argentine or Chile have installed proper sanitary landfills while in other countries like Peru, Colombia or Ecuador many disposal sites are called sanitary landfills even though they still burn the gas in funnels (Gamarra & Salhofer 2005). As an example von Buchwald (2005) describes the landfill site of “Las Iguanas” in Ecuador where no geomembrane was used as a 10m thick layer of clay was available even if the use of a geomembrane is recommended. Paredes (2008) mentions that, in Chile, the Sanitary Code which rules the regulation for sanitary landfills does not include geosynthetics. Nevertheless, a draft prepared by the Department of Health will strongly impact the situation, requiring the use of geomembranes and GCLs. in Brazil more than 70% of the municipalities still dispose of their waste in open dumps. In the Rio de Janeiro state only three municipalities have sanitary landfills licensed by the Environment State Agency (Ferreira *et al.* 2007). Where landfills are built, geosynthetics are not systematically used. Indeed, in the Pirai landfill the liner used was a 1m thick layer of compacted clay. Nevertheless as suggested by Vidal & Benvenuto (2008) the situation is evolving and HDPE geomembranes are more and more used in geoenvironmental applications.

In Africa Kehila *et al.* (2006) report on the Ouled Fayet landfill in Algeria where a 1.5mm thick HDPE geomembrane was used in conjunction with a compacted clay liner (CCL). In Jordan, the Ghabawi landfill is the first sanitary landfill in the Middle East. The liner includes a 2mm thick HDPE geomembrane on top of a GCL (Tadros 2007).

Until recently there was no facility in Mozambique for the safe disposal of HW (Legg 2007). The new Mavoco HW facility liner includes a 2mm thick HDPE geomembrane primary liner, a GCL, a 1mm thick HDPE secondary liner and a 0.3m compacted clayey sand liner. The GCL was prehydrated to about 100 % moisture content before placement of the primary geomembrane liner as it would otherwise have no access to moisture for hydration.

Part of the evolution is linked with international financial support and/or technical support from industrialized countries. Kavazanjian *et al.* (2006) report on four landfills with composite liner systems that were constructed in Chad and Cameroon as part of a World Bank financed pipeline construction project from 2000 to 2002 and three new sanitary landfills that were constructed in another World Bank project from 2000 to 2005 in Ghana among which only one incorporated a geomembrane. Serdarevic *et al.* (2007) mention an implementation of the World Bank project in this country as well as in Bosnia and Herzegovina. International technical assistance is also used in Kaliningrad Oblast to improve MSW management funded by the European Commission (Wilson *et al.* 2005).

Hüttner & Zurita (2005) indicate that in Chile since 2005 HW issues are regulated by a new HW management act and that the German Agency for Technical cooperation conducts technical assistance to improve the management of HW.

Kavazanjian *et al.* (2006) mention that the degree of sophistication of geosynthetic barrier systems in Asian countries is often a reflection, not only on local physical conditions but also of market factors. In many cases the more sophisticated systems including geomembrane/GCL composite barriers in single, and for HW, in double liner configurations, have been promulgated by private sector project proponents and exceed regulatory minimum standards.

Sitting constraints associated with new facilities in densely populated countries such as the Philippines and Vietnam mean that sites are often chosen based upon non-technical criteria such as land availability and ease of access. This in turn means that the use of geosynthetic barrier components will be increasingly required to provide environmental protection at sites that are inherently less suited technically for waste disposal that might otherwise have been the case (Kavazanjian *et al.* 2006). This situation regarding Asia is also true in other parts of the world. Indeed, Manga (2005) describes the case of Cameroon where many urban waste disposal dumps are poorly managed and sited and present a risk to human health and the environment. The same phenomenon occurs in Bosnia and Herzegovina (Serdarevic *et al.* 2007). Nevertheless there is a growing awareness in various developing countries of the need to adequately choose landfill sites with naturally occurring barriers, in areas where groundwater pollution might not be expected in order to maintain and improve living standards (Abu Qdais 2007, Ritskowski *et al.* 2007, Stokic & Popadic 2007, Tadros 2007).

Since many landfills in Europe have neither an acceptable geological barrier nor a base liner a capping will be of importance for the required protection against the potential hazard of landfills to ground water, soil and air (Simon & Müller 2004). Aspects regarding capping of landfills will be further discussed in this section.

Nature of the geomembrane and GCL for landfill bottom lining systems

A special session on geomembranes in landfills took place at the 7th International Conference on Geosynthetics followed by a long discussion that lead to a summary dealing in particular with the nature of the geomembrane for bottom liners (Giroud & Touze-Foltz 2003). The main points of the discussion were the following: (1) in Germany, only HDPE geomembranes are used in landfills, because only HDPE geomembranes have been certified, even though the certification system is, in principle, opened to other types of geomembranes; this is based on more than 20 years' experience with polyethylene in civil engineering applications and comparative testing of different other geomembrane materials (Zanzinger & Gartung 2002); (2) in most other countries, even though the certification system is less strict than in Germany, all or almost all of the geomembranes used in landfill bottom liner systems are HDPE geomembranes; (3) engineers and researchers would like to see a greater variety of geomembranes used in landfill bottom liner systems, but they recognize that this will be possible only when it is proven and accepted that geomembranes other than HDPE geomembranes have the required properties to be successfully used in landfill bottom liner systems; and (4) research efforts aiming at developing new types of geomembranes should be encouraged.

Imaizumi *et al.* (2006) report on the types and total amounts of geomembranes used in waste landfills in Japan in 1998 based on data from the Japanese Association of Barrier Sheet. Those data show that HDPE geomembranes were the most popular representing more than half of the geomembranes used in Japanese landfills. However, the use of geomembranes with medium rigidity such as thermo plastic olefin polypropylene (TPO-PP), thermo plastic olefin polyethylene (TPO-PE) and linear low density polyethylene (LLDPE) has been increased recently. The use of polyvinyl chloride (PVC) geomembranes is also mentioned for more recent applications in Japanese landfills (Kotake *et al.* 2006). One of the reasons for that leads in the fact that most landfills in Japan are constructed to bury the valley with surrounding embankments in mountainous areas, with typical disposal areas between 20,000 and 50,000m² so that the geomembrane is required to keep a good workability. Potié *et al.* (1997*a*) report on the use of a polypropylene (PP) geomembrane for the bottom lining of three landfill cells in France. Lopes & Gomes-Coelho (2007) indicate that 2mm thick HDPE geomembranes are systematically used in Portugal.

The same question could be raised regarding the nature of the bentonite contained in GCLs. If this may not be a crucial issue in Northern America where mostly natural sodium bentonite from Wyoming is available, it can be an issue in other parts of the world like Europe, Asia or Australia where various types of natural sodium bentonite, sodium activated calcium bentonites and natural calcium bentonites can be encountered (Chung 2004).

Recommendations expressed in the Guide of recommendations, for third party experts, relative to the assessment of equivalence (MEDD 2002) state that natural sodium bentonite GCLs are to be preferred to sodium activated calcium bentonite GCLs for bottom lining systems in landfills. This principle was reinforced by Guyonnet *et al.* (2005) who showed that a GCL containing natural sodium bentonite performed systematically better, in identical experimental conditions, than a GCL containing sodium activated calcium bentonite. It is believed that the main reason for this was the higher calcium carbonate content of the activated bentonite GCL (10 % in weight) in relation with the activation process (mixture with $NaHCO_3$ volcanic ash). Calcium carbonate, associated with the bentonite clay, provides a pool of divalent cations that are ready to exchange with the clay upon dissolution. Such results have prompted landfill operators in France to prefer natural Na-bentonite GCLs rather than Na-activated Ca-bentonite GCLs in agreement with recommendations expressed in the Guide of recommendations, for third party experts, relative to the assessment of equivalence (MEDD 2002). However, the distinction between natural and activated bentonite is only part of the issue and other criteria are needed in order to check whether a given product is suitable for a given application (Guyonnet *et al.* 2008).

This statement is reinforced by data presented by Lee & Shackelford (2005) who investigated the behaviour of two needle punched GCLs containing different sodium bentonites, one having a higher montmorillonite content, plasticity index and cation exchange capacity (CEC) than the other. While permeated with $CaCl_2$ solutions the hydraulic conductivity of both GCLs increased with increasing concentration and the hydraulic conductivity of the bentonite having the higher montmorillonite content was the largest. Thus the GCL with the higher bentonite quality was the more subject to chemical attack upon permeation by the $CaCl_2$ solution.

Gleason *et al.* (1997) compared the behaviour of thin layers of calcium and sodium bentonites permeated by tap water and 0.25M $CaCl_2$ solution. All thin layers permeated with tap water performed well. The sodium bentonite was approximately 10 times less permeable to tap water than the calcium bentonite. When permeated with the $CaCl_2$ solution, the calcium bentonite maintained a significantly lower hydraulic conductivity than granular sodium bentonite but a higher hydraulic conductivity than powdered sodium bentonite leading to the conclusion that the grain size of the sodium bentonite was found to be important.

Chung (2004) states that cation in exchange sites is the main factor affecting bentonite quality in GCLs and that the geo-chemical environment in which GCLs are used should be considered in order to ensure that GCLs perform as an effective barrier.

Additional investigations are required on this topic as regards the impact of granulometry of the bentonite, nature of the bentonite, activation and assembly mode to guarantee the adequation of a given GCL for a given application.

Regulations for top covers

According to the European legislation on waste disposal (OJEC 1999), if the competent authority after a consideration of the potential hazards to the environment finds that the prevention of leachate formation is necessary a surface sealing must be prescribed. Recommendations for the surface sealing include: (1) for non-HW a gas drainage layer, an impermeable mineral layer, a drainage layer with a thickness greater than 0.5m and a top soil cover with a thickness greater than 1m; and (2) for HW an artificial sealing liner, an impermeable mineral layer, a drainage layer with a thickness greater than 0.5m and a top soil cover with a thickness greater than 1m. Figure 3 presents an example of a typical top cover system for non-HW landfills.

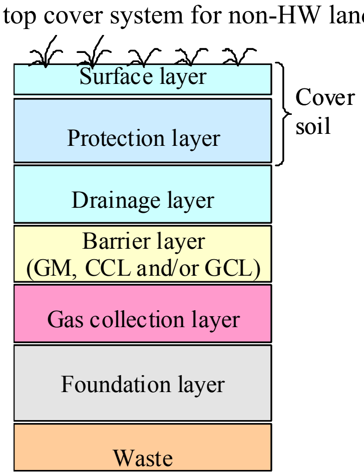


Figure 3. Example of a conventional top cover (adapted from Hauser & Gimon 2004)

In France, the design of the cover systems is now based on a performance standard. Indeed, French legislation prescribes in the case of MSW landfills a final cover aiming at limiting the infiltration of rainwater in waste and the leakage of leachate outside the landfill. This general objective of reduction of infiltration leaves the door opened to various cover alternatives, depending on the potential leachate recirculation. The Ademe (2001) guide gives among others: (1) elements to help choosing the type of cover based on performance objectives in terms of liquid and gas flow into and from the landfill, waste harmfulness reduction and durability greater than waste harmfulness period; and (2) recommendations for the choice of cover constitutive elements, including geosynthetics, to reach the performance objectives. Semi-permeable covers, impermeable covers with and without leachate recirculation are proposed. In the case of impermeable covers, both geomembranes and GCLs can be used. Figure 4 shows the installation geomembrane and drainage layers in top covers.

This regulation is consistent with the general change in philosophy from use of gas and liquid-tight cover systems to cover systems that allow the waste to "breathe". This change in philosophy is probably one of the major reasons that geosynthetic barriers are not used as widely in covers as they are in bottom liner systems for MSW landfills (Kavazanjian *et al.* 2006). Regarding HW landfills according to Kavazanjian *et al.* (2006) geosynthetics barriers are still the preferred barrier, as well as for capping of uncontrolled HW dumps and other sites where control of infiltration is paramount.

Innovative geosynthetics products appear to answer the demand on alternative cover systems. For example, an alternative to CCLs and GCLs for the construction of semi-permeable covers was proposed by Fourmont & Arab (2005). It consists of a drainage geocomposite made of mini-pipes comprised between a geotextile and a HDPE geomembrane. The infiltration through the cover occurs at the transversal and longitudinal overlaps between the rolls. Other solutions with other kinds of drainage geocomposites including a HDPE draining core and a geotextile used in the same way, i.e., with overlaps allowing water flow, can be encountered (Meydiot & Lambert 2000, Faure & Meydiot 2002).

In the case of leachate recirculation it seems to be common practice to include a geomembrane in the final cover. Bureau *et al.* (2005) clearly state that the bioreactor cell is characterized by a watertight and gastight top cover which means a HDPE geomembrane liner to minimize odours/gas emissions, to provide better gas quality and to permit improved control of the hydraulic mass balance. Thiel (2005) presents a generic cover design in case of leachate recirculation including a geomembrane. Barina (2005) presents the examples of two landfill sites in France and Italy where leachate recirculation is performed and where a geomembrane-CCL composite liner will be included in the final cover. Bourrassin *et al.* (1999) describe the top cover of the Vert-le-Grand landfill where one of the first leachate recirculation program was undertaken in France. The landfill was covered with a PVC geomembrane.

(*a*) (*b*)

Figure 4. Examples of top covers: (*a*) placement of the geomembrane; and (*b*) placement of the drainage layer over the geomembrane

Innovation also appears in the field of HDPE geomembranes. Indeed, a landfill surface sealing system made of a composite element of geomembrane and flexible thin film solar cells is at present under testing. The main advantage of this multifunctional system is emission minimization and convection prevention at the same time with photovoltaic plant. The short and long-term integrity of the system in regard to technical and operational demands and requirements is currently investigated (Kuehle-Wedemeier 2007).

Regarding the nature of the geomembrane used in cover liners, in some countries, including the United States, a variety of geomembranes are used in landfill capping systems, but a number of regulators in the United States do not permit the use of geomembranes other than HDPE geomembranes in landfill capping systems (Giroud & Touze-Foltz 2003). In France various examples of the use of bituminous geomembranes can be found (Potié *et al.* 1997*b*) included for low and moderate activity radioactive waste (Ossena *et al.* 1997, Marchiol *et al.* 2006). Potié *et al.* (1997*a*) also present the case of a realization of a landfill cover with a PP geomembrane. The geomembrane choice is mainly based on mechanical parameters i.e. potential for deformation and friction angle. Resistance to atmospheric agents of the waste nature can sometimes be taken into account (Silvestre *et al.* 2003). In Germany, even though the geomembranes of covers are not exposed to a corrosive chemical environment, the same types of geomembranes i.e. 2.5 mm thick HDPE, are used for caps and for basal liners (Zanzinger & Gartung 2002). The installation of thinner geomembranes or of geomembranes made with softer polymers, such as very low density polyethylene (VLDPE), would be more favourable with respect to the anticipated deformations of the landfill surface (Zanzinger & Gartung 2002).

A number of studies regarding performance of GCLs in covers deal with the application of the use of those materials. They will be further discussed in the section dedicated to geosynthetic barrier field performance data of this Keynote Lecture.

Fluid collection systems in landfills

Bottom liners

The European legislation on waste disposal (OJEC 1999) states in its annex 1 that in addition to the geological barrier described earlier a leachate collection and sealing system must be added in accordance with the following principles so as to ensure that leachate accumulation at the base of the landfill is kept to a minimum. For non-HW and HW landfills, an artificial sealing system which is interpreted as a geomembrane by most EU countries according to Kavazanjian *et al.* (2006) and a drainage layer with a minimum thickness equal to 0.5m are required. No hydraulic conductivity is given for this drainage layer. Figure 5 presents an example of a drainage layer consisting of 0.2m of sand and 0.3m of gravel.

In France the active barrier has functions of leachate drainage and short to intermediate-term protection of the subsurface and guarantees that leachate is collected for treatment during the active landfill period where leachate concentrations are high. Table 1 shows the various materials that can be considered in order to address these functions, including HDPE geomembranes. The French regulation opens the door to an equivalent to geomembranes even if no equivalent is known at present. The drainage layer with a minimal thickness equal to 0.5m can be replaced by an equivalent material, opening the door to the use of geocomposite drainage materials. The lack of hydraulic conductivity value of this drainage layer generates technical difficulties to justify equivalence (Jarousseau 2006). As the leachate table must not be in the waste, a minimum 0.3m thick drainage layer is required.

In Australia, the regulation requires a drainage layer to comprise gravel, or a combination of gravel and geonet. According to Cowland *et al.* (2006) an acceptable design could comprise a gravel layer 0.3m thick with a hydraulic conductivity, k, greater than 10^{-3}m/s. Jarousseau (2006) proposed such a solution for a French landfill.

In Florida, the leachate collection system (LCS) for the composite liner, and the primary LCS for the double liner have to be constructed with a granular drainage layer at least 0.6m in thickness with the lower 0.3m portion having a minimum hydraulic conductivity of 10^{-5}m/s. Geonets that are bounded to geotextiles are often used as part of these LCSs between the geomembrane and the granular layer to ensure the design heads can be achieved. In the case of the secondary leachate collection system (SLCS) for the double liner the rule requires it has a minimum hydraulic conductivity of 0.1m/s. Geonets are often used in this application (Tedder 2005).

Figure 5. Placement of a drainage layer in a bottom landfill liner containing 0.2m of sand and 0.3m of gravel

De Vita & Arnold (2002) present the use of the association of a 7.6mm tri-planar geonet-geotextile geocomposite covered by a 0.6m thick sand layer used as a drainage layer in two bioreactor cells providing, according to the authors, increased hydraulic performance, more suitable for the increased leachate recirculation associated with bioreactor operation activities.

Zhao & Giroud (2006) proposed design equations for the determination of the required hydraulic transmissivity and the maximum allowable spacing for geocomposite strip drains embedded within a continuous layer of sand that can be an alternative to conventional LCSs where the entire drained surface is covered with a unique material.

A variety of geocomposites can be encountered on landfill slopes for drainage purposes. Arab *et al.* (2002) present an application of a drainage geocomposite made of geotextiles and mini-pipes.

Top covers

The European legislation on waste disposal (OJEC 1999) regarding landfill covers clearly states the existence of a liquid and a gas drainage layer.

French legislation prescribes a final cover including a gas drainage layer in case of degradable waste. This layer would basically be a granular drainage layer that can be replaced by a geosynthetic with equivalent features. In France they usually consist of geospacers bounded by two geotextiles (Ademe 2001). This is consistent with usual practice in Europe (Zanzinger & Gartung 2002). Geonets, which are common in the USA, are used less frequently in Europe. A mention of their use in Portugal is made in Lopes & Gomes-Coelho (2007).

Rainwater drainage level is a key element in flow regulation. A small amount of precipitation runs off from the landfill surface directly. Most of it evaporates or is stored in the top soil layer where it is available for plant growth. The remainder water percolates and reaches the liner. In order to avoid the build-up of water pressure acting upon the seal and which can lead to instability, a dewatering layer is installed in the capping system upon the sealing layer (Zanzinger & Gartung 2002). Granular drainage materials can again be replaced by geosynthetics. In Florida, at this time the rule does not specify the hydraulic conductivity required for the drainage soils. Most designers, however, use geocomposites to ensure the adequate drainage of rainwater off the final cover (Tedder 2005). Drainage geocomposites have some advantages over granular drains. The mass of construction material to hold is smaller. Their thickness is small and waste storage volume can be saved. Also their placement is fast and easy. The quality of industrially manufactured geocomposites is more uniform than that of natural soils used for drainage layers. In some parts of the world, it may be difficult to find suitable granular drainage materials. For these reasons, drainage geocomposites are being used increasingly in landfill capping systems. Bourrassin *et al.* (1999) mention the use of a drainage geocomposite on the Vert-le-Grand landfill on top of the PVC geomembrane.

Von Maubeuge *et al.* (1999) insist on the fact that, at the time of conception of the drainage system using geosynthetics, long term performance in terms of filtration, drainage and mechanical stability – on slope for example – has to be evaluated, if one aims at replacing a classical mineral drain. Filtration issues were addressed in detail in Palmeira & Fannin (2002) and in Heibaum *et al.* (2006).

Lining systems in mining applications

Heap Leach Pads

Heap leach facilities (HLFs) are used as part of the mining process to extract metal from ore, and have been used for the recovery of gold, copper, silver, uranium, nickel, and other metals and non-metals. HLFs are engineered facilities which generally consist of a heap leach pad (HLP), solution collection/conveyance structures, process solution ponds, and process facilities. The HLP is a lined facility onto which ore is placed, either using truck haulage or conveyor placement. Once the ore is placed on the HLP, a leaching solution is applied at a controlled rate to percolate through the ore, thereby dissolving the contained minerals. The leaching solution varies depending on the type of ore to be processed, and may consist of a strong acid (e.g. sulphuric acid) or, in the case of gold and silver heap leaching, a dilute cyanide solution. A general schematic of a HLP operation is presented in Figure 6. The leach solution is then recovered and routed to process solution ponds, where it is sent for metal recovery.

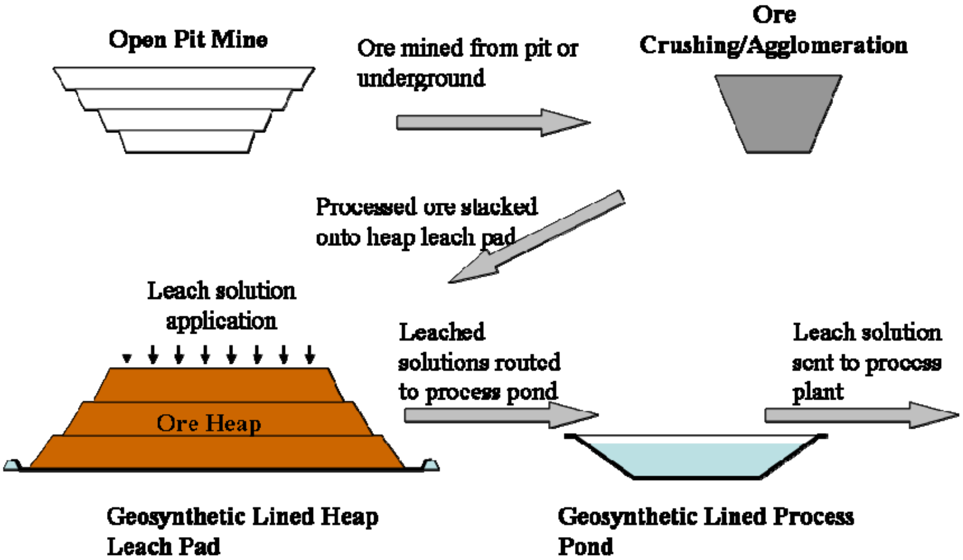


Figure 6. HLP Schematic

The HLP is commonly lined using natural or geosynthetic materials to promote solution recovery and to provide environmental containment of the leach solutions and ore. Liner systems for HLPs either consist of a single composite, or double composite systems with a leakage collection layer, as shown in Figure 7. Single composite liner systems generally consist of a geomembrane liner placed over a compacted liner bedding soil. The geomembrane liner materials typically consist of HDPE, LLDPE, PVC, and PP, although the primary materials used in modern HLFs are

HDPE and LLDPE. The extensive use of HDPE and LLDPE geosynthetics in heap leach operations has demonstrated that they are suitable for containment of corrosive acid rock drainage and metal leaching products, for periods of at least 20 years. Long term performance data (50 to 100 years) does not exist (Renken *et al.* 2007). This type of configuration is commonly used in areas that experience low hydraulic head (typically less than 1m). Double composite liner system consists of two geomembrane liners separated by a leakage collection layer. The lower secondary geomembrane is placed over a compacted liner bedding soil. A double composite liner system is used where high hydraulic heads may occur. Figure 8 presents a photograph of a leach pad under construction showing the components of a single composite liner system. It is important to note that GCLs may be considered as a substitute for the liner bedding soil; however care must be taken to address bentonite migration under load and internal shear strength (Lupo & Morrison 2007). Single geomembrane liners are still the most common liners for copper leach pads with composite liners more common for gold and silver leach pads (Breitenbach & Smith 2006).

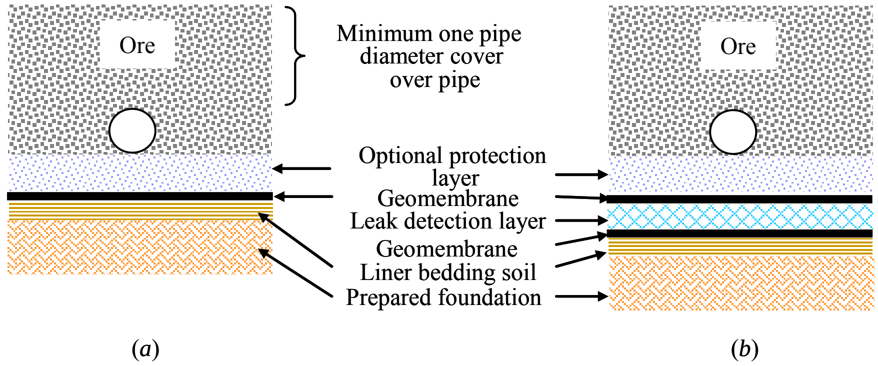


Figure 7. (*a*) Single Composite Liner System and (*b*) double Composite Liner System for HLPs

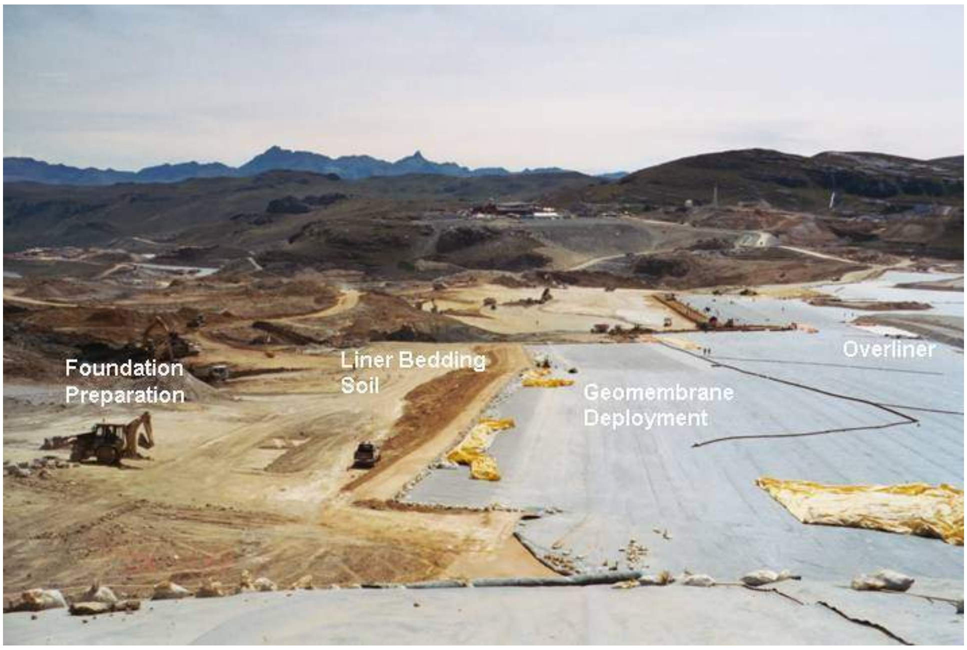


Figure 8. HLP Construction

In addition, it is interesting to note that geotextiles are not commonly used as a protection layer over the geomembrane. This is because the low friction at the geomembrane-geotextile interface has resulted in a number of failures in mining applications (Breitenbach 1995).

Recently, the size of HLPs has increased significantly with improvements in ore processing techniques, mining equipment, and the rising demand for metals and non-metals. Some leach pads now cover an area greater than 10 square kilometres with ore heights now exceeding 180m over the lined surface, with a resulting normal stress of over 4MPa. The potential resulting deformation of plastic pipes under high loads and design methods to address deformation and the geomembrane liner system design for high loads will be addressed in further sections of this Keynote Lecture.

Observations from the construction and performance of large and small HLPs have shown that it is important that liner system design considers the compatibility between the various components (e.g. foundation, liner bedding, geomembrane liner, and overliner). The overall performance of the liner system is dependent on the interaction between the various components. Therefore the design approach and testing requirements need to focus on inter-relationship of stress and strain between the liner system components, such as foundation settlements, interface shear strength, gradations of the liner bedding soil and overliner material, and geomembrane strength. The point will be further emphasized in the section of this Keynote Lecture dedicated to geomembrane liner design methods for high loads.

Tailings Storage Facilities

Tailings storage facilities (TSFs) are engineered structures that are constructed to impound slurried, thickened, paste, or dry-stacked waste materials (tailings) resulting from mineral processing activities. Ore that is mined from the ground is crushed to a prescribed size and then sent to a mill, where the rock is ground into a fine powder. The ground ore is processed as slurry using various methods to extract metals. After processing, the slurried tailings consist of fine particles suspended in process solution that may be processed further to form thickened, paste, or dry tailings. A general schematic of a TSF operation is presented in Figure 9. Depending on the geochemistry of the tailings and process solution, the tailings may be stored within a geosynthetic-lined TSF. This is often done if the tailings and/or process solution contain constituents that may have a negative impact on the environment.

Geosynthetic liner systems for TSFs are commonly designed and constructed as single composite liner systems. Double composite liner systems are less common due to the hydraulic properties of the tailings. Indeed it is important to note that a single composite liner system can be effectively used at TSFs even though the hydraulic head within the facility can exceed 100m. This is because the tailings often form a low permeability layer at the base of the TSF above the liner system. Indeed the hydraulic conductivity of consolidated whole (uncycloned) tailings can range from 10^{-6} to less than 10^{-10} m/s (Vick 1983), thereby forming a layer that can minimize seepage from the facility. The single composite liner system consists of a geomembrane liner placed over a compacted liner bedding soil. A GCL may also be considered as a liner bedding layer for TSFs. The geomembrane liner materials typically consist of HDPE, LLDPE, PVC, or Reinforced Polyethylene (RPE), although the most common materials are HDPE and LLDPE. A continuous or intermittent drainage layer may also be placed over the liner to enhance tailings consolidation or provide internal drainage for the TSF. As with HLPs, liner system designs for TSFs need to consider compatibility of the various components of the system. Important factors that influence the design of the liner system include, foundation settlements, expansion considerations for the facility (e.g. upstream, centreline, downstream construction), and environmental issues.

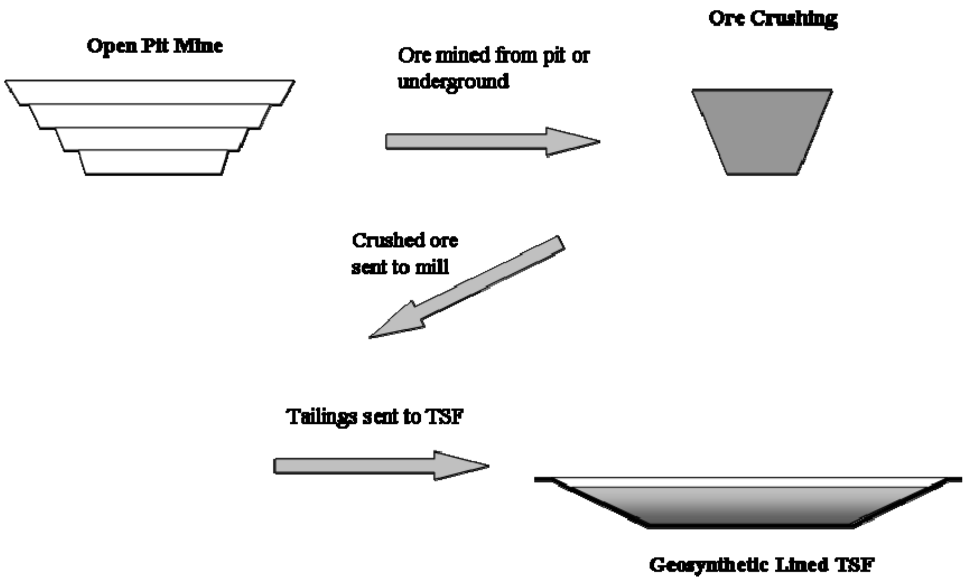


Figure 9. Mill/Tailings schematic

Waste Rock/Overburden Storage Facilities

During the mining process, waste rock or overburden is removed to provide access to the ore. The overburden materials are often placed in a dedicated facility that is engineered to provide stability and drainage. For the most part, overburden storage facilities are not typically lined with geosynthetic materials. This is because natural materials are often available for design and construction of liner systems. However, there are cases where geosynthetic materials have been incorporated into overburden storage facility liner systems.

Liner systems for overburden storage facilities are used when there is a concern for the development of poor quality seepage from the overburden. Poor quality seepage may develop as a result of percolation of meteoric water through the material and reacting with minerals in the overburden. The liner systems are most often designed as a single composite liner using the same methods as that for HLPs. The single composite liner system consists of a geomembrane liner (HDPE, LLDPE, PVC, RPE) placed over a compacted liner bedding soil (see Figure 7).

A key design consideration for design of liner systems for overburden storage facilities is the thickness of the overliner or protection layer to protect the geomembrane from damage by repeated loadings from haul truck traffic. This is particularly important when considering haul trucks that now have capacities of 300 tonnes or more, with operating weights of over 450 tonnes. Designers will often size the overliner material thickness to reduce the normal stress on the geomembrane from the haul truck traffic to between 36 and 70kPa. This approach has been tested at several mines (Crouse *et al.* 1999) and appears to provide adequate protection for the geomembrane liner.

Fluid collection systems in mining applications

Heap Leach Facilities

An important consideration in the design of an HLP is the leach solution collection system. The solution collection system not only needs to be designed to collect leach solutions, but it must also control the hydraulic head on the underlying liner system. Controlling the hydraulic head on the liner system will help reduce instability of the ore heap and the leach pad. Relatively small changes in the hydraulic head or drainage conditions within an HLP can result in ore heap failure. Controlling the hydraulic head can also reduce the driving head seepage through defects within the liner system, as will be further discussed in the section of this Keynote Lecture dedicated to modelling of flow through composite liners. The most common HLP solution collection systems consist of plastic pipes embedded in a drainage layer as shown on Figures 7 and 10.

Depending on the design and conditions within the HLP, the solution collection systems may also include geotextile, corrugated and perforated (single-walled and double-walled) pipes, perforated or non-perforated HDPE pipes, and geodrains.

The design of an HLP solution collection system (capacity, pipe spacing, and construction materials) must consider the following:

- Leach solution application rates and leaching cycle (primary, secondary, and tertiary leaching);
- Grading/design and type of the HLP;
- The prolonged exposure to strong acid and base leach solutions (>20 years);
- Crushing of the pipe associated with high ore loads (ore heights exceeding 180m with ore loads up to 4MPa;
- Pipe stress cracking;
- HLP foundation settlement;
- Degradation of the drainage material and fines generation; and
- Fouling of pipes and geotextiles.

Figure 10. HLP corrugated polyethylene solution collection pipes

Tailings Storage Facilities

Solution collection systems for TSFs are used to help collect process solutions for re-use in mineral processing and/or to promote consolidation of the tailings. Slurried tailings generally consist of fine particles suspended in process water. The percent solids of the slurry may range between 5 to 65 percent by weight, depending on the processing method. After the slurried tailings are deposited within the TSF, the tailings settle and consolidate, releasing process water that can be reclaimed and re-used for mineral processing. For the most part, the process water forms a supernatant pool over the consolidating tailings, where it can be reclaimed using a barge-mounted pump or a gravity decant system.

Within the TSF, a drainage system is often used on the floor of the impoundment to drain process water from the tailings and enhance the consolidation process. The drainage system may consist of a drainage layer placed over the entire floor of the impoundment, or a network of finger drains spaced throughout the impoundment. Geosynthetic materials are often incorporated into the design of the drainage system, and include plastic pipes, geotextiles, and geodrains. A schematic of typical TSF drainage systems is presented in Figure 11.

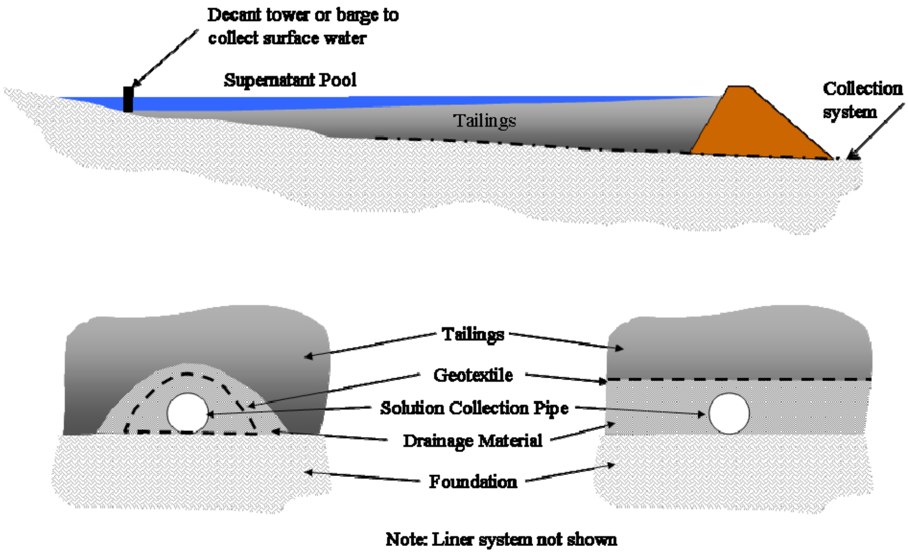


Figure 11. TSF Schematic

The performance of a drainage system is important to the design of the TSF, as it has the following benefits:

- Reducing the tailings consolidation time by providing another route for pore water drainage;
- Increasing the in-place tailings density which increases the capacity of the TSF;
- Enhancing process solution recovery for re-use in mineral processing; and
- Reducing the hydraulic head on the TSF impoundment floor.

The designs of solution collection systems for TSFs can vary significantly, depending on the desired goals and performance for the operation and characteristics of the tailings. For example, a large collection network covering the entire TSF impoundment may be used for operations where in-place tailings density and TSF capacity are critical. At other mine operations, only partial coverage of the TSF with a collection system is required to meet the needs of the operation. As illustrated in Figure 12, the collection system often consists of a series of plastic pipes embedded within a drainage material (gravel, sand, etc.). A geotextile may also be used in the design to minimize fines migration into the collection pipes. The collection pipes are routed to main headers that are connected to either a riser pipe or an external collection pond. Key design issues for TSF solution collection systems include:

- Collection spacing and geometry to achieve the desired goal or objectives for the mine operation; the drain spacing is often determined using analytical or numerical methods to predict the influence of the collection drains;
- Collection pipe sizing to accommodate the anticipated flows;
- Filter compatibility between the tailings and the drainage material/geotextile; filter compatibility on the drainage material may be assessed using the methods presented in International Commission on Large Dams (ICOLD 1994), Lafleur *et al.* (1989), and Sherard *et al.* (1984). The filter compatibility of the tailings to the geotextile may be assessed using the approach presented in Luettich *et al.* (1992); it should be noted that the particle size of the tailings will vary depending on the mineral processing method; finely ground tailings can have particle sizes with 60 to 80 percent of the material finer than 0.074mm, which is smaller than the apparent opening size of geotextiles; under these conditions, the solution collection system is over-designed to account for some loss of performance due to fines plugging;
- Stress on the solution collection piping imposed by the weight of the tailings; the stress on the collection system can be significant with some TSFs containing up to 130m of tailings.

Waste Rock/Overburden Storage Facilities

Waste rock or overburden storage facilities are used for temporary or permanent placement of non-ore bearing materials. In some cases, these materials are mineralized and have the potential to generate poor quality seepage that could impact the environment. Solution collection systems are often included in the design of overburden storage facilities to collect and route seepage to ponds or sumps. The seepage collected in the ponds or sumps is analyzed to determine if it can be discharged or if treatment may be required prior to discharge.

Solution collection systems for overburden storage facilities usually consist of collection drains excavated into the subgrade of the facility. The drains are lined with geotextile to prevent fines migration from the overburden and the foundation into the collection pipe. A drainage material is placed around the collection pipe and within the geotextile, as shown in Figure 12.

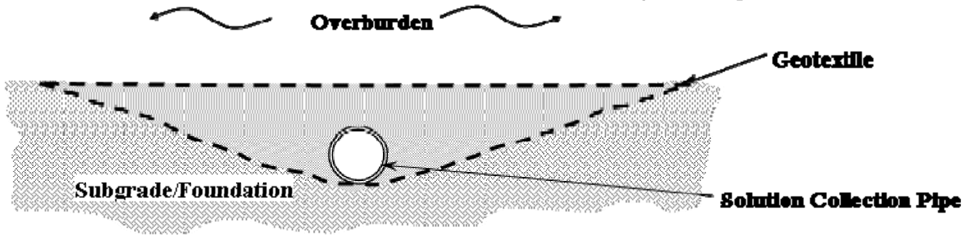


Figure 12. Typical Overburden Facility Seepage Collection Drain

Critical design issues with overburden storage facility solution collection systems include:

- Designing the collection pipe, drainage material, and geotextile to function under the very high stresses that can develop in the facility as overburden storage facilities can often exceed 150m in height, resulting in high normal stresses for the collection systems;
- The gradation of the drainage material selection to be compatible with the solution collection pipe and the type and weight of geotextile; and
- The hydraulic performance of the collection system evaluation considering the climate conditions at the mine site and the hydraulic characteristics of the overburden.

Lining systems in liquid waste ponds and channels

A number of geomembranes and/or GCLs may be used in ponds. An example of a pond lined thanks to a geomembrane is shown on Figure 13. It is also possible to install double-lining systems using two geomembranes with a drainage layer in-between them. This solution is still rare in liquid containment and is only applied where the risk must be reduced considerably. Only one reference to double lining systems was found by the authors in the literature for a containment pond (Girollet 1983).

