Chapter 7
Lessons Learned

7.1 Introduction

As discussed in Section 1.6.1 of this guidance document, there have been a number of documented cases where cover systems at waste management sites have not performed as intended. The primary factors contributing to the cover system problems in most cases were inadequate design and construction. Many of these problems occurred during, or shortly after, construction. Several, however, did not occur until one or more years after the completion of construction. The costs of remedying cover system problems can be significant, especially if the problems involve slope instability, or if they impact maintenance and can recur (e.g., excessive erosion). Daniel and Gross (1996) summarized the mechanisms that can adversely affect the performance of each component of a cover system. These mechanisms factors are presented in Table 7-1.

Table 7-1. Mechanisms that can adversely affect cover system performance (modified from Daniel and Gross, 1996).

Layer	Factor
Surface Layer	Insufficient or excessive slope Erosion by water and/or air Slope instability Insufficient nutrients or inadequate soil texture to support vegetation Inadequate soil thickness and thus water storage capacity to maintain adequate vegetation Undesirable vegetative species
Protection Layer	Erosion by water Slope instability Accidental human intrusion Intrusion by burrowing animals Root penetration Inadequate soil texture to support vegetation
Drainage Layer	Excessive clogging Insufficient flow rate capacity Insufficient number or flow rate capacity of outlets Freeze effects Slope instability
Barrier	Cracking due to wet-dry effects, freeze-thaw effects, differential settlements, seismic motions Deep root penetration Insufficient resistance to gas flow Slope instability Creep of all materials (CCL, GCL, GM, asphalt)
Gas Collection Layer	Insufficient coverage over waste Insufficient flow rate capacity
Foundation Layer	Insufficient strength

The purpose of the remainder of this section is to share recent information on experiences and lessons learned related to the design and construction of cover systems in a variety of situations. These experiences and lessons learned have been organized by the following subject areas:

- soil barriers;
- GM barriers;
- slope stability;
- waste settlement;
- stormwater management and erosion control;
- gas pressures; and
- miscellaneous.

Consistent with the discussion in Section 1.6.1 of this document, EPA believes that improvement can, and should, be made in the design and construction of cover systems. The information presented in this chapter is intended to alert engineers to past problems in the design and construction of these systems. By application of the lessons learned from this chapter, future design and construction can be improved and potential problems can be prevented.

7.2 Soil Barriers

Experiences and lessons learned with respect to the hydraulic performance of soil barriers in cover systems have focused primarily on the use of CCLs in this application. Bonaparte et al. (2002) discussed available case studies on CCLs and GCLs, from which the majority of the following information has been extracted.

7.2.1 Test Plots in Omega Hills, Wisconsin

One of the first detailed field studies on the performance of CCLs in landfill cover systems was described by Montgomery and Parsons (1989, 1990). Three large test plots with different cover system designs were constructed on top of the closed Omega Hills landfill and monitored for four years. The purpose of the field study was to compare the performance of the different cover systems. The landfill had accepted MSW and is located approximately 30 km northwest of Milwaukee, Wisconsin.

The cross sections of the three test plots are shown in Figure 7-1. Test plot 1, consisting of a 0.15-m thick topsoil layer overlying a 1.2-m thick CCL barrier, was representative of the existing cover system on part of the landfill. Test plot 2 involved the same thickness of CCL, but a thicker (i.e., 0.45-m thick) topsoil layer intended to promote better vegetative growth and thereby enhanced ET. Test plot 3 incorporated a layer of coarse-grained soil (sand) interbedded between two CCLs. The concept for the third plot was to take advantage of the capillary barrier effect (see Section 1.1.2), with the sand layer promoting retention of water in the upper CCL and enhanced ET. All test plots were constructed on the landfill's 3H:1V sideslopes. The CCL material was classified as CL according to the Unified Soil Classification System (USCS) and had a high silt content. The soil was placed and compacted in 0.15-m thick lifts to a hydraulic conductivity no greater than 1×10^{-9} m/s, based on laboratory hydraulic conductivity tests on

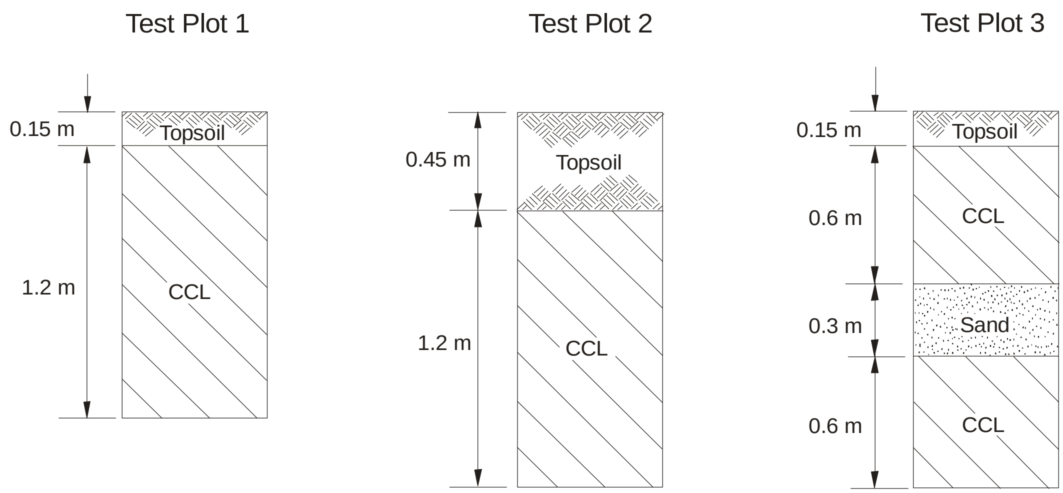

Figure 7-1. Cross Sections of Cover System Test Plots at a Landfill in Omega Hills, Wisconsin (modified from Montgomery and Parsons, 1989). Test Plot 1 is Representative of the Existing Landfill Cover System.

"undisturbed" small-diameter samples of the compacted soil. The sand in test plot 3 was a clean, washed, medium sand. The topsoil consisted of uncompacted clay loam to silty clay loam and was seeded with a mixture of grasses.

The test plots contained two principal data collection systems. The first system was a collection lysimeter installed beneath the test plot to collect water that percolated through the cover soils and allow quantification of the rate of percolation. The lysimeter consisted of, from top to bottom, a GT filter, a GC drainage layer, and a GM. The second data collection system was designed to collect and measure surface runoff.

The test plots were constructed from September 1985 to July 1986, and data collection and analysis began in August 1986. Measurements were obtained of precipitation, runoff, percolation, and other parameters such as temperature. Soil moisture content was monitored using neutron probes, and, until September 1988, soil matric potential was monitored using tensiometers.

The weather during the 12-month period from September 1986 through August 1987 was near normal. The period of September 1987 through August 1988 was dominated by a drought, which occurred during May through August. These months were characterized by substantially below average rainfall and temperatures that averaged 6 °C above normal. The drought reduced the cover vegetation to a dry, dormant state and caused cracking of the cover soils. The third year of data collection (September 1988 to August 1989) saw a return to normal conditions and a reduction in surface cracking. The nine-month period from September 1989 through April 1990 included a dry fall, a mild winter, and a spring with normal precipitation, but erratic temperature fluctuations. At the end of this monitoring period, cover vegetation was vigorous and included a number of plant species not in the original seed mix.

A summary of data through April 1990 is presented in Table 7-2. The key parameter is the quantity of percolation, i.e., flow rate of water into the lysimeter. In test plots 1 and 2, the percolation during the first year was 2 and 7 mm/year (6×10^{-11} to 2×10^{-10} m/s), respectively. However, by the third year, these values had increased to 56 and 98 mm/year (2×10^{-9} and 3×10^{-9} m/s), respectively. For test plot 3, which was designed with the intention of maintaining moisture in the upper CCL, the percolation rate remained more consistent and was found to range from 22 to 41 mm/year (7×10^{-10} to 1.3×10^{-9} m/s) during the first three years. In September 1988, at the end of the third year, 2-m deep test pits were excavated in each test plot, outside the area of the lysimeters. Examination of the test pits revealed that the CCLs in the test plots were in a similar condition:

- the upper 0.20 to 0.25 m of the CCLs were weathered and blocky;
- cracks 6 to 12 mm wide extended about 0.9 to 1 m into the CCLs in test plots 1 and 2 and through the entire thickness of the uppermost CCL in test plot 3;
- the base of the CCLs in test plots 1 and 2 appeared to be undamaged;
- roots penetrated 0.20 to 0.25 m into the CCLs in a continuous manner, and some roots extended as deep as 0.75 m into cracks in the CCLs; and
- the moisture contents in the upper portion of the CCLs were near the shrinkage limit.

The drought conditions in the second year of the study period apparently caused desiccation of the CCL in test plots 1 and 2, which led to a significantly increased CCL hydraulic conductivity in subsequent years. Although the CCLs in these tests plots may have initially had a hydraulic conductivity of 1×10^{-9} m/s or less, the desiccation damage caused the CCLs to no longer have this low level of hydraulic conductivity.

Table 7-2. Summary of performance-related information for field test plots at Omega Hills Landfill (data from Montgomery and Parsons, 1990).

Test Plot Designation	Year	Precipitation (mm)	Runoff (mm)	Percolation (mm)
1	1986-87	896	180	2
	1987-88	578	38	5
	1988-89	823	56	56
	1989-90	350	44	33
2	1986-87	896	109	7
	1987-88	579	38	30
	1988-89	823	51	98
	1989-90	350	22	31
3	1986-87	896	97	40
	1987-88	579	38	22
	1988-89	823	66	41
	1989-90	350	23	16

In May 1990, a second test pit was excavated in test plot 1. No major cracks were observed in the CCL, in contrast to the pronounced cracking of the upper portion of the CCL observed in the September 1988 test pits. The CCL appeared uniformly moist, probably as a result of spring precipitation. Roots did not appear to be deeper or more dense than observed in the earlier test pits. The base of the CCL still appeared to be homogeneous, moist, and intact. It is noteworthy that while the physical condition of the CCL in test plot 1 appeared to have improved, percolation through the CCL in 1990 remained at a high level.

For test plot 3, cracking of the uppermost CCL allowed significant amounts of water to enter the sand drainage layer. Discharge of water from the sand layer was found to occur within hours of the start of precipitation events, suggesting rapid transmission of water through the upper CCL due to preferential flow through cracks. Moisture in the sand drainage layer probably helped to protect the underlying CCL from damage. The capillary barrier in test plot 3 did not function as well as anticipated. It was expected that the sand drainage layer would help the overlying CCL retain moisture, but the uppermost CCL quickly cracked.

As of April 1990, percolation through test plots 1 and 2 was approximately 9% of precipitation, and percolation through test plot 3 was approximately 4.6% of precipitation.

The principal lessons learned from the Omega Hills study are that in a fairly short period of time (3 years), CCLs overlain by only 0.15 to 0.45 m of topsoil are subject to desiccation, cracking, and increases in hydraulic conductivity. The CCLs were incapable of "surviving" under these conditions with a hydraulic conductivity of 1 x 10^{-9} m/s or less.

7.2.2 Test Plots in Kettleman City, California

Corser and Cranston (1991) and Corser et al. (1992) described three test plots constructed at a landfill in Kettleman City, California. Cross sections of the test plots are shown in Figure 7-2. Test plot 1 consisted of a 0.9-m thick CCL overlain by an exposed 1.5-mm thick HDPE GM. Test plot 2 consisted of the same profile as test plot 1, except that 0.6 m of topsoil covered the GM. For test plot 3, 0.45 m of topsoil covered the CCL and no GM was present. A portion of the test plots was flat, and another sloped at 3H:1V. The test plots were constructed to study the factors that influence desiccation of CCLs used in cover systems.

The CCL material was a high plasticity clay that the site owner intended to use in cover system construction for approximately 30 ha of the landfill. The clay had an average liquid limit of 66% and plasticity index of 48%. Instrumentation for the test plots consisted of thermistors to monitor temperature in the topsoil and CCL and tensiometers to measure soil water potential. Corser and Cranston (1991) summarized the first six months of data collection. At the end of the six-month period, the surfaces of the CCLs were exposed over an area of 1.5 m x 1.5 m to observe and document cracking patterns.

Test plot 1 did not represent a cover system design but, instead, an exposed HDPE GM/CCL composite liner during the construction or operations phase. The clay exhibited some drying and cracking in areas where the GM was not in contact with the CCL. In other areas, where the GM was in contact with the CCL, the moisture content of the CCL at the surface had increased. It appears that the high temperature of the exposed HDPE GM caused heating and drying of the underlying CCL. In some areas (e.g., around wrinkles in the GM), moisture could migrate away via vapor transport. In other areas, the moisture could condense during cooler periods, causing moistening of the soil. In any case, there clearly was desiccation of the CCL beneath some portions of the exposed GM.

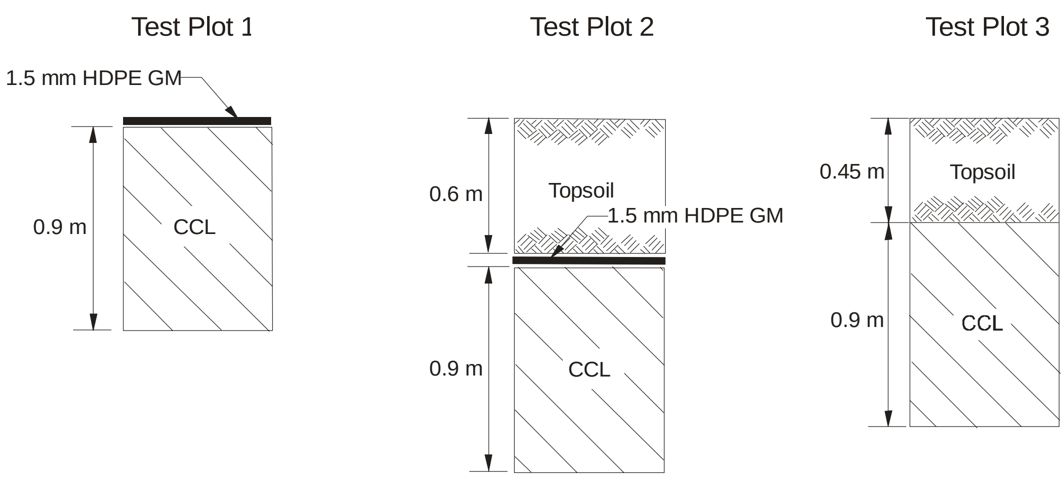

Figure 7-2. Cross Sections of Cover System Test Plots at a Landfill in Kettleman City, California (modified from Corser and Cranston, 1991).

Test plot 3 did not perform well during the summer season. The CCL dried, and cracking was observed at its surface. In contrast, test plot 2 performed well. There was no evidence of drying or cracking of the CCL.

Although the test plots were observed for only six months, significant deterioration of the CCLs was observed in test plots 1 and 3. Only test plot 2, in which the CCL was covered with a GM and 0.6 m of topsoil, performed well. The observations from Kettleman City are consistent with those of Omega Hills and suggest that perhaps the only practical way to protect a CCL from desiccation damage in typical cover system applications is to cover it with a GM overlain by a sufficiently thick layer of soil.

7.2.3 Cover Systems in Maine

The Maine Bureau of Remediation and Waste Management (1997) reported the results of laboratory and field hydraulic conductivity measurements for four CCL barriers in actual MSW landfill cover system applications. The laboratory tests were conducted on "undisturbed" small-diameter samples collected from the constructed CCLs. It appears that all four cover systems were installed using methods of construction and CQA practices that are representative of landfill industry practices presently used in the U.S.

Cumberland Site. The Cumberland MSW landfill, a 2 ha facility, was closed in 1992 with a cover system consisting of a 0.15-m thick vegetated topsoil layer underlain by a 0.45-m thick silty clay CCL. Underlying the CCL are sand-filled trenches that serve to collect and convey landfill gas. Laboratory hydraulic conductivity tests were performed on CCL samples during construction and in a post-construction investigation program conducted in 1994. A sealed double-ring infiltrometer (SDRI) test was also performed in 1994.

