A 25-YEAR PERSPECTIVE ON WASTE CONTAINMENT LINER AND COVER SYSTEM DESIGN USING GEOSYNTHETICS

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1. INTRODUCTION

This paper presents the perspective of one design practitioner regarding changes that have occurred in the industry over the past 25 years – particularly as related to the design of waste containment systems using geosynthetics. An effort has been made to present practical changes that have taken place in approaches to design, rather than advances in academic theory. Given the practical limitations of a single person writing an overview of this topic, there are bound to be biases in emphasis and subject matter related to the author's perspective and experience. In this regard the author asks forbearance from the readers, and welcomes any feedback from industry peers.

The author entered the field of waste containment engineering in 1986, at the very beginning of the 25-year retrospective period addressed in this paper, and his first projects were the design and evaluation of two major double-lined hazardous waste landfills, one in Illinois and one in California. It is interesting to observe that there is no significant difference between what we see looking at a set of design drawings from the Kettleman Hills Hazardous Waste Landfill project from 1987 (a detail is shown in Figure 1), and a set of drawings for a landfill design that would be produced today. We know, however, that there has, in fact, been a significant increase in the understanding we bring to current design approaches for these types of facilities, compared to the understanding we had 25 years ago. The aim of this paper is to highlight these changes in our approach to design that have taken place in the past generation.

Despite these advances over the past 25 years, important and fundamental questions continue to linger in the minds of designers, regulators, and owners, albeit tempered with greater understanding. Some examples of these persistent questions are:

- How long will these materials last?
- How much redundancy do we need?
- What should be the factor-of-safety? (or reliability?)
- Do we really need CQA?
- What kind of liner system is best, and how thick should it be?

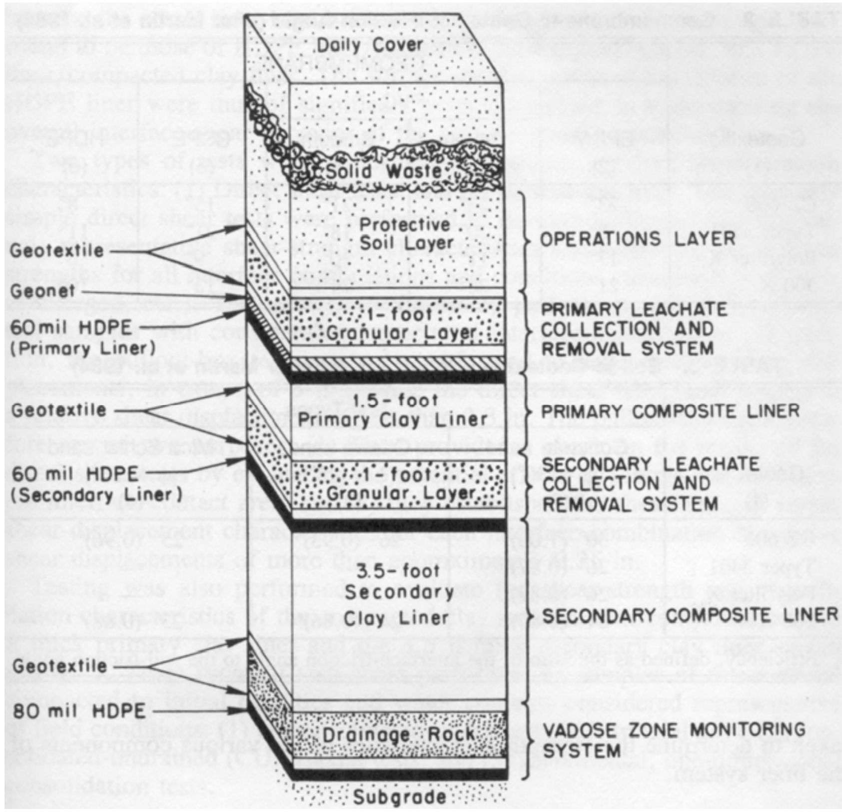

Figure 1. Detail of actual hazardous waste liner system design from 25 years ago.

2. WHAT IS A CONTAINMENT LINER SYSTEM?

What is a liner system? This is a fundamental question that deserves brief mention. In general, the author defines it as a barrier layer that has a lateral drainage layer above and/or below it, as illustrated in Figure 2. This simple concept helps us focus our discussion on the value of materials that serve as a barrier to fluids (e.g., geomembranes and GCLs), versus those that provide a transmissive layer to fluids (e.g., geonets and geocomposite drainage layers), or that serve as a protective layer for one of these two functions (e.g., a geotextile filter, or a geotextile puncture-protection layer).

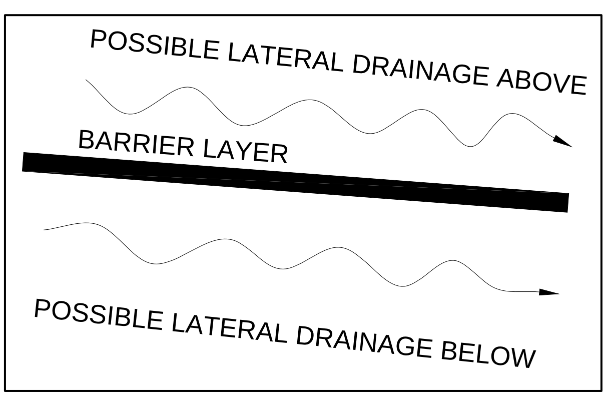

Figure 2: Schematic of author's definition of a liner system.

3. A COMMENT ON REGULATIONS GOVERNING WASTE CONTAINMENT LINERS AND COVERS IN THE USA

The containment industry is largely driven by regulations, as good regulations have proved essential for the proper long-term stewardship of our environment. Using U.S. regulations as a benchmark, we can generally say that the regulations that require liner systems for waste containment facilities came into being at the beginning of the 25-year period we are reviewing, as RCRA Subtitle C (hazardous waste) was promulgated in the early 1980's, and RCRA Subtitle D (municipal waste) in the early 1990's. These regulations essentially required single-composite bottom liners (with an extra geomembrane liner and lateral drainage layer for Subtitle C), with an overlying lateral drainage layer to collect leachate. The cover systems were also required to be single-composite liner systems. Very few changes have been made in these basic regulations since that time, other than allowing for some alternative design approaches. From this point of view, it is not surprising that a set of design drawings for a Subtitle C landfill in 1987 looks quite similar to a set of design drawings for a landfill today.

The environmental performance record of landfills constructed in the past 25 years has indeed been quite good, confirming the wisdom of the regulatory approach outlined in the 1980's. Indeed, we have definitely gotten our arms around the issues, and have them more or less contained (!). Meanwhile, as we examine new understandings gained from continued academic research, material improvements developed by manufacturers, experience gained in construction and operations, ingenious engineering solutions, lessons learned from failures, and approaches being developed by our neighbors around the world, we see definite indications that the industry has steadily advanced in its understanding over the past generation. And these improved materials, practices, and understandings will certainly continue in the next generation. Any suggestions for improvement that may be implied in this paper should by no means be considered as criticism of past implementations and approaches. Far from it. The U.S. - its industry regulators, owners, operators, engineers, technicians, contractors, and citizen-users - has done an excellent job of implementing a great infrastructure for "housecleaning" and environmental management that should be, and often is, the envy of other nations.

4. LIST OF TOPICS ADDRESSED IN THIS PAPER

To provide the reader with a quick reference of the topics that are addressed in this paper, the following list presents the order of the topics presented. Since the topics have been numbered to provide easy location, the headings before this section are included for completeness.

1. Introduction (previous to this section)
2. What is a containment liner system? (previous to this section)
3. A comment on regulations governing waste containment liners and covers in the USA (previous to this section)
4. List of topics addressed in this paper (this section)
5. Brief discussion of geosynthetic materials
6. Contaminant transport

7. Aging durability and geomembrane service life
8. Slope stability
9. Steep-slope bottom liner systems
10. Piggyback liner systems
11. Lateral drainage design
12. Details related to penetrations and attachments
13. Details related to anchor trenches
14. Exposed geomembrane covers
15. Ponds
16. Secondary containment around tanks
17. Construction, CQA, and specifications with regard to design considerations
18. Moving into the future

5. BRIEF DISCUSSION OF GEOSYNTHETIC MATERIALS

In deciding what to write about in greater detail and what to skim over with a passing reference, the author has elected to focus narrowly on specific design approaches. Several excellent papers are being presented at this conference that discuss our past generation of experience regarding different types of geosynthetic resins, additives, manufacturing methods, standards for testing, standards for specifying, and standards for installation and CQA. For this reason, geosynthetic materials in themselves are discussed only briefly in this section.

The following is a concise list of topics relative to geosynthetic materials which, from the author's perspective as a design practitioner, are significant relative to design. A check mark (√) appears beside those topics that are described in more detail. Where there is no (√), the reader is encouraged to review other papers from the proceedings of this conference to learn more about those subjects.

Geomembranes

A list of geomembrane-specific issues of acknowledged significance that have been addressed to a greater or lesser degree in the past 25 years, includes:

Long-term chemical compatibility: This was a major effort early on, and a lot has been learned in this regard. Early chemical immersion and index testing, which was developed by Dr. Henry Haxo, resulted in EPA Test Method 9090. That has since been superseded by ASTM D5322 and D5747 and related standards. Chemical compatibility charts are typically available from manufacturers, and these test methods are available for testing against specific chemical environments.

Advances in resins and products. Manufacturers have been the driving force for the development of an ever more efficient and useful selection of geomembrane materials, and market dynamics have conspired to retire some of the less efficient products. A good example of this is chlorosulphonated-polyethylene (CSPE), long-marketed under the trade name of Hypalon. Initially a major player in the exposed geomembrane market, its high cost and a number of technical issues caused it to all but disappear from the North American market (though the formulation is still available from an Asian producer), and its function has largely been replaced by reinforced polypropylene (PP-R). The PP-R material suffered a temporary setback in popularity in the early 2000s as durability failures started to show up. It turned out that a number of manufacturers had skimped on the antioxidant additive packages, but this problem has since been recognized and corrected. Another significant product development that has had a major impact on the design and use of geomembranes, and which did not exist at the beginning of our 25-year review period, was the use of textured surfaces on polyethylene geomembranes. These new surfaces provided much greater interface shear strength characteristics against various soils and other geosynthetics, which the industry has aggressively utilized ever since. These are only a few examples of numerous product developments that have occurred in association with geomembranes, and others will undoubtedly continue to be developed in the future.

Installation and seaming: There have been significant advances in our understanding of good installation and seaming practices, and of seaming equipment for thermoplastics. Perhaps the most significant development in seaming equipment was the introduction of hot-wedge welding for thermoplastic geomembranes, which took place near the beginning of our 25-year review period. Since that time, various improvements to seaming equipment have been made by different manufacturers, but the fundamentals of providing good welds have essentially remained unchanged over these past 25 years. Now as much as ever, good operator training, attentiveness, and CQA are the keys to successful welding.

Testing and specifying: Refined methods of testing and specifying have been developed through the efforts of GRI and ASTM, and the collaboration of manufacturers, designers, and testing laboratories. As an example in our industry, GRI GM13 became an industry cornerstone for specifying HDPE, and serves as an ongoing example for other materials. ASTM subcommittee D35 has developed a plethora of standards and guides for the testing and installation of geomembranes.

Wrinkles: An increased understanding of the development and control of wrinkles, and wrinkles as a design consideration. (√)

Allowable strain: An increased understanding of puncture protection and long-term durability issues. (√)

Soil cover: An increased understanding of procedures for covering with soils.