One exception is for lined ponds on mine sites. On mine sites, lined ponds are used to store solutions associated with ore processing and metal extraction, treated or fresh water, poor quality seepage, and sludge generated from water treatment facilities. Lined channels are used to convey process solutions between facilities and/or to provide double-containment for process solution pipelines. Lined ponds and channels are often included in the design of HLFs, TSFs, and overburden storage facilities. The liner systems for ponds and channels in mining applications typically consists of a double composite liner system including an upper primary geomembrane liner separated by a leak collection/drainage layer from lower secondary geomembrane liner placed over a compacted liner bedding soil. GCLs have been used in the design and construction of solution ponds for mining applications as a substitute for the liner bedding layer, more than in heap leach and tailings facilities. GCLs are used as the applied stress is much lower than that in heap leach and tailings facilities. Abramento *et al.* (2006) also mention the use of HDPE geocells filled with concrete to line a drainage channel in a mining area.


Figure 13. Example of a landfill pond in construction

Regarding the possible use of GCLs, it is important to consider the chemistry of the process solution: some mining process solutions are very aggressive (both acidic and basic) and could have a detrimental effect on the GCL performance. Simpson (2000) studied the effect of mine tailings solution on GCLs and found an increase in seepage rate due to chemical changes in the bentonite. Egloffstein *et al.* (2002) synthesised the possible effects of effluents on the evolution of bentonite features, especially its hydraulic conductivity. The main mechanism involved is cation exchange. Egloffstein *et al.* (2002) suggest that GCLs should not be used for lining of ponds containing concentrated solutions in non-polar organic liquids with a low dielectric constant like petrol, chlorinated hydrocarbons, xylol, and ethanol for example. Exceptions would only be possible when the GCL is saturated with water and when the contact remains short. No applications for permanent sealing purposes against acid with pH values lower than 3 and alkaline solutions with pH values greater than 13 are recommended. According to these authors preliminary tests should be performed for hard groundwaters (Ca^{2+}, Mg^{2+}), waters containing iron and leachate with high electrolytic concentrations. For highly concentrated pure solutions containing K^+ and NH_4^+ the application should be limited due to the specific interaction between those ions and montmorillonite. For heavy metals of high concentrations, the use of GCLs must also be evaluated by preliminary testing, like for hydrous solutions with polar organic liquids or leachate with high concentrations of organic cations. The authors also recommend for the use of GCLs with leachate and chemical solutions that the pre-swelling of the GCL be performed with water with low electrolyte content. The GCL should also be covered with at least 1m of soil. This is consistent with elements given by Renken *et al.* (2007) who indicate that a GCL may fail when it is needed to contain highly acidic solutions with a pH lower than 2 and that ion exchange may also be an issue for acid rock drainage as it contains high concentrations of divalent cations like Cu^{2+}, Zn^{2+} and Pb^{2+}. As observed by Touze-Foltz & Hatton (2006) in ponds dedicated to the collection of wastewater cation exchange between a needle-punched GCL containing natural sodium bentonite and a calcareous soil conducted to a change in hydraulic conductivity from 4×10^{-11}m/s to 10^{-9}m/s after 3 months of exposure. This resulted in a failure of the GCL liner. In this case the GCL was not prehydrated with water with a low electrolytic content, and the confining soil thickness was very low (See Table 2).

Table 2 provides an overview of references of geosynthetics in containment ponds. Cases studied presented in this table should not be considered as a substitute for specific studies pertaining to site-specific conditions. Table 2 shows that all kinds of geomembrane are used for those applications. In the case of ponds dedicated to storage of breeding effluents in France the use of soft geomembranes like PVC geomembranes is recommended (MAPA 1996).

Table 2. References of geosynthetics in containment ponds (Modified from Duquennoi 2002 and Duquennoi & Coquant 2003)

Source	Use	Age of GM (years)	Type and use of geosynthetic	Geosynthetic surface (m²)	Volume (m³)	Cover
Lescure (1983)	Sugar refinery	11	Butyl GM	N/a	N/a	N/a
Girollet (1983)	Sugar refinery	7	Bituminous GM (4.5mm)	60,000	133,000	Exposed
Girollet (1983)	Storm tank	5	Double lining system (polyane and elastomeric geomembrane) underliner geotextile, intra-lining drainage geotextile, protection geotextile	N/a	35,000	Shotcrete, precast blocks
Saintot & Bilancioni (1983)	Chemical industry	1	PP GM	10,000	N/a	N/a
Murillo-Fernandez (1994)	Waste water	2	PVC GM (1mm)	39,000	N/a	N/a
Breul & Herment (1995)	Storm tank	7	Bituminous GM (4mm), underliner geotextile on slopes	33,000	N/a	N/a
Fayoux *et al.* (1999)	Sugar refinery	1	HDPE GM (2mm), geotextile underliner, geospacer strips for underliner drainage	35,000	160,000	Exposed
Adams & Wagner (2000)	Food processing plant	11	HDPE GM (1mm) underliner nonwoven PP	14,970		Exposed

Source	Use	Age of GM (years)	Type and use of geosynthetic	Geosynthetic surface (m²)	Volume (m^3)	Cover
			geotextile (200g/m²)			
Garcin *et al.* (2002)	Road runoff	After installation	HDPE GM			Drainage net, reinforced geotextile and 0.3 to 0.4m soil included on slopes
Girard *et al.* (2002)	Manure pig pond	18	PVC GM (1mm)	N/a	N/a	Exposed
Girard *et al.* (2002)	N/a	18	PVC GM (1mm)	40,000	N/a	Exposed
Girard *et al.* (2002)	Leachate pond	10	PVC GM (1mm)	N/a	1,000 to 30,000	Exposed
Girard *et al.* (2002)	Storm tank	5	PVC GM (1.2mm)	N/a		Exposed
Girard *et al.* (2002)	Storm tank	15	PVC GM (1.3mm)	N/a		Exposed on slopes
Girard *et al.* (2002)	Storm tank	6	PVC GM (1.4mm)	N/a	1,000 to 30,000	Exposed
Girard *et al.* (2002)	Storm tank	7	PVC GM (1mm)	N/a		bottom granular drainage layer, exposed on slopes
Rollin *et al.* (2002)	Sewage treatment	N/a	Bituminous GM	2×2,050	2x2,460	Crushed rock
Spillemaecker *et al.* (2002)	Surface water and leachate	After installation	HDPE GM (2mm) 500g/m² puncture resistant geotextile	3×2,500		Exposed
Rowe *et al.* (2003)	Leachate	14	HDPE GM (1.5mm)		2,500	Exposed
Michelangeli *et al.* (2006)	Contamina-ted dredging sludge	After installation	HDPE GM (2mm)	Approx. 30,000		Protective geotextile 500g/m²
Nortjé & Meyer (2006)	Molasses	After installation	GCL + HDPE GM (1.5mm)		14,800	
Ouvry *et al.* (2006)	Brine	After installation	HDPE GM (1.5mm) on puncture resistant geotextile	20,760	130,000	Exposed
Touze-Foltz & Hatton (2006)	Sewage	After installation	GCL	4,200 2x1,750		0.3m calcareous soil
Wallace *et al.* (2006)	Sewage	After installation	HDPE GM (1.5mm) on geocomposite drain	Approx. 20,000		Exposed
Berube *et al.* (2007)	Salar evaporation	After installation	PVC GM on nonwoven geotextile in some cases	Approx. 300,000 each pond		Exposed

N/a: not available
GM: geomembrane

It also shows that the majority of geomembrane liner systems were left uncovered in containment ponds presented in this table. Where a cover was applied, it generally consists of granular layers, in-situ pour concrete covers, shotcrete cover and precast block cover.

The above mentioned information has to be considered with caution since they may be influenced by the effect of publication – most pond work do not lead to publication and only reference to exceptional cases are published in the literature (Duquennoi 2002). Whether or not it should be recommended to leave geomembrane lining systems uncovered is another question, since accelerated degradation of unprotected geomembranes attributed to excessive exposition has also been observed (Lambert *et al.* 1999). The actual trend is to recommend a protection on the whole surface of the geomembrane liner for runoff water ponds in France (SETRA & LCPC 2000). This is required by the need to get the bottom of the pond accessible for potential cleaning and for durability purposes. Garcin *et al.* (2002)

mention that ponds are more and more designed with green aspect solutions in order to take care of the integration in the surrounding landscape. This kind of application requires the use of anti-erosion or reinforced geosynthetics. In the case of the use of geomembranes for reed bed filters, Savoye *et al.* (2006) recommend the used of a geosynthetic to prevent the perforation of the geomembrane by reed rhizomes that can be very aggressive. Figure 14 gives examples of lining systems for road runoff ponds based on SETRA & LCPC (2000) showing the use of geotextiles under and over the geomembrane in a variety of cases.

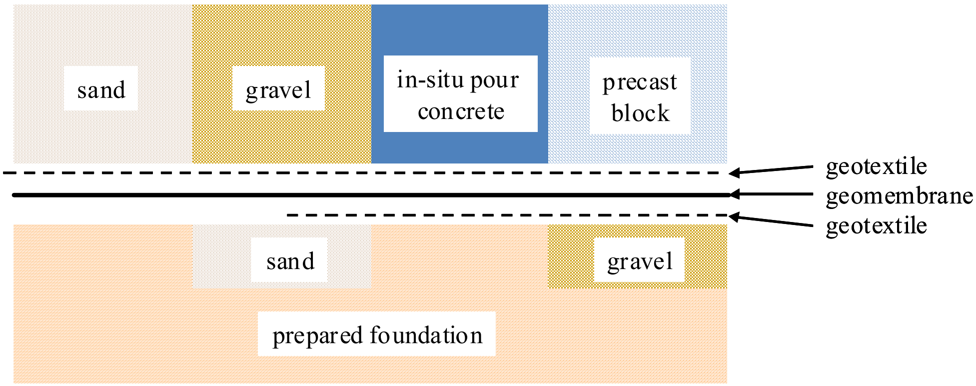


Figure 14. Example of lining systems for road runoff ponds (Adapted from SETRA & LCPC 2000)

The design of liner systems for lined ponds and channels need to consider the following issues:

- Exposure to ultraviolet and solar radiation which could affect the performance of the geosynthetic materials, thereby require more maintenance and repair work;
- Ice damage during winter month;
- Wear sheets or protection layers for solution pond decant structures and solution piping (for double containment designs);
- Compatibility of geosynthetic material with process solution chemistry;
- Stresses imposed on the liner system by decant pumping towers;
- High hydraulic head on the liner system;
- Leak collection sump and riser design; and
- Booting of pipes through the liner system, if needed.

Covers for liquid waste ponds and mining applications

A question which has not been often addressed up to now is floating cover of those reservoirs. Nortjé & Meyer (2006) present the case of a floating cover for a molasses reservoir made of a 1.4mm thick reinforced PP geomembrane. The design had to take account of possible gas generation from the stored molasses as well as water removal from the cover. A system of floats and weights was designed for gas and water removal. The gas removal system consists of a cuspated HDPE sheet strip installed along the perimeter to overlap with the floats underneath the cover and is connected to the atmosphere.

Temporary covers have been used on HLPs in countries where high rainfall dilutes operating solutions and surplus water can trigger the need for significant water treatment costs. Those cover generally include PVC geomembranes typically 0.75mm thick, for a temporary cover until the next dry season and thicker HDPE liners (0.75 to 1.5mm) for more permanent or reusable applications. Interlift liners have also been used in more than a dozen copper mine leach pads since 1993 to reduce acid consumption made of thin PVC (0.45 to 0.75mm thick) or LLDPE (0.75 to 1mm thick) liners (Breitenbach & Smith 2006).

A capillary barrier, combining of a non-woven geotextile with an overlying fine grained rock flour, was proposed by Park & Fleming (2006) for application in cover systems for mining wastes.

In Alcanena Industrial Liquid Waste Landfill a floating cover was installed (Machado do Vale 2007) made of reinforced PVC. Weights and floats were attached to the geomembrane to create bidirectional tension for the covers horizontal plats, eliminating slack. The PVC geomembrane was specially formulated and dimensioned to contact with such deposited materials, climate conditions, and to maintain its float capacity in time. Rainwater drains off of the top of the cover by gravity. In order to prevent retaining of the organic gases released during anaerobic digestion processes and to maintain the PVC geomembrane cover permanently adjusted, a pipe collection system was installed under the floating cover to contain and direct the gas fluids to external treatment.

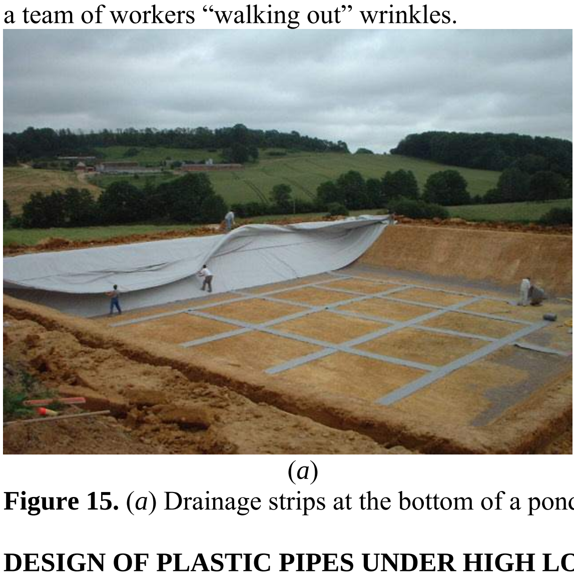
Gas collection systems in liquid waste ponds

It is generally not recommended to lay a geomembrane directly on the subgrade except in particular cases when the risks of the geomembrane puncturing and underliner pore water or gas pressure have been catered for (Duquennoi 2002). A protective layer may be interposed between the geomembrane and the subgrade when the latter is not smooth enough to guarantee geomembrane safety, especially below a high water head. Geotextiles or related products are now generally preferred because of their possible combined functions of gas drainage and mechanical protection of the underliner.

In France, in the case of ponds dedicated to disposal of breeding effluent, when a geomembrane is used for lining it has to be associated to a protective geotextile underneath the geomembrane together with a liquid and gas drainage system. The use of a protective geotextile underneath the geomembrane is also a recommendation from SETRA & LCPC (2000) illustrated on Figure 14 for ponds lined with geomembrane liners dedicated to the disposal of road runoff. SETRA & LCPC (2000) also mention that the use of a geotextile underneath the geomembrane can be a good way of protecting it from dust arising from soil particles that will disrupt the seaming process.

The purpose of underliner liquid drainage is to prevent the accumulation of liquid leaking from the pond as well as the uplift of the geomembrane lining system due to back-pressure from a raised water table. Underliner water drainage can be performed either by gravel layers, gravel-filled drainage trenches, or geosynthetic drainage strips, the latter being illustrated on Figure 15*a*. Depending on the volume of liquid to be drained, perforated geopipes may supplement gravel-based drainage systems. Drain pipes are always connected to a main collecting pipe or manhole and then to a pump or gravity outlet. Special attention must be given to filters associated with draining materials (Duquennoi 2002). In storage applications, filters are generally designed using geotextiles and state-of-the-art geosynthetic filter design methods are to be applied. An additional benefit of underliner liquid drainage, especially in containment ponds is the possibility of using it to monitor the drained liquid quantity and quality in order to detect leaks in the lining system.

Underliner gas drainage is as essential as liquid drainage, especially when organic fermentation gas or compressed soil pore air, resulting from a water-table rise, are expected. Underliner gas drainage has to be provided when gas fermentation may occur, especially where the total excavation of soils is not economically sustainable, where older storage structures may have caused undetected organic liquid infiltration (i.e. sugar factory plants, farms, etc.) and in the case of organic liquid leaks in the geomembrane lining system. Underliner gas and liquid drainage have frequently been combined in the same systems, but state-of-the art designs now tend to separate them, with water being drained in trenches or geosynthetic strips and gas being drained in a geotextile underliner connected to gas vents passing through the geomembrane at the top of the embankments. The latter solution is particularly attractive when the same geotextile is designed to perform both underliner gas drainage and protection. Nevertheless those gas drainage systems can be insufficient at the stage of the initial filling of a pond when the geomembrane is unloaded, or in case the floor is flat (Peggs 2008). It may then be desirable to use a sheet drain system such as a geotextile/geonet/geotextile geocomposite. Wallace *et al.* (2006) showed that a considerable amount of air can be entrapped in geomembrane wrinkles during installation. When ballasted by soil cover the wrinkles are flattened out due to the pressure applied by construction equipment and air is expelled into the ground or the underlying drainage layer. If the geomembrane is not ballasted by soil cover liquid pressure during the first filling of the reservoir may flatten some of the wrinkles by applying pressure on the wrinkles or by thermal contraction due to the low temperature of the liquid. As a result some air is expelled and may accumulate under the geomembrane in high areas of the reservoir bottom. This results in uplifted areas of the geomembrane buoyed by the underlying air pockets as shown on Figure 15*b*. The vents provided in the geomembrane and the drainage layer located beneath the geomembrane and connected to the vents are necessary but not sufficient to ensure air removal during the first filling of the reservoir. The solution found was to use a team of workers "walking out" wrinkles.


(*a*)

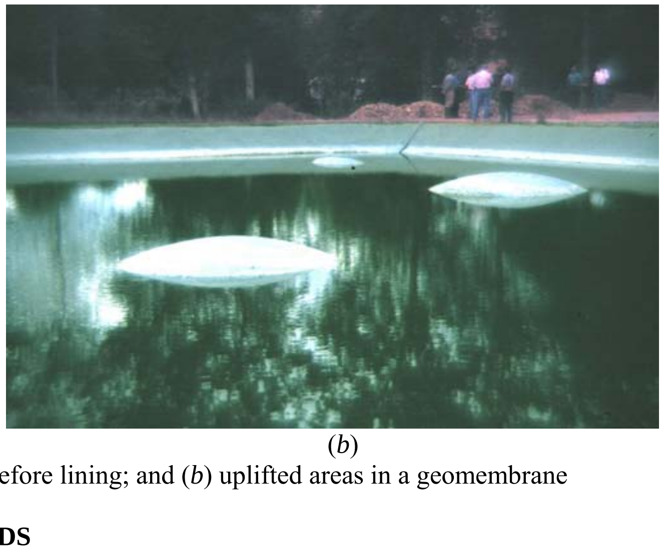

(*b*)

Figure 15. (*a*) Drainage strips at the bottom of a pond before lining; and (*b*) uplifted areas in a geomembrane

DESIGN OF PLASTIC PIPES UNDER HIGH LOADS

Introduction

In landfills and mining applications there is great interest in the design and performance of geopipes under high loads. Geopipes are used for leachate collection in landfills. In mining applications, geopipes are incorporated into the design of HLPs and TSFs for the collection of process solutions, and used beneath waste rock facilities to collect poor quality seepage. Geopipes act as high capacity, flexible conduits that, when properly designed and installed, provide an effective means for collection of solution, drainage, and overall solution management. The focus of this section is on the impact of high loads. After an overview of geopipes including design considerations, modelling design methods for buried pipes will be presented and design considerations discussed.

Overview of geopipes

A typical geopipe layout for a HLP is presented in Figure 16. Geopipes within HLPs are critical, as they do not only provide solution collection for process, but they also control the solution level within the ore heap, which affects stability of the entire facility.

Figure 16. HLP CPe solution collection pipes

Geopipes are generally constructed from Polyethylene (PE) or PVC, and are available as smooth (solid wall) or corrugated wall. Solid wall pipes are commonly used in applications where internal pressures, between 340 and 1,760kPa, are present and/or where the pipe may be subjected to high external loads. Corrugated PE (CPe) pipes are available as single-walled, double-walled, and triple-walled pipes. Corrugated wall pipes provide a light-weight alternative to solid wall pipe, and may be used in applications with high external loads. Recently, sealed CPe pipes have been used to convey solutions with internal pipe pressures up to 34kPa. Typical geopipe installations, with and without pipe trenches, are illustrated in Figures 17 and

(*a*) (*b*)

Figure 17. (*a*) PVC pipe installation and (*b*) installation of CPe dual and single wall pipes

(*a*) (*b*)

Figure 18. (*a*) Installation of solid HDPE pipe and (*b*) Pipe installation in trench

For many facilities, geopipes are used for both above ground and buried applications. The design methods for geopipes located at the surface or for shallow burial depths (less than 5m), are relatively well defined and established (e.g. Iowa Method and Modified Iowa Method, as discussed later in this section). However, design methods for geopipes with deep burial depths are not well defined nor understood. Understanding the performance and limitations of geopipes in deep burial applications is important as pipes are being designed and installed in harsh environments, such as in mining applications. At these facilities, plastic pipes may be exposed to high overburden pressures (up to 3.5MPa at some mines), be exposed to highly acidic (pH between 1 and 2) solutions, or exposed to basic (pH 10 or above) solutions containing dissolved metals. Clearly, at these deep burial depths, understanding pipe performance is critical to avoid pipe failure, such as that illustrated in Figure 19.

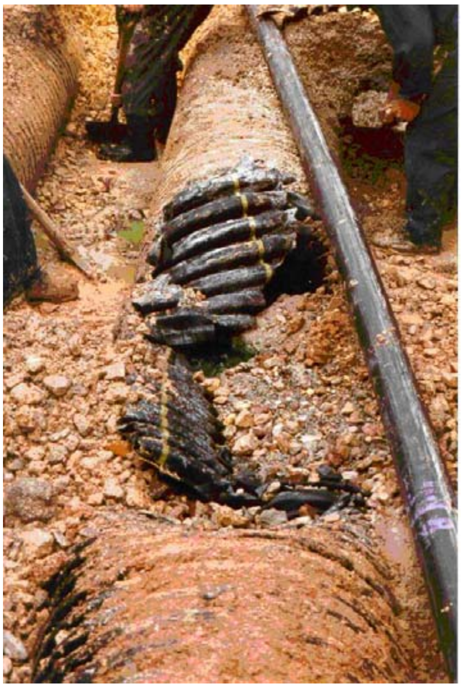


Figure 19. Pipe failure by crushing

Prior to presenting design concepts for geopipes, it is important to present definitions, which are common to most design methodologies. Figure 20 presents a schematic of a buried pipe, identifying some of the key names referenced in pipe design equations. As shown, the pipe crown is top of the pipe, while the springline is the central section of the pipe. The pipe envelope refers to the material that surrounds the pipe, and ring deflection is the ratio between the vertical change in pipe diameter under load to the original pipe diameter.

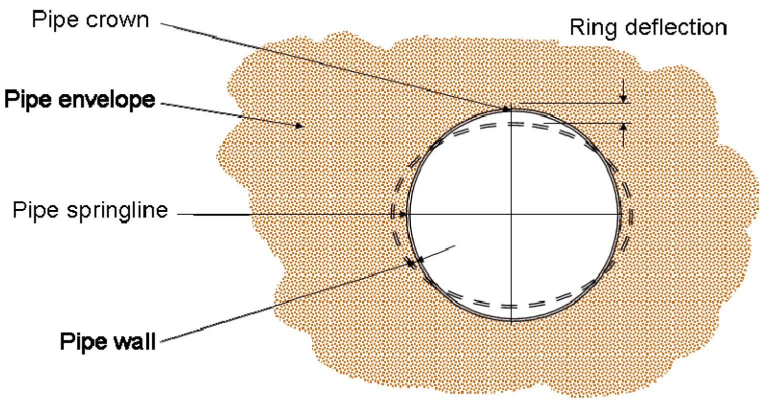


Figure 20. Pipe design common definitions

Modelling design methods for buried pipes

Historically, design methods for buried pipes considered both rigid and flexible pipes. Rigid pipes are generally recognized as pipes constructed from materials that are "stiffer" than the surroundingpipe envelope. Examples of rigid pipes include steel and concrete pipes. Steel and concrete pipes are often used in critical applications in both tailings and leach pads. Flexible pipes are constructed from materials that have a lower stiffness than the pipe envelope. For the most part, geopipes are considered to be flexible pipes. Both PE and PVC materials exhibit viscoelastic properties which have a lower stiffness than most pipe envelope materials, such as well graded, compacted sands and gravels. Both rigid and flexible pipes have unique performance issues that must be considered in design (Krizek 1990).

Before presenting a discussion on buried pipe design methods, it is important to recognize that the design criteria and performance for buried pipe designs will vary, depending on the type of project, life of the facility, solutions to be conveyed by the pipes, and method of construction. A single design criterion for all projects is not practical as it does not consider project specific issues or limitations. For example, the design criteria for drainage pipe installed at a landfill with a plus 100 year life will be completely different from solution collection pipe installed in HLP at a mining facility with a 10 year life. In the latter case, the mining facility has a limited life span, after which the facility will be reclaimed. Therefore, higher ring deflection and limited pipe buckling may be acceptable for solution collection pipes in the HLP, because it has a short operating life, after which it will not need to convey significant solution flow. The design criteria and desired pipe performance should be carefully considered as part of the overall pipe design (Watkins 1990).

Traditional methods for the design of buried geopipes follow from work of Marston & Anderson (1913) and Spangler (1941), which consider pipes with shallow burial depth. The work by Spangler (1941) was later revised into what is known as the Iowa Method and the Modified Iowa method (USDA 1990), and has been a standard for design for a number of years. These methods generally consider the following performance criteria:

- Pipe Wall Crushing: wall crushing occurs when the wall stress exceeds the long-term compressive strength of pipe material;
- Pipe Wall Buckling: wall buckling occurs when the total soil pressure exceeds the pipe critical buckling pressure; and
- Ring Deflection; typically, pipe deflection is calculated using the Modified Iowa method (USDA 1990); the design limit for ring deflection is commonly assumed to be approximately 5 percent; Krizek (1990) reports that this value was derived from the inspections of numerous pipe installations, where the average deflection before failure was determined to be about 20 percent of the pipe diameter; assuming a safety factor of 4 results in a design ring deflection of 5 percent.

While Modified Iowa method is still used for many buried pipe design, the method is based on a number of assumptions that are often not applicable to deep burial depths. For example, the pressure at the pipe crown (due to burial) is assumed to be equivalent to the soil density times the depth of burial, modified by various coefficients to account for different pipe configurations (e.g. pipe in trench, pipe projecting from trench, no pipe trench, etc). While this assumption may be applicable for burial depths less than 5m, it is not suitable for deep burial. A number of field stress measurements and laboratory studies conducted by Adams *et al.* (1988), Reeve *et al.* (1981), Sargand *et al.* (1993), Watkins (1990), Watkins *et al.* (1987) and Watkins & Reeve (1979) have demonstrated that, due to arching within the soil column above the pipe, the pressure at the pipe crown can be significantly lower than that predicted by the Modified Iowa method. The Modified Iowa method actually assumes a "flat" arch which contributes little support to the pipe.

An example of soil arching from actual field measurements presented by Adams *et al.* (1988) is presented in Figure 21. As shown, the actual stress near the pipe is significantly lower than the geostatic stress, used in the Modified Iowa method. Soil arching was first noted by Janssen (1895) in flow of materials in grain silos, where grain would form a stable "arch" over the silo chute, preventing flow. This phenomenon was extended by Terzaghi (1943) to soil mechanics and the design of retaining walls. Handy (1985) later developed a formal analysis of soil arching showing that as a stress arches forms, a rotation of the principle stresses occurs, resulting in an increased active pressure in the arch. The importance of soil arching is geopipe design is discussed further in this Keynote Lecture.

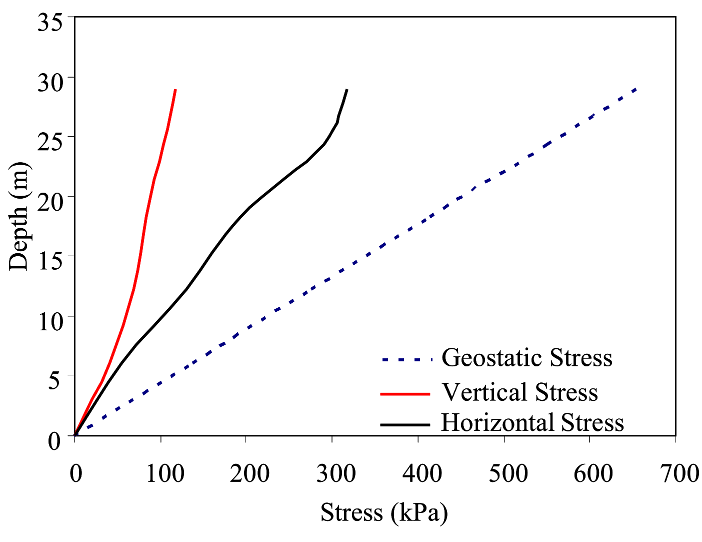


Figure 21. Stress arch above pipe (after Adams *et al.* 1988)

Studies by Adams *et al.* (1988), Lefebvre *et al.* (1976), Reeve *et al.* (1981), Sargand *et al.* (1993), Selig (1990), Valsangkar & Britto (1978), Watkins (1990), Watkins *et al.* (1987) and Watkins & Reeve (1979) show a strong correlation between pipe performance and the compressibility of the pipe envelope material. Faragher *et al.* (2000)

demonstrated that the pipe envelope plays an important role in pipe performance under repeated loading-unloading cycles, simulating traffic loads. A schematic illustrating the pipe envelope interaction with respect to pipe performance, based on the study by Watkins & Reeve (1979), is shown in Figure 22. As illustrated, the combinations of pipe envelope and pipe stiffness will affect the overall pipe performance and deformation patterns.

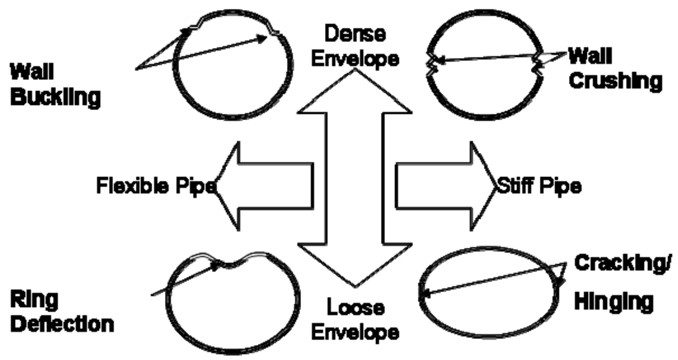


Figure 22. Pipe and pipe envelope interaction (from Watkins & Reeve 1979)

Willardson & Watkins (2002) present a discussion on the pipe envelope requirements for flexible pipe. In their study they also considered trenching versus non-trenching for pipe support and soil arching. Field tests showed that placing flexible pipes in a "V" trench, prior to covering, provides beneficial pipe support and reduces ring deflection.

Using field and laboratory data, Watkins & Reeve (1979) developed a semi-empirical design methodology for deep burial of flexible pipes using a ring compression criterion for pipes with dense pipe envelopes, and ring deflection for pipes with loose pipe envelopes. In their work, they concluded that, depending on the pipe envelope characteristics, the pipe ring deflection will be approximately equal to the vertical strain of the pipe envelope material, consistent with studies by Adams *et al.* (1988), Watkins (1990) and Willardson & Watkins (2002). In other words, the geopipes will remain open and not collapse as long as arching is present in the pipe envelope and the total strain does not exceed certain thresholds. For example, Willardson & Watkins (2002) found that geopipes will remain open with ring deflections up to 20%. Beyond 20%, the pipe will invert, but still remain open. Their study also found that pipe corrugations began to buckle at ring deflections of 5 percent. These findings are reinforced by field data from actual deep burial tests on flexible pipes. A compilation of measured pipe deformation data presented in Adams *et al.* (1988), and Howard & Selander (1974), Sargand *et al.* (1993), Shupe & Watkins (1985), Watkins *et al.* (1987) and Watkins & Reeve (1979) is presented in Figure 23. Data show that pipes remain open and functional even under high overburden stresses (500 to over 2,000kPa) and ring deflections (5 to over 35%).

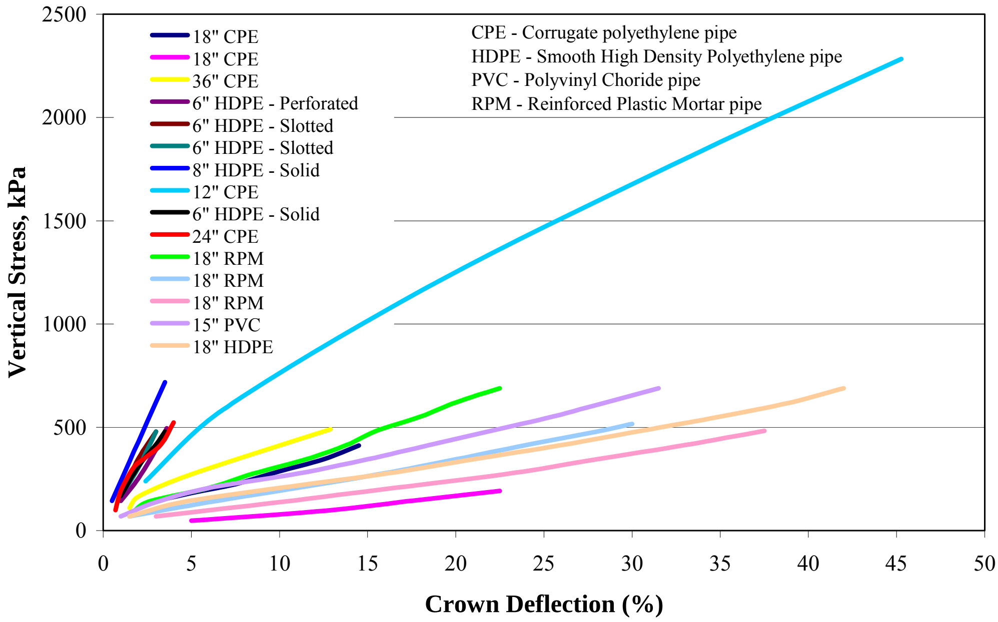


Figure 23. Measured pipe deformation from deep burial field tests

While the design methods developed by Watkins & Reeve (1979) are useful, the equations are limited in application as they cannot consider site specific pipe envelope material properties. Burns & Richard (1964) and Höeg (1968) developed closed form, plane-strain solutions for thin, circular conduits buried in an elastic soil. These equations are useful as they include the interaction of the pipe with the pipe envelope, both of which have independent material properties. In addition, the equations explicitly consider soil arching within the pipe envelope. These equations can be used to estimate pipe stresses and displacements for two limiting cases: “full slip” (zero shear stress between the pipe envelope and pipe interface) and “no slip” (continuity of displacements along the pipe envelope and pipe interface). The “full slip” case represents a condition where the soil is displaced (slips) around the pipe. The “no slip” case represents uniform deformation of the pipe and soil. These methods have been used by Brachman (2001) and Lupo (2001) to assess the performance of plastic pipes under high overburden pressures.

Numerical methods, such as finite element and finite difference, have also been used to analyses geopipe performance under various loading conditions. Katona (1988) used a finite-element soil-structure code to determine the maximum burial depth for CPe pipes up to 760mm diameter. In this analysis, the maximum burial depth was determined based on the load to exceed 7.5% ring deflection. Using this criterion, the maximum calculated burial depths for CPe pipes were found to be approximately 10m for most pipe sizes. Trickey & Moore (2007) studied the three-dimensional response of both stiff and flexible pipes using finite elements. Their study found that ring deflection of flexible pipes decrease with increasing depth due to a decrease in the peak moment acting on the pipe.

Design considerations

When designing geopipes for deep burial mining applications, it is important to define acceptable design criteria and pipe performance for the facility. The design criteria may vary depending on the type of facility, hydraulic considerations (head and flow rates), stability, construction materials, and facility operation. For example, in some mine facilities, high ring deflections (up to 20%) may be acceptable, as long as the geopipe has sufficient flow capacity. The design criteria should consider minimum open area (for solution flow), maximum crown deflections, type of pipe envelope materials, and construction method. The construction method is critical, as many pipes have failed at mines due directly to poor pipe and pipe envelope placement and compaction.

Figures 24 through 26 present photographs from field investigations and video surveys of geopipes at various mine sites. It is important to note that, although the pipes are damaged, showing wall crushing/buckling, and high ring deflection, the pipes are still functioning by collecting and conveying solution flow. All of these pipes are meeting their original design criteria.

(*a*)

(*b*)

Figure 24. (*a*) 0.6m CPe pipe under 90m of ore (~ 1.6MPa) with 20 percent ring deflection and (*b*) 0.45m diameter CPe pipe under 60m of ore (~ 1.1MPa) with 15 percent ring deflection

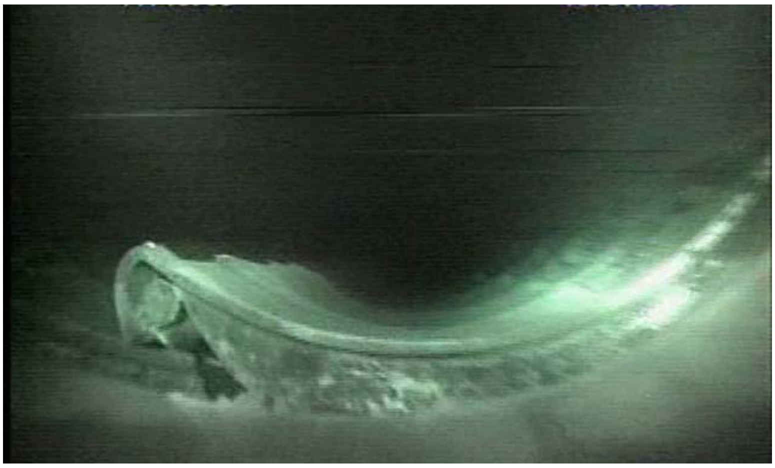


Figure 25. Wall buckling 0.91m Diameter CPe pipe under 50m of ore (~ 940kPa)

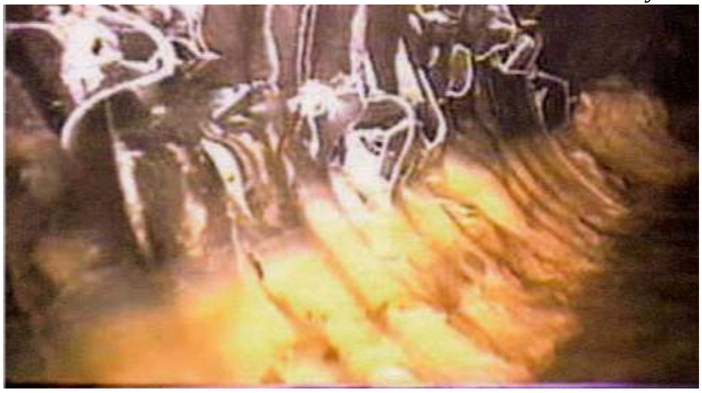


Figure 26. 0.6m diameter CPe Pipe wall buckling under 60m of ore (~ 1.0MPa)

The data from these field investigations and video surveys have been useful in understanding the performance of geopipes under deep burial conditions and formulating design methods for these pipes. Experience gathered from these investigations at mine sites for deeply buried geopipes has shown that the performance of the pipe envelope primarily governs the geopipe response. In instances where the geopipe has failed (see Figure 27), the failure was found to be related to poorly placed and compacted pipe envelope materials (Zhan & Rajani 1997, Willardson & Watkins 2002)

Figure 27. Geopipe failure

Given these observations, careful consideration should be given to the quality and compressibility of the pipe envelope material. Figure 28 presents compression curves from some pipe envelope materials. As shown, fresh crushed rock provides a relative "stiff" material, with a vertical compression of less than 7 percent at 2.5MPa. At the same vertical stress level, a well graded gravel had a compression of nearly 15 percent, while a weathered crushed rock had a vertical compression of over 20%. Assuming the geopipe ring deflection is equivalent to the vertical strain of the pipe envelope, the fresh crushed rock would be recommended to minimize ring deflection of the pipe and provide sufficient arching support.

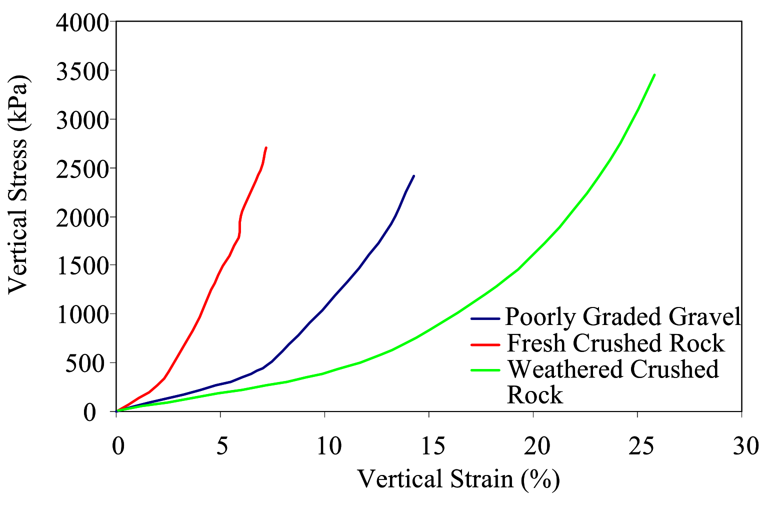


Figure 28. Pipe envelope compressibility

When selecting a suitable pipe envelope material for mining applications, the following guidelines are provided:

- Select an angular material that helps promote arching around the geopipe;
- Use fresh or unweathered materials to reduce compressibility;
- Test the pipe envelope for compatibility with the solutions that will contact the material; the pipe envelope should not degrade over time due to exposure to solution, or the soil arching will be reduced;
- In general, pipe envelope materials with maximum particle sizes less than 25mm work well as these materials can be packed around the geopipe to provide support;
- If the pipe envelope is to be used for drainage, the hydraulic characteristics of the material must be tested at stresses higher than those corresponding to the depth of burial. The stresses within the pipe envelope may exceed those imposed by the depth of burial as a result of soil arching around the geopipe. These stresses may be sufficient to crush the material, causing loss of permeability.