At the time of construction, the average CCL hydraulic conductivity measured in the laboratory was 5 x 10^{-10} m/s. In the 1994 investigation, the laboratory-measured hydraulic conductivity had increased to 1 to 2 x 10^{-9} m/s. The field hydraulic conductivity, measured with the SDRI in 1994, was 6 x 10^{-8} m/s. It is not certain whether the CCL originally had a field hydraulic conductivity greater than 1 x 10^{-9} m/s since field testing was not performed at the time of construction.

Vassalboro Site. The Vassalboro MSW landfill occupies 11.6 ha and was closed in 1990 with a cover system consisting of, from top to bottom: a 0.15-m thick sludge-amended topsoil layer; a 0.45-m thick glacial till CCL; and a gas collection layer. Laboratory hydraulic conductivity tests were performed at the time of construction, and again in 1994. An SDRI test was also performed in 1994.

The average hydraulic conductivity of the CCL measured in the laboratory at the time of construction was 2 x 10^{-9} m/s. In 1994, the laboratory-measured hydraulic conductivity values ranged from 9 x 10^{-9} to 5 x 10^{-8} m/s and the field-measured hydraulic conductivity was 2 x 10^{-8} m/s. It appears that the hydraulic conductivity of the CCL increased by about an order of magnitude from 1990 to 1994.

Yarmouth Site. The Yarmouth MSW landfill, a 2.5 ha facility, was closed in 1990 with a cover system consisting of, from top to bottom: a 0.15-m thick sludge-amended topsoil layer; a 0.45-m thick silty clay CCL; and a gas collection layer. Laboratory hydraulic conductivity tests were performed at the time of construction, and again in 1994 and 1996. An SDRI test was also performed in 1994 and 1996.

Laboratory hydraulic conductivity tests conducted in 1990 indicated an average CCL hydraulic conductivity of 8 x 10^{-10} m/s. In a 1994 investigation, the average measured laboratory hydraulic conductivity was 3 x 10^{-9} m/s, and, in 1996, the laboratory-measured hydraulic conductivity was in the range of 2 x 10^{-8} to 2 x 10^{-7} m/s, or about 20 to 100 times larger than in 1990. The field-measured hydraulic conductivity was 2 x 10^{-9} m/s in 1994 and 2 x 10^{-8} m/s in 1996. There is a clear trend of increasing hydraulic conductivity over time, with the magnitude of increase being one to two orders of magnitude over the six-year study period.

Waldoboro Site. The Waldoboro MSW landfill encompasses 1.6 ha and was closed in 1991 with a cover system consisting of, from top to bottom: a 0.15-m thick sludge-amended topsoil layer; a 0.45-m thick silty clay CCL; and a gas collection layer. Laboratory hydraulic conductivity tests were performed at the time of construction, and again in 1993 and 1996. An SDRI test was also performed in 1993 and 1996.

Laboratory hydraulic conductivity tests indicated that the CCL hydraulic conductivity increased over time from an initial average value of about 5 x 10^{-10} m/s (1991) to 1 x 10^{-8} m/s (1993) and then to 3 x 10^{-8} m/s (1996). The field hydraulic conductivities were 1 x 10^{-8} m/s (1993) and

4 x 10^{-8} m/s (1996). Thus, the data indicates that the hydraulic conductivity increased by about two orders of magnitude over a five-year period.

Discussion. The observations from these four cover system case studies are consistent with those of the other sites mentioned previously in this chapter. All of the available field performance data indicate that a CCL barrier overlain by a relatively thin layer of topsoil or protection soil (0.15 to 0.45 m thick), and without a GM above the CCL, cannot maintain a hydraulic conductivity of 1 x 10^{-9} m/s or less. From analysis of the condition of the four CCL barriers at these sites, it appears that desiccation was the most significant factor leading to an increase in field hydraulic conductivity. Freeze/thaw may also have contributed to the observed degradation in CCL performance. Penetration of plant roots into the CCL was also observed.

7.2.4 Test Plots in Live Oak, Georgia and Wenatchee, Washington

Lane et al. (1992), Khire (1995), and Khire et al. (1997, 1999) reported on field water balance studies for three 30 m x 30 m cover system test plots at two landfills, one near Atlanta, Georgia ("Live Oak") and the other near East Wenatchee, Washington ("Wenatchee"). The sites were selected to represent humid and semi-arid climates, respectively. The Live Oak test plot has a cover system with a 0.9-m thick CCL overlain by a 0.15-m thick silty topsoil layer. In Wenatchee, one test plot has the same cover system as at the Live Oak site except that the CCL is 0.6 m thick, and the other test plot models a capillary barrier consisting of a 0.75-m thick layer of uniformly-graded medium sand overlain by a 0.15-m thick silt topsoil layer. Climate, runoff, percolation, and soil moisture data collected between 1992 and 1995 were reported by Khire (1995) and Khire et al. (1997, 1999), and data collection is still ongoing as of 2001. Details of the water balance analyses for these test plots are provided in Chapter 4 of this guidance document. Importantly, the results of these field studies show nearly 250 mm of percolation through the Live Oak test plot in a period of 2½ years. Percolation through the CCL barrier test plot at the Wenatchee site over a roughly similar period was much less than at Live Oak, due to the more arid site conditions, but still significant (more than 30 mm). Percolation through the capillary barrier test plot was low, only about 5 mm. The conclusions for these data are consistent with those presented previously in this chapter. Percolation rates through inadequately-protected CCL barriers are relatively high. The limited results for the capillary barrier at the Wenatchee site are encouraging.

7.2.5 Test Plots in Hamburg, Germany

Melchior et al. (1994) and Melchior (1997a,b) described what may be the most extensive test plot program to date involving CCLs for cover systems. Test plots with four different cover system cross sections, shown in Figure 7-3, were constructed over a MSW landfill in Hamburg, Germany. The test plots with CCLs were constructed in 1987, and the test plots with GCLs were constructed in 1995. Each test plot is 10 m wide and 50 m long and is located on the relatively flat (i.e., 4% slope) top deck or on the 5H:1V sideslopes of the landfill. Climate, lateral drainage, runoff, percolation, soil moisture content, and soil water potential data are being collected.

The CCL material at the Hamburg site consisted of a glacial till comprising 17% clay, 26% silt, 52% sand, and 5% gravel. The principal clay minerals in the clay-sized fraction were (in decreasing abundance) illite, smectite, and kaolinite. The soil liquid limit was 20%, and the

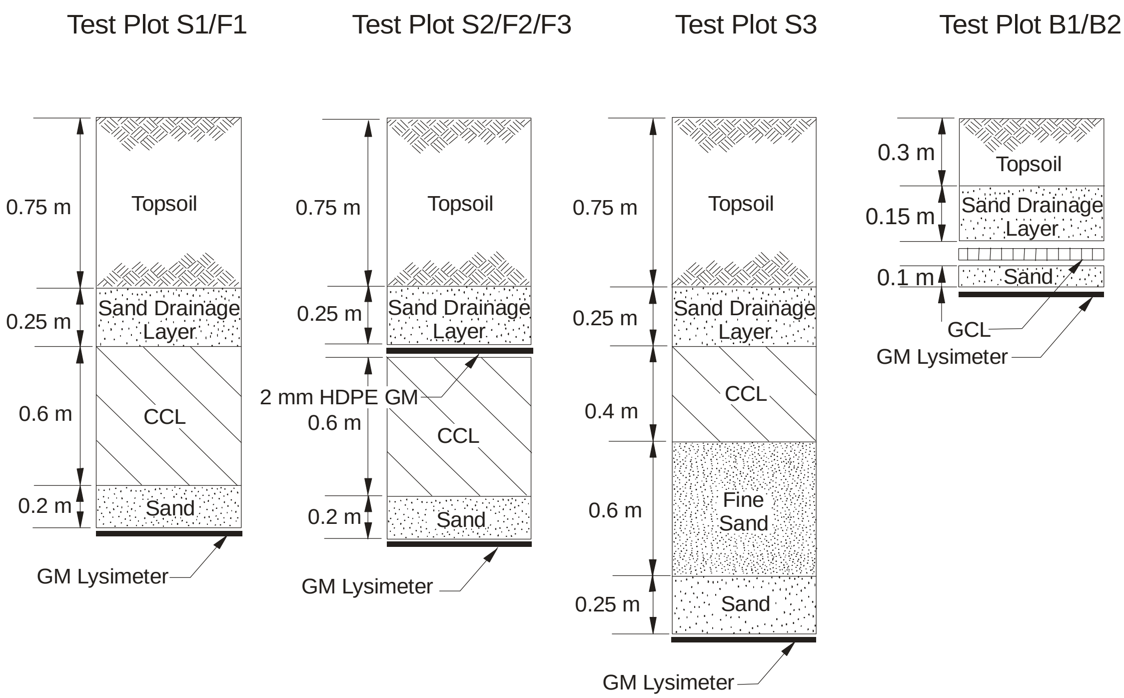

Figure 7-3. Cross Sections of Cover System Test Plots at a Landfill in Hamburg, Germany (modified from Melchior et al., 1994; Melchior, 1997a).

plasticity index was 9%. The soil was placed in 0.20-m thick compacted lifts at two percentage points wet of the standard Proctor optimum moisture content and to an average degree of compaction of 96% of the standard Proctor maximum dry density. The geometric mean hydraulic conductivity of the CCLs was 2.4×10^{-10} m/s, based on laboratory hydraulic conductivity tests on "undisturbed" small-diameter samples of the compacted soil. The CCL material at the Hamburg site was significantly different from that at the Omega Hills and Kettleman City sites. At Omega Hills, the CCL material was a low-plasticity clay (CL) containing a large amount of silt, which can make the CCL vulnerable to shrinkage cracking. The Kettleman City CCL material was a high-plasticity clay (CH). At Hamburg, the CCL material contained more than 50% sand- and gravel-sized particles and would therefore be classified as a clayey sand (SC). Clayey sands tend to be less vulnerable to shrinkage cracking than clays (especially highly plastic clays) that contain relatively few coarse-grained particles.

Percolation rates through the CCLs from 1988 to 1995 for the test plots with a 4% slope (test plots F1, F2, and F3) are summarized in Table 7-3. Percolation rates through the CCLs (i.e., drainage from the underlying lysimeters) from 1988 to 1995 for the test plots with a 20% slope (test plots S1, S2, and S3) are summarized in Table 7-4. Also shown in Tables 7-3 and 7-4 are the lateral flow rates from the sand drainage layers that overlie the CCLs. The last column in the tables expresses percolation through the CCLs as a percentage of the lateral flow from the sand drainage layers.

Table 7-3. Summary of field performance data for Hamburg, Germany test plots containing CCLs and at 4% slope (data from Melchior, 1997a).

Test Plot Designation	Year	Lateral Drainage (mm)	Percolation (mm)	Percolation/ Drainage (%)
F1	1988	368	7	2
	1989	183	8	4
	1990	286	18	6
	1991	187	9	5
	1992	226	103	46
	1993	253	174	69
	1994	247	166	67
	1995	156	164	105
F2	1988	293	3.5	1
	1989	156	0.6	0.4
	1990	263	0.4	0.1
	1991	171	0.5	0.3
	1992	313	0.8	0.3
	1993	412	1.3	0.3
	1994	409	1.8	0.4
	1995	310	1.7	0.5
F3	1988	367	4.1	1.1
	1989	155	1.4	0.9
	1990	262	2.6	1.0
F3	1991	168	2.0	1.2
	1992	326	3.5	1.1
	1993	481	5.0	1.0
	1994	431	5.2	1.2
	1995	328	5.2	1.6

As can be observed from inspection of the data in Tables 7-3 and 7-4, test plots F1, S1 and S3, which did not have a GM overlying the CCL, underwent large increases in percolation rate within three to four years after installation. In particular, the summer of 1992 was very dry in Hamburg, and the subsequent fall season was very wet. By 1992, actual percolation rates exceeded 100 mm/year in two of the three CCL test plots. The third CCL test plot exceeded this percolation value by 1993. Excavations made in 1993 confirmed that the CCLs in these test plots were cracked. Barely visible fissures were observed between soil aggregates (around 50 mm in diameter). By 1995, plant roots were observed to have extended more than 1 m into the cover system, reaching the upper parts of the CCLs. In summary, the performance of the test plots containing a CCL without GM protection has been poor. The apparent problem is gradual deterioration of the CCLs caused by desiccation during a particularly dry summer. Detailed results for test plot S1 are presented in Figure 7-4 for illustration purposes.

Percolation rates through test plots S2, F2, and F3, which contain a CCL overlain by a GM, have been very low. Test plots F2 and S3, which incorporate a continuously-welded HDPE GM, had average measured percolation rates of 1.3 mm/year, while test plot F3, which has an overlapped (not welded) GM, exhibited an average measured percolation rate of 3.6 mm/year. Melchior (1997a) indicated that the measured percolation is primarily due to thermally-driven unsaturated flow of pore water in the CCL, not to leakage through the GM.

Table 7-4. Summary of field performance data for Hamburg, Germany test plots containing CCLs and at 20% slope (data from Melchior, 1997a).

Test Plot Designation	Year	Lateral Drainage (mm)	Percolation (mm)	Percolation/ Drainage (%)
S1	1988	386	1.9	0.5
	1989	247	3.1	1.2
	1990	318	13	4
	1991	177	13	7
	1992	289	48	17
	1993	343	136	40
	1994	344	150	44
	1995	229	150	66
S2	1988	355	0.6	0.2
	1989	237	0.3	0.1
	1990	321	0.5	0.2
	1991	192	0.7	0.4
	1992	330	1.0	0.3
	1993	390	1.7	0.4
	1994	389	3.0	0.8
	1995	297	2.8	0.9
S3	1988	396	84	2
	1989	234	14	6
	1990	319	31	10
	1991	200	3	16
	1992	279	117	42
	1993	263	171	65
	1994	248	184	74
	1995	151	201	133

The two test plots (B1 and B2) containing GT-encased GCLs were constructed in early 1995 with an 8% slope. The GCLs were covered with a 0.15-m thick sand drainage layer and a 0.3-m thick topsoil layer. Melchior (1997a) reported that both GCL test plots performed very well through the first winter after installation. However, after a dry summer (1995), significant percolation occurred through both GCLs. Through four months in the fall of 1995, percolation through the two test plots was 45 and 63 mm. Melchior reported that during the 1995/1996 winter, the GCLs did not rehydrate and swell enough to completely heal the preferential flow paths caused by the previous summer's desiccation. In part, this may be due to calcium for sodium ion exchange within the bentonite.