ELL surveys: An expanded use of electrical leak location (ELL) surveys on both bare and covered geomembranes.

Slope stability: An increased understanding of slope stability. (√)

The desiccation of materials underlying exposed geomembranes: An increased appreciation of the dynamics involved.

Ponds and Reservoirs: Not much fundamental change in the approach to pond design, but a refined understanding of pond design issues. (√)

Development and use of Exposed Geomembrane Covers (EGCs). (√)

Contaminant transport: An increased understanding of leakage prediction and contaminant transport. (√)

Penetration and Connection Details: Increased appreciation for details related to connections and penetrations – an area where the industry can find great room for improvement. (√)

Repairability of aged and exposed liners: We are continually learning in this area, as facilities are continually aging.

GCLs

The past 25 years could be said to be "the generation of GCLs", since this is the period in which they were introduced and developed, and standards were established for hydraulic conductivity testing, index and performance testing, seaming, needle issues, deployment and handling, construction, and durability issues. The major technical issues that affect liner design, and which are still the subject of ongoing evaluations, include:

Product innovations: Many different types of products have been promoted that include different bentonites and different carriers, including polymer-amended bentonites.

Equivalency with compacted clays.

Cation exchange: Much attention has been given to cation exchange in bentonite for different conditions of hydration, normal loads, liquids, soils, peptized bentonite, polymer-amended bentonite, and solo GCL vs. composite vs. encapsulated GCLs.

Shear strength: An increased understanding of interface vs. internal shear strength; peak vs. post-peak; effects of different hydration methods; encapsulation. (√)

Hydration mechanisms: Advances in our understanding of the hydration of GCLs set on a subgrade; encapsulated GCLs, and the quantification of hydration mechanisms and rates (see Thiel et al., 2001).

Panel shrinkage. The issue of GCL panel shrinkage has received a lot of attention since the author first reported gaps in GCL panels that had originally been overlapped (Thiel & Richardson, 2005). Although there have been numerous published studies and great advances in our understanding of this issue since that time, the author of this paper feels satisfied that a pragmatic solution has been found, which is to simply heat-tack all fabric-based GCL seams, as described in Thiel & Thiel (2009).

Geonets and Geocomposite Drainage Layers

Geosynthetic drainage layer products have been available since the early 1980's. New products continue to be developed, which include bi-planar and tri-planar products, as well as cuspated sheets, nubbed-surfaces on geomembranes that are then covered with a geotextile, and draintubes encapsulated in geotextiles that are offered as alternatives. Stouter materials that are able to handle long-term high loads are also available.

Laminated geotextiles that form a 'geocomposite': In the late 1980's, geonet products laminated with geotextiles were not even available, and today they are so pervasive that the term "geocomposite" is used almost synonymously to refer to a geonet laminated with a geotextile. The techniques and specification for lamination (viz. peel strength) has improved over this time, along with our understanding. (√)

Transmissivity: We have seen greatly improved understanding of how to measure and how to design, using the transmissivity offered by these products for lateral drainage in different situations. (√)

Clogging: There is lack of long-term data regarding chemical and biological clogging of these materials, especially in bottom liner applications, and we can expect that more studies and understanding will be gained in the future.

Geotextiles (specifically those related to filtration and puncture protection)

Filtration: Over the past 25 years there has been improved understanding and elucidation of the geotextile filtration function (Giroud 2010). The science and theory of geotextile filtration has been greatly advanced in terms of a better understanding of the relationship between granular and geotextile filters, the role of the hydraulic gradient in filtration, the relationships between fiber geometries, minimum thickness and porosity requirements (or the percent of open area for woven geotextiles), the effective filtration opening size, etc. That said, there may be a gap between the available knowledge base versus the actual practices used in containment engineering (this might be different in another field such as dam engineering, where filters are much more critical, JP Giroud, pers. comm.). In general, there has not been a great incentive for sophisticated filter design in landfill engineering, so any number of older standard methods or rules of thumb are often utilized in material selection in this commodity-driven market. The author tends to use Leuttich et al. (1991), which is over 20 years old, as his filter design reference.

Puncture Protection: An improved understanding and improved specification for puncture protection (cushioning) methods. Rational approaches have been developed for evaluating the effectiveness of candidate geotextiles for specific puncture-protection applications. The long-term applicability of the different approaches remains an open question, as the advocates of different views can present significant differences in their final recommendations. (√)

NWNP Manufacturing: Although the manufacturing capabilities for nonwoven needlepunched (NWNP) geotextiles allow for the creation of fairly specialized products in terms of denier size, fiber cross-sections, fiber lengths, wettability, polymer, and degree of needle punching, in North America, these products have been, and continue to be widely commoditized using the same polymers (primarily PP in North America, though PET was formerly used and is still used in China; this is a market decision), with relatively fine 6-denier fibers. While there have been and will continue to be product innovations, the majority of products will likely continue to be offered in much the same fashion as they have for the past two decades.

Testing and specifying: Specified index and performance tests have remained relatively constant over the period, albeit with some fine tuning.

Durability. Geotextiles are used by themselves, as well as in conjunction with products such as geosynthetic drainage layers and GCLs. The long-term durability of geotextiles relative to their intended functions of providing filtration, separation, and interface shear strength is still a question of interest. Research is being conducted in this regard, and its outcomes will undoubtedly influence the practice of design in the future.

6. CONTAMINANT TRANSPORT

The goal of designing containment systems is to reduce the transport of contaminants into the groundwater to an acceptably low level for the contaminating life of the facility. Models of contaminant transport mechanisms and rates are used to make permitting and design decisions. In general, two different mechanisms of contaminant transport through landfill liners are actively discussed in today's literature: advection and diffusion. Advection, commonly referred to as 'leakage', is governed by Darcy's law, in which the flow through defects and soils is governed by the head buildup on the top of the liner system. Diffusion involves the migration of chemical constituents through intact media based on Fick's law; it is concentration-driven, independent of gravity and head conditions. Both transport mechanisms increase with an increase in temperature, so that landfill operating temperatures have a significant effect on contaminant transport. See Rowe (2005) for further discussion of these mechanisms.

The USEPA regulations for MSW landfill base liner requirements (RCRA Subtitle D as codified in 40 CFR Part 258, paragraph 258.40) are written to cover two different approaches. Paragraph 258.40(a)(1) is a performance-based standard to protect groundwater. Paraphrased, the regulation in 258.40(a)(1) states that contaminant transport through a liner system must result in no more than the specified prescriptive maximum contaminant levels stated in the regulation. This performance-based regulation requires the applicant to demonstrate, usually through the modeling of contaminant transport, that the proposed design will comply with its terms. As an alternative to the performance-based approach, paragraphs 258.40(a)(2) and (b) provide a prescriptive design basis for the liner and leachate collection system. If the prescribed design criteria are fulfilled, the design is presumed to meet the groundwater protection standards and no contaminant-transport modeling is required. These are the two well-known design standards for a single-composite liner and overlying leachate collection layer in the U.S.

The EPA (1993) provides technical background and guidance related to the Subtitle D ruling. Guidance is provided related to the contaminant transport modeling required to demonstrate compliance with the performance-based regulation, and an extensive list of computer modeling codes that were available at that time is provided in the reference. While many complex contaminant-fate-and-transport mechanisms were recognized by the EPA (1993), including diffusion, it is interesting to note that this guidance document puts a clear emphasis on advective-only transport through the liner system. In two distinct places the guidance states that "the factor that most strongly influences geomembrane performance is the presence of defects…or penetrations of the liner." The guidance suggests, for example, that the HELP model Version 3 (Schroeder et al., 1994), which incorporates the landmark Giroud leakage equations for composite liners (Giroud & Bonaparte, 1989, Giroud et al. 1989, and Giroud et al. 1992), should be used to estimate advective leakage through the liner, and that a model such as MULTIMED, developed by the USEPA (http://www.epa.gov/ceampubl/mmedia/multim2/) should then be used to evaluate the fate and transport of the leakage through the underlying hydrogeologic strata (the vadose and saturated zones) up to the defined point of compliance. Thus, for many years and even up to the present day, in many projects and many jurisdictions, contaminant transport modeling through liner systems focused exclusively on estimating the number and size of defects in the geomembrane liners, estimating the head over the geomembrane liners, and applying some form of the Giroud equation in order to estimate advective leakage through the liner system. Subsequently, significant efforts and advances in the application of the analytical and empirical equations for estimating advective flow were made in order to consider various types of overliner and underliner conditions (Giroud et al. 1997b,c), estimates of leakage and hole probabilities through double-liner systems (Giroud et al. 1997a), the effect of high head levels up to 3 m (Giroud, 1997), the effect of very large holes (Touze-Foltz & Giroud, 2005), long defects (Giroud & Touze-Foltz, 2005), the difference between the leakage rates of GCL and clay liners below geomembranes (Giroud et al., 1997d, and Rowe et al. 2004), statistical data on hole size and frequency (Marcotte et al., 2009; Forget et al., 2005; Darilek & Laine, 2001), the effects of wrinkles on calculations (Rowe, 1998), and various assumptions on the number of holes per hectare (e.g. see Giroud & Touze-Foltz, 2003). The effectiveness of these models has been confirmed, if not calibrated in hindsight, by field performance data reported by Bonaparte et al. (2002).

Although the theory and modeling ability to consider diffusion was available well before the 25-year period we review here, the author's perception is that only in the past dozen years or so has there has more serious consideration been given to contaminant transport through liners via the mechanism of diffusion. This perception is by no means universal. A number of jurisdictions in various locations around the world may have seriously considered it earlier, and some jurisdictions do not consider it even to this day. That said, it was as recent as 2001 that the author was first required to consider diffusion as part of an alternative liner demonstration. See an interesting discussion of the state of consideration of this issue in 2002 in Giroud & Touze-Foltz (2003).

Rowe et al. (2004) suggest that under design conditions prior to termination of the operation of a leachate collection system and/or failure of a geomembrane liner, the primary transport mechanism through modern liners is usually chemical diffusion, most notably for certain organic constituents. As leachate collection systems age and clog, leachate mounding may occur. At the same time, geomembrane liners age and potentially develop more defects as they approach the end of their service life. As these 'elderly' system failures occur, and depending on the nature of the natural underlying hydrogeology, the issue of advective transport will become more significant. Rowe (2005) documented several cases in which diffusion resulted in measurable contaminant transport even over relatively small timeframes. Rowe et al. (2004) demonstrated that there can be significant diffusion of certain organic compounds through geomembrane liners even while the volume of advective leakage is negligible. They further argue that calculations should be performed to assess the adequacy of a liner system combined with the underlying attenuation-layer soils, to determine if adequate environmental protection is provided over the contaminating lifespan of the facility.

Summary: It is commonly acknowledged that all liners leak, with or without geomembrane defects. The concept of composite liners is well founded, and industry feedback has vindicated the rationale for these liner systems in the results obtained by performance monitoring. At the same time, we recognize that even though we have been installing composite liners with good results for more than a generation, the lifetime of many facilities will far exceed this initial 25-year period, in terms of their operating lifetimes as well as their post-closure lifetimes.