In addition to selecting the suitable pipe envelope material, consideration must be given on the minimum thickness of material around the geopipe. This is an important aspect, as arching will only occur around the geopipe if there is sufficient pipe envelope thickness. A too thin pipe envelope will not be able to support arching around the pipe. Watkins & Reeve (1979) found that the pipe envelope should extend at minimum a distance of half the pipe diameter from the geopipe whether the pipe is in a trench or not. However, Lupo & Morrison (2005) recommend twice the pipe diameter for deep burial geopipes as an initial minimum thickness, based on elastic analyses. Experience from mines has shown that thicker pipe envelopes will generally provide better protection and better geopipe performance than thinner pipe envelopes.

GEOMEMBRANE LINER DESIGN METHODS FOR HIGH LOADS

Introduction

As discussed previously, the performance of geomembrane liners under most loading conditions is strongly influenced upon the interaction and compatibility of the various components within the liner system. This is particularly important in applications where high loads may be applied to the liner systems, such as lined reservoirs, mining facilities (HLPs, tailings impoundments, and waste rock facilities), and industrial applications. The selection and design of the geomembrane liner for applications with high loads requires a thorough understanding of the interaction between the liner system components and the type of applied load (normal and shear loads). The following of this section, after a brief overview of the use of geomembranes under high loads will focus on the presentation of a methodology to assess the adequate selection and design of the geomembrane. High load foundation settlement and internal settlement will also be briefly addressed.

Overview of the use of geomembrane sunder high loads

When the interaction of the liner system components is considered in the liner system design, geomembrane liners have been shown to perform well under very high loading conditions. Table 3 presents a summary of geomembrane liners 1.5mm to 2.5mm thick that are in operation today under high loads. As shown, the normal stresses on geomembrane liners range from 0.8 to 3.5MPa.

Table 3. Summary of high loads on geomembrane liners in operation

Geomembrane type	Maximum fill height (m)	Maximum vertical stress (MPa)
2.5mm LLDPE	180	3.5
2.0mm LLDPE	150	2.7
2.0mm LLDPE	180	3.2
1.5mm HDPE	150	2.7
1.5mm LLDPE	180	3.3
2.0mm HDPE	90	1.6
1.5mm LLDPE	90	1.7
1.5mm LLDPE	68	1.2
1.5mm HDPE	45	0.8
2.0mm HDPE	90	1.6
2.0mm HDPE	75	1.2
1.5mm HDPE	60	1.0
2.0mm HDPE	75	1.1
2.0mm HDPE	90	1.4

In the design of liner systems under high loading conditions, there are three primary considerations for the selection of the geomembrane liner:

- Potential puncture of the geomembrane liner under load;
- Development of unacceptable strain and tensile stress within the geomembrane liner due to foundation settlement; and

- Development of unacceptable strain and stress resulting from settlement of material placed over the geomembrane liner.

High load geomembrane liner puncture

For geomembrane liner applications under high loads, the selection of the geomembrane liner thickness and material type are generally based on performance under anticipated loading conditions, the angularity of the material placed immediately above or below the liner, estimated foundation settlement, and the material properties (friction, tensile strength, etc.).

In terms of geomembrane liner thickness, theoretical analyses such as that presented in Giroud *et al.* (1995) may be used, however it is more common to conduct a series of tests on the geomembrane liner using the methodology described in Environmental Agency (2006), Brachman *et al.* (2000), Lupo & Morrison (2007), Shercliff (1998) and Thiel & Smith (2004).These tests allow the overall performance of the entire liner system under load to be evaluated under highly confined conditions. A schematic of a testing frame and configuration is presented in Figure 29.

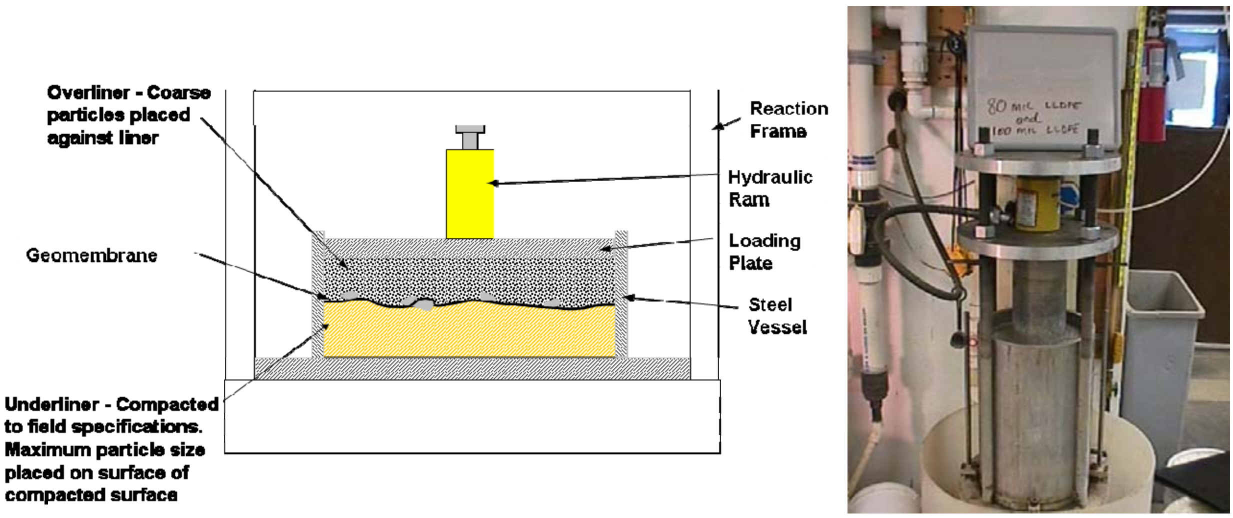


Figure 29. Liner load testing frame (*a*) schematic and (*b*) photograph

This type of testing frame is commonly referred to as a "cylinder test" (Environmental Agency 2006), and consists of a rigid vessel (either cylindrical or a rectangular box) with a loading ram and platens on either end. The testing procedure consists of constructing the liner system under consideration (single composite/double composite) using the same specifications as those to be used in the field construction. The liner system is then loaded under the anticipated loading conditions. The magnitude of the applied load and the time frame the load is kept on the liner system varies depending on the application. The Environmental Agency (2006) guidance recommends a maximum load 2.25 times the overburden pressure for tests at 20˚C lasting 1,000 hours, or a maximum load 2.5 times the overburden pressure for tests at 20˚C lasting 100 hours.

Upon completion of the test, the geomembrane liner is inspected for punctures, both visually and by applying a vacuum (vacuum pressure of 70mmHg). The Environmental Agency (2006) test procedure also includes the use of a lead impression sheet so that deformation of the geomembrane liner can be directly measured.

It is important to recognize that the cylinder test frame and testing procedure differ from the testing procedure presented in ASTM D5514/D5617, which are commonly used to assess the performance of geomembrane liners under load. The difference in the test frames and procedures reflect the different boundary conditions represented by each testing method. The cylinder test represents fully confined conditions, which are typical at the base of facilities under high normal loads. ASTM D5514/D5617 represent conditions whereby the geomembrane liner is less confined and exposed to greater biaxial strains.

A typical cylinder test set-up, showing the sequence of construction and post-test vacuum testing is shown in Figure 30. The test should be set-up to be representative of the actual liner system, and include materials placed below and above the geomembrane. In the test shown in Figure 30, liner bedding soil is placed and compacted in the bottom of the test vessel, the selected geomembrane liner sample is placed over the liner bedding soil and drainage gravel is placed over the liner. It is important to note that, due to the construction of the cylinder test, the test method assumes the foundation is nearly incompressible. The validity of this assumption should be assessed for each application, as discussed later in this section.

An important aspect of geomembrane liner testing for high load applications is to consider the presence of oversized particles in the subgrade and in the material placed directly on the geomembrane. The presence of oversize particles may lead to puncture of the geomembrane liner either during construction or after the application of high normal loads. An illustration of these conditions is presented in Figure 31. To evaluate the performance of the geomembrane liner under these conditions, tests are conducted using a cylinder test or similar set-up. In these tests, oversized particles are purposely placed above and/or below the geomembrane liner prior to load application. The maximum particle size is selected based on the project specifications and site conditions. For example, if the maximum acceptable particle size for the liner bedding soil (underliner) is 50mm, then 50mm rock particles are placed on the surface of the underliner. These particles represent loose rocks that may have been liberated from the underliner during placement or during geomembrane liner deployment.

(*a*) (*b*)

Figure 30. (*a*) Liner load test to 3 MPa and (*b*) Liner load test results

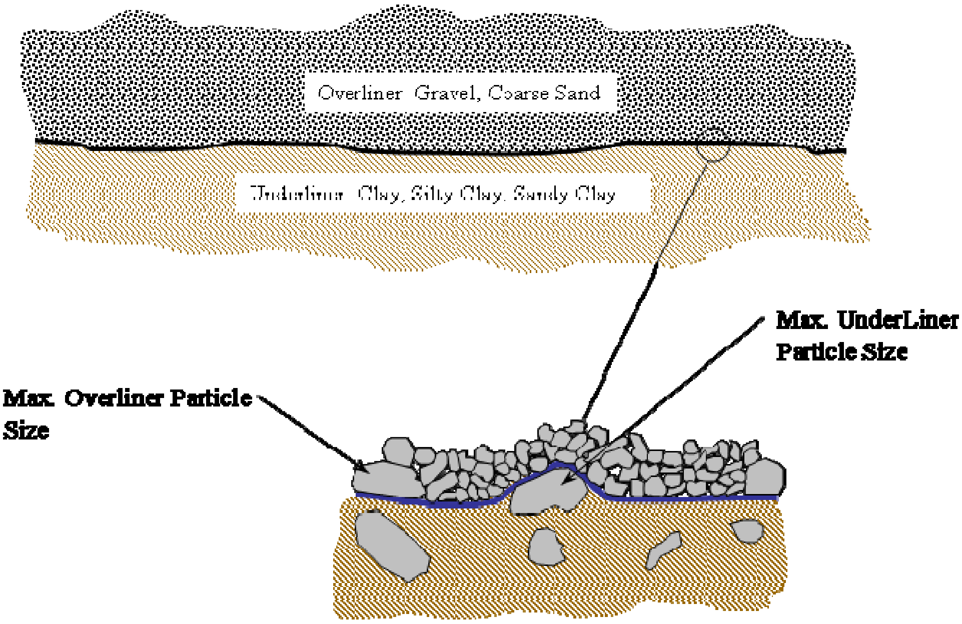
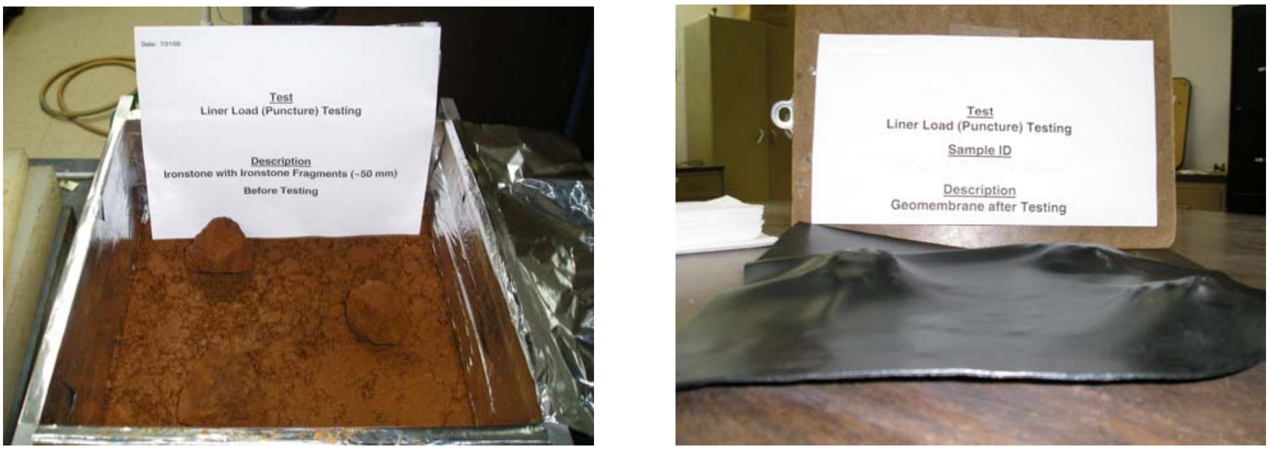


Figure 31. Rock particle in underliner

Figure 32*a* shows the test set-up to evaluate the performance of 2.0mm LLDPE with loose rock particles in the underliner. For this test, a rectangular loading box was used to provide a large section of geomembrane liner. The underliner was compacted to the project specifications. On the surface of the underliner, three 50mm diameter rock particles were placed. This condition reflects a condition of poor surface preparation before liner deployment. The LLDPE geomembrane liner was placed over the underliner and loaded to a maximum normal stress of 2.0 MPa.

The post-test geomembrane liner is presented in Figure 32*b*, showing a significant amount of strain. Although the geomembrane was deformed, the liner was not punctured. The rock particles placed on the underliner surface were compressed into the underliner and were broken.

Based on experience with high loads in mining applications, Lupo & Morrison (2007) developed recommendations presented in Table 4 providing a general guideline for geomembrane liner selection based on the applied load and characteristics of the foundation, overliner materials and underliner materials. This table may be used as a general starting point for geomembrane selection (type and thickness), however specific testing should be conducted to assess geomembrane liner performance for the given site conditions.



(*a*) (*b*)

Figure 32. (*a*) Test set-up with rock particles on liner bedding soil and (*b*) deformed 2.0mm LLDPE geomembrane liner.

Table 4. General Heap Leach Geomembrane Liner Selection Guide after Lupo & Morrison (2007)

Foundation conditions*	**Liner bedding soil†**	**Overliner material‡**	**Effective normal stress (MPa)§**		
			<0.5	0.5< <1.2	>1.2
Firm or high stiffness	Coarse-grained	Coarse-grained	2mm LLDPE or HDPE	2mm LLDPE or HDPE	2.5mm LLDPE or HDPE
		Fine-grained	1.5mm LLDPE or HDPE	2mm LLDPE or HDPE	2.5mm LLDPE or HDPE
	Fine-grained	Coarse-grained	1.5mm LLDPE or HDPE	1.5mm LLDPE or HDPE	2mm LLDPE or HDPE
		Fine-grained	1mm LLDPE or HDPE	1.5mm LLDPE or HDPE	2mm LLDPE or HDPE
Soft or low stiffness	Coarse-grained	Coarse-grained	2mm LLDPE	2mm LLDPE	2.5mm LLDPE
		Fine-grained	1.5mm LLDPE	2mm LLDPE	2.5mm LLDPE
	Fine-grained	Coarse-grained	2mm LLDPE	2mm LLDPE	2.5mm LLDPE
		Fine-grained	1.5mm LLDPE	2mm LLDPE	2.5mm LLDPE

*Description of foundation conditions is a relative measure of stiffness. The foundation conditions need to be investigated and tested to determine compatibility with the geomembrane

†Liner bedding soil refers to the soil in direct contact with the underside of the geomembrane. Testing and design calculations are required to assess compatibility with the geomembrane

‡Overliner refers to the material placed directly onto the geomembrane. Testing and design calculations are required to assess compatibility with the geomembrane

§Effective normal stress is the maximum stress onto the geomembrane due to the ore

High load foundation settlement

The application of high loads may also lead to large or unacceptable settlements, both total and differential, within the foundation of a geomembrane lined facility. The large settlements may lead to the development of tensile stresses exceeding the yield strength of the geomembrane liner. Discussions on the affects of settlement on geomembrane liners are presented in Giroud & Soderman (1995) and Giroud (2005), while allowable long-term geomembrane stresses are discussed in Berg & Bonaparte (1993).

Examples of liner systems designed for compressible foundations are presented by Dillon *et al.* (2004) and Lupo & Morrison (2005). Dillon *et al.* (2004) present a discussion on the design of a geomembrane liner system over a soft, peat foundation. This liner system consisted of a 2.0mm LLDPE liner placed over the peat foundation, with settlements anticipated to be 20 to 80 percent (consolidation strain). Lupo & Morrison (2005) present a discussion on the foundation settlement of a slope lined with a 2.5mm LLDPE geomembrane. The slope was subjected to 3MPa of normal stress, and was predicted to have settlements up to 4.5m.

The design of liner systems for large foundation settlements requires an analysis of the foundation conditions using either analytical or numerical methods. Analytical methods, such as the deflection method presented by Giroud *et al.* (1990), Giroud (1995), and Miura *et al.* (1990), provide a simplified approach that may be used to estimate liner strain and tensile stress. Numerical methods, using finite-difference or finite-element approaches, may be required to evaluate complex geometry. An example of using the finite-element method for liner system performance assessment is presented in Reddy *et al.* (1996) which used the finite element analysis to evaluate the interface shear strain between the components of a composite liner system.

Internal settlement

Internal settlement refers to settlement that occurs in the materials that are placed onto the liner system (e.g. waste, ore, etc). Settlement of materials on liner systems can result in the development of tensile stresses in the geomembrane liner above the yield strength. The potential for large-scale internal settlement increases with the thickness of material and the applied stress. In mining applications, there have been several geomembrane liner failures under high loads due to settlement and shear loading (Breitenbach 1997). Kodikara (2000) presents analytical solutions to estimate the magnitude of tensile stress and strain development in geomembrane liners from internal settlement for landfills, but these solutions may be used in other applications. Other methods that may be used to assess stress and strain with geomembranes include the following:

- Giroud (2005) developed a "co-energy" approach to evaluating strain development in geomembrane liners affected by settlement; and
- Liu & Gilbert (2005) presented a simplified graphical solution for estimating the stresses.

An alternative approach for evaluating the induced stress and strain in a geomembrane liner from internal settlement is to use numerical methods. Filz et al. (2001) used the finite element method to assess the progressive failure of the Kettleman Hills landfill. The advantage of using a numerical approach is that complex geometries can be evaluated. In addition, the non-linear settlement behavior of the materials can be simulated explicitly.

PUNCTURE PROTECTION OF GEOMEMBRANES AND GCLS

Introduction

As previously mentioned in the section dedicated to the overview of the use of geosynthetics for geoenvironmental applications, many modern landfill barrier systems require a LCS and primary composite liner involving a HDPE geomembrane over either a CCL or a GCL. The benefits of using geosynthetic liners as part of a barrier system may not be fully realized if physical damage occurs to either a geomembrane or a GCL. The particular focus here is on the damage that can occur to geomembrane and GCL liners from the drainage gravel of an overlying LCS.

The weight of the overlying waste is transferred to the foundation layer through the drainage gravel and composite liner. The grain size and grain size distribution of the drainage gravel along with the weight of the overlying waste will then control the potential physical damage that could occur to the liner. From the perspective of prolonging the service life of the LCS, it is desirable to use coarser 50mm gravel to minimize the implications from biologically induced clogging of the LCS (Fleming & Rowe 2004). However, as the gravel becomes coarser, the geomembrane experiences larger and more widely spaced contact forces (Brachman & Gudina 2008*a*). Thus, the requirements to provide adequate physical protection to the geosynthetic liner(s) depend on an overall assessment of barrier system design and the materials selected. To help alleviate the strains induced in the geomembrane by the gravel different types of protection layers i.e. geotextiles, a sand layer (see Figure 5) or combinations of geotextiles and sand layers, sand-filled cushions, rubber mats and even GCLs (see Figure 33) have been introduced between the geomembrane and coarse drainage gravel.

Figure 33. GCL used as a protection of the geomembrane at the Attainville MSW landfill (from Didier *et al.* 2006)

The question that arises is to know up to which extent a given material provides adequate protection of the geomembrane. Various experiments performed either in large scale laboratory tests or field tests aiming at quantifying the adequate protection for given geomembrane liner and granular layer will be presented in the following of this section after a brief presentation of the philosophy of puncture protection.

Philosophy of puncture protection

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 the geomembranes past the yield point, thus allowing deformations while preventing puncture of the geomembrane. There is no upper limit given for the local strain. 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 factor of safety against geomembrane yield (Bouazza *et al.* 2002). Along the same philosophy, Badu-Tweneboah *et al.* (1998) presented another approach for evaluating the effectiveness of a geomembrane liner protection. This approach is based on the multi-axial tensile test (ASTM D 5617, prEN 14151) performed on geomembrane specimens after exposure to anticipated field conditions. 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 in the case of landfills, over a long term (Bouazza *et al.* 2002). A value of 6% for elongation of a HDPE geomembrane was described as the geomembrane limit that the Quo Vadis working group agreed upon in the early nineties of the last century in Germany (Gallagher *et al.* 1999). A factor of safety of 2 was applied to the 6% strain value to give 3% total permissible strain. The Quo Vadis group are then said to have calculated the strain value due to subsoil settlement to be 2.75%, leaving 0.25% strain as the limiting value for local deformations, e.g. deformations caused by point loads from drainage aggregate. This has not been adequately proven. Justifications for the Quo Vadis value of 0.25% appeared some years after its introduction, but cannot be described as rigorous in their derivation. This point was made by Gartung (1994) who wrote that "On the basis of judgement it was concluded (…) that the (geomembrane) showing local strains of less than or equal to 0.25% would not experience any reduction in predicted lifetime (…). Since there was no better basis for a criterion to evaluate the efficiency of the geosynthetic protection layer, the limiting value of 0.25% local strain was temporarily adopted by the German regulators."

Typically, the performance criteria will be based on protection of the geomembrane liner from puncture, since the geomembrane liner is often the most critical component to solution containment. Narejo (1995) defined three levels of protection for geomembranes against puncture under typical loading conditions:

- Level I is typically applied to liner systems for HW facilities. This level requires that the liner system be designed such that less than 0.25 % strain occurs in the geomembrane liner from the imposed loading;
- Level II is for non-hazardous waste facilities, whereby the liner system is allowed to have geomembrane strains greater than 0.25%, but not result in yielding of an HDPE geomembrane liner (typically around 13%);
- Level III protection is defined for non-critical applications, where yielding of an HDPE geomembrane liner may occur, but does not puncture.

In order to assess the suitability and ability of a proposed protection layer to meet a performance criterion 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 (Bouazza *et al.* 2002). This paper will concentrate only on large scale performance tests or field tests since they have been widely used in recent years. An insight in a comparison on European protection efficiency tests was presented in Beckonert *et al.* (2002). A number of decimetric scale performance tests can be found in Artières & Duquennoi (1996), Gallagher *et al.* (1999) and Lambert *et al.* (2002) for example. First a presentation of experiments dedicated to the prevention of short-term puncture of geomembranes will be given, followed by presentation of studied aiming at ensuring the long term performance of the geomembrane.

Evaluation of protection on the short-term

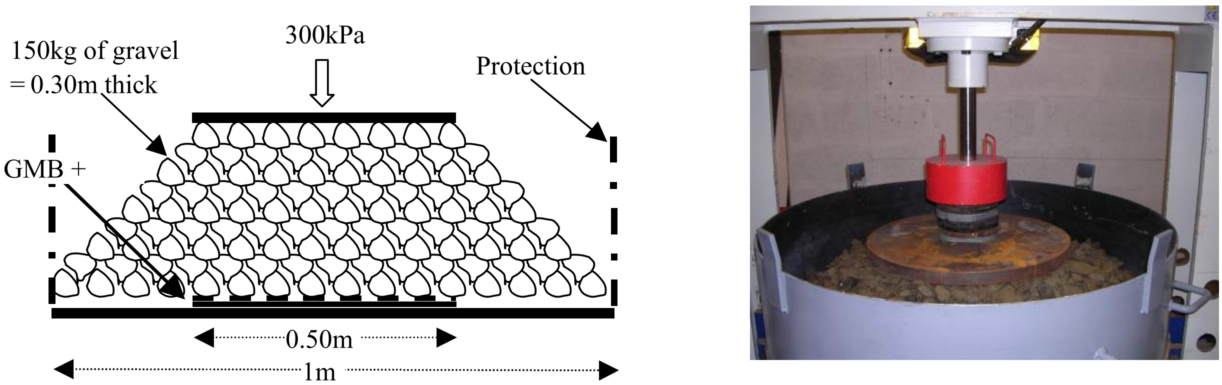
Large scale laboratory tests for puncture protection

Reddy & Saichek (1998) conducted large-scale laboratory tests to evaluate the protection of a 1.5mm HDPE geomembrane liner under long-term MSW loading conditions. Five different 0.3m thick granular soil layers that range from a coarse gravel to a medium sand were used in the testing program, with and without the presence of a 270g/m^2 nonwoven geotextile. The protective cover system and the geomembrane liner were subjected to incremental loading to a maximum pressure of 1.4MPa in a test box 508mm (width)×457mm (height)×914mm (length). A steel plate was placed on top of the protective cover system inside the test box for a uniform load application. A 13mm thick elastomer was selected to simulate a typical CCL. The loading for all of the tests was applied in increments of 205kPa every 30 minutes until the maximum pressure of 1.4MPa was attained in order to simulate long-term waste loading conditions. This maximum pressure was then maintained constant for 48 hours to allow for deformation and creep of the geomembrane specimen to take place. After 48 hours, the load was then incrementally decreased at the same rate as during application. The effect of long-term loading on the characteristics of the cover soils was assessed by performing particle size analyses, and the physical damage that occurred to the geomembrane liner was visually assessed in addition to performing multi-axial tension, wide strip tension, and water vapor transmission tests. The test results revealed that the degree of geomembrane liner protection decreases as the soil mean particle size increases and as the soil particle sphericity decreases. When a geotextile was combined with the protective cover soil layer, a significant increase in geomembrane liner protection was observed for all of the soils tested. This study demonstrated that a 0.3m thick granular soil layer consisting of particles with a mean size less than 30mm and a sphericity greater than 0.8, combined with a 270g/m^2 nonwoven geotextile provides adequate protection against installation damage for a 1.5mm thick HDPE geomembrane liner from long term MSW loading.

Khay *et al.* (2006) presented the results of a laboratory study on forty test pads aiming at quantifying the impact of the nature of the subgrade, the nature of the geomembrane, the mass per unit area of the protective geotextile and the type of granular material (see Figure 34). The subgrade was either a granular material 20-40mm or a compacted loamy soil. HDPE, PP and PVC geomembranes 1.5mm thick were used. Masses per unit area of the geotextiles were equal to 300, 500 and 700 g/m^2. In case the subgrade was a granular material, a geotextile was used underneath and on top of the geomembrane. Two types of granular materials, crushed and rolled, were used. A laser profilometer was used to quantify the geomembrane deformation and burst tests performed according to prEN14151 were the way selected to quantify the effectiveness of the geomembrane liner protection. After placement of the granular layer on top of the cover 50 passes were applied with the empty mover (9.5 tons) and 50 additional passes with the mover full (12 tons). In such conditions, even the 700 g/m² nonwoven geotextile was generally insufficient in protecting the geomembrane liner against damage during installation.

Figure 34. Installation of testing pads (from Khay *et al.* 2006)

Budka *et al.* (2007) also presented the principle of laboratory studies of static damages under load effects performed in 1m diameter cells for 100 to 1000 hours durations (see Figure 35). The tests principle is based on EN 13719 and on the work by Reddy & Saichek (1998). The testing procedure also included burst test after static loading of the geomembrane according to prEN14151. Results tended to show that on a rigid support, a 100 hours duration test is sufficient to obtain a representative simulation of the services load. Tests are ongoing on a soft support (elastomer).



(*a*) (*b*)

Figure 35. (*a*) Schematic drawing and (*b*) photograph of an ongoing laboratory test (from Budka *et al.* 2007, reproduced with the permission from SITA France)

Field scale tests for installation damage simulation

Field-scale tests have also been conducted to evaluate methods to protect geomembrane liners. Most of these tests were focused on preventing damage to the geomembrane liner from construction activities.

Reddy *et al.* (1996) conducted field tests using a 1.5mm thick HDPE geomembrane liner, loaded with light dozer (CAT D4 model) and a heavy dozer (CAT D7 model). A uniform 0.3m thick soil layer of medium or fine gravel was used in the tests. The geotextile used was a nonwoven needle punched PP geotextile with a mass per unit area of 270g/m^2. The subgrade consisted of silty clay. Results tended to show that the presence of a geotextile as light as 270g/m^2 in the protective cover system completely protected the geomembrane from construction loading. This study recommended that a geotextile be considered for use in the geomembrane liner protective cover system when gravel and/or heavy construction equipment are used.

Similar field tests were conducted by Crouse *et al.* (1999) on both a GCL and 1.5mm or 2.0mm HDPE geomembrane liners. The field tests also confirmed that a geotextile placed over the geomembrane liner would help reducing damage by construction equipment.

More recently, Budka *et al.* (2007) undertook a field test 18×30m² in which the efficiency of various geotextiles located under a 0.5m thick gravel layer in protecting a 2mm thick HDPE geomembrane was tested (see Figure 36). The HDPE geomembrane was overlying a CCL. Two earth movers (a hydraulic shovel and a dumper-truck) were used to deliver the gravel and apply load on the gravel layer. Twenty shovel passes (ten round turns) and twenty dumper passes were applied. Definitions given in the guide for the realisation and operation of field tests on bottom lining system (CFG 2001) were used to characterise the observed damages on the geomembrane samples. Burst tests (prEN 14151) were used to characterise the sampled geomembranes after testing. A distinction could be made between the various geotextiles tested as regards the efficiency to protect the geomembrane against puncture by rolled and crushed gravel.

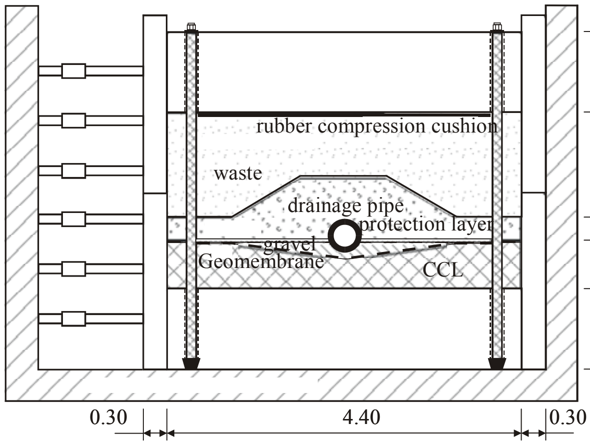
Figure 36. View of crushed gravel installation on testing pads (from Budka *et al.* 2007, reproduced with the permission from SITA France)

Evaluation of puncture protection for long-term performance of geomembranes

Zanzinger (1999) conducted large-scale model tests on 2.5mm thick smooth HDPE geomembrane to assess the performance of different protection systems. They were placed on a 0.6m thick mineral liner of medium plastic clay. A practical sand-bentonite bearing surface was prepared for a HDPE pipe with a typical stress-deformation behaviour of a silty sand. As in practice, it was covered by a geomembrane. The drainage layer was 16 to 32mm gravel with a minimum thickness of 0.3m. Wood shavings and sand were used to simulate waste. A total area of 19m² was used on both sides of the drainage pipe to examine the protection layers. Four protection layers were laid on each side. Five very different and partially very heavy products were tested, of which three (Nos. 2, 4 and 6) may be recognised in Figure 37. 8 products were examined. System No. 1 was a geocomposite with a filling of finely shredded tyres. System No. 2 consisted of a drainage geocomposite, which was tested in double-layer form. Systems Nos. 3 and 4 were HDPE geocontainers differing in the mass of sand filled in. Systems Nos. 5 and 6 consisted of heavy, needle punched nonwoven geotextile that were laid in double or single layers. The identical samples No. 7 and No. 8 consisted of a geomat filled with sand, welded to a geocomposite at regular intervals.

Deformations in the geomembrane in the course of the loading stress test were recorded thanks to soft sheet-metal "organ-pipe" plates that were installed at areas of measurement between the geomembrane and the underlying sand-bentonite layer of the pipe-bearing surface. Stress was increased over 13 phases within nine months, created vertical stress averaging 800 kPa at the measuring points. In the final phase of the compressive stress test, there was a loading stress of up to 1,000 kPa over a period of two weeks. The average strain measured in the large-scale test is slight with figures of up to 0.1%. The maximum strain was, without exception, over the 0.25% limit, but less than 1.0%. The nine-month examination under a load equivalent to a 60m high body of waste did not result in any tears, perforations, grooves or other damage to the geomembrane. After exposure to stress in the large-scale test, the deformed geomembranes showed constant rippled surfaces. The examined geosynthetic protection layers largely compensate for any local peak stress to be expected at the underside of the coarse-grained drainage layer. All protection layers, examined in this program, have shown satisfactory efficiency.

(*a*)



(*b*)

Figure 37. (*a*) Covering of protection layers, and (*b*) Cross-section of a large scale laboratory test, (numbers given in metres) (Adapted from Zanzinger 1999)

Tognon *et al.* (2000) conducted large-scale laboratory tests in a 2m diameter cell to assess the performance of protection layers on geomembrane liners. In their testing program, sand cushions, geotextiles and geomats were tested with a 1.5mm HDPE geomembrane with loads up to 900kPa. The test results indicated that local strains up to 13 % could develop in the geomembrane liner, but the liner did not puncture. The results indicate that the best protection for the underlying geomembrane was provided by a sandfilled geocushion or a special rubber geomat, which limited strains induced by coarse (40-50mm) angular gravel to 0.9% at 900kPa and 1.2% at 600kPa. The poorest performance was achieved using nonwoven geotextiles with a maximum strain of 8% being obtained with a 435g/m^2 geotextile at 250kPa and 13% with two layers of 600g/m^2 geotextile at 900kPa. Although there was adequate protection to avoid holes in the geomembrane in the short term, the short-term strain of 13% is dangerously close to yield and it is not clear what the long-term performance would be with these large locally induced strains. The results of the large-scale biaxial tests indicated that, for the conditions examined, none of the protection layers tested could limit the maximum actual total strain to <0.25% as required by German regulations. It is clear based on those results that the sand cushion and grid reinforced rubber mat provided far superior protection and were likely to give smaller short- and long-term strains than the nonwoven geotextile layers examined.

To illustrate the effectiveness of various protection layers, data from Dickinson & Brachman (2008) is summarized in Table 5 for one particular geomembrane/GCL composite liner installed beneath 50mm coarse gravel and subjected to an overburden pressure of 250kPa. The 150mm thick layer of sand was the most effective at limiting the geomembrane strain. Given that sand may not be practical in certain cases, 150mm thick layer of compacted silty-clay and 150mm thick layer of rubber tire shreds with a nonwoven needle punched geotextile (570 g/m^2) were also found to limit the geomembrane strains to acceptable levels. The results in Table 5 show that all three nonwoven needle punched geotextiles (GT1 and GT3) had geomembrane strains that were much smaller than with no protection. Increasing the mass (and hence thickness) of these geotextiles resulted in a decrease in the peak strain. However, none of the single geotextile layers tested at 250kPa were able to limit the tensile strains to less than 3%. The nonwoven needle punched geotextiles tested did provided cushioning (i.e. an increase in contact area between gravel particle and the geotextile as the thickness of the geotextile is reduced) but did not have adequate stiffness to further limit the tensile strains in the geomembrane because of initial slack in their force-deflection behaviour. This illustrates the challenge of using geotextiles to attempt to limit geomembrane strains. Since there is inherently going to be some degree of slack in the geotextile, by the time they have deformed enough to overcome this slack and start carrying some tensile force, the underlying geomembrane and GCL has already deformed. The thinner heatbonded geotextile GT5, which was much stiffer than the nonwoven geotextiles at small displacement, was ineffective by itself at reducing geomembrane strains as a result of the minimal cushioning it provided. However, a layered geocomposite with a thick nonwoven needle punched geotextile (GT3) in the middle to provide cushioning and stiffer heatbonded geotextiles (GT5) on the top and bottom to carry tensile force was able to limit the short term strain to less than 3%. Additional strain from geotextile creep and/or softening at higher temperatures may be expected to result in geomembrane strains greater than those reported in Table 5 as quantified by Brachman *et al.* (2008).

Table 5. Calculated geomembrane tensile strain and measured minimum GCL thickness for geomembrane/GCL composite liner beneath 50-mm coarse gravel at 250 kPa for 10 hr. No geomembrane wrinkle (from Dickinson & Brachman 2008).

Protection layer	Peak geomembrane* tensile strain (%)	Minimum GCL§ thickness (mm)
150mm thick sand†	0.1	5.2
150mm thick silty-clay‡	0.4	4.4
150mm thick tire shreds ❄ + GT1	2.8	4.3
GT5 + GT3 + GT5	2.6	4.6
2240 g/m^2 nonwoven GT3	5.1	4.0
570 g/m^2 nonwoven GT1	11	4.2
270 g/m^2 heatbonded GT5	15	3.9
None	19	3.5

*1.5mm thick HDPE; yield elongation strain = 20%; 2% secant modulus = 270 MPa.
§4,720g/m^2 sodium bentonite; 110 g/m^2 slit-film woven carrier geotextile; 240 g/m^2 virgin staple fibre nonwoven cover geotextile; needle punched; needle punched fibres thermally fused to carrier geotextile; initial water content = 120-150%; initial thickness = 7-9 mm.
†Poorly-graded medium sand; placed without compaction.
‡Compacted at Standard Proctor Maximum Dry Density.
❄Shredded rubber tires; particle gradation between 7mm and 27mm.

GCL indentations

Local thinning of a GCL from gravel directly on top of the GCL (i.e. without a geomembrane) was observed by Fox *et al.* (2000) and Shan & Chen (2003). The thickness of a GCL may also be affected by the presence of a wrinkle in the overlying geomembrane because of extrusion of bentonite in the GCL towards the lower stress region beneath the wrinkle (Anderson & Allen 1995, Stark 1998).

The change in thickness of a GCL when installed beneath a geomembrane with a wrinkle and coarse gravel that was subjected to applied pressures of 250 and 1000 kPa has been quantified by Dickinson & Brachman (2006, 2008). They showed that without protection or with ineffective protection (including a very heavy nonwoven needle punched geotextile with a mass of 2,240g/m^2) there was extrusion of bentonite beneath gravel contacts leading to local thin zones in the GCL. However, a 150mm thick sand protection layer was sufficient to prevent extrusion of bentonite in the GCL (up to applied pressures of 1,000kPa). With sand protection, the GCL experienced a beneficial decrease in thickness (and consequently void ratio) of the GCL by consolidation. The minimum measured GCL thickness (with no geomembrane wrinkle) is reported in Table 5 for various protection layers.

Specificities for wrinkled geomembranes

Geomembrane wrinkles that form during installation also influence the physical conditions experienced by the liner. The wrinkle causes a redistribution of vertical stresses acting on the liner with zero stresses directly beneath the wrinkle and increased stresses on both sides of the wrinkle. Gudina & Brachman (2006) showed peak tensile geomembrane strains from gravel particles are even larger when the geomembrane was wrinkled. Dickinson & Brachman (2008) showed local GCL indentations are also made worse if there is a wrinkle in the geomembrane. Thus the impact of wrinkles should be considered in the assessment of geomembrane and GCL protection unless special efforts are employed during installation to prevent wrinkles.

Soong & Koerner (1998) measured geomembrane wrinkle deformations for the specific case with sand above and below the geomembrane. Their results showed that although the wrinkle experienced a decrease in height and width, the gap beneath the wrinkle remained even when subjected to a pressure of up to 1,100 kPa and a test duration of 1,000 hours. When compacted clay was tested beneath the geomembrane, observed a different response where the gap beneath the wrinkle was completely filled with clay, depending on the applied pressure and the clay water content. For the case of a wrinkle in a geomembrane/GCL liner, Dickinson & Brachman (2006) found that the gap beneath the wrinkle was reduced, but remained, when the GCL was underlain by a firm sand foundation layer (**Figure 38**) but eliminated with a soft clay foundation layer. For both geomembrane/CCL and geomembrane/GCL composite liners, the deformed shape of the wrinkle depended on the type of protection layer, the applied vertical pressure, and the water content of the CCL or GCL. It was found that tensile strains in the geomembrane could also result from deformation of the wrinkle. However, these tensile strains were smaller than the local strains from gravel indentations;

consequently, the local indentations would govern in the assessment of overall tensile strain in the geomembrane. In the case of the sand protection layer, no tension was calculated from the wrinkle deformations because of greater wrinkle deformations.

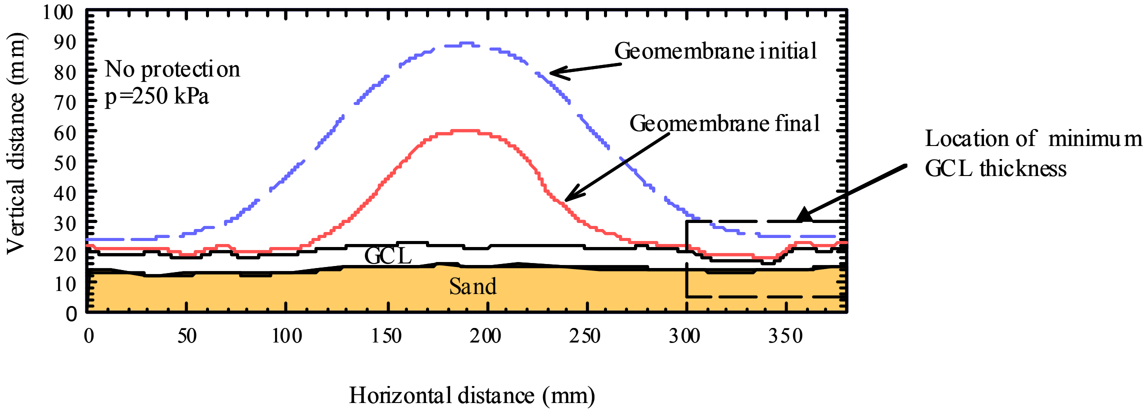


Figure 38. Deformed shape of geomembrane wrinkle and GCL (Scale 2V:1H). Modified from Dickinson & Brachman (2006).

Conclusion

Various tests addressing either damage during installation and protection efficiency were presented. Through the presentation of those tests the question arises of the impact of allowing any stress on the geomembrane as long as it it will not be punctured. To ensure long-term performance of the geomemnrane liner any constant stresses in the material should be avoided, as far as possible.