With respect to the CCL test plots, the results from the Hamburg test site are consistent with those from the Omega Hills and Kettleman City test sites, even though the CCL materials for the three sites were different. The up to 0.75 m of topsoil placed over the CCLs at the different sites was not sufficient to maintain the low hydraulic conductivity of the CCLs. It appears that a CCL placed in a cover system without a GM and a sufficient thickness of soil covering the GM is likely to fail to maintain a hydraulic conductivity $\leq 1 \times 10^{-9}$ m/s, at least for the considered sites and surface/protection soil thicknesses. It is emphasized that from a practical perspective, if the

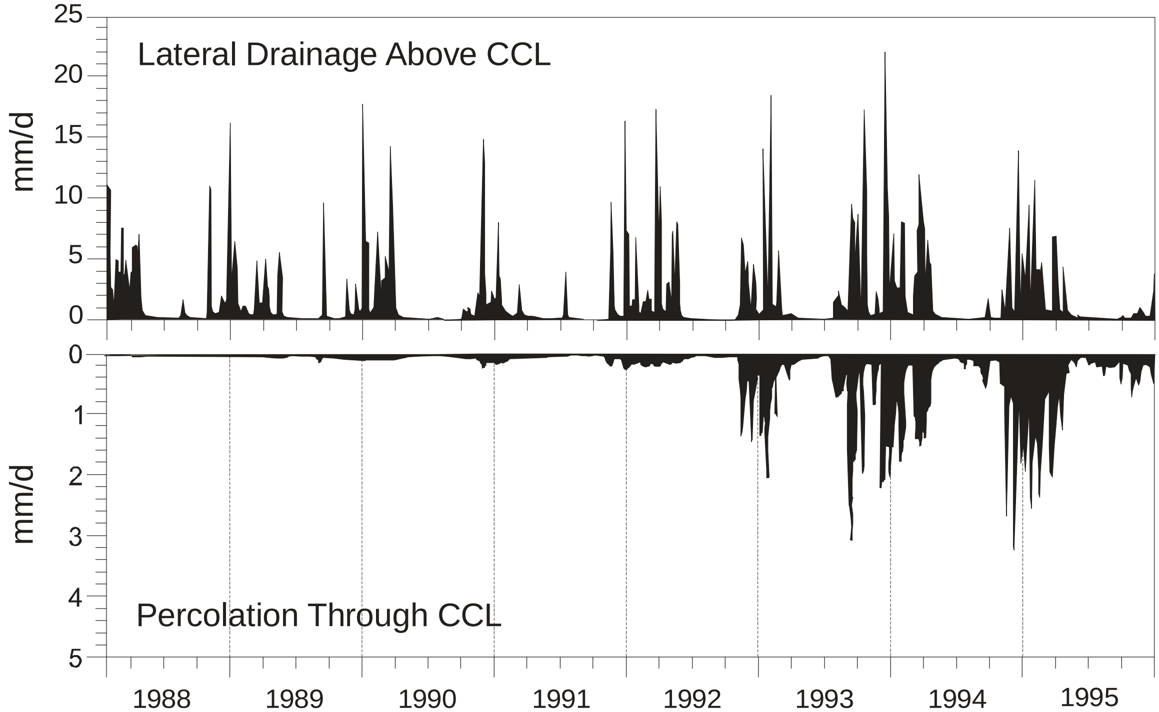

Figure 7-4. Summary of Field Data for Test Plot S1 with a CCL Barrier at a Landfill In Hamburg, Germany (modified from Melchior, 1997a).

CCL is to have a chance of maintaining a low hydraulic conductivity for an extended period, the CCL must be protected with both a GM and a sufficiently thick layer of cover soil above the GM. Furthermore, if a GCL is used in lieu of a CCL, the GCL must be chemically-compatible with adjacent soils.

7.2.6 Test Plots in Albuquerque, New Mexico

Dwyer (1997, 1998, 2001) described the U.S. Department of Energy (DOE) funded Alternative Landfill Cover Demonstration (ALCD) project, which involved the construction and monitoring of six test plots with different cover system configurations at the Kirtland Air Force Base in Albuquerque, New Mexico. The six cover system types being evaluated are shown in Figure 7-5. To provide good vegetation coverage during the growing season, the plots were seeded with a mixture of warm season and cold season native grasses.

The test plots were constructed in 1995 and 1996. Each test plot is 13 m wide by 100 m long, crowned in the middle, and sloped at 5% in both length directions from the crown. For each test plot, one slope ("western slope") is monitored under the ambient conditions existing at the site. The other slope ("eastern slope") has a sprinkler system to provide a hydrologic stress to the cover systems. Continuous water balance and meteorological data are being collected for the test plots. The plots are heavily instrumented to quantify measurable water balance variables (precipitation, surface runoff, lateral drainage, percolation, and soil moisture changes.) Instrumentation includes collection lysimeters to monitor percolation and time domain

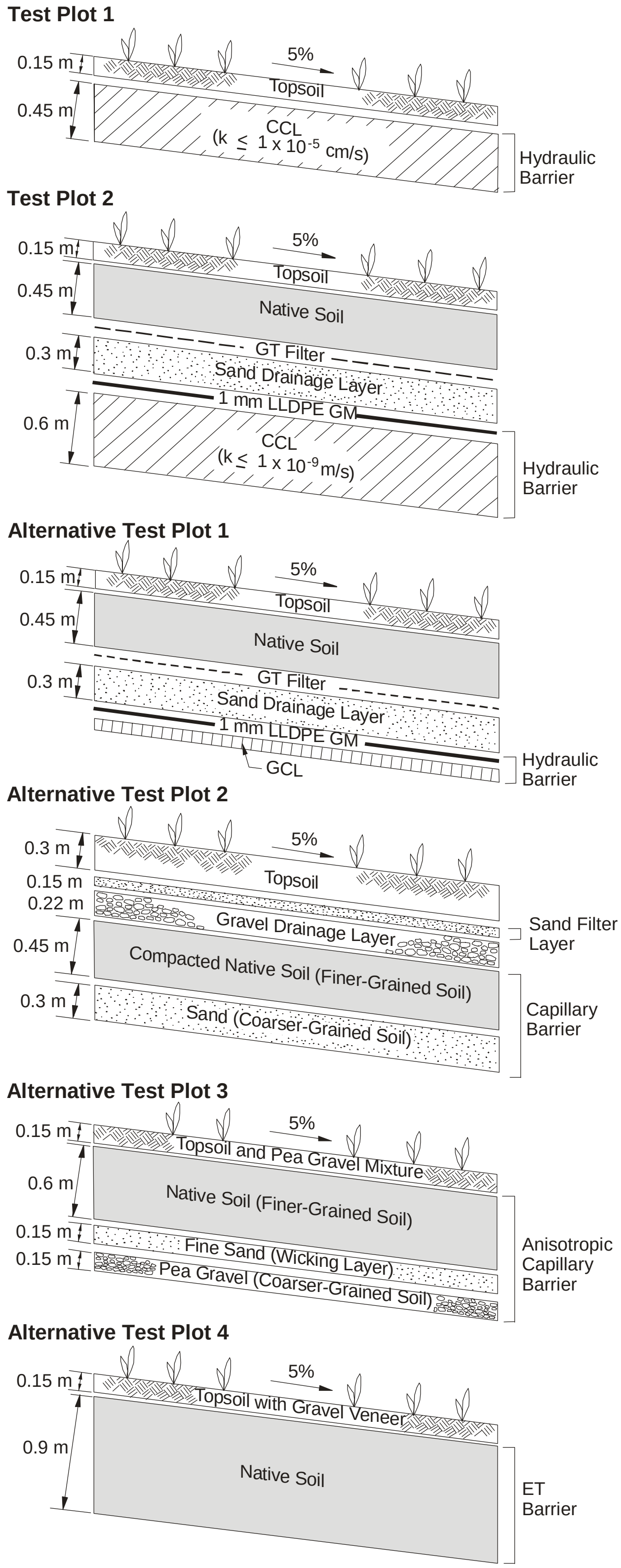

Figure 7-5. Cross Sections of Cover System Test Plots at Kirtland Air Force Base in Albuquerque, New Mexico (modified from Dwyer, 1997).

reflectometry (TDR) moisture sensors to monitor the soil water content within the cover system. Each test plot is briefly described below, and summary percolation data is presented in Table 7-5.

Test plot 1 has a RCRA "Subtitle D" prescriptive minimum criteria cover system (hydraulic barrier type). The hydraulic barrier for this system consists of a 0.45-m thick CCL ($k \leq 1 \times 10^{-7}$ m/s). The measured annual percolation through this cover system during the first three years of monitoring averaged 4.82 mm. Dwyer (1998) reported "*As expected, the subtitle D soil cover performed poorly…. Desiccation cracking, freeze/thaw cycles, root penetration, and earthworm and insect activity have acted to increase the permeability.*"

Test plot 2 has a RCRA "Subtitle C" equivalent minimum technology guidance cover system (hydraulic barrier type). The hydraulic barrier consists of a 0.6-m thick CCL ($k \leq 1 \times 10^{-9}$ m/s) overlain by a 1-mm thick LLDPE GM. Importantly, the GM had eight 1 cm^2 holes cut into it to simulate installation-induced defects. Reported average annual percolation is 0.13 mm. Dwyer (1998) reported: "*The other baseline cover - the subtitle C compacted clay cover - had little percolation for most of the year. However, in the past few months percolation has been evident, and the percolation rate is expected to slightly increase with time. One problem with this system is that the geomembrane hampers the ability of the barrier layer to dry by ET; consequently, as additional moisture infiltrates the barrier layer it eventually creates percolation.*" With respect to this cover system, two additional comments are provided: (i) the frequency and size of holes in the GM component of the cover system are significantly larger than would normally be anticipated in a good GM installation; and (ii) evaluation by the researchers involved in the project indicate that percolation through the CCL may be primarily due to concentrated flow through desiccation cracks that developed during construction.

Alternative test plot 1 is identical to test plot 2 except that a GCL is used in lieu of a CCL. Reported average annual percolation is 1.81 mm. Dwyer reported (1998): "*The GCL cover is not performing as well as expected. There are eight 1 cm^2 defects in the geomembrane. It is hypothesized that moisture moved through the geomembrane via defects or diffusion and penetrated the GCL seams prior to the seams' full hydration and sealing. The GCL could also have been damaged during construction, despite very tight quality control, or through root intrusion.*" Dwyer (2001) gave two additional hypotheses for the apparent increase in GCL permeability: (i) "*the bentonite within the geosynthetic clay liner has desiccated and does not fully repair itself after rewetting*"; and (ii) "*the soils in the Southwest or dry environments are susceptible to ion exchange problems that increase the permeability of the liner*".

Alternative test plot 2 has a capillary barrier type of cover system. The cover system has the following layers, from the surface down: (i) 0.3-m thick topsoil layer; (ii) 0.15-m thick graded sand filter layer; (iii) 0.22-m thick gravel drainage layer; (iv) 0.45-m thick compacted finer-grained soil component of the capillary barrier; and (v) 0.3-m thick sand coarser-grained soil component of the capillary barrier. Reported average annual percolation is 0.87 mm. Dwyer (1998) reported: "*The capillary barrier cover also showed a higher than expected percolation rate for the first year, but the rate is slowing significantly as the surface vegetation thickens with native grasses and shrubs.*"

Alternative test plot 3 includes an "anisotropic" capillary barrier, which is a type of capillary barrier intended to promote unsaturated lateral movement of water through certain soil layers. The components of this system are, from the surface down: (i) 0.15-m thick surface layer consisting of 75% local topsoil and 25% pea gravel; (ii) 0.6-m thick finer-grained soil component of the capillary barrier; (iii) 0.15-m thick fine sand interface layer (wicking layer) intended to promote lateral drainage under unsaturated flow conditions; and (iv) 0.15-m thick pea gravel coarser-grained soil component of the capillary barrier. Reported average annual percolation is 0.16 mm. Dwyer (1998) reported: "*The anisotropic barrier and ET cover are both performing very well. Their percolation rates have decreased, as with the capillary barrier, through increased transpiration from the vegetation growth. Recently, the percolation rates of both of these covers have fallen below that of the compacted clay cover.*"

Alternative test plot 4 is an ET barrier type of cover system consisting of, from the surface down: (i) 0.15-m thick topsoil layer; and (ii) 0.9-m thick native soil layer. Reported average annual percolation is 0.19 mm. Dwyer (2001) reported: "*The evapotranspiration cover appears to be leading the way in the third year of testing. This test reveals that in dry environments a well-designed simple soil cover is not only the cheapest alternative but also the most effective at controlling infiltration.*"

Table 7-5. Summary of field performance data for Albuquerque, New Mexico test plots (data from Dwyer, 2001).

Year	Precipitation Collected (L)	Percolation (mm)					
		Test Plot 1	Test Plot 2	Alt. Test Plot 1	Alt. Test Plot 2	Alt. Test Plot 3	Alt. Test Plot 4
1997 (May to Dec)	154,585	10.62	0.12	1.51	1.62	0.15	0.22
1998	169,048	4.96	0.30	0.38	0.82	0.14	0.44
1999	130,400	3.12	0.04	4.31	0.85	0.28	0.01
2000 (Jan to June)	28,151	0.00	0.00	0.00	0.00	0.00	0.00
Average	--	4.82	0.13	1.81	0.87	0.16	0.19

The ALCD project will provide additional valuable information as it is monitored for a period of at least five years. Already, the inadequacy of the "Subtitle D" minimum technology guidance cover system has been demonstrated and the effectiveness of the "Subtitle C" equivalent minimum technology guidance cover system is being confirmed. Percolation results to date for the test plots with the anisotropic capillary barrier and ET barrier are also promising. To date, data provided from this demonstration has been favorably considered by regulators to allow for the use of alternative cover systems in lieu of a prescriptive cover in several areas in the southwestern United States.

7.2.6 Test Plots in Los Alamos, New Mexico

Nyhan et al. (1997) described the performance of sixteen test plots constructed at Los Alamos National Laboratory for the Protective Barrier Landfill Cover Demonstration. The plots had four different cover system configurations, which were each constructed on slopes of 5, 10, 15, and 20%. None of the plots was vegetated, apparently to simulate conditions in which plants provided no transpiration. Precipitation, runoff, lateral drainage, percolation, and soil water content are being measured for each test plot.

The four cover system cross sections that were constructed are as follows:

- Test cover 1: the "conventional Los Alamos design" with, from top to bottom, 0.15 m of loam topsoil, 0.76 m of silty sand, and 0.3 m of gravel.
- Test cover 2: the "EPA design" with, from top to bottom, 0.15 m of loam topsoil, a GT filter/separator, 0.3 m of drainage sand, and a 0.6-m thick bentonite clay-sand CCL.
- Test cover 3: the "loam capillary barrier design" with 0.6 m of loam topsoil overlying 0.76 m of fine sand.
- Test cover 4: the "clay loam capillary barrier design" with 0.6 m of clay loam topsoil overlying 0.76 m of fine sand.

Test cover performance data presented by Nyhan et al. (1997) for the first 4½ years of monitoring show that Test cover 2 has performed better than the other cover system configurations. There has been no evidence of percolation for test cover 2 even though its CCL was only protected by 0.45 m of soil. The bentonite clay mixed in with sand to form the hydraulic barrier apparently helped the water balance at the site (Bonaparte et al., 2002). The highest amount of percolation was recorded for test cover 1; measured percolation rates for the test cover 1 plots ranged from 174 mm for the 5% slope to 31 mm for the 25% slopes over the 4½ -year monitoring period. Measured percolation rates for test covers 3 and 4, respectively, ranged from 76 and 48 mm for the 5% slope, 36 and 0 mm for the 10% slope, and 0 mm for both cover system cross sections on the 15% and 25% slopes.

Even though test cover 2 appears to have a favorable water balance, there is still the concern that the CCL may degrade over time. Based on the other field studies discussed in this section, desiccation of CCL barriers in cover systems is a distinct long-term possibility.

7.3 GM Barriers

7.3.1 Percolation through GM Barriers

Several of the soil barrier studies described in Section 7.2 included test plots containing GMs. The studies of Melchior et al. (1994) and Melchior (1997a,b) provide very good results for cover system test plots containing GM/CCL composite barriers. As reported in Section 7.2.5, average measured percolation rates for test plots containing seamed GMs averaged 1.3 mm/year, with the measured percolation being attributed to thermally-induced moisture movement in the CCL, not leakage through the composite barrier. The results from Dwyer (2001) for the GM/CCL composite hydraulic barrier are also quite good, even with the eight holes cut in the GM by the researchers. The average measured percolation rate for this cover system was 0.13 mm over the three-year monitoring period. Conversely, the percolation rates for the GM/GCL composite hydraulic barrier reported by Dwyer (2001) are high and may be due to the holes cut into the GM or other factors described in Section 7.2.6. More data for this test plot are needed, and further investigation into the percolation mechanisms is underway.

