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Office of Solid Waste and
Emergency Response
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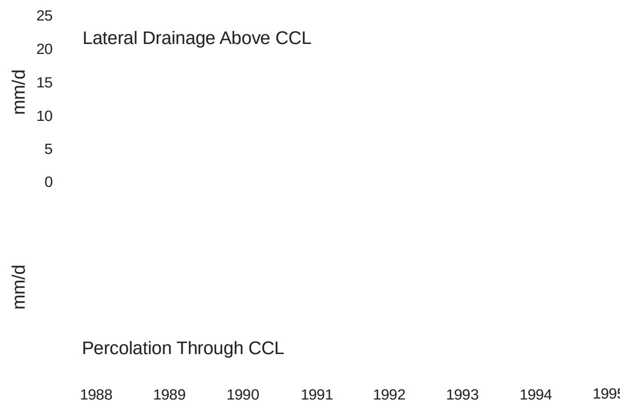

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List of Acronyms

ACAP	=	Alternative Cover Assessment Program
ALCD	=	Alternative Landfill Cover Demonstration
AOS	=	apparent opening size
ARAR	=	applicable or relevant and appropriate requirement
ASTM	=	American Society for Testing and Materials
BOD	=	biological oxygen demand
BTEX	=	benzene, toluene, ethylbenzene, and xylenes
C&DW	=	construction and demolition waste
CAA	=	Clean Air Act
CCL	=	compacted clay liner
CERCLA	=	Comprehensive Environmental Response, Compensation and Liability Act
CFR	=	U.S. Code of Federal Regulations
CH	=	high-plasticity clay (according to USCS)
CL	=	low-plasticity clay (according to USCS)
COD	=	chemical oxygen demand
CQA	=	construction quality assurance
CQC	=	construction quality control
CSPE	=	chlorosulfonated polyethylene
CSPE-R	=	chlorosulfonated polyethylene - reinforced
DOE	=	U.S. Department of Energy
EG	=	emissions guidelines
EIA-R	=	ethylene interpolymer alloy-reinforced
EPA	=	U.S. Environmental Protection Agency
EPS	=	expanded polystyrene
ET	=	Evapotranspiration
FDEP	=	Florida Department of Environmental Protection
FDR	=	frequency domain reflectometry
FHWA	=	Federal Highway Administration
FID	=	flame ionization detector
FOS	=	filtration opening size
FS	=	factor of safety
fPP	=	flexible polypropylene
fPP-R	=	flexible polypropylene reinforced
GC	=	geocomposite
GCL	=	geosynthetic clay liner
GM	=	geomembrane
GN	=	geonet
GPR	=	ground penetrating radar
GPS	=	global positioning system
GT	=	geotextile
HAP		hazardous air pollutants
HDPE	=	high density polyethylene

HELP	=	Hydrologic Evaluation of Landfill Performance
HW	=	hazardous waste
HSWA	=	Hazardous and Solid Waste Amendment
ISM	=	instantaneous surface monitoring
ISS	=	integrated surface sampling
IW	=	industrial waste
L	=	Liters
LandGEM	=	EPA Landfill Gas Generation Model
LCRS	=	leachate collection and removal system
LDLPE	=	low density linear polyethylene
LDR	=	Land Disposal Restrictions
LDS	=	leak detection system
LE	=	limit equilibrium
LEACHM	=	Leachate Estimation and Chemistry Model
LLDPE	=	linear low density polyethylene
LMDPE	=	linear medium density polyethylene
lphd	=	liter/hectare/day (1 lphd = 9.35 gallon/acre/day (gpad))
MACT	=	maximum achievable control technology
MCL	=	maximum contaminant level
MSE	=	mechanically stabilized earth
MSW	=	municipal solid waste
MSWLF	=	municipal solid waste landfill
NCDC	=	National Climatic Data Center
NCP	=	National Contingency Plan
NESHAP	=	National Emission Standards for Hazardous Air Pollutants
NMOC	=	non-methane organic compound
NPDES	=	National Pollution Discharge Elimination System
NRC	=	U.S. Nuclear Regulatory Commission
NRCS	=	National Resources Conservation Service
NSPS	=	New Source Performance Standards
OII	=	Operating Industries, Inc.
OU	=	operable unit
PCB	=	polychlorinated biphenyl
PCDD	=	polychlorinated dibenzo-p-dioxins
PCDF	=	polychlorinated dibenzo-furans
PE	=	polyethylene
PERM	=	permanent erosion and revegetation material
PET	=	potential evapotranspiration
PHGA	=	peak horizontal ground acceleration
PMP	=	probable maximum precipitation
PPL	=	priority pollutant list
ppm	=	parts per million
PVC	=	polyvinyl chloride
QA	=	quality assurance
QC	=	quality control
ROD	=	Record of Decision

RCRA	=	Resource Conservation and Recovery Act
RCPS	=	rigid cellular polystyrene
RUSLE	=	Revised Soil Loss Equation
RWEQ	=	Revised Wind Erosion Equation
SARA	=	Superfund Amendments and Reauthorization Act
SASW	=	spectral analysis of surface waves
SC	=	clayey sand (according to USCS)
SCS	=	USDA Soil Conservation Service
SDRI	=	sealed double-ring infiltrometer
SE	=	southeast
SITE	=	Superfund Innovative Technology Evaluation
SMCL	=	secondary maximum contaminant level
SVOC	=	semivolatile organic compound
SVT	=	solvent vapor transmission
SWRRB	=	Simulation for Water Resources in Rural Basins
TDR	=	time domain reflectometry
TDS	=	total dissolved solids
TERM	=	temporary erosion and revegetation material
TR-55	=	Technical Release 55 (SCS, 1986)
TRM	=	turf reinforcement mat
TSS	=	total suspended solids
TOC	=	total organic carbon
TOC	=	total organic compound (in Chapter 5)
UMTRA	=	Uranium Mill Tailings Remedial Action
UMTRCA	=	Uranium Mill Tailings Radiation Control Act
USCS	=	Unified Soil Classification System
USDA	=	United States Department of Agriculture
USFWS	=	U.S. Fish and Wildlife Service
USGS	=	U.S. Geological Survey
USLE	=	Universal Soil Loss Equation
VFPE	=	very flexible polyethylene
VLDPE	=	very low density polyethylene
VOA	=	volatile organic acid
VOC	=	volatile organic compound
WES	=	U.S. Army Corps of Engineers Waterways Experiment Station
WVT	=	water vapor transmission
XPS	=	extruded polystyrene

List of Variables

A	=	dimensionless parameter (dimensionless)
A	=	area of emitting source (m^2) (in Chapter 5)
A_b	=	area of drainage basin or subbasin per basin or subbasin width (m^2/m)
A_s	=	average annual soil loss by sheet and rill erosion (tonnes/ha/yr)
a_a	=	cohesion (for internal strength) or adhesion (for an interface) for the critical potential slip surface above the hydraulic barrier (Pa)
a_{ai}	=	apparent adhesion (for an interface) or cohesion (for internal strength) for the critical potential slip surface (Pa), as defined in Figure 6-8
a_b	=	cohesion (for internal strength) or adhesion (for an interface) for the critical potential slip surface below the hydraulic barrier (Pa)
a_i	=	adhesion (for an interface) or cohesion (for internal strength) for the critical potential slip surface (Pa)
B	=	dimensionless parameter (dimensionless)
B	=	Distance over which differential settlement, Δ, occurs (m)
C	=	vegetative cover and management factor (dimensionless)
C_d	=	empirical factor (dimensionless)
C_e	=	void ratio correction factor (dimensionless)
C_F	=	vegetal cover factor (dimensionless)
C_I	=	vegetal retardance curve index (dimensionless)
C_r	=	runoff coefficient (dimensionless)
C_s	=	surface layer coefficient (dimensionless)
CN	=	runoff curve number (dimensionless)
COG	=	combined crop factors (dimensionless)
$C_{\alpha\varepsilon}$	=	modified secondary compression index (dimensionless)
$C_{\alpha\varepsilon 1}$	=	modified secondary compression index during the intermediate secondary compression period (dimensionless)
$C_{\alpha\varepsilon 2}$	=	modified secondary compression index during the long-term secondary compression period (dimensionless)
$C_{i,1}$-$C_{i,2}$	=	concentration gradient of species i (Mg/m^3)
c	=	runoff coefficient (dimensionless)
c_s	=	cohesion of soil material above the critical potential slip surface (Pa)
D	=	flow depth (m)
D_i	=	depth of influence (m)
D_i	=	diffusivity of species i through cover material (m/yr^2) (in Chapter 5)
D_{15}	=	particle size at which 15% by dry weight of the soil particles are smaller (mm)
D_{50}	=	minimum gravel or riprap mean particle size to withstand the peak rate of runoff (mm)
D_{85}	=	particle size at which 85% by dry weight of the soil particles are smaller (mm)

d = depth of rainfall in time of concentration from a storm with a certain return period (m)
E = equivalency factor (dimensionless)
E_v = vertical evaporative flux (mm/day)
EF = erodible fraction (dimensionless)
ER_i = mass emission rate of species i (Mg/yr)
ET = evapotranspiration (mm/day)
F = flow concentration factor (dimensionless)
F_w = seepage force (N)
FS = factor of safety (dimensionless)
FS_A = factor of safety for critical potential slip surface above the hydraulic barrier (dimensionless)
FS_B = factor of safety for critical potential slip surface below the hydraulic barrier (dimensionless)
FS_{min} = minimum acceptable factor of safety (dimensionless)
$f(S)$ = slope function (dimensionless)
f_w = seepage force per unit volume (N/m^3)
G = dynamic shear modulus (Pa)
G/G_{max} = dynamic shear modulus reduction factor (dimensionless)
G_s = specific gravity of gravel or riprap (dimensionless)
G_{max} = maximum small-strain dynamic shear modulus (Pa)
g = acceleration of gravity (m/s^2)
H = height of the falling weight (m)
H_f = elevation difference along flow path (m)
H_s = soil layer thickness (m)
H_w = depth of water that can be stored in a soil layer for subsequent removal by plants
H_1 = height of waste at time t_1 (m)
H_2 = height of waste at time t_2 (m)
ΔH_s = secondary waste settlement (m)
h = height of slope (m), as defined in Figure 6-4
h_a = relative humidity of the air (dimensionless)
h_{avg} = average hydraulic head (m)
h_m = maximum head in drainage layer (m)
h_u = height of slope above the slope grade break (m), as illustrated in Figure 6-6
h_r = relative humidity at the soil surface (dimensionless)
h_z^* = minimum head at which flow into the coarser-grained layer first occurs (m)
I = infiltration into surface cover soil (mm/day)
i = hydraulic gradient (dimensionless)
i_r = rainfall intensity (m/s)
K = soil erodility factor (dimensionless)
K' = soil roughness factor (dimensionless)
k = hydraulic conductivity (m/s)

k	=	methane generation rate constant (yr^{-1}) (in Chapter 5)
k_{cs}	=	cover soil saturated hydraulic conductivity (m/s)
k_d	=	drainage layer hydraulic conductivity (m/s)
k_{ds}	=	granular drainage layer hydraulic conductivity (m/s)
k_f	=	long-term field hydraulic conductivity of granular drainage layer (m/s)
k_g	=	gas conductivity (m/s)
k_h	=	pseudo-static seismic coefficient (dimensionless)
k_l	=	laboratory hydraulic conductivity of granular drainage layer (m/s)
k_n	=	cross-plane hydraulic conductivity of geotextile (m/s)
k_s	=	saturated hydraulic conductivity (m/s)
k_u	=	unsaturated hydraulic conductivity (m/s)
k_y	=	pseudo-static seismic coefficient that produces a psuedo-static slope stability FS of 1.0 (dimensionless)
$k_y g$	=	yield acceleration (m/s^2)
L	=	lateral drainage (mm/day)
L_d	=	length of drainage layer flow path (m)
L_f	=	length of overland flow path (m)
L_{fg}	=	thickness of finer-grained soil layer (m)
L_0	=	methane generation potential (m^3/Mg)
LS	=	slope length and steepness factor (dimensionless)
l	=	slope length (m)
M_i	=	mass of solid waste in the i^{th} section (Mg)
M_W	=	earthquake moment magnitude (dimensionless)
n	=	Manning's roughness coefficient for the considered vegetative cover (dimensionless)
n_p	=	porosity of gravel or riprap layer (dimensionless)
n_s	=	Manning's roughness coefficient for the bare soil (dimensionless)
0_{95}	=	the 95% opening size of the geotextile (mm)
P_a	=	vapor pressure in the air above the evaporating surface (Pa)
P_c	=	conservation support practice factor (dimensionless)
P	=	precipitation (mm/day)
PERC	=	percolation through the cover system (mm/day)
PERC*	=	percolation through the cover soil (mm/day)
PET	=	potential evapotranspiration (mm/day)
Q(x)	=	mass transport of soil at downwind distance x (kg/m)
Q_M	=	maximum expected gas generation flow rate (Mg/yr)
Q_{max}	=	mass transport of soil (kg/m)
$Q(x)_{max}$	=	maximum mass transport of soil at downwind distance x (kg/m)
Q	=	peak rate of runoff (m^3/s/m)
q_c	=	flow capacity of drainage layer (m^3/s/m)
q_m	=	maximum flow rate in drainage layer (m^3/s/m)
R	=	runoff (mm/day)
R_e	=	rainfall energy/erosivity factor (dimensionless)
R_f	=	permissible velocity reduction factor (dimensionless)
R_n	=	net radiant energy available at the surface (mm/day)

S	=	slope inclination (dimensionless)
$S(z,t)$	=	Sink term representing uptake by transpiration (s^{-1})
SCF	=	soil crust factor (dimensionless)
S_r	=	retention parameter (mm/day)
$s(x)$	=	field length scale (m)
T	=	geosynthetic tension above the potential slip surface (N/m)
t	=	thickness of material above the critical potential slip surface (m) (in Chapter 6)
t	=	time (s) (in Chapter 4)
t_a	=	thickness of soil layer at point A (m), as defined in Figure 6-5
t_{avg}	=	average thickness of soil layer between points A and B, which are defined in Figure 6-5 (m)
t_b	=	thickness of soil layer at point B (m), as defined in Figure 6-5
t_c	=	time of concentration (s)
t_d	=	drainage layer thickness (m)
t_{ds}	=	granular drainage layer thickness (m)
t_i	=	age of the i^{th} section (yr)
t_m	=	required thickness of the internal drainage layer (m)
t_w	=	thickness of water flow parallel to the slope (m), as defined in Figure 6-3
t^*_w	=	thickness of water in Wedge 1 (m), as defined in Figure 6-4;
t_1	=	starting time for the period of secondary compression (s)
t_2	=	t_1 plus the time duration of secondary compression or intermediate secondary compression (s)
t_3	=	T_2 plus the time duration of long-term secondary compression (s)
U_a	=	wind speed (km/hr)
v	=	flow velocity (m/s)
v_s	=	shear wave velocity of material (m/s)
$v_{s,\ waste}$	=	shear wave velocity of waste (m/s)
W	=	mass of the falling weight (tonne)
W_b	=	buoyant unit weight (N)
WF	=	weather factor (kg/m)
x	=	downwind distance (m)
x	=	cover thickness (m) (in Chapter 5)
x_c	=	critical distance along a slope before gully formation begins (m)
z	=	vertical coordinate (m)
Γ	=	slope of the saturation vapor pressure versus temperature curve at the mean temperature of the air (dimensionless)
Ψ	=	geotextile permittivity (s-1)
ψ	=	matric potential (negative) due to capillary suction forces (N/m2)
α	=	empirical constant $(m/tonne)^{0.5}$
β	=	slope angle (degrees)
γ_b	=	average buoyant unit weight of material above the critical potential slip surface (N/m^3)
γ_{sat}	=	average saturated unit weight of material above the critical potential slip surface (N/m^3)

γ_t	=	total unit weight of material above the critical potential slip surface or total unit weight of material (N/m^3)
$\gamma_{t, waste}$	=	total unit weight of waste (N/m^3)
γ_w	=	unit weight of water (N/m^3)
Δ	=	differential settlement (m)
$\Delta W_{foliage}$	=	change in water storage on plant foliage (mm/day)
ΔW_{soil}	=	change in water storage in cover system soil (mm/day)
$\Delta W_{surface}$	=	change in water storage at surface (mm/day)
δ	=	shear displacement (m)
θ	=	soil volumetric moisture content (dimensionless)
θ_a	=	air transmissivity ($m^3/s/m$)
θ_{afc}	=	soil apparent field capacity (dimensionless)
θ_{allow}	=	allowable hydraulic transmissivity of geosynthetic drainage layer ($m^3/s/m$)
θ_{dg}	=	geosynthetic drainage layer transmissivity ($m^3/s/m$)
θ_{ds}	=	granular drainage layer transmissivity ($m^3/s/m$)
θ_{fc}	=	soil field capacity (dimensionless)
θ_h	=	hydraulic transmissivity ($m^3/s/m$)
θ_{sc}	=	soil water storage capacity (dimensionless)
θ_{ult}	=	ultimate hydraulic transmissivity of geosynthetic drainage layer ($m^3/s/m$)
θ_{wp}	=	soil wilting point (dimensionless)
λ	=	pore size distribution index (dimensionless)
μ_a	=	air viscosity (kg/m/s)
μ_w	=	water viscosity (kg/m/s)
ρ_a	=	air density (kg/m^3)
ρ_w	=	water density (kg/m^3)
σ_n	=	normal stress (kPa)
τ	=	shear stress (Pa)
$\tau_{\mathbf{a}}$	=	allowable shear stress (kPa)
$\tau_{\mathbf{ab}}$	=	allowable shear stress for the surface layer with bare soil (kPa)
$\tau_{\mathbf{ah}}$	=	allowable shear stress for the Horton/NRC method (kPa)
$\tau_{\mathbf{e}}$	=	effective shear stress applied to the surface layer by the flowing water (kPa)
ν	=	psychrometric constant (dimensionless)
ϕ	=	angle of repose or gravel or riprap (degrees)
ϕ_i	=	angle of internal or interface friction for the critical potential slip surface (degrees)
ϕ_a	=	angle of internal or interface friction for the critical potential slip surface above the hydraulic barrier (degrees)
ϕ_b	=	angle of internal or interface friction for the critical potential slip surface below the hydraulic barrier (degrees)
ϕ_s	=	angle of internal friction for the soil material (i.e., protection layer and/or granular drainage layer) above the critical potential slip surface (degrees)

ϕ_{si}	=	secant angle of internal or interface friction for the critical potential slip surface (degrees), as defined in Figure 6-8
ϕ_{ti}	=	tangent angle of internal or interface friction for the critical potential slip surface (degrees), as defined in Figure 6-8

Chapter 1
Introduction

1.1 Overview

1.1.1 Purpose

The guidance provided in this document is designed to be used as a tool to provide design information to facility owners/operators, engineers, and regulators regarding cover systems for municipal solid waste (MSW) and hazardous waste (HW) landfills being remediated under the Comprehensive Environmental Response, Compensation and Liability Act (CERCLA), Resource Conservation and Recovery Act (RCRA) Corrective Action, and sites regulated under the RCRA. For sites at MSW and industrial waste (IW) facilities that are subject to State permits, the technical information contained in this document may be used to supplement existing guidance in order to achieve compliance with those permits (EPA, 1993; and EPA, 2003). This guidance document provides an update to the previous U.S. Environmental Protection Agency (EPA) guidance on this subject "*Design and Construction of RCRA/CERCLA Final Covers*" (EPA, 1991a).

In comparison to the scope of the 1991 EPA cover system guidance document, the scope of this document has been expanded to address a number of new topics including design criteria development, new types of geosynthetics (such as geosynthetic clay liners (GCLs)), alternative materials and designs (including evapotranspiration (ET) barriers and capillary barriers), special design issues, lessons learned from the closure of existing landfills, performance monitoring of cover systems, maintenance of cover systems to achieve the required design life, and site end use. Significant advances in the technology for cover system design and construction have occurred since 1991. These advances are reflected in this document.

Final cover systems (hereafter referred to as "cover systems") are used at landfills and other types of waste management units (e.g., waste piles and surface impoundments) to contain waste and any waste by-products (e.g., leachate or landfill gas), control moisture and air infiltration into the waste, and prevent the occurrence of odors, disease vectors, and other nuisances. Cover systems are also used to meet erosion, aesthetic, and other post-closure site end use criteria for waste management sites. These systems are intended to achieve their functional requirements for time periods of many decades to hundreds of years.

As illustrated by Figure 1-1, cover systems form one component of the integrated group of engineered systems used at landfills to protect human health and the environment. Other components include liners, daily and intermediate covers, leachate collection and removal systems, gas collection and removal systems, and surface-water management systems.

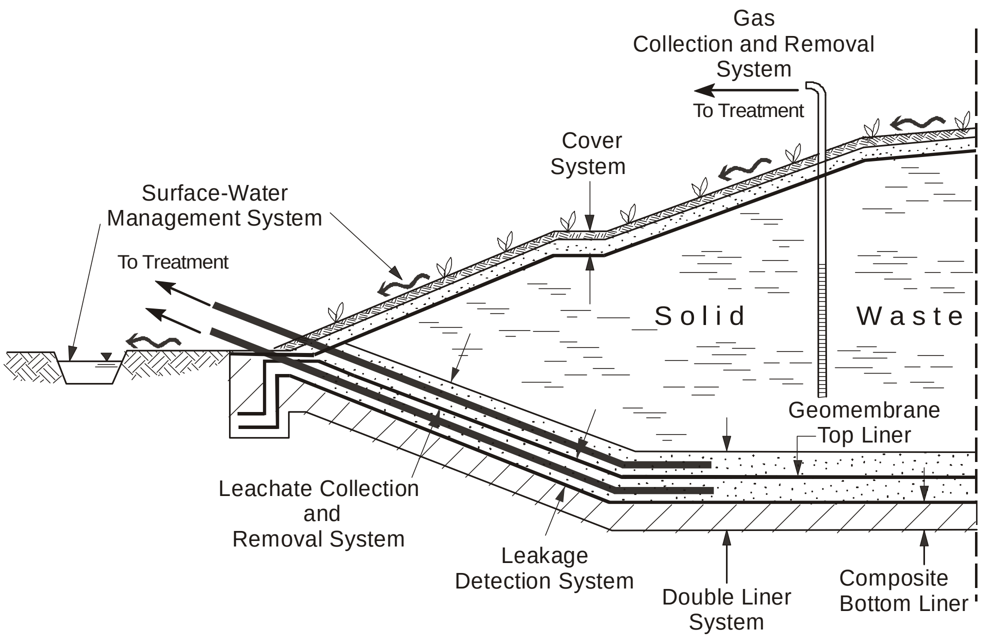


Figure 1-1. Example of Engineered Systems Used at Landfills.

Cover systems are also placed over old dumps as part of the remediation and final closure of these facilities and over contamination source areas that can be at the ground surface or at shallow depth. When used for these applications, the cover system may again be one component of an integrated group of engineered systems used for facility closure or source containment (Figure 1-2). The cover system components for these facilities are often similar to the components used to close new landfills. However, as discussed subsequently in this document, some of the design issues faced in closing dumps or in implementing source containment remedies at contaminated sites are different from the design issues faced in closing new landfills.

The cover system itself can consist of multiple layers of different types of soils and/or geosynthetics, each with one or more specific functions. The cover system components are briefly introduced in Section 1.5 and discussed in more detail in Chapter 2. Although gas management issues are discussed in Section 1.4 and Chapter 5 of this document, the information provided on regulatory requirements for MSW landfills under the Clean Air Act (CAA) is cursory and is not the intended use of this guidance document.

The waste to be contained can be municipal solid waste (MSW), hazardous waste (HW), low-level radioactive waste, industrial waste (IW), remediation waste, incinerator or coal-combustion ash, construction and demolition waste (C&DW), sewage treatment or industrial process sludge, or some other material. The cover system is installed on top of the waste shortly after a specific landfill cell or unit has been filled to capacity in the case of a new landfill, at the time of site remediation and closure in the case of an old dump, or at the time of site remediation in the case of a contaminated site.

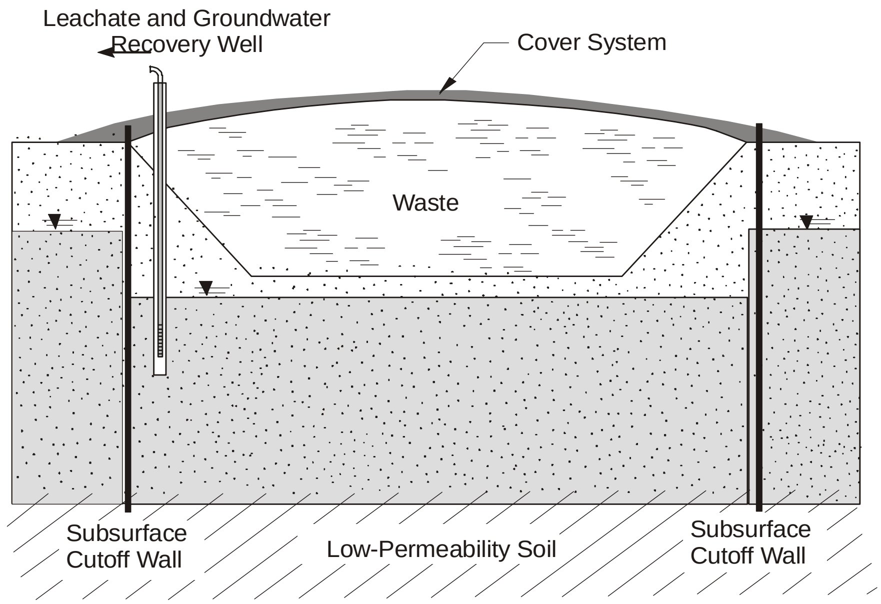


Figure 1-2. Example of Engineered Systems Used at Old Dumps or Contamination Source Areas.

1.1.2 Classification of Cover Systems

At present, cover system designs are based on one or more of three different principles for preventing or minimizing water percolation into waste. Each of these is briefly discussed below.

Hydraulic Barrier: This type of cover system uses a low-permeability physical barrier to impede the downward migration of water into the waste (Figure 1-3). Hydraulic barrier materials most commonly include compacted clay liners (CCLs), GCLs, geomembranes (GMs), and combinations of these materials. Other barrier materials (e.g. asphaltic concrete) have also been used. A hydraulic barrier is generally used with additional cover system components. However, recently, at a few MSW landfill sites, a GM barrier was used alone as a cover system (Gleason et al., 1998, 1999, 2001). In many cases and especially on sideslopes, an internal drainage layer is included above the hydraulic barrier to drain the overlying layers, promote lateral drainage, and prevent the buildup of hydraulic head in the cover system. A surface/protection layer is often installed as the topmost layer to protect the hydraulic barrier from erosion, exposure to wet-dry cycles, exposure to freeze-thaw cycles, biointrusion (intrusion by plant roots, burrowing animals, and humans), and ultraviolet degradation and for temporary storage of infiltrating water for subsequent uptake by vegetation, if present. Water movement through cover systems with hydraulic barriers can occur as either saturated or unsaturated flow, depending on site-specific conditions (particularly climate). Current EPA regulations and existing requirements for cover systems at landfills are developed around the use of hydraulic barriers.

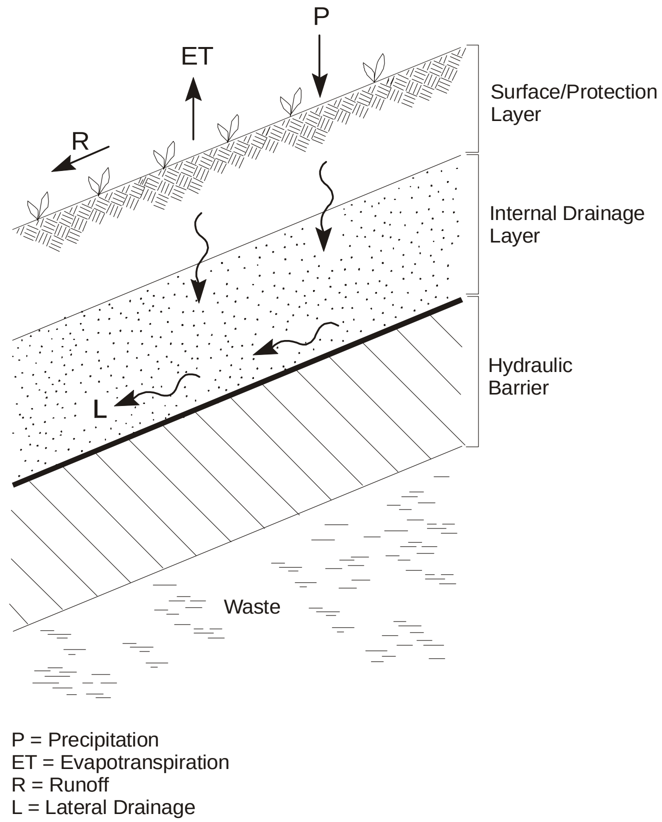


Figure 1-3. Hydraulic Barrier Type of Cover System.

ET Barrier: This type of cover system has been developed for use at arid and semi-arid sites. ET barriers consist of a thick layer of relatively fine-grained soil capable of supporting vegetation (Figure 1-4). Soil types used for construction of ET barriers include silty sands, silts, and clayey silts. ET barriers exploit two characteristics of fine-grained soils: (i) high water storage capacity (i.e., they can store a significant amount of water before gravity drainage: they have a high field capacity); and (ii) low hydraulic conductivity, even at high degrees of saturation. High soil water storage capacity allows storage of water that infiltrates the barrier until it can later be removed by ET. Low hydraulic conductivity limits advancement of the wetting front into the barrier during seasonal wet periods (rainfall or snow melt). An ET barrier should be sufficiently thick such that the soil water content does not increase near the base of the barrier; all changes in soil water storage should occur in the upper portion of the barrier (Figure 1-4). Otherwise, percolation through the cover system can occur. The required barrier thickness is a function of the frequency and intensity of precipitation, the unsaturated hydraulic properties of the soil, the type and vigor of vegetative cover, the rate at which water can be removed by ET, and other factors. Barrier thickness typically ranges from about 0.9 m to more than 2 m. ET barriers often have a surface layer to support vegetation and provide erosion protection.

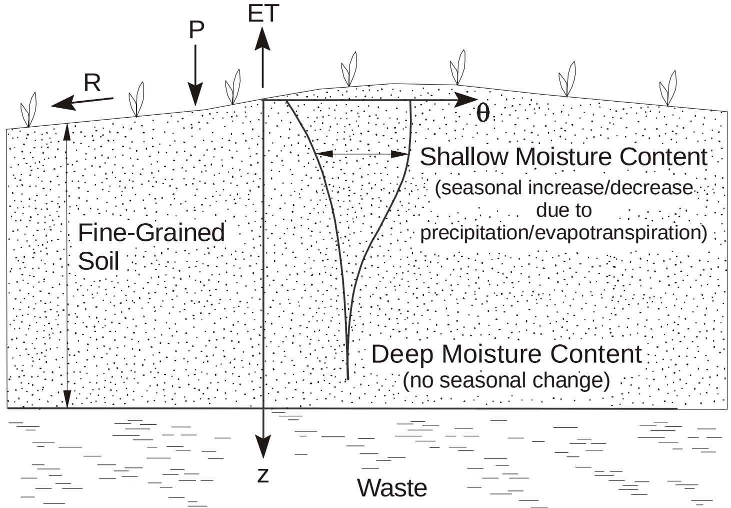


Figure 1-4. ET Barrier Type of Cover System and Representative Soil Moisture Content vs. Depth Profile.

Capillary Barrier: This type of cover system has also been developed for use at arid and semi-arid sites. Capillary barriers consist of one or more layers of finer-grained soil overlying one or more layers of coarser-grained soil. The finer-grained soil layer of the capillary barrier has a higher water storage capacity than a comparable layer at the same depth without the capillary break (i.e., a free-draining layer such as an ET barrier). Figure 1-5 illustrates the simplest configuration for a capillary barrier: a single finer-grained (e.g., clayey silt) soil layer over a coarser-grained (e.g., sandy) soil layer. At low degrees of soil saturation (i.e., at low matric potential in Figure 1-5), the hydraulic conductivity of the coarser-grained soil is much less than that of the finer-grained soil. This is the reverse of the condition that occurs at high degrees of soil saturation. Capillary barriers store infiltrating water in the finer-grained soil until the water can be removed by subsequent ET. If they are sloped, capillary barriers can also divert infiltrating water via unsaturated lateral flow in the finer-grained soil (above the soil interface). Sometimes a "wicking layer" (with intermediate characteristics to the coarser- and finer-grained layers) is installed between the coarser- and finer-grained layers to convey lateral flow. At high degrees of soil saturation (e.g., in a humid climate), the capillary effect breaks down and percolation through the cover system can occur. Like ET barriers, capillary barriers often have a surface layer to support vegetation and provide erosion protection.