Despite numerous experimental tests it is difficult to draw general recommendations regarding puncture protection that will ensure protection in any case. One can also notice the quasi absence of modelling approaches of puncture phenomena. Nevertheless results obtained by various authors suggest that a nonwoven needle punched geotextile selected solely to prevent puncture is generally insufficient to limit tensile geomembrane strains to allowable levels (Brachman & Gudina 2008*b*, Gudina & Brachman 2006, Tognon *et al.* 2000). Quantifying long-term tensions in geomembranes is challenging and currently underway (Brachman *et al.* 2008); however, at present, only an estimate of short-term tensions can be obtained from large-scale laboratory tests. The available data suggests that a 0.15m thick sand protection layer (Brachman & Gudina 2008*b*, Gudina & Brachman 2006) or a sand filled geocomposite (Tognon *et al.* 2000) is required to limit the geomembrane strains below the 3% limit of Seeger & Müller (2003).

GEOSYNTHETICS QUALITY ASSURANCE

Introduction

Variability in geosynthetics properties may occur as a result of manufacturing or as a result from damage incurred during installation. It is thus of the primary importance to properly follow both the manufacturing and installation processes. The focus of this section, after a brief description of quality assurance and quality control for manufacturing and construction will be on damage to geomembrane liners during installtion. This point will emphasize the need for strict quality assurance in the installation of geosynthetics that will be further discussed. Finally, this section provides a discussion on the suggested minimum scope for surveillance during installation of geomembranes.

Definitions of quality assurance/quality control for manufacturing and construction

Quality Assurance (QA) and Quality Control (QC) in the manufacture, design, and installation of geosynthetic materials play an important role in the overall performance of engineered structures which utilize geosynthetics. QA and QC procedures provide the means to quantify manufacture and installation quality of geosynthetics, which ultimately affects the performance of the material. Before presenting a discussion on QA programs in geosynthetics, it is worth briefly discussing the difference and role of QA and QC programs in the manufacturing and installation of geosynthetics, using the definitions presented by the Geosynthetic Research Institute (GRI) i.e. GRI GM13 (GRI 2006) and GRI GCL3 (GRI 2005).

Manufacturing Quality Control (MQC) is a planned system of inspections that is used to directly monitor and control the manufacture of a material which is factory originated. MQC is normally performed by the manufacturer of geosynthetic materials and is necessary to ensure minimum (or maximum) specified values in the manufactured product. MQC refers to measures taken by the manufacturer to determine compliance with the requirements for materials and workmanship as stated in certification documents and contract specifications.

Manufacturing Quality Assurance (MQA) is a planned system of activities that provides assurance that the materials were constructed as specified in the certification documents and contract specifications. MQA includes manufacturing facility inspections, verifications, audits and evaluation of the raw materials (resins and additives) and geosynthetic products to assess the quality of the manufactured materials. MQA refers to measures taken by the MQA organization to determine if the manufacturer is in compliance with the product certification and contract specifications for the project.

Construction Quality Control (CQC) is a planned system of inspections that are used to directly monitor and control the quality of a construction project. CQC is normally performed by the geosynthetics manufacturer or installer. It is necessary to achieve quality in the constructed or installed system. CQC refers to measures taken by the installer or contractor to determine compliance with the requirements for materials and workmanship as stated in the plans and specifications for the project.

Construction Quality Assurance (CQA) is a planned system of activities that provide assurance that the facility was constructed as specified in the design. CQA includes inspections, checks, audits and evaluations of materials and workmanship necessary to determine and document the quality of the constructed facility. CQA refers to measures taken to assess if the installer or contractor is in compliance with the plans and specifications for a project.

As noted, QC programs for geosynthetics generally rely on the manufacturer and/or installer, while QA programs may be conducted by the owner, designer, or an independent third-party. It is important to recognize and understand the difference between QA and QC programs for both construction and manufacturing as each has its own intent and requirements.

Manufacturing QA/QC

Designers rely on geosynthetic materials to perform based on their published physical, mechanical, and hydraulic properties. However, variations in their properties may occur during manufacturing. Since geosynthetic materials are a manufactured product, QA and QC programs during manufacturing are common as they demonstrate that the end-product meets the advertised specification.

USA and Europe follow different approaches. Koerner & Daniel (1993), Landreth (1993), Sieracke (2005), White & Allen (1993) provide an extensive discussion on the role and intent of QA/QC programs during manufacturing in the USA. In Europe, factory production control undertaken by manufacturers is monitored by a notified body (inspection body) in the context of the CE marking. To understand the CE marking and the correspondent implications, it is necessary to be aware of the European Construction Products Directive (CPD), which aims at removing technical barriers to trade within the European Economic Area.

In the context of the CPD, several Harmonized Technical Specifications have been developed, for specific applications and functions of the geosynthetics. These standards indicate the Attestation System that the manufacturer must adopt, in consequence of the CPD, to attest the conformity of the geosynthetics, according to the function that they can play in the different applications. For example EN 13493 describes the characteristics required for the use of geosynthetic barriers, i.e. geomembrane liners and GCLs, in the construction of solid waste storage and disposal sites.

To characterize the geosynthetics, many test methods and specifications have been developed by several organizations all over the word, including the Association Française de Normalisation (AFNOR), American Society for Testing Materials (ASTM), German Federal Institute for Materials Research and Testing (BAM), International Organization of Standards (ISO), Canadian General Standards Board (CSGB), European Committee for Standardizations (CEN) and Geosynthetic Research Institute (GRI). These standards are generally used within the industry to provide “bench mark” specifications for geosynthetic materials. These standards are important as they allow designers to specify geosynthetic materials. In France the ASQUAL (Association for Quality) is the certifier responsible for application of technical rules, organization of all the controls and quality certificates based on independent laboratories controls of measured properties both for geotextiles and geomembranes. The Asqual issues certificates for certified geotextiles and geomembranes, when requested by the manufacturer on a voluntary basis. Testing is performed by independent testing laboratories which have an accreditation.

Construction QA/QC

Variability in geosynthetics properties may occur as a result from damage incurred during installation. Traditionally, QA/QC programs are used to monitor the installation of geosynthetic materials so that installation damage may be minimized. In the design and installation of geosynthetic materials, QA/QC programs tend to be less standardized and may vary significantly and/or do not include requirements to assure long-term performance (Adams *et al.* 2001, Dobras & Yacko 1989, Donckers 1994, Fluet 1987, 1990, Hebert 1987, Monteleone & DiPippo 1989, Peggs 1995, 1997). QA/QC programs for geosynthetics tend to be linked to specific requirements (if applicable) by a governing agency for a particular application. However, throughout the world, there is no concensus on a standardized QA/QC program for applications involving geosynthetics. Indeed, in many countries there are no specific provisions for geosynthetic materials. As a result, QA/QC programs tend to be developed on a per-project basis, which may or may not be suitable for the given application.With the variability in standards, there is a possibility that geosynthetic materials may be improperly specified, tested incorrectly or infrequently, designed with incorrect parameters, installed improperly, handled improperly, or installed with poor documentation. This generally occurs when the role and/or benefit of QA/QC programs is not well understood or is not fully appreciated (Richardson & Bove 1993, Smith *et al.* 1996, Wallace 1989). The failure to implement a proper QA/QC program during design and installation may result in poor performance or failure of the geosynthetic material.

QA/QC programs during the design process are mainly left to in-house programs within the design engineering firms. These programs generally involve a series of internal/peer reviews by qualified persons and may also involve a third-party review by an external person or firm. The intent of the design QA/QC program is to determine if the design is based on sound engineering principles and that geosynthetic materials are being correctly integrated into the design using appropriate parameters.

Experiences in many geosynthetics failures, in many geosynthetics applications reveal that the majority of failures are due to inappropriate designs and inappropriate selection of materials by the design engineer (Peggs *et al.* 2003). Nevertheless, geosynthetic QA/QC programs during construction are as important as design and manufacturing QA/QC programs (Koerner & Daniel 1993, Mollard *et al*. 1996, Ramsey 2005). A properly designed facility can still perform poorly or even fail if the facility is not constructed properly or if it is damaged during construction (Darilek & Laine 2001, Shepherd *et al.* 1993).

Damage to geomembrane liners

Damage to geomembrane liners during installation has been the subject of several studies thanks to electrical methods, including those presented by Allen & Bathurst (1994), Bräu (1996), Crouse *et al.* (1999), Darilek *et al.* (1989), Giroud & Bonaparte (1989) and Reddy *et al.* (1996). These studies have generally shown that damage may occur as a result of falling objects (stones, tools, etc.), concentrated stress (e.g. stone under a liner), or being hit by equipment. Statistics on liner damage origin are presented in Colucci & Lavagnolo (1995), Nosko *et al.* (1995), Nosko & Touze-Foltz (2000) and Rollin *et al.* (1999) among others and summarized in Table 6.

As shown in Table 6 and discussed in Peggs (2003), the majority of liner damage is man-induced during construction. When construction-related damage is removed from the statistics, failures associated with stress-cracking and/or oxidation dominates. In addition to these statistics, Allen & Bathurst (1994) have shown that damaged geosynthetics may have a different load-strain (modulus) behavior than undamaged geosynthetics, which could impact performance and lead to premature failure.

Table 6. Statistics of Liner Damage (after Peggs 2003)

Activity/Location	Percent of Total	Details	Individual Percentage
Liner installation	24%	Extrusion	61%
		Melting	18%
		Stone Puncture	17%
		Cuts	4%
Covering	73%	Stone Punctures	68%
		Heavy Equipment	16%
		Grade Stakes	16%
Post-construction	2%	Heavy Equipment	67%
		Construction	31%
		Weather, etc.	2%
Flat floor	78%	Stones	81%
		Heavy Equipment	13%
Corners/edges	9%	Stones	59%
		Heavy Equipment	19%
		Welds	18%
Under pipes	4%	Stones	30%
		Welds	27%
		Heavy Equipment	14%
		Worker	15%
		Cuts	14%
Pipe penetrations	2%	Welds	91%
		Workers	8%
		Cuts	1%
Shipping, storage, etc.	7%	Heavy Equipment	43%
		Stones	21%
		Workers	19%
		Welds	17%

Another issue related with this topic is the defect density per liner area, i.e. number of defects per hectare (Colucci & Lavagnolo 1995). The variation of defect density as a function of the area of the facility surveyed is plotted in Figure 39. It can be observed that the density of defects tends to decrease as the surveyed area increases. However, it must be noted that there are many uncertainties regarding the varying conditions found in different sites (different types of geomembranes, different facilities, covered and uncovered geomembranes, etc). According to Colucci & Lavagnolo (1995), the reasons for the higher defects densities found in small installations may be due to the fact that:

- Smaller facilities have proportionally more complex features (corners, sumps, penetration);
- Small facilities tend to have higher percentage of hand seaming (extrusions);
- Large facilities have a stricter construction quality program; and

- Large installations generally receive less traffic per unit area installed.

It should be noted that relatively higher defect densities can be found on small containment facilities with complex features and where the geomembranes have been placed directly on poorly prepared subgrade soil. For example, Laine &Miklas (1989) reported a mean density of 26defects/ha from surveys conducted on containment facilities less than 2 ha. Similar observations have been drawn by other authors, such as Rollin *et al.* (1999, 2004).

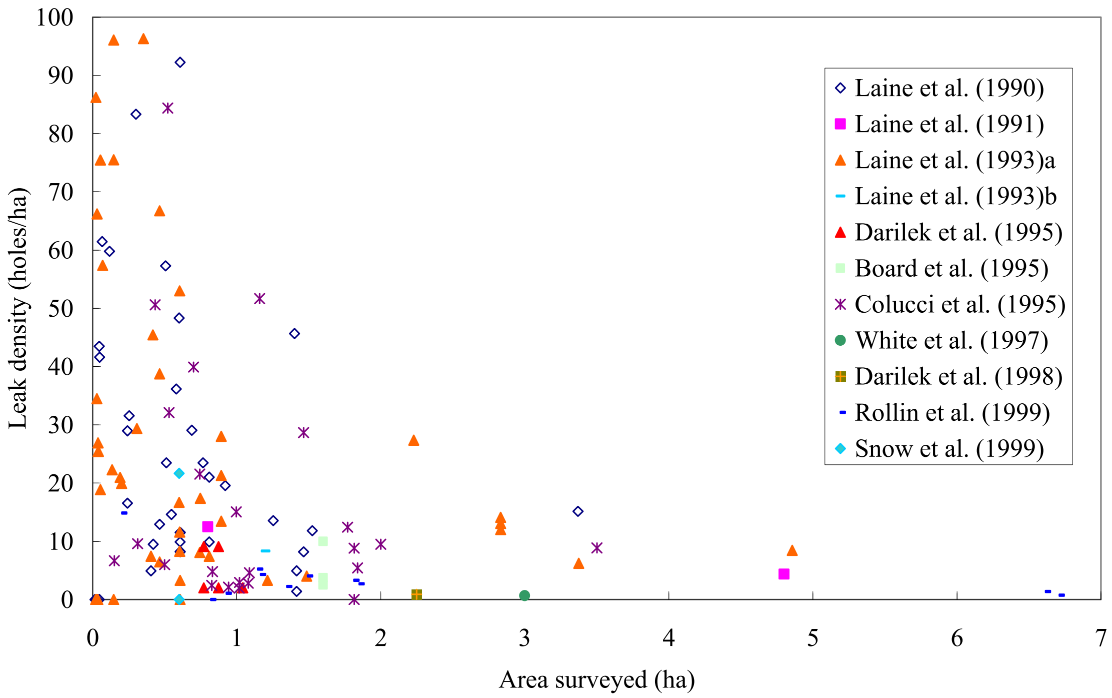


Figure 39. Variation of defect density as a function of the area surveyed (from Touze-Foltz 2001)

Table 7 shows defect densities reported by different authors for covered geomembranes. It can be seen that they range from 0.7 to 15.3defects/ha. Similar results were reported by Forget *et al.* (2005). These authors summarizing 10 years of leak detection surveys on geomembranes, found a mean defect density of 0.5defects/ha for covered geomembranes installed under a strict CQA program compared to a mean defect density of 16defects/ha in absence of a CQA program. In only two cases the maximum holes density was located in seams.

Table 7 includes the mean values obtained by Touze-Foltz (2001) from a synthesis of studies involving electrical leak location systems. Ths author reports a mean defect density of 2.8 defects/ha after installation of the geomembrane and 11.9 defects/ha after placement of the granular drainage layer. This result confirms that the majority of the defects occur during placement of the granular layer above the geomembrane.

Table 7. Reported defect density (modified from Touze-Foltz 2001)

Reference	Area surveyed (ha)	Status of geomembrane	Defects on geomembrane sheet (%)	Defects on geomembrane seams (%)	Mean defect density (defect/ha)
Laine & Mosley (1993)	1	Covered	20	80	8.3
Board & Laine (1995)	2	Covered	31	69	5.5
Colucci & Lavagnolo (1995)	25	Covered	85	15	15.3
White & Barker (1997)	1	Covered	100	0	0.7
Darilek & Miller (1998)	1	Covered	100	0	0.9
Snow *et al.* (1999)	2	Covered	100	0	10.9
Nosko & Touze-Foltz (2000)	325	Covered	93.7	6.3	12.9
Touze-Foltz (2001)	108.8	Covered	81.5	18.5	11.9

This defects density seems to be in agreement with the values previously recommended by Giroud & Bonaparte (1989) for design calculations. According to these authors a frequency of 12defects/ha should be considered to be representative for typical installation values. The US EPA Help program assumes a defect density of 2.5defects/ha for "excellent" installation quality and from 2.5 to 10defects/ha for "good" installation quality (Schroeder *et al.* 1994). It is also in agreement with the following conclusions made by Giroud & Touze-Foltz (2003) based on a discussion with experts regarding the number of holes in geomembrane liners:

- The number of holes at the end of geomembrane installation with construction quality assurance is typically believed to be from 1 to 5defects/ha;

- These defects are generally small holes and the number of defects decreases as the lined area increases (e.g. greater than two hectares);
- The number of defects caused by the placement of soil on top of the geomembrane varies in a wide range, from very few to 20 defects/ha, depending on the amount of care taken during placement of soil on top of the geomembrane and the type of geomembrane protection used; and
- These defects can be large (and often are). Table 8 presents data reported by different authors about type and size of defects. These data are consistent with data reported by Needham *et al.* (2004).

Another interesting aspect is the lack of development of defects occurring after the installation period. Leakage results from a permanent in situ leak detection system (grid system) at a landfill in the UK were reported by Needham *et al.* (2004). The leak detection system has been in operation since 1989 and covers a lined area of 5.5ha. Data show that there is no evidence of gradual development of holes from 1995 to 2003 at the end of the study. Based on these results, the authors concluded that once a liner is covered by several meters of waste, the mechanisms for future development of holes in the installed liner (e.g. stress cracking and puncturing under service stresses) are limited and they are unlikely to develop for at least the first decade of the service life of the geomembrane.

Table 8. Defect size as a function of defect type

Reference	Size	Holes	Tears/burns/ equipment	Cuts/ scraps/ gouges	Seams	Sites	Area surveyed (ha)
Colucci & Lavagnolo (1995)	0-0.2 cm^2	44	31	12	11	25	27.6
	0.2-1 cm^2	37	49	21	4		
	1-5 cm^2	60	49	2	8		
	5-10 cm^2	22	11	0	4		
	10-100 cm^2	10	22	0	1		
	100-1000 cm^2	15	4	0	0		
	1000-8400 cm^2	0	5	0	0		
Rollin *et al.* (1999)	<0.02 cm^2	3	-	0	18	11	24.1
	0.02-0.1 cm^2	6	-	4	7		
	> 0.1 cm^2	3	-	6	2		
Nosko & Touze-Foltz (2000)	< 0.5 cm^2	332		5	115	300	325
	0.5-2 cm^2	1720	236	36	105		
	2-10 cm^2	843	153	18	30		
	> 10 cm^2	90	496	-	15		
Peggs (2001)	< 0.1 cm	10	0	4	2	1	63.4
	0.2-1 cm	28	9	7	5		
	1-5 cm	7	2	21	3		
	5-10 cm	0	1	5	3		
	10-50 cm	1	0	2	1		
	50-100 cm	0	0	0	3		
	> 100 cm	0	0	2	2		
	unknown	4	1	5	3		

The role of QA/QC programs during construction is to minimize damage to geosynthetics and to ensure that the design specifications and the intent of the design are being met in the field (Peggs 1995, Wallace 1989). Given the variability and complexity of construction projects, it is difficult to define a single standard for construction QA and QC that would work for all projects (Adams *et al.* 2001). For example, a QA/QC program for the construction of a geosynthetic-lined industrial waste facility would differ from that used for the construction of a lined fresh water pond. However, it is possible to define minimum standards that can be used as the foundation for developing project-specific QA/QC programs. Insights regarding this approach are highlighted by Adams *et al.* (2001) and Peggs (1997).

The following section primarily focuses on the role of QA in the installation of geosynthetics, and mainly of geomembranes during construction, and provides a discussion on the suggested minimum scope for surveillance during installation.

Construction quality assurance plan

As data presented in previous section suggest, the mean defect density is much lower in geomembranes installed under strict CQA programs compared than in sites without CQA programs. Thus, the implementation of CQA programe in facilities that incorporate geosynthetics is of the upmost importance.

The primary tool in any CQA program is the CQA Plan or Manual (Cieslik 1988, Peggs 1995, 1997). The CQA Plan is a formal document that is tailored toward each specific project and, at a minimum, should include the following:

- Definitions to be used throughout the project to avoid confusion on acronyms and wording; this is important as acronyms can vary from project-to-project and from one country to another;
- Descriptions of responsibilities, qualifications, and obligations for each party involved in the CQA Plan (owner, project manager, designer, lead CQA representatives, geosynthetic manufacturer, geosynthetic installer, earthworks contractor, etc.);
- The lines of communication and authority for the project; in addition, it is important to define the process for addressing Request for Information (RFIs), Design Modifications (DMs), or changes in the project specifications;
- A formal process on handling deficiencies which defines responsibilities and the minimum documentation required to correct deficiencies;
- A project meeting schedule (location, time, frequency, required participants, etc); and
- A clear definition and scope of the Geosynthetic Quality Assurance Program, namely (Fluet 1990, Landreth 1993, Peggs 1995, Phaneuf 1993, Sasse & Saathoff 1996):
  - ✓ A requirement to review QC submittals by the geosynthetics installer; this is done to ensure the installers are meeting the minimum requirements for their QC programs and that the geosynthetic materials meet the project specifications; results from MQA and MQC should also be presented either by the geosynthetics installer or the geosynthetics supplier;
  - ✓ A definition of the scope for conformance testing to be conducted during the program; conformance tests are used to ensure the materials meet the project specifications; the scope should include: (1) the definition of who is responsible for collecting the test samples; (2) sample identification method; (3) the test method(s) to be used (specify method, including the organization like CEN, mandatory in EU, ASTM, ISO, etc., as there are differences in test methods between each organization); (4) the parameters to be tested; and (5) the frequency of testing; it is important to note the conformance testing will vary from project to project, based on the design and size of project;
  - ✓ A definition of the geosynthetic monitoring and testing to be completed during construction/installation;
  - ✓ Observations and monitoring to be conducted during geosynthetic deployment, seaming or joining (if applicable), destructive and non-destructive testing methods and frequency (if applicable), geosynthetic repairs, and geosynthetic protection after deployment; and
  - ✓ A definition of the minimum documentation requirements; these should include: (1) daily record-keeping, (2) documentation of construction problem resolutions, (3) documentation of design and specification changes, (4) photographic records, (5) weekly progress reports, and (6) chain of custody forms for test sample tracking.

An integral component of any geosynthetics CQA program is proper monitoring and documentation (surveillance) prior to delivery and after delivery, during installation, and after installation.

Prior to delivery

Prior to the delivery to the project site, it is important that certain data sheets and certifications be provided for each geosynthetic material to be used on the project. At a minimum, these submittals should include the manufacturer's certifications and quality control test results for the material to be delivered. In the context of the CE marking, for each type of product, the manufacturer should also supply a declaration of conformity and an accompanying document (which includes the mean values and tolerances for regulated characteristics of the product).

For geomembranes, if required, the manufacturer also has to provide information that verifies the resin, geomembrane rolls, and welding beads/rod to be used on the project conform to the project requirements. It is important to note that some projects have strict conditions on resin quality and additives for geomembrane liners. Ideally, these submittals should be provided prior delivery to the site so that only materials that meet the project specifications are delivered.

For needle punched GCLs the manufacturer should provide certification that the geotextile has been inspected continuously using permanent on-line full-width metal detectors and contains no needles that could damage other geosynthetic layers.

Material delivery on site

When the geosynthetic materials are delivered to the project site, all materials should be checked for the manufacture's name, name and type of geosynthetic, lot number (USA), roll number, roll dimensions (length, width, gross weight), mass per unit area, predominant raw material and CE marking (Europe). In case of ASQUAL certified products, all rolls should exhibit a label. In addition, documents supplied by manufacturer should be compared to the shipping manifest. It is important to verify that the delivered material agrees with the shipping manifest and project specifications. Geosynthetics without any labelling should be set aside and reported to owner or project manager. Materials without proper labels should not be deployed until they have been verified to meet the project specifications.

Upon arrival, each roll of material should be visually inspected for damage to the plastic cover and/or geosynthetic (punctures, rips, grease, etc). Damaged cover or geosynthetic rolls should be documented and reported to the owner or project manager. Depending on the project, the outer wrap is/should be rejected, or damaged geosynthetic may be returned to the manufacturer.

Material Storage

All geosynthetics should be handled and stored in accordance with the manufacturer's recommendations. Figure 40 illustrates good and poor storage conditions. Materials should be stored in an area where it will be protected from construction activity and away from excessive sunlight, heat, cold, moisture, and dirt or grease. For the particular case of GCLs, given the nature of the bentonite, it is particularly important that these materials be stored in an area where the rolls will not be exposed to moisture and kept dry until deployment.

(*a*)

(*b*)

Figure 40. Photographs illustrating (*a*) good and (*b*) bad storage of geosynthetic rolls

Conformance testing

After delivery and unloading, the geosynthetics should be sampled and tested for conformance to the project specifications. Alternatively, many projects require independent sampling of geosynthetic rolls while still in the manufacturing facility and prior to delivery to the construction site. These samples are then shipped to the testing laboratory for conformance testing and compliance assessment before rolls are allowed on site. This may appear to be redundant to the MQA and MQC programs, but conformance testing prior to installation is the final check to ensure the material meets the project specifications. The conformance testing samples should be collected according to the frequency stated in the QA plan. It is recommended the conformance samples be taken across the entire width of the roll and not include the outer wrap of the geosynthetic roll. Each sample should be identified, including: sample number, lot/batch number, roll number, date of sampling, project name, manufacturer, and name of person collecting the sample.

According to the CEN/TR 15019, the frequency should be a function of the importance of the geosynthetic for the safety of the works and the area of product used in the works. For safety standard "high" applications, i.e., applications where long-term strength is a significant parameter and/or where the product plays a decisive role for the safety of the construction and the stability of the works (eurocode 7 class 2) the frequency should be 1 sample every 6,000m^2, with a minimum of one sample above 1000m^2. For safety standard "normal" applications (eurocode 7 class 1) the frequency should be one sample every 10,000m^2, with a minimum of one sample above 1,000m^2.

Common geosynthetic conformance tests are discussed in Qian *et al.* (2001) and Koerner & Daniel (1993). They should include all relevant characteristics included in the project specifications. For geotextiles, conformance tests may include mass per unit area, grab tensile strength, trapezoidal tear strength, burst strength, puncture strength, apparent opening size and permittivity. Geomembrane conformance testing will depend upon the type of polymer and manufacture. For nonreinforced polyolefin geomembranes, conformance testing generally includes thickness, tensile strength and elongation, puncture, tear resistance, density and carbon black content and dispersion. GCL conformance tests include bentonite mass per unit area, bentonite free swell, GCL hydraulic conductivity, direct shear, and peel test for needle punched or stitch bonded GCL. Geopipe conformance tests may include density, carbon black content, compression strength, resin flexural modulus, and resin tensile strength.

Subgrade

Prior to deployment of the geomembrane, the subgrade surface should be visually inspected and tested to see if it meets the project specifications. Surface finish is an important aspect to the performance of geomembranes, geotextiles, and GCLs and will have a direct impact on the performance of the facility. As indicated earlier, defects in geosynthetic materials can occur during installation, and subgrade preparation and finish plays an important role on minimizing defects and damage to geosynthetics.

In general, it is desirable to have a firm and unyielding subgrade that has a relatively smooth surface finish so that the geosynthetic material is in good contact with the subgrade (see Figure 41). The contact between the geosynthetic and subgrade affects interface shear strength, leakage rate through defects in the geomembrane liner, puncture resistance and induced strain. It should be noted that the specifications for the subgrade finish should be tailored to suit the project conditions and geosynthetic materials. Zanzinger & Gartung (2002) provide a general guideline for a subgrade layer finish which indicates that the subgrade layer should have particle sizes that are less than 10mm diameter, deviations from the theoretical plane surface should not exceed 20mm over a distance of 4m and the ruts of the compaction equipment may not be deeper than 5mm. Specifications for subgrade finish will vary and should be developed carefully to meet the project requirements for performance.

If the surface meets specifications, it should be approved in writing by both the CQA monitor and the geosynthetic installer using a Subgrade Approval Form. Anchor trenches should also be inspected and approved in writing.

(*a*) (*b*)

Figure 41. Illustrations of (*a*) poor subgrade quality and (*b*) good subgrade under construction

Geosynthetic Installation

Prior to the deployment of the geosynthetic material, the installer should present details on the installation method and approach. These details should be reviewed and approved prior to deployment. Panel layout drawings should be provided for GCLs, geotextiles and geomembranes. The panel layouts should include penetration details, seam orientation, anchorage details, and seaming methods and materials, including overlapping between the rolls (if it is the case).

During installation, the CQA monitor should observe and document the equipment and method of installation for each geosynthetic material. Documentation should also include environmental factors, such as wind speed, direction, temperature, precipitation over the last 24 hours and cloud cover. General and specific site conditions should also be noted prior to and during installation.

As noted earlier, a significant percentage of geosynthetic damage occurs during installation, either during deployment or placement of cover materials. Care must be taken to document deployment and cover placement activities. If damage does occur to the geosynthetic material, all repair work should be inspected and verified to meet the requirements of the project specifications.

Geomembrane Inspection

Since geomembrane liners play a significant role in the use of geosynthetics for civil and industrial uses, this section is devoted solely to QA inspection requirements for geomembrane deployment and seaming.

During the deployment of geomembranes, each panel should be visually inspected for damage (e.g., holes, blisters, grease) after placement and prior to seaming. Any defects or damaged liner areas should be marked on the panel and on the panel layout. Areas of defects or damage should be documented in the daily report, indicating the location of damage, type of damage, repairs that have been performed, and panels that have been rejected. Figure 42 shows CQA monitors inspecting geomembrane panels before seaming.

(*a*) (*b*)

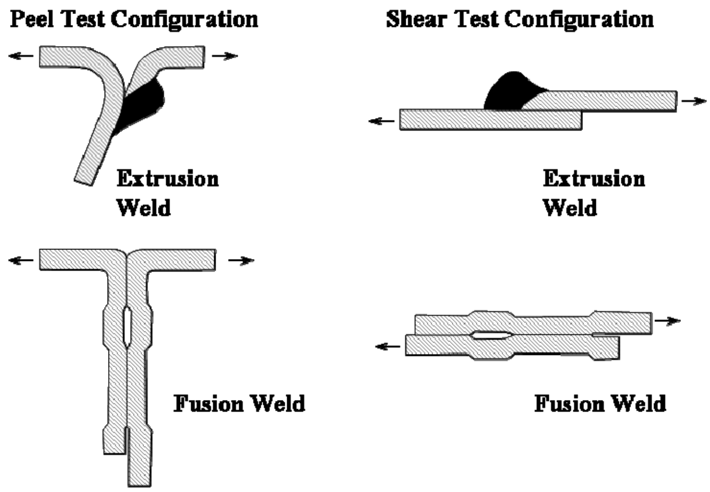
Figure 42. (*a*) CQA Inspection of Geomembrane Panel; and (*b*) Inspection of Several Panels Prior to Seaming (note panel and seam orientation)

All seams should be numbered with a system that is compatible with the panel numbering system. Prior to field seaming, all equipment should be inspected to make sure it complies with the manufacturer's recommendations and that the equipment is operating properly (e.g. temperature and feed rate are uniform).

Welding techniques and seam controls

For PE geomembranes, the primary welding technique is by hot wedge fusion. Hot wedge fusion welding involves heating the geomembrane while rolling the geomembrane between rollers. The rollers apply pressure, fusing the geomembrane. A double fusion weld is most common (Mollard *et al.* 1996) and has the advantage of using two sets of rollers to form an air gap in the central portion of the seam. The air gap formed by the double rollers can then be used to nondestructively test the seam using an air pressure test. The secondary method of seaming is to use extrusion welding. This method extrudes hot polyethylene onto the seam, joining the geomembrane panels. Extrusion welding is typically used only for patching and seaming around appurtenances and destructive seam sample locations. Each hot wedge seaming apparatus is equipped with automated variable speed devices with speed and temperature adjustment devices. Pressure is controlled by spring, pneumatic, or other system that allows for variation in sheet thickness. Extrusion-seaming equipment should include gauges indicating temperatures of the extrudate and nozzle. Prior to seaming, the seaming area should be inspected to ensure it is clean and free of moisture, dust, dirt, debris, and foreign material.

Prior to seaming, trial seams should be made on test strips of geomembrane under field conditions to verify acceptable seaming conditions. The CQA monitor should observe all trial seam procedures and record the trial seam identification number, the time of seaming, ambient temperature, number of seaming unit, name of seamer, machine temperature, preheat setting, and pass/ fail description. The trial seams are tested for shear and peel strength and compared to the project specifications. A schematic of shear and peel tests is presented in Figure 43.



(*a*)

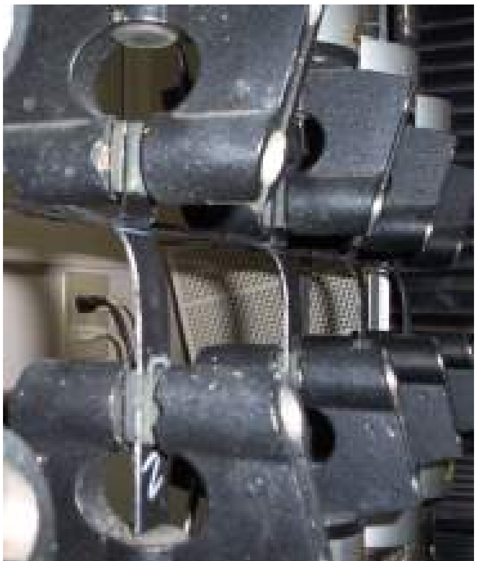


Figure 43. (*a*) Test seam configuration (after Mollard *et al.* 1996); and (*b*) Peel Testing Seam Samples

During seaming, the CQA monitor should record the seam number, location of seam, name of seamer, temperatures of the extrudate and nozzle, ambient surface temperature, and fusion-seaming apparatus speeds. Figure 44*a* presents a photograph of CQA monitors inspecting hot wedge seaming. Figure 44*b* presents a photograph of extrusion seaming of a patch.

(*a*)

(*b*)

Figure 44. (*a*) Hot wedge seaming and inspection; and (*b*) extrusion seaming of a patch

Destructive and non-destructive testing of the welded seams (fusion and extrusion) is conducted to test the quality of field manufactured seams. For destructive tests, geomembrane samples are cut from the seams for shear and peel tests (similar to that used for trial seams). The aim of theses tests is to evaluate the relative strength of the seam.

For non-destructive tests, air pressure test are typically used in double fusion seams. In this test, both ends of the seam are then sealed by clamps and the central air gap is pressurized and the air pressure is monitored over a specified time period. Leakage from the seam would result in a drop in the air pressure. Figure 45*a* presents an air pressure test on double fusion seams. The aim of non-destructive tests is to evaluate the seam continuity.

(*a*)

(*b*)

Figure 45. (*a*) Air Pressure Testing Double Fusion Weld; and (*b*) vacuum box

For extrusion welds, non-destructive testing consists of either using a vacuum box or by using electrical conductivity.

The vacuum box method (see Figure 45*b*) consists of placing soapy liquid onto the extrusion weld, then placing a vacuum box over the seam. A vacuum is drawn on the seam and air leaks are identified by "soap bubbles" along the seam.

The electrical resistivity method uses a copper wire that is placed in the junction of the seam, prior to extrusion welding. The extrusion welding goes over the top of the wire, sealing it within the weld. After welding, an electrical current is applied over the weld using a spark-gun. A spark will develop where the weld is incomplete, however as noted by Mollard *et al.* (1996), the testing method only identifies areas where the extrusion weld is thin or missing, it does not identify areas of poor bonding.

Destructive testing of fusion and extrusion welds require cutting portions of the seams from the field and testing them in peel and shear, as shown in Figure 43. Destructive testing can be controversial. As pointed out by Thiel *et al.* (2003), this type of testing dictates cutting holes per a frequency in the geomembrane by removing fusion welds and replacing them with patches that have extrusion welds. While destructive testing is still considered the state-of-practice, there are several alternative approaches that are being considered to minimize or replace prescriptive destructive testing. Most notably, infrared sensing technology (Peggs *et al.* 1994) has been evaluated on full scale projects. Ultrasonics (Peggs *et al.* 1985, Koerner *et al.* 1987) is frequently used with bituminous geomembranes.

Leak detection

The final stage of CQA program for a lined facility is leak detection testing of the entire system (seams, panels, penetrations, etc.) of the facility as discussed in Koerner & Koerner (2003), Koerner (1996), Peggs (1996), Phaneuf & Peggs (2001) and Thiel *et al.* (2003). Testing of the entire system provides an opportunity to check for defects or damage that may have occurred during installation and is a final check on the quality of installation. CQA leak detection should not be confused with leak detection and collection systems that are designed and constructed as part of a lined facility. CQA leak detection is to be conducted at the end of construction to verify the quality of the installation. It should also be noted that some facilities and industries do not require CQA leak detection to be conducted, therefore the method and approach for CQA leak detection will vary from project-to-project.

Methods of CQA leak detection for landfills and other lined facilities generally rely on electrical methods (ASTM 2002, Berube *et al.* 2007, Darilek 1989, Hix 1998, Hruby & Barrie 2007, Koerner & Koerner 2003, Nosko *et al.* 1995, Peggs 1999, Phaneuf & Peggs 2001, Thiel *et al.* 2003). The primary methods of leak detection include geomembrane spark testing, electrical leak location surveys (see Figure 46), and permanent in situ systems (grid systems).

Some geomembrane liner manufacturers are placing strips of conductive materials within the geomembrane so that the liner system quality may be spark tested (ASTM 2006). In spark testing, an electrified wand is passed over the liner surface. Defects or holes in the geomembrane that expose the conductive material will cause a "spark" indicating a potential leak. The advantages of spark testing are that they are relatively easy to conduct and can provide real-time results during construction. The disadvantages of spark testing are that they can only be effective prior to placing material onto the liner system and the efficacy of spark testing is subject to human error (adequate coverage and/or operation of equipment).

Electrical leak location surveys are conducted after construction has been completed, but before the facility is put into use. Taking into account that the majority of the holes in geomembrane liners occurs after the construction is complete, the conduction of this testing is the only hypothesis to repair the damages and, in this way, ensure the liner system performance. It should be noted that the cost associated to this testing are substantially lower than those inherent in potential repairs after the facility is put into use. Electrical surveys consist of applying a voltage across the liner system, then systematically measuring the electrical resistance within the lined area (ASTM 2002, Colucci *et al.* 1996, Rollin *et al.* 2002). Areas with high resistance indicate the liner system is intact, while areas with low resistance are indicative of holes or tears in the liner. The advantages of electrical resistivity surveys are that they are relatively easy to conduct and can provide real-time results during construction. The disadvantages of electrical surveys are that they can only be effective prior to placing material other than a drainage granular layer onto the liner system as after materials are placed on the liner, it can be difficult to locate and repair leaks. The surveys are affected by the conductance of the material overlying and underlying the liner system and contact of the liner to the ground (Peggs & Wallace 2008). Finally, the efficacy of resistivity survey is subject to human error (adequate coverage and/or operation of equipment). They thus require an experienced operator (Hruby & Barrie 2007).

(*a*) (*b*)

Figure 46. Electrical leak location survey (*a*) on exposed geomembrane; and (*b*) on covered geomembrane (courtesy of Solmers).

A permanent in situ leak detection system is often not considered part of a CQA program, but it may be considered in some cases. The method consists of burying a grid of electrodes within the foundation of the facility, prior to installation of the liner system. Similar to electrical resistance surveys, the electrical resistance is measured between each “node” of the electrode grid (White & Barker 1997). Areas with high resistance are indicative of “dry” conditions, while areas with low resistance indicate “moist” conditions, which may signify leakage through the liner. The advantages of the electrode grid method are that:

- The method can be used during any time of the facility operation;
- The method provides uniform coverage of the facility; and
- The method is more automated so it reduces the potential for human error.

The disadvantages of the electrode grid method are that:

- In order to test for leaks a liquid must be placed over the lined surface;
- The grid is installed beneath the liner so it cannot be accessed if there is an electrical problem in the grid; and
- The presence of moisture inferred by the resistance readings may not be indicative of a leak, but may reflect an increase in soil moisture due to a rising water table or local infiltration at the edge of the facility; the difficulty to repair leaks is even larger in this case than for electrical surveys (Hruby & Barrie 2007).

CQA leak detection is not yet generally accepted practice, however it is increasingly used and provides a powerful tool to evaluate the quality of the installation after construction. This is currently accomplished with diligent CQA surveillance and documentation, however the CQA leak detection process is only as good as the CQA monitors. Even with a high-level of training, CQA monitors may not be able to identify all flaws and defects in liner systems. CQA leak detection offers an opportunity to enhance our ability to detect defects or flaws that may impair the performance of the liner system.

This comment is also valid regarding general aspects of control. A process is undertaken at present in France to generalize certification to controllers as control quality strongly depends on the ability and competence of controllers to perform control (Gérard 2006).

GEOSYNTHETIC BARRIER FIELD PERFORMANCE DATA

Introduction

While discussing the field performance of liners, certainly the most discussed issue in the past two decades was leakage through liners involving a geomembrane. A wide number of authors have published data on this topic synthesised among others by Bouazza *et al.* (2002), Kavazanjian *et al.* (2006) and Rowe (1998, 2005). No new data have arisen to the authors' knowledge since the previously mentioned papers on this topic. Consequently the issue on hydraulic performance of composite bottom liners will be briefly discussed herein. Rather, the focus of this section will be on the performance of GCLs in covers, liners temperature which can impact transfers and GCL shrinkage.

Hydraulic performance of bottom composite liners

Some landfills have been constructed with double composite liner systems that means with a SLCS between the primary and secondary liners. Monitoring data from SLCS provides a rapid detection system of leaks through the primary liner, as well as it gives operator time for response before contaminants escape from the landfill and migrate into the subsurface. It also may provide insight regarding the effectiveness of the primary liners as emphasised by Rowe (1998).

A certain number of studies on flow rates have been made on landfills with SLCS by measuring the flow in these systems. Nevertheless, the interpretation of the data requires careful consideration of sources of fluid other than flow from the landfill (Gross *et al.* 1990). According to these authors, fluid may enter the SLCS as:

- Infiltration during construction of the system;
- Water arising from the compression and consolidation of the clay component of the primary liner under the weight of the waste;
- Groundwater infiltration from outside the landfill; and
- Flow through the primary liner due to defects in the geomembrane.