Future: Given the modest cost impact of adding a secondary liner and leakage collection system, it is reasonable to expect that the benefits of the long-term redundancy provided by double liner systems will continue to be recognized. These systems will therefore be promoted more broadly than they already are, both for the control of advective leakage and the greater reduction in diffusion that they provide. Because of our recognition that contaminant transport risks might be a function of the age of a particular infrastructure, some of the considerations described in the next section of this paper, which addresses aging durability, will likely come into play. The inclusion of diffusion considerations in contaminant transport analyses is likely to become increasingly prevalent in the prediction of performance. Also, ELL surveys will likely become more and more of a norm in the specifications for new liner construction, in order to reduce the number of geomembrane defects that appear early in the facility life.

7. AGING DURABILITY AND GEOMEMBRANE SERVICE LIFE

The industry is indebted to the service provided by Dr. Henry Haxo for contributions he made in the mid-1980s to our understanding of the long-term compatibility between various polymers and various chemical environments. This was the beginning of our understanding of the durability of aging polymeric materials in waste environments, and there have been many advancements and studies since that time. Although other papers in this conference may touch upon some of these same topics, the author felt obliged to include those aspects of the continuing research in this area that may influence immediate design decisions.

For buried applications that are intended to be a quasi-final solution (i.e., for which there is no long-term plan to replace, upgrade, or decommission the installed liner system), there have been few studies regarding the expected lifetimes of liners, except for those used with polyethylene materials. Polyethylene is the most studied and accepted polymeric material for long-life (multi-generation) projects (see interesting discussion on this in Giroud & Touze-Foltz, 2003). Although bituminous materials could perhaps be considered long-life as well, given that they are analogous to natural asphaltic material, they have not received a fraction of the attention and market share for containment lining systems as compared to polyethylene. Other polymeric membrane materials have also been used for long-term buried applications, PVC in particular, but estimated quantifications of their lifetime and aging-durabilities are lacking, and field exhumations have turned up mixed results. Other flexible membrane liner materials, such as PP, EIA, etc. have found excellent utilization in exposed applications such as reservoirs, ponds, and exposed secondary containment applications where there is an expectation of a finite-design lifetime on the order of one generation (i.e., 25 years, more or less). Though it would be fortuitous if some of these other materials had even longer lifetimes in service, at the present time they probably could not be used as design criteria, in the absence of more comprehensive aging-durability studies. There are a few excellent references on the selection of geomembrane materials for various uses, including Rollin et al. (2002), and Scheirs (2009). A number of attempts have been made in the past to create a selection matrix based on weighted or absolute criteria, and the author has even been involved in creating and evaluating such matrices. The author's experience with these matrices, though, is that they are too simplistic and do not usually lend themselves to an appropriate manner of evaluating materials. The best manner of evaluating materials is to clearly define the chemical, aging, and durability requirements for both long-term service and short-term construction survival. If more than one material happens to meet these requirements, then other considerations, such as cost, and the offsetting pros and cons can be used for the final selection.

Chemical compatibility aside (which is a large area in itself), prediction of the lifetime the various materials used in liners has been a holy grail from the beginning – how long will these materials last? However, the question itself requires some clarification if we are to answer it, for what does the question imply? Rowe (2012) has answered the question definitively for the industry: The lifetime of a geomembrane can be considered to have ended when it cracks so extensively that its presence as a fluid barrier is compromised on a massive basis over large areas.

Thus, to answer the question, "how long will the liner last?", we might do better to ask: "what will cause it to crack, and what will accelerate the appearance and propagation of cracks?"

Research related to the factors that affect the service life of a polyethylene geomembrane is described in the following paragraphs. For materials other than HDPE, insufficient data is available to assess their long-term performance in buried liners or covers (NRC, 2007).

<u>The basic chemical engineering of the resin itself</u>. Resins that are more crystalline, and therefore more chemically resistant, tend to have a lower stress-crack resistance. To this end, a great deal of work related to the development of stress-crack resistant resins has been done in the past 25 years, with perhaps the greatest changes made in the early 1990's. As a result, the term "high-density" polyethylene is now widely recognized as being a slight misnomer, because the density of all of the major "HDPE" geomembranes produced by the various manufacturers have been lowered to the point that they are technically considered a "medium density" polyethylene material, according to strict chemical engineering definitions. Nonetheless, the term "HDPE" is so ingrained in common usage that this semantic nuance is almost universally disregarded. The defining test with regard to stress cracking is the Notched Constant Tensile Load test, developed by the Geosynthetics Institute, and now the test standard established by ASTM as D5397. Just as a point of reference: while many resins in the late 1980s had NCTL transition times of less than 100 hrs, today one can easily obtain a resin with a transition time of greater than 1000 hrs.

<u>The anti-oxidant (AO) package in the resin formulation</u>. The effectiveness and lifetime of the additives that serve to protect the geomembrane from oxidation have been found to be a fundamental key to the ultimate lifetime of the geomembrane. While it is difficult for civil engineers to keep abreast of the nuances of polymer engineering, the GRI-GM13 standard guide is a good reference to use for specifying a good formulation for a geomembrane. The critical test in this regard has to do with the Oxidative Induction Time. Two tests are available that address this factor: Standard (ASTM D3895) and High-Pressure (ASTM D 5885). It appears that the High-Pressure test is much more indicative of the important anti-oxidants that contribute to long life (versus protecting against the high-temperature manufacturing process), and test values of 800 minutes are available from manufacturers.

Temperature. Temperature is a key factor in making long-term lifetime predictions for geomembranes. For example, it is estimated that the effective lifetime of HDPE at 40° C may be only 15% of its effective lifetime at 20° C (ref GRI 2010, or Rowe, 2005). Even short durations of exposure to high temperatures may significantly affect a geomembrane's lifetime (Rowe, 2012). In this regard, we would note the data from Koerner & Koerner (2006), which indicate that bioreactor landfill temperatures are significantly higher (on the order of 45-50 °C) than standard 'dry' landfill cell temperatures (on the order of 25-30 °C). Rowe (2012) also reported elevated landfill temperatures for wastes containing fly ash mixed with MSW, or wastes containing high aluminum content mixed with MSW (Stark et al., 2012a). Given the very significant effect of high temperatures on geomembrane service life, Rowe (2012) has suggested that the redundancy provided by secondary liners, considered separate from their operating temperatures, may effectively increase the service life of the overall liner system. The author would suggest that the monitoring of leachate temperatures just as they exit the landfill into sumps or pump stations can provide useful information feedback for site operators.

Allowable stresses and strains. Ongoing studies indicate that stress concentrations become the crack initiation sites for PE resins. Thus, the time-to-cracking, and the locations of cracks, will be largely influenced by the number of localized strains in the geomembrane. These types of strains may occur at all locations where gravel particles, which are present either in the subgrade or in the overlying leachate collection layer, cause the geomembrane to deform. Strains will also be found where there are wrinkles, which in North American installations are commonly prevalent in geomembranes as they are being covered. There is continuing debate and discussion regarding allowable strain level. For example an upper limit of 6-8% has been recommended by Peggs et al. (2005) and others, while the Germans are very conservative in their requirements, aiming for a design strain of 0.25% (Bishop, 1996) through the use of sand puncture-protection layers, and strict controls for the deployment-and-covering sequences used with geomembranes, in order to avoid burying wrinkles, for the reasons discussed above.

Past standard industry guidelines for allowable subgrade preparation and allowable overliner and puncture-protection materials have historically been established to promote geomembrane survivability and longevity of service. In light of ongoing research, these past standard practices were laudable and appropriate, and should be continued to be respected.

Recent research (Rowe, as yet unpublished) has shown that the highest quality subgrades are very stiff and smooth. Smoothness can be created using very well-graded materials that contain sands and silts, and even some fraction of gravel. Athanassopoulos et al. (2012b) have shown that subgrade soils that have a largely fine-grained soil matrix (e.g., clay liners) but some significant gravel content, can potentially result in significant geomembrane damage if there is some relative movement (e.g., due to seismic shaking), but that this potential damage can be avoided if a GCL is installed between this subgrade and the overlying geomembrane.

The author has repeatedly verified in the field that almost every soil subgrade over which GCLs and geomembranes are being deployed benefits from the application of water spraying and smooth-drum rolling within a narrow window of time before the geosynthetics are deployed. In this way, the displacement of soils and rocks caused by the geosynthetic deployment activities is minimized. It may be useful to include a general requirement of this method of execution in project specifications.

Regarding puncture protection from overliner materials, the industry standard for many years, and up through the present time in the USA, has been the geotextile puncture-protection formula developed by Koerner et al. (1996), and recently updated by Koerner et al. (2010). While geotextiles that are selected based on this approach may result in materials having a mass/area of 500-1000 g/m^2, the approaches used by the Germans (e.g., Witte, 1997) suggest that a minimum mass/area of 3000 g/m^2 is required, or in a more favorable approach, that sand blankets be used. Brachman and Sabir (2012) have shown that with coarse 50 mm drainage stone, even multiple layers of heavy NWNP geotextile protection totaling 2780 g/m^2 may allow exceedance of the suggested allowable strain values over time periods typical of a landfill design life. This has led to the suggestion that geomembrane service life can be extended by the use of a sand layer to provide puncture protection. Brachman and Gudina (2008) arrived at similar conclusions for drainage stones of sizes of 25 mm and 50 mm. The degree of geomembrane strain can be kept lower if a finer gravel, or a more well-graded gravel, or more rounded as opposed to angular gravel is used, or if a firmer subgrade is created. We would note that the tendency towards a smaller, more well-graded gravel size is in conflict with the desire for a larger, more well-sorted gravel size to resist leachate collection system clogging. Designers thus need to weigh the trade-offs involved and provide appropriate puncture protection for a given situation. More definition can be expected in the future with regard to the use of acceptable overliner gravel sizes in conjunction with various puncture-protection strategies.

Wrinkles. The subject of wrinkles is mentioned many times in this paper, partly because they are so endemic to liner installations, and partly because they affect many aspects of the function and integrity of the liner systems. Giroud & Morel (1992) provide an excellent theoretical understanding of the cause and prediction of wrinkles for different types of geomembrane materials in different conditions. NRC (2007) makes the simple observation that “wrinkles are common in North American landfills”. In the author’s experience this is likewise true in most of the world, except for Germany, where they insist on wrinkle-free geomembrane installations. While there has been no definitive standard on the control of wrinkles in geomembranes in North American installations, research over the past 25 years has progressively shown more and more why wrinkles are undesirable. The reasons for this include strains in the geomembrane that result in accelerated stress cracking at the locations of wrinkles (Rowe 2005), greatly increased leakage potential due to the loss of intimate contact with the underlying clay or GCL liners over significant areas (Rowe 2005), the reduced effectiveness of lateral drainage layers on top of geomembrane wrinkles, and the increased potential for construction damage, because wrinkles protrude upwards into the path of grading equipment blades. Thus, the most likely places where leaks will originate are also the worst places. Daniel and Koerner (2007) indicate that “the geomembrane must be flat when it is backfilled”, and they provide a list of proactive measures that can be taken to minimize the incidence of wrinkles. Scheirs (2009) suggests that at times, a compromise must be sought between the conflicting requirements of minimizing wrinkles while avoiding ‘trampolining’. The author’s practice in North American installations has been to set a maximum allowable wrinkle height of 2-3” (50-75 mm) during covering operations, and to use variations in daily temperatures to maximum advantage during this process. Using daily temperature as a primary control makes sense, since fundamentally, a geomembrane must be covered at or below its deployment temperature in order to achieve a wrinkle-free installation (Take at al., 2012). The question thus arises: on what basis should an acceptable wrinkle height be set? Chappel et al. (2012) showed that when the total area of wrinkles is less than 8-10% of the total area, then the maximum interconnected wrinkle length will generally be less than 200 m, which they considered an acceptable maximum length for the control of advective leakage due to random holes. At the sites that they studied, they observed that at geomembrane surface temperatures below 37 ºC, the interconnected wrinkle length was typically less than 100 m. A related study by Take et al. (2012) indicates that the wrinkle height at this same temperature was estimated to be between 50-75 mm, which precisely corroborates the author's experience-based specification. This approach cannot be taken so simply, though. If one reads the references and field studies carefully, it is actually the *onset of wrinkling to those heights* where the interconnectivity between wrinkles becomes a problem. In general, soil covering over polyethylene geomembranes needs to be halted a few hours after sunrise and can be begun again a few hours before sunset, with the exact times being specifically related to the maximum interconnected wrinkle length. Other approaches to limiting this maximum length could be as simple as placing sandbags between wrinkles to prevent their interconnection. This simple concept, however, may be not so simple to implement. We would note that these various approaches do not address the reduced service life at the locations of wrinkle. Take et al. (2012) suggest that any wrinkles with a height of 20 mm or more at the time of burial are likely to remain forever trapped, and will thus likely become sites of stress concentrations to varying degrees, depending on the particular wrinkle geometry.