This guidance document focuses primarily on the hydraulic barrier type of cover system with limited commentary on the other two types provided mainly in Chapter 3. It is noted, however, that the use of ET and capillary barrier types of cover systems is becoming more common, particularly in arid and semi-arid regions of the U.S. While these alternative designs can be adequate for hydraulic control, they should generally not be used without gas containment components at MSW landfill sites where landfill gas collection control are needed to prevent offsite gas migration and reduce emissions that are of concern to human health and the environment.

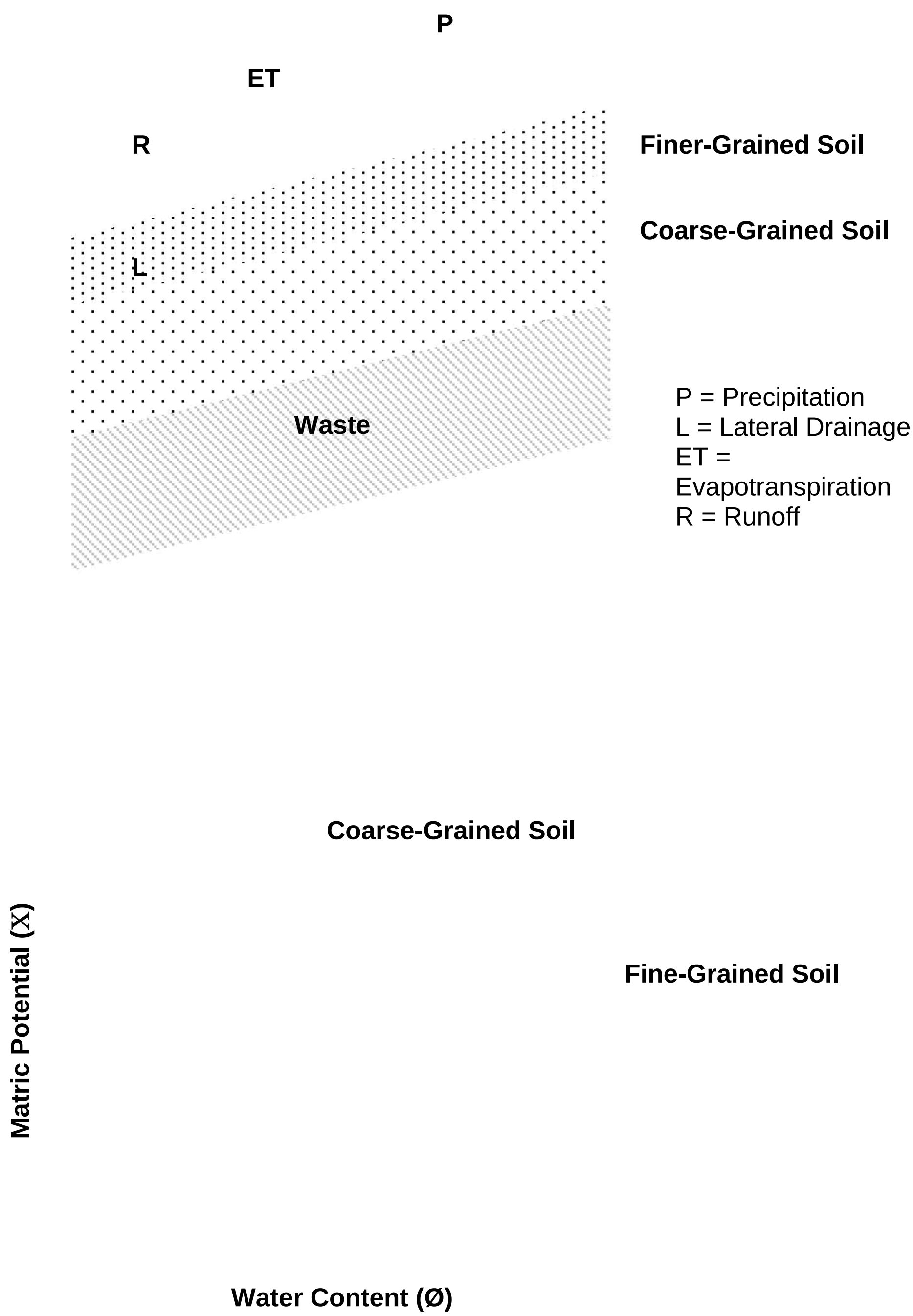


Figure 1-5. Capillary Barrier Type of Cover System and Representative Unsaturated Hydraulic Conductivity Functions.

1.1.3 Organization of Document

This document is organized into the following sections:

- List of Acronyms (page viii);
- List of Variables and Units (page xi);
- Introduction (Chapter 1);
- Individual Components of Cover Systems (Chapter 2);
- Alternative Design Concepts and Materials (Chapter 3);
- Hydraulic Analysis and Design (Chapter 4);
- Gas Emission Analysis and Collection System Design (Chapter 5);
- Geotechnical Analysis and Design (Chapter 6);
- Lessons Learned (Chapter 7);
- Performance Monitoring (Chapter 8);
- Post-Closure Maintenance and Site End Use (Chapter 9); and
- References (Appendix A).

1.2 Closure Regulatory Requirements

A starting point in understanding closure requirements for landfills or source area containment for contaminated sites is to become familiar with the regulations governing the landfill or environmental remediation project. Although generally well understood, the Federal regulations applicable to cover systems for RCRA and CERCLA projects are briefly reviewed in this section of the guidance document.

1.2.1 MSW Landfill Cover Systems

Minimum technical requirements for closure of MSW landfills (MSWLFs) regulated under RCRA Subtitle D are contained in Title 40 of the Code of Federal Regulations, Section 258.60 (40 CFR §258.60). The regulation allows either a minimum criteria cover system or a performance-based cover system design. The specific requirements of that regulation are as follows:

> "*(a) Owners or operators of all MSWLF units must install a final cover system that is designed to minimize infiltration and erosion. The final cover system must be designed and constructed to:*
> *(1) Have a permeability less than or equal to the permeability of any bottom liner system or natural subsoils present, or a permeability no greater than 1 x* 10^{-5} *cm/sec, whichever is less, and*
> *(2) Minimize infiltration through the closed MSWLF by the use of an infiltration layer that contains a minimum 18-inches of earthen material, and*
> *(3) Minimize erosion of the final cover by the use of an erosion layer that contains a minimum 6-inches of earthen material that is capable of sustaining native plant growth.*

(b) The Director of an approved State may approve an alternative final cover design that includes:
(1) An infiltration layer that achieves an equivalent reduction in infiltration as the infiltration layer specified in paragraphs (a)(1) and (a)(2) of this section, and
(2) An erosion layer that provides equivalent protection from wind and water erosion as the erosion layer specified in paragraph (a)(3) of this section."

After the foregoing regulations were issued in October 1991, EPA clarified their intent with respect to the permeability requirement of the prescriptive minimum criteria cover system in 40 CFR §258.60(a)(1). The Agency's clarification was contained in the Federal Register in June 1992, at 57 FR 28628 (EPA, 1992b). According to this clarification, the cover system is required to have a hydraulic conductivity less than or equal to that of any underlying liner system or natural subsoils. The purpose of this requirement is to prevent what the Agency calls the "bathtub" effect, wherein percolation into the landfill exceeds leakage through the liner system, causing the accumulation of liquid in the facility. The hydraulic conductivity must also be no greater than 1×10^{-7} m/s.

The EPA (1992b) clarification to the minimum requirements for MSW landfill cover systems is illustrated in Figure 1-6 for: (i) unlined landfills constructed prior to the effective date of Subtitle D regulations (Figure 1-6(a)); (ii) landfills with a CCL beneath the waste (Figure 1-6(b)); and (iii) landfills underlain by a Subtitle D composite liner consisting of a GM upper component and a CCL lower component (with the CCL having a maximum hydraulic conductivity of 1×10^{-9} m/s) (Figure 1-6(c)). While these minimum requirements seem to indicate that less protective cover systems are allowed at landfills with less protective liner systems, EPA believes that, all other factors being equal (e.g., comparable hydrogeologic setting, types of waste, etc.), more protective cover systems should be used at unlined landfills compared to lined landfills to minimize the percolation of water though the cover systems and, consequently, the formation of leachate and migration of such leachate from the units.

It should also be noted that the cover systems required by 40 CFR §258.60 regulations do not represent "complete" designs in the sense that they are based on a permeability design criterion only and do not address other design criteria. For example, the cover system shown in Figure 1-6(c) does not include a drainage layer above the GM barrier or an adequate thickness of cover soil to allow sufficient water storage for healthy surface vegetation. As another example, none of the designs presented in Figure 1-6 have an adequate thickness of soil protection above the CCL component of the cover system to protect the CCL from freeze-thaw damage for sites located in northern climates. As a final example, none of the designs addresses the important matter of landfill gas transmission beneath the cover system.

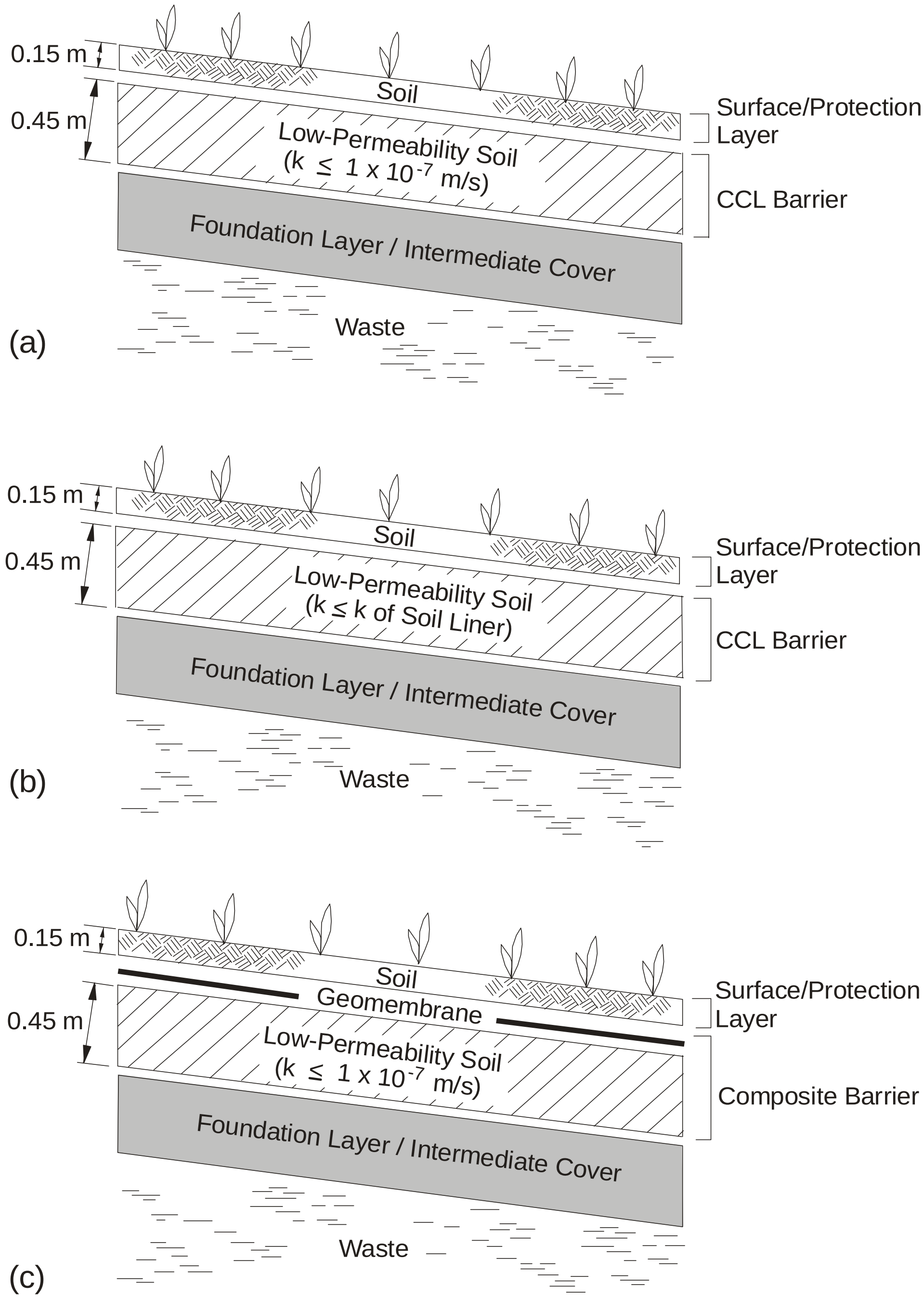


Figure 1-6. EPA Prescriptive Minimum Criteria Cover Systems for: (a) Unlined MSW Landfills; (b) MSW Landfills Underlain by a CCL; and (c) MSW Landfills Underlain by a GM/CCL Composite Liner.

1.2.2 Hazardous Waste Landfill Cover Systems

Minimum technical requirements for closure of permitted HW landfills regulated under Subtitle C of RCRA are contained in 40 CFR §264.310. Analogous requirements for interim status HW landfills are contained in 40 CFR §265.310. These regulations allow a performance-based cover system design; no prescriptive design criteria are provided for HW landfills. The specific requirements of the regulations for permitted landfills are given below:

> "*(a) At final closure of the landfill or upon closure of any cell, the owner or operator must cover the landfill or cell with a final cover designed and constructed to:*
> *(1) Provide long-term minimization of migration of liquids through the closed landfill;*
> *(2) Function with minimum maintenance;*
> *(3) Promote drainage and minimize erosion or abrasion of the cover;*
> *(4) Accommodate settling and subsidence so that the cover's integrity is maintained; and*
> *(5) Have a permeability less than or equal to the permeability of any bottom liner system or natural subsoils present.*"

Cover system requirements for interim status HW landfills (40 CFR §265.310) differ in several details from those given above for permitted facilities. EPA (1991a) and Koerner and Daniel (1997) discussed these differences. EPA generally recommends, however, that cover systems for interim status HW landfills be designed to the same standards as permitted facilities.

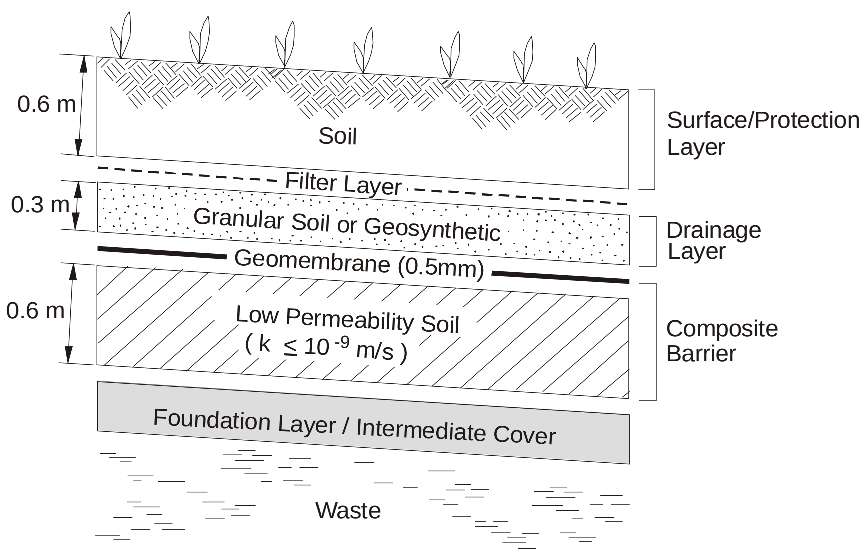


Figure 1-7. EPA (1989) Recommended Minimum Cover System for HW Landfills.

EPA previously issued minimum technology guidance for cover systems that meet the regulatory requirements of 40 CFR §264.310 (EPA, 1989). The cover system for HW landfills recommended in the 1989 EPA guidance consists of (Figure 1-7):

- a top layer containing two components: (i) either a vegetated or armored surface layer, selected to minimize erosion and, to the extent possible, promote drainage off the cover; and (ii) a protection layer, comprising topsoil and/or fill soil, as appropriate; the recommended top layer surface slope is 3 to 5%; the 1989 EPA guidance noted that the top layer soil component should be at least 0.6-m thick, and that a greater thickness may be required to assure that the underlying hydraulic barrier is below the frost zone;
- a soil drainage layer with minimum thickness of 0.3 m and a minimum hydraulic conductivity of 1×10^{-4} m/s that will effectively "*minimize water infiltration into the underlying low-permeability barrier*" and have a final slope of at least 3% after settlement and subsidence or a drainage layer consisting of a geosynthetic material with performance characteristics equivalent to the soil drainage layer; and
- a composite hydraulic barrier consisting of: (i) a GM with a minimum thickness of 0.5 mm; and (ii) a CCL with a minimum thickness of 0.6 m and a maximum hydraulic conductivity of 1×10^{-9} m/s; the EPA guidance notes that the entire hydraulic barrier should lie below the frost zone.

The 1989 EPA guidance indicated that optional layers may be used on a site-specific basis. According to the 1989 guidance, optional layers may include a gas collection layer placed below the hydraulic barrier, a biotic barrier component of the protection layer, and geosynthetic or soil filter layers. All of these types of materials are discussed in more detail in Section 1.5 and Chapter 2 of this document. The 1989 guidance also discussed the use of alternative designs. This subject too is discussed in Section 1.3 and Chapter 3 of this document. It is also reiterated that the 1989 document provides guidance on minimum design criteria. On a case-by-case basis, it may be necessary to provide additional components or capability to a HW landfill design. For example, it may be necessary to specify a drainage layer hydraulic conductivity greater than 1×10^{-4} m/s to assure no unacceptable build-up of hydraulic head in the cover system. As another example, the thickness of the protection layer may need to be greater than 0.6 m to adequately protect the hydraulic barrier component from freezing weather impacts in some northern climates.

1.2.3 Solid Waste Landfill Cover System Performance

Both the MSW and HW landfill regulations cited above specify as a performance criterion minimization of water percolation into the waste (or, equivalently, minimization of liquids migration through the landfill by preventing the bathtub effect). EPA is not yet recommending a design percolation rate for landfill cover systems.

1.2.4 CERCLA Site Cover Systems

The blueprint for remediation of CERCLA sites is contained in the National Contingency Plan (NCP) of 1990 and the Superfund Amendments and Reauthorization Act (SARA) of 1986. Remediation of these sites often involves installation of a cover system as part of a source control remedy for a landfill, waste pile or pit, or heavily contaminated area. EPA (1997a) reported that containment technologies, which typically include some form of cover system, have been used for approximately 40% of the source control remedies implemented through 1995 at CERCLA sites.

Design requirements for cover systems at CERCLA sites are generally based on the attainment of applicable or relevant and appropriate requirements (ARARs). ARARs for cover systems may include RCRA Subtitle C or Subtitle D regulations. EPA (1991a) provides a detailed discussion of ARARs in the context of CERCLA cover systems.

CERCLA MSW landfills represent a particular subset of CERCLA sites addressed by EPA's presumptive remedy guidance (EPA, 1993). CERCLA MSW landfills typically contain a combination of principally MSW and, to a lesser extent, wastes containing hazardous substances. CERCLA MSW landfills represent approximately 20% of the total number of CERCLA sites in the United States (EPA, 1991b). The Agency has developed some presumptive remedies using preferred technologies for common categories of sites, based on historical patterns of remedy selection for those categories of sites and EPA's scientific and engineering evaluation of performance data on technology implementation. For CERCLA MSW landfill sites, EPA generally considers containment as the presumptive remedy (EPA, 1993). Furthermore, the Agency has identified cover systems as a component of the source containment presumptive remedy. EPA (1993) provided the following guidance regarding ARARs for CERCLA MSW landfill presumptive remedies:

> "*In the absence of Federal Subtitle D closure regulations, State Subtitle D closure requirements generally have governed CERCLA response actions at municipal landfills as applicable or relevant and appropriate requirements (ARARs). New Federal Subtitle D closure and post-closure care regulations will be in effect on October 9, 1993 (56 FR 50978 and 40 CFR* §*258). State closure requirements that are ARARs and that are more stringent than the Federal requirements must be attained or waived.*
>
> *The new Federal regulations contain requirements related to construction and maintenance of the final cover, and leachate collection, ground-water monitoring, and gas monitoring systems. The final cover regulations will be applicable requirements for landfills that received household waste after October 9, 1991. EPA expects that the final cover requirements will be applicable to few, if any, CERCLA municipal landfills, since the receipt of household wastes ceased at most CERCLA landfills before October 1991. Rather, the substantive requirements of the new Subtitle D regulations generally will be considered relevant and appropriate requirements for CERCLA response actions that occur after the effective date.*"

"RCRA Subtitle C closure requirements may be applicable or relevant and appropriate in certain circumstances. RCRA Subtitle C is applicable if the landfill received waste that is a listed or characteristic waste under RCRA, and:

1. *The waste was disposed of after November 19, 1980 (effective date of RCRA), or*
2. *The new response action constitutes disposal under RCRA).*

The decision about whether a Subtitle C closure requirement is relevant and appropriate is based on a variety of factors, including the nature of the waste and its hazardous properties, the date on which it was disposed, and the nature of the requirement itself. For more information on RCRA Subtitle C closure requirements, see RCRA ARARs: Focus on Closure Requirements, Directive No. 9234.2-04FS, October 1989."

The decision of whether MSW or HW landfill cover system requirements are relevant and appropriate also depends on the level of cover system hydraulic performance that is necessary to achieve human or ecological receptor exposure point concentrations that produce acceptable post-remediation human health and ecological risk estimates.

1.2.5 Liquids Management Strategy

EPA policies and regulations for landfill cover systems have evolved within a framework originally described by the Agency as a "liquids management strategy." The two main goals of the strategy are: (i) minimizing leachate generation by keeping liquids out of the landfill (or source area for a CERCLA remediation); and (ii) detecting, collecting, and removing leachate as it is generated (EPA, 1991c, 1992a). With this liquids management strategy, keeping water out of the landfill (or source area) becomes a prime performance objective for the cover system. In fact, EPA has stated (EPA, 1989):

"Thus, the Agency believes that a properly designed and constructed cover becomes, after closure, the most important feature of the landfill structure. The Agency requires that the cover be designed and constructed to provide long-term minimization of the movement of water from the surface into the closed unit. Where the waste mass lies entirely above the zone of ground-water saturation, a properly designed and maintained cover can prevent, for all practical purposes, the entry of water into the closed unit, and thus minimize the formation and migration of leachate."

Figure 1-8 illustrates the benefits of cover system installation in reducing leachate generation rates. This figure shows leachate generation rates for a GM-lined MSW landfill cell through the period of active cell operation, the closure period, and the first few years of the post-closure period. The landfill site is located in Pennsylvania and receives approximately 1,000 mm of precipitation annually, on average (Bonaparte, 1995). The landfill cover system includes a GM barrier. Monthly average leachate generation rates during the period of cell filling were up to 3,400 lphd. Rates for the first three years of the post-closure period were only 70 lphd. The very significant effect of cover system installation on the rate of leachate generation is apparent. Figure 1-9 from Othman et al. (2002) shows similar behavior for a group of MSW and HW landfill cells that have a cover system that includes a GM. On average, leachate generation rates typically decreased by a factor of four within one year after closure and by one order of magnitude within two to four years after closure. Six years after closure, leachate generation rates were between 5 and 1,200 lphd (mean of 180 lphd). Nine years after closure, leachate generation rates were negligible. These data show that well designed and constructed cover systems can be effective in reducing leachate generation rates to very low or near zero values.

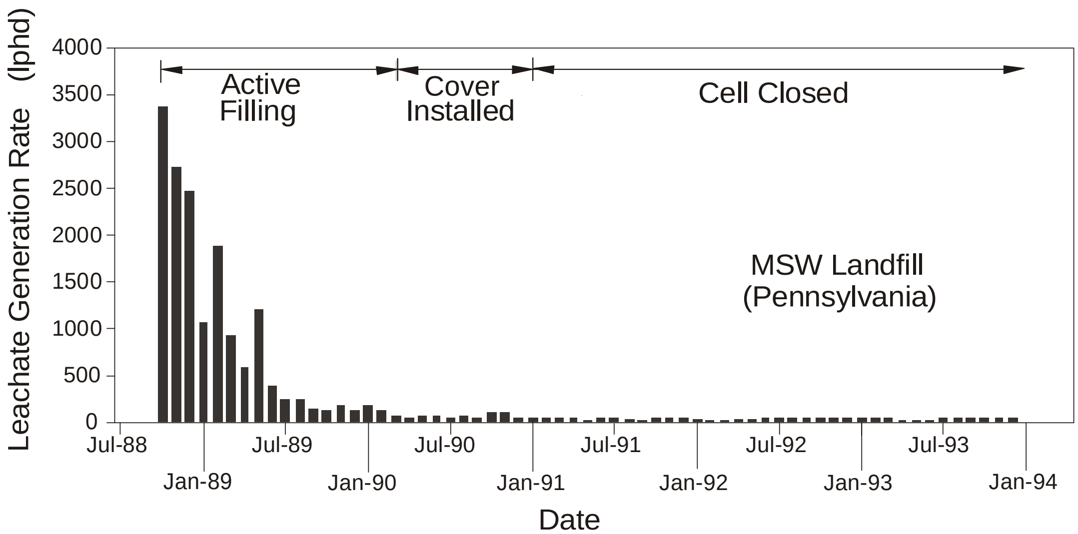


Figure 1-8. Leachate Generation Rates Over Time at a MSW Landfill in Pennsylvania (from Bonaparte, 1995). (Flow rates are in liters/hectare/day (lphd).)

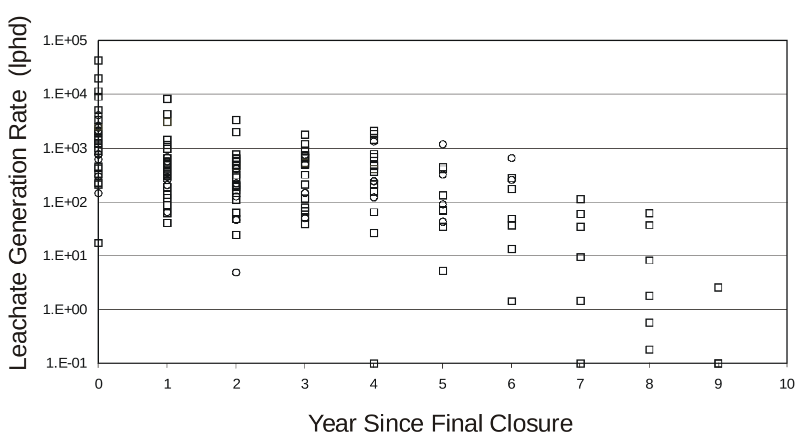


Figure 1-9. Effect of Cover system Installation on Leachate Generation Rates for 12 MSW Landfill Cells (shown as circles) and 22 HW Landfill Cells (shown as squares) (from Othman et al., 2002). (Note: flow rates of 0 lphd are shown as 0.1 lphd on this figure.)

1.2.6 Design Life

Consistent with the Agency's liquids management strategy, discussed above, the design life goal for RCRA and CERCLA cover systems is to minimize infiltration into the waste for as long as the enclosed waste poses an unacceptable risk to human health and the environment. A distinction should be made between the minimum post-closure care period of 30 years given in RCRA regulations and the design life of the cover system. The latter is much longer than 30 years and is defined primarily by the service life characteristics of the material used to construct the cover system. The service life of CCLs protected from freeze-thaw and other environmental effects, and not subjected to excessive differential settlements, should be indefinitely long (Mitchell and Jaber, 1990). The service life of any GM component of the cover system is dependent on the specific material used and how well the material is protected. The most extensive service life data currently available are for high density polyethylene (HDPE) GMs. The data indicate that the service life for commercially-available HDPE GMs will be measured in terms of at least several hundred years (Hsuan and Koerner, 1998; Hsuan and Koerner, 2002). Other types of GMs may have different service lives from that for HDPE GMs. Great care should be used in specifying GM materials to require products that, through polymer type, additive (e.g., antioxidant) packages, physical robustness, etc., are capable of achieving as long a service life as possible.

Achieving a design life measured in terms of hundreds of years requires more than just the selection of durable materials of construction. The design itself should be developed to achieve the design life criteria. This involves developing a design with adequate slope stability factors of safety, adequate flow capacity for the internal drainage system, adequate surface-water runoff controls, adequate freeze-thaw protection, adequate resistance to surface erosion, and appropriate vegetation or other surface treatment. Many of these design topics are addressed in subsequent chapters of this document. Recognizing the dynamic nature of the ecosystem in which cover systems function, post-closure monitoring and maintenance are important elements in achieving the required design life. Long-term maintenance with respect to surface erosion, biointrusion, and plant succession (i.e., grasses to shrubs to trees) are particularly important issues in addressing the design life of a cover system. Monitoring of cover systems after closure is necessary to both satisfy regulatory requirements and assure the performance of the cover system. While performance monitoring is important for all closed facilities, it is particularly such for closed sites, such as old dumps and contamination source areas, and for sites with alternative cover systems. Monitoring of infiltration, soil moisture, gas emissions, and settlement is discussed in Chapter 8. The cover system should generally be inspected and maintained to assure adequate performance of the site in the long term and to comply with regulatory requirements. Cover system maintenance is discussed in Chapter 9.

1.2.7 Other Regulatory Requirements

In addition to the regulatory requirements cited above, other regulatory requirements may be ARARs to a landfill closure or CERCLA remediation project. These additional requirements must be considered on a case-by-case basis. It is essential for proper design and legal compliance of the project that all potentially applicable regulations be identified during the design criteria development phase of the project (see Section 1.6 of this document). Other potential regulatory requirements or ARARs may include:

- State-mandated cover system regulations that impose additional requirements beyond the minimum technical requirements of EPA;
- requirements imposed by other regulations for specific types of wastes, regulated under the Toxic Substances Control Act (40 CFR §700), such as polychlorinated biphenyls (PCBs), or Uranium Mill Tailings Remediation Act (40 CFR §192);
- State or Federal (including Federal Emergency Management Agency) requirements for site surface-water management, landfill gas management, seismic design, or other requirements that could influence the design of the cover system;
- provisions for management, treatment, and/or discharge of stormwater runoff, leachate, gas condensate, or other liquids under provisions of the Clean Water Act, including National Pollution Discharge Elimination System (NPDES) requirements (40 CFR §122) and proposed landfill point source effluent limitation guidelines (40 CFR §445);
- State requirements for maximum allowable soil erosion rates, erosion control structure design and performance, and surface-water management structure design and performance;
- Federal or State requirements for siting, including limitations on construction in floodplains, disturbance of wetlands, and construction on or near Holocene faults.

1.3 Alternative Design Concepts and Materials

RCRA regulatory requirements provide flexibility for innovation and alternatives by limiting the use of specific minimum design specifications as much as possible, by providing performance criteria in lieu of design specifications, and/or by providing administrative procedures for gaining approval of waivers from RCRA regulatory requirements. Also, under CERCLA §121(d), ARARs may be waived (refer to guidance).

EPA is open to considering alternative designs on a case-by-case basis. Determinations on the acceptability of alternative designs for HW landfills are the responsibility of the Regional Administrator. Statutory requirements must be satisfied by any approved alternative. This document provides guidance on several of the alternative design approaches and materials that the Agency may consider on a case-by-case basis. It is anticipated that new design approaches and materials expect to be considered by EPA in the future as the performance of these alternatives is demonstrated and proven. As an example of an alternative design, Region 1 of EPA has issued alternative minimum technology guidance for closure of unlined HW RCRA landfill sites in that region. The rationale and technical analyses supporting the Region 1 alternative minimum technology guidance is given in EPA (2000a). It is noted that, in Region 1, this type of landfill often has relatively steep sideslopes (i.e., greater than 6 horizontal:1 vertical (6H:1V)) and soils suitable to construct a hydraulic barrier may not be locally available. A comparison of the minimum technology guidance from EPA (1989) and EPA (2000a) is presented in Figure 1-10. Other types of alternative designs may involve ET or capillary barriers as discussed in Section 1.1 of this document. Alternative design concepts and materials are discussed in more detail in Chapter 3.

The minimum technical requirements for cover systems were developed by EPA to achieve the liquids management strategy goal previously described. These requirements still represent the Agency's preferred approach for most types of landfills under most situations. In recent years, however, the Agency has begun to consider other management strategies for landfill facilities. Potential strategies include, for example, landfill leachate recirculation and bioreactors (EPA, 1995). EPA believes that new landfill management strategies may lead to new alternative cover system designs and materials. The Agency is currently considering these types of alternatives on a case-by-case basis.