Of particular interest, in the context of the present work, is the study conducted by Bonaparte *et al.* (2002). These authors examined data from 72 landfill cells containing one of a single geomembrane primary liner, a geomembrane/CCL primary composite liner, a geomembrane/GCL or a geomembrane/GCL/CCL primary composite liner with either sand, gravel or geonet SLCS. The authors used the data to calculate average and peak SLCS flow rates for three distinct landfill development stages: (1) initial period of operation; (2) active period of operation; and (3) post-closure period. Only the second and third periods are of interest here as they provide a better indication of liner performance (Rowe 2005). The main findings from this study are as follows:

- For geomembrane primary liners, the flow rate collected often exceeds that for a primary CCL with $k=5\times10^{-11}$m/s but is normally considerably less than that for a primary CCL with a typically specified $k=1\times10^{-9}$m/s. However, in some cases, the leakage through the geomembrane alone exceeds that of the CCL with $k=1\times10^{-9}$m/s;
- The combination of a geomembrane and GCL substantially reduces leakage relatively to a geomembrane alone;
- The interpretation of leakage rates for geomembrane/CCL composite liners is more complicated due to the multiples sources of water in the SLCS as previously mentioned. Despite of those uncertainties, field data tend to show that geomembrane/GCL composite liners perform better than geomembrane/CCL composite liners. According to Rowe (2005) those differences cannot be explained as the result of consolidation water.
- From data presented by Bonaparte *et al.* (2002), Rowe (2005) deduced mean values of average monthly flows in the active and post-closure periods respectively equal to 90 and 50lphd for geomembrane/GCL composite liners. Typical field leakage rates from geomembrane /GCL composite liners ranged between 0.6 and 1.5lphd.

However as mentioned by Bouazza *et al.* (2002) the above observations regarding GCLs may not be indicative of the long term behavior of composite liners containing GCLs, because most of the GCLs in the US EPA's field study reported by Bonaparte *et al.* (2002) were in service for less than 5 years.

It is important to note that higher flow rates appear to be achieved in landfills with no CQA. For example, Bonaparte & Gross (1990, 1993) reported that 19 % of landfills with CQA had SLCS flow rates of 50 lphd or less and 57% of landfills had SLCS flow rates of 200 lphd or less, whereas for landfills with no CQA, only 20 % had SLCS flow rates of 200 lphd or less. Higher average flow rates for landfills without CQA were also indicated by Tedder (1997).

Hydraulic Performance of GCLs in covers

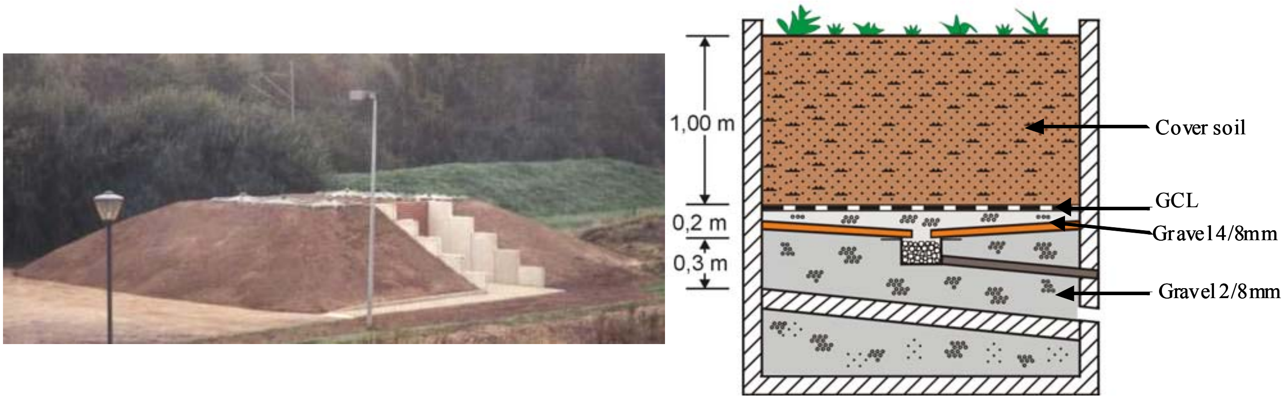
The focus of this section is to make a synthesis based on the existing literature on the behaviour of GCLs in covers based on field data. This point is important to discuss as a significant number of studies have been published on this topic and no agreement has yet been obtained on GCL performance in covers. The objective of this section is thus to try and clarify the parameters of influence on field performance for the case of this specific application of GCLs. Analyses of phenomena that could explain the observed results is also given in the following.

The oldest study on this topic was detailed by James *et al.* (1997) who presented a case history of an adhesive-bonded GCL used to provide seals to the roofs of five water service reservoirs. The predominantly sodium bentonite, containing about 2% calcite exchanged sodium for calcium and this exchange was extensive within a matter of months. The GCL was placed on compacted clay and was covered for all five water reservoirs by a 0.15m thick non calcareous gravel layer and a 0.3m thick soil cover. Finely cracked zones were frequently observed. Hydraulic conductivity tests were not conducted on the exhumed GCLs.

Heerten & von Maubeuge (2000) reported the results of a study performed by Maile (1997) and Maile *et al.* (1998) in 2m diameter lysimeters at Essen, containing either CCLs or a GCL. The GCL used had a 200g/m² PP woven, a 300g/m² nonwoven carrier geotextile impregnated with 800g/m² of natural sodium bentonite and 5kg/m² of natural sodium bentonite in total. The GCL was covered by a 1m thick soil layer. No permeation was shown on the GCL lysimeter at all during the three years of test under natural climate. Artificial precipitation was then created during which the efficiency of the cover was about 99%. When the GCL was excavated, roots were observed to have penetrated the entire soil cover and partially through the drainage mat covering the GCL. The hydraulic conductivity of the exhumed GCL was measured to be 2 to 4×10^{-10}m/s in agreement with the field permeation rate through the GCL. Comparison of physical properties of the initial bentonite to the one obtained after excavation showed that cation exchange had taken place and that no further changes were to be expected.

More recently, Mansour (2001) investigated the behaviour of a sodium bentonite needle punched GCL after 5 years of field exposure in a semi-arid climate. The test plot GCL was overlain by approximately 0.66m of on-site soil. The soil was not vegetated which may have affected the results of the study as vegetation can increase the potential for drying out of the soil and can also adversely affect the GCL through root damage. Soil samples taken at different depths below and under the GCL showed an increase in water content with depth. At a depth equal to 0.5m the soil did not show any desiccation. This in return kept the GCL hydrated. Results of GCL index flux, swell index and fluid loss did not reveal any significant evolution in those features after 5 years of exposure. Soil samples analysed for cations showed a very little amount of calcium.

Blümel *et al.* (2002, 2003, 2008) give results obtained in 3m² lysimeters (see Figure 47) with three different needle-punched GCLs containing sodium bentonite either powdered or granular with masses per unit area of bentonite in the range 4.5 to 5 kg/m². All GCLs were covered by an approximately 1m thick layer of silty sand and exposed to humid climate conditions in northern Germany. The portion of the rainfall (around 800mm/year in average) that permeated through the GCLs during seven years is in the range 0.4 to 1.3%. GCLs filled with bentonite powder seemed to have a somewhat lower permeability than the GCL containing granular bentonite. Detailed analyses seemed to give some indication of a small increase in this rate in the long term view. Despite an almost complete cation exchange in the GCL from sodium to calcium during the first three years, no significant increase in flow was noticed. Furthermore there was no significant evidence that the water content of the bentonite had been reduced in the summer to such critical values that shrinking with cracks had occurred. The sealing efficiency of the GCL was shown to be directly related to the amount of drainage water, which is a reversible effect. After a temporary increase of permeation the GCLs presented a full sealing efficiency in the following winter period. The authors also recommend to choose a layer of more than 1m in thickness of such soil and densities that provides a steady wet climate in the soil pores above the GCL.



(*a*) (*b*)

Figure 47. (*a*) View of soil protected lyimeters; and (*b*) general lay-out of a single lysimeter test set-up (from Blümel *et al.* 2002)

Melchior (2002) studied two needle punched GCLs, two stitch bounded GCLs and a composite geomembrane/bentonite GCL in the field under a 0.45m thick cover containing humic topsoil and drainage layer. Results obtained in 100m² lysimeter test pads in the field show that during the first winter the GCLs were swollen and very effective. After the first dry summer leakage through the needle punched and the stitch bounded GCLs dramatically increased up to values greater than 180mm/year. The average annual precipitation on the period of time studied was close to 750mm. The reasons for the damage are desiccation and root penetration which led to the formation of a typical soil structure with cracks in the GCLs. The desiccation was the result of the upward directed water loss to the dry covering layers and the direct water uptake by plant roots. Due to ion exchange from sodium to calcium the bentonite of the desiccated GCLs did not re-swell to such a level that the GCL re-gained a sufficient low permeability. After two years all single layers and upper layers in the overlaps of the needle punched and stitch bounded GCLs showed desiccation cracks. After 4 years, the lower layers in the overlaps were also severely damaged.

The geomembrane/bentonite GCL exhibited a different behaviour as the geomembrane prevented desiccation and root penetration into the bentonite layer as the geomembrane was located upwards.

Wagner & Schnatmeyer (2002) investigated the behaviour of six different types of cover in 3×15m test pads including capillary barriers, a GCL, CCLs and blast furnaste dust. The average annual precipitation was 876mm. The GCL used was needle punched with a total mass per unit area equal to 4.2kg/m², with an initial hydraulic conductivity lower than 5×10^{-11}m/s. The GCL was covered by a 0.25m thick gravel layer located under a 0.75m thick covering material. After two years of testing, the GCL cover exhibited the best hydraulic performance of all liners. Nevertheless the hydraulic conductivity estimated for the test pad result after two years was estimated to be 2×10^{-10}m/s. This increase in hydraulic conductivity was attributed to cation exchange.

Mackey & Olsta (2003) investigated the behaviour of two GCLs used in two landfill closures in a coastal area in Florida. In the first landfill (alpha) the mass per unit area of the GCL was 5.9kg/m² at a 19% water content, the swell index ranged from 24 to 26ml/g and fluid loss from 5.6 to 14.8 ml. The hydraulic conductivity ranged from 4.1×10^{-12}m/s to 9.5×10^{-12}m/s at 13.8kPa confining pressure. Three samples were removed after 7 years and 2 months in a single area where it is believed that changes in the GCL could be directly related to the soils used during construction and climatic (drought) conditions. The soil thickness on top of the GCL samples was in the range 0.61-0.81m. Roots were observed in some samples (see Figure 48).

Figure 48. A-3 GCL sample specimen at manufacturer's laboratory exhibiting roots (from Mackey & Olsta 2003)

In the second landfill (beta) the mass per unit area of the GCL was 5.76kg/m² at a 19% water content, the swell index 33 ml/g swell index and the maximum fluid loss 9.7ml. The hydraulic conductivity ranged from 1.2×10^{-11}m/s to 2.3×10^{-11}m/s at 13.8kPa confining pressure. Samples were taken at two locations on top of the landfill 5 years and 8 months after construction. Shell fragments were observed within the topsoil matrix in one location. The soil thickness on top of the samples was in the range 0.46 to 0.56m for sample B1 and 0.86m for sample B2. For all samples a significant reduction in swell index and large increase in fluid loss was observed except for one sample of landfill beta (sample B2) where the fluid loss results were just above the specification requirements. Unless what was observed for the other samples the hydraulic conductivity of this sample only increased slightly. There was only a small amount of sodium remaining in the bentonite of all samples even if the minimum calcium content was 0.24% in the soil, for the landfill beta. There was a higher percentage of calcium ions in samples of landfill alpha than in samples of landfill beta in relation with the higher calcium content in the surrounding soil. The same relative difference in calcium content for all samples from landfill beta is not reflected by a significant difference in the hydraulic conductivity of the two GCL samples from this landfill. The test results show that at lower calcium content within the bentonite the confining pressure exerts a more determining factor in the hydraulic conductivity of the GCL. Another explanation might be the fact that the sample from landfill beta with the lowest hydraulic conductivity came from a location of an overlap seam where the greater amount of bentonite may have allowed a lowered sodium-calcium ion exchange impact to the GCL. This point requires further investigation according to the authors. Several factors could explain the increase in hydraulic conductivity among which root penetration observed for two samples at landfill alpha, and soil saturation in the other sample leading to buoyancy of the soil in its saturated conditions.

Cazzuffi & Crippa (2004) report on the excavation of GCL samples 7 years after installation from the capping in the confined area of a brownfield in the southern part of Italy. The GCL was a needle punched GCL incorporating natural sodium bentonite, with a mass per unit area greater than 4890g/m² (manufacturer data sheet, personal communication from the first author). The soil thickness on top of the GCL is not known precisely (personal communication from the first author). Hydraulic conductivity and tensile tests were performed on the samples in the laboratory. Presence of roots was noticed in various excavated areas. In one area a single root reached and crossed the GCL. In this area samples with and without crossing roots were tested. An increase in the hydraulic conductivity was noticed as compared with the virgin material with the exception of one sample. In the worst case the hydraulic conductivity had increased by almost one order of magnitude. In the case of the sample with roots the hydraulic conductivity measured is lower than for sample without roots, the explanation proposed being self-healing capacity of bentonite.

Henkel-Mellies & Zanzinger (2004) investigated the behaviour of a stitch bonded GCL containing calcium bentonite with a mass per unit area equal to 9.5kg/m² in a 520m² test pad. At the location of the landfill the average annual precipitation is about 750mm. The cover consists from top to bottom of a 0.2m thick sand layer, a 0.8m thick slightly loamy sand restoration layer, a drainage geocomposite, the calcium-GCL, a 0.1m thick regulating layer made of slightly loamy sand and finally a drainage bottom geocomposite and a geomembrane in order to collect flow going through the GCL (see Figure 49*a*). Leakage flow through the GCL is in the order of a few millimetres per years or 0.3 to 1% of the precipitation even after about 4 years of use. The process of leakage through the GCL is not a low and constant flow but a sporadic flow occurring mainly during time of high drainage flow where it can be assumed that a hydraulic head builds up temporarily in the drainage layer, which then causes leakage flow (see Figure 49*b*). During most of the time the drainage layer effectively keeps the hydraulic head at about zero and consequently no leakage flow occurs. The results of the in-situ moisture measurements within the restoration profile show that during most summers the extraction of soil moisture reached down to the base of the 1m thick restoration layer. According to these authors the thickness of the top soil cover of surface sealing should not be designed just to satisfy the minimum requirements but to take into account the conditions at each individual site.

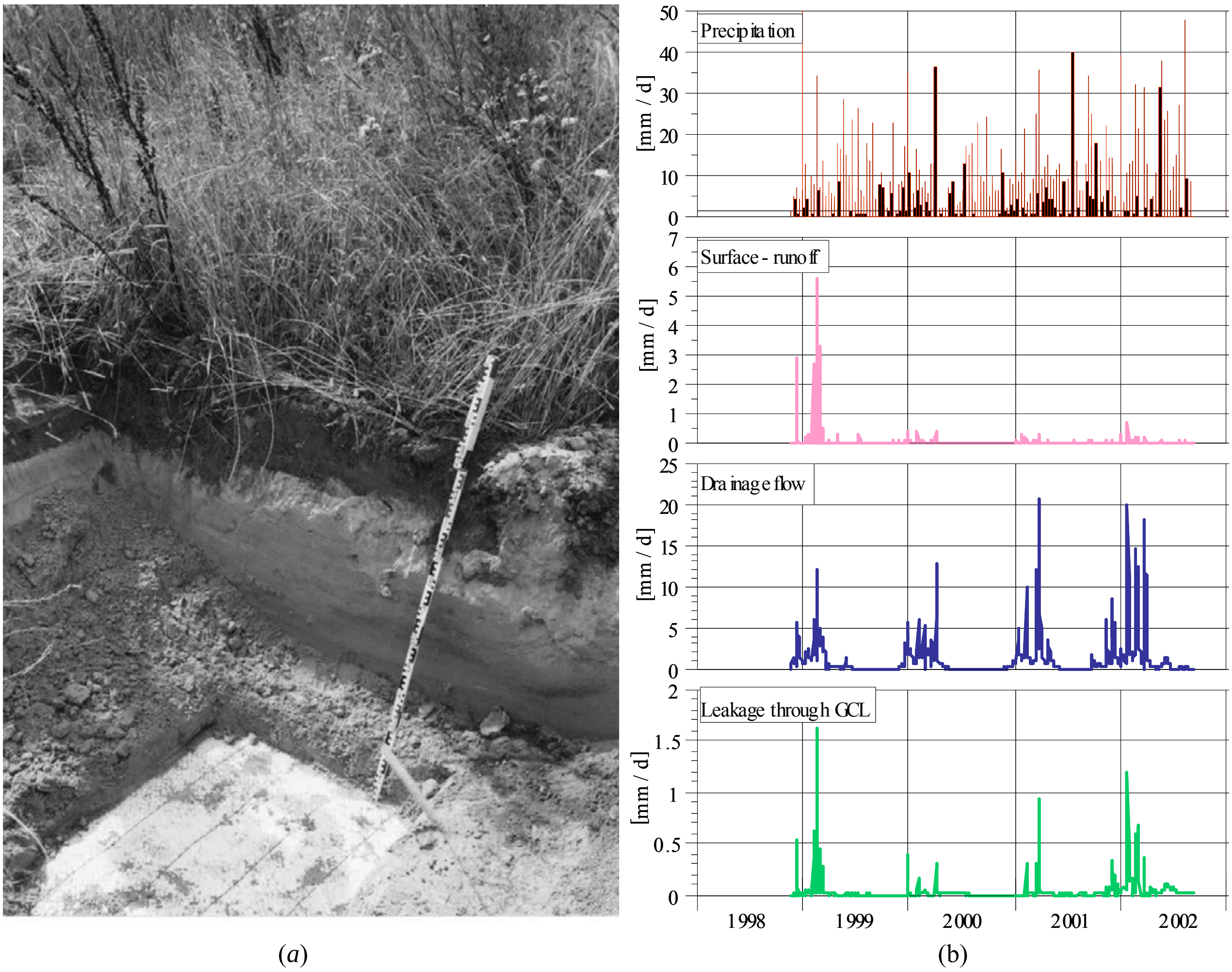


Figure 49. (*a*) Test-pit showing a section of the test field with vegetation, top soil (dark), subsoil (light grey) and drainage geocomposite (white); and (*b*) Results of test field measurements: daily values of precipitation, surface runoff, drainage flow and leakage through GCL from 1998 to 2002 (from Henken-Mellies & Zanzinger 2004).

Adu-Wusu &Yanful (2006) report three years performance results of a GCL used to mitigate acid rock drainage at Wistle mine in Ontario. The test plot was 12.2m wide and 24.4m long incorporating a 8mm thick GCL. As compared to a 0.46m thick sand-bentonite barrier and a 0.6m thick layer of sandy silt, the GCL was the most-effective barrier in reducing percolation into the underlying waste rock.

Barral *et al.* (2007) describe field test pads in which two needle punched GCL containing natural sodium bentonite with a mass per unit area of bentonite close to 5 kg/m² were installed on top of a 0.4m thick siliceous sand and under the same thickness of siliceous sand covered by a 0.3m thick cover soil. Six lysimeters 34m² in surface were installed under two different needle punched GCLs (one containing granular bentonite and the other one powdered bentonite) for three different configurations: pre-hydrated in situ, installed at the water content of reception and under an overlap. Results obtained after 9 months show that the GCL is protected by the cover from freezing and desiccation as the extreme temperatures measured at the depth of the GCL in the soil were equal to 8 and 35°C in the winter and the summer respectively. These preliminary results must be confirmed on the long term. Lysimeters located under the GCLs which were pre-hydrated received lower volumes than those located under the dry GCL. The lysimeters installed under overlaps also collected very small amounts of water. The estimated hydraulic conductivities based on this value was equal to 5×10^{-11}m/s for a hydraulic gradient equal to 1.

Benson *et al.* (2007*b*) describe the hydrologic performance of a final cover for a coal ash landfill where the barrier layer consisted of GCL or a composite GCL. The site located southwestern Wisconsin receives 892mm of precipitation and has a potential evapotranspiration of 838mm annually, on average. The cover profile consists of a 0.76m thick vegetated surface layer (silty sand), the GCL and a 0.15m thick layer of interim silty sand cover placed over the ash. The conventional GCL installed contained 3.6kg/m² of granular Na-bentonite and was encased between a woven slit-film PP geotextile and a lightweight spunlace polyester geotextile. Two lysimeters 4.3×4.9m² were installed beneath the cover to monitor the percolation rate. Percolation rates measured in both lysimeters were low within the first months after installation but increased dramatically up to 450mm/year in less than 18 months. The authors noticed that bentonite placed in the overlaps had numerous shrinkage cracks. Extensive cation exchange occurred in the GCLs resulting in a marked reduction in the swelling capacity of the bentonite. Percolation from the overlying soils was the likely source of the Ca and Mg involves in the exchange. Upward diffusion from the underlying sand may also have been important as Ca and Mg are the dominant exchangeable ions present in the underlying sand too. The hydraulic conductivity of the GCL in the vicinity of both lysimeters ultimately ranged between 1.4×10^{-8} and 9.1×10^{-7}m/s whereas the hydraulic conductivity of the new GCL ranged between 2.7 and 7.8×10^{-11}m/s. Preferential flow due to root intrusion appear not to be a factor causing the large increase in hydraulic conductivity. The conventional GCL was ultimately replaced by a composite GCL contained 3.6kg/m² of granular Na-bentonite encased between nonwoven and woven geotextiles and laminated with a polyethylene geofilm 0.1mm thick. Much lower percolation rates (2.6 to 4.1mm/year on the average) have been transmitted since the composite GCL was installed and similar percolation rates have been recorded regardless of whether the GCL was installed with the geofilm oriented upward or downward.

Meer & Benson (2007) report on the investigation of GCLs exhumed from four landfills in Wisconsin and Georgia and tested for water content, swell index, saturated hydraulic conductivity and exchangeable cations. The silty sand soil liner thickness on top of the various GCLs was in the range 0.75 to 0.9m. Most of the exchangeable Na initially on the bentonite in the exhumed GCLs was replaced by Ca and Mg and the bentonites had swell index close to 10ml/2g, typical of Ca bentonites. Hydraulic conductivities of the exhumed GCLs were in the range 5.2×10^{-11} to 1.6×10^{-6} m/s. Hydraulic conductivity of the exhumed GCLs was strongly related to the gravimetric water content at the time of sampling. GCLs with gravimetric water content less than 85% had hydraulic conductivities in the range 10^{-8} to 10^{-6}m/s whereas GCLs with gravimetric water contents greater than 100% had lower hydraulic conductivities in the range 10^{-10} to 10^{-9}m/s.

Zanzinger (2008) reports on excavations performed in Germany by Heerten (2004), Heyer (2000) and Sporer (2002) and measurements on sites performed by Blümel *et al.* (2006) and Henken-Mellies (2005). From those various experiences it appears that as a rule, in Germany, and by extension middle European climatic conditions, a 1m thick layer is sufficient in protecting GCLs from desiccation. No observation of desiccation with cracks was made under such conditions in Germany. Zanzinger (2008) suggests that climatic conditions may have an influence on the minimum soil cover thickness to use while comparing the studied he reported to observations performed by Meer & Benson (2007). Zanzinger (2008) also indicates that the right choice of cover layers reduces the risk of penetration by roots.

A synthesis of those various studies is presented in Table 9 in terms of GCL type, bentonite nature and mass per unit area, service life, cover thickness, mole fraction of sodium and calcium, swell index, presence of roots and hydraulic conductivity, in case those data are available.

From these results it can be concluded that the use of GCLs in capping systems is problematic if they are not adequately protected. It has to be expected that when desiccation and ion exchange take place after a short period of time irreversible damages to the GCL will occur. Indeed as shown by Sporer & Gartung (2002*a*) the conversion from Na to Ca bentonites is complete within a few years under the widest range of conditions. According to values from 12 excavations mainly of landfill cappings reported by Egloffstein (2001) the ion exchange usually takes approximately 1 to 2 years when the GCL is used in unsaturated conditions. Further, it has been established that due to cation exchange, the permittivity of the GCL is about one order of magnitude greater than that of the original Na bentonite as long as there are no desiccation cracks. When desiccation cracks occur the cation exchange in the interlayers prevents the desired self-healing of the fissures. The ensuing leakage cannot be avoided. Even a single desiccation event has an extremely pronounced negative effect on the self-healing capacity of Ca bentonites (Sporer & Gartung 2002*b*). The observation made by some authors that cracks in bentonite resulting from desiccation do not re-close after re-wetting may not be solely attributed to ion exchange but also to an irreversible change in the structure of the bentonite. This emphasises the need to prevent desiccation of the bentonite in cap seals of landfills by suitable cover soil thickness or an overlying geomembrane.

Table 9. Type of GCLs, Field hydraulic conductivity, cover system and other related data from literature (Adapted from Meer & Benson 2007)

Source	**GCL type**	**Bentonite type mass/unitarea**	**Service life (years)**	**Cover thickness (m)**	**Mole fraction sodium**	**Mole fraction calcium**	**Swell index (ml/2g)**	**Roots**	**Hydraulic conductivity (m/s)**
Cazzuffi & Crippa (2004)	NP	Na 4.9kg/m²	7	NR NR NR NR NR	NR NR NR NR NR	NR NR NR NR NR	NR NR NR NR NR	no no no yes no	8.61×10^{-11} 1.01×10^{-10} 1.32×10^{-10} * 7.92×10^{-11} § 3.3×10^{-11}
Heyer (2000)	NP	Na powdered	3	0.4	NR	NR	NR	NR	$5.0\text{x}10^{-10}$ ❅
James *et al.* (1997)	AB	Na	1.5	0.45	0.16	0.69	NR	NR	NR
Melchior (2002)	NP SB C	Na Ca act. granular	4 4 5	0.45 0.45 0.45	0.04 0.02 0.63	0.81 0.83 0.28	9 8	yes yes yes	3.5×10^{-7} ⌘ 1.0×10^{-7} ⌘ NR
Mansour (2001)	NP	Na	5	0.66	NR	NR	33	NR	from 1.4 to 2.2×10^{-11}
Mackey & Olsta (2004)	NP NP	Na 5.9kg/m² Na 5.76 kg/m²	7.2 5.5 5.5	0.61-0.81 0.46-0.56 0.86	0.01⌘ 0.02⌘	0.64⌘ 0.49⌘	8 to 8.5 7.5 11 (GCL) 14 (overlap seam)	yes	1.2×10^{-9} to 6.4×10^{-8} † 5.8×10^{-10} to 6.4×10^{-8} ‡ 2.2 to 2.3×10^{-10} 3.5 to 4.8×10^{-11}
Barral *et al.* (2007)	NP NP	Na 5kg/m² granular Na 5kg/m² powdered	0.75 0.75	0.7 0.7					5.0×10^{-11} 5.0×10^{-11}
Benson *et al.* (2007)	NP C	Na 3.6kg/m² granular	4.1 2 5	0.76 0.76 0.76	0.10 0.09	0.67 0.55	7-15 8-9	yes yes	1.7 to 9.1×10^{-7} 1.4 to 8.1×10^{-7} no increase in percolation rate
Meer & Benson (2007)	NP AB AB NP	Na Na Na Na	4.6 11.1 5.6 4.1	0.75 0.8 0.8 0.9	0.04 0.06 0.16 0.22	0.72 0.74 0.56 0.65	9 11 8 10		3.9×10^{-7}⌘ 6.2×10^{-10}⌘ 4.7×10^{-7}⌘ 1.1×10^{-6}⌘
Wagner & Schnat-meyer (2002)	NP	4.2kg/m² Powdered	2	1.0	NR	NR	NR		8.3×10^{-10} ❄,⌘ 2.0×10^{-10} ❆
Heerten & von Maubeuge (2000)	NP	5kg/m²	3	1.0	0.71-0.88	0.015-0.07	8-10	no	2 to 4×10^{-10}
Henken-Mellies & Zan-zinger	SB	Ca 9.5kg/m²	4	1.0	NR	NR	NR		1.1×10^{-8} ❄

Source	GCL type	Bentonite type mass/unitarea	Service life (years)	Cover thickness (m)	Mole fraction sodium	Mole fraction calcium	Swell index (ml/2g)	Roots	Hydraulic conductivity (m/s)
(2004)									
Blümel *et al.* (2002, 2003, 2008)	NP	Na 4.5kg/m² powdered	3	1.0	NR	NR	NR	NR	slight increase
	NP	Na 5kg/m² powdered		1.0	NR	NR	NR	NR	slight increase
	NP	Na 4.5kg/m² granulated		1.0	NR	NR	NR	NR	slight increase
Heerten (2004)	NP	Na,3 to 5 kg/m² powdered	5	1.4	full conversion from Na to Ca		NR	NR	1.2 x 10^{-10}❉
Sporer (2002)	NP and SB	Na	2-8	0.5-1.5	almost full conversion from Na to Ca		7-14	till 0.2m depth	$3x10^{-10}$ to $2.2x10^{-9}$⅄

AB: adhesive bounded, C: composite GCL, NP: needle punched, SB: stitch bounded
NR: not reported
* Sample without root from the same location than sample §
§ Sample with root
† Hydraulic conductivity of exhumed GCLs reported by independent testing laboratory
‡ Hydraulic conductivity of exhumed GCLs reported by GCLs manufacturer
❄ Computed based on peak daily percolation rate assuming unit gradient flow
⌘ Mean values from Meer & Benson (2007)
❆ Computed based on annual percolation rate
❉ Obtained under the assumption that the hydraulic gradient is equal to 30
⅄ Increase in permittivity of one order of magnitude in comparison with the initial permittivity

To create a system to cope with the risk of desiccation, all components of the whole capping system including subgrade, drainage layer and topsoil should be taken into account. The material properties, the placement of the soil and the thickness of the topsoil layers as well as the choice and care of the vegetation should be considered on a site specific basis. In particular the weather conditions in an average as in an extreme year should be known and taken into account while planning a cover system. It has been shown that a 0.75m thick cover can be insufficient to protect GCLs. This is consistent with finding from Reuter (2006) who based on the background of actual experience from test fields indicates that a minimum thickness of 0.3m is not sufficient as a protective action against weathering influences. It supports only the sealing effect during initial hydration and protects against mechanical damaging. An optimal sealing effect will be reached, if between the loading of 0.3m minimum cover during the installation and the loading of the further cover layers in an height of minimum 1.0m not more than 2 to 3 weeks have passed. It has also been shown that climatic conditions may have an influence on the minimum soil cover thickness to use and that 1m proves to be sufficient in Central Europe. In all mentioned cases the use of composite GCLs resulted in better results as the geofilm was not crossed by roots. The impact of roots however is not clear as it seems that in certain cases better performances can be obtained where roots are present, maybe in conjunction with self-healing effects when this phenomenon can take place which will not be the case if a significant cation exchange has occurred.

Liner temperature

Increases in landfill temperature arise from heat generated by biodegradation of waste or the heat of hydration of incinerated residues (ash) (Rowe 2005). Breitenbach & Smith (2006) also mention that most copper leach operations are using bio-chemical processes to recover copper from sulphide ores. The biological reactions are exothermic and operating temperatures at the base of a large sulphide heap are estimated to reach up to 50°C.

The temperature at a HDPE geomembrane liner has a crucial influence on the long-term integrity and service life of the liner as elevated temperatures greatly accelerate the rate of depletion of the protective antioxidants added to the geomembrane during manufacture (Needham & Knox 2007). This point will not be further discussed here as a Keynote Lecture is dedicated to durability and lifetime prediction of geosynthetics in this conference (Hsuan *et al.* 2008). Temperature also influences both hydraulic conductivity and diffusion coefficient (Collins 1993). Rowe (2005) indicates that diffusive and advective transport are respectively 40% and 30% higher at 20°C than at 10°C, and 100% and 80% higher at 35°C. Consequently if the liner temperature changes with time, the effect of the change in diffusion and permeability parameters with temperature on contaminant impact should be considered. Rowe & Booker (2005) proposed an approach that readily models the effects of changes in diffusion coefficient and hydraulic conductivity with time. A literature review of liner temperatures is presented in the following of this section. The impact of temperature on diffusion through GCLs will be further discussed in the section of this Keynote Lecture dedicated to transfer and attenuation through GCLs. Literature data exist on liner temperature for CCLs, GCLs and geomembranes. Some authors also present an evaluation of the impact of leachate recirculation on liner temperature in some cases as will be presented in the following.

Yoshida & Rowe (2003) discussed the observed temperatures at the base of the Tokyo Port Landfill where 35m of waste were placed directly on the surface of a natural clayey liner. They also report temperature data from two other landfills in Tokyo. In most cases the temperatures were high 7 to 10 years after the beginning of landfilling and then declined. The maximum temperature rose to 50°C.

Rowe (2005) gives the example of the Keele Valley landfill where liner temperature has been monitored above a CCL over a 21-year period. Data indicate rapid increases in liner temperature with increases in leachate mound (see Figure 50). The maximum CCL temperature observed after 20 years approaches 40°C. Based on those data, Rowe (2005) indicates that the final maximum liner temperature is higher for landfills where there is a high waste moisture content due to leachate mounding.

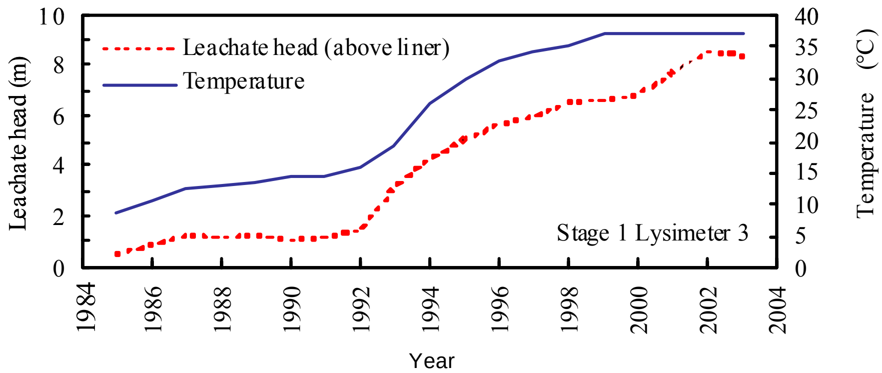


Figure 50. Variation in leachate head and temperature at one location in the Keele Valley Landfill (after Rowe 2005; reproduced with permission of Geotechnique).

Regarding GCLs, Yesiller & Hanson (2003) and Hanson *et al.* (2005) present temperature data monitored at the bottom of three cells at a landfill in Midwestern USA for more than five years. Temperature of the GCL have reached 30°C at liner locations near the center of one cell under approximately a 5 year old waste, corresponding to a maximum waste height of 43m. In another cell the temperature reached 21°C in the same configuration under a 4 year old waste. The temperature reached 17°C in the last cell under approximately 1.5 year old waste corresponding to a maximum waste height of 26m. The rates of temperature increase in the liners were approximately 2.9, 3.6 and 5.2°C/annum for the three previously mentioned cells respectively.

Klein *et al.* (2001) present the results of temperature measurement at a 2.5mm thick HDPE geomembrane installed on a 0.6m thick CCL and separated from a 9m thick incinerator bottom ash by a gravel drainage layer and a geotextile. In the geomembrane the initial temperature rise of 0.14°C per day which lasted during 4 weeks was followed by a levelling off during the next two months. Afterwards a second increase in temperature at a rate of 0.065±0.005°C per day was observed. The maximum temperature of 45.9°C was reached 17 months after installation. Subsequently the temperature decreased at a rate of 0.6°C per month and was around 40°C at the end of the measurement period presented in the paper. The authors estimated that the geomembrane temperature was to stay close to 40°C in the following years.

A concern regarding bioreactor operation is that exothermic reactions associated with waste degradation may cause temperatures to increase. Benson *et al.* (2007*a*) indicate that they did not notice any significant difference in leachate and liner temperatures between bioreactor and conventional landfills. On the contrary, Koerner & Koerner (2006) who presented the evolution of the liner temperature at a dry and a wet cell indicate a significant difference between both management strategies. Indeed for the conventional landfill, the liner temperature was constantly 20°C for a long period of time and then abruptly rose to 30°C after 6 years (see Figure 51). The trends were slightly increasing during the four additional years of measurement. In the bioreactor cell, the liner temperatures began higher than those at the dry cell from 5 to 8°C and have a uniformly increasing slope. After almost 4 years they are between 41 and 46°C and they continue to rise. It may be hypothesised that the delay before the temperature increases reflects the time required for significant heat generation and conduction of that temperature downward to the liner (Rowe 2005). Those data are also consistent with data from Rowe (2005). Nevertheless, it will be necessary to undertake liner temperature measurement on a long term basis where there is no significant leachate mound to confirm this result because of the slower way of temperature increase with time (Rowe 2005).

Needham & Knox (2007) present the results of more than 6 years of temperature monitoring at the liner of a mainly MSW landfill with leachate recirculation during and after waste disposal. London clay was compacted to form a basal liner covered by a 0.3m thick layer of drainage stone. The start of infilling in October 2000 coincided with heavy rainfalls. Since the wastes were capped in November 2001 leachate recirculation was achieved and continued until January 2005 to be the sole method used to control leachate head. Since then both recirculation and discharge have been used. Temperature increased during the monitoring period from initial values in the range 15-20°C to peak temperatures of 35-36°C during the last year of measurement. Leachate heads measured were in the range 0.3m to 9.6m for peak values.

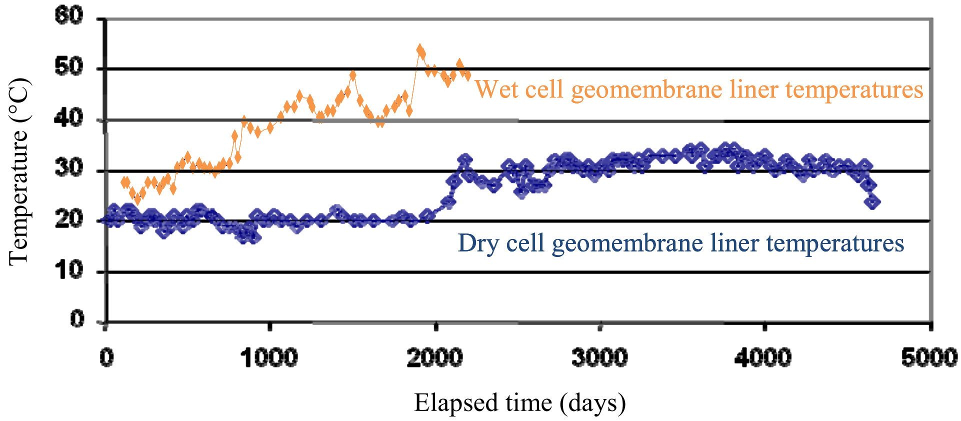


Figure 51. Geomembrane temperature from wet and dry cells (reploted from Koerner *et al.* (2008))

The length of time for which the temperature remains this high is unknown at present but based on available data is likely to be at least a few decades for large landfill such as those where data is currently available (Rowe 2005, Yoshida & Rowe 2003).

Various solutions were proposed intending at reducing geomembrane temperatures. Legge (2006) and Legge *et al.* (2007) present the results of an experimentation performed in a 4m wide and 5m long reservoir with a 0.6m thickness aiming at studying the potential for liner temperature reduction by fluid flow through a leakage detection system (LDS). (see Figure 52).

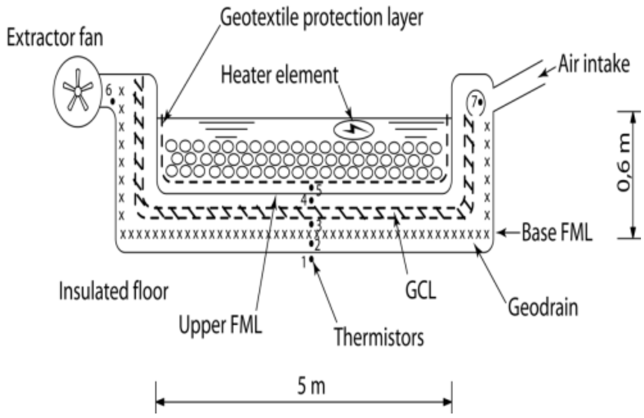


Figure 52. Schematic layout of thermal gradient evaluation apparatus (from Legge *et al.* 2007)

An extractor fan was provided through which ambient temperature air could be drawn through the LDS. The authors noticed the existence of a thermal gradient through the LDS which increases with fluid flow through the LDS placing the secondary liner and geosynthetic drainage layer at a lower temperature. Rowe *et al.* (2007) report a preliminary modelling investigation of the feasibility of both controlling liner temperature and extracting heat for on-site use. The proposed design solution involves the installation of an active coolant horizontal pipe array in a sand protection layer above the gravel drainage layer of the LCS.

GCL shrinkage

There is a growing awareness that an uncovered geomembrane overlying a GCL can result in the GCL panels underlying the geomembrane shrinking to an extent that causes GCL panels to part in the overlap area and a certain number of papers have put this phenomenon in light in the past few years (Kavazanjian *et al.* 2006, Davies 2007). Acknowledgement of this issue were by Thiel & Richardson (2005) (see Figure 53) and Koerner & Koerner (2005*a*,*b*) who reported five cases of GCL panel separation due to GCL shrinkage presented in Table 10. These observations are related to GCLs covered with a geomembrane which was left exposed from two months to five years prior to soil covering. In all cases the side of the geomembrane in contact with the cover geotextile of the GCL was textured. The GCL panels had shrunk in the cross machine direction by an amount ranging from 0.15m to more than 1.35m in an extreme case. In all the documented cases, no increase or decrease was noted in the machine direction of the GCL panels.

Thiel & Richardson (2005) and Koerner & Koerner (2005*a*,*b*) summarised several possible reasons why GCL panel shrinkage could occur. The potential causes of panel shrinkage can be summarized as follows (Thiel *et al.* 2006):

- Bentonite shrinkage due to desiccation, possibly exacerbated by hydration-drying cycles;
- Shrinkage of one or both of the geotextile components of the GCL;
- GCL panel necking due to Poisson's effect linked to tension in the longitudinal direction caused by gravity on slope (or caused by the next mechanism in the longitudinal direction);
- GCL panel lateral gathering due to repeated extension-contraction of the overlying textured geomembrane that somehow dragged the GCL toward the centre of the panel in the lateral direction; and
- Bentonite shrinkage due to cation exchange.