7.3.2 GM Barrier Seam Problem Due to Contamination

Calabria and Peggs (1996) described a cover system project in Pennsylvania where a high rate of HDPE GM barrier seam failures occurred during construction. The 1.0-mm thick textured HDPE GM was installed over a MSW landfill between November 1994 and March 1995. The project specifications required that both the inside and outside tracks of GM fusion seam samples be destructively tested. Initially only the inside track of fusion seam samples was destructively tested in shear and peel. After about 50% of the GM installation had been approved, based on passing destructive test results, and the approved portion of the GM had been covered with a soil layer, it was determined that the outside track of fusion seam samples had not been tested. Archived fusion seam samples were subsequently obtained and tested. About 60% (i.e., 25 of 42) of the archived seam samples had inside track peel test failures, primarily due to seam separation exceeding the minimum specified value of 10%. Most of the failures were associated with four of nine seaming machines and two of nine operators. Fifty percent (i.e., 6 of 12) of the extrusion seam samples taken from the section of GM not covered with topsoil also failed. These failure frequencies for fusion and extrusion seam samples do not include samples collected and tested to isolate poor quality seams. The installer attributed the high seam sample failure frequencies to certain volatile constituents (i.e., benzene, toluene, ethylbenzene, and xylenes (BTEX)) in landfill gas being absorbed by the HDPE GM and inhibiting the fabrication of good seams. However, after the installer sent a new supervisor to the site, the failure rate for extrusion seams dropped.

Calabria and Peggs (1996) performed an investigation to determine if the amount of BTEX absorbed by the HDPE GM impacted seam quality at the site. The investigation included obtaining archived seam samples for destructive testing and microstructural examination and analyzing GM from the site for BTEX constituents. They also exposed site-specific GM samples to BTEX, seamed them, and tested them in peel and shear. Calabria and Peggs found that most of the archived fusion seam samples showed rippling along the seam tracks and extensive warping. They attributed the ripples to GM overheating (setting the seaming machine temperature too high and/or speed too low). They attributed the warp to manual adjustment of the seaming machine to change its direction. They also noted that the GM at the outer edge of the seam tracks was notched, creating a location where stresses could be concentrated, which could potentially lead to stress cracking. Other seams had linear features oriented along the length of the seam in areas of the seams where the GM was shiny and not heated sufficiently to melt its surface. Calabria and Peggs attributed these linear features to soil particles being dragged along the seam by the hot wedge of the seaming machine.

Selected seam samples from the installed GM were collected and analyzed for BTEX constituents and subjected to peel testing. None of the constituents was detected at a concentration greater than 1 mg/kg. No relationship was found between constituent concentration and seam failure rate. Site-specific GM samples exposed to BTEX, seamed, and then tested in peel were found to have good quality seams. Based on their investigation, Calabria and Peggs concluded that the high failure rate for GM seam samples was predominantly caused by soil in the seams (i.e., inadequate cleaning prior to seaming). Other causes of failure were overheating and, for extrusion seams, inadequate grinding. The BTEX absorbed by the GM had no apparent impact on seam quality. The following lessons can be learned from this case study:

- The absorption of relatively low concentrations of BTEX by HDPE GM appears not to affect the quality of seams subsequently constructed.
- HDPE GM must be thoroughly cleaned along a seam path before the seam is constructed since dirt in the seam adversely impacts seam integrity.
- Dual track fusion seaming machines are designed to make high quality seams along two tracks. Both tracks should be destructively tested since failure of one track is generally indicative of overall seaming problems, and failure of one track can increases the stress in the adjacent track.

7.3.3 GM Barrier Seam Problem Due to Moisture

In a cover system application in the southeastern U.S., a 0.9-mm thick CSPE-R GM was installed using a solvent seaming method. The overlap width was 75 to 100 mm. Seaming was typically performed in the early morning hours from sunrise until 9:00 or 10:00 a.m. so as to avoid intense heat during the day. As the project progressed, there were observations of unbonded blisters within the seam area particularly in the afternoon. The blisters varied in size (from 10 to 50 mm and either circular or elliptical in shape) and were numerous.

Upon sampling and seam testing, it was determined, primarily from the results of peel tests, that there were indeed unbonded areas within the seam at the locations of the blisters. Microscopic examination showed that the solvent did not dissolve the resin in these same areas. The reason for the afternoon observation of the blisters is that the air in the unbonded areas expanded as the GM temperature increased.

After considerable evaluation, it was concluded that the high relative humidity and resulting moisture during the evening and early morning left the GM wet. The installation crew was not diligent in making the opposing surfaces in the area to be bonded completely dry and the undulating surface of the scrim-reinforced GM contributed to their resistance to drying the GM using rags or wipes. After the installation crew began using a portable heater to dry the area to be bonded, the problem was avoided for the remainder of the project. Repair of the seamed areas with blisters was performed using a 0.3-m wide cap strip over the entire width of the original seam.

This case history, along with the previous one in Section 7.3.2, emphasizes that, regardless of seaming method, field seaming of GMs has two paramount requirements: (i) the area to be seamed must be <u>clean</u>; and (ii) the area to be seamed must be <u>dry</u>.

7.3.4 Temperature Fluctuations During GM Installation

This project involved closure of an industrial hazardous waste landfill in the southeastern U.S. during hot, mid-summer conditions. Large temperature fluctuations during cover system installation presented the installer of a 2-mm thick HDPE GM barrier with several challenges. Daytime temperature fluctuations of 11 to 17 °C were commonly observed during the installation. The excessive heat made welding conditions difficult. The expansion and contraction of the GM also caused problems.

The closure design required pipe boot penetrations for gas vents and for cover system geosynthetics to be tied into the existing liner system along the perimeter of the landfill. The majority of the GM barrier production welds were of the double-track fusion type. The perimeter tie-in was performed manually using an extrusion welder. Due to high daytime temperatures, extrusion welded seams for the tie-in had to be cooled immediately after seaming to ensure that seam separation did not take place prior to extrudate hardening. Water-cooled towels were used to accelerate hardening of the weld. Extreme care was required to maintain a continuous weld. In addition, during hotter periods of the day, compensation wrinkles were added upslope and parallel to the perimeter tie-in. These compensation wrinkles had a tendency to creep downslope, accumulating at the tie-in, and in some places requiring repair (Figure 7-6a).

During welding of the tie-in, after a cooling rain, the GM barrier contracted sufficiently to pull on the gas vent pipe boots at the landfill crest. The stress in the GM was sufficient to distort the gas vent pipes from a vertical to an inclined position (Figure 7-6b). Repairs were made to the affected pipe penetrations by the installation of additional compensation wrinkles near the pipe penetrations.

(a) (b)

Figure 7-6. Effect of Temperature Fluctuations During GM Installation: (a) GM Wrinkles at Sideslope Toe; and (b) GM Contraction after a Rain.

The installation of cover system geosynthetics under variable high-temperature conditions, as in this case history, requires not only an understanding of GM thermal expansion and contraction characteristics, but also limitations of welding techniques and other factors. To reduce the effects of temperature for these types of conditions, the design engineer can specify GMs with lower coefficients of thermal expansion, light colored GMs, or provisions for keeping dark colored GMs covered with temporary light-colored protection (e.g., light-colored GTs) at all times. Also, the design engineer can specify that GM seaming be performed during relatively cool periods only (i.e., early morning or evening, under cloudy conditions, etc.).

7.3.5 Fate of GM Wrinkles

A 10-ha landfill vertical expansion in the mid-Atlantic U.S. required installation of geosynthetics over the existing waste mass. The geosynthetics serve not only as a cover system over the existing waste but also as a part of the liner system for the new expansion area. The cover system consisted of, from top to bottom: cover soil; GC drainage layer; and 2-mm thick HDPE GM barrier.

Close coordination was needed between the geosynthetics installer and the earthwork contractor during placement of the cover soil over the GC drainage layer. When wrinkles were observed during the placement of soil over the geosynthetics, spotters were used to “walk out” the wrinkles. Part way through the installation, the CQA engineer determined that an area of cover soil did not meet specification. When the non-complying soil was removed and the geosynthetics uncovered, it was found that the GM wrinkles had persisted and several had folded over and crimped (Figure 7-7). The crimping occurred even though the overburden stress was small, due only to the protective cover soil. These observations are consistent with the laboratory findings on the fate of wrinkles presented in Koerner et al. (2002).

Repairs were made to the GM that had been creased during the prior placement of cover soil. Additional restrictions were then imposed on earthwork operations. Placement of protective cover soil was restricted to early mornings and evening hours (when the GM was cool and contracted) to minimize wrinkle formation. Full time spotters and personnel were required to be present at all times during protective cover placement.

Even though spotters had been used during the initial placement of the cover soil, wrinkles in the GM were found to occur. This case history further highlights the need to control placement of soils over geosynthetics and to minimize wrinkling in GMs. It is important to keep GM wrinkles from folding over since this creates strain concentrations, and hence stress concentrations, in the GM. It is noted that it has been shown analytically that the size of the wrinkles can be reduced by increasing the shear strength between the GM and the underlying material (Giroud, 1994). Therefore, for example, the use of textured rather than smooth GM may reduce the risk that large wrinkles will form.

Figure 7-7. Wrinkles Developed in an HDPE GM and Folded Over During Placement of Cover Soil.

7.4 Slope Stability

7.4.1 Overview

Gross et al. (2002) identified cover system slope instability as the most common type of problem encountered at landfills. Gross et al. collected available information on cover system slope stability failures, for which they found:

- four landfills at which cover system slope failures occurred during construction;
- eleven landfills at which cover system slope failures occurred after rainfall or thaw; and
- three landfills at which soil cover damage occurred after an earthquake.

Each of these three types of cover system slope stability problems is discussed below. In addition, the results from the EPA-sponsored GCL test plot slope stability program are also discussed.

7.4.2 Cover System Slope Failure During Construction

Cover system slope failures during construction have been described by Paulson (1993), Boschuk (1991), and Gross et al. (2002). The primary causes of failure were identified as: (i) placing soil over the sideslope geosynthetics from the top of the slope downward, rather from the toe of the slope upward; (ii) using unconservative presumed values for critical interface shear strengths; and (iii) using interface shear strength values from laboratory tests performed under conditions not representative of the actual field conditions.

At a landfill described by Paulson (1993), the design called for geosynthetic reinforcement to be installed over a nonwoven GT cushion and then covered with soil. The GT cushion was underlain by a smooth GM barrier. The reinforcement was to be anchored on the top of the landfill by covering a length of the reinforcement with soil. Slope stability analyses conducted during design assumed that soil would be placed over the reinforcement from the bottom of the slopes upward, after the reinforcement had been anchored. However, this requirement was not incorporated into the construction specifications. When construction began, access to the bottom of the slope was not available, so the contractor started placing soil from the crest of the slope downwards. Shortly afterwards, a section of cover system involving the soil, reinforcement, and GT cushion slid along the interface between the GT and the underlying GM barrier. The main factor leading to the failure was placement of cover soil from the top down. Moreover, the construction specifications did not place any limitations on the size or ground pressure of the construction equipment used, nor on its mode of operation. Consistent with the recommendation of Daniel and Koerner (1993), soil layers should normally be placed over geosynthetics from the toe of slope upward to minimize construction-induced tension in the geosynthetics and take advantage of passive soil resistance at the toe of slope.

At a landfill described by Boschuk (1991), a gravel drainage layer placed on top of a smooth GM barrier on a 3H:1V slope, slid down the slope, damaging the underlying GM. The contractor had tried to place the gravel by pushing it up the slope with a bulldozer and then by placing it on the slope using a clamshell bucket, but neither method worked. Apparently, the drainage layer material did not develop adequate interface shearing resistance with the underlying GM.

Adequate design-phase interface shear testing and slope stability analyses with materials representative of final construction would have prevented this problem.

At another landfill discussed by Boschuk (1991), as soil was being placed over the already-installed sand drainage layer on a 3H:1V a slope, the sand slid downslope over a heatbonded nonwoven GT. Apparently the sand was too coarse to penetrate into the heatbonded GT openings. Project-specific interface direct shear tests between the sand and GT performed prior to construction resulted in an interface friction angle of about 21° indicating the slope would be stable. The tests were performed, however, at normal stresses much larger than the actual field loading condition. Tilt table interface shear tests performed after the failure and at a lower normal stress representative of field conditions produced a sand/GT interface friction angle of about 18°. This latter test result indicates marginal slope instability for this interface on a 3H:1V (18.3°) slope. The cover system was reconstructed with a needlepunched nonwoven GT that had a higher interface shear strength with sand than the calendered GT. The lesson from this case study is that interface direct shear tests should be performed under laboratory test conditions representative of those expected in the field.

Gross et al. (2002) described a project involving closure of 32-m long, 3H:1V landfill sideslopes. The design called for geogrid reinforcement to be installed over a smooth HDPE GM barrier and then covered with overlaying soil layers, with the first such layer being a sand drainage layer. The construction specifications required the reinforcement to be anchored on the top of the landfill by extending the reinforcement onto the top deck and covering it with the soil layers prior to placing soil over the reinforcement on the sideslope. Slope stability analyses were conducted assuming that the soil layers would be placed over the reinforcement from the bottom of the slope upward. However, this condition was not incorporated into the construction specifications. When construction began, existing gas wells on the top deck interfered with geogrid installation. Where the gas wells interfered with installation, the adjacent geogrid strips stopped short and did not extend back to their full design anchorage length. Access to the bottom of the sideslopes was limited at some locations due to wetlands near the slope toe. As a consequence of these conditions, the contractor placed the sand by pushing it from the crest downward. This mistake was compounded by the fact that the contractor created a sand stockpile on the slope near the crest. Shortly after sand placement began, the anchored geogrid layers ruptured at the slope crest beneath the sand stockpile and construction equipment. The GM then tore near the slope crest and along outward diagonals down the length of the GM on both sides of the stockpile. The cover system was subsequently redesigned using textured rather than smooth HDPE material. The lessons from this case study are that geosynthetics need to be properly anchored prior to placing soil cover, soils should not be stockpiled on top of geosynthetics on slopes (unless accounted for in the design), and soil cover should be placed from the bottom of the slope up.

7.4.3 Cover System Slope Failure After Rainfall or Thaw

Gross et al. (2002) presented case studies of cover system slope failures due to rainfall or thawing conditions at eleven landfills. The primary causes of failure were identified as: (i) not accounting for seepage forces; (ii) clogging of the internal drainage layer, which leads to increased seepage forces; and (iii) not accounting for moisture at the GM/CCL interface (which weakened the interface) due to both rain falling on the CCL surface during construction and freeze-thaw effects.

Inadequate Design for Seepage Forces: Five cover system slope failures were primarily attributed to rainfall-induced seepage pressures in soil layers above the failure surface. The cover systems for the landfills involved in these failures have 3H:1V or 2.5H:1V sideslopes and are up to about 60 m in slope length. Available details on the cover system failures are given below.