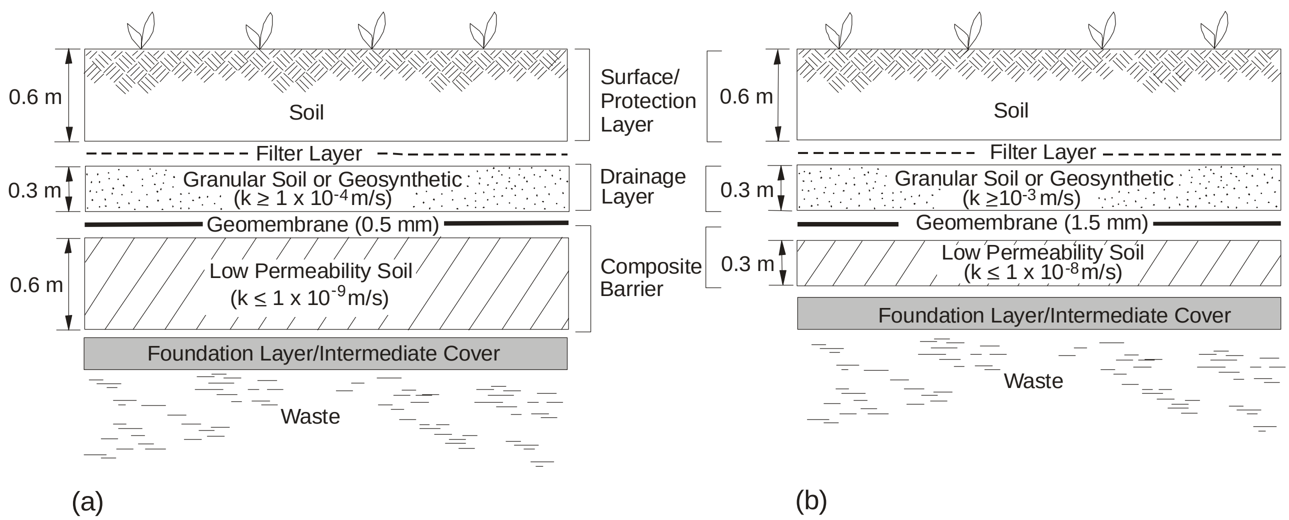


Figure 1-10. Comparison of Cover Systems for HW Landfills: (a) EPA (1989) Recommended Minimum Technology Cover System; and (b) Region 1 Alternative Minimum Cover System.

The use of monitored natural attenuation is recognized by EPA as a viable technique for remediation of soil and groundwater at certain sites (EPA, 1999a). The term “monitored natural attenuation” refers to the reliance on natural attenuation processes to achieve site-specific remedial objectives within a time frame that is reasonable, compared to that offered by other more active remediation methods. The “natural attenuation processes” that are at work in such a remediation approach include a variety of physical, chemical, and/or biological processes that, under favorable conditions, act without human intervention to reduce the mass, toxicity, mobility, volume, or concentration of contaminants in soil or groundwater. These in-situ processes include biodegradation, dispersion, dilution, sorption, volatilization, radioactive decay, and chemical or biological stabilization, transformation, or destruction. EPA is aware of situations where monitored natural attenuation has been proposed along with a permeable (e.g., granular) cover system as a source control remedy for a CERCLA landfill. Since this approach is not consistent with the Agency’s liquids management strategy, EPA will evaluate these cover systems very carefully on a case-by-case basis and, in some cases, will require that an in-situ treatment technology be used with this approach to complement natural attenuation and a demonstration of the technical practicability of the technology. As an example, this remediation approach is being used by EPA Region 1 for the Somersworth Sanitary Landfill Superfund Site. As outlined in the 1995 Consent Decree for the site, the Preferred Source Control Remedy includes:

- "*placement of a permeable cover over the landfill allowing precipitation to flush contamination from the waste management area. This cover will remain as long as contaminants continue to leach from the waste within the waste management area and the chemical treatment "wall" is functioning. After the Final Cleanup Levels have been achieved and can be maintained with use of the treatment "wall," an appropriate landfill cover to close the landfill that is consistent with the ROD* (Record of Decision) *shall be installed and maintained.*";
- "*installation of a treatment wall composed of impermeable barrier sections and permeable, chemical treatment sections to provide in-situ (in-place), flow-through treatment of contaminated ground water at the down-gradient edge of the waste management area.*" (the site and the pilot-scale treatment wall is described in EPA (1999b)); and
- "*enhancements … and additional source control measures, if necessary*".

The Consent Decree for this site places the burden of using the alternative source control remedy on the party implementing the remedy. If the Preferred Source Control Remedy does not meet the specified performance standards, a Contingent Source Control Remedy, including installation of a cover system that meets RCRA Subtitle C requirements and other ARARs, may need to be implemented.

1.4 Gas Management Requirements

Landfill gas collection and control is necessary at some MSW landfills, a limited number of HW landfills, and some CERCLA remediation sites. Most modern MSW landfills built to current regulatory standards have landfill gas collection and control systems. Some sites recover the gas for its energy potential, which may help to offset regulatory compliance costs. As of January 1999, there were about 300 MSW landfill gas-to-energy projects active in the U.S and several hundred more planned or in construction (Thorneloe, 2000).

Anaerobic decomposition of organic material in waste is the principal source of landfill gas and a significant cause of settlement of the waste mass. Some industrial wastes, however, can generate gas by inorganic chemical reactions. Gas production rates vary with the composition and age of waste, waste volume, waste moisture content, and other factors. MSW landfill gas consists mainly of methane and carbon dioxide, with lesser concentrations of nitrogen, oxygen, sulfides, ammonia, and other constituents, and trace concentrations of a variety of volatile organic compounds, including vinyl chloride, ethylbenzene, toluene, and benzene (Tchobanoglous, 1993). Landfill gas can be a significant threat to human health and the environment (EPA, 2000). Because of this, CAA regulations establish requirements for MSW landfill gas collection and control at certain facilities. Gas generation in a MSW landfill cell can extend over a period of 25 years or more, or gas generation can be accelerated through the use of leachate recirculation. An idealization of gas generation rates in MSW landfills is presented in Figure 1-11.

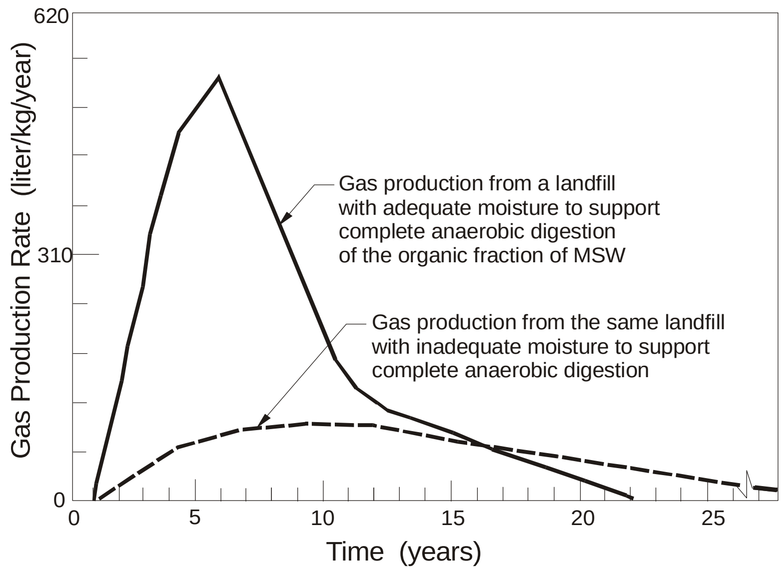


Figure 1-11. Idealization of Gas Generation Rates in MSW Landfills (from Tchobanoglous, 1993).

Gas emissions from MSW landfills are presently governed by two sets of regulations that may influence the design of landfill gas collection and control systems associated with the cover systems. A third regulation was proposed in November 2000 (EPA, 2000). RCRA Subtitle D regulations address the personal and fire/explosion safety aspects of landfill gas under 40 CFR §258.23, which requires:

> "*(a) Owners or operators of all MSWLF units must ensure that:*
> *(1) The concentration of methane gas generated by the facility does not exceed 25 percent of the lower explosive limit for methane in facility structures (excluding gas control or recovery system components); and*
> *(2) The concentration of methane gas does not exceed the lower explosive limit for methane at the facility property boundary.*"

The second set of regulations governing MSW landfill gas is the New Source Performance Standards (NSPS) and Emissions Guidelines (EG) promulgated under the Clean Air Act (CAA). The NSPS and EG regulate emissions of non-methane organic compounds (NMOCs) as a surrogate to total landfill gas emissions (40 CFR §60.755). MSW landfills with design capacities equal to or greater than 2.5 million megagrams and 2.5 million cubic meters with NMOC emission estimates of 50 megagrams or more per year must have: (i) a well designed and operated gas collection system; and (ii) a control system device capable of reducing NMOC mass in the collected gas by 98%.

The third regulation proposes national emission standards for hazardous air pollutants (NESHAP) for MSW landfills identified as major sources of hazardous air pollutants (HAP) listed in Section 112(b) of the CAA and some MSW landfills identified as area sources (EPA, 2000). The proposed NESHAP contains the same requirements as the EG and NSPS as well as some additional requirements to further reduce HAP emissions to the level reflecting the maximum achievable control technology (MACT). The total impact on MSW landfills is expected to be limited.

Additional information on gas management regulations for MSW landfills can be found at http://www.epa.gov/ttn/atw/landfill/landflpg.html.

Waste-generated gas affects cover systems in several ways. The presence or absence of gas influences the selection of the type of hydraulic barrier material. GMs are generally better barriers to gas migration than soils, with the possible exception of intact CCLs at or near saturation (although low-permeability soils at saturation can have low shear strength and drying of the soil is a concern). Also, it may be necessary to install a gas collection layer beneath the barrier to convey gas to outlets through the cover system, or alternatively to install gas extraction wells or trenches at a sufficiently close spacing to prevent gas build-up beneath the barrier.

A factor sometimes overlooked in the closure of old landfills and in remediation of contamination source areas is that placement of a cover system will trap any gas being generated by the waste. Gas generation rates at these facilities may be slow enough that gas generation is not even recognized as a design issue. Yet after cover system installation, gas pressure can slowly build up. This process may eventually lead to one or more of the following: (i) problems with cover system performance, including a reduction in the factor of safety along interfaces in the cover system below the hydraulic barrier; and (ii) for unlined or inadequately lined landfills and contamination source areas, subsurface gas migration. Subsurface gas migration has caused adverse groundwater quality impacts at many older, unlined landfills and may also cause increases in atmospheric emissions of gas and safety or health impacts to nearby residences both from gas migrating through the soil and being released from groundwater passing beneath a residence. EPA recommends that the potential for landfill gas generation and impacts to nearby residences or businesses always be carefully evaluated as part of any landfill closure or remediation project.

Another factor to be noted is the trend towards recirculation of leachate and addition of other liquids to promote decomposition of landfilled waste. This results in faster and greater gas production than conventional landfills. These factors need to be considered in the final cover design to ensure adequate protection to near-by residents and the environment.

1.5 Typical Cover System Components

Components of a typical hydraulic barrier cover system are briefly introduced here and discussed in more detail in Chapter 2. The usual sequencing of these components is illustrated in Figure

1-12. Depending on a site's regulatory status, not all components listed below may be required as part of the final cover system.

1.5.1 Surface Layer

The topmost component of a cover system is the surface layer. The primary functions of this layer are to resist erosion by water and wind, be maintainable, and provide a growing medium for vegetation, if present. The surface layer may also serve other purposes, such as promoting ET or satisfying project aesthetic, ecological, or end use criteria.

Materials that can be used for cover system surface layers include: (i) topsoil; (ii) amended topsoil; (iii) gravel-soil mixtures; (iv) gravel; (v) riprap; (vi) articulated block systems; (vii) asphaltic concrete; and (viii) other materials. Of these materials, topsoil is, by far, the one most commonly used. Suitable topsoil promotes growth of vegetation, thereby maximizing the ET component of the cover system water balance. Vegetation also decreases the quantity and velocity of stormwater runoff on the cover system slopes and reinforces the topsoil; both of these effects reduce the rate of topsoil erosion in comparison to a topsoil layer without vegetation. At sites with conditions unfavorable to maintaining an adequate growth of vegetation (e.g., sites with steep slopes or in semi-arid or arid climates), gravel-soil mixtures, gravel, riprap, articulated block systems, or other materials may be used for the surface layer.

1.5.2 Protection Layer

A protection layer may serve several functions:

- protect underlying layers from erosion;
- protect underlying layers from exposure to wet-dry cycles, which may cause degradation of these layers;
- protect underlying layers from exposure to freeze-thaw cycles, which may cause degradation of these layers;
- serve as a barrier to human, burrowing animal, or plant root intrusion (i.e. a biobarrier);

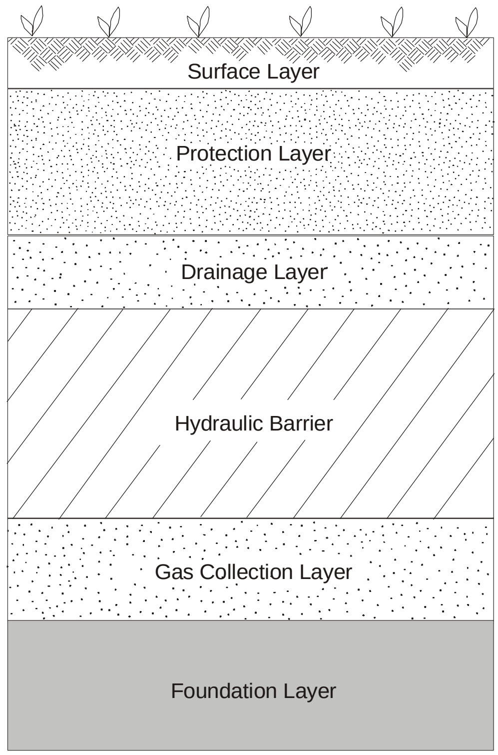


Figure 1-12. Typical Hydraulic Barrier Cover System Components.

- temporarily store water that has infiltrated through the surface layer until the water returns to the atmosphere through ET; this action provides a water reservoir to support plant growth and reduces infiltration into underlying cover system layers; and
- restrict emissions of radon gas for those wastes, such as uranium mill tailings, that emit radon.

On-site or locally available soil is usually suitable for protection layer construction if the primary functions of the layer are to support vegetation and protect underlying layers from cracking due to wet-dry and freeze-thaw effects. However, if the primary role of the protection layer is to prevent biointrusion, cobbles, asphaltic concrete, or similar materials are typically required.

1.5.3 Drainage Layer

In a hydraulic-barrier type cover system, a drainage layer may be required beneath the protection layer and above the hydraulic barrier, particularly on sideslopes. A drainage layer may serve several functions:

- limit the buildup of hydraulic head on the underlying hydraulic barrier, which minimizes percolation of water through the barrier;

- drain the overlying surface and protection layers, which increases the available water-storage capacity and helps to minimize erosion by reducing the time during which the layers remain saturated with water; and
- reduce the seepage forces in the protection, surface, and drainage layers, which improves cover system slope stability.

Materials used for drainage layers include sand, gravel, geotextile (GT), geonet (GN), and geocomposite (GC) drainage materials. The material used should have adequate hydraulic conductivity to minimize the buildup of hydraulic head above the hydraulic barrier and adequate hydraulic transmissivity to convey the design flow rate. If gravel or a GN is used for the drainage layer, a filter layer will usually be needed between the drainage layer and the overlying protection layer to prevent fines from clogging the drainage layer. GT filter layers are typically used to achieve this function, although soil filter layers can also be used. If the drainage layer consists of gravel, and the underlying barrier is a GM, a GT cushion layer is typically needed between the GM and gravel. One of the most important aspects of designing a drainage layer is providing for free drainage at the drainage layer outlet.

1.5.4 Hydraulic Barrier

The function of the hydraulic barrier is to minimize percolation of water through the cover system by impeding infiltration into the barrier and by promoting storage or lateral drainage of water in the overlying layers. Properly designed, constructed, and maintained hydraulic barriers can virtually eliminate infiltration into the waste. Hydraulic barriers also restrict migration of gas or volatile constituents from the waste mass to the atmosphere.

Materials used for hydraulic barrier construction include GMs, GCLs, and CCLs. Each of these barrier materials may be used alone or in combination. It has been shown, however, that, all else being equal, a cover system with a composite barrier consisting of GM/CCL, GM/GCL, or GM/GCL/CCL allows less percolation than a cover system with a GM, GCL, or CCL barrier.

1.5.5 Gas Collection Layer

Gas collection layers may be necessary beneath cover system barriers for wastes that generate gas or emit volatile constituents. These layers are designed to have adequate in-plane gas transmissivity to convey gas to passive gas vents, active gas wells, or trenches. Gas collection layers are typically a necessary complement to systems that utilize passive gas vents. Gas collection layers may not always be needed for landfills with active gas extraction systems, depending on gas generation rates in the landfill, extraction well spacing, presence or absence of horizontal gas trenches, and other factors.

Gas collection layers may be constructed of granular materials (e.g., sand or gravel) or geosynthetics (e.g., GT, GN, GC). The selected material must have adequate transmissivity to minimize the build up of gas pressures beneath the barrier and convey the design gas flow rate. When a granular material is used, a separation layer (typically a GT) may be needed to separate the granular material from the overlying barrier.

1.5.6 Foundation Layer

The foundation layer is the bottom-most component of the cover system. The functions of the foundation layer are to provide grade control for cover system construction, adequate bearing capacity for overlying layers, a firm subgrade for compaction of overlying layers, a smooth surface for installation of overlying geosynthetics, and, in some applications, a buffer zone to reduce the potential effects of waste differential settlements on the cover system components.

Materials most often used for the foundation layer include on-site or locally available soils. Sometimes, intermediate cover soil already in place is used for all or a portion of the foundation layer. In a few situations, waste material can be used to construct the foundation layer. If constructed of granular material, the foundation layer may also serve as a gas collection layer.

1.6 Design Criteria Development and Conceptual Design

1.6.1 Overview

Gross et al. (2002) present the results of a survey conducted for EPA on problems and lessons learned at representative landfill facilities located throughout the U.S. The survey identified 69 modern landfill facilities that had experienced 80 liner system or cover system problems. For the study, a "modern facility" was considered one with components substantially meeting current EPA regulations and constructed and operated to the U.S. state-of-practice from the mid-1980's forward. Almost 30% of the problems identified in the study involved landfill cover systems. The percentage of cover system problems for the 69 facilities will likely be higher in the future since a number of these facilities were active and did not yet have a cover system. The primary factor contributing to the cover system problem in most cases was inadequate design.

The number of facilities in the EPA study is small compared to the total number of modern landfills nationwide. However, the search for problem facilities was not exhaustive. The Agency believes many more facilities than identified in the study have experienced the types of problems identified in the study. As pointed out in the EPA study, the single factor that can most improve the performance record for waste containment systems is improved design practice by the engineering community. It is imperative and consistent with the standard of professional care that engineers prepare complete, detailed, and proper designs of cover systems. Simple and incomplete design approaches intended to simply satisfy regulatory requirements and "get the grass growing" are not acceptable. This guidance document is intended to contribute to improved practices with respect to the design of cover systems.

The critical first steps in designing a landfill cover system involve: (i) developing the criteria that will be used to guide the design; (ii) preparing a conceptual design using these criteria; (iii) identifying data gaps based on the conceptual design; and (iv) performing predesign studies to generate the data needed to prepare the detailed design and construction plans and specifications. Design criteria development is addressed in more detail below.

1.6.2 Regulatory Requirements

The first step in establishing design criteria is to identify all applicable regulatory requirements (for a RCRA Subtitle D or Subtitle C facility) or ARARs (for a CERCLA site remediation). General guidance on applicable regulatory requirements was given in Section 1.2 of this document. Federal regulations are found in the Code of Federal Regulations and are available on the U.S. Government Printing Office website at http://www.access.gpo.gov/nara/cfr/cfr-table-search.html. State and local regulations may also be available on-line.

1.6.3 Climatic Criteria

Climate significantly affects cover system design and performance. For example, the typical approach to preventing water percolation through a cover system for a facility in the eastern U.S. is to use a low-permeability hydraulic barrier. In arid regions of the western U.S., however, the same design objective can be achieved using an ET barrier. As another example, climatic factors influence the thickness of the cover soil required to protect an underlying hydraulic barrier from the effects of freeze-thaw. Further, climatic factors greatly affect the types of vegetation that can be grown on a cover system.

Climatic criteria to consider in the design of a cover system include the amount and seasonal distribution of precipitation, duration of specific storm events (e.g., 1-hour storm event, 24-hour storm event, etc.), intensity of specific storm events (e.g., 25-year recurrence interval storm event, 100-year recurrence interval storm event, probable maximum precipitation (PMP), etc.), seasonal temperature variations and extremes, depth of frost penetration, quantity of snow melt, wind speed and direction, solar radiation and humidity. In some areas (e.g., cold, arid), the controlling climatic criterion for percolation may be snowmelt.

1.6.4 Physical and Engineering Criteria

Physical criteria that should be considered in designing a cover system include the lateral limits of waste, property setback requirements, if any, height of facility above surrounding ground, sideslope length and inclination, top deck length and inclination, depth of waste within the facility, type and thickness of interim cover, and potential for the waste to generate gas. A distinction should be made at this stage between proposed landfills where the design engineer has control over essentially every physical parameter for the facility versus an existing landfill or CERCLA remediation site where the design engineer starts the design process by considering the pre-existing site conditions. The consequences of a certain design action can be quite different for these two situations. For example, it is a relatively straightforward matter to design and construct a stormwater management or slope stability terrace or bench for a new landfill. Conversely, design and installation of a terrace or bench for a cover system on a steep pre-existing landfill slope can be difficult or infeasible. The latter type of design requires either that a cut be made into the existing waste slope or, alternatively, that the terrace be built up above the waste slope using soil fill and foundation/slope reinforcement techniques. These kinds of differences should be considered by the design engineer during design criteria development.

Design criteria development for a cover system should also consider a number of engineering criteria. The design engineer should carefully consider which engineering criteria are relevant for a particular facility, and then apply them appropriately. For each engineering criterion that must be satisfied, the design engineer should define: (i) performance requirements for that criterion; (ii) method of analysis or evaluation; and (iii) required input parameters with numerical values for each parameter and at least qualitative, if not quantitative, consideration of the uncertainty (i.e., standard deviation, standard error, etc.) associated with the selected numerical values. As an example, a common engineering criterion for landfill cover systems is long-term static stability of the waste mass beneath the cover.

Table 1-1. Common engineering criteria for RCRA and CERCLA cover systems.

Slope Stability • Foundation stability • Waste mass stability • Cover system veneer stability • Pseudo-static stability analysis • Other stability conditions	**Settlement (Total and Differential)** • Foundation total settlement • Waste mass total settlement • Foundation differential settlement • Waste mass differential settlement
Seismic Deformation Analysis • Foundation liquefaction • Waste mass deformation • Cover system deformation	**Surface-Water Runoff Control** • Estimated peak flow rate • Surface-water control structure design (benches, channels, and retention ponds)
Geosynthetic Component Performance • GT filter layer requirements • GT clogging potential • GN/GC flow rate • GN/GC clogging potential • GN/GC compression resistance • GN/GC outlet design • GT cushion layer requirements • GM design (type, thickness, elongation and strength requirements) • GCL design (type, internal reinforcement, overlap)	**Erosion Control and Vegetation** • Rill and interrill erosion • Gully formation (tractive force analysis, critical distance for gully formation, permissible velocity analysis) • Wind erosion • Vegetation requirements (type, planting, fertilizer, amendments) • Temporary and permanent erosion control material requirements
Soil Component Performance • Erosion resistance of surface layer • Biointrusion resistance • Water storage capacity • Frost penetration depth • Drainage layer flow rate • Drainage layer clogging potential • Drainage layer outlet design • Granular filter layer requirements • Soil barrier hydraulic design (suitable soil availablity, thickness, hydraulic conductivity)	**Hydraulic Performance** • Cover system water balance • Percolation through cover system • Water flow in drainage layer • Maximum head in drainage layer **Gas Emission Control** • Gas emission rate analysis • Gas flow and pressure in collection layer • Gas collection system (active or passive) • Gas treatment requirements

The performance requirement for this criterion is usually expressed in terms of a factor of safety against slope instability. The minimum acceptable factor of safety might be 1.5, for example. (A discussion of recommended slope stability factors of safety is given in Chapter 6 of this document.) A method of analysis that could be used to evaluate this criterion is a two- or three-dimensional limit equilibrium method of slices. Input parameters for the evaluation include the geometry of the waste, unit weight and shear strength of the waste, existence of any perched or continuous zones of leachate in the waste, existence of landfill gas pressures beneath the cover system, and the thicknesses, unit weights, internal shear strengths, and interface shear strengths of the cover system installed over the waste.

A partial list of engineering criteria that are frequently considered in the design of RCRA or CERCLA cover systems with the components shown in Figure 1-12 are listed in Table 1-1. Not all criteria apply to all cover systems. The foregoing list of criteria, while extensive, is by no means exhaustive. Additional criteria will need to be considered on a case-by-case basis. Also, particular attention should be given to applicable engineering criteria any time an alternative or innovative cover system is proposed, as past precedent for such systems will, by definition, be limited or nonexistent.

1.6.5 Aesthetic and Land Use Criteria

Aesthetic and land use criteria are becoming more important in the design of cover systems. More and more, facility owners, regulators, and the local community are sensitive to the aesthetics of closed waste management sites. Today, it is not uncommon to design aesthetic enhancements into site closure projects. When such enhancements are to be used, they should be adequately designed in their own right, and any impact they may have on any other engineering criterion identified previously in this section should be addressed. Examples of aesthetic enhancements that have been incorporated into cover systems include:

- construction of an undulating sideslope to provide a more natural looking landform (compared to long, planar sideslope);
- planting of trees and shrubbery on terraces; and
- construction of decorative block retaining walls.

Increasingly, beneficial post-closure land uses are being considered in the design of cover systems for landfill closures and CERCLA remediations. The most common types of end uses are parks, hiking trails, sports fields, and golf courses. The selected end use can have a significant impact on cover system design. For example, if a site is to be used for a golf course or other facility with a vegetated surface layer that requires irrigation, the cover system may require an internal drainage layer and a barrier that includes a GM to control percolation through the cover system (Hauser, 2000). Figure 1-13 shows a completed CERCLA remediation in southern California where the site was closed with a multi-component soil and geosynthetic cover system, and a golf course was developed on the cover system. Further discussion of aesthetic and post-closure land uses for cover systems is given in Chapter 9 of this document.

1.6.6 Ecological Criteria

Conventional engineering approaches for designing cover systems often fail to fully consider ecological processes at work in the local environment. Natural ecosystems effective at capturing and/or redistributing materials in the environment have evolved over millions of years. Consequently, when contaminants are introduced into the environment, ecosystem processes begin to influence the distribution and transport of these materials, just as they influence the distribution and transport of nutrients that occur naturally in ecosystems (Hakonson et al, 1992). As the ecological status of a cover system changes, so will performance factors such as water infiltration, water retention, ET, soil erosion, and biointrusion. An objective often overlooked in designing cover systems is to cause subsequent ecological change to enhance and preserve the encapsulating system. Only through a holistic ecological approach can long-term maintenance requirements for cover systems be truly minimized (Caldwell and Reith, 1993). Consideration of natural analogs can enhance the design of a cover system by disclosing those processes that are active in a given environment or the mechanisms that could lead to failure. These mechanisms can then be avoided through appropriate design and construction. Natural analog studies provide clues from past environments that can be applied to the long-term behavior and performance of a cover system. Analog studies involve the use of logical analogy to investigate natural and archaeological occurrences of materials, conditions, or processes that are similar to those known or predicted to occur in some part of the cover system (Waugh, 1994). Perhaps the simplest examples of a natural analog for a cover system are the stable soil geomorphology in the locality of a project. Local soil geomorphology may be an indicator of the erosional stability of local soils used for the surface/protection layer in a cover system. For example, if a local glacial till is to be used for the surface/protection layer of a landfill, and all the local landforms containing that fill have evolved with slopes no steeper than a certain value, then use of that till on steeper cover system slopes contravenes the local geomorphological evidence, suggesting a greater likelihood of long-term maintenance requirements than might otherwise be the case.

A primary goal of design is to achieve a cover system that is as maintenance-free as possible. While it is debatable as to whether the need for all long-term maintenance can be eliminated, significant progress is possible with respect to current engineering practice. Moreover, in virtually all cases, some degree of maintenance or post-construction refinement may be necessary until the cover system reaches a state of equilibrium with its inherent environment.

An important point often not recognized is that a cover system should be stabilized with vegetation comprising plant communities that closely emulate a selected local “climax” community (Caldwell and Reith, 1993). A climax community, in ecological terms, is defined by the environmental parameters of the community (e.g., climate, soil, and landscape properties, fauna, and other flora). Central to the concept of “climax” is the community’s relative stability in the existing environment (Whittaker, 1975). A diverse mixture of native plants on a cover system maximizes water removal through ET (Link et al., 1994). The cover system is then more resilient to natural and man-induced catastrophes and fluctuations in environments. Similarly, biological diversity in cover system vegetation is important to community stability and resilience given variable and unpredictable changes in the environment resulting from pest outbreaks, disturbances (overgrazing, fires, etc.), and climatic fluctuations.

Figure 1-13. Example of Post-Closure Land Use: Closed California CERCLA Site Used as a Golf Course.

fig 1-13

Local native species that have been selected over thousands of years are best adapted to disturbances and climatic changes (Waugh, 1994). In contrast, planting of non-native species, as is common in the current standard-of-practice for landfill and containment system engineering, is genetically and structurally monotonous (Harper, 1987) and therefore more vulnerable to disturbances. Pedogenic processes gradually change the physical and hydraulic properties of earthen material cover systems (Hillel, 1998). Plant communities inhabiting the cover system will also change in response to these changes in soil properties.

A cover system that is to last for hundreds of years, or longer, should be designed as an integral component of a larger dynamic ecosystem. Cover system components initially designed for a specific purpose such as a barrier or drainage layer will not function independent of one another. Therefore, these systems should be considered not only individually, but also as a system (linked assemblage of components). Inevitable changes in physical and biological conditions should be taken into account to help ensure the long-term effectiveness of the cover system.

Chapter 2
Individual Components of Cover Systems

2.1 Introduction

As described in Section 1.5, a typical hydraulic barrier cover system will have the following components: surface layer, protection layer, drainage layer, hydraulic barrier, gas collection layer, and foundation layer (Figure 1-12). Not all components are necessary for all cover systems. For example, a gas collection layer is unnecessary if the underlying waste does not generate gases that require collection or control. Each component in a cover system serves a specific purpose and must function for its intended design life. For instance, the gas collection layer facilitates collection and control of decomposition gases or vapors generated by the waste or remediation source area material and must function as long as the gases or vapors are produced. The components of a cover system should interact as a system. The gas collection layer, for example, works properly only if one of the overlying layers (typically the hydraulic barrier) serves as a barrier to gas migration, allowing the gases to accumulate in the gas collection layer, where they can be removed. Also, attention must be paid to the interfaces between the components. For example, fine soil from one layer should not migrate into coarse soil in an adjacent layer (a separation or filter layer should be used if particle migration is a concern). In addition, adjacent materials sometimes have low shear strength along their interface (e.g., GN/GM, GM/CCL). Thus, the design of a multi-component cover system involves careful analysis of each component, consideration of how the components interact in a system, and evaluation of interfaces.

The functions, materials, and design principles for the six typical cover system components of hydraulic barrier cover systems are discussed in this chapter. Where components interact with one another, those interactions are discussed as well. Examples of cover systems for different applications are given at the end of the chapter.

2.2 Surface Layer

The primary functions of the surface layer are to resist erosion by water and wind, support easy maintenance, and provide a growing medium for vegetation, if present. The surface layer can also serve other purposes, such as promoting ET or meeting aesthetic, ecological, and site end use criteria.

2.2.1 General Issues

Perhaps the most important concern with respect to the surface layer is the potential for erosion. Excessive erosion can lead to exposure of underlying layers and can cause the cover system to be ineffective. Erosion can be controlled by managing surface-water runoff (see Section 2.2.4), minimizing seepage forces within the cover system soils (see Section 2.4), and selecting a surface layer material that can withstand the anticipated erosive stresses due to water and wind (see Sections 2.2.2.2 and 2.2.5).