Figure 53. Photograph showing gap in GCL panels after removal of geomembrane (from Thiel & Richardson 2005)

Table 10. Known panel separation cases (from Koerner & Koerner 2005*a*)

Date (duration of exposure)	Location	Slope	GCL Type (Geotextiles)	Geotextile orientation	Moisture (%)	Maximum separation (m)
1993 (5 years)	Massachussetts	2.5:1	Nonreinforced (double NW)	W-up NW-down	appr. 20	0.3
2000 (5months)	Virginia	3:1	Reinforced (double NW	NW-up NW-down	24-31	0.3
2001 (appr. 15 months)	Confidential	3:1	Reinforced (NW-W)	NW-up W-down	25-30	0.2
2004 (2 months)	South America	2°-4°	Reinforced (double W)	NW-up NW-down	29-44	0.15
2004 (3 years)	California	1.5-1	Reinforced (double NW)	NW-up NW-down	25-29	1.2

W: woven; NW: nonwoven

For reinforced GCLs, drying alone during GCL installation has not been observed to cause panel shrinkage (Thiel *et al.* 2006). This observation is enforced by laboratory testing by Koerner & Koerner (2005*a*) indicating that a maximum shrinkage of reinforced GCLs of the order of 2% with drying alone was observed while the separation of a 0.15m in a 4.5m wide panel would require a shrinkage of at least 3.3% (Thiel *et al.* 2006).

In contrast to drying only, cyclic hydration and drying can have a profound impact on GCL shrinkage as demonstrated by the laboratory testing performed by Thiel *et al.* (2006). The following phenomenon was supposed to occur: during the daytime the exposed geomembrane will increase in temperature due to the sun. A maximum temperature of approximately 70°C has been measured on black geomembranes (Pelte *et al.* 1994, Koerner & Koerner 1995).This elevated temperature causes the GCL to dry during the day. As a result water vapour becomes trapped between the GCL and the geomembrane. During the night, when temperature decreases, the water vapour trapped below the geomembrane will condense into droplets. If there is an appreciable slope, the droplets may run downgradient and, after a number of day-night cycles, may gather at the toe of slope. This accumulation of water may saturate the GCL and underlying soil in the vicinity of the toe of the slope and/or from a water pillow beneath the geomembrane. If there is only a slight slope, the condensed water would be available to go back into the GCL. During these cycles, the natural matric-suction of the bentonite in the GCL will always have a tendency to draw moisture from the subgrade. The rate at which the GCL will draw moisture from the subgrade will be site specific and depend on the subgrade moisture conditions and matric-suction characteristics of the subgrade soil. The subgrade moisture provides the GCL a water source for extended hydration-drying cycles. The magnitude of hydration and drying would vary substantially at different sites and under different exposures as well as from day-to-day, week-to-week and season-to-season.

Thiel *et al.* (2006) developed an experimental program to evaluate the amount of GCL shrinkage due to cyclic changes in temperature and water content. Six different needle punched reinforced geotextile-encased GCLs were tested from two different manufacturers with variations in geotextile carrier, bentonite source and granularity, water content and density of needle punching reinforcement. In addition to those GCLs a geomembrane-supported GCL with 0.4mm thick HDPE backing was tested. Five different woven and nonwoven geotextiles representative of those used in the GCLs were tested. The samples were cut to a dimension of 0.35m in the cross-machine direction by 0.6m in the machine direction. The samples were placed in a relaxed, stress-free state of aluminium pans with their as-received water content (see Figure 54). The two small ends of the samples were clamped using a continuous bar-clamp screwed to the pan intending to simulate the conditions in the field where most geomembrane/GCL slope installations include anchorage or ballast at both ends. The aspect ratio of the relevant portion of the sample was 1.8 (0.55/0.3). The samples were hydrated by even application of a 0.5L (water content of approximately 65%) or 0.3L of water, then covered with a plastic sheet and allowed to hydrate at room temperature. Samples were then placed in an oven at 60°C and left to dry for approximately 15 hours. Forty cycles were performed for GCLs and seven for geotextiles.

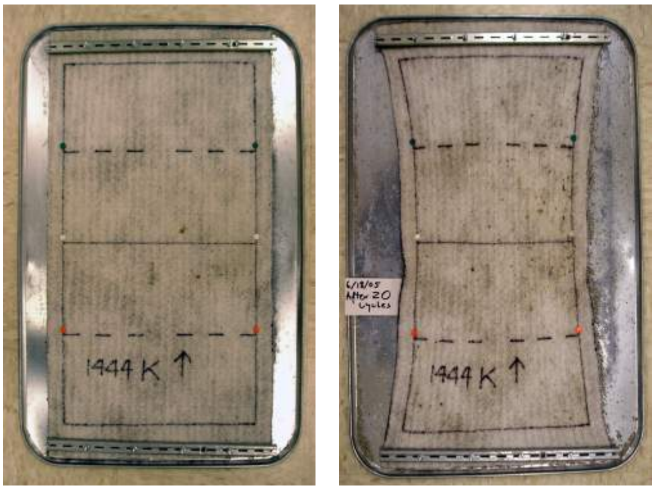


Figure 54. GCL sample (left) before test cycles and (right) after 20 hydration-drying test cycles (from Thiel *et al.* 2006).

The geotextiles by themselves exhibited very little shrinkage and stabilized after few hydration-drying cycles so that it was determined that geotextile shrinkage by itself is relatively small and a minor contributor to overall GCL shrinkage. The geomembrane-supported GCL did not exhibit noticeable shrinkage. Regarding the reinforced geotextile-encased GCLs, the following results were obtained:

- After 40 hydration-drying cycles, some samples still exhibit a trend towards further potential shrinkage;
- The testing methodology was able to produce an amount of shrinkage that is consistent with some of the field observations; those results were also supported by laboratory tests performed by Bostwick *et al.* (2007);
- Smaller water addition results in less shrinkage per cycle;
- The presence of a woven fabric in the GCL whether it is a woven carrier or a scrim associated with a nonwoven carrier, reduces the amount of shrinkage; this result was confirmed by Bostwick *et al.* (2008);
- Increased needle punching results in a lower tendency for shrinkage;
- Variable results were obtained for GCLs of the same type from different manufacturers without a clear explanation.

More testing would be required to:

- Determine the limit of shrinkage for each product;
- Determine the difference in the limit of shrinkage in relation with the amount of water supplied for hydration;
- Understand the difference in results for GCLs of the same type from different manufacturers that may depend on many variables among which type and granularity of bentonite, initial water content, type of geotextile fibers, methods of needle punching; and
- Refine the testing methodology to ensure that it is representative of the field conditions especially as regards amount of water used for hydration, the drying temperature, the cycle duration and the aspect ratio of the samples.

Regarding the aspect ratio, Bostwick *et al.* (2007) conducted laboratory tests using samples of different sizes and aspect ratios in the range 1 to 10. They showed that size did not play a significant role in either the rate or the magnitude of GCL shrinkage. No definitive conclusion was drawn regarding the influence of the aspect ratio. The initial moisture content did not appear to affect shrinkage for the range of values tested. Field data taken by Bostwick *et al.* (2008) following one year of liner exposure suggest that shrinkage in a cooler climate may be less than that noticed in the laboratory tests.

Longitudinal steep slope tensioning of the GCL was pointed out by Koerner & Koerner (2005*a*,*b*) as an important mechanism. Indeed, along with the daily expansion and contraction of the exposed geomembrane comes a gradually downslope moving wave which accumulates at the toe of the slope. For textured geomembranes placed above GCLs which have an upper nonwoven needle punched geotextile a tensioning drag force can be mobilized. Assuming that the GCL remains in its anchor trench at the top of the slope, this action represents a full width panel tensile stress of the GCL. In so doing a transverse panel contraction or necking occurs. In order to investigate the magnitude of this effect Koerner & Koerner (2005*a*,*b*) performed a series of laboratory tests on two reinforced and one unreinforced GCLs. Tensile tests were performed on GCL samples according to ASTM D6768 with aspect ratios in the range 0.5 to 5. Results obtained readily provide the necessary explanation for GCL panel separation.

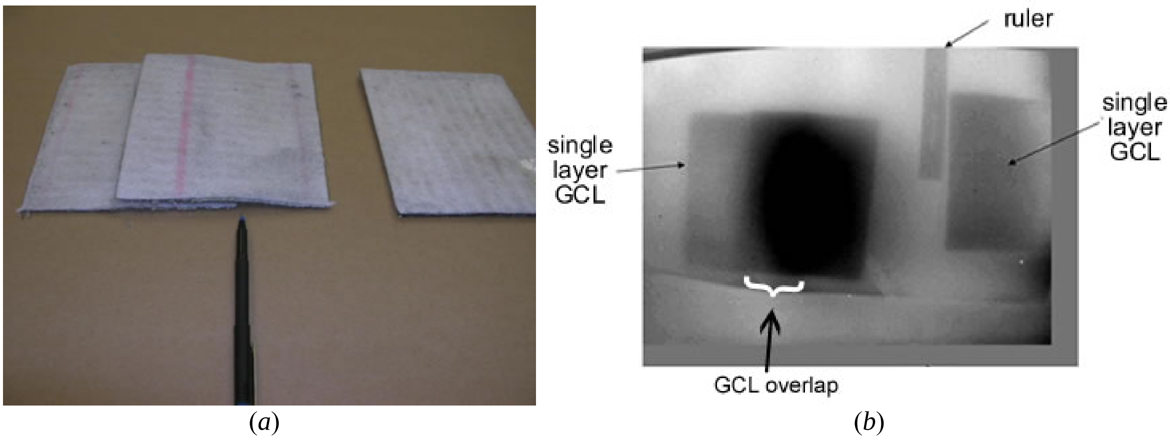
Thiel *et al.* (2005) have also observed some down drag of an unrestrained GCL on a 2:1 slope that appeared to be caused by the diurnal expansion and contraction of the overlying textured membrane. However regarding the cases with a relatively flat installation, there should not be a suspicion of tension in the GCL. Thiel *et al.* (2005) also give a case history in which tension-necking may not be the leading mechanism that causes GCL panel separation.

Transverse contraction or “gathering” of GCL should be investigated in the field since laboratory simulations are difficult to properly scale. The mechanism is believed to be possible on relatively flat surfaces, but is felt to be less of an effect than the longitudinal stressing mentioned previously (Koerner & Koerner 2005*a*) even if likely explains the one case in South America where the GCL panels separated on a relatively flat surface (see Table 10).

The impact of cation exchange in conjunction with hydration/drying cycles has not been studied up to date to the authors' knowledge.

Based on the experiments they performed Koerner & Koerner (2005*a*) make the following recommendations:

- Do not leave geomembrane/GCL composite liners exposed to the atmosphere; this recommendation is consistent with recommendations made by EAG-GTD (2002) specifying that GCLs should be covered in a non swollen condition; therefore the amount of GCL sheets deployed depends on what can be covered within the same day; the covering has to be finished before the swelling of the bentonite and has to take place immediately after the installation of the sheets; for the backfilling a minimum thickness of 0.3m is required;
- Do not use GCLs with needle punched nonwoven geotextiles on both sides unless one of the geotextiles is scrim reinforced, which is consistent with findings by Thiel *et al.* (2006);
- Increase the GCL overlap to compensate for the potential panel separation; Koerner & Koerner (2005*b*) suggest an increase from 0.15m to 0.25m or 0.45m depending on the GCL; Thiel *et al.* (2005) also recommend greater than 0.15m overlaps on most projects and to consider 0.3m overlaps as a standard minimum basis;
- Protect the exposed composite liner during its exposure time by using thermal blankets, geofoam, or other insulation techniques;
- Develop a non-destructive testing method to detect GCL panel separation; Koerner & Koerner (2005*b*) indicate that they investigated the use of ground penetrating radar and ultrasonic pulse-echo techniques to investigate if GCL panel separation has occurred without removing the geomembrane; Davies (2007) indicates that infrared thermography could also be used for this application based on preliminary laboratory tests (see Figure 55); and
- If possible create full-scale field test sites to study the conditions under which GCL panel separation occurs.



(*a*) (*b*)

Figure 55. (*a*) GCLs with overlap and gap and (*b*) infrared thermograph (Courtesy of Ian Peggs)

MODELLING OF FLOW THROUGH COMPOSITE LINERS

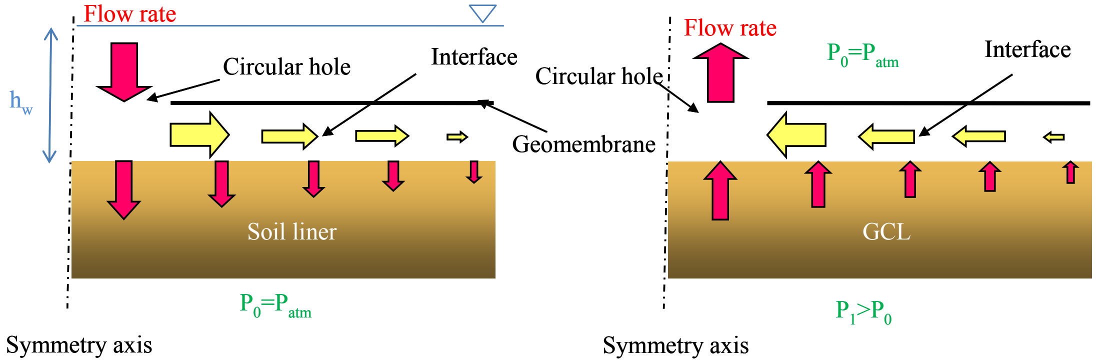
Introduction

The main goal of this section is to present a synthesis including recent advances on the existing methods available for the prediction of flow rates through composite liners due to the existence of defects in the geomembrane for situations where there exists an interface between the geomembrane and a soil liner that can be either a CCL or a

GCL. The methods that will be presented are analytical solutions, empirical equations and numerical modelling when they exist. Liquid and gas flow trough composite liners are studied herein.

Contact conditions

Before addressing these methods it is important to underline that the flow through a defect in the geomembrane depends, as indicated by Brown *et al.* (1987), on the contact between the geomembrane and the underlying soil liner. According to these authors, if the contact is not perfect, once fluid has migrated through the defect, it spreads laterally through the gap existing between the geomembrane and the underlying soil, called interface. This interface flow covers an area called wetted area. Finally, the liquid migrates into and through the soil liner as presented on Figure 56*a*.



(*a*) (*b*)

Figure 56. Flow through a composite liner due to a circular defect in the geomembrane (*a*) for a hydraulic head of liquid on top of the geomembrane, and (*b*) for a gas pressure under the GCL of a composite liner

There are three main sources of imperfect contact between a geomembrane and a CCL according to Rowe (1998): (1) protrusions related to particle size distribution in the liner material, which create a gap in which the fluid may flow; (2) undulations/ruts which result in the surface not appearing smooth; and (3) wrinkles in the geomembrane. The quality of the geomembrane installation is thus a key issue.

Focusing on the quality of the geomembrane installation, three geomembrane-CCL contact conditions are typically considered: excellent, good and poor contact conditions. The definitions of good and poor contact conditions were initially defined by Giroud (1997), based on the original concept by Giroud *et al.* (1989). The excellent contact condition was added to the previous ones by Touze-Foltz & Giroud (2003). Definitions of these contact conditions are presented below:

- Poor contact conditions (PCC) correspond to a geomembrane that has been installed with a certain number of wrinkles, and/or has been placed on a low-permeability soil that has not been adequately compacted and does not appear smooth;
- Good contact conditions (GCC) correspond to a geomembrane that has been installed with as few wrinkles as possible, on top of a low permeability soil layer that has been properly compacted and has a smooth surface. Furthermore, it is assumed that there is sufficient compressive stress to maintain the geomembrane in contact with the low-hydraulic conductivity soil layer; and
- Excellent contact conditions correspond to a geomembrane that has been installed with no wrinkles on top of a soil component of a composite liner that consists of a GCL installed on top of, and in close contact with, a low-hydraulic conductivity soil layer that has been adequately compacted and has a very smooth surface. Furthermore, it is assumed that there is sufficient compressive stress to maintain the geomembrane in contact with the GCL.

Qualitative definitions of contact conditions are subjective. This may lead to different interpretations of a given field case. To overcome this limitation, Rowe (1998) proposed quantitative definitions linking the soil liner hydraulic conductivity to the interface transmissivity for poor and good contact conditions. These quantitative definitions were extended by Touze-Foltz & Giroud (2003) for excellent contact conditions. Later on, Barroso (2005) proposed a new contact condition, the geomembrane-GCL contact condition, based on experimental data. Quantitative definitions of contact conditions are given below:

$$\log\theta = -1.7476 + 0.7155 \log k_L \quad \text{for excellent contact conditions} \qquad (1)$$

$$\log\theta = -1.3564 + 0.7155 \log k_L \quad \text{for good contact conditions} \qquad (2)$$

$$\log\theta = -0.5618 + 0.7155 \log k_L \quad \text{for poor contact conditions} \qquad (3)$$

$$\log\theta = -2.2322 + 0.7155 \log k_L \quad \text{for geomembrane-GCL contact conditions} \qquad (4)$$

Where θ is the hydraulic transmissivity of the interface in m²/s and k_L is the hydraulic conductivity of the soil liner in contact with the geomembrane in m/s.

Analytical solutions

A number of analytical solutions have been developed to quantify the flow rate through defects in flat or wrinkled geomembranes based on Darcy's law (Brown *et al.* 1987, Jayawickrama *et al.* 1988, Rowe 1998, Touze-Foltz *et al.* 1999) where the interface between the geomembrane and the underlying layer is of uniform thickness and, consequently, where the hydraulic transmissivity is uniform.

The most commonly used equations were proposed by Rowe (1998) and Touze-Foltz *et al.* (1999). The first author developed analytical solutions to quantify liquid flow for the case of a circular defect in a flat geomembrane and in a wrinkled geomembrane. Touze-Foltz *et al.* (1999) extended the solution for a damaged wrinkle for various boundary conditions and to the problem of liquid flow for two, or more, parallel interacting damaged wrinkles. Equations by Touze-Foltz *et al.* (1999) were again extended by Touze-Foltz *et al.* (2001) to take into account the non uniform hydraulic transmissivity at the interface of geomembrane-CCL or geomembrane-GCL composite liners.

The basic problem configuration follows from Rowe (1998) and Touze-Foltz *et al.* (1999) and is depicted in Figure 57.

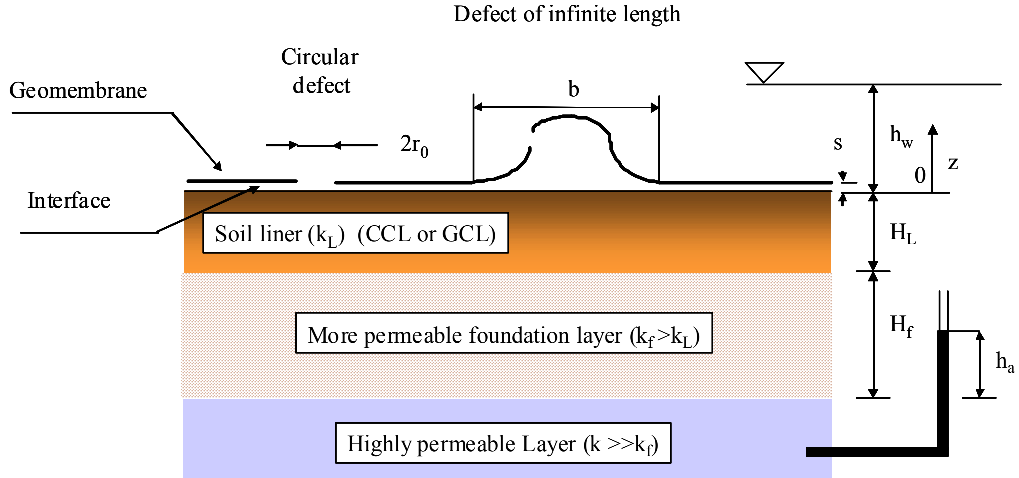


Figure 57. Schematic drawing showing a composite liner with a geomembrane exhibiting different types of defects: circular defect of radius r_0 and a damaged wrinkle of width b (modified from Touze-Foltz *et al.* 1999)

It includes a geomembrane resting on a low-permeability soil layer. This layer can be either a CCL or a GCL. From now on, it will be simply designated as "soil liner". The z-axis origin corresponds to the top of the soil liner with upward being positive. The soil liner rests on a more permeable foundation or attenuation layer which, in turn, rests on a highly permeable layer that can be either an aquifer or a leakage collection layer. Accordingly, it can be assumed that the flow through the composite liner is not influenced by the hydraulic conductivity of subgrade layers. It is assumed that the features of the interface (contact conditions) can be characterised by a uniform hydraulic transmissivity, θ. The interface transmissivity that governs interface flow can either be based on experimental data (Barroso *et al.* 2006, Chai *et al.* 2005, Harpur *et al.* 1993), or on empirical equations (equations 1 to 4).

Furthermore, it is assumed that: (1) liquid flow is under steady-state conditions; (2) the soil liner and the foundation layer are saturated; and (3) liquid flow through the liner and the foundation layer is vertical (Rowe 1998, Touze-Foltz *et al.* 1999). According to the continuity of liquid flow, the equivalent hydraulic conductivity, k_s, corresponding to the liner and the foundation layer is given by (Rowe 1998; Touze-Foltz *et al.* 1999):

$$\frac{H_L + H_f}{k_s} = \frac{H_L}{k_L} + \frac{H_f}{k_f} \tag{5}$$

Where H_f is the foundation or attenuation layer thickness in m, H_L is the low-permeability soil layer thickness in m, k_f is the foundation or attenuation layer hydraulic conductivity in m/s, k_L is the low-permeability soil layer hydraulic conductivity in m/s. When a hydraulic head, h_w, is applied on the top of the composite liner, the mean hydraulic gradient, i_s, through the liner and foundation is given by (Rowe 1998):

$$i_s = \frac{H_L + H_f + h_w - h_a}{H_L + H_f} = 1 + \frac{h_w - h_a}{H_L + H_f} \tag{6}$$

where h_a, the hydraulic head in m in the highly permeable layer that is not fully saturated, is often assumed to be equal to zero.

Case of a circular defect in the geomembrane

Touze-Foltz *et al.* (1999) obtained an analytical solution for a flow in the interface equal to 0 at a distance R_c from the defect centre in geomembrane corresponding to the radius of the wetted area. This equation valid for saturated conditions in the composite liner can be written as:

$$Q = \pi r_0^2 k_s i_s - 2\pi r_0 \theta \alpha \left[A_p I_1(\alpha r_0) - B_p K_1(\alpha r_0) \right] \tag{7}$$

Where

$$\alpha = \sqrt{\frac{k_s}{(H_L + H_f)\theta}} \tag{8}$$

$$A_p = -\frac{(h_w + C)K_0(\alpha R_c) - CK_0(\alpha r_0)}{K_0(\alpha r_0)I_0(\alpha R_c) - K_0(\alpha R_c)I_0(\alpha r_0)} \tag{9}$$

$$B_p = \frac{(h_w + C)I_0(\alpha R_c) - CI_0(\alpha r_0)}{K_0(\alpha r_0)I_0(\alpha R_c) - K_0(\alpha R_c)I_0(\alpha r_0)} \tag{10}$$

$$C = H_L + H_f \tag{11}$$

Where r_0 is the circular defect radius in m and h_w is the hydraulic head applied on top of the composite liner in m. In these equations K_0 and I_0 are modified Bessel functions of zero order. R_c is obtained by solving the following equation for a zero hydraulic head at radius R_c:

$$AI_0(\alpha R_c) + BK_0(\alpha R_c) - H_s = 0 \tag{12}$$

This solution even if expressed differently is strictly similar to the solution presented by Rowe (1998) for the same boundary conditions and geometric and hydraulic parameters. It has been presented in this way to show the parallelism with equations obtained for gas flow that will be presented subsequently.

Case of a damaged wrinkle of width b and a defect of infinite length and width b

In the case of a damaged wrinkle, flow is controlled by the “footprint” of the wrinkle, defined as the zone where the wrinkled geomembrane is not in contact with the underlying liner (width b in Figure 57). From a calculations point of view, there is no fundamental difference between a damaged wrinkle of width b and a defect of infinite length and width b since it is assumed that the holes in a wrinkle do not control the flow and no assumption is made regarding the height or the shape of the wrinkle. Thus, the two types of defects are defined by a single parameter: their width b (Figure 57). The analysis is two-dimensional, that is why these two types of defects are generally referred to as two-dimensional defects, and the rate of liquid flow is expressed in terms of rate of liquid flow per unit length. In steady-state conditions, for saturated conditions in the composite liner, the flow rate per unit length, Q_L, can be obtained by Equation 13, (Giroud & Touze-Foltz 2005):

$$Q_L = bk_s\left(1 + \frac{h_w}{H_s}\right) + 2\sqrt{k_s \theta h_w \left(2 + \frac{h_w}{H_s}\right)} \tag{13}$$

As highlighted by those authors, the first term of the right side of this equation quantifies the rate of flow into the soil liner (CCL or GCL) located directly under the defect. The second term quantifies the rate of interface flow.

Analytical solutions such as the ones previously presented have the advantage of being rigorous. The drawback of these tools is their complexity, particularly the existing equation for circular defects.

Empirical equations

Numerous empirical equations for predicting the flow rate through composite liners comprising a geomembrane and a CCL due to defects in geomembranes have been developed and successively updated. Giroud & Bonaparte (1989) and Giroud *et al.* (1989) developed the first sets of equations. These equations provide an approximate solution assuming that the hydraulic gradient is close to unity. This assumption may be reasonable for low leachate mounds (design mounds ranging from 0.03 to 0.3m) but are not strictly valid for the levels of leachate mounding that may occur during post-operation, in cases of excessive clogging of a LCS, or a modest leachate mound over a GCL (Rowe 1998, 2005). Aware of these limitations, Giroud *et al.* (1992) extended the approximate solution to consider higher hydraulic heads. They also proposed equations for defects of infinite length. A limitation in these equations was that they required charts to obtain the value of one of the terms of the equation.

Giroud (1997) updated previous empirical equations, providing an entirely empirical means of calculating the flow rate through defects in geomembranes. In addition, he summarised the developed equations in regard of the shapes of the defects, the liquid head above the geomembrane liner, and the contact conditions. Later on, Giroud *et al.* (1998) developed a new set of equations for calculating: (1) the rate of liquid flow through composite liners due to geomembrane defects; (2) the rate of liquid flow through defects in a geomembrane placed on a semi-permeable medium; and (3) the rate of liquid flow through defects in a geomembrane overlain by a permeable medium and underlain by a highly permeable medium.

Foose *et al.* (2001*a*) and Touze-Foltz (2001) compared the flow rate through composite liners comprising a geomembrane and a CCL calculated using either empirical equations or analytical solutions. For small circular defects, the results obtained using empirical equations developed by Giroud (1997) showed good agreement with the results obtained using analytical equations developed by Rowe (1998) and Touze-Foltz *et al.* (1999). Conversely, for defects of infinite length, the results obtained using empirical equations by Giroud *et al.* (1992) were inconsistent with the results obtained using the analytical solutions. Analysis conducted by Foose *et al.* (2001*a*) attributed this inconsistency to the fact that the empirical equations for small circular defects and defects of infinite length correspond to different values of interface transmissivity even though the same contact conditions are considered. In other words, the interface transmissivity was a function of the type of defect, which should not happen. Based on these findings, these authors proposed new empirical equations for defects of infinite length (Foose *et al.* 2001*a*) and damaged wrinkles (Touze-Foltz & Giroud 2003), based on the assumption that the transmissivity is independent from the type of defect. As a very simple analytical solution to the quantification of flow rates for longitudinal defects exists the use of the empirical equation, less precise, is not necessarily recommended. Nevertheless, in case the reader would like to use empirical equations for those defects, the use of equations developed by Touze-Foltz & Giroud (2005) have to be preferred to equations developed by Giroud *et al.* (1992) for the above mentioned reasons.

Chai *et al.* (2005) proposed a modification of equations by Giroud (1997), for circular defects, and by Touze-Foltz & Giroud (2003), for defects of infinite length, in order to consider the effect of the effective overburden pressure applied by the waste over the lining system. The modification proposed consists in multiplying the flow rate by a dimensionless correction factor that is equal to one when the overburden pressure is equal to zero.

As pointed out by Giroud *et al.* (1992), Chai *et al.* (2005) and Giroud & Touze Foltz (2005) in most cases, flow at the ends of longitudinal defects cannot in most situations be neglected in flow rates calculations. Assuming that the plan view of the defect is a rectangle with a half circle at each end as presented on Figure 58, Giroud & Touze Foltz (2005) suggested to add to the flow rate calculated thanks to the analytical solution for longitudinal defects to flow rates obtained thanks to empirical equations specifically developed for circular defects. Empirical equations for the case of large circular defects (diameters in the range 0.1 to 0.6m) were thus developed by Touze-Foltz & Giroud (2005) to quantify flow rates due to the ends of damaged wrinkles.

Following this principle, Touze-Foltz & Barroso (2006) proposed a set of empirical equations for composite liners involving GCLs for small and large circular defects (i.e. one for defect diameters ranging from 2×10^{-3} to 2×10^{-2}m and other for defect diameters ranging from 0.1 to 0.6m), as well as semi-empirical equations to predict flow rates through defects of finite length (i.e. narrow defects, such as tears, cuts or defective seams) and wide defects (such as damaged wrinkles).

Summarising, empirical equations are now available for the evaluation of flow through circular (small and large) defects in the geomembrane, narrow defects like tears, cuts or defective seams wide defects like damaged wrinkles, that consider four types of contact conditions (geomembrane-GCL, excellent, good and poor).

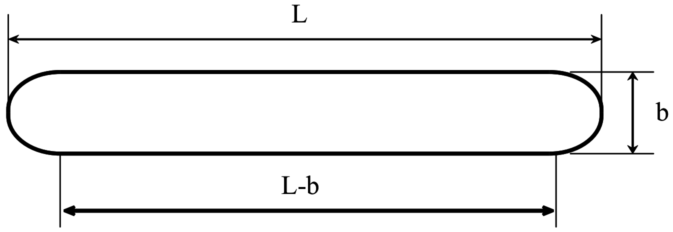


Figure 58. Plan view of a defect of finite length (from Giroud & Touze Foltz 2005)

Table 11 summarises the latest empirical equations developed for assessing the flow rate through composite liners caused by geomembrane defects that take into account the limits of validity previously mentioned in this section, especially as regards the unicity of transmissivity values, regardless of the defect type. Through this table it can be observed that an increase in hydraulic head, hydraulic conductivity or defect size leads to an increase in flow rate.

It should be noted that equations included in this table can only be used for the following values of the parameters (Touze-Foltz & Giroud 2003, Touze-Foltz & Barroso 2006):

- Hydraulic heads ranging from 0.03 to 3m;
- Hydraulic conductivities of the soil component of the composite liner ranging from 1×10^{-10} to 1×10^{-8}m/s (for excellent, good and poor contact conditions);

- Thicknesses of the soil layer component of the composite liner ranging from 0.3 to 5m (for excellent, good and poor contact conditions);
- Hydraulic conductivities of the GCL component of the composite liner ranging from 1×10^{-12} to 1×10^{-10}m/s (for geomembrane-GCL contact conditions);
- Thickness values of the GCL component of the composite liner ranging from 6×10^{-3} to 14×10^{-3}m (for Geomembrane-GCL contact conditions).

Table 11. Existing empirical equations for assessing the flow rate through composite liners due to geomembrane defects

Defect		Contact conditions	Empirical equations	Reference
Circular	2 mm< hole diameter < 20 mm	Excellent	$Q = 0.096 h_w^{0.9} a^{0.1} k_k^{0.74}\left[1 + 0.1(h_w/H_s)^{0.95}\right]$	Touze-Foltz & Giroud (2003)
		GCC	$Q = 0.21 h_w^{0.9} a^{0.1} k_k^{0.74}\left[1 + 0.1(h_w/H_s)^{0.95}\right]$	Giroud (1997)
		PCC	$Q = 1.15 h_w^{0.9} a^{0.1} k_k^{0.74}\left[1 + 0.1(h_w/H_s)^{0.95}\right]$	
		GM–GCL	$Q = 2.4\times10^{-3} h_w^{0.9} a^{0.1} k_k^{0.74}\left[1 + 0.1(h_w/H_s)^{0.95}\right]$	Touze-Foltz & Barroso (2006)
	100 mm< holes diameter < 600 mm	Excellent	$Q = 0.33 h_w^{0.84} a^{0.18} k_k^{0.77}\left[1 - 0.1(h_w/H_s)^{0.027}\right]$	Touze-Foltz & Giroud (2005)
		GCC	$Q = 0.64 h_w^{0.84} a^{0.18} k_k^{0.77}\left[1 - 0.1(h_w/H_s)^{0.027}\right]$	
		PCC	$Q = 2.604 h_w^{0.84} a^{0.18} k_k^{0.77}\left[1 - 0.1(h_w/H_s)^{0.027}\right]$	
		GM–GCL	$Q = 0.116 h_w^{0.54} a^{0.4} k_k^{0.82}\left[1 - 0.22(h_w/H_s)^{-0.35}\right]$	Touze-Foltz & Barroso (2006)
Defect of infinite length	2 mm < width < 20 mm	Excellent	$Q = 0.42 h_w^{0.45} b^{0.004} k_k^{0.87}\left[1 + 0.52(h_w/H_s)^{0.59}\right]$	Touze-Foltz & Giroud (2003)
		GCC	$Q = 0.65 h_w^{0.45} b^{0.004} k_k^{0.87}\left[1 + 0.52(h_w/H_s)^{0.59}\right]$	
		PCC	$Q = 1.64 h_w^{0.45} b^{0.004} k_k^{0.87}\left[1 + 0.52(h_w/H_s)^{0.59}\right]$	
Damaged wrinkle	100 mm < width < 600 mm	Excellent	$Q = 0.63 h_w^{0.45} b^{0.1} k_k^{0.87}\left[1 + 0.282(h_w/H_s)^{0.82}\right]$	Touze-Foltz & Giroud (2003)
		GCC	$Q = 0.89 h_w^{0.45} b^{0.1} k_k^{0.87}\left[1 + 0.282(h_w/H_s)^{0.82}\right]$	
		PCC	$Q = 1.98 h_w^{0.45} b^{0.1} k_k^{0.87}\left[1 + 0.282(h_w/H_s)^{0.82}\right]$	

Q = flow rate in m^3/s; Q_L = flow rate per unit length in m^2/s; h_w = hydraulic head on top of geomembrane in m; a = circular defect area in m^2; b = width of defect of infinite length or damaged wrinkle in m; k_s = soil layer hydraulic conductivity (in case of composite liners involving GCLs, it represents the equivalent hydraulic conductivity of the soil liner plus the GCL) in m/s; H_s = soil liner + GCL layer thickness in m; and θ = transmissivity of the interface in m^2/s.

Numerical methods

One limitation of the analytical solutions presented in the previous section is that they assume total saturation of the soil liner. This implies a restriction of the validity of those equations on a limited area where saturation can be guaranteed, which is the so-called wetted area. Furthermore, as soil liners and GCLs are not initially saturated when installed, the question arises of the possibility to take account of this partial saturation in the quantification of flow. The only existing solution at the moment to study this point is numerical modelling.

Cartaud *et al.* (2005*a*) proved through numerical modelling using a finite element model solving Richards' equation that the initial hydration of the CCL has a limited impact on the flow through composite liners. Saidi *et al.* (2006) undertook a similar study with the same numerical code, for composite liners incorporating GCLs where significant differences between flow rates obtained thanks to analytical solutions and numerical modelling were observed. Those were attributed to the significant discrepancy between wetted area corresponding to the saturated zones calculated thanks to analytical solutions and empirical equations thus showing the importance of taking partial saturation of GCLs into account. They also studied the impact of the shape of the end of longitudinal defects on flow rates. Results obtained tended to show that while considering a circular or a square end of defect does not change much to the result, the way the longitudinal defect is decomposed will have, thus suggesting that a more precise result would be obtained through 3D numerical modelling while quantifying advective flow.

The impact of non-uniformity of interfaces opening on advective flow could also be investigated though numerical modelling. Cartaud *et al.* (2005*b*) investigated the influence of the position of a circular hole in the geomembrane of the composite liner. They could show the importance of the respective positions of the hole and of non-uniformities of the interface opening on advective flow rates that cannot be accounted for through empirical equations or analytical solutions.

Numerical modelling can also be a useful tool while quantifying the possible hydraulic interaction between defects. If analytical solutions exist for this purpose when one is dealing with longitudinal defects, no analytical solution allows to investigate the influence of the distance between adjacent defects on the advective flow. A recent investigation of this point was performed by Saidi *et al.* (2008) for geomembrane-GCL composite liners, putting in light a very limited hydraulic interaction between adjacent square holes in the geomembrane.

Navarro *et al.* (2008) evaluated the impact of a hole on the flow at the interface of a composite liner on slope where a geotextile was placed to facilitate drainage of small water impoundments. They developed a simple model coupling a piston model to account for the flow into the underlying soil and a plane flow in the geotextile.

Illustration of flow rates quantification using analytical solutions

Figure 59 presents the leakage obtained for poor, good and geomembrane-GCL contact conditions respectively defined by equations 3, 2 and 4. Two circular holes radii were taken into account in the calculations, respectively equal to 10^{-3}m for small holes and 10^{-2}m for large holes. Calculations were performed assuming a 1m thick clay layer with a hydraulic conductivity equal to 10^{-9}m/s located on top of a 5m thick attenuation layer with a hydraulic conductivity equal to 10^{-6}m/s, corresponding to the basic configuration of French landfills. In case a GCL is located under the geomembrane its hydraulic conductivity was supposed to be equal to 2×10^{-10}m/s taking account of ageing of the GCL based on values that will be presented in the section dedicated to migration and attenuation through GCLs of this Keynote Lecture. The GCL used as a reinforcement layer is underlain by a 1m thick CCL with a hydraulic conductivity equal to 10^{-9}m/s. The hydraulic head on top of the composite liner is equal to 0.3m. As shown by Rowe (2005) a large number of circular holes would be necessary to reach main ranges of flow rates observed by Bonaparte *et al.* (2002), in case of geomembrane-CCL composite liners. With the value of interface transmissivity adopted for geomembrane-GCL composite liners which is equal to 6.7×10^{-10}m^2/s based on Equation 4, a reasonable number of small circular holes can explain the flow rates observed. The conclusion obtained by Rowe (2005) was different. Indeed the transmissivity value he adopted was 2×10^{-10}m^2/s leading to larger number of holes required to account for mean measured flow rates.

Additional points were plotted on Figure 59, obtained for a repartition of defects including 14 circular holes, 4 cuts and a damaged wrinkle. Small defects correspond to 10^{-3}m diameter circular holes, 10^{-3}m wide and 0.1m long cuts and 0.15m wide and a 10m long wrinkle. Large defects correspond to 10^{-2}m diameter circular holes, 10^{-2}m wide and 0.3m long cuts and 0.3m wide and a 30m long wrinkle. In the calculations performed, flow due to the ends of longitudinal defects either cuts or wrinkle was taken into account. In this case, results obtained show that flow rates obtained can be explained taking account of the mean density of defects observed on site thanks to electrical surveys, including a reasonable number of large defects (only one damaged wrinkle).

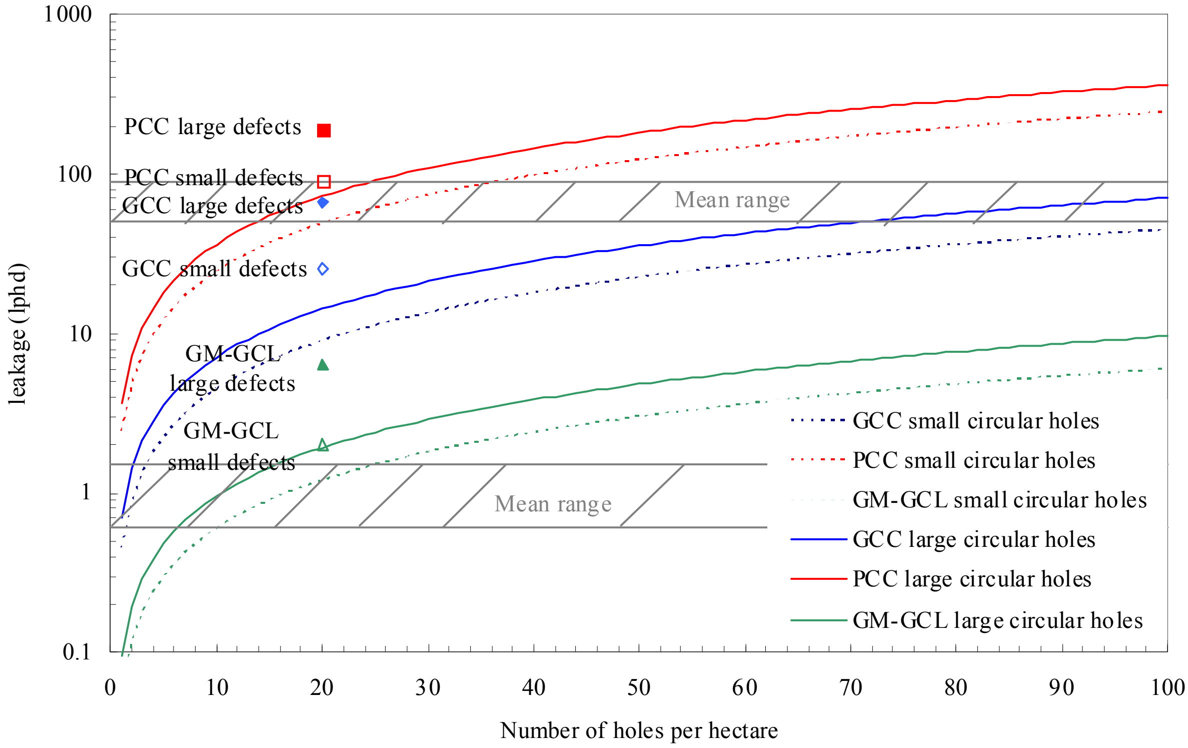


Figure 59. Comparison of calculated and measured leakage rates for various contact conditions and defects repartitions (adapted from Rowe 2005).