<u>Thickness</u>. All other things being equal, thicker geomembranes will be more durable during construction, and will last longer (Rowe et al. 2010).

8. SLOPE STABILITY

Significant advances in our understanding, testing, and evaluation of slope stability have been made in the past 25 years. In landfill slope stability issues, the distinction is often made between bottom-liner stability and veneer (e.g., final cover) stability. Although the underlying geo-mechanics that govern stability are the same in both cases, the normal loads and sensitivities to pore pressures are vastly different, so it is useful to discuss them separately.

Bottom Liner Stability Issues

At the beginning of this paper's 25-year retrospective period, we had a major bottom-liner failure at the Kettleman Hills landfill in 1987. This was, and still is perhaps, the largest permitted hazardous waste landfill in the world. The liner system was fairly complex, as shown in Figure 1. This event created a great awareness of slip surfaces and the need to check the slope stability in lined containment systems, and perhaps inspired more technical papers on slope stability than any other containment project (e.g., Filz et al., 2001; or Stark and Poepple, 1994). The key lessons learned from this failure include:

- The importance of phasing and fill planning relative to slope stability. The failure would never have happened if the filling of the landfill had been performed in a proper sequence. The landfill operations essentially attempted to fill one side of a slippery bowl. If the other side of the bowl had been constructed sooner, and if the landfilling had been more balanced between the two sides, the failure would never have occurred. Since that time, this same lesson, unfortunately, has been learned the hard way at a few other major landfill projects. There is really no need for anyone else to have to learn this lesson; we can all definitely learn from this one mistake that occurred in 1987.

- Stark and Poepple (1994) used this failure to suggest to the industry that it might be good to use post-peak shear strengths on the backslopes of lined containment areas, while peak strengths may or may not be acceptable on the base areas, depending upon the seismic hazard. Ten years later, Stark and Choi (2004) elegantly refined this discussion.

- Filz et al. (2001) and Esterhuizen et al. (2001) wrote a pair of landmark papers on the progressive mobilization of shear strength in lining systems, using the Kettleman Hills failure as their case study. A similar paper was written by Reddy et al. (1996). The key lesson we can learn from these papers is that the mobilization of shear strength is not equal and uniform as is assumed by limit-equilibrium analyses. It is of overwhelming importance for practitioners to understand that the rigid-block modeling of slope stability, which is what is done in all 2-D limit-equilibrium models, does not represent the dynamics of reality. In fact, the shear strength is mobilized in a very specific localized fashion, such that all failures are essentially progressive failures. This is a significant reason why it is prudent to assume post-peak shear strengths for significant portions, especially the backslopes, of lined facilities. While the author does not advocate that slope stability analyses should use the finite-element approach in general, it is valuable for practitioners to study these references in the literature in order to gain an appreciation of the true mechanics of shear strength mobilization.

In the same vein as the lessons learned from the fill sequencing at Kettleman Hills, Smith and Giroud (2000) and Breitenbach (1997) emphasized that heap leach mining projects have also demonstrated that filling considerations for the first lift are the most important, relative to slope stability, and are typically more critical than the final fill configuration.

Based on the Kettleman Hills and similar solid waste failures, as well as numerous failures in the mining industry, it has become clear that the critical situations for slope stability risk are during construction, in the early phases of filling, and in the critical intermediate phases of filling. This concept must be borne in mind, even though many regulatory requirements have historically required only the examination of the final fill configurations to ensure slope stability. Thus, a key message for geotechnical practitioners involved in such projects is that not only do these different stages of geometry need to be modeled, but the shear strength testing for critical interfaces needs to be conducted at normal loads appropriate to these different conditions.

Peak vs. Post-Peak Strength. Partly as a result of the Kettleman Hills failure, but also because of the greater awareness and emphasis on slope stability evaluations in general, much work and many publications have been dedicated not only to understanding the shear strength characteristics of different materials and interfaces, but also to coming to grips with which shear strengths should be used relative to peak strength, post-peak strength, or residual strength. Some key publications in this regard include:

- ASTM D7702 - Standard Guide for Considerations When Evaluating Direct Shear Results Involving Geosynthetics. This recent guideline provides a very good discussion of the relevant issues related to the commissioning and interpretation of shear strength testing, and provides a good list of references that include many key related publications. This ASTM Standard reviews important issues such as the fundamental aspects of measuring and reporting shear strength using ASTM direct shear methods; evaluation of the Mohr-Coulomb envelope over appropriate normal load ranges; how to evaluate the subject of cohesion (or adhesion); how to evaluate the shear-displacement curves reported from tests; suggestions for reviewing results and test methods against historical data and published guidance; the value of inspecting specimens after testing; and multi-interface test approaches.

- For a modern reinforced GCL-geomembrane interface, and internal GCL shear strengths at moderately high normal loads, the author considers Fox and Ross (2011) to be the most comprehensive and relevant paper, as it provides useful insights on hydrated GCL shear strengths. This publication also includes references to numerous other key papers on the subject.

- In the same vein, an interesting publication on geomembrane and GCL interfaces under ultra-high normal loads, such as those that are experienced in large heap-leach facilities, was published by Athanassopoulos et al. (2012).

- A comprehensive discussion of unreinforced GCL shear strength, under both hydrated and dry (encapsulated) conditions was published by Thiel et al. (2001).

- There has been discussion of different approaches to the selection of peak versus post-peak, or even residual, shear strength parameters for use in slope stability analyses, as discussed by Thiel (2001). The Ohio EPA (2004) presented the interesting approach of requiring residual interface shears strengths for all liner systems on slopes steeper than 5%. GRI (2011) considered the selection of shear strength to be the "most-sensitive-unknown-variable" in performing stability analyses.

- When assigning post-peak shear strengths, a standard industry approach has been developed (Thiel 2001) that uses the post-peak shear strength of the interface that has the lowest peak strength. It is important for this concept to be applied for specific normal load ranges, since the lowest peak strength may shift from one interface to another under different normal load ranges. For example, for interfaces involving textured geomembranes with reinforced GCLs, it is commonly acknowledged (e.g., Athanassopoulos et al. 2012) that at increasing normal loads there comes a point at which the peak GCL internal shear strength is lower than the strength of the interface between the textured geomembrane and the hydrated GCL. Thus, if a post-peak strength analysis were being conducted, it would be necessary to determine the normal load at which the transition from the interface to the GCL internal strength would take place.

- A number of designers and academics (e.g., Gilbert and Byrne 1996) recommend that a factor of safety greater than one be achieved in all containment system slope designs, assuming that residual strengths are mobilized along the entire slip surface. This view was later echoed by Stark and Choi (2004).

<u>Pore Pressure</u>. There has been an increasing recognition of the importance of considering pore pressures in landfill and heap leach stability analyses (Thiel, 2001; and Castillo et al., 2005). In its 2011 report on 20 large landfill failures, GRI found that liquids were considered a major mobilizing factor in more than half of the failures, and liquids in general were involved in all the failures. It is thus worth highlighting the fact that after gravity, pore pressures are the single most significant destabilizing element, especially for sites practicing liquid recirculation. It is known

that at least one major landfill failure was caused by the aggressive recirculation of leachate (Hendron et al. 1999).

Method of Analysis. The actual methods of analysis that are used have not changed appreciably over the past 25 years. Slope stability analyses are most commonly assessed using computer programs that evaluate the limit equilibrium of a 2-D cross-section. Although 3-D and finite-element analysis methods are available, from a pragmatic point of view, the everyday stability analysis has been and will continue to be 2-D in actual practice. There are three main reasons for this, which are clearly laid out by Duncan (1996): the inherent conservatism of the practice, its ease of application, and its avoidance of errors. One aspect of this practice worth noting is that the judgement of experienced practitioners is required in order to select critical cross-sections.

Acceptance Criteria. Apart from the mechanics of selecting appropriate material properties, geometries, phreatic surfaces, and methods of analysis, the other significant decision that must be made by the geotechnical practitioner regards the acceptance criteria. Traditionally, static stability analyses were evaluated on the basis of a "factor of safety" (FS); based on the historical practices in the profession, a value of 1.5 is commonly considered acceptable. For seismic analyses (in California, for example), 25 years ago the industry was commonly using variations of a "pseudo-static" analysis that used a predetermined horizontal acceleration roughly on the order of 0.15 g, and required that FS $\geq$ 1.0 under those conditions. While both of these acceptance criteria are still in use, more sophisticated and appropriate approaches have been developed in the past 25 years.

The legacy of requiring a minimum static factor of safety of 1.5 is still a relatively universal standard in the solid waste industry, although the author has noted that in many heap leach facilities, values of 1.3 are commonly accepted. The acceptance of a specific value implies a certain reliability of all of the factors that comprise the analysis, such as the geometry, shear strength parameters, and phreatic surfaces. Given that every single input going into an analysis involves some degree of uncertainty, a more intelligent manner of evaluating slope stability is based on the concept of 'reliability', or its inverse, namely 'probability of failure'. The implementation of a reliability analysis was made very approachable for the industry at large by a landmark publication by Duncan (2000). In this paper, Duncan provided the tools (spreadsheet equations) and approaches needed to create the necessary input to estimate the reliability of just about any type of engineering analysis, including slope stability. Design practitioners in all disciplines are highly encouraged to obtain this reference and follow its methods for all significant calculations. The output of the exercise will result in a "most-likely-value" factor-of-safety, as well as a reliability value (or its inverse, probability of failure). A significant benefit of this approach is that the manner in which decisions that affect slope stability are made can be easily communicated to the project sponsor, and the risks involved in making those decisions can be shared. While this approach will give both the designer and the client a feel for the sensitivities of certain parameters related to the project's reliability, it still begs the question of what is an appropriate probability of failure. Guidance on this can be found in other industry publications, such as Whitman (1984) or D'Hollander (2002).

Seismic. Seismic analyses for landfills and leach pads have undergone significant advances. The "design earthquake" definition used in the 1980s used concepts of 'maximum-probability' and 'maximum-credible' earthquakes. With the enactment of Subtitle D in 1993, the design earthquake for permitted landfills was defined as an event that would have a "10% probability of occurrence in 250 years". Since that time, the USGS has developed excellent online seismic hazard analysis tools, which are available at http://earthquake.usgs.gov/hazards/. If one inputs a latitude and longitude, the website will provide the peak ground acceleration for the selected probabilistic design event, mean and modal moment magnitudes, as well as spectral accelerations for a range of fundamental periods.