2.2.2 Elements of Design

Important questions that typically need to be addressed when considering the design of the surface layer include:

- What materials are available to construct the surface layer?
- What thickness of surface layer material is needed?
- What maximum slope inclination can be used with the surface layer material while providing acceptable erosion rates?
- For vegetated cover systems, what plant species should be established?
- How should surface-water runoff be managed?
- What minimum slope inclination is required to promote runoff after accounting for settlement?
- What temporary and permanent erosion control measures should be used?
- How should the surface layer be constructed?
- What type and frequency of maintenance should be employed?
- What type and frequency of monitoring should be employed?

2.2.2.1 Slope Inclination

Slope inclination can be expressed in different ways, as shown in Figure 2-1. The ratio of horizontal and vertical (e.g., 3H:1V) is perhaps the most common way of expressing the inclination of landfill sideslopes. Slope inclinations are often expressed as a percentage when referring to landfill top decks, runoff, or internal drainage issues. When slope stability is analyzed, the inclination is typically expressed in degrees.

As shown in Figure 2-2, some cover systems have a relatively flat top deck and steeper sideslopes. In such situations, the cover system components might be different in the flatter and steeper areas. For example, the surface layer might be topsoil on the top deck and rock riprap on the sideslopes. However, in most instances, the same components are used on both the flatter and steeper areas.

Most landfill cover system top decks are designed to have a minimum inclination of 2 to 5%, after accounting for settlement, to promote runoff of surface water. Slopes flatter than 2% may allow water to pond on the surface, if localized settlements occur, and are usually avoided. However, in some cases involving the closure or remediation of existing landfills, waste piles, or source areas, flatter slopes may already exist and the cost to increase the slope inclination by fill placement or waste excavation may be significant. In these cases, slightly flatter inclinations can be considered if the future settlement potential can be demonstrated to be small, if concerns about localized subsidence can be adequately addressed, and if monitoring and maintenance provisions exist to repair areas of grade reversal or subsidence.

The potential for excessive erosion or slope instability increases as the cover system inclination increases. Sideslope inclinations can range from flatter than 5H:1V to steeper than 2H:1V.

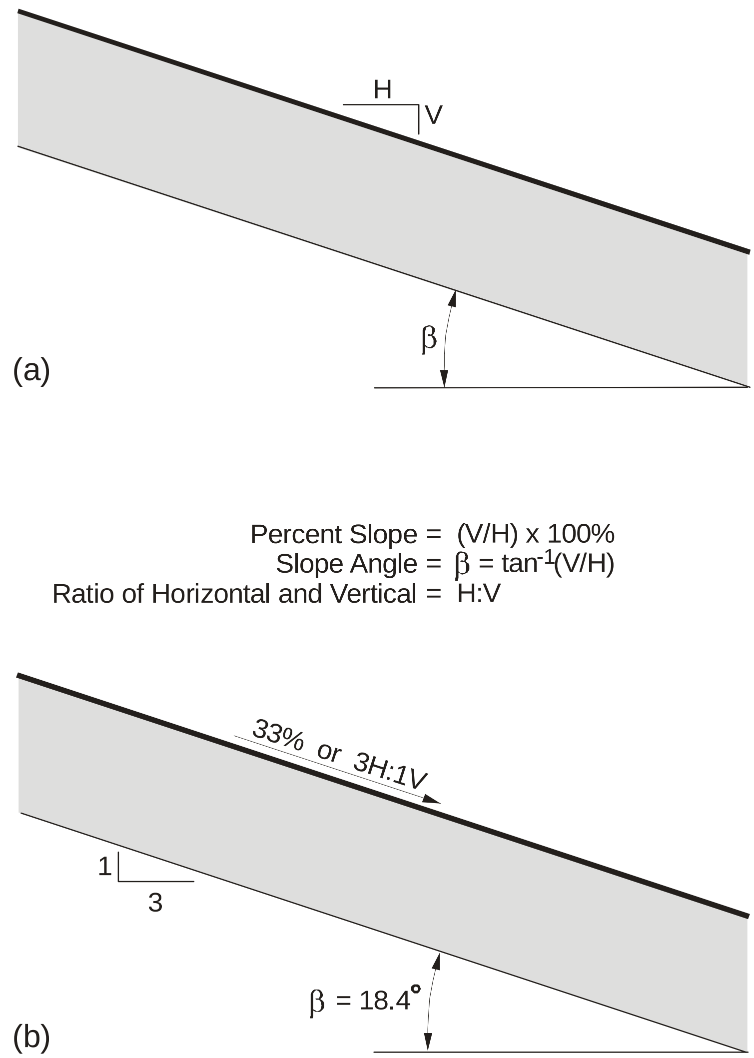


Figure 2-1. Slope Inclination: (a) Definitions; and (b) Example.

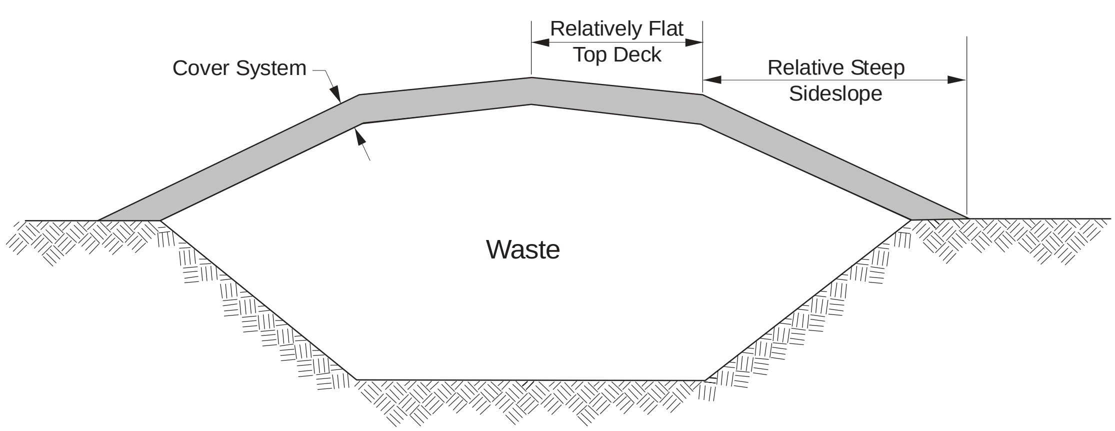


Figure 2-2. Relatively Steep and Flat Sections on a Typical Landfill Cover System.

Flatter sideslope inclinations are typically associated with surface impoundments, HW landfills, low-level radioactive waste landfills, and sites with soft remediation wastes. Some landfill cover system sideslopes are as steep as 2H:1V (and even steeper for some old landfills). Most modern MSW landfills have maximum cover system inclinations in the range of 4H:1V to 3H:1V, values selected to balance the need for facility capacity with considerations related to facility operational efficiency, waste mass and cover system slope stability, and surface erosion. Slopes with inclinations near the flatter end of this range (4H:1V or flatter) are typically used for cover systems when less maintenance will be performed or for projects in which erosion or slope stability is a particularly critical issue.

2.2.2.2 Materials

In humid climates, a vegetated topsoil layer substantially reduces the potential for surface erosion in comparison to bare ground. Vegetation serves to reduce the quantity and velocity of runoff, reduce soil mobilization due to raindrop impact, and bind soil particles together through root systems. Vegetation also promotes ET of infiltrating water. Alternatives to a topsoil surface layer are typically only considered when it is difficult to maintain vegetation (e.g., on steep slopes or in arid or semi-arid areas). At sites with this condition, the vegetative cover may not have sufficient density to provide adequate erosion protection. Grasses and shrubs may tend to be clumped, leaving a substantial percentage of the surface devoid of vegetation and unprotected from wind and runoff. In such circumstances, alternative, erosion-resistant materials may be warranted to help encourage native vegetation establishment and growth and to reduce erosion. In this type of environment, the addition of organic matter and plant nutrients to the surface soils and the use of soil-gravel mixtures (see Section 2.2.2.2.3), gravel (see Section 2.2.2.2.4), riprap (see Section 2.2.2.2.5), geosynthetic erosion control materials (see Section 2.2.5.4), or other materials may be required. Alternatives to a topsoil surface layer may also be considered to achieve a desired end use for the property, e.g., a parking lot or building.

2.2.2.2.1 Topsoil

The most common material used to construct the surface layer is locally available topsoil. Because the soils and rocks of different regions are variable, topsoils are variable, as well. However, all topsoils tend to be relatively rich in organic matter and contain a broad mixture of particle sizes. General information on the surface soils for a particular area of the U.S. is summarized in the U.S. Department of Agriculture (USDA) National Resources Conservation Service (NRCS) soils surveys. Soil surveys may be obtained from the State or local office of the NRCS. Some of these surveys are also available online at http://www.statlab.iastate.edu/soils/nssc/.

Soils used for cover systems are typically classified using either engineering or agricultural soil classification systems. The agricultural system, employed by the USDA and summarized in Figure 2-3, classifies soil based on the relative amounts of sand, silt, and clay. A mixture of sand, silt, and clay is called “loam.” Soils that promote and sustain plant growth are typically loamy soils. The sand in the loam provides a stable matrix that does not tend to shrink and crack when the soil dries, and the sand helps promote good drainage. A fine material (silt and clay) fraction is important in topsoil for retention of moisture. For these reasons, a loamy soil that contains organic matter and nutrients is ideal for topsoil.

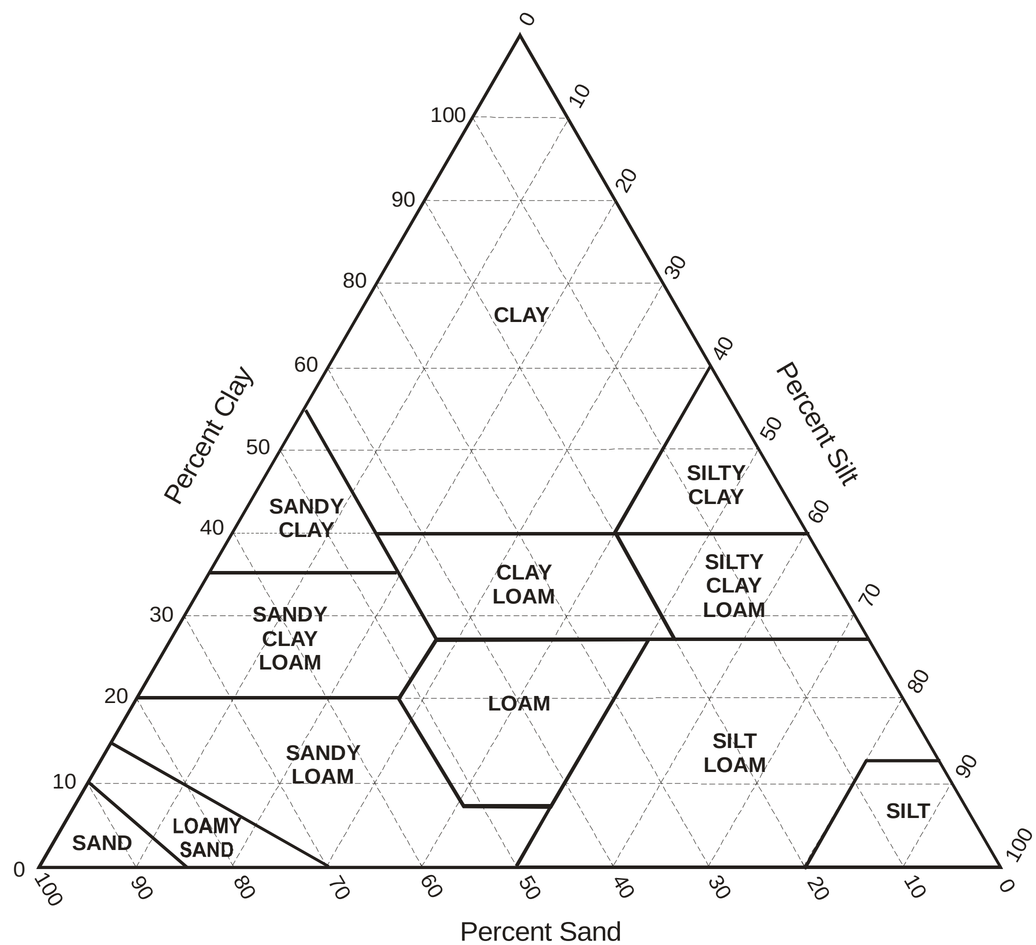


Figure 2-3. USDA Soil Classification System. USDA Particle Sizes: Sand, 0.05 – 2 mm; Silt, 0.002 - 0.05 mm; and Clay, < 0.002 mm.

The design engineer should consult local agricultural specialists when evaluating the soil proposed for the surface layer. The most appropriate type of soil to use may depend on the type of vegetation that will be planted. Site-specific factors, such as soil pH and salinity, may be very important.

2.2.2.2.2 Amended Topsoil

It is important that topsoil contain adequate organic matter and plant nutrients. If not, supplements (e.g., compost, fertilizers) may be added. An increasingly common practice is to amend topsoil with organic matter that would otherwise constitute a waste material, such as wastewater treatment sludge or fibrous waste from production of paper. The organic matter in these materials helps to promote growth of vegetation, and the use of these materials in surface layers leads to productive use of a material that would otherwise be a waste material. Care should be taken if these types of waste materials are used to ensure that surface-water runoff from the amended topsoil is safe when discharged to surface waters. The organic amendment should also be demonstrated to be non-pathogenic and to not create a nuisance (e.g., odor, vectors, etc.)

2.2.2.2.3 Soil-Gravel Mixture

At sites where excessive erosion may occur with topsoil alone, a soil-gravel mixture may be suitable. Erosion (Ligotke, 1994) and water balance studies (Waugh, 1994) suggest that moderate amounts of gravel (e.g., 25% by weight) mixed into topsoil can control both water and wind erosion with little effect on the vegetation or the soil water balance. As wind and water pass over the cover surface, some winnowing of fines from the gravel-soil mixture is expected, creating a vegetated erosion-resistant surface sometimes referred to as a "desert pavement". The size of gravel used in the mixture is typically in the range of 10 to 50 mm in diameter.

This design was utilized in an alternative cover system as part of a landfill research project in Albuquerque, New Mexico. The surface treatment consisted of mixing 25% by weight pea gravel with topsoil in the uppermost 6 inches of the fine layer of a capillary barrier. Results have shown this to be very effective to date. (Dwyer 2001)

As another example, a 1-m thick silt loam-pea gravel mixture was used as the top deck surface layer for a prototype cover system constructed over a contamination source area at the U.S. Department of Energy (DOE) Hanford Site. The prototype cover system was constructed in 1994 and its performance was monitored for four years as part of a treatability study (http://hanfordbarriers.pnl.gov/sum_tests.asp). Results of the study demonstrated that the cover system performance criteria were met or exceeded, and the cover system design components are highly effective for the Hanford Site.

2.2.2.2.4 Gravel Veneer

A thin surface layer consisting of 10 to 50-mm diameter gravel may be used to provide more erosion protection than a topsoil surface layer and can also result in the establishment of vegetation. The gravel can trap seeds until they germinate. In addition, there is more near surface moisture available for plants since there is generally less surface evaporation from a gravel layer than from a topsoil layer. At the low matric potentials typically experienced in the semi-arid and arid climates where a gravel surface layer may be used, finer-grained soils generally have a higher hydraulic conductivity and, thus, higher evaporation rate than coarser-grained soils. Consequently, after the gravel dries, the finer-grained soil below the gravel will tend to remain moist because the overlying coarser-grained gravel layer is, at this point, essentially non-conductive. The tendency of granular material to behave in this manner is utilized by gardeners who apply mulch to bare soil. The mulch allows water to percolate down to the underlying soil but shields the soil from evaporative loss of water (Kemper et al., 1994).

In comparison to a soil-gravel surface layer, a gravel veneer surface layer affects the soil water balance. The significance of this effect has not been well studied, but its potential impact must be acknowledged when use of a gravel surface layer is considered. The use of a gravel surface layer reduces evaporation. However, the added vegetation established on the gravel layer and the additional available moisture in the surface soils increases transpiration. Depending on the site conditions, the reduction in evaporation may or may not be balanced by the increase in transpiration.

A gravel veneer surface layer was utilized in an alternative cover system as part of a landfill cover research project in Albuquerque, New Mexico. The treatment installed gravel 0.6 cm diameter and 2 to 4 cm in depth. Results have shown this method to be effective. (Dwyer 2001)

Sources of clean gravel are often limited, which means that the gravel must frequently be quarried from rock. Before gravel is selected for the surface layer, the cost of the material should be established to ensure that the use of gravel is practical.

When gravel is used for the surface layer, a separation layer (e.g., GT) may be necessary between the gravel and the underlying material to prevent this latter material from being eroded by water.

2.2.2.2.5 Riprap

At sites where is it difficult to establish and maintain vegetation, a riprap (cobble) surface layer may be preferred. Clean riprap may adversely impact the water balance of the cover system. Precipitation that falls on the riprap percolates downward with virtually no impedance. Evaporation is limited because riprap has large openings and water falling though the riprap and into the underlying soil will not be brought back by capillarity to the riprap surface for evaporation. In addition, plants, other than occasional deep rooted plants such as shrubs and trees, do not normally grow through the riprap and, therefore, do not remove water from the subsoil and transpire it back to the atmosphere. Thus riprap serves as a one-way conduit for water movement by allowing water to percolate downward into the underlying materials but contributing almost nothing to upward water migration via ET. For example, field experiments at Hanford, Washington, demonstrated that the placement of an unvegetated gravel surface layer over a silty soil caused approximately half of the annual 150 mm of rainfall to percolate through the upper 2 m of soil (Gee et al., 1992). In contrast, when silt (even without vegetation) was exposed at the surface and not covered with gravel, there was zero percolation through the 2-m thick soil profile during the monitoring period.

There are instances in which it may desirable to have a relatively large amount of infiltration penetrating into the cover system. One such case involves a soil cover system constructed over radioactive wastes that emit radon gas. For this case, surface emissions of radon can be controlled by covering the waste with a thick, wet layer of clayey soil. Wet, clayey soils are practically impermeable to gas. Maintaining a high water content in the soil is desirable in such situations, and a layer of riprap at the surface can help to keep the underlying soil wet. The increased infiltration may, however, result in increased percolation through the cover system, and it may be more advantageous to incorporate a gas collection layer and overlying GM barrier into the cover system.

In earthwork projects, riprap is often the most expensive material used on the project. This is because sources of clean cobbles are fairly rare, which means that the riprap must often be quarried from rock. Frequently, the closest source of riprap may be tens or hundreds of kilometers from the project site. Thus, before riprap is selected for the surface layer, the cost of the material should be established to ensure that the use of riprap is practical.

For cover systems, riprap is often sized based upon experience, judgment, and the size of material that is available. The typical minimum particle size of stones in riprap used for cover systems is 10 to 300 mm. However, the minimum particle size depends on the steepness of the slope and the anticipated water flow velocity. If relatively steep slopes are used, large, angular stones may be necessary to maintain the stability of the stones on the slope. Some cover systems at large landfills have somewhat irregular surfaces with high and low areas. Natural drainage swales or channels may exist. There is more potential for higher-velocity water flow in these swales or channels, compared to other areas, and larger stones (up to approximately 150 to 300 mm or greater) may be appropriate in such areas.

When riprap is used for the surface layer, a bedding layer (e.g., cobbles) or a separation layer (e.g., GT) may be necessary between the riprap and the underlying material to prevent this latter material from being eroded by water. When riprap is used to line drainage swales or channels on the cover system, the riprap is sometimes placed on a piece of GM to limit infiltration into the underlying cover system components. If this detail is used, an outlet should be designed to accommodate the water collected on the GM.

As an example, a basalt riprap (less than 250 mm diameter) surface layer was used on the 2H:1V sideslopes along the perimeter of the prototype cover system constructed over a contamination source area at the DOE Hanford Site (http://hanfordbarriers.pnl.gov/sum_slope.asp). As previously mentioned in Section 2.2.2.2.3, the performance of the prototype cover system was monitored for four years and found to be satisfactory.

2.2.2.2.6 Asphaltic Concrete

Asphaltic concrete is a mixture of aggregate (usually sand and gravel) and asphalt, sometimes with additional materials such as polymers. Heated asphalt is mixed with aggregate, spread in a thin layer (typically 50 to 100 mm thick), and compacted with heavy, steel vibratory drum rollers. Asphaltic concrete can be placed as a single layer or in multiple layers.

Asphaltic concrete can be quite permeable unless special attention is given to minimizing air voids during mixing and application (Repa et al., 1987). To achieve low hydraulic conductivity, 1.5 to 2 times more asphalt is used than is typical for roadway pavements. This type of asphaltic concrete is referred to as "low-permeability asphaltic concrete." Both ordinary and low-permeability asphaltic concrete have been used in cover systems. In some cases, the low-permeability asphaltic concrete layer is the only cover system component and functions as the surface layer and hydraulic barrier.

A low-permeability asphaltic concrete layer should not be considered as a permanent hydraulic barrier, unless it is maintained. Asphalt becomes brittle over time as a result of exposure to ultraviolet radiation and oxygen. In addition, an asphaltic concrete layer in a cover system may develop cracks due to differential settlement of underlying waste. If the intent is to maximize design life, the asphaltic concrete layer should normally be buried beneath a protection layer and not subjected to differential settlements that would induce cracking.

The following are examples of cover systems in which asphaltic concrete was used as the surface layer. One case involved a 1-ha area of contaminated soil that was located next to an office building. The cover system was paved with ordinary asphaltic concrete and used as a parking lot. In a second case, a small section of a landfill cover system was paved with low-permeability asphaltic concrete to create an area that could be used to park maintenance vehicles. The third case was a remediation project in which there was particular concern for minimizing or eliminating erosion. Again asphaltic concrete was used as the surface layer. In the latter two cases, the asphaltic concrete was a low-permeability material that contained an asphalt application rate intended to produce a hydraulic conductivity of 1×10^{-9} m/s or less. In both of these cases, the asphaltic concrete served as a surface layer and hydraulic barrier.

The National Risk Management Research Laboratory of the EPA is currently evaluating the application of a low-permeability asphaltic concrete cover system to two CERCLA sites under the Superfund Innovative Technology Evaluation (SITE) Program (http://www.epa.gov/ORD/SITE/). Each cover system consists of a 100-mm thick layer of proprietary-blend low-permeability asphaltic concrete.

2.2.2.2.7 Other Materials

Practically any material, including articulated block systems, some construction and demolition wastes, and some lightweight manufactured aggregates (e.g., expanded shale), could potentially be used as a material in a surface layer or could be mixed with other materials and used for the surface layer. However, if something other than soil, gravel, or riprap is considered, it will generally be because there is a special desire or incentive for utilizing a particular material. Alternative materials should be considered if they are safe, stable, and can meet applicable design criteria.

2.2.2.3 Thickness

The minimum thickness of the surface layer is established based on consideration of the rooting depth of any surface vegetation, anticipated erosion rate, and construction tolerances. With respect to the latter, it is usually not practical to construct a layer thinner than about 0.15 m using typical earth moving equipment. If topsoil or a topsoil-gravel mixture is used, the soil should be thick enough to accommodate a healthy growth of plant roots. For shallow-rooted plants such as certain grasses, a 0.15-m thick layer of soil usually provides adequate rooting depth. Thus, the minimum thickness of a vegetated surface layer is generally 0.15 m. If plants with deeper roots are planted or represent a desirable climax community, the thickness of the topsoil should be increased to accommodate root growth. The underlying protection layer (if present) may also accommodate plant roots, in which case 0.15 m of topsoil may be all that is needed for the surface layer.

In some instances, the surface layer and protection layer are constructed from the same type of material, making it impossible to distinguish one layer from the other. The combined layers may be referred to as “cover soil” or “cover material”. If the surface and protection layers are combined into a cover soil, then the minimum thickness of the cover soil should be evaluated considering the plant rooting depth. A typical minimum thickness of the cover soil is 0.45 to 0.6 m for cover systems with hydraulic barriers. For cover systems with ET or capillary barriers, EPA recommends a minimum cover soil thickness of 0.9 m or greater (see Section 3.2.5). Thicknesses greater than 1 m are occasionally used to provide a suitable medium for growth of plants in relatively arid areas, which commonly have deep-rooted plants. Greater thicknesses of cover soil may also be needed to provide a hydraulic barrier with protection from desiccation or frost.

If gravel or riprap is used for the surface layer, the minimum thickness is usually 0.15 m or twice the average particle size of the material, whichever is larger.

If asphaltic concrete is used for the surface layer, the minimum thickness should be determined from an analysis of vehicular loading, but would typically be in the range of 75 to 150 mm.

2.2.3 Vegetation

Selection of plant species is an important consideration in the design of a vegetated surface layer. The vegetation serves several functions:

- Plant leaves intercept some of the rain before it impacts the surface layer, thereby reducing the energy of the water and the potential for erosion.
- Plant vegetation also helps dissipate wind energy.
- The shallow root system of plants enhances the surface layer resistance to water and wind erosion.
- Plants promote ET of water, which increases the available water storage capacity of the cover soils and decreases drainage from these soils.
- A well-vegetated surface layer is generally considered more natural and esthetically pleasing than an unvegetated surface layer.

In selecting the appropriate vegetation for a site, the following general recommendations are offered:

- Locally-adapted, low-growing (less than 1 m high) grasses and shrubs that are herbaceous or woody perennials should be selected. Native plants are recommended to maintain long-term ecological stability.
- The plants should survive drought and temperature extremes. They should also tolerate inhospitable site conditions (e.g., exposure to landfill gas).
- The plants should contain roots that will penetrate deep enough to remove moisture from beneath the surface but not so deep as to disrupt the drainage layer, hydraulic barrier, or gas collection layer.
- The plants should be capable of thriving with minimal addition of nutrients.
- The plant population should be sufficiently diverse to provide erosion protection under a variety of conditions.
- The plants should not be an attractant to burrowing wildlife.
- The vegetative cover should be capable of surviving and functioning with little or no maintenance (e.g., without excessive irrigation, fertilization, and mowing).

Guidance on selection of vegetative materials is found in Wright (1976), Thornburg (1979), Lee et al. (1984), and EPA (1985). These references provide information about plant species, seeding rate, time of seeding, and areas of adaptation. Growth information for a number of plant species is available in the USDA Plant database at http://plants.usda.gov/. Local plant specialists, such as the NRCS, are usually consulted to select the appropriate mixture of seeds for a site. Local NRCS and Department of Transportation specifications may also be useful. Experience also is very helpful, and once a seed mixture has been shown to provide satisfactory performance in a particular region, it tends to continue to be used.

At many sites with cover systems located in humid and temperate parts of the country, the cover systems are seeded with a mixture of grasses. The mixture may contain several grass species to provide diversity in the grass population, promote vegetative growth for as much of the year as possible, and maintain a vegetative layer with the desired mixture of shallow- and medium-depth roots. Information on grasses is available in Hanson and Juska (1969), who subdivide the U.S. into the four regions shown in Figure 2-4. Native or locally-adapted grasses that they generally recommend for permanent vegetative covers are listed in Table 2-1.

Figure 2-4. Major Regions of Grass Adaptation in the U.S. (modified from Hanson and Juska, 1969).

Table 2-1. Grass species recommended for use as permanent vegetative covers in the four regions of grass adaptation (modified from Hanson and Juska, 1969).

Region	Species	Seeding[1] Time	Seeding Rate[2] (kg/ha)	Comments
Cool-humid (Region 1)	Kentucky bluegrass (*Poa pratensis* L.)	Spring & Fall	20	Do not use named varieties
	Tall fescue (*Festuca arundinacea* Screb.)	Spring & Fall	40	Use K-31 or Alta varieties; can winter kill north of Interstate 80
	Perennial ryegrass (*Lolium perenne* L.)	Spring & Fall	40	Do not use named varieties
	Smooth brome (*Bromus inermis* Leyss.)	Spring & Fall	20	Use southern type except in extreme northern part of region
	Redtop (*Agrostis alba* L.)	Spring & Fall	15	Not very tolerant of mowing; good for wet conditions
	Weeping lovegrass (*Eragrostis curvula* Schrad.)	Spring & Early Summer	5	Use in southern ¼ of region only since less winter hardy than other species
Warm-humid (Region 2)	Bermudagrass (*Cynodon dactylon* L.)	Spring & Early Summer	10	Do not use named varieties
	Bahiagrass (*Paspalum notatum* Fluegge)	Early Summer	20	Do not use named varieties unless cold tolerance is important
	Zoysia (*Zoysia japonica* Steud)	Summer	See Reference	Propagated vegetatively
	St. Augustine grass (*Stenotaphrum secundatum* Kuntze)	Early Summer	See Reference	Propagated vegetatively; common is coarser textured than named varieties
Warm-arid & semi-arid (Region 3)	Bermudagrass (*Cynodon dactylon* L.)	Spring	10	Do not use named varieties
	Buffalograss (*Buchloe dactyloides* Englem.)	Spring	25	Use only in the eastern ¼ of the region
	St. Augustine grass (*Stenotaphrum secundatum* Kuntze)	Early Summer	See Comment	Use only in extreme southern part of region and at low elevations
Cool-arid & semi-arid (Region 4)	Bermudagrass (*Cynodon dactylon* L.)	Early Summer	10	Do not use named varieties; use only in extremely southern part of region
	Buffalograss (*Buchloe dactyloides* Englem.)	Spring & Early Summer	25	Use only in eastern ¼ of region
	Sideoats grama (*Bouteloua curtipendula* Torr.)	Spring	35	Use Blue grama (*Bouteloua gracilia* Lag.) if less than 380 mm precipitation
	Fairway wheatgrass (*Agropyron cristatum* Gaertn)	Spring	25	Best adapted to northern ½ of region; use Crested wheatgrass (*A. desertorum* Schult.) in the southern part at elevations of 1,500 to 2,700 m

[1] For species that can be seeded spring and fall, fall seedings are almost always more successful.
[2] Seeding rates assume single species. Reduce rates by the number of components in mixtures.
Minimum % pure live seed of 70 is assumed (% pure live seed = % germination x purity).
If the % pure live seed is less than 70, increase seeding rate accordingly.

Sometimes the vegetation is selected to maximize ET of water. For example, O'Donnell et al. (1997) describe the use of juniper plants to minimize infiltration of water through a cover system. Hybrid poplar trees, planted at a high density (e.g., 2,700 trees/ha), have also been used for the same application (Licht et al., 2001).

For cover systems in humid or temperate climates vegetated with grasses, the grasses are usually mowed periodically to discourage the growth of shrubs, trees, or other types of deep-rooted plants. Deep-rooted plants are usually undesirable because their root systems could plug the drainage layer or penetrate the hydraulic barrier, if it consists of only a CCL or GCL without an overlying GM. Trees can also create problems if they are blown over, uprooting large masses of soil and leaving a crater in the surface.

For sites designed to allow the development of climax communities, plant roots are typically deeper than for sites vegetated only with grasses. To prevent clogging of the drainage layer by plant roots, the thickness of the cover soils is increased or the drainage layer is sometimes treated with a biocide. Alternatively, the cover system is designed with relatively shallow sideslopes so that the ability of the drainage layer to function is not as critical. For example, native plants, including coastal sagebrush, were established on several closed landfills with thick ET cover systems in southern California in the late 1990's. When the native plants on these covers were studied to assess their growth characteristic, the roots of some of the native species had penetrated up to 2 m into the cover system soils.

To help in the initial establishment of vegetation, adequate soil nutrients should be available. In addition, soils detrimental to vegetation growth (e.g., soils with high salt contents) should be avoided. While soil amendments will improve the soil's characteristics as a rooting medium, any additional processing or amendments will increase costs.

2.2.4 Surface-Water Control

Surface-water runoff from the cover system should be controlled using a surface drainage system. The channelization of runoff is critical with respect to managing flow and controlling erosion. The drainage system may consist of a network of swales, ditches, downchutes, drop pipes, and culverts. Each component of the drainage system should be designed for the peak flow conditions anticipated from the design storm. Downchutes represent a particular challenge due to the high water velocities that occur on steep slopes. Flows from the cover system are typically directed to sediment traps, basins, and/or ponds to minimize the release of sediments and control rates of water flow from the site.

The design of a surface drainage system often constitutes a significant exercise in surface-water hydrology. The process can be very complex, involving statistical analysis of storm events, prediction of runoff for situations where minimal quantitative data exist, consideration of the potential occurrence of storms during interim stages of landfill development, consideration of changing cover system inclinations over time as the underlying waste settles, and other complications.