Gas flow quantification

Bouazza & Vangpaisal (2006, 2007) recently addressed the quantification of flow rates through geomembrane-GCL composite liners. Bouazza *et al.* (2008) recently proposed analytical solutions, on a set of hypothesis close to the one presented in the previous section dedicated to liquid flow modelling through composite liners. The model assumes that gas flow through a defect in the geomembrane of a geomembrane-GCL composite liner consists of flow through the underlying GCL followed by a radial flow in the interface to the circular defect in the geomembrane. Thus the reverse mechanism from the one assumed regarding liquid flow in the previous section is assumed here, as show on Figure 56*b*. No equations are developed at the moment for two-dimensional defects contrarily to what was done for liquid flow. The proposed model is function of differential gas pressure, moisture content of the GCL, the interface transmissivity and hole diameter. The flow rate can be calculated thanks to the following equation:

$$Q = -\frac{\pi r_0}{\gamma P_0}\left[\frac{K_g r_0}{2L}\left(P_0^2 - P_1^2\right) - \sqrt{\frac{K_g \theta}{L}}\left(AI_1(\lambda r_0) - BK_1(\lambda r_0)\right)\right] \quad (14)$$

Where r_0 is the hole radius in m, γ is the gas unit weight in $kg/(m.s)^2$, P_0 and P_1 are respectively the outlet and the inlet pressures in kPa, K_g is the cross plane gas permeability of the GCL in m/s, L is the GCL thickness in m, θ is the gas interface transmissivity in m²/s which can be measured according to the procedure presented by Bouazza & Vangpaisal (2006), A and B are constants that can be calculated depending on the boundary conditions adopted and I_1 and K_1 are modified Bessel functions of first order. λ is given by the following equation:

$$\lambda = \sqrt{\frac{K_g}{\theta L}} \quad (15)$$

Which is identical to the coefficient α previously presented for liquid flow through composite liner when one changes H_s to L. Two different boundary conditions were defined by Touze-Foltz *et al.* (1999) that are specific head conditions and no flow boundary conditions at the edges of the interface for liquid through composite liners that were used by Bouazza & Vangpaisal (2008). In case of no flow boundary condition at a distance R from the circular defect centre, Bouazza & Vangapaisal (2008) obtained:

$$A = \frac{\left(P_0^2 - P_1^2\right)K_1(\lambda R)}{K_1(\lambda R)I_0(\lambda r_0) + K_0(\lambda r_0)I_1(\lambda R)} \quad (16)$$

$$B = \frac{\left(P_0^2 - P_1^2\right) I_1(\lambda R)}{K_1(\lambda R) I_0(\lambda r_0) + K_0(\lambda r_0) I_1(\lambda R)} \tag{17}$$

In case of a specific pressure equal to P_1 at a distance R from the hole centre A and B are respectively given by Equations 18 and 19:

$$A = \frac{-\left(P_0^2 - P_1^2\right) K_0(\lambda R)}{K_0(\lambda r_0) I_0(\lambda R) - K_0(\lambda R) I_0(\lambda r_0)} \tag{18}$$

$$B = \frac{\left(P_0^2 - P_1^2\right) I_0(\lambda R)}{K_0(\lambda r_0) I_0(\lambda R) - K_0(\lambda R) I_0(\lambda r_0)} \tag{19}$$

Those expressions are very close from those presented in the case of liquid flow in Equations 9 and 10.

A good agreement was obtained between analytical solutions and experimental results from Bouazza & Vangpaisal (2006) for specimens with moisture contents greater than 70%. For lower moisture contents, the model predictions overestimated the experimental results, probably in relation with the change in the gas flow pattern from the adopted conceptual gas flow model.

MIGRATION AND ATTENUATION THROUGH GCLS

Introduction

Any assessment of long-term environmental impacts from landfills requires contaminant transport modelling (Rowe 2005). One dimensional contaminant transport for a single reactive solute without degradation, through a saturated GCL can be modelled using the following equation, when one is neglecting first order decay (Rowe *et al.* 2004):

$$n_t \frac{\partial C}{\partial t} = \left(n_t D_t \frac{\partial^2 C}{\partial z^2} - \nu_a \frac{\partial C}{\partial z} \right) - \rho K_d \frac{\partial C}{\partial t} \tag{20}$$

where C is the concentration in the GCL in kg/m^3 at depth z in m and time t in s, n_t is the total porosity of the GCL, D_t is the diffusion coefficient deduced from the total porosity in m²/s, ν_a is the Darcy velocity in m/s, ρ is the dry density in kg/m^3, and K_d is the partitioning coefficient in m^3/kg.

Evidence has been presented to show that diffusive transport (contaminant migration driven by the difference in concentration between the upper and lower portions of the liner) is often the dominant mode of contaminant transport and that liner analyses based solely on leakage rate can be misleading (Foose *et al.* 2002). However, for large hydraulic heads or increases in hydraulic conductivity of a GCL, advection can become an important transport mechanism. Consequently hydraulic conductivity of GCLs to leachate and mining solutions will be discussed in this section as well as diffusion and sorption through GCLs. Finally a brief overview of oxygen diffusion through GCLs will be given. As no recent works have been published regarding diffusion through geomembranes, this topic will not be discussed herein. Published values for the partitioning coefficient and diffusion coefficient for geomembranes have been summarised by Rowe *et al.* (2004).

Hydraulic conductivity of GCLs to leachate and mining solutions

A variety of studies has shown that the hydraulic conductivity and swelling of bentonite can be influenced by inorganic permeant solutions (Alther *et al.* 1985, Ashmawy *et al.* 2002, Egloffstein 1997, Egloffstein 2001, Guyonnet *et al.* 2005, Jo *et al.* 2001, Petrov & Rowe 1997, Quaranta *et al.* 1997, Ruhl & Daniel 1997, Shackelford *et al.* 2000, Shan & Daniel 1991, Shan & Lai 2002, Vasko *et al.* 2001). In many of these studies, focus was placed on the type of hydrating liquid and permeating liquids. Table 12 provides a synthesis of GCL testing conditions and main results obtained by some of these authors for permeation of GCLs with leachate, mining solutions fluids or specific fluids. Table 13 provides the main characteristics of solutions utilized in the studies discussed in Table 12.

Influence of prehydration

Despite variations in bentonite, leachate composition, experimental devices and testing conditions, some general trends can be observed regarding the effect of prehydration and its influence on hydraulic conductivity after permeation with leachate. The type of prehydration fluid appears to be a possible factor in the observed hydraulic conductivity. According to Didier & Comeaga (1995) a lower hydraulic conductivity can be obtained when the first wetting liquid is water rather than a chemical solution or leachate. Nevertheless Petrov & Rowe (1997) observed that for a synthetic MSW leachate, the initial hydrating medium did not appear to influence hydraulic conductivities at a confining stress of about 33kPa, suggesting no potential hydraulic benefits of a water-hydrated bentonite core;

however those authors indicate that hydration with water would still be preferred. This result is consistent with finding from Shan & Lai (2002) who determined the hydraulic conductivity of two GCLs. They observed that even if the GCLs were hydrated with seawater, acid rain water, or leachate, they would remain to be an effective hydraulic barrier to leachate. Indeed, the hydraulic conductivity was controlled by the last permeant fluid. Nevertheless as the number of pore volumes in tests presented in the study by Shan & Lai (2002) was rather low this conclusion is questionable. The GCL containing natural sodium bentonite tested by Guyonnet *et al.* (2005) showed a better hydraulic behaviour to a MSW leachate than to a 10^{-3}M sodium chloride solution, in relation with the presence of ammonium in the real leachate. This result is not consistent with what is usually observed with single-salt species solutions where the influence of the first hydrating fluid is very important (Petrov & Rowe 1997, Ruhl & Daniel 1997, Shackelford *et al.* 2000). This may be linked with the fact that the equilibrium is not reach in the various tests or that multiple-salt species solutions have a different impact on bentonite than singles-salt species solutions.

Table 12. Synthesis of hydraulic conductivities to leachate or mining solutions from the literature.

Source	**GCL type**	**Bentonite type mass/unit area**	**Load (kPa)**	**Hydrating fluid**	**Permeating fluid**	**Pore volume**	**Hydraulic conductivity (m/s)**
Didier & Comeaga (1995)	NP	Na 5kg/m²	8	DW	DW	20	3×10^{-11}
				DW	Leachate	36	6×10^{-11}
				Leachate		30	5×10^{-8}
	NP	Na 5kg/m²		DW	DW	18	2×10^{-11}
				DW	Leachate	30	$4.5\text{x}10^{-11}$
Petrov & Rowe (1997)	NP	Na 3.9kg/m²	33	DW	Synthetic MSW	4.5 to 9	8.8×10^{-11} to 2.1×10^{-10}
				Synthetic MSW leachate		18	7.3×10^{-11}
Ruhl & Daniel (1997)	NP	Na 5kg/m²	35	Synthetic MSW leachate		> 2	2×10^{-8}
				Synthetic HW leachate		> 2	1×10^{-11}
				MSW leachate		> 2	$<1\times10^{-12}$
	CR NP	5kg/m²		Synthetic MSW leachate		> 2	1×10^{-9}
				Synthetic HW leachate		> 2	8×10^{-12}
				Simulated fly ash leachate		> 2	6×10^{-12}
				MSW leachate		> 2	2×10^{-10}
	AB	Na 3.6kg/m²		Synthetic MSW leachate		> 2	3×10^{-10}
				Synthetic HW leachate		> 2	1×10^{-10}
				Fly ash leachate		> 2	2×10^{-11}
				MSW leachate		> 2	7×10^{-12}
Schroeder *et al.* (2001)	PH	Na	15	Purified water			5.4×10^{-13}
				Water after 2.5 years			1.2×10^{-11}
				Real leachate after 2.5 years			2×10^{-11} to 4.6×10^{-11}
				Synthetic leachate			2×10^{-11} to 4×10^{-11}
Shan & Lai (2002)	NP	3.6kg/m²	34.5	Tap water		0.2	4.4×10^{-11}
				Seawater		6	1.7×10^{-7}
				Acidic water		2.7	2.5×10^{-11}
				MSW leachate		1	3×10^{-11}
				Tap water	MSW leachate	1.3	3.7×10^{-11}
				Seawater	MSW leachate	5	1.5×10^{-9}
				Acidic water	MSW leachate	1.8	4.8×10^{-11}
	AB	3.6kg/m²		tap water		0.2	4.8×10^{-11}
				Seawater		3	1.2×10^{-8}
				acidic water		1	2.8×10^{-11}
				MSW leachate		1.3	2.6×10^{-11}
				Tap water	MSW leachate	1.7	1.9×10^{-11}
				Seawater	MSW leachate	13	2×10^{-11}
				Acidic water	MSW leachate	1.5	2.3×10^{-11}
Guyonnet *et al.* (2005)	NP	Na 5.3kg/m²	10	NaCl 10^{-3}mol/l	NaCl 10^{-3}mol/l	3	1×10^{-11}
					MSW leachate	3	1.4×10^{-11}
	NP	Ca act			NaCl 10^{-3}mol/l	6	1.7×10^{-10}

Source	GCL type	Bentonite type mass/unit area	Load (kPa)	Hydrating fluid	Permeating fluid	Pore volume	Hydraulic conductivity (m/s)
		4.5kg/m²			MSW leachate	7	5×10^{-11}
Katsumi *et al.* (2007)	NP	Na 4.73kg/m²	20-30	Leachate A		20	1×10^{-11}
				Leachate H		20	1.95×10^{-11}
				Leachate S		20	1.89×10^{-11}
				Leachate K		325	$6,16\times10^{-9}$
				Leachate K, 1 year		220	5.19×10^{-10}
				Leachate K	LK x 1.5 diluted	50	1.23×10^{-10}
					LK x 2 diluted	50	1.09×10^{-10}
					LK x 4 diluted	50	8.66×10^{-11}
					LK x 8 diluted	35	1.2×10^{-10}
					LK x 16 diluted	35	1.17×10^{-10}
					LK x 32 diluted	35	8.14×10^{-11}
					LK x 64 diluted	35	5.5×10^{-11}
Rauen & Benson (2008)	NP	Na 3.66kg/m²	70	DI water		8	8.3×10^{-10}
			70	Conventional leachate		8	5.8 to 6.7×10^{-10}
			70	Recirculation leachate		8	0.92 to 1×10^{-9}
Brown & Schackel-ford (2007)				Deionized water, aerobic		20	1.3×10^{-11}
	NP	Na	34.5	Deionized water, aneaerobic		20	1.3×10^{-11}
				SAWS, aerobic		75	5.5×10^{-11}
				SAWS, aneerobic		26	1.4×10^{-11}
Lake *et al.* (2008)			30	DW	DW		4.3×10^{-12}
			30	DW	SL	9	1.5×10^{-11}
			90	DW	DW		3.5×10^{-12}
			90	DW	SL	6	1.0×10^{-11}
Lake *et al.* (2007)	NP	Na 5.3 kg/m²	56	DW	DW	4	6×10^{-12}
			56	DW	Al solution 0.0185M	4	4.6×10^{-11}
Lange *et al.* (2007)	NP, TL	Na 5.5 - 6.6 kg/m²	25	DDW	Synthetic ARD	5	5×10^{-11}
						21	1.3×10^{-10}
					Synthetic GMT	5	1.7×10^{-11}
						21	3.5×10^{-11}
					DDW	5	1.6×10^{-11}
						21	2×10^{-11}
Benson *et al.* (2008)	NP	Na	50	Tap water	Tap water	8	1.1 to 1.2×10^{-11}
				Tap water	Al rich leachate	8	2.3 to 2.7×10^{-11}
				Al rich leachate		8	0.15 to 2×10^{-9}
		Ca activated		Tap water	Tap water	8	1.6 to 3.6×10^{-11}
				Tap water	Al rich leachate	8	0.42 to 1.8×10^{-8}
				Al rich leachate		8	5.1 to 2×10^{-7}

AB: adhesive bounded
ARD: acid rock drainage
CR: contaminant resistant
DW: distilled water
DDW: deionized distilled water
GMT: gold mine tailings
NP: needle punched
PH: prehydrated
SAWS: simulated animal waste solution
SL: simulated livestock mortality leachate
TL: thermally locked

Table 13. Main Concentration of main cations (in mg/l), anions (in mg/l) and index characteristics for leachate and mining solutions mentioned in this section

	Study Number (see legend below)														
Property	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
pH	5.4	6.23	4.4		7		7.43	8.1	7.77	8.19	6.1	6.4	6.8	3.3	12.2
Conductivi-ty (S/m)	1.16					1.7			1.24	2.25	0.61				4.33
CS (mM)									56.7	112					
IS (mM)									100	210	59				774
RMD ($M^{1/2}$)									1.16	1.11					1.15
H															
Be								0.024							
B								9.9							0.89
F									<0.01	<0.01					
Na	460	1615			368	4930		1748	1800	2980	502	1800	650	785	9416
Mg	400	473			100	104		102	127	448			92	1	0.04
Al									<0.5	<0.5			4	91	3470
K	780	354				890		1521	445	821		1100	11	667	95.06
Ca	1200	1224	10^3		112	89		84	27.6	4.2	139	200	116	4.5	1.89
Cr						0.881	0.37	0.8							
Mn								0.297	0.03	0.37			1	23	0.02
Fe							1.21		<0.8	<0.8			1.2	218	0.11
Co								0.020							
Ni						0.847		0.19						20	
Cu							0.031		<0.05	<0.05				18	0.09
Zn							0.61	0.169	0.12	0.2				102	0.21
As								0.210					4.5	2.5	
Cd				<200			0.0037						12.5	6.4	
Sr								0.039							
Ba								0.192							
Pb							0.11	0.013							
NH_4	500	618				57		1980			379	1900	-	20	
P															1.36
S						87							502	1034	315
Cl	3989	4414			520	197	959.2	2804	1889	2982	1697	3300	219	1591	
Br									17.7	12.4					
NO_3		40				266			<0.01	<0.01	0		5	-	
NO_2									<0.01	<0.01					
OH^-	319					493									
HCO_3		4876				437		9760	4034	7996			128	-	
SO_4^{2}	384	137			4340	58	23.7	54.7	322.5	30.3		3900			
CO_3^{2}		156				5070			11	57.3			182	-	
CH_3CO_2	2360														
HPO_4		18													
PO_4									6.2	7.2	150				

CS: Cationic strength
IS: Ionic Strength
RMD: ratio of total molarity of monovalent cations to square root of total molarity of divalent and polyvalent cations.
1: Didier & Comeaga (1995), 2: Petrov & Rowe (1997) plus trace metals, 3 to 5: Ruhl & Daniel (1997) respectively synthetic MSW leachate, simulated HW leachate and real MSW leachate, 6: Schroeder *et al.* (2001), 7: Shan & Lai (2002), 8: Guyonnet *et al.* (2003), 9 & 10: recirculation and conventional leachate from Rauen & Benson (2008), 11: Brown & Schackelford (2007), 12: Lake *et al.* (2008), 13 & 14: GMT and ARD from Lange *et al.* (2007), 15: Benson *et al.* (2008).

Influence of the concentration of the permeant solution

Many studies show that the hydraulic conductivity of GCLs is sensitive to the concentration of the permeant solution and the cation valence. In general, the highest hydraulic conductivity and the lowest swelling were obtained in concentrated solutions or solutions with a preponderance of divalent cations. Nevertheless many of these studies have shown that GCLs maintain low hydraulic conductivity in the range 1×10^{-12} to 1×10^{-10}m/s when permeated with real MSW leachate, HW leachate, ARD solutions or simulated fly ash leachate (Didier & Comeaga 1995, Guyonnet *et al.* 2005, Lange *et al.* 2005, 2007, Ruhl & Daniel 1997, Schroeder *et al.* 2001, Shan & Lai 2002).

Impact of leachate compared to water

Didier & Comeaga (1995) analysed the total or partially hydrated behaviour of GCLs after a long time exposure to leachate. It was found for the leachate saturated GCLs that the hydraulic conductivity increased by three orders of magnitude compared to the hydraulic conductivity to water, from 3×10^{-11} to 5×10^{-8}m/s.

Brown & Schackelford (2007) evaluated the potential use of a GCL as the primary hydraulic barrier for animal waste lagoons on the basis of hydraulic conductivity testing. The GCL was permeated under both aerobic and anaerobic conditions with both deionized water and a simulated animal waste solution to determine the effect, if any, of the simulated solution on the hydraulic conductivity of the GCL. The hydraulic conductivity of the GCL increased only slightly (8%) on average when permeated with the simulated solution under anaerobic conditions relative to the value of 1.3×10^{-11}m/s based on permeation with deionized water, but increased by a factor of 4.2 on average when permeated with the simulated solution under aerobic conditions.

Lake *et al.* (2008) describe a GCL hydraulic conductivity testing program being utilized to assess the potential use of GCLs for small on-farm burial applications of typical livestock mortalities. The polymer coated GCL exposed to a simulated livestock mortality leachate exhibited less than an order of magnitude increase in hydraulic conductivity relative to that of water for the preliminary pore volumes tested (6 to 9).

This review also reveals that real leachates were generally less aggressive than simulated leachates. For the study performed by Ruhl & Daniel (1997), relative to the simulated leachate tested, the real MSW leachate:

- Contained roughly equal amounts of monovalent and polyvalent cations;
- Contained suspended solids and micro-organisms that tended to plug the pores of the bentonite; and
- Produced gases of decomposition that tended to block flow paths.

Influence of recirculated leachate

Rauen & Benson (2008) found no significant influence of a leachate on the hydraulic conductivity of a GCL containing natural sodium bentonite whether a conventional leachate or a recirculation leachate was used. Nevertheless their findings represent a point in time as exchange reaction with influent cations were still occurring after one year of permeation (8 pore volumes of flow).

Influence of mining solutions

Lange *et al.* (2007) investigated the capacity of GCLs to attenuate metals and metalloids from acid rock drainage (ARD) water and a neutral-pH, As-rich water associated with gold mine tailings (GMT). The thermally locked needle punched GCL studied consisted of a scrim reinforced-nonwoven carrier geotextile and a nonwoven cover geotextile encapsulating a layer of granular sodium bentonite 5.5 to 6kg/m². The long term hydraulic conductivity of the GCLs increased from 1.6×10^{-11}m/s (water for 5 pore volumes) to 5×10^{-11}m/s and 1.3×10^{-10}m/s after permeation with the GMT and ARD waters respectively (21 pore volumes). The different behaviour of the same metal in the different mining solutions tested demonstrated how the occurrence of multiple ionic species in solution can complicate the transport process, producing unanticipated chemical combinations due to opposing or collaborative effects.

Impact of aluminium solutions

Lake *et al.* (2007) studied the permeability of a needle punched reinforced GCL containing sodium bentonite to aluminium solutions simulating aluminium-rich residual solids produced from water treatment plants. Permeation with 0.0185M and 0.015M aluminium sulphate solutions was performed respectively under 56kPa and 114kPa normal stresses. The GCL maintained a hydraulic value lower than 5×10^{-11}m/s. Substantial amounts of aluminium attenuation were observed in the hydraulic conductivity test most likely as a result of cation exchange. Additional research presented on the same kind of solution by Benson *et al.* (2008) showed that permeation with leachate from an aluminia refinery after 8 pore volumes of flow led to a significant impact on the hydraulic conductivity, with differences observed based on the nature of the bentonite contained in the GCL.

Prehydration also had a significant effect on the hydraulic conductivity. It was found that the hydraulic conductivity of the GCL containing an Australian bentonite was 60 to 800 times larger to leachate than to tap water. For the calcium activated Chinese bentonite, the ratio of hydraulic conductivities ranged between 120 and 390.

Influence of the solution electrical conductivity

Katsumi *et al.* (2007) conducted GCL hydraulic conductivity tests to leachate for time periods up to three years. In the range of low electrical conductivity, the hydraulic conductivity for inorganic solutions such as NaCl and $CaCl_2$ appeared to estimate rather well the barrier performance of GCLs as compared to the real waste leachate having similar electric conductivity.

Influence of calcium carbonate

Guyonnet *et al.* (2005) investigated the evolution of two GCLs hydraulic conductivity after prolonged contact with four fluids, including one real leachate. Results suggested that calcium carbonate in the bentonite formed during activation of the calcium bentonite may re-dissolve during contact with a dilute permeant, releasing calcium ions that exchange with sodium in the clay.

Influence of polymer treatment

Ashmawy *et al.* (2005) examined the advection, diffusion and sorption characteristics of untreated and polymer-treated bentonite clays. Contaminant-resistant bentonite maintained a lower hydraulic conductivity than sodium bentonite when the clay was prehydrated and in case of basic permeants.

Influence of test duration

It is important to also point out the importance of ensuring GCL compatibility tests are performed for sufficient duration. Shackelford *et al.* (2000) pointed out that termination of hydraulic conductivity tests involving prehydrated GCLs before chemical equilibrium is established, might result in measured hydraulic conductivities that do not represent equilibrium and might be unconservatively low. Accordingly, values of hydraulic conductivities presented in Table 12 obtained for low number of pore volumes must be considered with caution, as they may be conservatively low.

Correlation between free swell and hydraulic conductivity

Egloffstein *et al.* (2002) indicate that the result of free swell tests could be used to detect the leachate effect on GCLs. It can be argued that even if the free swell test method has a low precision due to water entrapping or material settling, this test gives an interesting overview of the material-fluid interactions. Figure 60 shows the relationship between free swell and hydraulic conductivity of GCL bentonite from several authors. These results need to be considered carefully as hydraulic conductivity methods differ from author to author. However results tend to show that high swelling bentonites generally correspond to a low hydraulic conductivity and vice versa.

Katsumi *et al.* (2007) noticed that GCLs hydraulic conductivity are lower than 1×10^{-10}m/s when free swell is larger than 15ml/2g regardless of the type and concentration of the permeant solution. Nevertheless Ashmawy *et al.* (2005) noticed that the swell index cannot be used as a universal indicator of the hydraulic conductivities to leachate nor of the ratio of the hydraulic conductivity to leachate to the hydraulic conductivity to water for all bentonites, as illustrated by the degree of scatter in Figure 60.

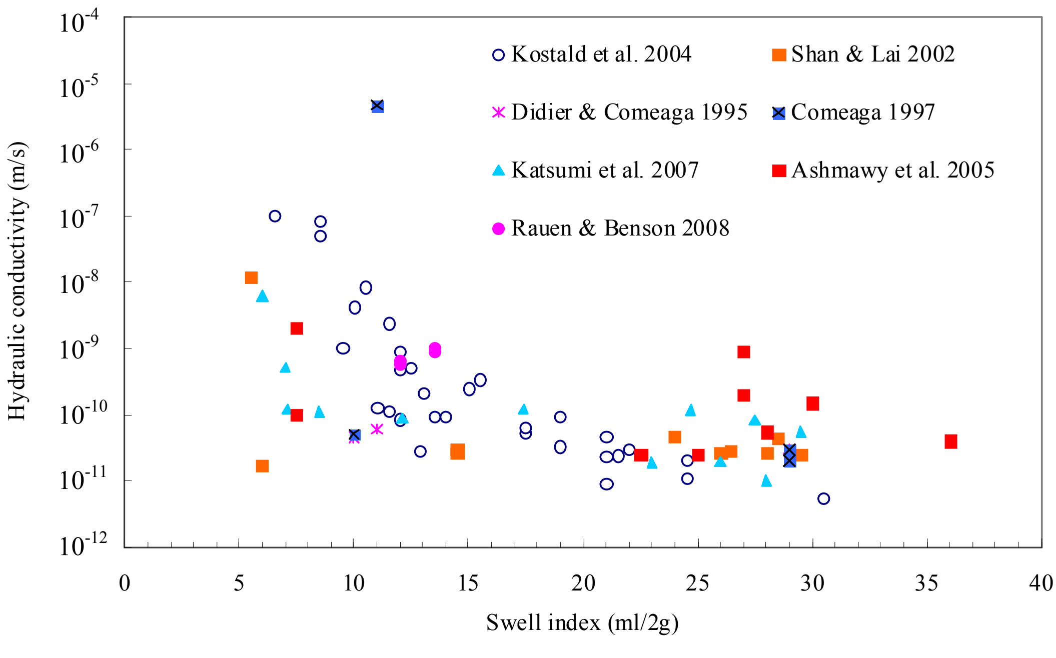


Figure 60. Relationship between hydraulic conductivity and swell index (modified from Kolstad *et al.* 2004)

Diffusion and sorption through GCLs

Inorganic species

Lake & Rowe (2000*a*) examined the diffusion of sodium chloride through three GCLs containing sodium bentonite, either granular or powdered with masses per unit area greater than 5.3kg/m², all being needle punched. The effect of final bulk GCL void ratio, contaminant concentration and test method was also investigated. The concept of bulk GCL void ratio was first introduced by Petrov & Rowe (1997) in order to homogenize the effects of variable mass of bentonite on GCL thickness. Bulk GCL void ratio is defined as the ratio of volume of voids within the geotextile and bentonite components of the GCL to the volume of voids within the GCLs. Further details on the computations can be found in Petrov & Rowe (1997). Two types of diffusion tests were performed. Specified volume tests allowed comparison of diffusion result by controlling the final bulk GCL void ratio while constant stress diffusion tests allowed comparison of diffusion coefficients at similar specified applied stresses. Lake & Rowe (2000*a*) showed that the diffusion coefficient decreased linearly with decreasing final bulk GCL void ratio. They also showed that the diffusion coefficient is dependant on the source solution: when the NaCl concentration was increased significantly from 0.05-0.08M to 0.6-2.0M there was an increase in the diffusion coefficients. This increase was largely mitigated when a constant stress was applied to the sample through the testing as compared to test performed at constant void ratios. The method of GCL manufacture (type of geotextiles, bentonite impregnation, needle punched fibers being thermally treated) did not significantly affect the diffusion coefficients at a given void ratio; however, the method of manufacture can influence swelling (Lake & Rowe 2000*b*) and hence the diffusion coefficient especially for samples hydrated at low stress (Rowe 2005). For the range of conditions examined, the chloride diffusion coefficient was generally between 1×10^{-10}m²/s and 4×10^{-10}m²/s.

Pivato & Raga (2006) studied the sorption of ammonium on bentonite powder and a GCL through batch tests. Tests were performed with real leachate in order to take into account the competition effect of different compounds in the leachate. The results obtained showed that the sorption was non-linear for the range of concentrations utilized. The partitioning coefficient values obtained ranged between 4.9 and 6.6l/kg for NH_4^+ confirming that ammonium adsorption was higher in bentonite than in other materials used in landfill liners.

Ouhadi *et al.* (2006) studied the interaction between bentonite and several heavy metals (Zn, Pb) at different concentrations and various pH levels. Focus was placed on the chemical, desorption response and degradation of buffering capability of bentonite subjected to heavy metal contaminants. A series of batch equilibrium tests were performed. They showed that the heavy metal retention decreased when H concentration is increased. The highest adsorption occurred at pH levels higher than 6. At pH lower than 4, the retention potential decreases as much as 50%.

Lake *et al.* (2007) studied the diffusion of aluminium at an effective stress of 100kPa through the same GCL as previously presented regarding permeability to aluminium sulphate solutions. The initial aluminium concentration was 0.037M. Batch test were also performed. Those tests resulted in a sorption coefficient value of 30l/kg and a diffusion coefficient equal to 1.5×10^{-10}m/s. Batch testing suggested high sorption at low concentrations and a non-linear sorption behaviour for the range of concentrations utilized.

Lange *et al.* (2007) studied the capacity of a GCL to attenuate metals and metalloids like As, Al, Cd, Cu, Fe, Mn, Ni, Sr and Zn, from mine ARD water and neutral-pH As-rich water associated with GMT. Gypsum precipitation occurred in the GMT samples and was responsible for Ca and S attenuation and could have been responsible for Cd and As attenuation. Cd remained more mobile than the other metals, experiencing earlier breakthrough times and smaller soil concentrations. Lange *et al.* (2007) also observed that the precipitation of the ferrihydrite occured in the ARD samples and gypsum occurred in the GMT samples. These minerals were responsible for retention of metals in addition to the cation exchange in the GCL. Effective diffusion coefficients for Ni, Cd, Zn and Mn given by Lange *et al.* (2008) ranged from 7.6×10^{-11}m^2/s for Ni to 9.9×10^{-11}m^2/s for Mn.

Volatile organic compounds

For older landfills, volatile organic compounds (VOCs) can be ubiquitous in landfill leachate (Edil 2007). Investigations presented by Edil (2007) for landfills in Wisconsin indicate that the potential for VOC migration remains a problem associated with clay and composite liners as shown on Figure 61. Both numerical analyses and field data imply that the current state of practice in Wisconsin to contain VOCs is not adequate and VOCs may present a potential environmental problem as time passes. Then a careful review of landfill containment design with a focus on VOCs, as presented by Rowe *et al.* (2004), is needed to prevent wide-spread groundwater contamination around landfills in time. It is thus of interest to study diffusion of VOCs through GCLs.

Lake & Rowe (2004) examined diffusion of dichloromethane (DCM), 1,2 dichloroethane (DCA), trichlorethylene, (TCE), benzene and toluene through a GCL at room temperature. The GCL was a nonwoven carrier, nonwoven cover thermally treated needle punched fibres GCL containing granular natural sodium bentonite. They concluded that the order of the rate of mass transport through the GCL was DCM≈DCA>benzene>TCE>toluene. The diffusion coefficients at room temperature and at confining pressure lower than 10 kPa ranged from approximately 2×10^{-10}m²/s to 3×10^{-10}m²/s. The difference in mass transport was attributed largely to varying degrees of sorption of the different compounds to the geotextile components of the GCL and to the bentonite. Given that VOC diffusion coefficients were obtained at relatively high GCL bulk void ratios compared with what would be encountered on the field, the results fall in the upper range of VOC diffusion coefficients expected for the case of a GCL in a base liner situation.

Rowe *et al.* (2005) extended this work by examining the effect of temperature on the diffusion of BTEX (benzene, toluene, ethylbenzene, m&p-xylene and o-xylene) through a needle punched reinforced GCL. This GCL with a mass per unit area of about 4.2kg/m² was composed of granular sodium bentonite encapsulated between a scrim reinforced nonwoven polyproplylene carrier and a staple fiber nonwoven PP cover geotextile. Tests were performed under a 9.5kPa load. Rowe *et al.* (2005) confirmed that the geotextile component of the GCL was the primary contributor to sorption of hydrocarbons by the GCL. The diffusion coefficients followed the order benzene>toluene>ethylbenzene>m&p-xylene≈o-xylene. Partitioning coefficients for the entire GCL followed the order m- and p-xylene>ethylbenzene>o-xylene>toluene>benzene (see Table 14). The reduction in both the diffusion and partitioning coefficients with decreasing temperature from 22°C to 7°C had opposite effects on mass transport through the GCL. However the decrease in transport due to a reduced diffusion coefficient is more significant than the increased transport due to smaller sorption and the net effect was reduced mass transport at lower temperature (Rowe 2005).

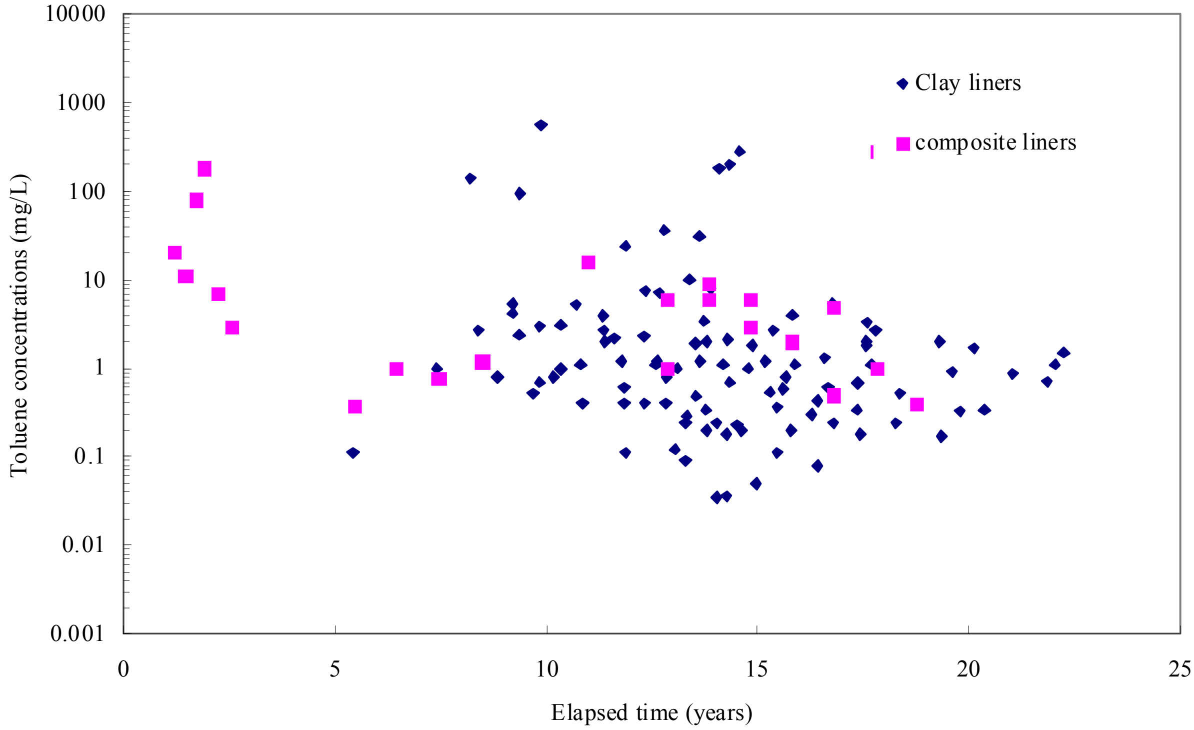


Figure 61. Concentrations of Toluene in lysimeters under clay and composite liners (from Edil 2007)

Table 14. Synthesis of diffusion and partition coefficients for the entire GCL of BTEX at 22 and 7°C from Rowe *et al.* (2005)

	Diffusion coefficient (m²/s)		Partitioning coefficients (ml/g)	
Temperature	22°C	7°C	22°C	7°C
benzene	3.7×10^{-10}	2.2×10^{-10}	4.4	2.6
toluene	3.1×10^{-10}	1.8×10^{-10}	15	8.7
ethylbenzene	2.9×10^{-10}	1.7×10^{-10}	36	22
m&p-xylene	2.5×10^{-10}	1.5×10^{-10}	42	25
o-xylene	2.6×10^{-10}	1.5×10^{-10}	27	14

Ganne *et al.* (2008) studied the sorption to geotextiles constitutive of GCLs of various VOCs and quantified the diffusion of 1,2 dichloroethane through a needle punched GCL containing powdered bentonite with a dry mass per unit area equal to 5.7kg/m². As regards the value of partition coefficients obtained, the same trend as the one noticed by Rowe *et al.* (2005) could also be observed. Indeed toluene and trichloroethylene exhibited a trend for a larger sorption followed by benzene and 1,2-dichloroethane. The diffusion coefficient of 1,2dichloroethane at room temperature was found to be 1.5×10^{-10}m²/s and corresponded to a bulk GCL void ratio equal to 3.51.

In an attempt to improve the sorption capacity of GCL bentonites, Lake & Rowe (2005) examined the potential improvement in the sorption that could be achieved for several organoclays and bentonite-activated carbon mixtures. Batch tests performed showed that all organoclays could potentially increase VOC sorption to GCLs by several orders of magnitude but activated carbon generally appeared to provide the most improvement for the samples tested. This result is consistent with data from Bartelt-Hunt *et al.* (2003) who studied the sorption of non-ionic organic species on organobentonites and with data from Richards & Bouazza (2007) who studied the sorption of phenol on organo-modified basaltic clay and bentonite. Lorenzetti *et al.* (2005) showed that at organobentonite amendments greater than 20% the hydraulic conductivity of GCLs could increase by as much as three orders of magnitude. Numerical simulations performed by these same authors modelling benzene flow through a single GCL tended to show that for one of the two organobentonites tested, the increased sorption would "balance" the increase in hydraulic conductivity. However, Lake & Rowe (2005) showed that considering the whole liner, increased sorption would only provide marginal benefits in terms of increased attenuation capacity for DCM and may even result in slightly higher benzene peak impacts for the single composite liner examined. Because of the relatively thin nature of the GCL, the increased sorption has a limited impact on contaminant migration (Rowe 2005). Thus the increased costs associated with modifying GCLs likely outweigh the benefit of such additives when considering the use of the GCLs as part of the liner system for MSW landfills (Rowe 2005).

Oxygen diffusion through GCLs

The purpose of cover systems in mine storage facilities containing reactive mining waste such as sulphide-bearing waste is to reduce the oxidation of those residues in the presence of oxygen and/or water. As ARD develops the pH drops progressively from near neutral to slightly acidic levels to very low pH levels (Renken *et al.* 2005*b*). These highly acidic waters can dissolve residual heavy metals still contained in the sulphidic waste and contaminate receiving waters via surface waters or groundwater flow. As GCLs could represent an alternative to capillary barriers or composite liners, even if they are not regularly used in this configuration at the moment, it is important to understand their performance as regards oxygen diffusion (Bouazza & Rahman 2007).

Laboratory investigations

Aubertin *et al.* (2000) presented the results of a laboratory study performed with a needle punched GCL containing 3.3kg/m² of bentonite. The load applied on the samples was in the range 10 to 30 kPa. Most measurements were performed on fully hydrated GCLs. A comparison to results obtained at a saturation degree equal to 71% emphasized the need to maintain the degree of saturation close to 100% at all time to minimize oxygen diffusion as the diffusion coefficient varied between $1\times10^{-7}m^2/s$ and $2\times10^{-11}m^2/s$ in this range of saturation for the GCL studied.

Bouazza & Rahman (2007) performed a laboratory investigation involving three different GCLs containing natural sodium bentonite either in powder or granular. Two of the GCLs were needle punched and the third one stitch-bonded. The mass per unit area of bentonite was in the range 3 to 6kg/m². They investigated the influence of the saturation degree of the bentonite on the diffusive coefficient of oxygen through those GCLs. An approximate four order of magnitude decrease from 2×10^{-6} to $2\times10^{-10}m^2/s$ of the oxygen diffusion coefficient was observed as the degree of saturation increased from 20 to 97% (see Figure 62). The diffusion coefficient was found to vary by two orders of magnitude for water saturation varying from 20 to 80 % whereas for a saturation degree larger than 80% a change of only 15% in saturation was needed to achieve an additional two orders of magnitude variation in the diffusion coefficient. The stitch-bonded GCL exhibited higher diffusion coefficients than the needle punched GCLs. This result was attributed to the fact that the bentonite tends to become partly confined along the stitch lines and swell freely between them. This suggests that GCLs should have a high degree of saturation to be effective as a barrier against oxygen (Bouazza & Rahman 2007). Then the cover will have to be designed to maintain the GCL close to saturation.