Early in the 25-year period under discussion, in addition to the pseudo-static method described above, the solid-waste industry adopted a chart-based deformation-analysis approach that was based on the use of the estimated yield acceleration and the assumed Richter magnitude of the design earthquake. This approach was based on the Newmark method (Newmark, 1965) as developed by Makdisi and Seed (1978), which is used to estimate the earthquake-induced deformation of dams and embankments. Bray et al. (1998) significantly advanced and refined this chart-based deformation concept. The most recent widely used chart-based approach is the one published by Bray and Travaserou (2007). More sophisticated approaches include performing a 1-dimensional seismic response analysis using an actual or simulated earthquake motion that is input to a computer program such as SHAKE, and performing double-integration on the acceleration vs. time response in order to estimate Newmark-type sliding magnitudes, or conducting 2-dimensional finite-element site-response analyses for actual or simulated time histories. The latter approach is typically used only in situations in which the more simplified conservative analyses do not provide acceptable results. Regardless of the method used, there is still the question of what constitutes an acceptable result. At this point, the main reference that is cited in the industry is by Seed and Bonaparte (1992), who conducted a survey of leading engineering firms, and found a general consensus that acceptable deformation under a design earthquake should be limited to no more than 150-300 mm.

Future: Going forward, the key questions relative to slope stability will continue to revolve around the selection of an appropriate shear strength and the consideration of appropriate pore pressures. Questions still remain regarding the appropriate selection of peak vs. post-peak shear strength parameters, long-term durability with regard to interface and internal shear strengths, and controls and impacts of waste saturation on stability. The importance of these issues still remains to be fully appreciated, especially as bioreactor technology continues to expand, and as leachate collection and removal systems (LCRS's) continue to clog.

Veneer System Stability Issues

Veneer systems in landfills that might experience stability issues are those on sloped locations of final covers systems, or bottom-liner sloped areas that have a geomembrane and/or GCL barrier that is/are covered with relatively thin layers of leachate collection and/or operational soil materials during construction. Because this layer is often relatively thin compared to the slope length, these systems are commonly and conservatively treated as 'infinite slopes' from a slope stability point of view, although toe-resistance can be incorporated if desired. From this point of view, a veneer situation can be simply considered to be a block on a sloping surface.

While significant failures of landfill bottom liners may number only a couple of dozen in the past 25 years, veneer failures undoubtedly number in the hundreds. Many of these probably go unreported, and those that make the industry news-circuit are sufficient testimony that they are a significant problem. The geotechnical aspects of final cover system slope stability follow the same principles as those used for other geotechnical stability problems. What is unique and important to recognize relative to final cover systems is their sensitivity to relatively small changes in loading, slope angle, pore pressures, or shear strengths, all of which make them more susceptible to sliding failure (Thiel 2008).

While the author does not have a statistical list to quantitatively present the causes and triggering mechanisms of such failures, his familiarity with industry issues and discussions with peers over the past 25 years indicate the following reasons for veneer failures:

- Pore pressures acting at the liner interface. There are two types of pore pressures that can act at the veneer-cover interface: those from above, and those from below.

- Pore pressures above the geomembrane. The most pervasive cause of veneer failure is liquid (namely, rain) from above that percolates through the topsoils and builds up over the liner. It is easy to demonstrate that if there is no drainage layer between the geomembrane and the cover soil, the factor of safety for the stability of the soil sitting on the liner is in one stroke cut roughly in half. This is explicitly described, along with a number of design solutions, by Thiel and Stewart (1993), and in several other follow-up publications by others in the industry, such as Giroud et al. (1995b) and Koerner and Soong (1998). The main principle of the design solution is clearly to provide a dependable transmissive layer, such as gravel or a geocomposite drainage layer, between the geomembrane and the soils. The author notes that one of the changes in the industry over the past 25 years has been the elimination of the misnomer “cap-net”, which was used to refer to less expensive geonets that were being promoted for landfill caps, under the perceived view that a lower level of performance would be needed in caps than in bottom liners. Analyses very often prove that the transmissivity requirements for landfill covers may significantly exceed the requirements for bottom liners due to slope stability concerns. The largest transmissivity reduction factor for these applications is typically “biological”, as plant roots have been demonstrated to reduce transmissivity by 50%.

- Pore pressures below the geomembrane. Landfill gas pressures that are exerted upon the bottom surface of a cover geomembrane have caused several documented veneer-cover slope failures in the industry. Again, the design solution is to provide a lateral drainage layer below the cover system that collects and ultimately vents these pressures. The only industry design methodology published to date is that of Thiel (1998).

- Lubrication at the geomembrane interface. The author is aware of two types of “lubrication”-induced veneer failures. One is caused by the extrusion of bentonite from the woven side of a needle-punched GCL against a geomembrane. Such a failure was well-documented at a test site in Cincinnati (Daniel et al., 1998). A GCL was placed on a moist subgrade with the woven side face up, and was then covered by a geomembrane and 3 feet of soil on a 2:1 slope. Shortly after installation, two of the test plots slid. This demonstrated that bentonite extrusion from the woven side of a GCL needs to be taken into account if it is placed against a geomembrane, or perhaps that it is best to use a non-woven geotextile-based GCL against a geomembrane. The other “lubrication” failure, which was documented by Thiel (2009), involved a PVC geomembrane that slid on a nonwoven geotextile on a 4:1 slope about three months after being covered with soil. In the forensic investigation it was discovered that the condensation of moisture between the PVC geomembrane and geotextiles significantly decreased the interface shear strength, as compared to the dry interface. The lesson in this case is that interface shear tests with geomembranes should always be sprayed with water before assembling the test sandwich, because this truly replicates field conditions.

- Equipment-loading-induced failures. It has been documented that inertial forces from equipment can locally cause the peak interface strengths to be exceeded, resulting in post-peak interface strengths. Progressive failures of this type, which have eventually led to large-area slope failures, have been evaluated by the author and documented in the literature (e.g., Stark et al. 2012b, Thiel and Narejo, 2005). The lessons learned in general are that small, wide-track, low-pressure equipment should be used to spread cover soils over geosynthetics, and that the pushing of soils should be done in an uphill, not a downhill direction. Also, the failure evaluation presented by Thiel and Narejo (2005) led to a change in the industry-standard for geocomposite peel strength, which was upgraded from 0.5 pound-per-inch (ppi) to a MARV value of 1 ppi for projects in which shear strength is a critical factor.

- Simple failures due to inadequate shear strength. 'Simple' veneer-type failures due to inadequate interface shear strength, without any other extenuating factors, have been regularly experienced in the industry. These are typically non-dramatic and are not popularly documented because they are corrected as soon as they are discovered.

- Other. There are other types of veneer slope failures, such as veneer-reinforcement failures, that are less common and more specialized.

As with bottom-liner systems, perhaps the "most-sensitive-unknown-variable" is again the assumed shear strength parameters. In veneer cover design, it is critical for testing to be performed within the low normal load ranges under consideration. One of the most significant abuses of the interpretation of laboratory shear strength data is the cavalier backwards-extrapolation of the cohesion intercept. This is typically a very unconservative approach that should really never be used, especially for the low normal load regimes for cover systems.

Seismic analyses for final cover systems can use the same chart-based methods described in the preceding section. The author recommends the Bray et al. (1998) reference as the most appropriate method, since it offers explicit solutions for the tops of landfills, taking into account amplification up through the waste mass.

As described in the preceding section, a significant decision that must be made by the geotechnical practitioner regards the acceptance criteria. As with bottom liners, FS = 1.5 is a commonly accepted basis of design for covers. As regards the allowable deformation in seismic analyses, the most germane reference is Kavazanjian (1998), who suggests that deformations of 1 m or more are allowable, as long as the responsible party is ready, willing, and funded to make repairs. If it is desired that the damage caused by the design earthquake would be relatively insignificant, then the estimated deformations should be less than 150-300 mm.

Future: Going forward, the key questions relative to veneer slope stability will continue to revolve around the selection of an appropriate shear strength and the consideration of appropriate lateral drainage layers above and below the barrier layer (viz. geomembrane). The long-term reliability of the assumed drainage layer transmissivity is also a subject of concern, as is described later, in the section 'Lateral Drainage Design'.

9. STEEP-SLOPE BOTTOM LINER SYSTEMS

As the industry has developed more confidence in liner-system construction over the past generation, and as the challenge of siting new waste repositories has become more difficult, steep-slope locations such as sidehills, canyons, and quarries have become more common disposal sites. Steep-slope bottom liner systems require special consideration not only in terms of slope stability, but also in terms of subgrade preparation, installation challenges, wrinkle management, covering, and the effects of settlement and downdrag on the liner systems. Each of these issues is discussed briefly below.

<u>Subgrade preparation</u>. The specifications for appropriate subgrade preparation should really be no different for a steep-slope application than they are for a flat-bottom application, and yet the achievement of a firm, smooth surface is much more difficult on a steep slope. The contractor's resourcefulness often makes it possible to successfully prepare firm, smooth surfaces on soils as steep as 1.5(H):1(V). At other times, engineered solutions such as puncture-protection layers, geofoam, or shotcrete may be in order.

<u>Installation challenges</u>. Installer ingenuity is required to safely deploy and seam geosynthetics on steep slopes. Special considerations include the effort to avoid overly disturbing the prepared subgrade, welding machine burnouts, CQA access, and wrinkle management. Because of the effect of gravity and daily expansion and contraction, the management of wrinkles at the toes of long slopes can be troublesome, especially for multiple layers. Often there is no good choice but to cut out the multiple layers of wrinkles and add a repair-seam near the toe of the slope. This activity is best done after the soil cover layers have been placed on the bench at the toe of the slope, so that trampolining of the lining system across the bench does not occur. For several reasons, including the problem of wrinkles, it is advisable for benches to be installed at maximum vertical intervals of approximately 15 m.

<u>Settlement and downdrag</u>. Settlement and downdrag along steep-lined slopes will occur to varying degrees during initial construction, initial landfilling, and long-term waste settlement. The key questions in this regard are: "how much downdrag will occur?" and "at what degree of slope inclination does downdrag become a concern?" There has been little field monitoring and limited research on this subject. The research that has been done indicates that at a slope inclination of perhaps 2(H):1(V) or steeper, downdrag could be a significant concern (Jones and Dixon, 2005), but it could occur on flatter slopes as well, depending on the forces and the relative interface shear resistances within the liner system (Lui and Gilbert, 2003).

The occurrence and amount of downdrag that will ultimately occur is acknowledged to be a quite complex matter, and perhaps beyond our ability to accurately predict. In the design of steep slopes, the author believes it is necessary for the design to be able to accommodate downdrag without incurring any damage to the primary liner system. In the author's opinion, protection of a steep-slope liner against downdrag damage can be accomplished in two ways. One is to provide one or more preferential slip surfaces above the primary liner. The second is to provide a veneer-reinforcement layer above the primary liner. The two methods can also be combined. For example, a slip surface consisting of a single-sided geocomposite could be deployed over a geomembrane on a steep slope (see Snow et al., 1994, as perhaps the first published example of this). The interface between the bare geonet and the geomembrane is much more slippery (i.e., has less shear resistance) than any other interface in the lining system, and will therefore slip when there is relative movement above the primary liner due to waste settlement and downdrag. In a geocomposite, the greatest stress due to interface slippage will be located near the crest of the slope, and at some point it will rip at that location. If a high-strength geotextile is anchored on the bench at this location, directly beneath the geocomposite, then as the geocomposite continues to be dragged down-slope, an ever-greater window-area of the high-strength geotextile will be exposed to the overlying soil that is engaging the downdrag forces. As this is taking place, a high-strength geotextile can provide two functions: (1) it can protect the underlying primary geomembrane from being directly exposed to the overlying soil materials, and (2) it can bear the downdrag load. Depending on the normal forces and downdrag forces, the high-strength geotextile would eventually reach its load-bearing capacity when the window of exposure reaches a certain size, and at this point the high-strength geotextile would rip. The design engineer would have to make an estimate whether or not the total amount of downdrag movement would exceed the ripping-point of the high-strength geotextile. If it was deemed probable that the downdrag movement would continue past this point, then a second layer of high-strength geotextile could be installed below the first layer, thus allowing another sequence of downdrag movement before this layer, in its turn, has its tensile strength challenged. This paper presents the first reported description of this technique; a more complete and publicized exposition is planned in the coming year.