It is common practice to construct swales and ditches on cover systems with long vegetated sideslopes to intercept runoff and water from any cover system drainage layer outlets (Figure 2-5). Swales may be formed by constructing soil add-on berms on a uniformly sloping cover system (Figure 2-5(a)) or by constructing benches into the cover system sideslopes (Figure 2-5(b)). Ditches may be constructed adjacent to cover system access roads (Figure 2-5(c)). The swales and ditches are often connected to armored downchutes or to drop pipes, which convey runoff from the cover system sideslopes. A supplemental hydraulic barrier may be installed beneath the surface layer of swales, ditches, and downchutes to decrease the potential for infiltration of water into underlying cover system components. If the cover system surface layer consists of riprap or asphaltic concrete, surface drainage features, such as swales and ditches, may not be necessary.

The vertical spacing of swales and ditches on a cover system slope should be designed considering the need to manage surface water and limit erosion. In many cases, the spacing is controlled by erosion concerns (see Eq. 2-5 in Section 2.2.5.4 and Eq. 2-9 in Section 2.2.5.5.3) and is a function of slope inclination, surface layer material and vegetation properties, rainfall intensity, and other factors. As a general rule of thumb, surface-water interception may be necessary on cover system sideslopes at intervals of 10 m vertically or 30 m along the slope, whichever produces more frequent benches. Leaving out benches altogether on slopes with lengths greater than approximately 30 to 50 m may lead to excessive erosion and is usually avoided for slopes with inclinations greater than 5%. Erosion rills forming gullies as deep as 1 m can develop, and hundreds of cubic meters of soil can be washed away in a few days of inclement weather if adequate surface water controls are not employed. The actual vertical spacing of swales and ditches on a cover system should be based on local factors and detailed hydraulic and erosion analyses and should not be arbitrarily established.

Since swales, ditches, and downchutes convey concentrated flow from cover systems, they may need to be armored with turf reinforcement mat, riprap, or other material (see Section 2.2.5.7) to have adequate resistance to erosion. Extra erosion control measures may also be required at surface drainage system transitions (e.g., at the intersection of a swale and a downchute or down pipe).

Surface drainage system design typically involves the following general steps: (i) divide the cover system into several distinct drainage areas, as necessary; (ii) estimate the hydrologic properties of each area using size, soil type, and vegetative cover type; (iii) evaluate the rate of runoff from the design storm for each drainage area and the peak rate of runoff at each surface drainage system component; and (iv) size each component of the surface drainage system to handle the estimated peak flow associated with it. When the drainage system includes a sedimentation pond for stormwater management, the required storage volume of the pond also needs to be evaluated.

The design storm is usually specified for temporary and permanent conditions in federal, state, and local waste management, flood control, and soil conservation regulations. For example, federal regulations for MSW landfills (40 CFR §258.26) and HW landfills (40 CFR §264.301(h) and 40 CFR §265.301(h)) require these facilities to be designed to manage at least the 24-hour storm with a 25-yr return period. For containment applications with a higher level of risk to

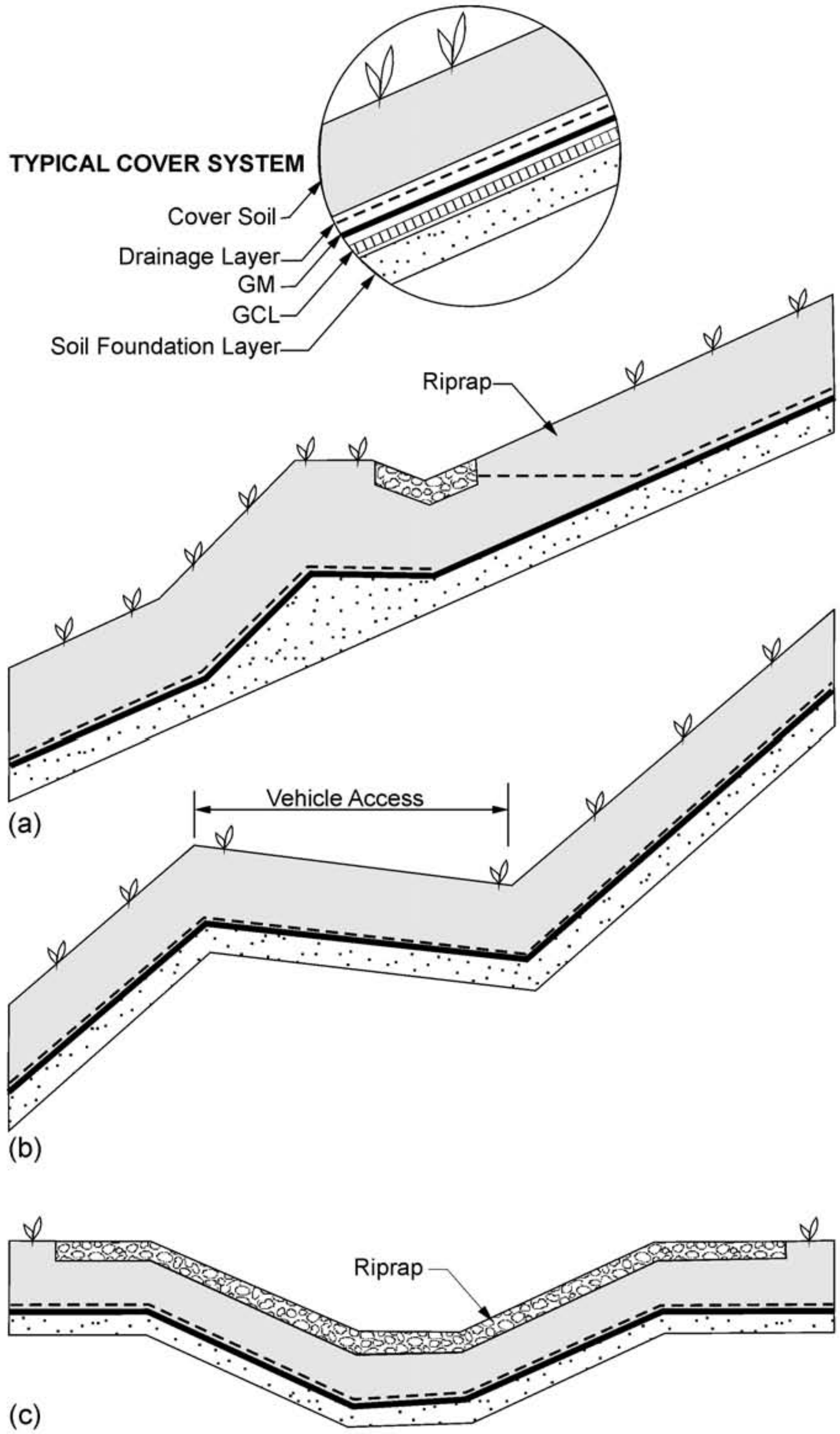


Figure 2-5. Details of Typical Swales and Ditches for Cover Systems (from Koerner and Daniel, 1997): (a) Swale Constructed with Add-on Berm; (b) Swale Constructed by Benching Sideslopes; and (c) Ditch Sometimes Constructed Adjacent to Access Road.

human health and the environment, such as for low-level radioactive waste disposal facilities, the design storm may be developed based on human health risk, statistical analysis of precipitation events, the PMP event, and other factors. As an example, the 2,000-yr design storm was considered when designing the on-site disposal facility at the DOE Fernald Environmental Management Project site.

Several urban drainage models are available for surface-water analysis for small (i.e., less than about 500 ha) urban watersheds. Two of the most commonly used models are: (i) the "rational method"; and (ii) the USDA Soil Conservation Service (SCS) Technical Release Number 55 (TR-55) method. (Note that the SCS is now the NRCS.) Both of these methods are described below.

The "rational method" is one of the simplest and best-known analysis methods routinely applied in urban hydrology. It is commonly used in civil engineering applications and is a method approved by the DOE (1989) for design of cover systems for sites regulated by the Uranium Mill Tailings Radiation Control Act (UMTRCA) of 1978 (i.e., Uranium Mill Tailings Remedial Action (UMTRA) sites). The rational method is based on the assumption that rainfall occurs uniformly over the watershed and at a constant intensity for a duration equal to the time of concentration. This method is typically used for areas under 40 ha in size. Using the rational method, the peak rate of runoff, q ($m^3/s/m$), is calculated as:

$$q = c\, i_r\, A_b\, F \qquad \text{(Eq. 2.1)}$$

where: c = runoff coefficient (dimensionless) and is equal to runoff divided by precipitation, i_r = rainfall intensity (m/s) for the period of interest; A_b = area of the drainage basin or subbasin per basin or subbasin width (m^2/m); and F = flow concentration factor (dimensionless).

Input values to the rational equation are as follows:

- The runoff coefficient is a function of ground cover, soil antecedent moisture, ground slope, and other factors. Runoff coefficient values are given in many hydrology textbooks and can range from near zero for shallow-sloping, grassed sandy soils to essentially 1.0 for impervious cover. Typical runoff coefficient values for different vegetation and slope conditions are shown in Table 2-2. For storms with return periods longer than 100 years, DOE recommends the use of c = 1.0 (DOE, 1989).
- Rainfall intensity is calculated as:

$$i_r = d / t_c \qquad \text{(Eq. 2.2)}$$

where: d = depth of rainfall in time of concentration from a storm with a certain return period (m); and t_c = time of concentration (s). The equation used to calculate the time of concentration depends on the surface layer material. For a soil, vegetated, or paved surface layer, the time of concentration can be calculated using the method of Brant and Oberman presented in DOE (1989):

$$t_c = 0.0328\, C_s \left[\frac{L_f}{S\left(i_r\right)^2} \right]^{1/3} \qquad \text{(Eq. 2.3)}$$

where: C_s = surface layer coefficient (dimensionless) and is 0.5 for paved areas, 1.0 for unvegetated soil; and 2.5 for turf; L_f = length of overland flow path (m); S = slope inclination (dimensionless); and all other terms are as defined previously. For a riprap surface layer, the time of concentration can be calculated using the method of Kirpich presented in U.S. Nuclear Regulatory Commission (NRC) (1990):

$$t_c = 0.0192 \left[\frac{\left(L_f\right)^3}{H_f} \right]^{0.385} \qquad \text{(Eq. 2.4)}$$

where: H_f = elevation difference along flow path (m), and all other terms are as defined previously. Whatever the surface layer, DOE (1989) recommends that the minimum time of concentration used in Eq. 2.2 be no less than 150 seconds. This is because for very small values of t_c, small decreases in t_c will cause relatively large increases in i_r, resulting in over-conservative estimations of the peak rate of runoff. Values for d in Eq. 2.2 are obtained from rainfall intensity maps (e.g., Hershfield, 1961; Miller et al., 1973; Hansen et al., 1982).

- The flow concentration factor accounts for flow possibly concentrating in rills and gullies. When calculating the peak rate of runoff to size drainage structures, F = 1. When evaluating the potential for gully formation (see Section 2.2.5.5), the flow concentration factor generally ranges between 1 and 3. For vegetative covers, Caldwell and Reith (1993) recommend using flow concentration factor values between 2 and 3. For riprap-lined channels, Abt et al. (1987, 1988) recommend using values between 1 and 3.

Table 2-2. Runoff coefficient values (modified from Barfield et al., 1983).

Vegetation and Slope Conditions	Soil Texture		
	Open sandy loam	Clay and silty loam	Tight clay
Woodland			
Flat, 0-5% slope	0.10	0.30	0.40
Rolling, 5-10% slope	0.25	0.35	0.50
Hilly, 10-30% slope	0.30	0.50	0.60
Pasture			
Flat, 0-5% slope	0.10	0.30	0.40
Rolling, 5-10% slope	0.16	0.36	0.55
Hilly, 10-30% slope	0.22	0.42	0.60
Cultivated			
Flat, 0-5% slope	0.30	0.50	0.60
Rolling, 5-10% slope	0.40	0.60	0.70
Hilly, 10-30% slope	0.52	0.72	0.82

The TR-55 method (SCS, 1986a) is based on the unit hydrograph method of analysis, and, thus, unlike the rational method, it can be used to calculate runoff volume and sediment pond storage volume as well as the peak rate of runoff. It also can better accommodate sites with varying topography and surface layer characteristics. Like the rational method, TR-55 starts with a "runoff coefficient", called a "runoff curve number"(CN) in TR-55, and a rainfall amount uniformly imposed on a watershed over a specified time. At the start of a precipitation event, some rainfall is considered lost to plant interception, evaporation, infiltration into the surface soil, and storage in surface depressions. After the initial loss, called the "initial abstraction" is satisfied, any additional rainfall may generate runoff. TR-55 calculates the runoff volume considering the initial abstraction and then transforms the runoff into a hydrograph using unit hydrograph theory and routing procedures that depend on runoff travel time through each segment of the watershed. Four different unit hydrographs are used to represent storm events across the U.S. Two of the rainfall distributions, Types IA and I, are representative of the Pacific maritime climate that occurs in Alaska, the western half of Washington and Oregon, and most of California. The Type 3 distribution is representative of the Atlantic and Gulf of Mexico coastal areas. The Type 2 distribution is similar to the Type 3 and occurs in the rest of the country. After hydrographs for watershed segments have been routed to a specific location, the peak runoff rate at that location can be calculated by adding the hydrographs.

Once the design flow rate is determined, the surface drainage system can then be designed to handle the flow. Open channel flow in swales, ditches, or downchutes is analyzed for the depth and velocity of water to ensure that the system has adequate capacity to convey flow with sufficient freeboard and that flow velocities are not greater than those specified for the specific drainage structures. The book by Chow (1959) is often used as a reference for analyzing open channel flow. Down pipes can usually be designed using open channel flow equations. Standard equations for flow in pipes are presented in numerous fluid hydraulics texts and provided by the pipe manufacturers.

2.2.5 Erosion Protection

2.2.5.1 Overview

Excessive erosion of the surface layer has been a significant problem for a number of cover systems. Gullies extending to a depth of 100 to 200 mm are not unusual. In the extreme, the underlying drainage and barrier layers can be eroded. Although erosion problems can often be addressed as a maintenance activity, there have been instances of major erosion that displaced hundreds of cubic meters of soil from inadequately protected landfill covers. Swope (1975) studied 24 landfill cover systems in the U.S. and found that 33% had slight erosion, 40% had moderate erosion, and more than 20% had severe erosion. Johnson and Urie (1985) report that erosion can be made more severe by the installation of a hydraulic barrier within a landfill cover system. Without an overlying drainage layer, the barrier can cause the cover soils to become soaked. Saturation decreases soil strength, increases particle detachment, and increases erosion potential (NRCS, 1998a). Even in natural soil systems, cover soils over a compacted layer on a steep slope may slide downslope as a mass if the soils become saturated (NRCS, 1998a).

Gross et al. (2002) described several cases of significant cover system erosion, including one for a cover system with 60-m long 3H:1V sideslopes (see Section 7.6.2). This cover system included sand berms to divert surface-water runoff from the top deck of the landfill to riprap-lined downchutes on the landfill sideslopes. Sand add-on berms were also located at a few locations on the sideslopes. The sand berms on the top deck developed gullies at several locations allowing concentrated flow of runoff down the sideslopes. Though this cover system included a sand drainage layer, it was not designed to outlet on the cover system and did not have sufficient capacity to convey drainage from the cover system top deck and sideslopes. The combination of inadequate management of surface water, insufficient drainage layer capacity, and long steep sideslopes contributed to the erosion problems at the site (Figure 2-6).

2.2.5.2 Nature of Erosion

Soil erosion involves a process of both particle detachment and transport by water or wind. It is initiated by drag, impact, or tractive forces acting on individual particles of soil at the surface. Water erosion starts when raindrops impact soil particles, dislodging them and sending them

Figure 2-6. Deep Gullies Through the Topsoil and Sand Drainage Layers Exposed the GM Barrier on 60-m Long, 3H:1V Landfill Sideslopes.

upward into the air and some distance away. As water collects on the soil surface, it begins to run off in small rivulets and then sheets of uniform flow. The sheet flows carry soil particles dislodged during impact and particles dislodged by tractive forces exerted from the flow. As the sheet flows move downslope, the flows concentrate due to irregularities in the soil surface and topography. The resulting concentrated flows cut more deeply into the surface, creating small channels called rills that may be tens of millimeters deep. Rill erosion accelerates with increase in runoff, slope inclination, and slope length. Rills can be removed from a slope and will return in different patterns and shapes. If rill development is allowed to progress, the rills will form deep cuts in the soil surface and become gullies. Because of the high velocities of flow in gullies, massive removal of soil is possible. Gullies may be several feet or more deep and, unlike rills, can generally not be repaired with a simple tilling of the soil surface. They also grow and deepen, as sheet flow passing above the headcut of a gully exerts forces on the flow channel boundary and removes accumulated soil debris from the channel. The types of water erosion that may occur on a cover system are illustrated in Figure 2-7.

The erosion potential of soil is primarily a function of the size of the soil particles, interparticle cohesive forces, and the velocity of the transporting fluid (air or water). This relationship is illustrated in Figure 2-8. Erosion potential increases with decreasing particle size and increasing velocity of the transporting fluid. Clays, however, which have the smallest particle size, also have cohesion, meaning that they stick to each other, which helps to prevent erosion. Some sodium-rich clays do not adhere to one another very well and, therefore, are highly vulnerable to erosion. Such clay soils are called “dispersive clays.” Several tests exist to identify potentially dispersive clays (Sherard et al., 1976). Silt has a small particle size but lacks cohesion. Silt is, therefore, almost always highly erodible. Neither dispersive clays nor silts should be used for the surface layer, unless it can be clearly demonstrated that erosion will not be a problem.

In arid and semi-arid climates, which have sparse vegetation and dry surficial sediments, winds can cause significant erosion. Winds can pick up and carry in suspension the lighter, less dense soil constituents (e.g., organic matter, clays, and silts with particles sizes primarily less than 0.1 mm) (Gray and Sotir, 1996). This is why soil-gravel mixtures or gravel veneers are often considered as a surface layer for cover systems constructed at arid and semi-arid sites. By transporting the lighter soil particles, wind removes the most fertile part of the soil and lowers soil productivity (Lyles, 1975). The majority (approximately 62 to 97%) of wind-eroded soil is carried near the ground surface at heights less than 1 m. Windbreaks can be used to impede soil movement within this height interval. Though wind can cause significant soil loss, most erosion of soil covers in arid and semi-arid areas is caused by water.

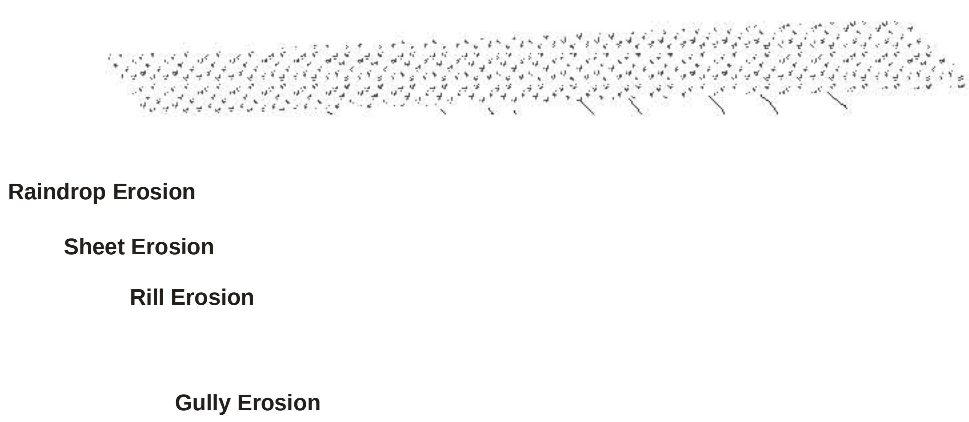


Figure 2-7. Types of Water Erosion That May Occur on a Cover System.

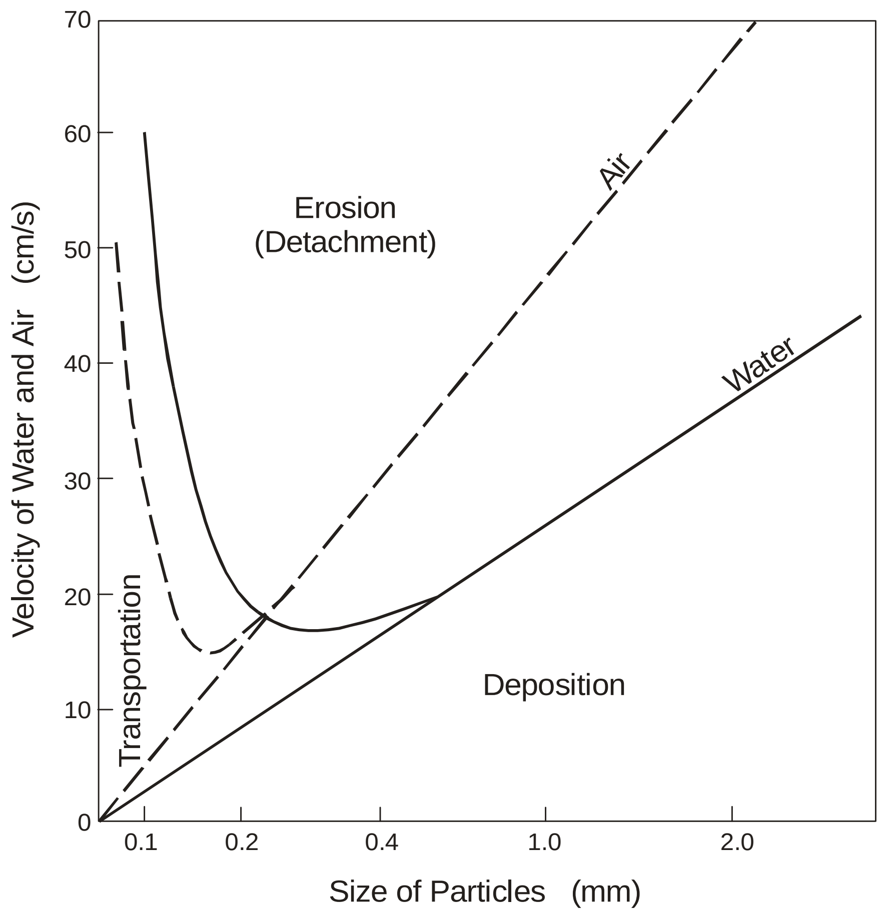


Figure 2-8. Relationship Between Erosion Mechanism (Air or Water), Particle Size and Fluid Velocity (Garrels, 1951 as referenced by Mitchell, 1993).

2.2.5.3 Short-Term and Long-Term Erosion

The cover system design should address the potential for short-term erosion (i.e., before a good stand of vegetation is established), and make use of temporary erosion-control measures as necessary. The design should also address long-term erosion after vegetation has been established especially for the site-specific rainfall or wind event. Erosion can be damaging not only to the cover system but also to areas into which eroded soil is deposited. It is also important that constructed erosion-control measures be installed correctly and maintained.

The timing for completion of cover system construction can impact the potential for erosion. In northern climates, the end of the construction season coincides with the end of the growing season. A common problem is that the cover system is seeded at a time of year that is not conducive to growing grass. In some climates, it may be impossible to initiate growth of the vegetative cover during certain parts of the year. It is recommended that construction be scheduled to allow vegetation to become established as soon as practicable and before the end of the growing season, if at all possible (EPA, 2002). If this is not achievable, erosion control materials may be needed to protect the surface layer.

The construction contractor is usually made responsible for maintaining temporary erosion control measures and repairing erosion damage during and shortly after construction. However, the contractor usually has only limited expertise in soil erosion control. Further, the contractor is not privy to the design decisions that affect the potential for severe short-term erosion. Thus, caution should be exercised in placing responsibility upon the contractor, who may be ill equipped to make informed decisions about appropriate erosion control measures. It is recommended that the design engineer consider carefully the potential for and consequences of short-term erosion and be proactive in specifying appropriate control measures (e.g., silt fences, rolled erosion control materials, sediment traps, hay bales, etc.) in the construction documents. The NRCS has developed conservation practice standards for a number of erosion control measures. Most state NRCS offices have websites with downloadable conservation practice standards. There may also be local requirements and standards for erosion control.

The NRCS (2000) makes the following recommendations to limit short-term erosion during construction:

- cover disturbed soils as soon as possible with vegetation or other materials (mulch) to reduce erosion potential;
- divert water from disturbed areas;
- control concentrated flow and runoff to reduce the volume and velocity of water and prevent formation of rills and gullies;
- minimize the length and steepness of slopes (e.g., use benches);
- prevent off-site sediment transport;
- inspect and maintain any structural control measures;
- where wind erosion is a concern, plan and install windbreaks;
- avoid soil compaction by restricting the use of trucks and heavy equipment to limited areas; and
- break up or till soils compacted by grading prior to vegetating or placing sod.

Long-term erosion is an important consideration in the design of the surface layer. In spite of the admittedly approximate nature of predictive equations for erosion control, most cover systems will require an analysis of long-term and, sometimes, short-term erosion. Typical design criteria are as follows:

- The design sheet and rill erosion rate should not be exceeded. Although it is advisable to select allowable rates of soil erosion on a project-specific basis, many design engineers follow the general guidance that the design sheet erosion rate not exceed 4.5 tonnes/ha/year (EPA, 1991).
- Using the sheet and rill erosion rate from this calculation, the thickness of cover soil at the end of the design life should be calculated to verify that there is adequate thickness

remaining and that sheet and rill erosion has not progressed through the cover soil and into the underlying layers. There should also be sufficient soil thickness to support vegetation and provide freeze-thaw protection of a CCL barrier, if present.

- The surface layer should resist gully formation under the tractive forces of runoff from site-specific design storm.
- If the potential for wind erosion is a concern (e.g., for some arid sites), wind erosion should also be evaluated.

The analysis of sheet and rill erosion, gully formation, and wind erosion is discussed in Sections 2.2.5.4, 2.2.5.5, and 2.2.5.6, respectively.

2.2.5.4 Sheet and Rill Erosion

2.2.5.4.1 Universal Soil Loss Equation

The average annual rate of soil loss by water erosion is often estimated by design engineers using some form of USDA's Universal Soil Loss Equation (USLE). The Revised USLE (RUSLE) (Renard et al., 1997) is an improved version of USLE and is currently recommended by the USDA for calculation of soil loss. RUSLE was developed to estimate soil loss caused by raindrop impact and sheet flow (collectively referred to as "interrill" erosion) plus rill erosion. It is derived from the theory of erosion processes, data from natural rainfall plots, and results for rainfall-simulation plots.

The RUSLE method is directed toward the prediction of erosion from construction sites, mined lands, reclaimed lands, and other disturbed areas. The areal extent and surfacing of many cover systems provide similar conditions to those for the above landforms. RUSLE, however, is limited to the estimation of average annual erosion rates and cannot establish erosion from specific events. The soil loss prediction represents an average for many storms and years. In addition, there is no direct method within the RUSLE procedure to determine the depth or magnitude of gully erosion on a cover system. It is, therefore, recommended that this method be used with another method that considers gully development.

The RUSLE is expressed as:

$$A_s = R_e\ K\ (LS)\ C\ P_c \qquad \text{(Eq. 2.5)}$$

where: A_s = average annual soil loss by sheet and rill erosion (tonnes/ha/yr); R_e = rainfall energy/erosivity factor (dimensionless) and is a measure of rainfall energy and intensity rather than just rainfall amount; K = soil erodibility factor (dimensionless), is a measure of the relative resistance of a soil to detachment and transport by water, and varies based on seasonal temperature and rainfall; LS = slope length and steepness factor (dimensionless) and is the ratio of soil loss from a given field slope to that from a slope that has a horizontal length of 22.1 m (from the origin of sheet flow to the point where runoff is concentrated in a defined channel) and a steepness of 9%; C = vegetative cover and management factor (dimensionless) and is the ratio of soil loss from land cropped under the specified conditions to the corresponding loss from clean-tilled, continuous fallow; and P_c = conservation support practice factor (dimensionless) and is the ratio of soil loss with a specific support practice to the corresponding soil loss with uphill and downhill tillage.

Input values for RUSLE are developed using site-specific information and the database that is part of the RUSLE computer program. Version 2 of the program can be downloaded from http://bioengr.ag.utk.edu/rusle2/.

Using A_s computed from Eq. 2.3, the thickness of cover soil at the end of the cover system design life can be calculated to verify that there is cover soil remaining and that the thickness of this remaining cover soil is sufficient to protect the any CCL component of the cover system.

2.2.5.4.2 Water Erosion Prediction Project (WEPP) Model

The WEPP model was developed in the 1980's when an increasing need for improved erosion prediction technology was recognized by the major research and action agencies of the United States Department of Agriculture and Interior, including the Agricultural Research Service (ARS), Natural Resource Conservation Service (NRCS), Forest Service (FS), and Bureau of Land Management (BLM). In 1985, these agencies embarked on a 10-year research and development effort to replace the Revised Universal Soil Loss Equation. Some of the differences between the WEPP model and the RUSLE are as follows:

- The RUSLE equation is based on undisturbed agricultural and rangeland top soil conditions, whereas any kind of soil can be described with WEPP. Thus, WEPP is well suited to describe a landfill cover, which is a disturbed condition.
- The WEPP model is capable of predicting erosion and deposition in more complex situations, such as when berms are involved. WEPP can predict the erosion on a cover as well as the deposition in berm channels in the watershed mode. The WEPP model's ability to determine runoff and channel flow can also aid in determining stability issues with berms, such as overtopping. RUSLE can only predict the upland erosion between berms.
- RUSLE can only predict average annual upland erosion. WEPP's climate generator includes stochastically generated events. This is an important point in arid environments where there are very few precipitation events annually, but when they occur, they are often torrential events that have major impacts on the site. Thus, a landfill in an arid climate is unlikely to fail in an average year, whereas, it is very likely to fail in a year when a major storm event has occurred. WEPP can predict the impacts from a major storm event, but RUSLE cannot.

Additional information regarding the WEPP model, software, and documentation can be found at: http://topsoil.nserl.purdue.edu/nserlweb/weppmain/wepp.html.

2.2.5.5 Gully Erosion

2.2.5.5.1 Overview

The concentration of runoff under many circumstances encourages the formation of rills, which, if unchecked, grow into gullies. This is arguably the most severe type of erosion of cover systems soils at landfill and waste remediation sites.

The dynamics of gully formation are complex and not completely understood. Gully growth patterns are cyclic, steady, or spasmodic and can result in the formation of continuous or discontinuous channels. Gully advance rates have been obtained by periodic surveys, measurements to steel reference stakes or concrete-filled auger holes, examination of gully changes from small-scale maps, or from aerial photographs. Studies are producing quantitative information and some procedures that combine empirically- and physically-based methods have been advanced. Vanoni (1975) presented six methods used for prediction of gully growth and/or gully head advance. They all follow some type of multiplicative or power law and are replete with empirical constants that are generally site specific. McCuen (1998) updated and further described gully erosion prediction equations with the observation that five factors underlie the relevant variables of the process: land use, watershed size, gully size, soil type, and runoff momentum. Having investigated the relevant factors, however, McCuen found that none of the equations treat all terms. Better methods of evaluating gully formation that are more physically based are needed.

The potential for gully development in vegetated soil surface layers has been assessed at landfill sites using the tractive force method described by Temple et al. (1987) and DOE (1989) and developed for channel flow (see Section 2.2.5.5.2), the Horton/NRC method for computing the critical distance for gully formation (NRC, 1990) (see Section 2.2.5.5.3), and the permissible velocity method described by Chow (1959) and NRC (1990) and also developed for channel flow (see Section 2.2.5.5.4). These methods are presented below and are based on the approach of NRC (1990) guidance. This approach is to prevent gully initiation during the occurrence of a single, extremely large, design rainfall. By designing for such an event, it is expected that smaller, continual events will have little or no cumulative influence on gully initiation. Of course, such a conservative approach results in relatively flat, and relatively short, slopes.

Similar approaches, typically using the permissible tractive force and velocity methods, can be used to design other types of surface layers. For example, design methodologies for riprap covering uranium mill tailings piles have been developed and used with apparent success. Nelson et al. (1986) discuss general design methodologies, and Abt et al. (1988) present design criteria based on flume tests. The NRC (NRC, 1990) recommends specific methodologies and equations for the calculations. For example, the Stephenson method, described by Abt et al. (1988) (see Section 2.2.5.5.5), can be used to select the mean particle diameter to withstand a design storm. The Stephenson method is recommended for evaluating the erosion resistance of a gravel or riprap layer with a slope inclination greater than 10% (NRC, 1990). For steeper slopes (e.g., slope inclinations greater than 5H:1V), the Hartung and Scheuerlein method (Hartung and Scheuerlein, 1970) has been used.