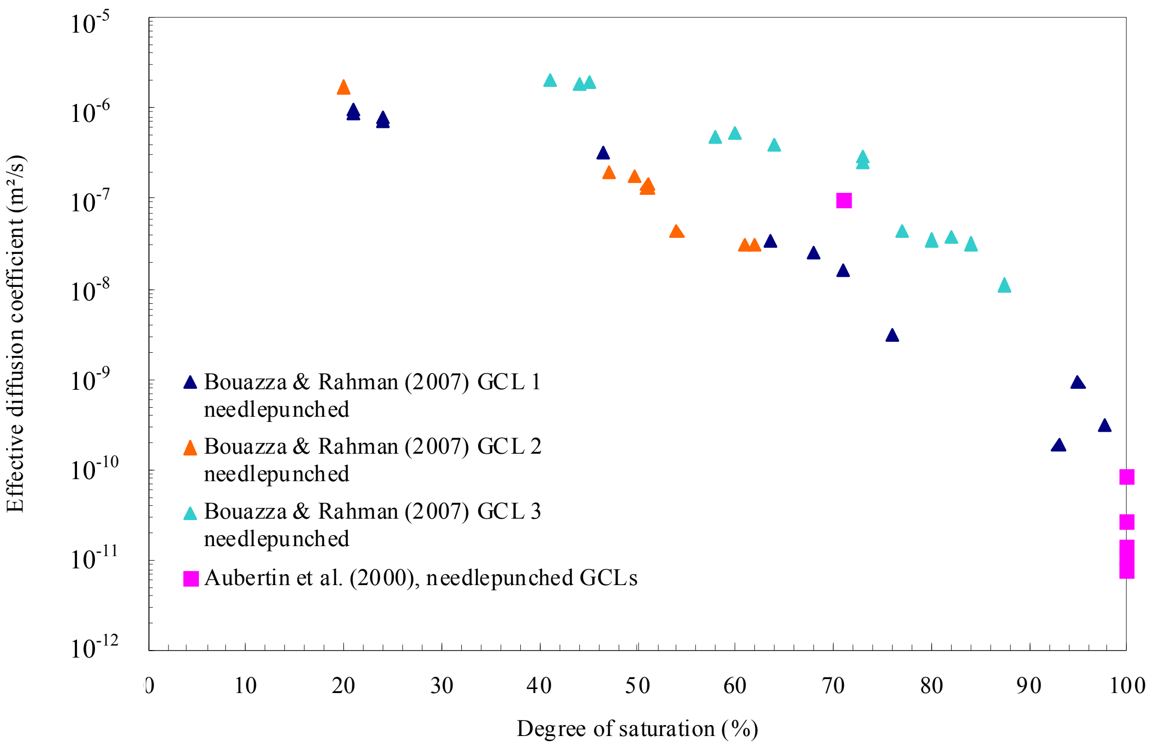


Figure 62. Effective diffusion coefficient of oxygen against degree of saturation in different GCLs (from Bouazza & Rahman 2007)

Field investigations

Various experiments were performed in order to investigate the in situ performance of GCLs to oxygen diffusion. Field oxygen concentrations were measured by Renken *et al.* (2005*b*) on a 15m×15m test plot indicated that the GCL used containing 3.6kg/m² of bentonite would have to be upgraded substantially to adequately reduce oxygen diffusion. An explanation could lie in a partial drying of the GCL during the summer. Adu-Wusu & Yanful (2006) also present the results obtained on a 12.2m×24.4m test plot containing a 8mm thick GCL. After three years of data collection it was observed that the GCL did not significantly reduce the oxygen gradient across the barrier. As no water content sensors were installed in the GCL, no clear explanation could be given.

EQUIVALENCY FOR LINING SYSTEMS

Introduction

While initial approaches to barrier equivalence considered equivalence in terms of times of contaminant migration through the barrier, it is now widely acknowledged that the most important is not how long it takes for contaminants to break through but rather the level of impact, in terms of groundwater concentration levels, the contaminant flux has on groundwater resources located below the barrier system (Foose *et al.* 2002, Guyonnet *et al.* 2007). Failure to include an analysis of diffusion of VOCs through intact liners can lead to incorrect conclusions regarding equivalency (Edil 2003). Models have been developed in order to estimate contaminant migration from landfills taking into account basic landfill features such as composite liners characteristics. Various approaches will be presented in the following that are being used either to quantify flow through soil liners or composite liners.

Case of soil liners

Summary of the approach adopted in France

In France it is recommended that calculations performed for the assessment of geological barriers consider the passive barrier only and not the active barrier as defined in the section relative to the overview of the use of geosynthetics for geoenvironmental applications. The main reason for this is to avoid that reinforcement of one barrier should be achieved at the expense of the other (Guyonnet *et al.* 2007). Also, the active and passive barriers do not perform in the same way over different time-frames, an issue which is poorly addressed by calculation models as they are unable to deal with the temporal evolutions of barrier performance. In order to avoid a misuse of theoretical calculations, it is recommended that for the assessment of equivalence in a French context, calculations consider the passive barrier only, what is done in practice (Ouvry *et al.* 2002), even though the essential role of the active barrier for subsurface protection is fully recognized.

The exact analytical solution proposed by Guyonnet *et al.* (2001) for the steady-state concentration in an aquifer overlain by a multiple-layer mineral-layer mineral barrier system is presented on Figure 63.

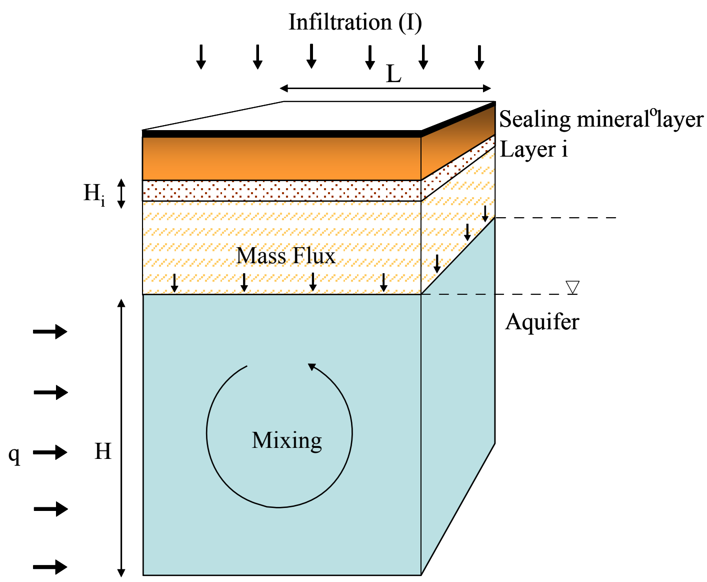


Figure 63. Conceptual model for the analytical solution of Equation 21 (from Guyonnet *et al.* 2007)

This solution, which assumes perfect mixing in an aquifer (or drainage layer in the case of Rowe & Booker (1987)) underlying the barrier, is:

$$C_D = \frac{1}{1+\varepsilon-\varepsilon \exp\left[-\sum_{1}^{n}\frac{I H_i}{\theta_i D_i}\right]} \tag{21}$$

with:

$$C_D = \frac{C}{C_o} \tag{22}$$

$$\varepsilon = \frac{q\,H}{I\,L} \tag{23}$$

$$D_i = \alpha_i v_i + D_0 \tau_i \tag{24}$$

where C is steady-state concentration in the aquifer in kg/m^3, C_o is constant source concentration in kg/m^3 over length L in the direction of groundwater flow, n is the number of layers in the mineral barrier, I is the steady-state vertical infiltration rate through the barrier system (controlled by infiltration through the sealing layer in m/s.

I is given by equation 25:

$$I = Ki \tag{25}$$

with

$$i = \frac{h_w + H_s}{H_s} \tag{26}$$

where h_w is the head of leachate in m, and H_s is the soil liner thickness in m, H_i is thickness of the ith layer in m, θ_i is the volumetric water content of the ith layer, D_i is effective dispersion coefficient in the ith layer in m^2/s, α_i is the longitudinal dispersivity coefficient in the ith layer taken as one tenth of the layer thickness in m, v_i is the pore water velocity in the ith layer in m/s:

$$v_i = \frac{I}{\theta_i} \tag{27}$$

D_0 is the free-solution diffusion coefficient in m^2/s, τ_i is the tortuosity of the ith layer, q is the aquifer Darcy flux in m/s and H is the aquifer thickness over which perfect mixing is assumed in m.

A Transient approximate solution is given by Equation 28:

$$\overline{c^*(p)} = \frac{C_o}{p} \cdot \frac{\exp\left[\frac{i\varphi}{2}\right]\chi}{\left(\theta_a H\, p\, \varphi + \frac{Hq}{L}\varphi\right)\left(\exp\left[\frac{\chi}{2}\right] - \exp\left[-\frac{\chi}{2}\right]\right) + \frac{1}{2}(i\varphi + \chi)\exp\left[\frac{\chi}{2}\right] - \frac{1}{2}(i\varphi - \chi)\exp\left[-\frac{\chi}{2}\right]} \tag{28}$$

with : $$\varphi = \int_0^{H_s} \frac{1}{v(z)D(z)}\,dz \tag{29}$$

and

$$\chi = \sqrt{i^2\varphi^2 + 4H_s\, p \int_0^{H_s} \frac{1}{D(z)}\,dz} \tag{30}$$

where $\overline{c^*(p)}$ is the aquifer concentration expressed in Laplace space, p is the Laplace variable and θ_a is the aquifer porosity. Guyonnet *et al.* (2001) studied the three scenarios presented in Table 15.

Table 15. Description of scenarios in the example calculation (From Guyonnet *et al.* 2001)

	Scenario 1	**Scenario 2**	**Scenario 3**
	Thickness (m)	Thickness (m)	Thickness (m)
GCL	none	0.01	0.01
Compacted clay	1	0.2	1
Silt	5	5	none
Sandy loam	none	none	5

Results obtained are presented in Figure 64. They would suggest that if a GCL is present the thickness of the clay layer can be reduced for an equivalent or even superior level of protection. However, a certain number of phenomena that will be further discussed in the practical considerations at the end of this section were not taken into account.

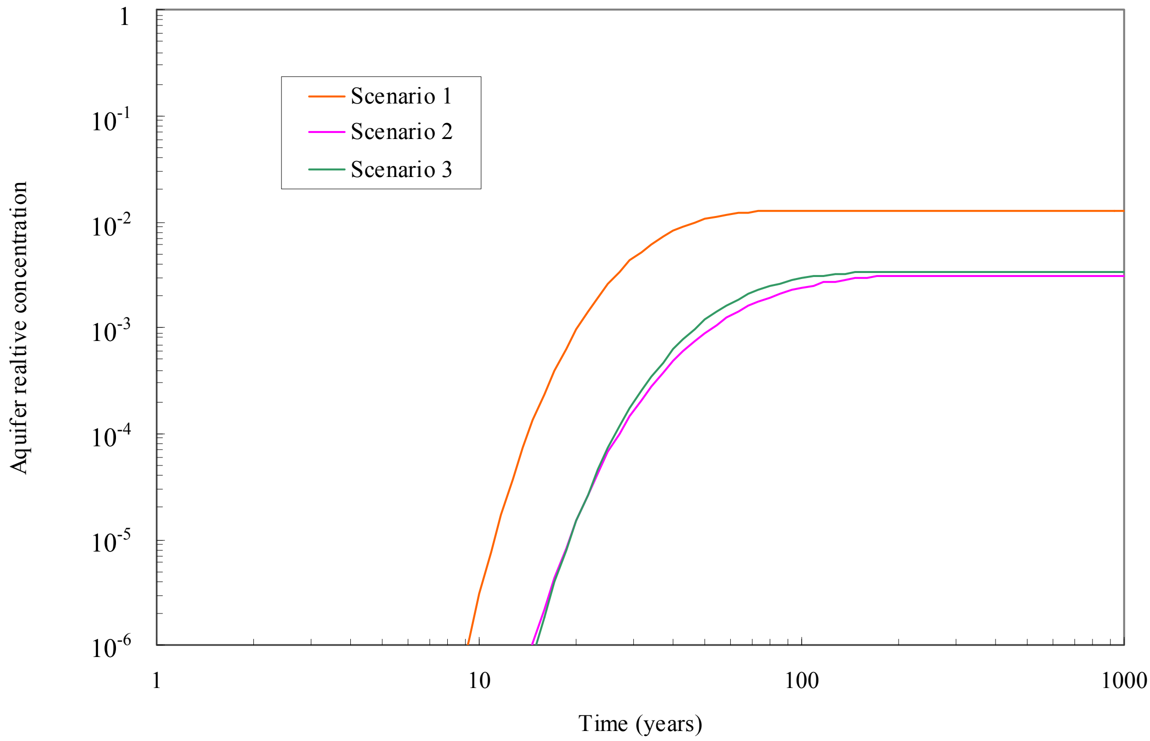


Figure 64. Results of the example application fro scenarios 1, 2 and 3 from Guyonnet *et al.* (2001)

A different approach was presented by Manassero *et al.* (2000) allowing the calculation of the contaminant mass flux and concentration in an aquifer at steady-state. The source concentration is assumed to be constant. The concentration C_x at distance x from the upstream boundary of the landfill is obtained as:

$$\frac{C_x - C_b}{C_0 - C_b} = \left(1 + \frac{qx}{Q_0}\right)^{\left(\frac{e^P}{1-e^P}\right)} \tag{31}$$

Where C_0 is the contaminant source concentration in kg/m^3, C_b is the background concentration in the aquifer upstream the landfill in kg/m^3, q is the Darcy seepage velocity through the liner in m/s, Q_0 is the volumetric flow of the aquifer upstream of the landfill per unit horizontal width perpendicular to seepage direction in m²/s, and P_L is the Peclet number defined as:

$$P_L = q\sum_i \frac{H_i}{n_i D_i} \tag{32}$$

Where H_i is the thickness of the ith layer of the liner in m, n_i is the porosity of the ith layer of the liner and D_i is the diffusion-dispersion coefficient of the ith layer of the liner in m²/s. This solution can be used equally for single or composite liners.

Case of composite liners

Analytical solutions

Katsumi *et al.* (2001) proposed a simplified method to design and evaluate landfill liners that allows solving the 1D advection-dispersion equation that accounts for adsorption given by Equation 33 in case of a constant source concentration C_0, for a semi-infinite medium:

$$\left(1 + \frac{\rho_d K_p}{n}\right)\frac{\partial C}{\partial t} = D\frac{\partial^2 C}{\partial z^2} - v_s\frac{\partial C}{\partial z} \tag{33}$$

Where C is the concentration of the solute in mol/l, ρ_d is the dry density of the clay, K_p is the clay-solute partition coefficient, D is the dispersion coefficient for the solute in m²/s and v_s is the solute velocity in m/s. R the retardation coefficient is defined as:

$$R = 1 + \frac{\rho_d K_p}{n} \tag{34}$$

Where n is the porosity. The following initial and boundary conditions were used:

$$\begin{cases} C(z = 0, t) = C_0 \\ C(z, t = 0) = 0 \quad \text{for } z > 0 \\ \dfrac{\partial C}{\partial z}(z = \infty, t) = 0 \end{cases} \tag{35}$$

Where t is time in s. If the soil properties are assumed to be homogeneous and time invariant and no chemical reaction occurs, concentration at the bottom of the liner can be obtained by:

$$\frac{C(H_s, t)}{C_0} = \frac{1}{2}\left\{ \operatorname{erfc}\left[\frac{1 - T_R}{2\sqrt{T_R / P_L}}\right] + \exp(P_L)\operatorname{erfc}\left[\frac{1 + T_R}{2\sqrt{T_R / P_L}}\right]\right\} \tag{36}$$

Where H_s is the thickness of the soil liner in m. The parameter T_L is the dimensionless time factor and P_L is the Peclet number given respectively by Equations :

$$T_R = \frac{\nu_s t}{RH_s} \tag{37}$$

$$P_L = \frac{\nu_s H_s}{D} \tag{38}$$

When analyzing transport of inorganic chemicals through composite liners, leakage through geomembrane defects is the primary transport mechanism. The transport calculations are then made using the 1D advection-dispersion equation on an area equal to the sum of effective areas of transfer defined for a given hole in the geomembrane using Darcy's law:

$$A_e = \frac{Q_e}{Ki} \tag{39}$$

Where i is the average vertical gradient for 1D flow and the magnitude of Q_e the flow rate for a given defect depends on the shape of defect, contact between soil liner and geomembrane and can be calculated for example using equations presented in the section dedicated to modelling of flow through composite liners among others.

For organic chemicals the following assumptions are made:

- The contribution of leakage through the geomembrane defects is negligible because molecular diffusion is far more significant;
- Diffusion through the geomembrane is ignored because the geomembrane is significantly thinner than the soil liner; and
- Advection is zero because the geomembrane limits leakage to very small quantities. The concentration expression then reduces to:

$$\frac{C(H_s, t)}{C_0} = \operatorname{erfc}\left[\frac{1}{2\sqrt{Dt / H_s^2 R}}\right] \tag{40}$$

Katsumi *et al.* (2001) also give expressions for the mass flux J of solute through the liner with time. They indicated that calculations performed using this method were in excellent agreement with the results from a more exact 3D finite difference analysis performed by Foose *et al.* (1999). Foose *et al.* (2001*b*) extended this solution to a medium of finite thickness and to the case the soil liner includes various layers i.e. a GCL, attenuation layer and foundation layer for example.

Numerical modelling

Foose *et al.* (2002) developed models based on a three-dimensional finite-difference groundwater flow model and a three-dimensional block-centered finite-difference computer model for solving the three-dimensional advection-dispersion equation for analysing contaminant transport in composite liners taking into account (1) advection and diffusion of inorganic and organic solutes through defects in the geomembranes and subsequently through the soil liner; and (2) diffusion of organic solutes through the intact geomembrane and subsequently through the soil liner. A comparison of three composite liners systems all including a 1.5mm thick HDPE geomembrane was conducted for the flow of toluene with:

- The liner prescribed in subtitle D of RCRA including a compacted clay layer at least 0.61m thick;
- A 6.5mm thick GCL;
- The Wisconsin NR 500 containing a compacted clay layer at least 1.21m thick.

No attenuation layer was considered under the liners. In this analysis the leachate source was assumed to have a constant concentration and depth. The frequency of geomembrane defects was assumed to be 2.5defects/ha with an area assumed to be 0.66m². Contact between the geomembrane and the soil liner was assumed to be intimate which means that there was no interface between both components of the composite liner. The depth of leachate was set at 0.3m. The length of the simulation was 100 years. The hydraulic conductivities were taken to be 1×10^{-9}m/s for the CCL and 10^{-11}m/s for the GCL. The diffusion coefficients and porosities were taken as 8.47×10^{-10}m^2/s and 0.54 for the CCL, 5.3×10^{-11}-5×10^{-10}m^2/s and 0.7 for the GCL. The value of diffusion coefficient through the geomembrane was 3×10^{-13}m²/s. Sorption parameters adopted were respectively equal to 135 ml/g for the geomembrane, 1 ml/g for the CCL and 2.6 to 5.2 ml/g for the GCL. Results obtained are presented on Figure 65.

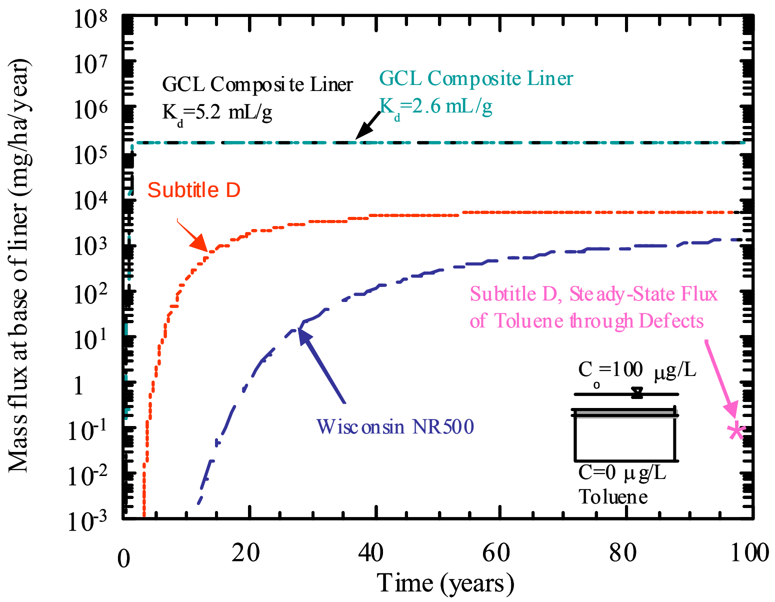


Figure 65. Mass flux of toluene in three composite liners with concentration at base equal to 0 (modified from Foose *et al.* 2002)

The mass flux of VOCs through defects was found negligible as compared to the mass flux through the intact liner which was six orders of magnitude greater. Furthermore most of the mass flux breaks through was found to be less than a decade or two depending on the type of liner, result which was consistent with the observation of VOCs found in pan lysimeters in Wisconsin within a decade (Edil 2003).

The mass flux of toluene after 100 years for the GCL composite liners was 1.5 orders of magnitude greater than that for subtitle D liner and 2.1 orders of magnitude greater than that through the Wisconsin NR500 liner. Results on the analysis show that equivalency assessment based purely on advection can lead to incorrect conclusions. Indeed for VOCs composite liners having thicker soil barriers had lower mass flux and greater sorptive capacity than the GCL composite liner; an analysis based on leakage rate would lead to the opposite conclusion.

Rowe & Brachman (2004) evaluated the equivalence of various composite liners. The specific details behind the typical results presented herein have been reported by Rowe & Brachman (2004) and Rowe *et al.* (2004); only a brief overview of the barrier properties are given here. The single composite liners considered here are shown in Figure 66. The geomembrane was assumed to be a properly specified and installed HDPE geomembrane. It was assumed the geomembrane contained 2.5 undetected holes/ha with a radius of 5.64mm that were not located at or near a wrinkle. Interface transmissivities of 1.6×10^{-8} and 2×10^{-10}m^2/s for the geomembrane in contact with CCL and GCL, respectively, were used to quantify leakage. The hydraulic conductivities were taken to be 1×10^{-9}m/s for the CCL, 2×10^{-10}m/s for the GCL and 1×10^{-7} m/s for a sandy silt attenuation layer. The diffusion coefficients and porosities were taken as 0.02m^2/a and 0.4 for the CCL, 0.005-0.009 m^2/a and 0.7 for the GCL, and 0.022m^2/a and 0.3 for the attenuation layer.

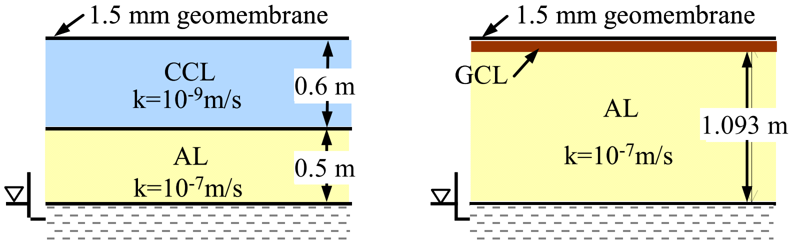


Figure 66. Single composite liner systems.

The leachate head above the elevation of the geomembrane was taken as 0.3m up to time T1 to correspond to an operational LCS. Larger leachate heads of 6m (T1 < t < T2) and 12m (t > T2) represent the mounding of leachate when the effectiveness of leachate collection is reduced (e.g., due to biologically induced clogging of the LCS). The piezometric head in the underlying aquifer was taken as the elevation of the top of the aquifer, as shown in Figure 66. The primary geomembrane and any secondary geomembrane were assumed to reach their service lives at times T3 and T4, respectively.

The calculated concentration of dichloromethane (DCM) – a chlorinated organic solvent often found in low but significant concentration in MSW leachate – through a single composite liner is given in Figure 67. For the three different barrier thicknesses, all show an increase to a peak DCM concentration in the aquifer followed by a subsequent decrease in concentration. This response arises from the finite mass of contaminants available in the landfill.

For the purely hypothetical case of a geomembrane + 0.6m CCL directly on top of the aquifer (as there would most likely be some sort of separation between the composite liner and the aquifer) contaminant transport is controlled by diffusion of DCM through the geomembrane. A permeation coefficient for DCM of 1.2×10^{-4} m^2/a was used based on data for a HDPE geomembrane (Rowe *et al.* 2004). For the number of holes and conditions assumed, leakage is sufficiently small through the composite liner such as to have negligible effect on the aquifer. The CCL provides diffusive resistance to the DCM that passes through the geomembrane. By delaying the time for DCM to reach the aquifer there is a greater potential for breakdown of DCM in the waste and clay liner. In this case the peak concentration of DCM exceeds 80 μg/l.

The calculations in Figure 67 show that, if any soil beneath the composite liner is included in the contaminant assessment, the peak DCM concentration is greatly decreased.

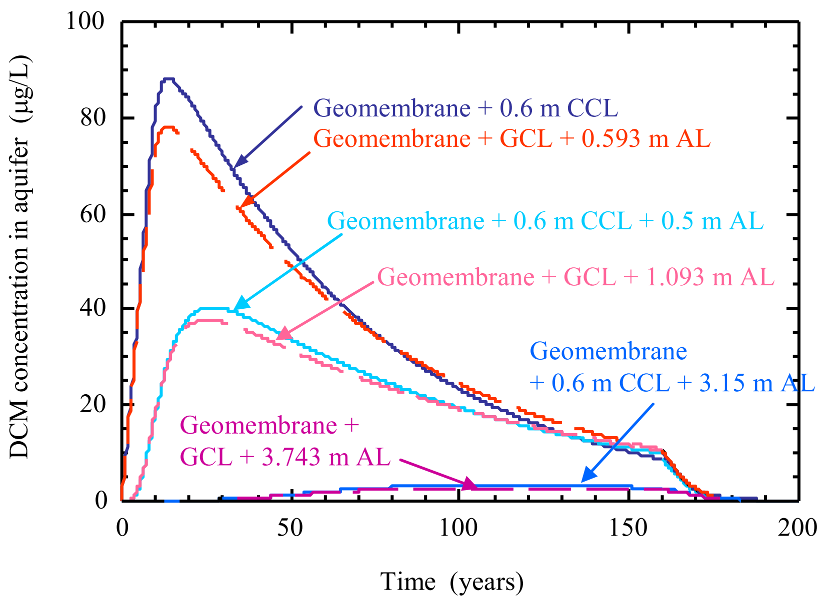


Figure 67. Calculated dichloromethane concentration for single composite geomembrane/CCL and geomembrane/GCL liner systems. T1=40 years, T2=60 years, and T3=160 years (Modified from Rowe & Brachman, 2004).

This layer need not be of very low hydraulic conductivity; in these calculations the attenuation layer was assumed to have properties typical for a sandy-silt. Increasing the total thickness of the barrier system by 0.5m reduces the peak DCM impact by more than one-half relative to the geomembrane + 0.6m CCL alone. An attenuation layer greater than 3.15m reduces the peak impact to less than 5 μg/l, which is one possible maximum concentration level for DCM. Lower impact occurs for a thicker attenuation layer because the greater thickness of soil beneath the geomembrane increases the time it takes for DCM to diffuse through the barrier system and thus allows more time for the organic contaminants to biodegrade. In the cases considered, the peak DCM impact was not sensitive to the service life of geomembrane provided it exceeded the time to reach the peak impact.

The calculated impact on the aquifer for chloride – a typical inorganic contaminant that is not retarded by sorption or biodegradation – differs substantially from DCM. The calculated impact in Figure 68 shows that essentially no chloride reaches the aquifer provided the geomembrane has not reached its service life, which was assumed to be 160 years in this assessment. This is because the leakage is very low and the geomembrane is an excellent diffusion barrier to chloride. A partitioning coefficient of $1.04 \times 10^{-9} m^2/a$ was used for chloride and a HDPE geomembrane (Rowe *et al.* 2004). However once the geomembrane is assumed to reach its service life, the chloride moves towards the aquifer dominated by advection. The calculated peak chloride impact in such scenarios principally depends on when the primary geomembrane was assumed to reach its service life.

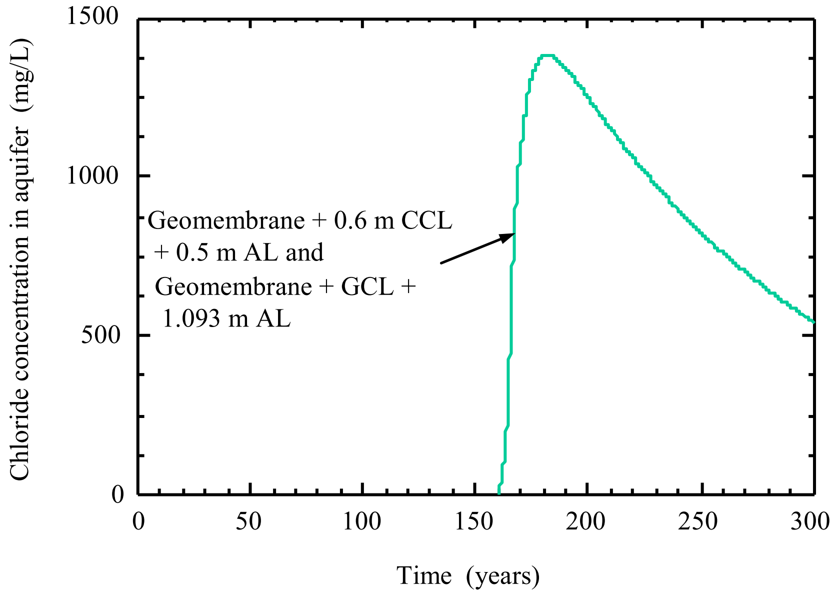


Figure 68. Calculated chloride concentration for single composite geomembrane/CCL and geomembrane/GCL liner systems. T1=110 years, T2=130 years, and T3=160 years. Modified from Rowe & Brachman (2004).

A double composite liner may be required when the calculated peak impact with a single composite liner exceeds maximum allowable levels. Inclusion of a SLCS between the two composite liners allows the opportunity to collect and remove additional contaminants from the system and thereby reduce the peak impact relative to the single composite liner. Also, a secondary geomembrane may be expected to have a longer service life than a primary geomembrane, largely owing to potentially lower service temperatures for a secondary geomembrane since it would be further from the warmer waste and closer to the cooler groundwater. The calculated concentrations of chloride in the aquifer for the case of the double composite geomembrane/CCL liner shown in Figure 69 is given in Figure 70.

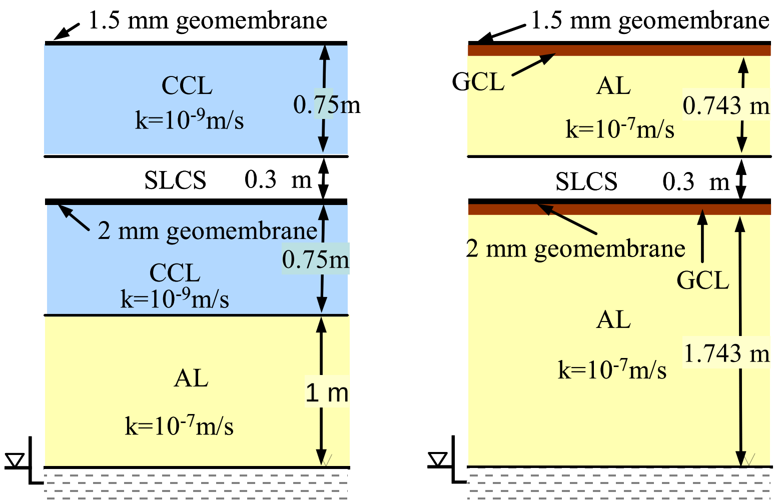


Figure 69. Double composite liner systems

As for the single composite liner, no chloride reaches the aquifer as long as the secondary geomembrane has not reached its service life. The peak impact with the double composite liner can be reduced to less than 200mg/l provided the secondary geomembrane lasts at least 400 years. The peak chloride impact then depends on the service life assumed (T4) for the secondary geomembrane. Calculations like these can be performed for the range of service life values expected for a given project depending principally on the long-term temperatures expected for the geomembrane and the polymer formulation and additives used to manufacture the geomembrane.

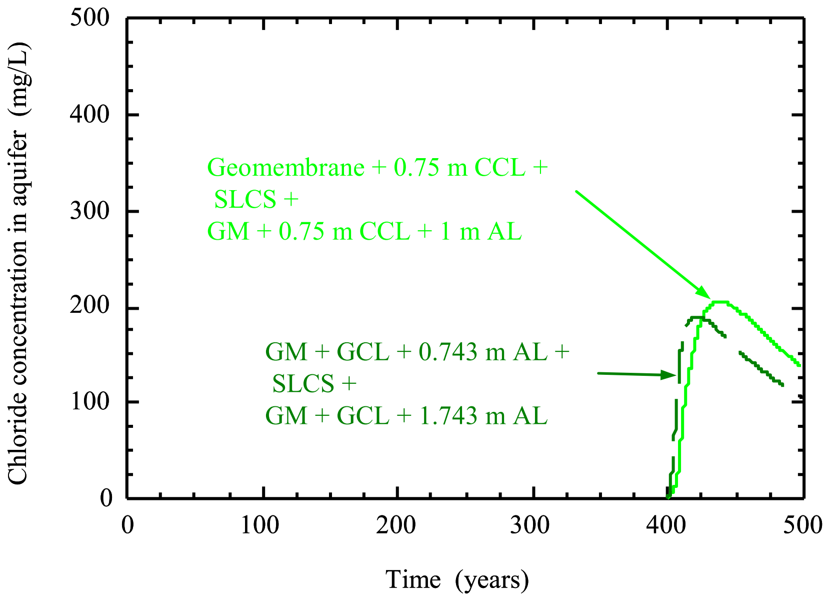


Figure 70. Calculated chloride concentration for double composite geomembrane/CCL and geomembrane/GCL liner systems. *T1*=110 years, *T2*=130 years, and *T3*=160 years, *T4*=400 years. Modified from Rowe *et al.* (2004).

Calculations like those shown in Figures 67, 68 and 70 may be used to examine the environmental protection provided by a geomembrane/GCL composite liner relative to a geomembrane/CCL liner, in terms potential contaminant impact on the aquifer. For DCM, if the geomembrane + 0.6m CCL was replaced with a geomembrane and only a GCL, the calculated peak DCM impact would increase to 500μg/l. Contaminant transport would still be dominated by diffusion, but since the GCL is very thin relative 0.6m of compacted clay it provides little resistance to diffusion. However, as shown in Figure 67, if the CCL is replaced with a GCL and a 0.593m thick attenuation layer (to keep the thickness of the GCL + AL = 0.6m), the peak DCM impact is actually reduced.

The geomembrane + GCL + 0.593m AL has a slightly lower peak impact than the geomembrane + 0.6m CCL largely because the sandy-silt attenuation layer acts as a better diffusive barrier than the compacted clay for the range of parameters investigated. The geomembrane + GCL + 0.593m AL does have slightly lower leakage than the geomembrane + 0.6m CCL case; however, since the leakage is so small that the transport is dominated by diffusion, small changes in leakage have negligible impact on DCM concentrations in the aquifer.

For the three different barrier thicknesses in Figure 67, the geomembrane/GCL/AL composite liner systems examined were found to be equivalent to (and indeed a little better than) the geomembrane/CCL/AL systems in terms of minimizing the impact of DCM on the aquifer. Rowe & Brachman (2004) also concluded the same for calculations considering benzene as a typical aromatic hydrocarbon. In terms of chloride, there is essentially no difference between the geomembrane/CCL and geomembrane/GCL composite liners for the single and double composite liners considered in Figures 68 and 70 (Rowe & Brachman 2004, Rowe *et al.* 2004).

El-Zein & Rowe (2008) examined the interaction between leakage through a hole in a wrinkle and diffusion using a finite element analysis with CONFEM2D. This model applies a special treatment of a leaking geomembrane to yield an equivalent boundary conditions that conserves mass in the waste. The transport of dichloromethane was simulated for eight cases involving composite liners, including both CCLs and GCLs. The effect of wrinkles on the transport of DCM into groundwater was found to be significant under conditions oh high leakage. Pollutev7 predictions of peak concentrations in the aquifer were typically within about 20% of the two-dimensional predictions under conditions of horizontal mixing in the aquifer. Results obtained tended to show that the one dimensional analysis conducted with Pollute V7 are adequate for most practical purposes when compared to results of CONFEM2D.Conversely, for situations where predictions are very close to regulatory limits, a full two-dimensional analysis may be justified to confirm predicted compliance.

Practical considerations

Given the complexities, uncertainties and variabilities involved in contaminant transport from a landfill, through a barrier system and into a receptor aquifer, such quantitative assessments of potential impact are not meant to be precise predictions of concentration at any point, at any time. However, they are able to permit one to calculate trends, identify a likely range of expected values and identify important parameters. They also provide a means to compare alternate barrier designs and assess the equivalency of the environmental protection provided by a geomembrane/GCL composite liner relative to a geomembrane/CCL composite liner.

Studies performed confirm that consideration of an attenuation layer makes a significant difference to the mass flux through the composite liner system and needs to be considered when assessing equivalence.

Models still have serious limitations and so while models may be useful for design purposes, it is essential that equivalence assessments include elements of qualitative reasoning (and common sense). Models must achieve increased capabilities in order to address key issues relevant to equivalence assessments. An important area of progress is the coupling between chemical, structural and hydraulic mechanisms (Guyonnet *et al.* 2007). A significant body of research has documented the structural changes undergone by bentonite following ion exchange. These structural changes may lead to permeability increases as shown in the section of this Keynote Lecture dedicated to migration and attenuation through GCLs. While the coupling of ion exchange with transport phenomena is already handled by several models (for example Parkhurst & Appelo 1999), taking into account the influence of clay chemistry evolution on clay structural mechanical and hence hydraulic properties is still in its infancy (Guyonnet *et al.* 2007).

The equations for predicting solute transport described in the present section have a number of limitations (Foose *et al.* 2001*b*). Indeed there is a paucity of laboratory or field data regarding contaminant transport in composite liners. Therefore the accuracy of the equations in replicating field conditions is unknown. Furthermore most of the equations presented are for simplified boundary conditions which may not be representative of field conditions.

Other equivalency issues including the availability of suitable clay for constructing a CCL, resistance to environmental degradation, catastrophic puncture, quality of construction and mechanical stability must also be considered when assessing equivalency of liners for waste containment (Foose *et al.* 2001*b*, Rowe & Brachman 2005, Guyonnet *et al.* 2007).

In addition the type and size of defects were assumed to be the same regardless of whether the composite liner had a CCL or a GCL and rather small as compared to densities mentioned in the section dedicated to CQA.

CONCLUSIONS

Despite the reduction of waste production in various developed countries, waste landfilling remains one of the major geoenvironmental applications of geosynthetics with the growing awareness of the need to improve waste storage in a number of countries through Eastern Europe, Africa, Asia and South America. Furthermore design and construction of liner systems in mining applications has increased over the last two decades as the performance of geosynthetic materials in the mining environment becomes better understood. Not only well known usual products are used but there is still some room for innovation in products that can answer specific requirements.

There is also a need for answers to specific questions raised by those new applications where loads can be very high as compared to loads classically encountered in other geoenvironmental applications like landfills or ponds for environmental protection.

An insight was thus given through this Keynote Lecture on design of plastic pipes under high loads and design of geomembranes under high loads, arising from the need in mining applications to consider the presence of high stress levels up to 3.5MPa. Understanding the performance and limitations of geopipes in deep burial applications is important as pipes are being designed and installed in harsh environments. The state of the art in the area of design methods for designing of geopipes under high loads was given. The selection and design of the geomembrane liner for applications with high loads requires a thorough understanding of the interaction between the liner system components and the type of applied load (normal and shear loads). Insight was given regarding testing procedures to evaluate the suitability of the lining system, foundation settlement and internal settlement.

The topic of puncture protection of geomembrane and GCL liners was then addressed trhough the presentation of large laboratory or field tests. Those tests addressed either damage during installation or protection efficiency. Results obtained by various authors suggest that a nonwoven needle punched geotextile selected solely to prevent puncture is generally insufficient to limit tensile geomembrane strains to allowable levels. Quantifying long-term tensions in geomembranes is challenging and currently underway. At present, only an estimate of short-term tensions can be obtained from large-scale laboratory tests. The available data suggests that a 0.15m thick sand protection layer or a sand filled geocomposite is required to limit the geomembrane strains below the 3% limit.

An insight was then given in quality assurance and quality control which is a necessary step to ensure that a properly designed facility will perform satisfactorily ensuring proper construction. After a presentation of features of defects in geomembranes state of the art recommendations were given together with an overview of geomembrane liners control methods.

The following section was dedicated to the in situ performance of liners, focusing mainly on GCLs. Various topics were addressed among which performance of GCLs in landfill covers. For this particular topic a number of experimental results from field excavations or in situ testing were reported in literature that are contradictory, underlining the need for more research in this field. Nevertheless, a quasi systematic satisfactory performance could be obtained under a 1m soil cover under middle European climatic conditions. GCL panels shrinkage was also addressed. The various explanations proposed in the literature were presented in detail and insight given in possible measures to prevent or avoid this phenomenon. Liner temperature which can influence transfer through geosynthetics and soil materials was also briefly discussed.

The various methodologies existing at present to address liquid and gas flow through composite liners in case a defect exist in the geomembrane were then presented, i.e., analytical solutions, empirical equations and numerical modelling, giving a state of the art in this field.

As any assessment of long-term environmental impacts from landfills requires contaminant transport modelling issues regarding leachate or mining solutions transfers through GCLs, either advective or diffusive, was then addressed. A good performance of GCLs to advective transfers of leachate or mining solutions is generally observed, whatever the hydration condition contrarily to what can be observed for permeation of single-salt solutions based on the literature review performed. More research may be required to confirm those results as the equilibrium was not probably reached in all tests. Attenuation of heavy metals in GCLs was shown to be an important phenomenon. Oxygen diffusion through GCLs can also be an issue when the degree of saturation of the GCL is not high enough.

Finally, the question of equivalence for soil liners and composite liners was addressed. A presentation of the various existing approach was given with a particular emphasis on composite liners. Results presented show the great influence of an attenuation layer underneath the liner on transfers to the aquifer. The various approaches, despite some limitations that were described provide a successful means to compare alternative barrier designs and assess the equivalency of the environmental protection provided by a GCL liner relative to a CCL liner, whether part of a soil or composite liner.

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