- Bonaparte et al. (1996) and Vander Linde et al. (2002) described the failure of a cover system for a landfill in north Georgia. The cover system consisted of a 0.3-m thick topsoil layer over a stitch-bonded reinforced GCL barrier. Sideslopes were 3H:1V and up to 54 m in length. The cover system did not have an internal drainage layer and was designed without consideration of rainfall-induced seepage forces in the topsoil layer. Construction of the system was completed in the fall of 1994. During the winter of 1995, the cover system experienced several episodes of downslope movement. Each episode of movement was immediately preceded by a significant rainfall event. The nature of the slope movement is illustrated in Figure 7-8 and photographs of the failure are presented in Figure 6-1. Analyses performed after the failure demonstrated substantial seepage force buildup due to rainfall, resulting in a calculated factor of safety of less than 1.0 for sliding of the topsoil layer on top of the GCL. The main lesson from this case study is that seepage forces should be considered in evaluating cover system stability. When seepage forces are accounted for, they will typically lead the design engineer to incorporate an internal drainage layer into the cover system design whenever a conventional design approach (involving hydraulic barriers and maximum slopes in the range of 4H:1V to 3H:1V) is used.
- Boschuk (1991) described a project involving a cover system on a 3H:1V slope. The cover system consisted of, from top of bottom: topsoil layer; medium-coarse sand drainage layer; woven GT reinforcement layer; and GM barrier. Project-specific interface shear testing was not performed. The design engineer assumed a sand/GT interface friction angle of 24°, or about two-thirds of the sand angle of internal friction. The sand slid on the underlying GT after a rainfall event estimated by Boschuk to have a two-year recurrence interval. Gross et al. (2002) calculated slope stability factors of safety of 1.34, 0.98, and 0.63 for this project assuming infinite slope conditions, a 24° interface friction angle and, respectively, conditions of no seepage force, seepage in the sand layer, and full seepage in the sand and overlying topsoil layer. The main lesson from this case study, like the previous one, is that seepage forces should be accounted for in evaluating cover system stability. A secondary lesson from this case study is that project-specific interface shear testing should be performed.
- Boschuk (1991) described an additional cover system failure where the primary causes of failure were inadequate (or no) consideration of seepage forces and/or inadequate characterization of interface shear strengths. The cover system cross section consisted of, from top to bottom, topsoil layer, sand drainage layer, and GM barrier. The sand drainage layer had a specified minimum hydraulic conductivity of 1×10^{-4} m/s. The type of GM is not identified in the case study. Sliding occurred along the sand/GM interface after three days of rainfall. A steady-seepage infinite slope analysis was conducted by

Gross et al. (2002) for this case study. In their analysis, a secant friction angle of 20° was assumed for the sand/GM interface. The calculated slope stability factors of safety are 1.09 and 0.80, respectively, without and with full seepage forces in the sand layer. A lesson from this case study is that sand drainage layers with a hydraulic conductivity of 1 x 10^{-4} m/s may not be permeable enough to convey flow without the buildup of seepage forces. A higher permeability drainage medium would perform better.

- Soong and Koerner (1997) described the 1995 failure of a cover system on a 40-m long, 2.5H:1V slope that occurred after a heavy rainfall. The cover system consists of a 0.75-m thick silty sand layer (approximate hydraulic conductivity of 1 x 10^{-5} m/s) underlain by a CCL barrier. About two to three years after the cover system was constructed, the sand slid downslope over the CCL during a storm. The slide was relatively small and localized. Soong and Koerner attributed the failures to seepage forces that developed in the sand layer. An infinite slope analysis was conducted by Gross et al. (2002) for this case study. In their analysis, the friction angle for the sand was assumed to be 30°. The calculated slope stability factors of safety are 1.44 and 0.66 without and with full seepage forces in the sand layer, respectively. The lesson from this case study is similar to the previous one: cover system internal drainage layers may need to have a hydraulic conductivity much larger than 1 x 10^{-5} m/s to prevent significant seepage forces. A higher permeability drainage medium would perform better.
- Soong and Koerner (1997) also described the 1996 failure of a cover system on a 50-m long, 3H:1V slope. The cover system consists of a 0.6-m thick topsoil layer overlying a 0.3-m thick sand drainage layer (approximate hydraulic conductivity of 1 x 10^{-4} m/s), which in turn overlies a CCL barrier. About five to six years after the cover system was constructed, the sand slid downslope over the CCL immediately after a storm. At least four localized slides occurred. Soong and Koerner attributed the slides to relatively high seepage forces that developed in the cover system because the drainage layer hydraulic conductivity was too low. The timing of the slides (5 to 6 years after closure) suggest that clogging of the sand drainage layer may have occurred to some extent. An infinite slope analysis was conducted by Gross et al. (2002) for this case study. In their analysis, the friction angle for the sand was assumed to be 30°. The calculated slope stability factors of safety are 1.73 and 1.40 without and with full seepage forces in the sand layer, respectively. With seepage forces in the sand and topsoil layers, the calculated factor of safety is 0.77. The lessons from this case study are that: (i) the hydraulic conductivity of cover system internal drainage layers may need to be larger than 1 x 10^{-4} m/s to prevent significant seepage forces; and (ii) clogging of an internal drainage layer can reduce its effectiveness. This latter effect is discussed in more detail below.

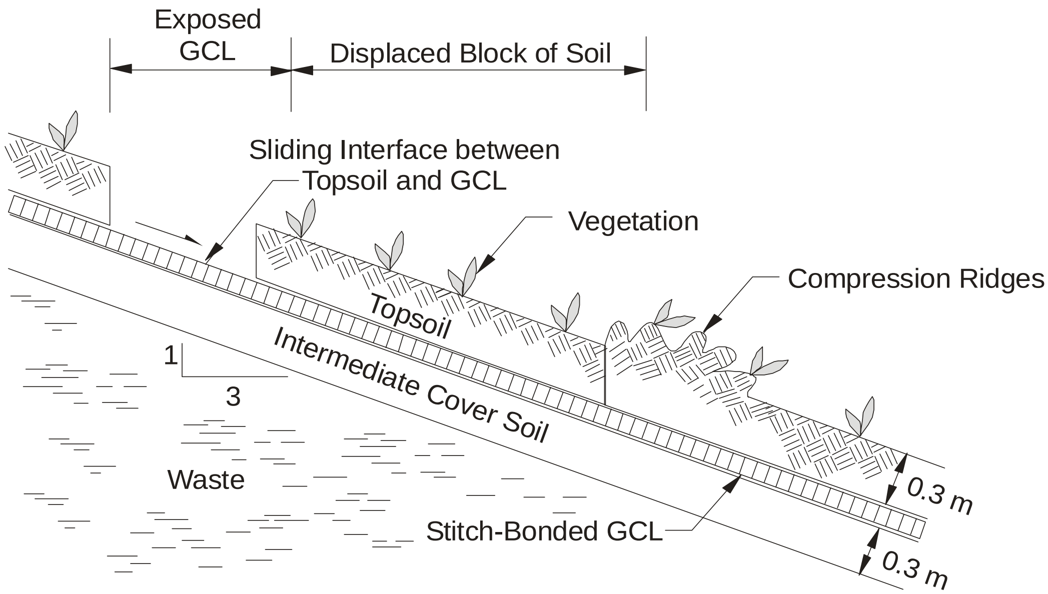

Figure 7-8. Observed Failure Mechanisms for Sliding of Soil Layer Over Stitch-Bonded GCL.

Clogging of Internal Drainage Layer: Clogging of the cover system internal drainage layer can impair the ability of the layer to freely drain, resulting in a buildup of hydraulic pressure and failure of the cover system. This mechanism was identified as the primary factor contributing to slope stability problems at five landfills. Available details on these cover system slope failures are given below.

- Boschuk (1991) described a cover system slope failure that appeared to be related to clogging of the sand drainage layer. The cover system consists of, from top to bottom: topsoil layer; gap-graded sand drainage layer (minimum hydraulic conductivity of 1×10^{-4} m/s); and smooth GM barrier. The cover system slopes ranged from about 50 to 90 m in length. Within one year of the completion of construction, the entire lower third of the cover system slid downslope along the sand/GM interface. The sand drainage layer in the slide zone contained significant fines, presumably washed into the sand from the topsoil layer and the sand in the upper two-thirds of the slope. Boschuk (1991) indicated that fines migration had so reduced the hydraulic conductivity of the sand drainage layer at the bottom of the slope that the layer liquefied under the induced hydraulic head buildup. Lessons learned from this case study are as follows:
  - Gap-graded soils are more prone to migration of finer-sized particles (i.e., internal instability) than well-graded soils. Particle migration may result in clogging of the soil. Therefore, if gap-graded soils are used as drainage materials, the potential for particle migration should be evaluated during design.
  - A granular soil drainage layer needs to have a filter to protect against migration of particles from the overlying topsoil or protective soil layer. This aspect of the design should be performed using available filter criteria (see Chapter 4 of this

document) and/or laboratory testing. GT filter layers can be used in the design if a sand filter layer is not available or is too costly.

  - Cover system slopes should always be evaluated for stability using rigorous analysis methods that consider the anticipated seepage forces and interface/internal shear strengths applicable to the cover system.

- Boschuk (1991) described a project where the cover system consists of from top to bottom: topsoil layer; sand drainage layer; and smooth GM barrier. Perforated collection pipes wrapped with a nonwoven GT filter were installed in the sand drainage layer. After a period of time, fines clogged the GT at the pipe perforations, hydraulic head built up the sand drainage layer, and the cover system slid downslope. Failure occurred at the sand/GM interface, primarily on the lower third of the slope. After the failure, the pipes were observed to be dry and the surrounding sand saturated. This failure might have been prevented if the GT filter wrapped around the pipe had been adequately designed. A thinner, more open, GT, that allows fine soil particles to pass through but which retains the sand, would have performed better. Much better, however, would have been to not wrap the pipe in a GT filter at all, but rather to bed the pipe in drainage gravel and place a properly designed GT filter around the gravel, or to design the system to allow unimpeded migration of fines through the pipe perforations. The problems associated with placement of GT filter layers around pipes (as was done in this case study) have been clearly described by Bass (1986), Koerner et al. (1993), and Giroud (1996).

- Another failure described by Boschuk (1991) involved a cover system consisting of, from top to bottom: topsoil layer; nonwoven GT filter; gravel drainage layer; and GM barrier. Over time, the GT became clogged by the topsoil. As a consequence, infiltrating rainwater did not drain freely from the topsoil into the underlying gravel. Pore pressures increased in the topsoil layer, and the topsoil slid downslope over the GT. Failure occurred primarily on the lower third of the slope. Boschuk (1991) did not indicate if filter design, interface direct shear testing, or a slope stability analysis were performed as part of the cover system design. The GT should have been designed to be compatible with the topsoil using filter criteria calculations and/or laboratory testing. Compatibility between topsoil and GT filter layers should always be carefully evaluated because the topsoil may have a low degree of internal stability. Internally unstable soils will typically be poorly graded, with significant fines and little cohesion.

- Soong and Koerner (1997) described the failure of a cover system on a 45-m long, 3H:1V slope that occurred in 1996. The cover system consists of, from top to bottom: 0.75-m thick topsoil layer; 0.3-m thick sand drainage layer; and CCL barrier. The design called for water in the sand drainage layer to flow to the toe of the slope where it would be collected in a gravel toe drain and then conveyed through a pipe to a discharge point. The gravel toe drain was not wrapped with a GT filter. Five to six years after the cover system was constructed, a number of localized slides of the sand over the CCL occurred. When the gravel toe drain was exhumed, the gravel was found to be very contaminated with fines, which presumably migrated into the gravel from the overlying sand and topsoil. Soong and Koerner attributed the failure to relatively high seepage forces that developed in the cover system after the gravel toe drain became clogged. Lessons learned from this case study are similar to those learned from the previous case studies.

- Soong and Koerner (1997) described the failure of a cover system on a 45-m long, 2.5H:1V slope between benches that occurred in 1996. The cover system consists of, from top to bottom: 0.6-m thick topsoil surface/protection layer; 0.2-m thick sand drainage layer; and CCL barrier. The design called for water in the sand drainage layer to flow to the toe of the slope where it would be collected in a gravel toe drain and then conveyed through a pipe to a discharge point. The pipe was wrapped with a GT filter. As with the previous case study, about five years after the cover system was constructed, a number of small localized slides of the sand over the CCL occurred. When the gravel toe drain was exhumed, the GT filter layer was found to be clogged with fines at pipe perforations. The fines presumably migrated to the GT from the sand and topsoil. Soong and Koerner attributed the failure to hydraulic head that developed in the cover system after the GT around the pipe became clogged. As previously discussed, wrapping of perforated pipes in GTs should be avoided if at all possible due to the relative inefficiency of placing the filter layer at this location and the potential for clogging (Bass (1986), Koerner et al. (1993), and Giroud (1996)).

<u>Moisture Changes at GM/CCL Interface:</u> Gross et al. (2002) described a case study involving a cover system for which construction was not completed until late fall. The project site is located in northern Ohio. The cover system cross section is illustrated in Figure 7-9. During the first winter after landfill closure, the cover system was covered with snow and the ambient temperature was below freezing until the spring.

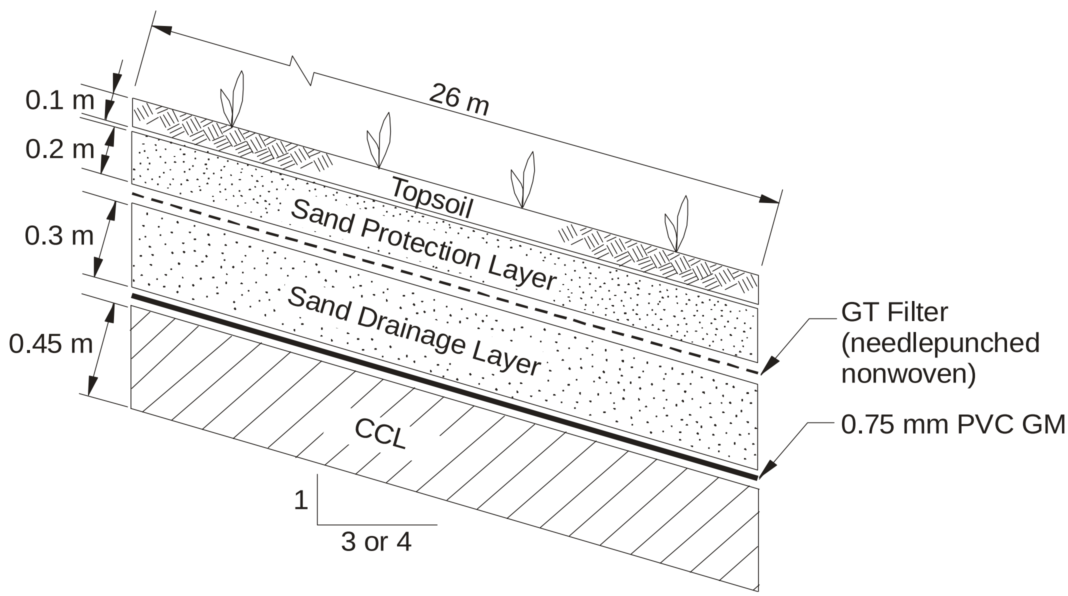

Figure 7-9. Cover System Cross Section for Northern Ohio Landfill that Underwent a Slope Failure after Thaw.

A few days after the first spring thaw, the PVC GM component of the cover system slid over the CCL component on a portion of 4H:1V slope. An initial investigation after the failure revealed that water could not exit from the sand drainage layer because the lower end of the drainage layer was blocked by ice and snow. As a result, the cause of the slide was initially assumed to be the buildup of hydraulic head resulting from the thawing of the blocked drainage path. However, subsequent slope stability analyses demonstrated that seepage forces above the GM would have had little effect on the factor of safety with respect to a slide that occurs at an interface located beneath the GM (see Chapter 6 of this guidance document). With seepage forces identified as only a minor potential contributor to the slope failure, an additional investigation was conducted to evaluate the shear strength characteristics of the GM/CCL interface and, in particular, the effect of temperature fluctuations on interface strength. Interface shear tests simulating the conditions during the winter (-7 °C) followed by thaw (+0.5 °C) showed that the formation of ice lenses at the GM/CCL interface at below-freezing temperature increased the water content at the interface during thaw. This resulted in a marked decrease of the interface shear strength after the thaw, compared to the interface shear strength before freezing. Slope stability calculations incorporating the results of the interface shear strength testing program showed that the cover system would be unstable on a 4H:1V slope if the moisture content of the CCL exceeded 23%. Systematic measurements of field CCL moisture content showed that this moisture content was likely exceeded in the area where the slide occurred, while the condition was not met in other areas. This localized effect (i.e., higher water content) was attributed to heavy rainfall that preceded the installation of the GM in the area where the slide eventually occurred.

The main lessons from this case study is that freeze-thaw cycles have a significant effect on interface shear strengths. To avoid potential problems, the interface should be located below the depth of frost penetration. Also, rainfall onto a CCL immediately prior to GM placement can lead to lower interface strengths than obtained in interface shear tests performed at "as compacted" moisture contents.