2.2.5.5.2 Tractive Force Method for Vegetated Surface Layers

The tractive force method (Temple et al., 1987; DOE, 1989) can be used to calculate the allowable shear stress, τ_a (kPa), of a vegetated surface layer as:

$$\tau_a = \tau_{ab} C_e^{\,2} \geq 0.9\,\text{kPa} \qquad \text{(Eq. 2.6)}$$

where: τ_{ab} = allowable shear stress for the surface layer with bare soil (kPa); and C_e = void ratio correction factor (dimensionless). Temple et al. (1987) and DOE (1989) provide graphs for both τ_{ab} and C_e values.

The allowable shear stress must be equal to or greater than the effective shear stress applied to the surface layer by the flowing water, τ_e (kPa):

$$\tau_a \geq \tau_e = \gamma_w\, D\, S \left(1 - C_F\right)\left(\frac{n_s}{n}\right)^2 \qquad \text{(Eq. 2.7)}$$

where: γ_w = unit weight of water (kN/m^3); D = flow depth (m); S = slope inclination (dimensionless); C_F = vegetal cover factor (dimensionless); n = Manning's roughness coefficient for the considered vegetative cover (dimensionless); and n_s = Manning's roughness coefficient for the bare soil (dimensionless). Guidance on the selection of values for the vegetal cover factor and the Manning's coefficients is provided by Temple et al. (1987) and DOE (1989).

The depth of flow can be calculated using the Manning's equation (DOE, 1989):

$$D = \left(\frac{q\,n}{S^{0.5}}\right)^{0.6} \qquad \text{(Eq. 2.8)}$$

where: q = peak rate of runoff (m^3/s/m) from Eq. 2.1 (and incorporating the flow concentration factor), and all other terms are as defined previously.

2.2.5.5.3 Horton/NRC Method for Vegetated Surface Layers

The Horton/NRC method (NRC, 1990) is also used for prediction of gully formation for vegetated surface layers. The method is used to estimate the critical distance, x_c (m), along a slope before gully formation begins. The slope lengths of a cover system should be designed to be less than x_c between runoff collection points (e.g., between drainage swales) to minimize the potential for gully development. The equation for x_c is as follows:

$$x_c = \frac{\tau_{ah}^{\;5/3}}{45\,F\,i_r\,n\left(f(S)\right)^{5/3}} \qquad \text{(Eq. 2.9)}$$

where: τ_{ah} = allowable shear stress for the Horton/NRC method (kPa); F = flow concentration factor (dimensionless) from Eq. 2.1; i_r = rainfall intensity (m/s) from Eq. 2.2; n = Manning's roughness coefficient for the considered vegetative cover (dimensionless), calculated using the tractive force method described in Section 2.2.5.5.1; and f(S) = slope function (dimensionless).

The allowable shear stress can be calculated as the minimum of:

$$\tau_{ah} = \frac{\tau_a}{(1 - C_F)\left(\frac{n_s}{n}\right)^2} \qquad \text{and} \qquad \text{(Eq. 2.10)}$$

$$\tau_{ah} = \tau_{va} = 0.75 C_I \qquad \text{(Eq. 2.11)}$$

where: τ_a, C_F, n_s, and n are calculated using the tractive force method described in Section 2.2.5.5.1; τ_{va} = limiting vegetal stress (stress at which vegetation will break) (kPa); and C_I = vegetal retardance curve index (dimensionless). Guidance on the selection of values for the vegetal retardance curve index is provided by Temple et al. (1987) and DOE (1989). Eq. 2.10 is based on allowable soil stress, and Eq. 2.11 is based on allowable vegetal stress.

The slope function can be calculated as follows (NRC, 1990):

$$f(S) = \frac{\sin\beta}{(\tan\beta)^{0.3}} \qquad \text{(Eq. 2.12)}$$

where: β = slope angle (degrees).

2.2.5.5.4 Permissible Velocity Method for Vegetated Surface Layers

The permissible velocity method (Chow; 1959; NRC, 1990) can also be used to assess the potential for gullies to form in a vegetated cover. The flow velocity of runoff should be less than the permissible velocity for the surface layer material. NRC (1990) recommends checking results of the Horton/NRC Method against those of the permissible velocity method.

The flow velocity, v (m/s), is calculated in the conventional manner:

$$v = q/D \qquad \text{(Eq. 2.13)}$$

where all other terms are as defined previously.

Permissible velocities recommended by SCS (1986b) for a range of vegetated cover conditions (e.g., grass type, surface layer slope, soil erosion sensitivity, etc.) in drainage channels are presented in Table 2-3. When the flow depth, D, is less than 1 m, NRC (1990) recommends that the permissible velocity in the channel be reduced by a reduction factor, R_f (dimensionless):

$$R_f = 1 + 0.46\ \log(D) \quad \text{for}\ \ 0.08\text{ m} \le D \le 1\text{ m}$$

$$R_f = 0.5 \quad \text{for}\ \ D < 0.08\text{ m} \qquad \text{(Eq. 2.14)}$$

Table 2-3. Permissible velocities recommended by SCS for vegetated drainage channels (modified from SCS, 1986b).

Vegetation Type	Slope Range (%)	Permissible Velocity[1]	
		Erosion resistant soils (ft/s)	Easily eroded soils (ft/s)
Bermudagrass	0-5 5-10 over 10	8 7 6	6 5 4
Bahiagrass Buffalograss Kentucky bluegrass Smooth brome Blue grama Tall fescue	0-5 5-10 over 10	7 6 5	5 4 3
Grass mixtures Reed canarygrass	0-5 5-10[2]	5 4	4 3
Lespedeza sericea Weeping lovegrass Yellow bluestem Redtop Alfalfa Red fescue	0-5[3]	3.5	2.5
Common lespedeza[4] Sudangrass[4]	0-5[5]	3.5	2.5

[1] Use velocities exceeding 5 ft/s only where good vegetated covers and proper maintenance can be obtained.
[2] Do not use on channel slopes steeper than 10%, except for vegetated sideslopes in combination with a stone, concrete, or highly resistant vegetative center section.
[3] Do not use on channel slopes steeper than 5%, except for vegetated sideslopes in combination with a stone, concrete, or highly resistant vegetative center section.
[4] Use annuals on mild slopes or as temporary protection until permanent vegetated covers are established.
[5] Use on slopes steeper than 5% is not recommended.

2.2.5.5.5 Stephenson Method for Gravel or Riprap Surface Layers

The Stephenson method (NRC, 1990) is used to compute the minimum gravel or riprap mean particle diameter, D_{50} (mm), to withstand the peak rate of runoff:

$$D_{50} = 1{,}000 \left[\frac{q(\tan\beta)^{7/6}\, n_p^{1/6}}{C_d\, g^{1/2} \left[\left(1 - n_p\right)\left(G_s - 1\right) \cos\beta \left(\tan\phi - \tan\beta\right) \right]^{5/3}} \right]^{2/3} \qquad \text{(Eq. 2.15)}$$

where: n_p = porosity of gravel or riprap layer (dimensionless); C_d = empirical factor (dimensionless) ranging from 0.22 for gravel to 0.27 for crushed granite (Stephenson, 1979); g = acceleration of gravity (9.81 m/s^2); G_s = specific gravity of gravel or riprap (dimensionless); ϕ = angle of repose of gravel or riprap (degrees); and all other terms are as defined previously. Guidance on the selection of values for the porosity and angle of repose of the gravel or riprap is provided by Abt et al. (1987) and NRC (1990). Gravel or riprap with a mean particle diameter of

D_{50} will be on the threshold of movement under flow q. The surface layer will collapse at a flow varying from 1.2q (for gravel) to 1.8q (for crushed granite) (Stephenson, 1979).

2.2.5.6 Wind Erosion

2.2.5.6.1 Revised Wind Erosion Equation.

The average annual rate of soil loss by wind erosion (for that portion of sediment that moves between the soil surface up to a height of 2 m) can be estimated using the Revised Wind Erosion Equation (RWEQ) computer program (Fryrear et al., 1998). RWEQ was developed for agricultural fields and is currently being used by the NRCS to assess soil loss. The model is derived from the theory of erosion processes and data from laboratory and field wind tunnel studies.

Using finite difference techniques, RWEQ solves an equation for horizontal mass transport across an eroding surface:

$$\frac{dQ(x)}{dx} = \frac{2x}{s(x)^2}\left(Q_{max}(x) - Q(x)\right) \qquad \text{(Eq. 2.16)}$$

where: Q(x) = mass transport of soil (kg/m) at downwind distance x; x = downwind distance (m); $Q_{max}(x)$ = maximum mass transport of soil (kg/m) at downwind distance x; and s(x) = field length scale (m).

The maximum mass transport of soil, Q_{max} (kg/m), is calculated as:

$$Q_{max} = 109.8\ (WF\ EF\ SCF\ K'\ COG) \qquad \text{(Eq. 2.17)}$$

where: WF = weather factor (kg/m) and is a function of wind speed, soil wetness, snow cover, and other factors; EF = erodible fraction (dimensionless), is the fraction of the surface 25 mm of soil that is smaller than 0.84 mm, and is computed empirically as a function of the percentages of clay, silt, and sand-sized particles, organic matter, and calcium carbonate in the soil; SCF = soil crust factor (dimensionless) and is computed empirically as a function of the percentages of clay and organic matter in the soil; K' = soil roughness factor (dimensionless) and is a function of soil clod roughness, ridge height and spacing, and other factors; and COG = combined crop factors (dimensionless) and is related to plant canopy and residues.

RWEQ uses monthly weather data, soils and field data, and management inputs to assess wind erosion. The management inputs include cropping systems tillage and operation dates, windbarrier descriptions, and irrigation information. Time periods from the management input file are used to partition the weather factor for each management time period. The dominant wind direction is assessed, and the wind factor is computed for four directions using weather data and considering hill and wind barrier effects, snow cover, and soil moisture content. Operation dates are also used to determine time periods for computation of residue decay, soil roughness decline, and soil erosion. Residue decomposition is computed for each period based on weather conditions and accumulated decomposition days since crop harvest. Soil roughness is decayed for each time period based on rainfall characteristics and clay content. The residue and soil roughness for each time period are used with the length of eroding field to determine the average soil erosion for that field length. The soil erosion from the different time periods are then summed to get the average annual rate of soil loss by erosion.

Input values for RWEQ are developed using site-specific information and the database that is part of the RWEQ computer program. The program is available for download from http://www.csrl.ars.usda.gov/wewc/rweq/readme.htm.

2.2.5.6.2 Wind Erosion Prediction System

The Wind Erosion Prediction System (WEPS) is a process-based, daily time-step, computer model that simulates weather, field conditions, and erosion. WEPS development involves an Agricultural Research Service (ARS) led, national multidisciplinary team of scientists, intended to replace the predominately empirical Wind Erosion Equation (WEQ) (Woodruff and Siddoway, 1965). Agencies involved include the ARS, Natural Resource Conservation Service (NRCS), and Forest Service (FS) from the U.S. Department of Agriculture, along with the EPA and Bureau of Land Management (BLM). The purposes of WEPS are to improve technology for assessing soil loss by wind from agricultural fields and to provide new capabilities such as assessing soil movement, plant damage, calculating suspension loss, and estimating PM-10 (particles less than 10 microns in diameter) when wind speeds exceed the erosion threshold (Wagner, 1996)

WEPS consists of an instructional program, a user-interface program, seven submodels, and an output section. WEPS allows users to input their own data files or use previously prepared data base files. It also possesses the ability to provide users with individual values for suspension, saltation, and surface creep. WEPS' seven submodels, each based on the fundamental processes which occur in the field, are used to predict and give estimates for wind erosion.

More information on WEPS and wind erosion can be found at the USDA-ARS Wind Erosion Research Unit (WERU), available at http://www.weru.ksu.edu/.

2.2.5.7 Erosion Control Materials

One often-effective means for controlling erosion is through the use of erosion control materials. Such materials can be temporary or permanent and, depending on the materials, are placed before, during, or after seeding. Once installed, the measures may require maintenance to maintain their effectiveness.

2.2.5.7.1 Temporary Erosion Control Materials

Temporary erosion and revegetation materials (TERMs) consist of materials that are in whole or part degradable. TERMs provide temporary erosion control and are either disposable after a given period, or only function long enough to facilitate vegetative growth. After the growth is established, the TERMs are no longer needed. Some of the TERMs are completely biodegradable, but others are only partially so. Theisen (1992) groups the various materials listed in the upper part of Table 2-4 as being in the TERM category.

The first two products listed in the TERM category in Table 2-4 consist of traditional methods of erosion control using straw, hay, or mulch loosely bonded by asphalt or adhesive. The stability of this type of material is may not be very good. Geofibers in the form of short pieces of fibers or microgrids can be mixed into soil with machines or rototillers to aid in laydown and continuity. The fiber or grid inclusions provide for greater stability over straw, hay, or mulch broadcast over the ground surface.

Table 2-4. Erosion control materials (after Theisen, 1992)

Type of Material	Examples of Material
Temporary Erosion and Revegetation Materials (TERMs)	Straw, hay, and hydraulic mulches Tackifiers and soil stabilizers Hydraulic mulch geofibers Erosion control meshes and nets Erosion control blankets Fiber roving systems
Permanent Erosion and Revegetation Materials (PERMs) - Biotechnical Related	UV-stabilized fiber roving systems Erosion control revegetation systems Turf reinforcement mats Discrete length geofibers Vegetated geocellular containment systems
Permanent Erosion and Revegetation Materials (PERMs) - Hard Armor Related	Geocellular containment systems Fabric formed revetments Vegetated concrete block systems Concrete block systems Stone riprap Gabions

Erosion control meshes and nets are biaxially oriented materials manufactured from polypropylene or polyethylene. These materials do not absorb moisture, nor do they shrink or expand over time. They are lightweight and are stapled to the seeded ground using hooked nails or U-shaped pins. The purpose of affixing the material to the ground is to improve stability. Erosion control blankets are also biaxially oriented nets or meshes manufactured from polypropylene or polyethylene. With these materials, a blanket of straw, excelsior, cotton, coconut, or polymer fiber is attached to one or both sides of the net or mesh. The fibers are held to the net or mesh by glue, lock stitching, or other methods.

Fiber roving systems are continuous strands, or yarns, usually of polypropylene, that are fed continuously over the surface to be protected. They can be placed by hand or using compressed air. After placement on the ground surface, emulsified asphalt or other soil stabilizer is used for controlled positioning.

2.2.5.7.2 Permanent Erosion Control Materials

Permanent erosion control materials (PERMs) can be biotechnical or hard armor (Table 2-4). The biotechnical materials are discussed first.

Most of the biotechnical materials are polymer products that control erosion, aid in vegetative growth, and eventually become entangled with the vegetation to provide reinforcement to the root system. As long as the material is shielded from sunlight, via shading and soil cover, it will not degrade (at least within the limits of polymeric materials). The polymers can be stabilized with carbon black and/or chemical stabilizers. The seed is usually applied after the PERM is placed.

Erosion control revegetation mats and turf reinforcement mats are closely related materials, the basic difference being that erosion control revegetation mats are placed on the ground surface with a soil infill, while turf reinforcement mats are placed on the ground surface with soil filling in and above the material. Thus, turf reinforcement mats can be expected to provide better vegetative entanglement and longer performance. Seeding is usually done prior to installation of an erosion control revegetation mat, but while backfilling within the structure of turf reinforcement mats.

Discrete length geofibers are short pieces of polymer yarns mixed with soil to provide a tensile strength component that can resist forces such as those occurring at athletic fields and on slopes. Vegetated geocellular containment systems consist of three-dimensional cells of GMs or GTs, which are filled with soil and vegetated (Figure 6-33).

Hard armor systems provide their own erosion protection, independent of vegetation. Geocellular containment systems are permanent when the infill material is concrete. Fabric formed revetments are GTs that are filled with concrete or grout. As the GT deteriorates over time from UV degradation, the concrete or grout is left behind.

Numerous concrete block systems are available for erosion control. Hand placed interlocking masonry blocks are popular for low traffic pavement areas such as driveways. The voids in the blocks and between them are usually vegetated. Alternatively, the system can be factory fabricated as a unit, brought to the job site, and placed on prepared soil. The prefabricated blocks are either laid on, or bonded to, a GT substrate. The finished mat can bend and torque by virtue of the blocks being articulated with joints, weaving patterns, or cables. A concrete cribwall has also been used as a surface layer (Figures 6-30 and 6-31).

Stone riprap can be very effective as was discussed earlier. A GT placed on the soil surface before placement of riprap serves as a filter and separator.

Gabions consist of discrete cells of wire netting filled with hand-placed stone. The wire is usually galvanized steel hexagonal wire mesh, but in some cases can be a plastic geogrid.

2.2.6 Construction

If topsoil is used to construct the surface layer, the soil is only compacted nominally, if at all, to facilitate plant root development. Even moderate amounts of compaction can result in decreased root depth and density. As described by the NRCS (1996), compaction restricts rooting depth, which reduces the uptake of water and nutrients by plants. It also decreases infiltration, which increases runoff and, thus, erosion potential. To promote the growth of vegetation, it is generally recommended that cover soils be placed at bulk densities less than the values given in Table 2-5.

A gravel-soil mixture will require some compaction, but heavy compaction is neither necessary nor desired. Rock riprap is normally placed loosely with little or no compaction. Where asphaltic concrete has been used as the surface layer, road-paving equipment was used for construction.

2.2.7 Maintenance

Maintenance is discussed in Chapter 9. The most important maintenance activities for the surface layer involve maintaining the intended vegetative cover and the erosion control measures, repairing erosion gullies, filling surface depressions caused by localized settlement, and, as an associated activity, maintaining and repairing surface-water management structures.

Table 2-5. Minimum soil bulk density at which a root restricting condition may occur (NRCS, 1996).

Soil Texture	Bulk Density (g/cm^3)
Coarse, medium, and fine sand and loamy sands other than loamy very fine sand	1.80
Very fine sand, loamy very fine sand	1.77
Sandy loam	1.75
Loam, sandy clay loam	1.70
Clay loam	1.65
Sandy clay	1.60
Silt, silt loam	1.55
Silty clay loam	1.50
Silty clay	1.45
Clay	1.40

2.2.8 Monitoring

Monitoring is discussed in Chapter 8. The surface layer should be monitored to identify problems with excessive erosion, excessive differential settlement, or slope instability, assess the health of the vegetative cover, and evaluate gas emissions, if gases are a concern. If the cover system water balance is being assessed, the surface layer moisture content or matric potential and surface-water runoff may also be monitored.

2.3 Protection Layer

The protection layer lies directly beneath the surface layer and, in some cases, may be combined with the surface layer to form the “cover soil”. The primary functions of the protection layer are to protect the underlying cover system components and to temporarily store water that has percolated through the surface layer until it can be returned to the atmosphere by ET. The underlying layers may need protection from erosion, exposure to wet-dry cycles, exposure to freeze-thaw cycles, exposure to ultraviolet light, and biointrusion by plant roots, burrowing animals, and humans. The storage of water in the protection layer provides a water reservoir to support plant growth and reduces infiltration into underlying cover system components. The protection layer may also serve to attenuate emissions of radon gas for those wastes that emit radon.

2.3.1 General Issues

Occasionally, cover systems are designed without a protection layer. In such cases the surface layer is placed directly on a drainage layer or hydraulic barrier. This design approach is usually not recommended because erosion gullies may sometimes cut through the surface layer (if it is relatively thin) and expose or even erode the underlying layers. The underlying layers may then become damaged under prolonged exposure to the environment. For example, exposed CCLs will usually develop desiccation cracks. As discussed in Section 7.2, even up to 0.75 m of cover soil may not be sufficient to protect underlying CCLs from degradation. Geosynthetics are also vulnerable to degradation from exposure to ultraviolet light. If the surface layer is vegetated topsoil and there is no protection layer to provide stored water to plants, the vegetation may experience excessive stress and even die when the topsoil moisture content decreases to low levels. In most situations, the only justification for omitting the protection layer is if the underlying layers require no protection and the surface layer is not vegetated.

With this in mind, the most important concerns with respect to the protection layer are generally the level of protection required by the underlying layers and the water storage capacity required to support any vegetation.

2.3.2 Elements of Design

Important questions that typically need to be addressed when considering the design of the protection layer include:

- What materials are available to construct the protection layer?
- What thickness of protection layer material is needed?
- How should the protection layer be constructed?
- What type and frequency of maintenance should be employed?
- What type and frequency of monitoring should be employed?

2.3.2.1 Materials

The protection layer is usually constructed from on-site or locally available soil. As discussed in Section 2.2.2.2.1, medium-textured soils, such as loams, have the best overall characteristics for seed germination and the development of plant root systems. Fine-grained soils, such as silts and clays, have excellent water-holding capability, which provides roots with water for plant growth but limits the transport of oxygen to plant roots. In addition, fine-textured soils are vulnerable to cracking when desiccated. Conversely, coarse-grained soils, such as sands and gravels, have low water retention capacity and high saturated hydraulic conductivity. Coarse-grained soils can drain and dry out quickly, resulting in an insufficient moisture supply for plants. For example, there have been instances in which cover soils at landfills became so dry that cover system irrigation was required to maintain adequate soil moisture to support grass (post-closure maintenance of a vegetative cover was required in the permits for the facilities). The addition of water to the surface of a cover system is generally not recommended because one of the primary purposes of a cover system is usually to limit infiltration of water into the underlying waste.

If a soil protection layer is placed above a drainage layer, filter criteria for the two layers should be met. Filter criteria can be met in one of two ways: (1) ensuring that the materials themselves meet the criteria (thus eliminating the need for a filter); or (2) installing a soil or GT filter at the interface between the layers. Filters are discussed in Section 4.7.

If the primary role of the protection layer is to prevent biointrusion, cobbles, asphaltic concrete, recycled concrete pavement, or similar materials are typically required. If both vegetative support and preventing biointrusion are critical, the protection layer may consist of two or more components, for example a layer of cobbles overlain by a GT filter and then a silty loam soil layer.

2.3.2.2 Thickness

The required thickness of the protection layer depends on many factors including:

- need to protect underlying layers from damage due to wet-dry and freeze-thaw cycles;
- maximum depth of frost penetration;
- need to prevent accidental human intrusion, penetration by burrowing animals, or root penetration into underlying materials;
- need to support vegetative growth by accommodating plant roots;
- need to temporarily store water in the protection layer to attenuate rainfall infiltration into the underlying layers and to sustain vegetation through dry periods;
- need to provide other types of protection unique to a particular waste (e.g., attenuate radon emissions if the underlying waste emits radon); and
- need for a capillary barrier (discussed in Section 3.3), if this is a design strategy.

As previously mentioned in Section 2.2.2.3, thicknesses of cover soils (surface layer plus protection layer) are often in the range of 0.45 to 0.6 m, although thicknesses greater than 1 m are sometimes necessary to provide adequate rooting depth, soil moisture storage capacity, and freeze-thaw protection or to meet other design requirements. The protection layer may need to be still thicker if both vegetative support and protection from biotrusion is required. As will be subsequently discussed, the typical thickness of a biointrusion-resistant cobble layer is on the order of 0.5 to 1 m.

2.3.2.2.1 Desiccation Protection

Depending on the cover system components, the protection layer may need to be designed to be thick enough to protect the underlying layers from desiccating. For example, the hydraulic integrity of a CCL will be compromised if it is allowed to desiccate and crack after being exposed to wet-dry and/or freeze-thaw cycles. The degree of desiccation protection required for a CCL depends upon whether the CCL is covered with a GM. If the barrier is a GM/CCL composite, the GM will provide the CCL with some protection from desiccation (see Section 7.2). However, a soil protection layer with a thickness on the order of 0.45 m or more is still required over the GM.

If the hydraulic barrier is a CCL alone, the problem of protecting the CCL from desiccation is particularly challenging. As discussed in Section 2.5.2.6, cover soils have exhibited severe desiccation to depths of up to 1 m, and possible deeper. It thus appears that the thickness of protection layer required to slow desiccation of an underlying CCL that is not covered with a GM for a time period of 30 years or more is at least 1 m, and probably more. Because only limited information is available on this subject, a conservative approach is recommended.

Depending on the chemistry of the permeating water, GCLs may or may not be vulnerable to permanent damage from desiccation (see Section 2.5.2.6). If the permeant contains cations that may exchange with the sodium in the GCL bentonite, the barrier will loose some capability to swell and recover from desiccation over time. As described in Section 2.5.2.6, GCLs have been damaged for this reason in at least several field installations.

If it is desired to protect a CCL, GCL, or other type of barrier from desiccation (and it almost always is desired to do so), the best approach is to place a GM over the barrier, and then cover the GM with soil.

2.3.2.2.2 Frost Penetration Protection

The protection layer is generally designed with the intent of preventing underlying layers from freezing. This is especially a concern in northern climates. As temperatures drop and soil layers within the cover system freeze, water drawn towards the freezing front can cause desiccation cracking, freeze-thaw cracking, and frost heaving. As discussed in Section 2.5.2.7, desiccation and frost cracking may cause CCLs located within the frost zone to have increased permeability to water and gas. Neither GCLs nor GMs appear to be vulnerable to freeze-thaw damage. However, based on the information presented in Section 2.3.2.2.1, if freezing temperatures cause a GCL to desiccate, it may become damaged if it rehydrates with water containing certain exchangeable cations. To avoid damage to a CCL, the protection layer and overlying surface layer should be thick enough to place any CCL below the maximum depth of frost penetration. If may be advisable to also use this approach for GCLs. Alternatively the GCL may be covered with a GM to reduce its potential to desiccate due to freezing conditions.

The protection layer should generally prevent the drainage layer (if one is present) from freezing as well, particularly on relatively steep sideslopes. If the drainage layer freezes, it is not functional for part of the year. During the thaw period, it is particularly important that the drainage layer work properly, i.e., drain freely, and that the protection layer be sufficiently thick to provide the protection that is required. If the drainage layer is to be within the depth of frost penetration, the layer should be made permeable enough that it drains rapidly and has little capillarity (i.e., has a low field capacity) so that the voids in the layer are filled with air and not water during the winter months.

The depth of frost penetration in a cover system may vary from that of the native deposits due to differences in soil texture, moisture content, density, organic matter, and other factors. For example, because clay particles have a higher insulation value than silt or sand particles and since clay soils normally hold more moisture than silts and sands, the depth of frost penetration is usually greater in silt and sandy soils (light-textured soils) than in clays and silty clays (heavy-textured soils).

There are several techniques available for estimating the depth of frost penetration. One common practice is to use frost penetration maps for native soils, such as the one in Figure 2-9. This map shows contours of maximum frost penetration depth based on estimates made by the U.S. Weather Bureau. Frost penetration maps may be of limited accuracy. According to DeGaetano et al. (1997), available maps for maximum frost penetration depths in the U.S. are based on unofficial, poorly documented, and antiquated (1899-1938) measurements.

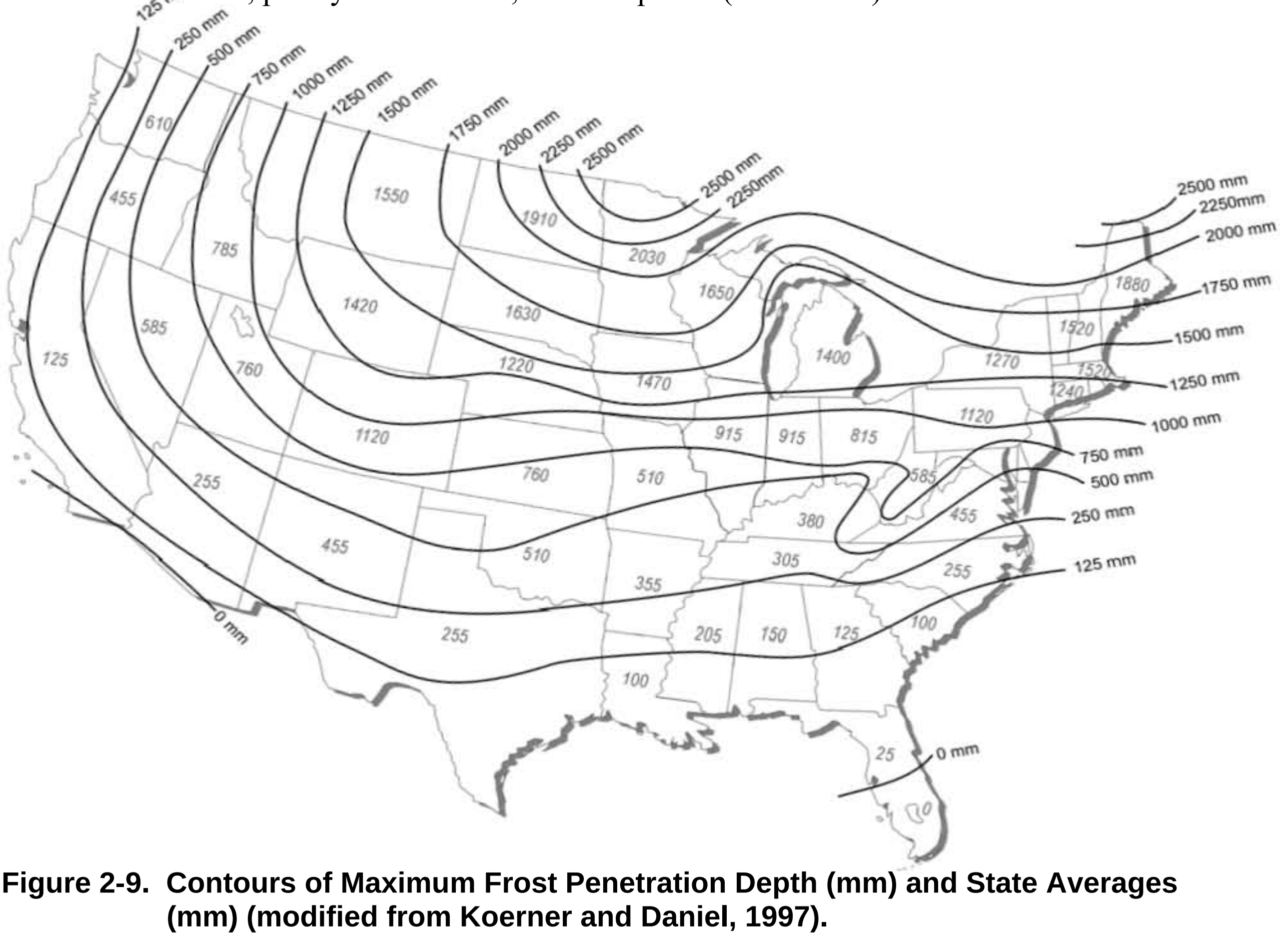


Figure 2-9. Contours of Maximum Frost Penetration Depth (mm) and State Averages (mm) (modified from Koerner and Daniel, 1997).

As an alternative to using frost penetration maps, the depth of frost penetration may be computed using the air freezing index and other site-specific factors. The air freezing index is the total number of degree-days of freezing for a given winter. One degree-day of freezing results when the mean air temperature measured at 137.3 cm above the ground for one day is 1F degree below 32°F. Air freezing index data and statistics (based on 1951-1980 data) for a number of weather stations across the U.S. can be downloaded from the National Climatic Data Center (NCDC) website (http://lwf.ncdc.noaa.gov/oa/fpsf/fpsf.html); data documentation for the air freezing index statistics is presented by Steurer (1998). The NCDC website also includes a map of 100-year return period air freezing indices (Figure 2-10). There are a number of semi-empirical and physical models for evaluating the frost penetration depth using the air freezing index. The most commonly used model to evaluate the frost depth is the modified Berggren method. This semi-empirical method, which is not presented in this guidance document, considers the thermal properties of the soil layers, the air freezing index, and other parameters. Information on the Berggren method can be found in Aldrich and Paynter (1953).