The Future: We can expect more finite-element modeling of stresses and strains in steep-slope liner systems, and hopefully, more controlled instrumentation and reporting of actual steep slope liner systems.

10. PIGGYBACK LINER SYSTEMS

As old landfills reach their capacity and new sites become difficult to find, the practice of filling a new lined landfill against the side, or on top of, of an old existing landfill has become an attractive option that is routinely considered, and will always be a potential choice in local solid waste management plans. While these "piggyback" lateral and vertical expansion designs may not be everyday events, they will also not be rare. In addition to the typical design issues related to slope stability, the design goals for such systems are generally to allow the leachate and gas generated from the new waste mass to be effectively captured and directed to the new leachate collection and removal system, without leaching through the old waste mass that likely has a less reliable, or nonexistent leachate collection system. The collection of landfill gas from the old waste mass below the new liner system may also be an issue, but this is a separate discussion that is outside the scope of this paper.

A piggyback liner system has to be able to withstand the anticipated future total and differential settlements from the underlying waste foundation upon which it is constructed. Thus, one of the design tasks would be to model the future settlement of the waste mass upon which the piggyback liner will be installed, in order to verify that the anticipated total settlement would not result in a substandard slope (which would mean a generally established minimum of 2% of residual slope), for drainage of the new leachate collection system on the piggyback liner. Design approaches could include compacting the existing waste (which could even be as extreme as deep dynamic compaction), and steepening the slope of the existing waste using soil or non-putrescible waste, so that when it settles, there is less of a chance of it becoming too flat.

Localized differential settlements, e.g., sinkholes that might develop due “rusty refrigerators” and thus create a void in the underlying waste, can potentially be managed with ‘brute force’, i.e., a geogrid or high-strength geotextile reinforcement below the new liner system. The lateral extent of effective protection provided by subgrade reinforcement is limited to something on the order of 3 m in diameter, plus or minus, depending on the depth of the overlying new waste and the type of reinforcement selected. Original discussion and details can be found in Giroud et al. (1990), and excellent summaries are presented in a number of informative PowerPoint presentations that are available from the Geosynthetics Institute (Folsom, PA) and Koerner (1994).

11. LATERAL DRAINAGE DESIGN

The intelligent use of lateral-drainage design elements is recognized as an aspect of containment-system design that requires perhaps the most insightful and experienced engineering talent that can be brought to bear, because their impacts on slope stability and containment are so highly significant. In the world of geosynthetics, it is traditionally understood that geonets provide a lateral drainage function. Although cuspated drainage panels have also been used to provide lateral drainage, their use in containment systems is relatively rare, and the author has never used them in these applications. There is one ‘nubbed-surface’ geomembrane product that, when covered with a geotextile, provides in-plane drainage, and the author has successfully used this. While some very specialized, loosely needle-punched, very-heavy denier nonwoven fabrics have been used for the lateral drainage of gases, in general geotextiles are not by themselves appropriate for lateral drainage applications.

At the beginning of the 25-year period under discussion, only a limited selection of bare bi-planar polyethylene geonets was available for geosynthetic lateral drainage materials. If a geotextile filter was needed adjacent to the geonet, it had to be deployed separately. Beginning in 1988, the ability to heat-laminate nonwoven needlepunched (NWNP) geotextiles to one or both sides of geonets was developed, and many different thicknesses and cross-sections of bi-planar and tri-planar geonets are now being offered. The heat-lamination of a geotextile to a polyethylene geonet is performed by melting the outside of the polyethylene geonet and then pressing the NWNP geotextile onto the melted surface. The lamination is actually created by the mechanical grip of the cooled polyethylene around the geotextile fibers. The hotter the melted surface, and the more pressure that is applied while it is melted, the more securely the geotextile fibers will be ‘gripped’ by the cooled polyethylene, though to the detriment of the geonet’s transmissivity. The integrity of the lamination is measured in an index test called a ‘ply-adhesion’ or ‘peel’ test (ASTM D7005). Originally, manufacturers only specified a minimum average roll value (MARV) of 0.5 lb/inch (ppi) for this property. As described above in the section ‘Veneer System Stability Issues’, a couple of slope stability failures where this interface failed led to a new industry MARV of 1.0 ppi. The manufacturing control of this heat-laminated interface has greatly improved over the years, to the point where a more uniform and consistent peel strength is achieved, with less ‘holidays’. The lamination of geotextiles over geonets has become so common that one rarely finds bare geonets being used anymore. The ability to laminate geotextiles not only provided a commonly needed filter against one side of the geonet, but also provided a good frictional interface against textured geomembranes on the other surface. The combination of a geotextile laminated to a geonet forms a 'geocomposite', and these have become so prevalent that the term 'geocomposite' is practically an industry synonym for this type of product, even though the term 'geocomposite' is generic and could refer to any number of geosynthetics that are combined together. In the discussion that follows, the author will use the term 'geocomposite' to intend both geonets that are laminated with geotextiles, and nubbed-geomembranes that create a transmissive layer when covered with a separate geotextile.

The most important performance property of a geonet or geocomposite is its transmissivity, which is essentially equivalent to its in-plane hydraulic conductivity multiplied by its thickness. Units of transmissivity are gallons per minute per foot of width (English units), or cubic meters per second per meter of width, which ends up being reported as m^2/s. Testing and specifying the transmissivity of geonets and geocomposites has been greatly refined and improved over the past dozen years. The most significant advance came with GRI Test Standard GC8 - Determination of the Allowable Flow Rate of a Drainage Geocomposite (Geosynthetic Institute, Folsom, PA) in 2001. In this test method, the manufactured material is tested against geosynthetic or soil super- and sub-strates that are representative of the field conditions, under normal pressures that are representative of field conditions, for 100 hrs. The material should also be tested under a gradient equal to or greater than the design gradient (never at a lower gradient), in accordance with ASTM D4716 (Test Method for Constant Head Hydraulic Transmissivity (In Plane Flow) of Geotextiles and Geotextile Related Products). The GC8 method provides that appropriate reduction factors should then be applied to the test results in order to account for long-term geotextile and soil intrusion, biological clogging, chemical clogging, and creep. Finally a global factor of safety should be applied to this value. Excellent discussions regarding this approach are provided in Giroud et al. (2000a).

A very fundamental aspect of the discussion of the use of lateral drainage layers in all applications is the fact that all design methods assume an unconfined in-plane flow. If this assumption is violated, then all the calculated benefits of lateral drainage layers in terms of reducing leakage, or reducing pore pressures to preserve slope stability, may be invalidated. To this end, Giroud et al. (2000a) has provided simple and elegant design approaches for designing the required transmissivity of the lateral drainage layers for given conditions of liquid input, slope, and drainage outlet spacing. In addition, Giroud et al. (2000b) has shown that a relatively thin geocomposite drainage layer requires a greater transmissivity than a much thicker granular drainage layer in order to obtain an equivalent flow capacity and maintain unconfined flow. Additional design equations for compound slopes, stacked double-drainage layers, and radial flow can be found in the same special publication as the Giroud et al. (2000a and 2000b) references.

Five specific areas of containment engineering design in which lateral drainage plays a major role are described below, along with advances in our understanding, and current design approaches that employ geosynthetic lateral drainage layers.

<u>Bottom-liner primary LCRS in landfills (or "overliner" drain layer in heap leach pads)</u>. A cornerstone of the USEPA Subtitle D and C regulations is the requirement of an LCRS (which is a lateral drainage layer) that limits head buildup on the liner to a maximum of 30 cm. The initial recommendations in the 1980s for minimum required permeabilities of 1E-03 cm/s were much too low to ensure good long-term performance, and tended to result in short- and long-term

clogging. Primary landfill LCRS's clog to a significant degree over time. Rowe (2005) presents several examples and literature citations that document cases in which systems lost over 3 orders of magnitude of permeability over a 4-10 year period. Clogging is cited as being caused by the combination of biological and inorganic precipitate, with the inorganic precipitate being a long-term clog residue that is composed primarily of calcite.

A landmark paper was provided by Koerner et al. (1994), regarding large-scale drainage correction factors (DCF) for LCRS's, in which DCF was defined as the total area of the LCRS divided by the geotextile filter flow area. The extremes of this factor are DCF = 1, for the case in which a blanket filter covers the entire LCRS gravel collection layer, and DCF =24,000, for when a perforated pipe is wrapped with a geotextile filter and all the leachate must flow through the portions of the geotextile filter that cover the perforations in the drainage pipe. An intermediate value of DCF = 40 might exist where the geotextile filter is wrapped around the gravel envelope immediately surrounding the leachate collection pipes. Experience has shown that relatively thin blanket filters, with DCF = 1, provide a leachate-treatment function that reduces the clogging of the LCRS layer (Rowe 2005), without becoming overly clogged themselves. Wrapping geotextiles around pipes or constrained areas is not recommended. Rowe & Van Gulck (2003) describe how the presence of a blanket filter performed better than no filter, and how a NWNP filter performed better than a woven filter. The author's practice is to provide a lightweight (135 g/m^2) NWNP geotextile as a compromise, thus providing a filter material that will provide the filtration/treatment benefit and will be less prone to clogging than heavier materials.

The author has found that designers use geonets and geocomposite drainage layers in primary LCRS's with caution, because of the clogging issue, and with even more caution in the case of 'wet' landfills in which liquids are recirculated. Even if new geocomposite drainage layers can be shown to permit a flow equivalent to that of a 30-cm-thick coarse granular system, they may have only one-tenth of the total porosity, and are therefore much more susceptible to clogging when put into service. Also, it is commonly accepted that a 60-cm-thick layer of soil material is typically required as a minimum protective layer above a geosynthetic liner prior to waste placement, so using a 30-cm -thick granular drainage layer already satisfies at least half of that requirement. If gravel drainage materials are not readily available, and there is a temptation to use geosynthetic drainage layers as the primary LCRS material, the author would still suggest providing, in general, 'gravel windows' that are 60-cm thick by approximately 4-m wide (the width of construction equipment) that cover the major leachate collection pipes and sump area, and a total coverage of at least 15% of the surface area of the landfill bottom.

In the future, we can expect more research related to reduction factors for clogging, and recommendations for providing greater redundancy in permeability, slope, pipe size, and spacing in LCRS's.

<u>Leakage detection layer in landfill bottom liner systems (secondary collection)</u>. Double-lined landfills with leakage detection layers (also called secondary LCRS) are common in the USA because they provide redundancy of environmental protection and performance feedback. Indeed, the limited feedback we have on the excellent performance of composite liner systems is mainly a result of the double-lined landfill data provided from New York landfills (Bonaparte et al., 2002).