7.4.4 Soil Cover Damage Due to Earthquakes

Loma Prieta Earthquake: The epicenter of the 17 October 1989 (moment magnitude M_W 6.9) Loma Prieta earthquake was located approximately 16 km northeast of the City of Santa Cruz. The focal depth was approximately 18 km, with a fault plane dipping about 10 degrees from the vertical to the west. The Loma Prieta event produced observational data on the seismic performance of older, unlined solid waste landfills. Orr and Finch (1990), Johnson et al. (1991), and Buranek and Prasad (1991) reported on post-earthquake inspections of fifteen landfills. None of the landfills subjected to strong shaking in the Loma Prieta event were instrumented. The estimated bedrock peak horizontal ground accelerations (PHGA) at the base of the landfills in the Loma Prieta event ranged from 0.1 g to 0.5 g. All of the post-earthquake damage investigators reported only minor or moderate damage (as defined by Matasovic et al. (1995)) to landfills in this event, with the most common damage being cracking of the cover soil on the landfill slopes and at transitions between waste and natural ground. Johnson et al. (1991) and Buranek and Prasad (1991) noted that it was often difficult to distinguish between "normal" cracks induced by waste settlement and/or decomposition and earthquake-induced cracking. Repair of this type of cover soil cracking is performed regularly as part of routine landfill maintenance activities. The earthquake induced cracks in the cover soil were repaired by landfill maintenance crews immediately following the earthquake without disruption to landfill operations. Orr and Finch (1990) note that some of the landfill gas recovery systems were temporarily affected by power loss and that there was above-ground pipe breakage at a number of the landfills impacted by the Loma Prieta earthquake. However, according to these investigations, all landfill gas recovery systems were repaired and back in operation within 24 hours of the earthquake, and there were no reported post-earthquake changes in quantities of leachate and extracted landfill gas.

Among the landfills closest to the Loma Prieta earthquake zone of fault rupture, observational data exist for the Guadalupe, Ben Lomond, Kirby Canyon and Santa Cruz landfills. The estimated bedrock PHGAs for these landfills are 0.43 g, 0.38 g, 0.34 g and 0.30 g, respectively. As reported by Johnson et al. (1991), even the highest slopes at these landfills, which include 2H:1V slopes up to 45 m high at the Santa Cruz landfill, 3H:1V slopes up to 45 m high at the Ben Lomond landfill, and 2H:1V slopes up to 75 m high at the Kirby Canyon landfill, performed well, with only minor cracking (25 to 75 mm in width) of cover soils observed. Only at the Guadalupe landfill, as reported by Buranek and Prasad (1991), was minor downslope cover soil movement observed.

Northridge Earthquake: Augello et al. (1995), Matasovic et al. (1995), and Matasovic and Kavazanjian (1996) documented damage to soil cover materials at three landfills in the 17 January 1994 Northridge earthquake (moment magnitude M_w 6.7). This earthquake occurred on a blind thrust fault at a depth of approximately 15 km at the northern end of the San Fernando Valley within the greater Los Angeles area. Estimated PHGA in bedrock at the landfill sites ranged from 0.20 g to 0.42 g. Consistent with observations in the Loma Prieta earthquake, damage in the Northridge event was limited to surficial cracking of cover soils occurring primarily near locations with contrasting seismic response characteristics (e.g., top of waste adjacent to canyon slopes). At two of the landfills, the cracking was relatively minor. At one landfill, a major crack occurred near and parallel to a liner system anchor trench. This crack was about 200-m long, up to 150-mm wide, with the two sides of the crack vertically offset by up to 100 mm. No waste was exposed. At all three landfills, the damage was dealt with as an operation issue through post-earthquake inspection and repair (i.e., regrading and revegetating the cracked soil layers).

The main lesson from these case studies is that surficial cracking of soil cover layers, especially near locations with contrast in seismic response characteristics (e.g., top of waste by sideslopes), should be anticipated and dealt with as an operation issue through post-earthquake inspection and maintenance.

7.4.5 Results of EPA GCL Test Plots

Carson et al. (1998), Daniel et al. (1998), and Daniel (2002) describe the results of an evaluation of 14 GCL field test plots constructed at a landfill test site in Cincinnati, Ohio. The test plots were designed and constructed as prototype landfill cover systems. The purpose of the test plots was to evaluate the internal and interface shear strength characteristics of the commercially-available GCLs under in-service conditions. Five test plots were constructed on a 3H:1V (nominal) slope, and nine test plots were built on a 2H:1V (nominal) slope. Plots on the 2H:1V slope were nominally 20 m long, while those on the 3H:1V slope were 29 m long. All plots were two GCL panel widths (9 m) wide and were covered with 0.9 m of silty, clayey sand.

A typical cross section of a test plot constructed on a 3H:1V slope is shown in Figure 7-10. In general, the test plots were constructed with a double-sided textured GM overlying the GCL, which would be typical of a cover system for a landfill. However, GCLs are also used in cover systems without GMs. Hence, three plots were constructed with no GM. The plots were drained internally above the GM using a GC (GT/GN/GT) drainage layer or, for the plots that did not contain a GM, a sand drainage layer.

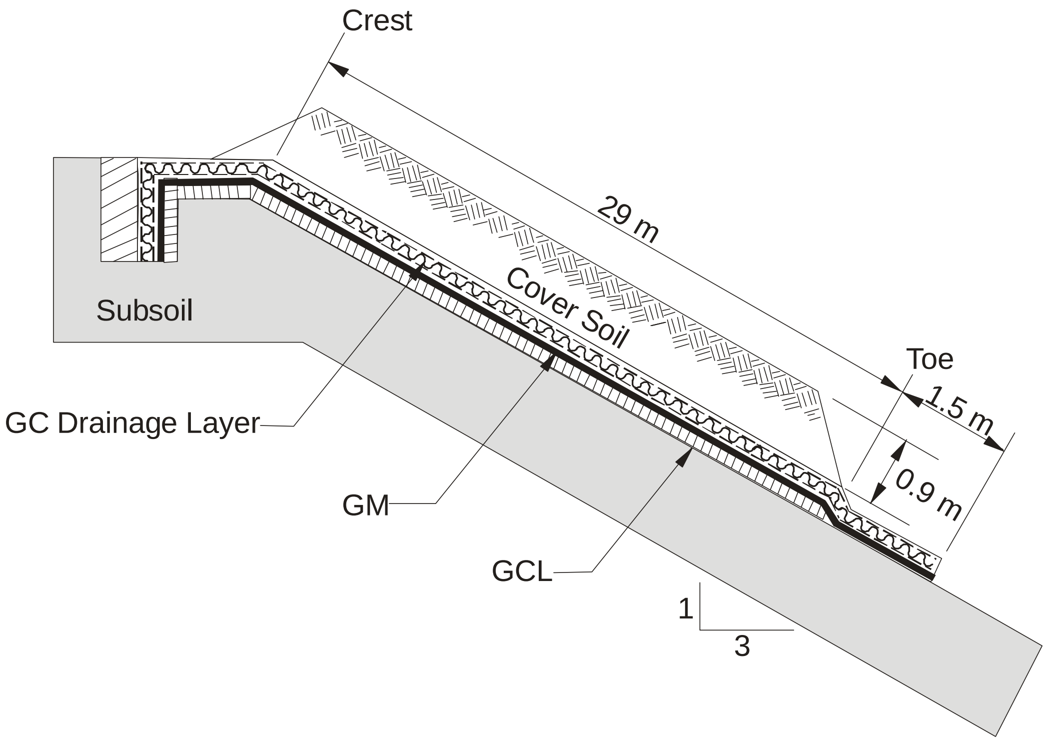

Figure 7-10. Typical Cross Section for 3H:1V Cover System Test Plot in Cincinnati, Ohio (modified from Carson et al., 1998).

The rationale for selecting the 2H:1V and 3H:1V slope inclinations was as follows. The 3H:1V slope was selected to be representative of typical cover systems for landfills in use today. In order to confirm that GCLs are safe against internal failure on 3H:1V slopes, it must be shown that they are not only stable, but are stable with an adequate factor of safety. Slope stability analysis methods are discussed in Chapter 6 of this guidance document. As discussed in Section

6.2.6, a minimum acceptable factor of safety (FS_{min}) for static stability analyses of 1.5 will often be appropriate for permanent cover system applications. The ratio of tanβ for a 2H:1V slope to tanβ for a 3H:1V slope is 1.5. Subject to the assumptions listed above, if a GCL is demonstrated under a given normal stress to be stable on a 2H:1V slope (i.e., FS > 1.0), the same GCL is demonstrated to be stable on a 3H:1V slope at the same normal stress with FS > 1.5. Therefore, the 2H:1V slopes were chosen to demonstrate internal stability of GCLs on 3H:1V slopes with FS > 1.5. However, it was recognized that constructing 2H:1V slopes was pushing the GCLs to (and possibly beyond) their limits of stability.

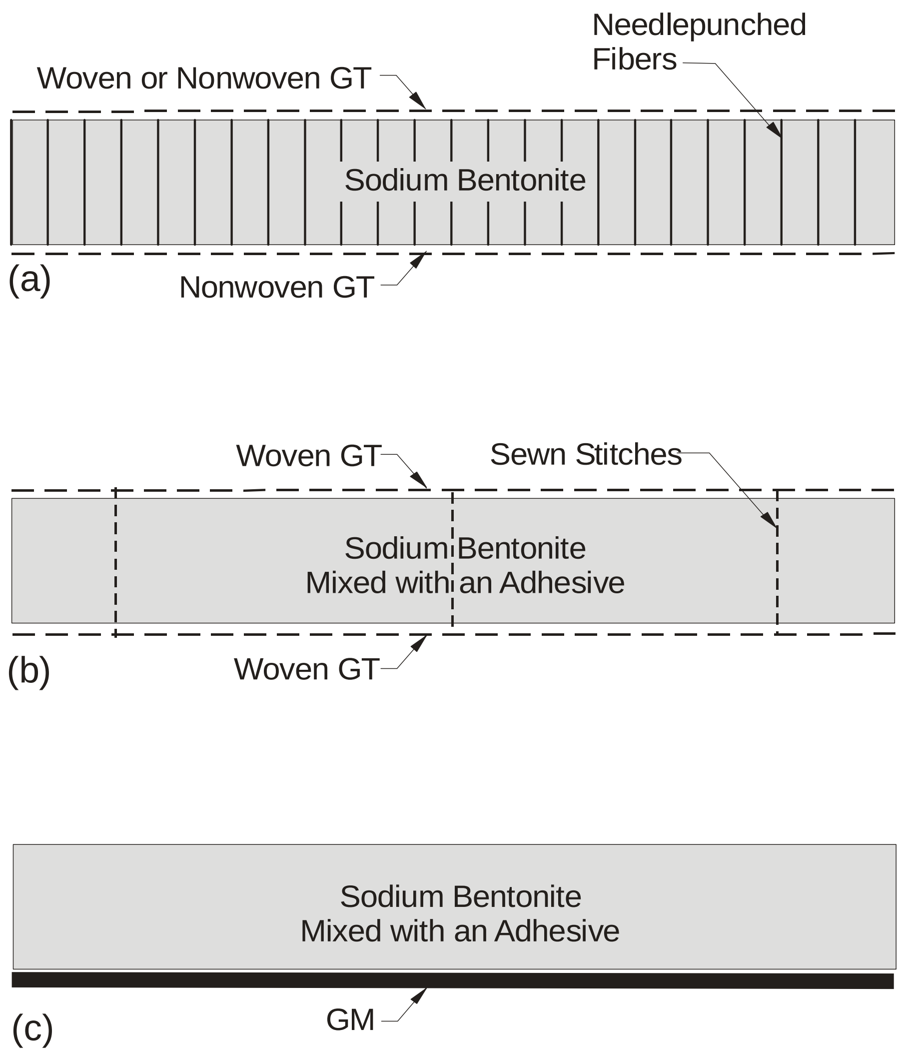

Figure 7-11. Schematics of GCLs Used in Cover System Test Plots in Cincinnati, Ohio: (a) Reinforced, GT-Encased, Needlepunched GCL (e.g., Bentofix and Bentomat); (b) Reinforced, GT-Encased, Stitch-Bonded GCL (e.g., Claymax); and (c) Unreinforced, GM-Supported GCL (e.g., Gundseal).

Three types of GCLs, shown schematically in Figure 7-11, were used in the test plot program: (i) reinforced, GT-encased, needlepunched GCLs (e.g., Bentofix and Bentomat); (ii) reinforced GT-encased, stitch-bonded GCL (e.g., Claymax); and (iii) unreinforced, GM-supported GCL (e.g., Gundseal). For the ten test plots in which a GM was placed over the GCL, the GM was a 1.5-mm thick textured HDPE GM.

Construction of the test plots began on November 15, 1994 and was completed on November 23, 1994. However, one plot (P) was constructed on June 15, 1995. The test plots were first graded to provide a smooth subgrade. Next geosynthetics were installed by pulling them down from the crest of the slope (Figure 7-12), and then cover soil was placed (Figure 7-13) by starting at the bottom of the slope and working upslope. In plots incorporating a GC drainage layer, the GM and GC were extended beyond the GCL at the toe of the slope and another 1.5 m past the end of

Figure 7-12. GCL Panels Deployed on Slopes of Cincinnati, Ohio Test Plots by Pulling Them Downslope from a Spreader Bar at the Slope Crest.

Figure 7-13. Cover Soil on the Cincinnati, Ohio Test Plots Placed over the Geosynthetics from the Slope Toe Upward.

the cover soil (Figure 7-10). For plots constructed with a sand drainage layer, a piece of GC material was embedded in the sand at the toe of the slope and then extended 1.5 m beyond the end of the cover soil.

All of the geosynthetic materials in each test plot were brought into their respective anchor trenches, which were then backfilled. The toe of each test plot was excavated at the completion of construction so that no buttressing (i.e., passive) force could be mobilized at the toe of the slope. To prevent the development of tension in the geosynthetic components above the mid-plane of the GCLs, all components above the mid-plane, including the upper GT of the GCL, were cut at the crest of the slope (Figure 7-14). Cutting occurred in the spring of 1995, after the winter thaw and about five months after construction of the test plots. However, the geosynthetics were not cut in plot P, which was constructed later in the program for the sole purpose of evaluating hydration of bentonite encased between two GMs.

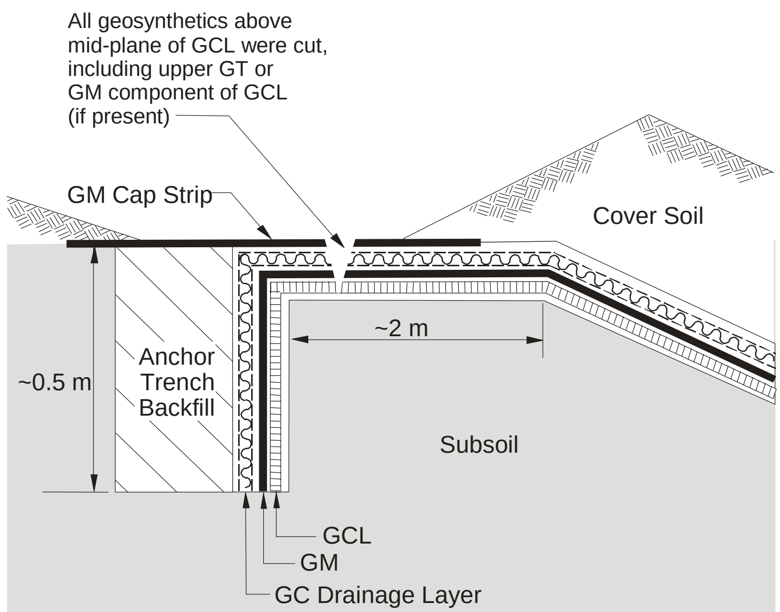

Figure 7-14. Cut in Anchor Trench Geosynthetics above Mid-Plane of GCL on Cincinnati, Ohio Test Plots.

Instrumentation for the test plots included gypsum blocks and fiberglass moisture sensors and wire displacement gauges (extensiometers). A discussion of moisture sensors is provided in Chapter 8 of this guidance document.