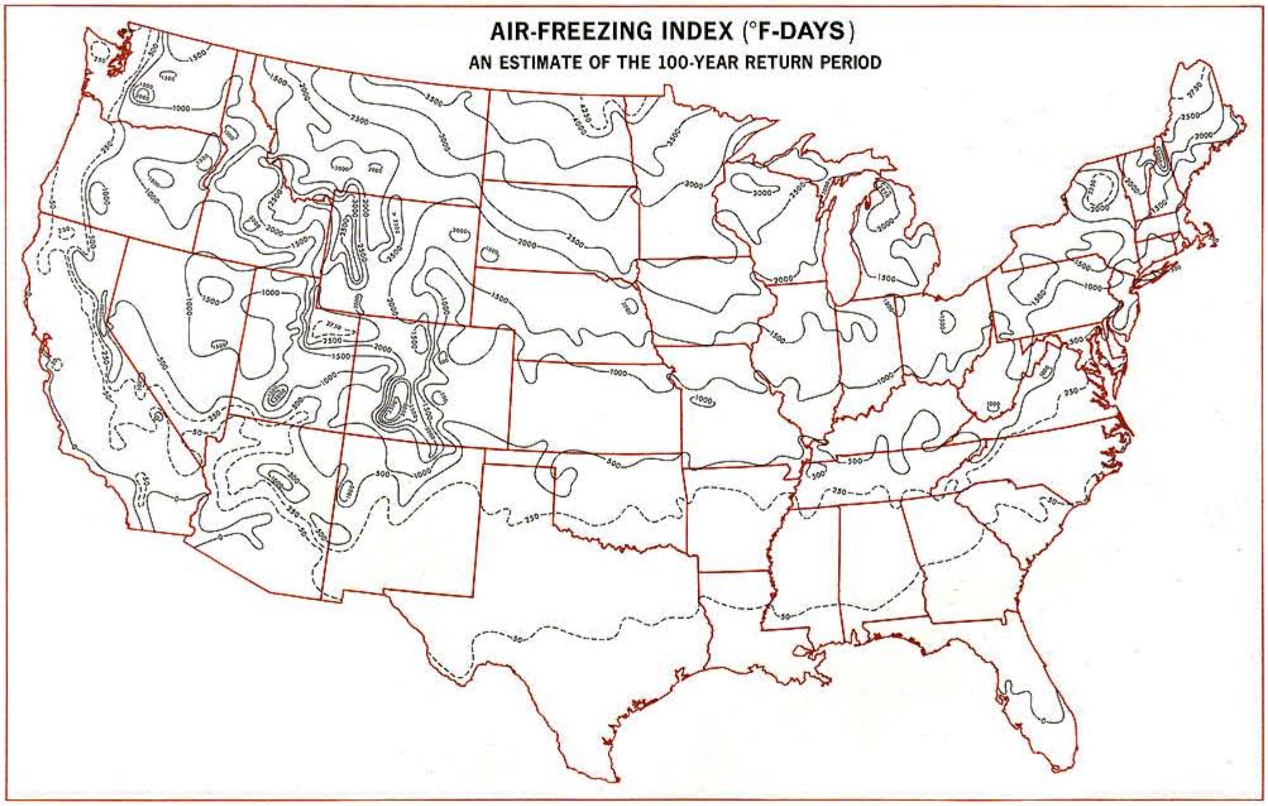


Figure 2-10. Contours of Air Freezing Indicies (°F-days) with a 100-yr Return Period (downloaded from http://lwf.ncdc.noaa.gov/oa/fpsf/fpsf.html).

2.3.2.2.3 Accidental Human Intrusion Protection

Accidental human intrusion has generally not been a design consideration for cover systems on most landfills or waste remediation sites. However, ordinary human activities can damage the cover system. For example, ruts may be created if vehicles are driven on the cover system when the surface layer is wet. Normally, if an adequate cover soil thickness is provided to support vegetation and protect the underlying cover system components, the thickness will also be sufficient to protect the cover system from ordinary human impacts such as vehicle ruts.

Essentially the only type of waste for which accidental human intrusion has been a design consideration is radioactive waste. It is not clear why radioactive waste has been singled out. Human intrusion into MSW or HW could also be dangerous to the intruder. When human intrusion has been considered, the principal concern has been with accidental exposure (e.g., excavation to lay a buried pipeline or to construct a basement for a home). Though the cover system can be thickened to approximately 5 m or more to prevent such occurrences, the problem is more typically handled by assuming that deed restrictions and security measures will prevent intrusion. No amount of thickness can prevent "intentional" intrusion, such as drilling a boring or digging a deep utility excavation.

Some cover systems, especially those at redeveloped sites, may incorporate visible barriers with bright, readily identifiable colors within or beneath the protection layer to indicate that the cover system may be damaged if the intrusive activity continues any further downward. For example, bright orange plastic netting has been used for such a purpose. Other types of visible barriers may also be used to provide an additional safeguard against accidental digging or other construction-related damage to the cover system.

2.3.2.2.4 Root Penetration Protection

The penetration of plant roots below the protection layer is undesirable. Suter et al. (1993) summarize the potential mechanisms by which plant roots can damage a cover system:

- Roots may enter the drainage layer or gas collection layer and cause clogging.
- Roots may penetrate the hydraulic barrier, causing an increase in hydraulic conductivity.
- Decomposing roots leave channels for movement of water and vapors.
- Roots may desiccate CCLs, causing shrinking and cracking.
- Uprooted trees may lead to soil erosion and leave depressions in the cover system.
- Roots may enter the wastes, take up constituent chemicals, and transport them to above ground components. For radioactive wastes, this is a particular concern.
- Roots may modify the waste by increasing decomposition rates and by releasing chemicals that mobilize metals.

Suter et al. (1993) provide examples of several of these potential problems. Different plant species develop root systems that penetrate to different depths. Root systems of shallow-rooted grasses may penetrate no deeper than 0.15 m into the subsoil. Grasses with deeper root systems may have roots that penetrate to depths of 0.3 to 0.5 m. Root systems of shrubs can penetrate to depths in excess of 1 m. Some desert plant species have roots that can penetrate many meters into the subsurface. Trees also have deeper root systems. In generally, the establishment of deep-rooted shrubs and trees on a cover system should be prevented via routine maintenance such as periodic mowing unless the cover system has been specifically designed to accommodate the deep roots.

Climate influences the depth of root penetration, and even the materials into which roots penetrate have an influence on root depth. Roots generally seek out lightly-compacted soils that contain moisture. Roots will not, as a general rule, penetrate into dry or heavily compacted soils.

In soil profiles containing a finer-grained soil overlying a coarser-grained soil, roots will remain in the relatively moist, finer-gained soil and will not penetrate into the coarser-grained soil as long as the coarser soil remains dry. If the coarser-grained soil becomes wet, then the roots will seek moisture in this soil.

The coarser-grained material used to construct a barrier to plant roots often consists of cobbles. When cobbles are used as a barrier to plants roots, the placement of a fine-textured soil over the cobbles will create a capillary barrier. If the cobbles remain dry, they should stop further downward penetration of plant roots (Hakonson, 1986). The cobbles may also help increase plant growth by keeping moisture on the upper soil layer. Experiments with cobble biobarriers have been carried out at arid and semi-arid sites (Cline, 1979; and Cline et al., 1982). Research indicates that 0.9 m of cobbles, or 0.15 m of gravel over 0.75 m of cobbles, is effective in stopping root penetration of deep-rooted plants (DePoorter, 1982).

Another alternative is to utilize materials that inhibit root growth, to stop further penetration of roots into the soil. Cline et al. (1982) examined the effectiveness of several phytotoxins impregnated into or onto GTs that were placed within the soil protection layer, just above the drainage layer. Some of the phytotoxins met the goal of being effective in stopping the downward progress of root growth, with no other effects. However, some of the phytotoxins killed the plants when the roots encountered the fabric. The longevity of these products requires further evaluation.

2.3.2.2.5 Burrowing Animal Protection

For some types of waste (particularly radioactive waste), the protection layer may need to provide the cover system with a high level of protection from intrusion by burrowing animals. Suter et al. (1993) summarize the effects that burrowing animals can have on cover systems as follows:

- Animals may burrow through the cover system, resulting in direct channels for movement of water, vapors, roots, and other animals.
- Even when they do not penetrate the entire cover system, burrows may increase the porosity of the soil, thereby increasing infiltration rates in some situations (although, in arid areas, burrows may actually do the opposite by provide channels for enhanced evaporation).
- If burrows penetrate the entire cover system, animals may become externally contaminated or consume the waste, thereby spreading the waste in their feces, urine, and flesh.
- Animals may carry waste directly to the surface during excavation if the burrows fully penetrate the cover system.
- By working the soil and transporting seeds, burrowing animals may hasten the establishment of deep-rooted plants on the cover system.
- Burrowing animals cast soil on the surface, thereby increasing erosion of the cover system.

Research by Cline (1979), Cline et al. (1982), and Hakonson (1986) found that if objects, such as cobbles, placed in a burrowing animal's path are sufficiently large and/or tightly packed, the animal's progress is effectively stopped. Thus, a barrier to burrowing animals typically consists of a 0.5 to 1-m thick layer of cobbles. The maximum particle size should be established based on the burrowing animals of concern but is typically on the order of 100 to 200 mm. Care should be taken to provide adequate filter layers both above and below the cobbles, to prevent overlying and underlying soil particles from migrating into the cobbles. Filter design is presented in Section 4.7.

A GM may also be viewed as a barrier to burrowing animals. Studies indicate that animals will not make their way through GMs such as those made from HDPE (Steiniger, 1968). Also, welded wire mesh and certain polymeric erosion control mats may also be barriers to burrowing animals.

2.3.2.2.6 Vegetation Support

Vegetated cover soils should be thick enough to accommodate a healthy growth of plant roots and store sufficient water to support plant growth. Plants should generally have relatively shallow roots so that the roots do not penetrate too deep into the cover system because, as described in Section 2.3.2.2.4, deep penetration threatens the integrity of underlying components. However, roots should be deep enough to enable the plants to extract moisture from a sufficient depth. Most grasses are thought to have effective rooting depths of about 0.15 to 0.5 m. If plants with deeper roots are planted or represent a desirable climax community, the thickness of the cover soil should be increased to accommodate root growth. For example, deeper-rooted plants may become established over time and displace the grasses that were initially planted. The minimum thickness of the cover soil is typically 0.45 to 0.6 m to accommodate plant roots. Even thicker cover soils are required to accommodate certain shrubs and desert plant species.

2.3.2.2.7 Water Storage

Most of the rainfall that contacts the surface of a cover system infiltrates into the underlying cover soil and is retained in the soil by capillary forces. The ultimate fate of this water is primarily ET. For cover systems with a vegetated surface layer, it is critical that the cover soils be capable of retaining sufficient moisture to support plant growth.

The greater the percentage of fines in a soil, the greater the water retention after gravity drainage. The volumetric water content of a soil after gravity drainage is referred to as the soil's field capacity, θ_{fc} (dimensionless). This parameter is often reported as the volumetric water content at a matric potential of -0.03 MPa (-3.3 m). At water contents less than field capacity, the soil hydraulic conductivity is often assumed to be so low that gravity drainage of the soil becomes negligible and the soil moisture is held in place by capillarity. Some of this stored water can be removed via transpiration. Vegetation can reduce the soil moisture content from field capacity to wilting point, θ_{wp} (dimensionless). This parameter is often defined as the volumetric water content at a matric potential of -1.5 MPa (-150 m)). At water contents below the wilting point, plant activity is assumed to stop. Evaporation from the soil surface can further reduce the soil moisture content from wilting point to residual saturation, which is the water content at an infinite matric potential. The relationship between these different soil water contents is shown in Figure 2-11 for soil textures ranging from sand to clay.

Though plastic clays have a high field capacity, they are typically not used for the protection layer because they can desiccate and crack, providing preferential pathways for infiltrating water to bypass the clay matrix and thereby bypass storage. In addition, there is less water storage for plants in these soils than in silty loam soils, as shown in Figure 2-11 and Table 2-6. In some regions, such as the Texas Gulf coast, the surface soils are almost entirely highly plastic clays. In such cases, there may be no practical alternative to the use of a heavy clay soil. If a loamy soil is available, it is usually selected because it is the best soil in terms of combining good moisture retention, workability, resistance to desiccation cracking, and moderate hydraulic conductivity. Sandy clays, clayey sands, and lean clays may also be suitable for use in protection layers.

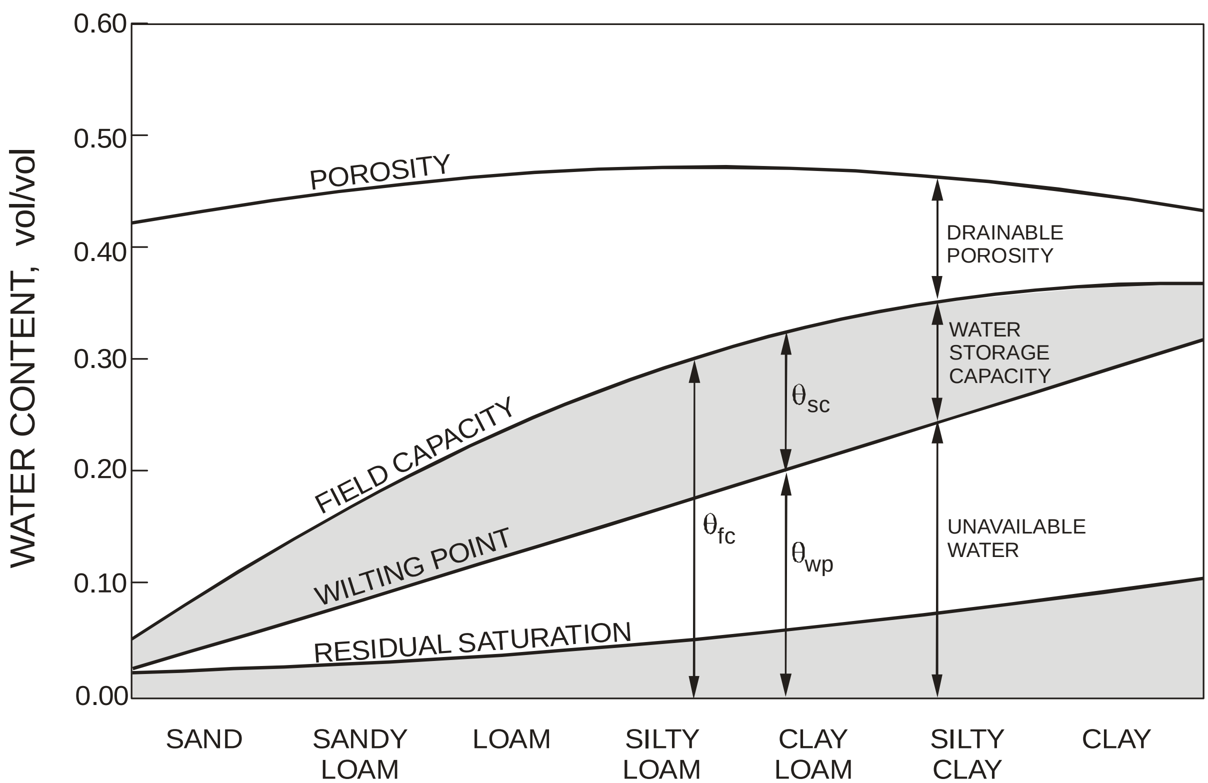


Figure 2-11. Relation Among Moisture Retention Parameter and Soil Texture Class (modified from Schroeder et al., 1994).

A soil's available water storage capacity (i.e., θ_{fc} - θ_{wp}) depends on its texture and density. Representative moisture content values for soils of different textures are given in Table 2-6. Since cover soils are only lightly compacted (unlike hydraulic barriers which are heavily compacted), only data for low-density soils are presented. As shown in the table, silty or clayey sands, silts, and silty clays typically have a storage capacity of about 0.1 to 0.15.

The depth of water, H_w (m), that can be stored in a soil layer for subsequent removal by plants can be calculated as follows:

$$H_w = \theta_{sc} H_s = (\theta_{fc} - \theta_{wp}) H_s \qquad \text{(Eq. 2.18)}$$

where: θ_{sc} = water storage capacity of soil (dimensionless); H_s = soil layer thickness (m); and all other terms are as defined previously. It is important to note that the use of field capacity and wilting point is arbitrary and ignores other factors that affect the amount of moisture retained in a soil layer, such as rock fragments and salts in solution (Cassel and Nielsen, 1986; NRCS, 1998b). Nevertheless, these are simple and commonly used concepts and are applicable for approximating the water storage capacity of a soil layer.

Table 2-6. Representative water contents for low-density soils with different textures (modified from Schroeder et al., 1994).

Soil Description	USDA Classification	Porosity (–)	Field Capacity (–)	Wilting Point (–)	Storage Capacity (–)	Saturated Hydraulic Conductivity (m/s)
Clean, poorly-graded sand	Coarse sand (CoS)	0.417	0.045	0.018	0.027	1.0×10^{-4}
Clean, well-graded sand	Fine sand (FS)	0.457	0.083	0.033	0.050	3.1×10^{-5}
Silty sand	Sandy loam (SL)	0.453	0.190	0.085	0.105	7.2×10^{-6}
Low-plasticity silt	Loam (L)	0.463	0.232	0.116	0.116	3.7×10^{-6}
Low-plasticity silt	Silty loam (SiL)	0.501	0.284	0.135	0.149	1.9×10^{-6}
Low-plasticity clay	Clay loam (CL)	0.464	0.310	0.187	0.123	6.4×10^{-7}
Clayey sand	Sandy clay (SC)	0.430	0.321	0.221	0.100	3.3×10^{-7}
High-plasticity clay	Clay (C)	0.475	0.378	0.251	0.127	2.5×10^{-7}

The depth of water that can be stored in a soil layer can be substantial. For example, from Table 2-6 and Eq. 2.18, the representative storage capacity of a 0.6-m thick protection layer constructed with silty loam is 0.149 and the depth of water that can be stored in this layer is approximately 90 mm. If the protection layer was constructed with fine sand, only about one-third of this storage capacity would be provided.

2.3.2.2.8 Radon Attenuation

Some radioactive wastes emit radon-222 (^{222}Rn) in the form of a heavier-than-air gas. Inhalation of radon gas at sufficient concentrations is a human health hazard. Federal regulations limiting radon releases to the atmosphere are contained in 40 CFR §192.02 and are applicable to the control of emissions from UMTRA sites that must comply with UMTRCA. The regulations are also typically applied as an ARAR to DOE sites undergoing remediation. These regulations require that release of ^{222}Rn to the atmosphere not exceed: (i) an average release rate of 20 picocuries per square meter per second; or (ii) increase the annual average concentration of ^{222}Rn in the air at or above any location outside of the disposal site by more than one-half picocurie per liter. To attenuate the release of radon to the environment, the cover system may need to incorporate a radon gas barrier. This barrier may be incorporated in the hydraulic barrier or it may be located closer to the surface, in which case the gas barrier may be considered to be part of the protection layer.

GMs can also be used as barriers to radon gas release. While the half-life of ^{222}Rn is short (3.8 days), radon is a part of the uranium-238 (^{238}U) decay series. Uranium-238 has a half-life of about 4.5 billion years. Given this long half-life, there has been some concern about the longevity of GM barriers used for radon control. Although GMs will not last forever, a properly selected and appropriately formulated GM, adequately protected by design, can last for a presumed timeframe measured in hundreds of years. Because the cost of GMs is relatively low, a GM can provide a cost-effective means of radon gas control for the timeframe just indicated.

For a soil layer to function as an effective barrier to gas diffusion, air-filled voids in the soil have to be discontinuous. Gas diffuses very slowly through wet soils that contain only occasional, unconnected air bubbles. Relatively thick (up to about several meters) layers of clay-rich soil are typically employed when protection from radon emissions is needed. For clayey soils to function effectively as gas barriers, they must be at a high degree of saturation and free of cracks. Over a design life of hundreds of years, maintaining a wet, undesiccated layer of clayey soil under natural conditions can be a tremendous challenge. To maintain a high water content in the soil, a riprap surface layer may be considered to increase infiltration. The increased infiltration may, however, result in increased potential for percolation through the cover system.

Specific procedures for designing soil layers to provide radon protection are beyond the scope of this guidance document. One methodology documented by DOE (1989) involves determining the allowable radon emission, estimating the radon diffusion coefficient through the soil, and sizing the thickness of the soil layer based on the calculated diffusive flux. Additional information on radon attenuation through cover systems is presented in NRC publications by Rogers and Associates Engineering (1984a,b).

2.3.3 Construction

When the cover system is vegetated, the soil protection layer is only lightly compacted to allow plant roots to penetrate the soil, as discussed in Section 2.2.6. For unvegetated cover systems, the soil protection layer may be placed and compacted using procedures for structural fill or may have no specific compaction criteria. Depending on the properties of the materials underlying the protection layer, and especially if there are geosynthetics underlying the protection layer, there may be limitations on the stresses exerted by the construction equipment. For example, if a soil protection layer overlies a GC drainage layer, the soil may need to be placed with a low-ground pressure bulldozer and a minimum first lift compacted thickness of 0.2 to 0.3 m.

2.3.4 Maintenance

Maintenance is discussed in Chapter 9. Since the protection layer is covered by the surface layer, protection layer maintenance is generally not needed unless the surface layer is breached due to erosion or there are problems with excessive differential settlement or slope instability.

2.3.5 Monitoring

Monitoring is discussed in Chapter 8. If the cover system water balance is being assessed, the protection layer moisture content or matric potential may be monitored.

2.4 Drainage Layer

Water that permeates through the surface and protection layers can be removed from the cover system by an internal drainage layer. The primary functions of the drainage layer are to: limit the buildup of hydraulic head on the underlying hydraulic barrier, which minimizes percolation of water through the barrier; drain the overlying protection and surface layers, which increases the available water-storage capacity of these layers and helps to minimize erosion of these layers; and reduce the seepage forces in the protection, surface, and drainage layers, which improves cover system slope stability.

2.4.1 General Issues

In many cases and especially on sideslopes, an internal drainage layer is included above the hydraulic barrier to promote lateral drainage and prevent the buildup of hydraulic head in the cover system. As discussed by Bonaparte et al. (2002), the design of existing cover system drainage layers has been found to be inadequate in a significant number of cases, leading to a significant number of instances of excessive cover system erosion and slope instability. The main issues with drainage layer design are related to flow capacity, transitions and outlets, and filtration. Each of these issues is discussed below.

The drainage layer should be designed to have adequate flow capacity. As described in Section 7.4.3, there have been cases of cover system instability due to the build up of seepage forces on sideslopes after a rainfall. For some of these cases, the drainage layer was not designed with adequate flow capacity; in one case, the cover system did not include a drainage layer. The drainage layer should be designed to convey the maximum anticipated flow rate from a design storm, and the maximum flow rate should be calculated considering the cover system water balance for the selected storm. Methods for calculating the maximum flow rate in a drainage layer are presented in Section 4.5. The allowable flow rate of a drainage layer can be calculated as described in Section 2.4.2.3.

It is noted that in arid and semi-arid climates a water balance may show that a cover system does not require a drainage layer. Instead, it may show that infiltration is stored in the overlying cover soils and later removed by ET.

Drainage layer transitions and outlets should be designed to provide free-flow of water. Otherwise, cover soils can become saturated, leading to increased erosion, and seepage forces can increase, leading to an increased potential for slope instability. The design of drainage layer slope transitions is discussed in Section 4.6. Outlet design is discussed in Section 2.4.2.4.

The need for a soil or GT filter above the drainage layer should be evaluated. Sometimes the drainage material (particularly if it is sand) is inherently a filter for the adjacent materials, in which case a separate filter layer is not required. However, a filter (soil or GT) is usually required, particularly if the drainage layer is gravel or a GN. As described in Section 7.4.3, there have been cases of cover system instability where the cause of the instability was attributed to clogging of a GT filter or clogging of a granular drainage layer when a filter layer was omitted. If a filter is required, it should be designed to retain the overlying soil, resist clogging, and have adequate permittivity. The design approach for soil and GT filters is presented in Section 4.7.

2.4.2 Elements of Design

Important questions that typically need to be addressed when considering the design of the drainage layer include:

- What materials are available to construct the drainage layer?
- What thickness of drainage layer material is needed?
- What are the maximum design flow rate and allowable flow rate in the drainage layer?
- How should drainage layer transitions and outlets be designed?
- How should the drainage layer be constructed?
- What type and frequency of maintenance should be employed?
- What type and frequency of monitoring should be employed?

2.4.2.1 Materials

Both granular materials (typically sand or gravel) and geosynthetics (GT, GN, and GC) have been used as drainage layer material in cover systems. The material used should have adequate hydraulic conductivity to minimize the buildup of hydraulic head above the hydraulic barrier and adequate hydraulic transmissivity to convey the design flow rate. The drainage layer material should also meet filter criteria with adjacent layers.

2.4.2.1.1 Granular Materials

Granular drainage materials are normally composed of relatively clean sand or gravel. Gravel is material that does not pass through the 4.74-mm wide openings of a No. 4 sieve. Sand consists of material that passes through the No. 4 sieve but not through the 0.075-mm wide openings of a No. 200 sieve. “Clean” sand or gravel refers to sand or gravel that contains very little or no material that passes through the openings of a No. 200 sieve. Clean sands and gravels are often produced by washing natural sands and gravels to remove any “fines,” which are particles that pass through the openings of a No. 200 sieve.

The drainage layer should meet filter criteria with the overlying protection layer. If the drainage layer material will not retain the protection layer material, a soil or GT filter is required. A discussion of filter layer design is presented in Section 4.7.

Specifications for granular materials often require:

- no more than 5% (dry-weight basis) of material passing the No. 200 sieve;
- a maximum particle size on the order of 25 to 50 mm; however, smaller particles will typically be required if a GM will underlie the drainage layer; alternatively, a GT cushion layer can be used;
- restrictions on gradation, stated in terms of allowable percentages for specified sieve sizes (these restrictions may exist for various purposes, including filtration considerations);
- limitations on mineralogy (often the drainage material is required to be a non-carbonaceous material, with a limit on the amount of calcium carbonate in the material, although hard evidence that carbonaceous materials are truly unsuitable is lacking, as discussed below);
- restrictions on the angularity of the material, if the material will interface with geosynthetics, which are vulnerable to puncture by large, sharp objects (or, alternatively, a GT cushion may be employed);
- that no deleterious material be present; and
- a minimum acceptable saturated hydraulic conductivity.

The specified material requirements attempt to ensure that the materials will not puncture adjacent geosynthetics, will be chemically stable, and will provide adequate drainage. Perhaps the two most complex requirements relate to presence of calcium carbonate and to hydraulic conductivity.

Nearly all granular construction materials are natural, excavated materials (e.g., river sand or gravel) or are produced from crushing rock. In either case, granular materials that are rich in calcium carbonate (e.g., crushed limestone or dolostone) are commonly available in many parts of the U.S. and are frequently considered for use as drainage layer material. There are two concerns over the use of drainage material containing calcium carbonate. First, if GCLs are used as the hydraulic barrier, leachable calcium may undergo ion exchange with the sodium in the bentonite causing an increase in the GCL's hydraulic conductivity. (CCLs can also be adversely impacted by ion exchange, but generally to a much lesser extent because of their thickness and minerology.) Second, calcium carbonate may slowly dissolve, threatening the integrity of the drainage material and potentially causing chemical clogging if the dissolved material is precipitated elsewhere in the system. There is little hard, published evidence that dissolution of calcium carbonate from drainage materials in cover systems is, in fact, a serious problem. However, the mechanism is obvious and the potential for problems commands caution. This is an area of on-going research, and, within the next few years, it should be possible to develop additional design guidance. However, until more definitive information becomes available, it is recommended that the calcium carbonate content of the drainage material be limited.

Although there are no definitive guidelines, specified maximum values for calcium carbonate content typically range from 5 to 20%. Local experience and practice, coupled with knowledge of the calcium carbonate content of locally available granular materials, tend to dictate the specified value. In some areas, it may be impossible to find granular materials that are completely free of calcium carbonate. In addition, of the two ASTM tests that are often specified for calcium carbonate content (ASTM D 3042 and ASTM 4373), one has been criticized for not providing reproducible or reliable test results for granular drainage materials and both use strong acids to dissolve the calcium carbonate.

No specific minimum hydraulic conductivity is recommended for a granular drainage material because the required value is site dependent. When there is a regulatory guidance or requirement (e.g., Federal guidance regarding cover system drainage layers for HW landfills), the minimum specified hydraulic conductivity is generally 1×10^{-4} m/s. However, analysis indicates that this value may be too low for many applications. The problem with a minimum hydraulic conductivity of 1×10^{-4} m/s is that it may not provide the drainage layer with sufficient capacity to convey the maximum flow rate from a design storm. To minimize the potential for excessive erosion and slope instability, the drainage layer should be able to convey the maximum flow rate entirely in the layer without buildup of excess head.

Also, a soil with a hydraulic conductivity of 1×10^{-4} m/s will typically retain a significant amount of moisture under gravity drainage conditions (i.e., have a significant field capacity). The presence of this moisture increases the potential for root penetration into the layer. The moisture also increases the potential for freeze-thaw effects.

Hydraulic conductivity is usually measured in the laboratory using ASTM D 2434. The degree of difficulty in accurately measuring hydraulic conductivity increases as the hydraulic conductivity increases. With very high-hydraulic conductivity materials (e.g., large gravels), it is necessary to maintain a very low head loss in order to avoid turbulent flow, and the small head loss is difficult to measure. Specialized laboratory equipment is required to test these materials.

Care should be taken to ensure that representative samples of material are tested for hydraulic conductivity, and that the density (hence, porosity) of the samples are representative of the value expected for the drainage layer as constructed in the field. As materials are handled in the field, they tend to get ground up slightly, producing additional fines and lowering hydraulic conductivity, particularly in the lower part of the drainage layer. As a rule of thumb, approximately 0.5 to 1% of additional fines by weight will be generated every time a drainage material is handled. When a sample is collected from a material stockpile, there is a tendency to select a sample near the surface. Such samples may be cleaner than material from deeper in the stockpile and also cleaner than the material will be after it is handled and placed in the field.

2.4.2.1.2 Geosynthetics

Because the normal stresses on a cover system drainage layer are relatively low, a number of different types of geosynthetics can be considered for use as the drainage layer. Geosynthetic drainage materials most frequently used in cover systems include:

- GNs of solid ribs with diamond-shaped apertures;
- GNs of foamed ribs with diamond-shaped apertures; and
- needlepunched nonwoven GTs.

Other geosynthetics drainage materials that may also meet project-specific requirements include:

- "high flow" GNs of solid ribs in a parallel orientation;
- drainage cores of single cuspations or dimples;
- drainage cores of double cuspations or dimples;
- drainage cores of built-up columns;
- drainage cores of stiff three-dimensional entangled mesh;
- resin bonded nonwoven GTs.

Like granular drainage layers, a geosynthetic drainage layer should meet filter criteria with the overlying protection layer. A GN or core drainage layer requires an overlying GT filter to keep the protection layer material from directly clogging the apertures of the drain. Furthermore, if a GM hydraulic barrier underlies a GN or core drainage layer, as is often the case, a GT may be required between the drain and GM to provide higher interface friction on steep sideslopes and, possibly, reduce deformation-related intrusion of the GM into the drain and/or protect the GM from puncture or other damage by the drain. Often, the GT is heat bonded or glued to the GN or drainage core, creating a GC, to enhance interface shear strength, decrease the potential for fugitive soil particles to enter the drain during construction, and facilitate installation. If a GT drainage layer is used, it is also designed to meet filter criteria with the overlying protection layer material.

A potential advantage of thin geosynthetic materials as drainage layers is that the weight of these materials is very low, which is advantageous when compressible waste or soil underlies the cover system. Also, geosynthetics, being thin, occupy less airspace than an equally transmissive granular drainage layer. (This same advantage applies to the use of a GCL over a CCL as a hydraulic barrier and a geosynthetic over granular material in a drainage layer.)

Specifications for geosynthetic drainage layers often require:

- resin and additive requirements;
- minimum thickness;
- minimum mass per unit area;
- minimum hydraulic transmissivity at a specified normal stress and hydraulic gradient;
- minimum strength requirements to survive installation;
- if the drainage material is a GN or core, inclusion of a GT filter above the drain; and
- if the drainage material is a GN or core, inclusion of a GT beneath the drain, if necessary, to increase interface friction, reduce deformation-related intrusion of an underlying

hydraulic barrier material into the drain, and/or protect the hydraulic barrier from puncture or other damage by the drain.

As with the hydraulic conductivity of a granular drainage layer, no specific minimum hydraulic transmissivity is recommended for a geosynthetic drainage material because the required value is site dependent. To minimize the potential for excessive erosion and slope instability, however, the drainage layer should be able to convey the maximum flow rate entirely in the layer without buildup of excess head. It is noted that a geosynthetic drainage layer is generally required to have a higher transmissivity than that for a granular drainage layer to convey the required design flow rate under unconfined flow conditions. As discussed by Giroud et al. (2000), the geosynthetic drainage layer hydraulic transmissivity that is equivalent to a granular drainage layer hydraulic transmissivity for these conditions can be calculated as:

$$\theta_{dg} = E\,\theta_{ds} = E\,k_{ds}\,t_{ds} \qquad \text{(Eq. 2.19)}$$

where: θ_{dg} = geosynthetic drainage layer transmissivity (m^3/s/m); E = equivalency factor (dimensionless); θ_{ds} = granular drainage layer transmissivity (m^3/s/m); k_{ds} = granular drainage layer hydraulic conductivity (m/s); and t_{ds} = granular drainage layer thickness (m). The equivalency factor can be approximated as (Giroud et al., 2000):

$$E = \frac{1}{0.88}\left[1 + \left(\frac{t_{ds}}{0.88 L_d}\right)\left(\frac{\cos\beta}{\tan\beta}\right)\right] \qquad \text{(Eq. 2.20)}$$

where: L_d = length of drainage layer flow path (m), and all other terms are as defined previously.