There are generally two key design criteria for properly designed leakage collection layers: (1) provision for rapid reporting of a significant leak in the primary liner system to the secondary sump (a 24-hour detection time is commonly specified), and (2) limiting of the head acting on the secondary liner system to less than the thickness of the secondary lateral drainage layer, or less than 30 cm, whichever is less.

One of the main benefits of using a geocomposite lateral drainage layer material for the secondary collection system is that its high transmissivity is conducive to very rapid reporting of leakage to the sump. Methods of calculating the travel time of a leak through the secondary lateral drainage layer to the sump are provided in Giroud et al. (1997a) and Richardson et al. (2000), and perhaps the only reference that provides a method for calculating the head buildup on a secondary liner subjected to leakage from above is provided by Giroud et al. (1997a).

Leakage detection and management layer in a double-lined pond. The discussion later in this paper on 'Ponds' provides a background discussion of why on most important pond projects, double-liners with intervening leakage detection layers are used. Geosynthetic drainage layers are often used for the leakage detection and collection layers. Thiel and Giroud (2011) also discuss how it has been proven that the air space between two textured geomembranes has been successfully utilized to provide leakage detection and control in a number of pond cases.

Groundwater underdrain systems. The use of underdrains to provide hydraulic control of groundwater conditions is a standard practice in geotechnical foundation engineering. Advances made in geocomposite drainage materials over the past generation have certainly given us increased flexibility in addressing these design issues.

Final-cover lateral-drainage layer above the cover geomembrane. As described in the section 'Veneer System Stability Issues', the provision of a lateral drainage layer directly above a geomembrane's final cover is often essential for maintaining the vegetative soil cover stability. The methods described by Thiel and Stewart (1993) and Giroud et al. (2000a) allow geocomposite lateral drainage layers to be properly designed to preserve the cover veneer stability. Experience has shown that it is important to maintain good outflow conditions that allow the lateral drainage layers to discharge. Freezing conditions, for example, are suspected of having precipitated localized failures due to water backup in the drainage layer (Bonaparte et al., 2002, and the author's personal experience).

Final cover lateral-drainage layer below the geomembrane. As described above in the section 'Veneer System Stability Issues', providing a lateral drainage layer directly below a geomembrane's final cover is often essential for maintaining the final cover stability, as described by Thiel (1998). Geocomposite drainage layers have been found to be perfectly suited for this application, and in addition, suited for providing gas relief and enhanced landfill gas collection. The author has also found that they provide a secondary benefit of capturing side-slope leachate seeps, which can then be directed to the toe, or to a collection gallery beneath the final cover system, and then conveniently reintroduced into the LCRS at an appropriate location.

Note on installation issues. The installation of geonets and geocomposites is relatively simple, but there are a few aspects of the process that must be performed with care to avoid defeating its intended purpose. One aspect that has historically received very little attention from manufacturers and was not addressed in the earlier literature concerns geocomposite butt seams.

Butt seams require the laminated geotextile to be stripped back from the bottom of one roll and from the top of the other roll in order to provide net-to-net contact at the seam. On several jobs I have heard the installers, when asked to do this, make the 'remarkable remark': '…but we never do this'. This means that their standard practice has always been to incorrectly overlap the geocomposites with the geotextiles intact, which severely reduces their transmissivity. Only recently has this issue been correctly addressed in the literature, by Koerner & Koerner (2009). A second aspect that is often overlooked is that geocomposites have flow-directionality, with the maximum flow occurring in the machine direction. The flow capacity can be reduced between 30% and 90% in the transverse direction, depending on the type of geonet. Thus, panel placement instructions may need to be provided in the design and verified by CQA. Thirdly, the outlet condition details of geocomposites are critical for their proper long-term functioning. This issue is partly addressed by Koerner & Koerner (2009).

12. DETAILS RELATED TO PENETRATIONS AND ATTACHMENTS

Experience in the review, design, and performance of field inspections, and in the attachment of geomembrane boots and connections to structures, reveals that a wide variety of standards and approaches are employed. The execution of the details involved is very much an art of workmanship, and depends a great deal on the experience and understanding of the installer. There has historically been very little guidance in the literature regarding the fine points of specifying and implementing these critical details. The typical manufacturer's details and guidelines are not much more than concepts that have been repeated for over two decades. Thus, there is a big difference between what we assume and expect, versus what is actually constructed, in terms of the leak resistance of geomembrane penetrations and attachments to structures. Before 2009, the only substantive references on the subject were the ASTM Guide D6497, Standard Guide for Mechanical Attachment of Geomembrane to Penetrations or Structures, and Wells (1993). Thiel & DeJarnett (2009) touch upon some of the detailed and critical aspects that should be addressed when specifying and constructing geomembrane seals around penetrating pipes (referred to as "boots") and attachments to structures. In addition to specific recommendations, Thiel also provides the following general recommendations as guiding principles for the design and construction of geomembrane penetrations and boots:

- Penetrations and attachments are more susceptible to leakage than a free-field geomembrane liner. Owners and designers should always be prepared to manage leakage at these locations. Critical applications should always be designed with redundancy (double liners, double boots, leakage detection layers, etc.).

- For exposed and serviceable installations, it is prudent to have a regular inspection and maintenance program for geomembrane penetrations and attachments.

- Penetrations and attachments require a great deal of care and craftsmanship to construct. Designers and CQA personnel should give extra attention to detailing and inspecting these items. Installers should develop in-house standards to assure the quality of their own installations.

Giroud & Soderman (1995a) provide guidance for the connections between a geomembrane and rigid structures, and propose a method to determine the amount of wall batter that is required to decrease the tensions and strains in the geomembranes to an allowable level. Giroud &

Soderman (1995b) provide guidance for analyzing the mechanism of deformation of a geomembrane that is supported by a soil dike and is subject to differential settlement at its connection to a rigid structure.

It is clear that the subject of leak-resistant details is complex and cannot be taken for granted. The few technical guidance documents that are available related to these critical aspects of containment construction suggest that more development can be expected in the future in this area. Meanwhile, a lot of faith will continue to be put in the skill and craftsmanship of the installer.

13. DETAILS RELATED TO ANCHOR TRENCHES

Geomembrane anchor trenches are often a matter of convenience for general contractors and installers, and are probably commonly over-designed. For common types of anchor trenches where the liner system will be buried, not much has changed in the past 25 years. Good practices to consider when designing and specifying standard anchor trenches are:

- It may be good to design standard-type anchor trenches so that the geomembrane pulls out before tearing, so that more material stays intact in the undesirable event of slope movement.
- Keep anchor trench designs simple and flexible. Since they are generally for the convenience of the contractor, it may be good to allow for the contractor to suggest alternatives.
- The author believes that for double-liner systems, it is always best to seam the primary and secondary liners together in the anchor trench. This then rules out the possibility that liquids will enter the leakage detection system through this avenue.
- Backfilling in anchor trenches should be performed carefully so as not to damage the geosynthetics, and should always be done well and in a controlled fashion. Loose backfill has the potential to become water-logged, which can only create problems of various sorts.

Critical anchor trenches are required for exposed geomembranes (e.g., ponds and EGC's) or high-strength anchorage applications (e.g., reinforcement). For these applications, a more detailed engineering analysis may be necessary in order for such trenches to resist the tension forces that may develop in the geomembrane due to forces such as wind (see the next section on EGC's). Historically, most textbook methods for evaluating anchor trench pullout only considered the shear resistance along the planar surfaces of the anchor trench (e.g., the trench walls), and they assumed "frictionless rollers" at the corners. The most common approaches, such as those suggested by Koerner (1994) and Qian et al. (2002) were perceived as providing inadequate consideration of the pullout resistance around the corners of an anchor trench. The most significant advance in anchor trench design methodology in the past 25 years was proposed by Villard and Chareyre (2004). Based on a combination of analytical reasoning, finite element modeling, and laboratory testing, they recommended a design approach for L- and V-shaped anchor trenches that accounts for corner forces using the Euler-Eytelwein equation for belt-friction. The analytical methodology that they proposed is considered by the author to be far superior to any other methodologies that were previously proposed. Thiel (2010) presents a case study that provides some refinements to the Villard and Chareyre (2004) method and shows how their method could be used to optimize construction on large projects.

14. EXPOSED GEOMEMBRANE COVERS (EGCs)

Within the past 25 years, the concept of having a relatively long-term exposed geomembrane cover (EGC) has found a definite niche in real-world applications, and such installations that have been designed to last at least one generation are proving successful. A good summary of the rationale and justification for such covers is provided by Koerner (2012).

Design challenges in the installation of an EGC include: (a) providing adequate anchorage of the geomembrane so that it can resist the typically strong wind forces; (b) making sure that all important geomembrane welds are constructed so that they would only be stressed in a shear mode, and not a peel mode; (c) managing intense stormwater runoff from the exposed geomembrane area; (d) ballast for low-wind conditions; and (e) managing the large number of penetrations through the cover geomembrane that would cause localized stresses during wind storms. Exposed conditions also mean that the cover is susceptible to damage from animals (birds, deer, etc.), meteoric events (e.g., hail), vandalism, and fire. Even so, many successful and substantial EGCs have been constructed, some now approaching 20 years old, including some in Florida and Louisiana that have survived major hurricanes.

As mentioned above, one of the largest challenges in the design of an EGC is accounting for wind loads. At high wind velocities, all geomembranes are likely to be uplifted. In this regard, Giroud et al. (1995a) presented a landmark paper that provides a method for evaluating the tension, strain, and deformation of a geomembrane that is uplifted by the wind. Zornberg and Giroud (1997) provided refinements to the original method, and Giroud et al. (1999) provided additional discussion regarding anchorage design.

15. PONDS

The primary difference between containment liner systems involving ponds versus landfills is the intentional design of the operational conditions of relatively high heads. Leakage under high head conditions will produce more consequences than it will under low heads.

Ponds are designed and constructed for many different uses; these include architectural or decorative purposes, golf courses, recreation or sport facilities, habitats, fisheries, stormwater detention, sedimentation, water storage, chemical containment, and wastewater containment. Different types of ponds can have different design criteria related to liquid containment. Most ponds are intended to contain liquids with the desirable goal of having as little leakage or infiltration to the ground as is reasonably possible. In the extreme case of chemical and strong-wastewater ponds, significant leakage to the environment is unacceptable and may also be illegal, depending on the specific circumstances. Leakage from a pond can be undesirable for the following several reasons, according to Thiel and Giroud (2011):

- Potential contamination and pollution of soils and groundwater from the leaking fluids.
- Possible underground erosion and/or formation of solution cavities, which is one type of "geotechnical damage".

- Possible slope instability due to phreatic surface buildup below the liner, which is a second type of "geotechnical damage".

- Potential uplifting of the liner, reducing the pond capacity and exposing the liner to mechanical damage and excessive stresses due to the pressure of gas and/or liquid present under the liner and/or to the buoyancy of the liner.

- Potential loss of valuable clean water, or potential loss of a valuable solution (in the case of chemical and production ponds).

- Potential difficulty in maintaining an acceptable liquid level, which may be important in decorative ponds, water reservoirs for recreation or sport activities, reservoirs for pump-storage stations, etc.