As described by Carson et al. (1998), Daniel et al. (1998), and Daniel (2002) the test plots were observed for over three years. A summary of information on the test plots is given in Table 7-6. A summary of results of the test plot program is given in Table 7-7.

Table 7-6. Information on GCL test plots (from Daniel et al., 1998).

Test Plot	Type of GCL	Nominal Slope (H:V)	Target Slope Angle (°)	Actual Slope Angle (°)	Actual Slope Length (m)	Actual Plot Width (m)	Cross Section (Top to Bottom)[1]	GCL Side Facing Upward	GCL Side Facing Downward
A	Gundseal	3:1	18.4	16.9	28.9	10.5	Soil/GC/GM/GCL	Bentonite	GM
B	Bentomat ST	3:1	18.4	17.8	28.9	9.0	Soil/GC/GM/GCL	Woven GT	Nonwoven GT
C	Claymax 500SP	3:1	18.4	17.6	28.9	8.1	Soil/GC/GM/GCL	Woven GT	Woven GT
D	Bentofix NS	3:1	18.4	17.5	28.9	9.1	Soil/GC/GM/GCL	Nonwoven GT	Woven GT
E	Gundseal	3:1	18.4	17.7	28.9	10.5	Soil/GC/GCL	GM	Bentonite
F	Gundseal	2:1	26.6	23.6	20.5	10.5	Soil/GC/GM/GCL	Bentonite	GM
G	Bentomat ST	2:1	26.6	23.5	20.5	9.0	Soil/GC/GM/GCL	Woven GT	Nonwoven GT
H	Claymax 500SP	2:1	26.6	24.7	20.5	8.1	Soil/GC/GM/GCL	Woven GT	Woven GT
I	Bentofix NW	2:1	26.6	24.8	20.5	9.1	Soil/GC/GM/GCL	Nonwoven GT	Nonwoven GT
J	Bentomat ST	2:1	26.6	24.8	20.5	9.0	Soil/GT/Sand/GCL	Woven GT	Nonwoven GT
K	Claymax 500SP	2:1	26.6	25.5	20.5	8.1	Soil/GT/Sand/GCL	Woven GT	Woven GT
L	Bentofix NW	2:1	26.6	24.9	20.5	9.1	Soil/GT/Sand/GCL	Nonwoven GT	Nonwoven GT
M	Erosion Control	2:1	26.6	23.5	20.5	7.6	Soil	No GCL	No GCL
N	Bentofix NS	2:1	26.6	22.9	20.5	9.1	Soil/GC/GM/GCL	Nonwoven GT	Woven GT
P	Gundseal	2:1	26.6	24.7	20.5	9.0	Soil/GC/GM/GCL	Bentonite	GM

[1]GC = GT/GN/GT.

All test plots were initially stable, but over time as the bentonite in the GCLs became hydrated, three slides (all on 2H:1V slopes) involving GCLs occurred. One slide involved an unreinforced GCL in which bentonite that was encased between two GMs unexpectedly became hydrated. The other two slides occurred on 2H:1V slopes at the interface between the woven GT components of the GCLs and the overlying textured HDPE GMs. A photograph of the test plots at which these two interface slides occurred is presented in Figure 7-15.

Table 7-7. Summary of calculated factor of safety (FS) and actual slope stability (from Daniel et al., 1998).

Test Plot Designation	Slope Angle (°)	Peak Friction Angle (°)	Large-Displacement Friction Angle (°)	Peak FS	Large-Displacement FS	GCL Performance
A	16.9	37[2(D)]	35[2(D)]	2.5[2(D)]	2.3[2(D)]	Stable
B	17.8	23[1]	21[1]	1.3[1]	1.2[1]	Stable
C	17.6	20[1]	20[1]	1.1[1]	1.1[1]	Stable
D	17.5	29[1]	22[1]	1.8[1]	1.3[1]	Stable
E	17.7	20[2(H)]	20[2(H)]	1.1[2(H)]	1.1[2(H)]	Stable
F	23.6	20[2(H)]	20[2(H)]	0.8[2(H)]	0.8[2(H)]	Internal Slide
G	23.5	23[1]	21[1]	1.0[1]	0.9[1]	Interface Slide
H	24.7	20[1]	20[1]	0.8[1]	0.8[1]	Interface Slide
I	24.8	37[1]	24[1]	1.6[1]	1.0[1]	Stable[4]
J	24.8	~31[1]	~31[1]	1.3[1]	1.3[1]	Stable[4]
K	25.5	31[3]	31[3]	1.3[1]	1.3[1]	Stable[4]
L	24.9	~31[1]	~31[1]	1.3[1]	1.3[1]	Stable[4]
N	22.9	~37[1]	~24[1]	1.8[1]	1.1[1]	Stable
P	24.7	20[2(H)]	20[2(H)]	0.8[2(H)]	0.8[2(H)]	Stable

[1] GM/GCL interface
[2] Internal GCL strength for dry (D) or hydrated (H) bentonite
[3] GCL/drainage sand interface
[4] Large displacement occurred in subsoil below GCL, but not in or at the interface with GCL

As discussed by Daniel et al. (1998), the experience from these test plots provides several conclusions of practical significance to engineers. At the low normal stresses associated with cover systems, the interface shear strength is generally lower than the internal shear strength of internally-reinforced GCLs. The weakest interface will typically be between a woven GT component of a GCL and the adjacent material, which in this case was a textured HDPE GM. The interface strength may be low in part because of the tendency of bentonite to extrude through the openings in the relatively thin, woven GT and then into the interface as the GCL hydrates. Design engineers are encouraged to consider GCLs with relatively thick, nonwoven GT components in critical situations where high interface shear strength is required.

Current engineering practice for evaluating the stability of GCLs on slopes is to conduct direct shear tests and then to use LE methods of slope stability analysis to calculate factors of safety using the results of those tests. This approach was described in detail in Chapter 6 of this document. The experience from the test plot program has validated this approach. All three test plots that slid had calculated factors of safety less than 1.0. All remaining (stable) test plots had factors of safety greater than 1.0. Based on the experience from this study, cover systems containing GCLs cannot achieve slope stability factors of safety normally considered adequate on 2H:1V slopes. It appears, however, that 3H:1V slopes (depending on materials) can be constructed with factors of safety of at least 1.5 for the conditions existing in this project.

Figure 7-15. Plots G (Left) and H (Right) Approximately Two Months After Construction and Several Days after the Slide in Plot G.

7.5 Waste Settlement

Gross et al. (2002) described a project in which landfill settlement caused tearing of cover system GM boots around gas well penetrations. The landfill cover system has a 1-mm thick HDPE GM barrier and was constructed in 1991 and 1992. By late 1992, a gas collection system, including vertical HDPE gas collection wells that penetrate the GM barrier, had been installed in the landfill. At each penetration, an HDPE GM boot was clamped to the well and extrusion welded to the GM barrier to seal the barrier around the well. When several of the GM boots around the wells were inspected in 1995, the boots were observed to be torn from the GM barrier. The boots were not designed to accommodate settlement of the waste, which would cause downward displacement of the GM barrier relative to the wells. Since the cover system had been constructed, the landfill top deck had settled from 0.3 to 0.9 m. The problem was resolved by replacing the gas extraction well boots with new expandable boots that can elongate up to 0.3 m. These boots can also be periodically moved down the well to accommodate landfill settlement. The lesson from this case study is that GM boots in cover systems must be designed to accommodate landfill settlements.

Another example of the impacts of settlements on a cover system is shown in the photographs in Figure 7-16. Surface tension cracks caused by differential settlement of underlying MSW are clearly evident. The cracks occurred in a soil cover system at an arid site in the western U.S.

Figure 7-16. Surface Tension Cracks in Cover Soils from Differential Settlement of Underlying MSW.

The soil material used to construct the cover consists of a silty, gravelly sand, intended to be resistant to erosion and desiccation cracking. These cracks were observed to occur throughout the cover system, with most aligned perpendicular to the slope (constant elevation). The cracks were observed to act as drains for surface runoff during infrequent storm events, allowing percolation into the waste mass.

7.6 Stormwater Management and Erosion Control

7.6.1 Failure of Erosion-Mat Lined Downchute

Harris et al. (1992) described the failure of a geosynthetic erosion mat-lined downchute on the cover system of a landfill in Missouri. An erosion mat was used to line one downchute that conveyed runoff from approximately 2 ha of cover system and 8 ha of adjacent property; riprap was used to line the remaining three downchutes that drained a total of about 10 ha. The erosion mat consisted of a polyethylene, three-dimensional, turf reinforcement mat (TRM). The mat-lined downchute was installed on the top deck, starting in about 3 m from the slope crest, down the sideslope, and along a perimeter section of the landfill. At the inlet, the downchute slope is about 5%, and runoff is diverted into the downchute by small diversion berms. The downchute grade increases to 33% on the sideslope. Near the slope toe, the downchute has a more gentle inclination of about 8%. Riprap was placed in the downchute at this lower slope transition for energy dissipation. TRM was supplied in rolls that were 1.5 m wide and 30 m long. Adjacent rolls were overlapped at least 75 mm and secured to the underlying soil with 200-mm long staples installed at 0.75 m spacings. Roll ends overlapped a minimum of 0.45 m and were shingled downward. TRM was also anchored in 0.3-m deep trenches at the top of each roll and along the sides of the downchute. After the mat was placed, grass seed was applied and covered with about 13 mm of topsoil. Within one month after construction, following a series of significant rainfall events, the channel was unserviceable. Soil had raveled along the sides of the downchute, soil had eroded underneath the mat and along mat panel overlaps, and the mat had moved downslope about 2 m. Failure of the mat appeared to have started at the top of the slope and progressed downward. Though grass was becoming established across the cover system by this time, there was little grass in the downchute at the time of failure.

The most severe damage to the downchute is believed to have occurred after a peak rainfall intensity of about 64 mm/hr, estimated to represent a 1-hr storm with a 5-year recurrence interval. The peak runoff from this storm in the downchute on the sideslope was estimated by Harris et al. (1992) to be 1.33 m^3/s. The corresponding peak velocity in the downchute was calculated to be 2.9 m/s. After the failure, a detailed laboratory testing program was conducted to evaluate the relationship between flow velocity and erosion of a mat-lined surface for a simulated flow duration of 0.5 hr. The results of the study indicated that fully-grassed, mat-lined channels had noticeable erosion at flow velocities of about 5 m/s. However, without grass, the velocity required to develop noticeable erosion was about 3 m/s. Harris et al. (1992) concluded that the combination of large drainage area, steep slope, and the inability of grass to sprout quickly in the channel lead to failure of the downchute.

Based on the information in Harris et al. (1992), the following lessons can be learned from this case study:

- Flow velocities in drainage channels under the design storm should be calculated so the appropriate channel lining can be selected. If an erosion mat is selected for a channel and the erosion mat cannot withstand the design flow velocities until grass is established, significant maintenance and/or failure of the downchute should be anticipated.
- If the downchute had been constructed earlier, within the plant growing season, the grass may have become established faster and erosion of the downchute may have been less severe. The mat was installed and seeded in the fall, when plant growth is relatively slow, resulting in an extended period with poor to no grass cover in the downchute. The average plant growing season at the site starts in April and ends in October, the month in which construction of the downchute was completed. Every effort should be made to establish cover system vegetation prior to the onset of cool fall weather.

7.6.2 Excessive Erosion and Gullying

Gross et al. (2002) described a 16 ha landfill cover system, with 60-m long slopes inclined at 3H:lV. The design called for sand berms to divert surface-water runoff from the top deck of the landfill to six riprap-lined downchutes on the landfill sideslopes. Sand diversion berms were also located at a few locations on the sideslopes. The cover system consists of the following components, from top to bottom:

- vegetated topsoil layer, 0.2 m thick on the top deck and 0.3 m thick on the sideslopes;
- sand drainage layer with a specified minimum hydraulic conductivity of 1 x 10^{-5} m/s, 0.2 m thick on the top deck and 0.4 m thick on the sideslopes; and
- 1-mm thick HDPE GM barrier.

Within three years after construction, about 0.8 ha of the cover system was severely eroded and about 0.1 ha of cover soil had slid downslope. Sixteen deep gullies developed on the landfill sideslopes in the vicinity of the riprap-lined downchutes and in areas where the sand berms at the slope crest had been breached due to differential settlement and sheet flow concentration on the top deck. Gullies typically started near the slope crest and propagated downslope. The gullies extended through the topsoil and sand drainage layers down to the GM barrier (Figure 7-17). In two areas, major sliding of the topsoil and sand drainage layers occurred. In several locations, the GM was damaged by punctures and tears, and the subgrade beneath the GM was irregular. EPA HELP model simulations conducted after the erosion was observed indicated that the sand drainage layer had insufficient capacity to convey surface-water infiltration from the 25-year, 24-hour storm. Under this condition, the flow that could not be conveyed within the drainage layer backed-up into the overlying topsoil layer and as surface flow. Seepage forces in the sand drainage layer and topsoil layer reduced slope stability and increased surface erosion. Other project details that contributed to the development of erosion and gullies at the site include: (i) sand diversion berms and downchutes were designed such that they did not intercept lateral flow in the sand drainage layer; (ii) runoff collected by berms and downchutes could infiltrate through the topsoil layer and enter the drainage layer; and (iii) a lack of access control resulted in unauthorized trafficking of four-wheel drive vehicles and dirt bikes on the landfill. The following lessons can be learned from this case study:

- The surface-water runoff management strategy for this landfill, which did not result in diversion of internal drainage from the top deck to the downchutes and allowed uninterrupted sheet flow over the 60-m long, 3H:lV sideslopes, proved inadequate to prevent surface erosion and localized slope instability. A design that incorporated both drainage layer interceptors and surface-water runoff interceptors (such as benches or swales) on the sideslopes would likely have been more effective in limiting erosion and localized failure.
- Design analyses for this facility did not adequately characterize potential peak flows in the sand drainage layer. For future projects, it is recommended that the guidance given in this document be used to estimate the required flow capacity. Also, as previously discussed in this document, a hydraulic conductivity of 1×10^{-5} m/s for a cover system drainage layer is too low for many applications, including this case study. Hydraulic conductivity values in the range of 1×10^{-3} m/s, or even higher, will often be necessary to allow unimpeded drainage while minimizing the build-up of seepage forces in the sideslope.
- Design of the drainage layer at slope transitions (e.g., drain outlets and benches) is critical to the effective functioning of the drainage layer. If not properly designed, flow will back up and generate hydraulic pressure at the slope transition. For flow not to back up in a drainage layer flowing full, flow capacity across the slope transition must not decrease. Chapter 4 of this document provides guidance on the design of internal drainage layers at slope transitions and outlets.

Figure 7-17. Deep Gullies Through the Topsoil and Sand Drainage Layers Exposed the GM Barrier on 60-m Long, 3H:1V Landfill Sideslopes.

Figure 7-18 presents a photograph (Dwyer 1997) of a RCRA Subtitle D cover system for a closed MSW landfill in an arid area of the western U.S., with no surface-water runoff control system for the portion of the landfill shown in the photograph. Erosion gullies can be clearly seen in the photograph. These gullies were formed by a single intense storm. The gullies were deep enough to cut through the entire cover, exposing waste. The cover system sideslopes are about 3H:1V and the surface layer consists of a silty, gravelly sand.

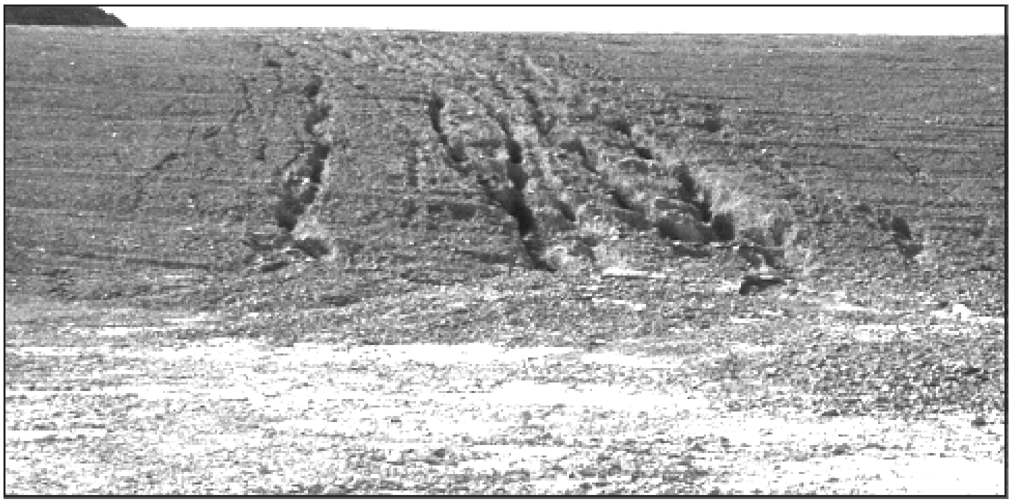

Figure 7-18. Gullies on a RCRA Subtitle D Cover System without Surface-Water Runoff Control System and Located in an Arid Setting.

7.6.3 Failure of Surface-Water Runoff Collector

Figure 7-19 presents photographs of a failed surface-water runoff control system for another portion of the closed MSW landfill at an arid climate site described above. The control system consists of corrugated metal pipe-arch culverts installed in the cover system to both intercept downslope surface-water runoff (culverts placed perpendicular to slope) and convey collected runoff to a designated collection area (culverts placed in downslope direction). The design basis for the culverts is not known. The hydraulic capacity of the culverts was not adequate to contain the runoff and overflow occurred during previous storm events. It appears that the sides of the culvert blocked entry of runoff into the culvert, causing surface water to flow parallel to the culvert. This resulted in erosion adjacent to, and beneath, the culvert, exposing waste. Also, the culverts were prone to siltation and infilling. The lessons learned from this case study are that non-vegetated cover soils must be designed to convey surface-water runoff without excessive erosion, runoff interceptors and conveyance structures must be adequately sized, and inlets to the structures must be designed to not impede the inflow and cause erosion around the structure.

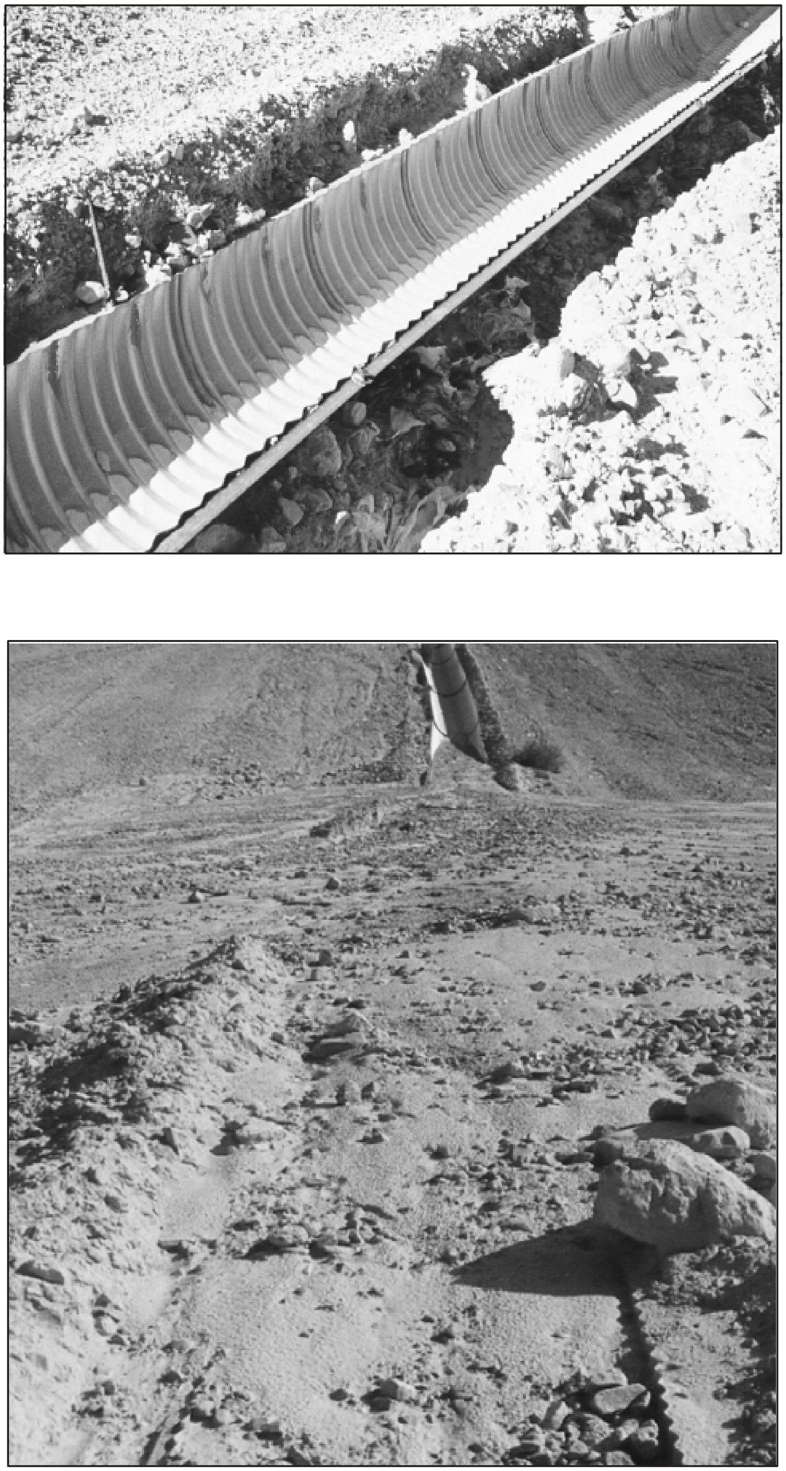

Figure 7-19. Failed Surface-Water Runoff Control System for Another Portion of the Closed MSW Landfill Located at an Arid Climate Site and Shown in Figure 7-18.

7.7 Gas Pressures

A 40 ha closed MSW landfill was being utilized for gas recovery via deep wells that were placed at approximately 100 m spacings. The cover system included a GM barrier overlain by a combined 1 m thickness of various soil layers (Figure 7-20). The well perforations began 8 to 10 m below the cover system (i.e., the upper portion of the wells were not perforated). As a consequence of the wide well spacing combined with the absence of perforations in the upper part of the well, gas generated in the upper portion of the landfill accumulated beneath the cover system, generating uplift pressures on the underside of the GM. As the gas pressure beneath the GM increased, the normal stress, and, thus, the shear strength, between the underside of the GM and the GT beneath it decreased (Figure 7-20). This resulted in the GM and overlying materials gradually moving downslope. The GM and overlying GC (GT/GN/GT) strained considerably at the top of the slope (Figure 7-21) and folded over at the toe of the slope (Figure 7-22). Tension cracks were also evident at the top of the slope, and bulging of vegetation, cover soil, and geosynthetics was apparent at the toe of the slope. The lesson from this case study is that the spacing of gas extraction wells must be close enough to prevent the buildup of gas pressure on the underside of the cover system. Also, well perforations must not be so deep as to create a "dead zone" with respect to gas collection beneath the cover system. In some cases, a granular or geosynthetic gas transmission layer should be used to provide for more efficient movement of landfill gas to well locations.

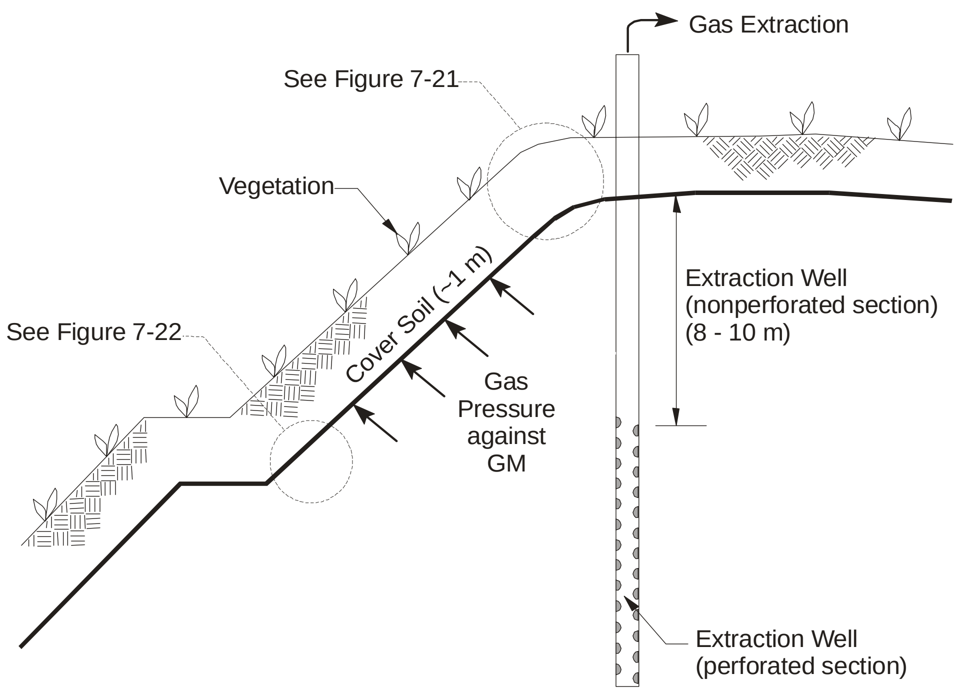

Figure 7-20. Gas Pressures Built Up Beneath Cover System of Closed MSW Landfill Because Upper Portion of Gas Extraction Wells Was Not Perforated.

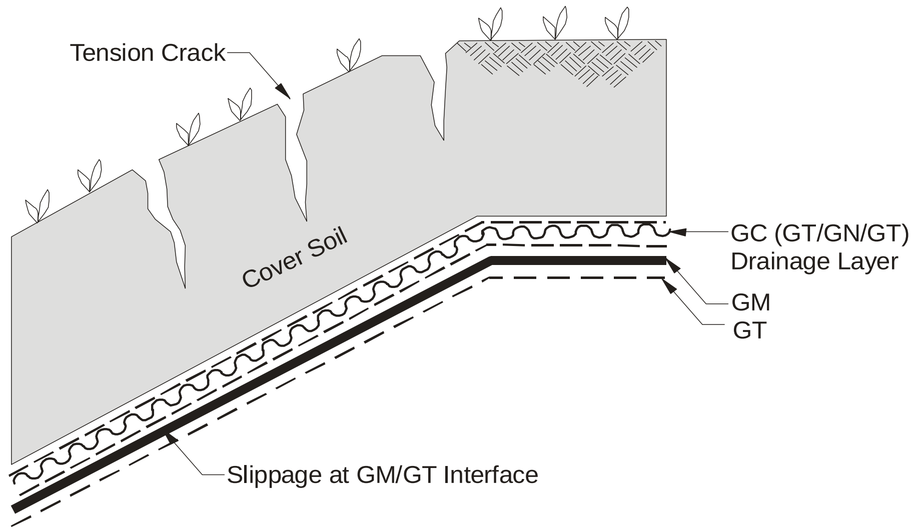

Figure 7-21. Gas Pressures Beneath Cover System GM Resulted in Slippage at GM/GT Interface with Straining of the GM and GC at the Slope Crest.

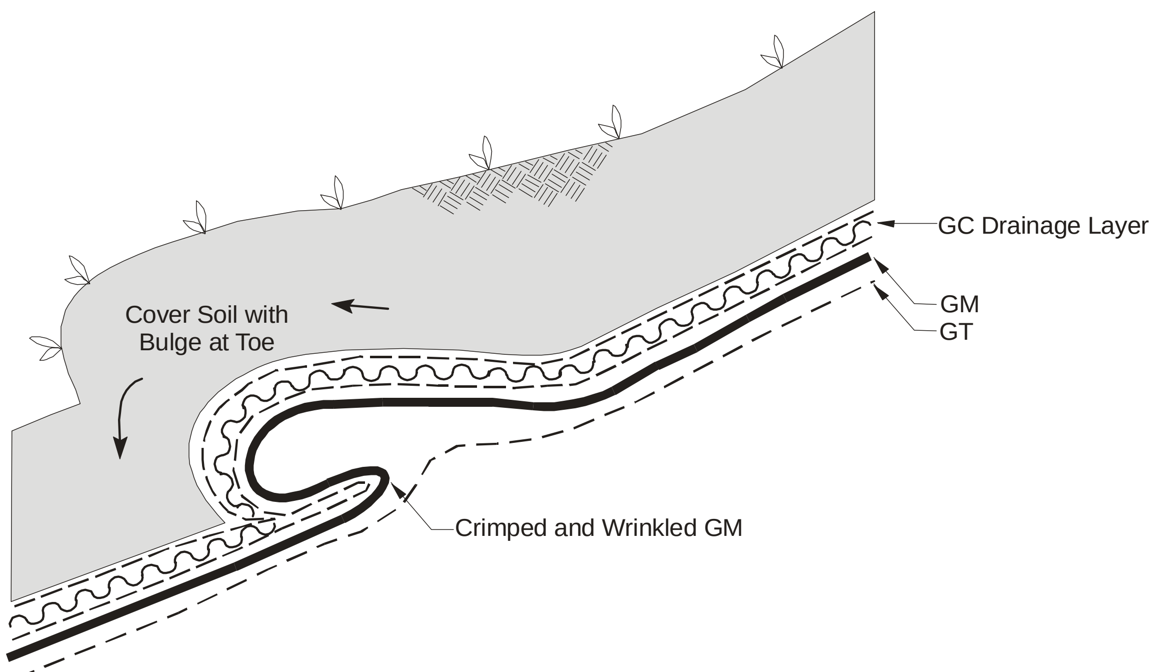

Figure 7-22. Slippage at GM/GT Interface Also Caused the GM and GC to Fold Over at the Slope Toe.

At the extreme, gas uplift pressure can be so great as to cause the GM to push the cover soil aside and expand into a large “whale” as shown in Figure 7-23. The reader should note that this photograph was not taken at the site of the case study described above, but (in other cases) this extreme situation has been observed to occur. Landfill gas, if not collected, will also impact cover systems that do not contain GMs. Vegetation on many landfill cover systems has been killed by landfill gas emissions. Figure 7-24 presents a photograph (Dwyer 1997) illustrating this problem. Even in arid climates with non-vegetated surface layers, the impacts of gas migration can be evident. Figure 7-25 shows surface stains produced by landfill gas throughout the cover of a closed MSW landfill.

Figure 7-23. GM “Whale” Caused by Gas Pressures Beneath the GM.

7.8 Miscellaneous Problems

Gross et al. (2002) described a project in New York involving the inadvertent use of contaminated topsoil. During placement of the topsoil layer for a landfill cover system, several truckloads of soil brought to the site by the contractor had an aromatic odor. The project specification for topsoil prohibited deleterious material in the topsoil, so topsoil hauling was ceased until the affected soil could be tested. Samples of the affected soil were collected and analyzed for VOCs and metals. Based on the results of the testing, the soil was found to contain unacceptably high concentrations of lead. Topsoil that smelled aromatic or contained chemicals ionized by a photoionization detector was removed from the site. Each truckload of topsoil subsequently brought to the site was screened using the above criteria. EPA recommends that soil borrow sources be investigated by the owner unless the materials are supplied by a commercial materials company (Daniel and Koerner, 1993). In the case study described above, topsoil was excavated by the contractor from an off-site property. If the owner had required that test pits be excavated so the topsoil could be inspected prior to construction, the topsoil contamination may have been identified earlier. The soil contamination also might have been identified earlier if the contractor had been required to submit chemical analyses on samples of the topsoil brought to the site.

Figure 7-24. Landfill Cover System Vegetation Killed by Landfill Gas.

Figure 7-25. Surface Stains on Landfill Cover System Caused by Landfill Gas.