The basic design of pond liner systems has not substantially changed in the past 25 years. Digging holes in the ground to contain liquid is not a particularly new idea, and lining these holes with synthetic liners to reduce leakage losses has been done for over 80 years. In fact, Koerner (1994) reports that the term '*pond liner*' was eventually superseded by the term 'geomembrane', which tells us how geomembranes were originally used. Single geomembrane liners constructed over a smooth, firm, relatively low-permeability subgrade, and covered with a ballast layer of at least approximately 30 to 60 cm, may provide a high degree of resistance to leakage, even when the geomembrane contains defects. The reason for this is that the ballast layer will generally prevent gas pressures from uplifting the geomembrane, and will maintain intimate contact between the geomembrane and the soil subgrade, thereby keeping the leakage rate at a very low level, even where there are defects in the geomembrane. Depending on the ballast type, the size of the geomembrane defects, and the degree of liquid head in the pond, the leakage rate from this type of pond can be estimated using empirical equations that are available in the literature. There are also other considerations that must be considered in the installation of soil ballast layers over pond geomembranes, such as soil type, placement method, pond volume impact, cost, and veneer stability.

Ponds that are important from a geotechnical, commercial, or environmental point of view are typically double-lined with intermediate leakage detection and collection layers. These types of ponds have been in use and have been regulated for at least the past 25 years. Thiel and Giroud (2011) describe in detail how deductive engineering and operational experience clearly show that any critical pond design using an exposed geomembrane primary liner requires a well-designed, well-monitored, and well-maintained leakage collection system if one expects the pond to function properly. Often, geosynthetic drainage layers are used. The key design features for ponds typically include (top-to-bottom) a properly anchored primary geomembrane sloped down to a sump, a lateral drainage layer that controls leakage by efficiently draining to an extraction sump, a secondary geomembrane, and a prepared foundation. Thiel and Giroud (2011) discuss four levels of leakage control, why it is important that the leakage collection layer be designed so that it has adequate transmissivity to control the head buildup in the leakage collection layer to a level that is less than the thickness of the leakage collection layer, and the concept of an Action Leakage Rate (ALR) for ponds.

Empty ponds, especially large ones, may present significant wind uplift considerations for exposed geomembranes. Anchorage for these liners can follow the methods discussed above for

EGCs. The author has been involved in projects in which intermediate anchor trenches were created across the bottoms of very large ponds to manage the wind uplift.

The range of geomembrane materials used in pond applications is typically much more diverse than that used for landfills. In fact, for potable-water applications, the use of polyethylene geomembranes is in the minority, as compared to the use of other materials such as reinforced polypropylene. The manufacturers and installers of flexible geomembranes typically promote their products for pond applications because their satisfactory performance in these applications has been verified for periods of up to 20 years, and also because of the much smaller number of field seams that are needed due to the use of prefabricated panels. As with any project, specific geomembrane resins must be chosen that will provide appropriate chemical compatibility, exposure and construction durability, and repairability.

Wrinkles are often endemic to pond designs in which the geomembrane is exposed. Because of the undesirable consequences of hydrostatic forces on geomembranes that ‘bridge’ or ‘trampoline’ across the corners and toes of slopes, exposed geomembranes are often installed with enough slack to prevent bridging in the coldest conditions. As a consequence, the exposed geomembranes in ponds typically exhibit different degrees and patterns of wrinkling during warm periods. Wrinkles that experience hydrostatic forces typically take the shape of flattened ‘fins’ protruding normal from the subgrade planar surface. The largest ‘fins’ will typically occur with HDPE geomembrane materials, because of that material’s relatively high thermal expansion coefficient, and its propensity to have larger wrinkles further apart than more flexible materials. The author is aware of at least two large pond projects in recent years (one reported by Peggs, 2012; the other not published) in which very strange cracking failures occurred at the tops of these fins. The hypothesized explanation for this is twofold: (1) the protruding exposed tips of these fins finally stress-cracked because they had experienced numerous cyclic stress-strain reversals, and (2) they had also potentially experienced localized loss of anti-oxidants at the surface, possibly exacerbated by being exposed and stretched passed the yield point at the tip of the fold. While these types of failures are rare in the author’s experience, and while numerous successful exposed HDPE-lined ponds have provided 20 or more years of service without such events, the fact that this has recently occurred twice on very large ponds is worth noting by any engineer who is considering the lining of large ponds.

16. SECONDARY CONTAINMENT AROUND TANKS

Diked secondary containment that is used to contain potential spills around the outside of fuel and chemical storage tanks is commonly provided using geosynthetic materials such as geomembranes. The use of single GCLs to provide this function has not seemed popular, probably because of the need to provide a minimum of 30 cm of overburden soil confinement on top of the GCL, and the propensity of shallow-buried GCLs exposed to variable meteoric conditions and wet-dry cycles to experience a degradation of their low hydraulic conductivity in these applications. In the author’s experience, geomembranes used in these applications are most often exposed and are not buried, with some exceptions. Because of the need for chemical compatibility with fuels and strong chemicals, the geomembrane materials typically used for this application are HDPE, reinforced ethylene-interpolymer-alloy (EIA), and thick bituminous geomembranes. Spray-on liner materials, such as polyurea, are typically the most expensive, but have also been used in these applications.

HDPE offers the advantages of lower cost, compatibility with HDPE pipe for welding boots and embedment strips, and good repairability. The disadvantage of HDPE is its expansion and contraction characteristics. Since an exposed secondary containment diked area is similar to an empty pond, as described above, numerous wrinkles are endemic to these installations. These wrinkles trap stormwater and create extra opportunities for wind damage and stress concentrations.

The advantage of reinforced EIA material is its greatly reduced expansion and contraction that is the result of its reinforcement. Its disadvantage, in the author's experience, is that the large number of hand-welds that must be made using hot air guns and rollers seems to result in numerous small adhesion failures over time; these are essentially sites where leakage takes place.

The advantage of thick, roll-out bituminous geomembranes is that they seem very durable for foot and light equipment traffic and for maintenance activities, and are highly resistant to wind uplift. There may be some disadvantages to these materials if exposed to chronic fuel exposure, such as drips that sometimes occur at these installations.

Numerous attachments and penetrations are endemic to these installations, which require a lot of attention to detail (see discussion on 'Penetrations and Attachments').

17. CONSTRUCTION, CQA, AND SPECIFICATIONS WITH REGARD TO DESIGN CONSIDERATIONS

Although there will be other papers at this conference that discuss the experience of the past 25 years related to specifications, installation, and the seaming of geosynthetics, the author feels obliged to mention a few items as they relate to design.

<u>CQA</u>. The author would like to acknowledge the continued need now, as much as ever, for construction quality assurance (CQA). The author's extensive and ongoing experience in performing designs and CQA in this industry has shown that there is no less need for CQA today than there was 25 years ago.

<u>Conformance Testing.</u> Related to CQA, the author believes that there is a continued need for conformance testing of geosynthetic products, but that this could perhaps be done in a more intelligent manner now than it was 25 years ago. There are a number of index tests that continue to be performed with relatively high frequency that were established 25 years ago (e.g., 'every 100,000 sq ft'). Perhaps with GAI-LAP accreditation and extensive MQC reporting, there could be relaxation of many of the industry standard requirements for certain conformance tests, and more acceptance of MQC results and manufacturer certifications. On the other hand, there are certain index and performance tests that may be more critical for design performance criteria that still require frequent testing. Depending on the project-specific requirements, these tests could include items such as peel strength for various materials, transmissivity testing, interface shear strength, oxidative induction times, and other critical properties.

<u>ELL</u>. This conference would not be complete without mentioning the progress that has been made in the past 25 years in the use of electrical geophysical methods to locate defects in installed geomembranes. This activity goes by various names, including electric leak location (ELL), which is the acronym used in this paper. Development of the ELL method began in 1980 at the Southwest Research Institute in San Antonio, Texas under cooperative contracts with the

U.S. Environmental Protection Agency. Commercial surveys conducted on water-covered geomembranes began around 1985, and on soil-covered geomembranes around 1988, right at the beginning of the 25-year period we are discussing (Laine & Darilek, 1993). Since then, the capabilities of this method and its relatively low cost have propelled it to the forefront of the geosynthetics world as the most state-of-the-art quality-control/quality-assurance method for installed geomembranes. Required by an increasing number of regulatory boards for new landfill expansions, it is now also being applied worldwide to heap leach facilities in the mining industry.

The basic method of ELL is to connect an electrical power supply to electrodes above and below the liner, and to then detect areas where there is localized electrical current flow through leaks in the otherwise insulating liner. ELL methods are standardized by ASTM D7002; Standard Practice for Leak Location on Exposed Geomembrane Using the Water Puddle System, and ASTM D7007; Standard Practices for Electrical Methods for Locating Leaks in Geomembranes Covered with Water or Earth Materials. The ELL method has the ability to detect defects that would not ordinarily be detected using standard CQA methods, especially those caused by placement of the initial soil layers over the top of the liner system. It has been known for some time (see Nosko et al., 1996) that most significant geomembrane damage is caused by construction machinery during the placement of earth materials on the geomembrane. Smith et al. (2007) describe specific measures that can be included in the design and specifications to enhance the electrical leak- location signal and improve the quality of the survey. For over a decade, these techniques have been part of the author's standard design practice in bottom liner systems for landfills and heap leach pads.

Experience. A general word of advice, based on our last 25 years of experience, is to recommend that an experienced, responsible party who is intimately familiar with construction procedures be involved in decisions related to design details, and to any issues related to slope stability (this could be different professionals for different parts of the design).

Along the vein of experience, a lot can be learned by studying failures that have occurred in the industry over the past 25 years. Many of the references cited in this paper contain discussions of failures with observations of useful lessons that were learned. Perhaps the largest compendium of containment-system failures of various sorts is contained in Appendix F of Bonaparte et al. (2002).

18. MOVING INTO THE FUTURE

The discussions in the preceding sections suggest that while we have come a long way, significant questions remain to be answered, and there are areas that show clear room for improvement. While regulations assume a certain static immobility once they've been established, the understandings we gain through experience never stop growing. It is only natural and desirable, then, for new understandings to eventually render existing minimum compliance regulations inadequate, and to inspire designs that are more effective and efficient.

Key items discussed in the preceding sections that could contribute to more effective designs include:

- An emphasis on high-performance, very highly stress-crack-resistant resins for geomembranes (and other geosynthetics) with more robust AO packages.
- More robust leachate collection (lateral drainage) layers.
- Greater deliberation given to long-term allowable strain in geomembranes, with greater attention given to puncture protection and wrinkle management.
- Further investigation into the long-term durability of shear strength interfaces and appropriate acceptance criteria for slope stability for the full range of bottom liners, covers, static conditions, and seismic conditions.
- More requirements with regard to monitoring the impacts of operations on liquid levels and temperatures, especially in light of the bioreactor concept.
- More consideration for shortening the post-closure care period by weighing the benefits of enhanced waste degradation and waste stabilization using anaerobic versus aerobic methods.
- More consideration of design redundancy in specific design elements (e.g., extra pipes, closer spacing) and for the design as a whole (e.g., double liner systems).
- More attention to robust details.

One might well ask: "If the industry's performance has been so stellar in the past 25 years, why consider changing a winning game? Especially since any of the proposed so-called-improvements might lead to increased cost of construction or operations, why would we want to change?" The answers to these questions can be found in the learning process we are all engaged in. Part of this learning is the realization that our excellent track record over the past 25 years may only represent a relatively small fraction of the life legacy presented by constructed landfills. For this reason, we should continue to query the durability and aging characteristics of our designs in light of the expected duration of the operational life and post-closure care period, especially for the very large, regional facilities that are now being constructed. The recommendations in the NRC (2007) report mainly focus on increased funding and requirements for the monitoring of existing facilities, with the aim of assessing the long-term performance of engineered systems.

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