The hydraulic transmissivity of geosynthetic drainage layers can be measured in the laboratory using ASTM D 4716. The test setup should simulate the actual field system as closely as possible in terms of boundary conditions, stresses, and gradient.

2.4.2.2 Thickness of Granular Layers

The recommended minimum thickness of a granular drainage layer is usually 0.3 m. This allows sufficient thickness for ease of construction and to avoid damage to underlying geosynthetics, such as a GM. With extremely careful control of thickness, it is possible to construct thinner granular drainage layers (down to a thickness of about 0.15 m), but granular drainage layers thinner than 0.3 m are not very common.

2.4.2.3 Required Flow Capacity

The flow capacity, q_c (m^3/s/m), of a drainage layer must be equal to or greater than the product of the maximum flow rate, q_m (m^3/s/m), considered for design and the factor of safety, FS (dimensionless):

$$q_c \geq q_m\,FS \qquad \text{(Eq. 2.21)}$$

As previously mentioned, the maximum flow rate can be calculated considering the cover system water balance for the selected design storm. Methods for calculating the maximum flow rate are presented in Section 4.5. The FS selected for design should be based on the level of uncertainty inherent in the design input parameters and the consequences of failure. A minimum FS value of 2 is recommended for cases where the uncertainty in input parameters is low and the consequences of failure are small. For many situations, a larger FS may be appropriate. Koerner and Daniel (1997) have recommended using a FS value of at least 5 to 10 to account for uncertainities in the hydraulic conditions.

For granular drainage layers, the drainage layer hydraulic conductivity is selected to provide adequate flow capacity and unconfined flow conditions. For geosynthetic drainage layers, the drainage layer hydraulic transmissivity is selected to provide adequate flow capacity and unconfined flow conditions. For all drainage layer materials, the required field hydraulic properties for design are evaluated considering the material properties measured in the laboratory and reduction factors that consider the potential for reduction in the property over time due to long-term clogging, deformation, etc. in the field.

For granular drainage layers, the field hydraulic conductivity can be computed as:

$$k_f = k_l \left(\frac{1}{RF_{CC} RF_{BC}} \right) \qquad \text{(Eq. 2.22)}$$

where: k_f = long-term field hydraulic conductivity of granular drainage layer (m/s); k_l = hydraulic conductivity of granular drainage layer (m/s) measured in the laboratory; RF_{CC} = reduction factor for chemical clogging (dimensionless); and RF_{BC} = reduction factor for biological clogging (dimensionless).

For geosynthetic drainage layers, the field hydraulic transmissivity can be computed as:

$$\theta_f = \theta_l \left(\frac{1}{RF_{IN} RF_{CR} RF_{CC} RF_{BC}} \right) \qquad \text{(Eq. 2.23)}$$

where: θ_f = long-term field hydraulic transmissivity of geosynthetic drainage layer ($m^3/s/m$); θ_l = hydraulic transmissivity of geosynthetic drainage layer ($m^3/s/m$) measured in the laboratory; RF_{IN} = reduction factor for elastic deformation and/or or intrusion of the adjacent geosynthetics into the drainage layer (dimensionless); RF_{CR} = reduction factor for creep deformation of the drainage layer and/or creep deformation of adjacent materials into the drainage layer (dimensionless); and all other variables are as defined previously.

It may occasionally be necessary to consider other reduction factors, such as factors for installation damage or elevated temperature effects. If necessary, they can be included on a site-specific basis. On the other hand, if the reduction factor has been included some way in the test procedure for measuring the hydraulic property, the reduction factor would appear in the foregoing formulation as a value of unity. Information on preliminary reduction factor values is given in Koerner (1998).

2.4.2.4 Drainage Layer Outlets

As previously discussed, water collected in a drainage layer should be conveyed to an outlet. If there are not a sufficient number of outlets or if the outlets become clogged, the hydraulic head in the drainage layer can build up and exceed the drainage layer thickness, leading to saturation of cover soils and increases in seepage forces. There have been cases of significant cover system erosion and slope instability caused by inadequate outlet design.

Drainage layer outlets are usually designed to release water into drainage ditches or swales on the cover system or along the facility perimeter. The drainage layer may extend to the ditch or swale, as in Figure 2-5(a) or may be connected to the drainage structure via pipes or other means. When it is necessary to prevent the drainage layer from freezing, the drainage layer is usually insulated with an adequate thickness of cover soil (see Section 2.3.2.2.2). However, the prevention of freezing (and, hence, plugging) of outlet points can be challenging because outlets are usually exposed to freezing temperatures. Pipe outlets may be more problematic than areal outlets because they concentrate flow from a larger area. Thus, if a pipe is plugged with frozen water, water would have to flow laterally for some distance to reach another pipe. The authors are aware of situations where pipes plugged with ice have been dealt with as a maintenance issue by removing the ice using a heat source.

2.4.3 Construction

The construction, quality control (QC), and CQA of granular drainage layers and the manufacturer, installation, QC, and CQA of geosynthetic drainage layers are discussed in detail by Daniel and Koerner (1993, 1995). This discussion is not repeated herein.

In brief, granular drainage material is usually loosely dumped from a truck and spread with a low-ground pressure bulldozer. Low-ground pressure equipment is used to minimize the generation of fines and the potential for damage of any underlying geosynthetics. Granular drainage layers are generally not compacted.

Geosynthetic drainage layers are manufactured in panels of certain widths and lengths. The panels are placed in the field and connected by overlapping, seaming, tying, interlocking, or other means.

2.4.4 Maintenance

Maintenance is discussed in Chapter 9. Since the drainage layer is overlain by the surface and protection layers, drainage layer maintenance is generally not needed unless the cover soils are breached due to erosion or there are problems with excessive differential settlement or slope instability.

2.4.5 Monitoring

Monitoring is discussed in Chapter 8. If the cover system water balance is being assessed, lateral drainage from the drainage layer may be monitored.

2.5 Hydraulic Barrier

The primary function of the hydraulic barrier is to limit percolation of water through the cover system to an amount less than or equal to the maximum acceptable value. The hydraulic barrier achieves this by impeding infiltration into the barrier and by promoting storage or lateral drainage of water in the overlying layers. For wastes that generate gases or have volatile constituents, the hydraulic barrier can also restrict migration of these pollutants through the cover system and into the atmosphere.

2.5.1 General Issues

By definition, the hydraulic barrier must provide high impedance to flow of water, typically by having a very low saturated hydraulic conductivity. The most important concern with respect to the hydraulic barrier is the ability of the barrier to function as intended over time. Depending on the barrier material selected, the water impedance capabilities of a barrier can become substantially reduced when the barrier is subjected to deformations, wet-dry cycles, freeze-thaw cycles, and biointrusion. Even when not subjected to these stresses, barriers may degrade over time, for example, as GMs do as they lose their oxidizers by volatilization.

2.5.2 Elements of Design

Important questions that typically need to be addressed when considering the design of the hydraulic barrier include:

- What materials are available to construct the hydraulic barrier?
- What thickness of hydraulic barrier material is needed?
- What is the expected performance of the hydraulic barrier in terms of quantity of water percolation through the layer?
- What is the expected performance of the hydraulic barrier in terms of prevention of gas release to the atmosphere?
- How much differential settlement is expected, what level of tensile strain will this create in the hydraulic barrier, and how is the barrier expected to perform under this stressor?
- What is likelihood that the hydraulic barrier will be subjected to wet-dry cycles and how is the barrier expected to perform under this stressor?
- What is likelihood that the hydraulic barrier will be subjected to freeze-thaw cycles and how is the barrier expected to perform under this stressor?
- What hydraulic barrier properties are required to provide the required shear strength?
- What is likelihood that the hydraulic barrier will be subjected to biointrusion and how is the barrier expected to perform under this stressor?
- What is the anticipated lifetime of the barrier material(s)?
- How should the hydraulic barrier be constructed?
- What type and frequency of maintenance should be employed?

2.5.2.1 Materials

Materials used for hydraulic barriers include GMs, GCLs, and CCLs. Although other materials have been used (e.g., asphaltic concrete, as discussed in Section 2.2.2.2.6), the vast majority of all barriers are composed of one or a composite of the three materials listed above. Choices in the composite category typically are GM/GCL, GM/CCL, or GM/GCL/CCL. It has been shown that, all else being equal, a cover system with a composite barrier consisting of GM/CCL, GM/GCL, or GM/GCL/CCL allows less percolation than a cover system with a GM, GCL, or CCL barrier alone.

Each type of barrier has advantages and disadvantages. No one type should be viewed as optimal for all cover systems. The appropriate material(s) should be selected based on the specific objectives of a particular project and the expected site conditions.

2.5.2.1.1 GMs

GMs are thin, factory-manufactured polymeric materials that are widely used as hydraulic barriers in cover systems due to their non-porous structure, flexibility, and ease of installation. GMs have the advantages of extremely low rates of water and gas permeation through intact GMs and, depending on the material, the ability to stretch and deform without tearing. They also protect underlying CCLs from desiccation or root penetration. Disadvantages of GMs include leakage through occasional GM imperfections, the potential for slippage along interfaces between GMs and adjacent materials, and, for some applications, uncertainty about the length of the GM useful service life.

GMs form an essential part of many cover system hydraulic barriers. They are manufactured in panels, which vary in dimension depending on the manufacturing process and project-specific criteria. The most common types of GM polymers used in cover systems include:

- HDPE;
- very flexible polyethylene (VFPE) (this classification includes linear low density polyethylene (LLDPE), low density linear polyethylene (LDLPE), and very low density polyethylene (VLDPE));
- flexible polypropylene (fPP);
- flexible polypropylene reinforced (fPP-R), which is fabricated with a reinforcing scrim between two plys of polymer sheets; and
- polyvinyl chloride (PVC).

New materials are under development, and the above list of currently-used GMs should not be viewed as a complete list of all types of GMs that might be suitable for use in a landfill cover system. All of these GM materials are available with smooth and textured surfaces for increased friction and, thus, shear strength when used on steep sideslopes. Additionally, spray-on elastomeric GMs are possible, as are bituminous GMs. However, these groups are rarely used in cover systems and, therefore, are not discussed further.

Much has been written about the relative advantages and disadvantages of various GM materials. It is important that the requirements of a GM for a liner system not be confused with requirements for a cover system. In a typical liner system application, the GM is exposed to leachate and subjected to relatively high normal stresses. Replacement or repair of the GM after waste placement is not typically possible. Most liners are installed on firm subgrade, so the stress-elongation characteristics of the GM are of secondary importance. The most commonly used GM material for liner systems has historically been HDPE. Engineers have often selected this material because of its very good chemical resistance and service life characteristics.

In cover systems, the GM is not usually exposed to leachate, although it may be exposed to rising gases, which will often contain trace amounts of volatile constituents, or to vapors. Cover system GMs are subjected to relatively low normal stresses. However, as cover system GMs are often placed over compressible waste materials, which undergo post-closure differential settlement, the stress-elongation characteristics of the GM can be an important design consideration. While HDPE GMs have been widely used in cover systems, flexible GM barriers made of PVC, VFPE, and fPP are finding wider use.

In the current state-of-practice, chemical compatibility is rarely considered for cover system GMs since the upper surface of the GM is only exposed to water infiltration through the cover soils. However, the lower surface of the GM may be exposed to gases and vapors that may contain chemicals that are harmful to certain GM formulations. Thus, chemical resistance is an issue that may need to be considered under site-specific conditions.

Specifications for GM hydraulic barriers often require:

- resin and additive requirements;
- limitations on the amounts of fillers, carbon black, and regrind/recycle material that can be added to the resin;
- texture quality (e.g., minimum asperity height), if texturing is used;
- minimum thickness;
- mass per unit area; and
- minimum strength and elongation requirements.

Protection layers are often placed above a GM if angular gravel or crushed rock will be placed on the GM. A protection GT used in this application is sometimes referred to as a cushion. In cover systems, the overburden stresses produced by cover soils are normally not very large, which makes the design of a GT cushion relatively simple compared to a situation in which the angular stone overlying the GM is subjected to high compressive stresses. Procedures for selecting a GT mass per unit area to adequately protect the GM are provided by Koerner (1998).

2.5.2.1.2 GCLs

GCLs are thin, factory-fabricated products containing a layer of sodium bentonite (a very low permeability clay) that is supported by one or two layers of geosynthetics. GCLs have attractive features for cover system applications, including a very low saturated hydraulic conductivity (e.g., typically less than 5 x 10^{-11} m/s, which is lower than for CCLs), preservation of low hydraulic conductivity when subjected to different stressors, and ease of installation. Disadvantages of GCLs include low internal shear strength of hydrated bentonite, potentially low interface shear strength at its upper and lower surfaces (depending on the type of GCL and interfacing materials), potential for increased hydraulic conductivity due to cation exchange reactions under certain conditions, potential for premature hydration during installation desiccation cracking of the bentonite layer, and root intrusion for unprotected GCLs. Although GCLs are relatively new (first used in a waste containment application in the late 1980s), their use has increased rapidly in the past decade. One of the more common applications of GCLs is as the soil component of composite hydraulic barriers. Less frequently, they are used alone as a barrier. The results of a large-scale field test program sponsored by EPA to evaluate GCL use in cover systems are summarized in Section 7.4.5.

GCLs consist of sodium bentonite placed between GTs and mechanically held together by adhesive or fibers, or bentonite adhesively bonded to a GM or GT/GM laminate. The types of GCLs most commonly used in cover system applications are shown in Figure 2-12. The bentonite is the low-hydraulic conductivity component; the geosynthetics act as carrier materials or, in the case of GCLs incorporating GMs, as a supplemental hydraulic barrier. The carrier geosynthetics support the bentonite component and help to maintain a uniform layer of bentonite that can be handled, transported, and placed as a barrier. The manufactured material has a nominal clay thickness of 5 mm and is produced on rolls that measure about 4 m in width and 30 to 60 m in length. The mass of bentonite per unit area (dry weight basis) is typically at least 3.6 kg/m^2.

Bentonite is the critical component of GCLs. Bentonite is a naturally occurring, mined clay mineral material that is extremely hydrophilic. When placed in the vicinity of water (or even water vapor), the bentonite attracts water molecules into a complex configuration that leaves little free water space in the voids. This significantly decreases the hydraulic conductivity of the bentonite. When the bentonite is saturated and permeated with fresh water, the hydraulic conductivity is typically on the order of 1 to 5 x 10^{-11} m/s, or less, depending on the bentonite and the effective confining stress used in the measurement of hydraulic conductivity. Because hydraulic conductivity decreases with increasing effective confining stress, it is important that the effective confining stress be reported along with hydraulic conductivity. For cover system applications, it is common to report hydraulic conductivity at an effective confining stress of approximately 35 kPa, which is the lower limit of effective confining stress that is recommended for routine commercial hydraulic conductivity testing of GCLs.

GCLs can be reinforced by needlepunched fibers or stitching that increases the internal shear strength of the GCL, which can help to maintain stable slopes. A variety of woven and nonwoven GTs can be used. For GM-supported GCLs, the GM can be smooth or textured, and the thickness can be as little as 0.3 mm or as much as 2 mm. New types of GCLs are being developed, and the materials and configurations are continually expanding and improving.

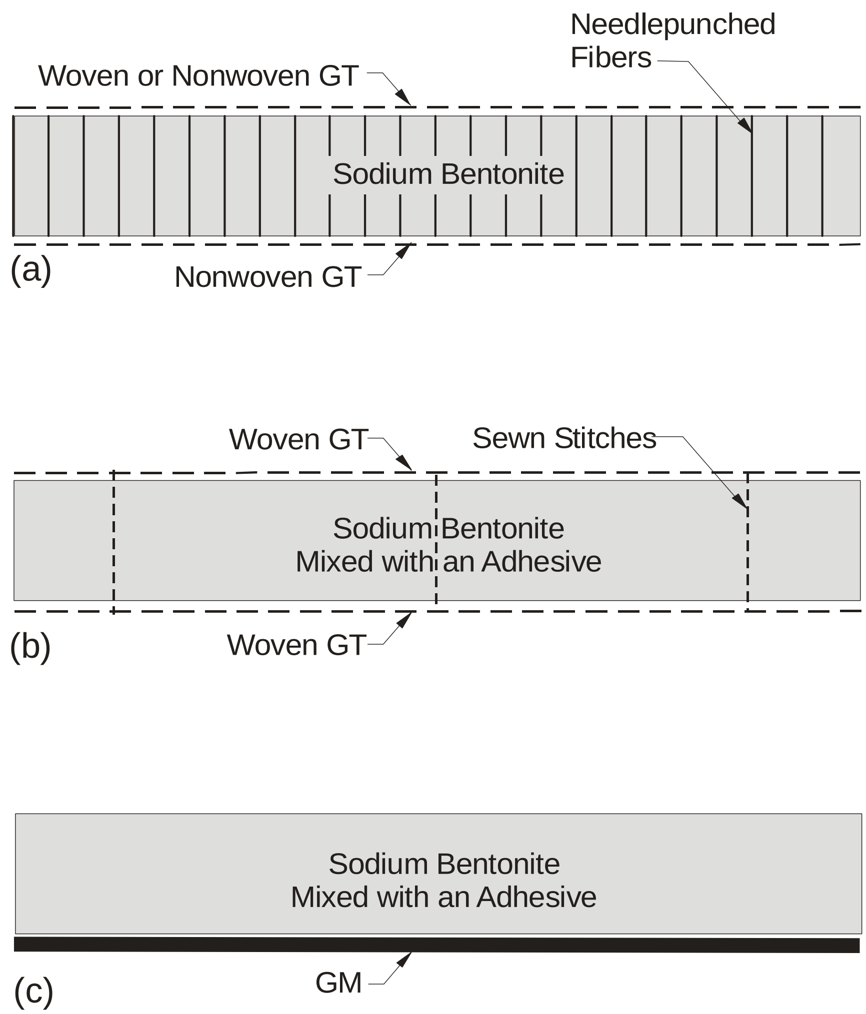


Figure 2-12. Types of GCLs Commonly Used as Cover System Barriers: (a) Reinforced, GT-Encased, Needlepunched GCL; (b) Reinforced, GT-Encased, Stitch-Bonded GCL; and (c) Unreinforced, GM-Supported GCL.

Specifications for GCL hydraulic barriers often require:

- restrictions on bentonite properties (minimum free swell, maximum fluid loss);
- minimum mass per unit area;
- minimum strength and strain requirements; and
- maximum hydraulic conductivity.

Three EPA reports on GCLs have been published (Daniel and Estornell, 1991; Daniel and Boardman, 1993; and Daniel and Scranton, 1996). A detailed discussion of GCLs is provided by Koerner (1998).

In GCL applications, it is important to ensure that the hydraulic conductivity of the GCL is not adversely affected by post-installation chemical changes. The bentonites used in GCLs are sodium-based, which means that the dominant exchangeable cation in the pore water of the bentonite is sodium. When GCLs are placed in contact with soils, the bentonite in the GCL begins to absorb water immediately from the adjacent soils, unless a GM separates the bentonite from the adjacent material. The hydration process is relatively rapid, with significant hydration occurring in a few days and nearly complete hydration occurring within a few weeks. If the cations in the hydrating liquid contain a mix of monovalent and polyvalent cations, little alteration in hydraulic conductivity normally occurs. However, if the hydrating water is rich in polyvalent cations such as calcium, the GCL may not swell adequately or attain the desired low hydraulic conductivity. Even if a GCL is initially hydrated with a water containing few polyvalent cations, the GCL may be affected in the long term if it is permeated by an infiltrating water rich in polyvalent cations. Over time, the indigenous sodium cations in the GCL may be replaced by the polyvalent cations. Calcium-rich soils, or aggregates containing limestone, are of particular concern because they leach calcium. Melchior (1997a,b) and James et al. (1997) document cases in which cation exchange converted the sodium bentonite in GCLs used for cover systems to calcium bentonites, causing an increase in hydraulic conductivity. If the potential exists for leachable cations in overlying surface, protection, or drainage layers to adversely impact GCL hydraulic conductivity, this impact should be evaluated by index testing (e.g., free swell and fluid loss tests) and by hydraulic conductivity testing, for example, as described by Ruhl and Daniel (1997). If necessary, the GCL should be protected with a GM or different materials should be used above the GCL.

One of the potential problems with GCLs is thinning of bentonite if the GCL is placed on sharp objects such as stones or sharp changes in local topography, such as ruts left by vehicles. To avoid these problems, it is recommended that no protruding stones larger than approximately 12 mm be present on the subgrade surface, and that no ruts deeper than about 25 mm be present.

GCLs need to be covered with a GM or an adequate thickness of soil as soon as possible after installation to prevent unconfined hydration. If the GCL hydrates while unloaded, the GCL can swell excessively and potentially extrude laterally as overburden soil is placed. The hydrated GCL also has relatively low shear strength and may impact slope stability. Even if the GCL is covered with a GM, there is still potential for hydration if the underlying subgrade materials are wet or if the waste emits gases that are saturated with water vapor. Daniel et al. (1993) and Bonaparte et al. (1996) provide data on GCL hydration due to contact with compacted subgrade soil.

GCLs also need to be covered with an adequate thickness of soil prior to operating heavy vehicles above the GCL. If adequate protection is not provided, the bentonite can extrude laterally, causing localized thinning (Koerner and Narejo, 1995). Experience from tests reported by Koerner and Narejo (1995) and Fox (1998) indicates that bentonite will not be squeezed laterally in the GCL as long as the thickness of cover soil is at least one to two times greater than the width of the tire load at the surface of the protective soil layer. Based on this, the minimum thickness of cover soil should be about 0.45 to 0.6 m. This should be accomplished in practice since at least 0.3 m of soil is generally maintained between geosynthetics and low-ground pressure tracked equipment and at least 0.6 m of soil is generally maintained between geosynthetics and rubber-tired vehicles.

2.5.2.1.3 CCLs

CCLs are constructed from materials that are mineralogically stable and are well known to design engineers, regulators, and contractors. CCLs offer the advantage over GMs and GCLs in that they are much thicker, which makes them much less susceptible to accidental puncture. Historically, CCLs have been the most frequently used cover system barrier material. Procedures for construction of CCLs to meet permeability criteria are well-established. However, information developed more recently indicated that, when used alone, CCLs in cover systems may not maintain their low permeability in the long term. This is particularly true if a CCL hydraulic barrier is used at an arid or semi-arid site, is located above the depth of frost penetration, or has insufficient overlying cover soil to prevent desiccation cracking. Section 7.2 summarizes a number of field case histories where CCL barriers in cover system applications exhibited increasing permeability with time, even when the CCLs were overlain by cover soils. The increase in permeability is attributed to wet-dry and freeze-thaw effects, root penetration, and differential settlement. Bonaparte et al. (2002) suggest that the best way to maintain low CCL permeability in this application is to overlay the CCL with both a GM and a cover soil with a thickness sufficient for the site-specific conditions. Another limitation of CCLs is their inability to conform to all but the smallest differential settlements of the underlying waste without cracking. Tension cracks starting from the underside of the CCL and propagating upwards through the thickness of the CCL can render them nearly useless as barriers to water infiltration or gas release.

CCLs are constructed primarily from natural soil materials that are rich in clay, although the barrier may also contain processed materials such as bentonite. Specifications for CCLs that must have a hydraulic conductivity of not more than 1×10^{-9} m/s often require (Koerner and Daniel, 1997):

- minimum percentage of fines (particles passing the No. 200 sieve (0.074 mm openings)) ≥ 30-50%;
- minimum plasticity index ≥ 7-15%;
- maximum percentage of gravel (particles retained on the No. 4 sieve (4.76 mm openings) ≤ 20-50%; and
- maximum particle size ≤ 25-50 mm (perhaps less for lifts overlain by a GM).

Local experience may dictate different requirements, and, for some soils, more restrictive criteria may be appropriate. However, if the criteria tabulated above are not met, it is unlikely that a natural soil liner material will be suitable without additives such as sodium bentonite.

If there is concern that rocks or stone in the CCL material may damage an overlying GM, the stones should be removed. Vibratory screens can be used to sieve stones prior to placement or mechanical devices that remove stones in a loose lift can be used. A different material, or a differentially processed material that has fewer and smaller stones, may also be used to construct the uppermost lift of the CCL to be covered by a GM.

CCLs used in cover systems should be as ductile as possible (to accommodate differential settlement) and should be resistant to cracking from moisture variations (e.g., desiccation). Sand-clay mixtures are ideal materials if resistance to shrinkage and desiccation-induced cracking are important (Daniel and Wu, 1993). Ductility is achieved by avoiding use of dense, dry soils that tend to be brittle. If suitable materials are unavailable, local soils can be blended with commercial clays (e.g., bentonite) to achieve low hydraulic conductivity. A relatively small amount of sodium bentonite (typically 2 to 6% by weight) can lower hydraulic conductivity as much as several orders of magnitude. The percent bentonite is usually defined as the weight of bentonite (including a small amount of hydroscopic water) divided by the weight of soil (dry and moist weight have been used, but the dry weight is recommended) to which bentonite is added. Soils with a broad range of grain sizes usually require a relatively small amount of bentonite (i.e., less than 6%). Uniform-sized soils, such as dune sand, usually require more bentonite (i.e., up to 10-15%). Sometimes different soils are blended to provide a material with a broad range of grain sizes, thus reducing the amount of bentonite needed to achieve the specified hydraulic conductivity criterion. For instance, on one project, a coarse to medium sand was successfully blended with bentonite (Alston et al., 1997). By adding 30% of fine, inert material (waste fines from a materials processing plant), the amount of bentonite required was halved. In some cases, GCLs are selected over soil-bentonite CCLs due to economics or ease-of-construction considerations.

2.5.2.2 Thickness

2.5.2.2.1 GMs

The thickness of a GM used in a cover system is selected based upon several factors, the most important of which are durability and capability of being seamed. GMs should be adequately thick to resist construction damage and puncture. The minimum recommended thickness for this purpose is thought to be 0.75 mm. The minimum thickness for adequate field seaming varies with material but is typically in the range of 0.75 to 1 mm. As the GM thickness increases, other mechanical properties also increase. Koerner (1998) suggests that the GM properties given in Table 2-7 be used as a guide to installation survivability, i.e., the ability to be installed without significant damage. GMs should be selected with sufficient thickness to meet the material properties in this table.

2.5.2.2.2 GCLs

GCLs are manufactured with a nominal clay thickness of 5 mm. Like GMs, GCLs are thin and may potentially be punctured during installation. Unlike GMs, however, GCLs possess significant self-sealing capability due to the swelling of dry bentonite upon hydration or the plastic flow of hydrated bentonite. Shan and Daniel (1991) found that holes as large as 25 mm in diameter in a dry GCL swelled shut when the GCL was hydrated, and that the hydraulic conductivity was not significantly affected by the large puncture. However, it is possible to puncture GCLs (e.g., with construction equipment) to the point that self-sealing will not occur.

Table 2-7. Minimum properties for general GM installation survivability suggested by Koerner (1998).

	Required Degree of Installation Survivability[1]			
Property and Test Method	*Low*	*Medium*	*High*	*Very High*
Thickness (ASTM D 1593) (mm)	0.63	0.75	0.88	1.00
Tensile (ASTM D 682, 25 mm strip) (kN/m)	7.0	9.0	11	13
Tear (ASTM D 1004 Die C) (N)	33	45	67	90
Puncture (ASTM D 4833) (N)	110	140	170	200
Impact (ASTM D 3998 mod.) (J)	10	12	15	20

[1] *Low* refers to careful hand placement on a uniform, well-graded, smooth subgrade with light loads of a static nature, typical of vapor barriers beneath building floor slabs.
Medium refers to hand or machine placement on a machine-graded subgrade with medium loads, typical of canal liners.
High refers to hand or machine placement on a machine-graded subgrade of rough texture with high loads, typical of landfill liner and cover systems.
Very high refers to hand or machine placement on machine-graded subgrade of very rough texture with high loads, typical of liners for heap leach pads and floating covers for impoundments.

2.5.2.2.3 CCLs

CCLs are constructed in layers called "*lifts*" that typically have a thickness before compaction ("loose lift") of 0.2 to 0.25 m and a thickness after compaction ("compacted lift") of not more than 0.15 m. Typically three to six lifts are used to produce a CCL hydraulic barrier with a final thickness of 0.45 to 0.9 m. Since each lift of CCL may potentially have areas that do not meet the hydraulic conductivity criterion (as construction of CCLs is, by nature, less controlled than the manufacture of GMs and GCLs), the use of multiple lifts decreases the likelihood that these areas would be continuous through the CCL thickness. A minimum of three compacted lifts is recommended. If the CCL hydraulic barrier is not overlain by a GM, four of more compacted lifts is preferred. It is noted that these recommendations on minimum CCL thickness are based on constructability and performance considerations, not minimum regulatory guidance, which in some cases may allow a thinner CCL.

2.5.2.3 Percolation

The selection of the hydraulic barrier depends to some extent on the allowable rate of water percolation through the cover system. In most instances, the cover system is intended to allow very little infiltration of water into the waste, and the hydraulic barrier is essential to achieving low percolation rates. In other instances, particularly those involving risk-based corrective actions, the amounts of percolation may be less restrictive.

It is recommended that the percolation objective for the cover system be defined, at least qualitatively, prior to design. Methods for estimating percolation rates through cover systems are presented in Chapter 4.

Liquids can migrate through GMs by two mechanisms: (i) permeation through an intact GM; and (ii) flow through GM holes. Fluids permeate GMs by molecular diffusion. The process involves adsorption of the diffusing chemical or compound into the surface of the GM, diffusion through the GM, and desorption from the opposite surface of the GM. Some diffusion rates reported in the literature for GMs are as follows:

- 1.0 mm-thick HDPE: water vapor transmission (WVT) rate = 0.020 g/m^2/day;
- 1.0 mm-thick HDPE: solvent vapor transmission (SVT) rate = 0.02 to 20 g/m^2/day depends on solvent type); and
- 0.75 mm-thick PVC: WVT rate = 1.8 g/m^2/day.

The WVT values are relevant for infiltrating water coming through the cover soil and eventually entering into the underlying waste mass. The SVT values are relevant if there are rising vapors or gases from the waste mass. For MSW landfills, the gases are saturated with water vapor and may contain low concentration of solvents derived from volatilization within the landfill. Diffusion coefficients for various organic solvents and polyethylene GMs are summarized by Rowe (1998). The above WVT rates are relatively low and do not result in significant amounts of water percolation through the hydraulic barrier. While the SVT rates are higher, solvent mass transfer through GM hydraulic barriers will, in most cases, be very low due to the low concentration of solvents in any gas in contact with the barrier layer. The authors caution, however, that while solvent mass transfer through the cover system will be insignificant in most cases, it should be considered in evaluating GM barriers used for capping of remediation source areas which may contain a significant solvent mass.

Of greater significance than water vapor diffusion is flow through GM holes, such as tears, punctures, or imperfect seams. Flow through such holes in a GM alone usually significantly exceeds the diffusion values listed above (EPA, 1991). If the GM is underlain by a GCL or CCL to form a composite barrier, water migrating through a GM hole or defect will be impeded by the underlying GCL or CCL. Flow through the GCL or CCL will then be limited by the area of the GM hole(s), which is only a small fraction of the total area of the barrier, and any lateral flow at the interface of the GM and the GCL or CCL. The amount of interface flow is a function of the "intimacy" of the contact between the GM and GCL or CCL components (Giroud and Bonaparte, 1989b; Gross, et al., 1990). If there is good contact between the GM and underlying GCL or CCL, the flow rate through a GM hole will be very low (unless the hydraulic head acting on the hole becomes very large, which is usually not the case). The relative performance of GM and composite barriers is apparent when analyzing field data on apparent leakage rates through the top liners of double-lined landfills. As described by Gross et al. (1997) and Othman et al. (2002), the data indicate that GM barriers have a representative hydraulic efficiency of 99% and GM/GCL and GM/CCL composite barriers have a representative efficiency of 99.9%, where efficiency is defined as the percentage of lateral drainage that flows from the drainage layer rather than percolates through the barrier. Methods of estimating leakage though holes in GMs alone and GM/CCL and GM/GCL composite barriers have been presented by Giroud and Bonaparte (1989a, 1989b), Giroud et al. (1989), Giroud et al. (1992), Giroud (1997), Rowe (1998), and Foose et al. (2001). Recommendations on the use of the different leakage models are presented by Foose et al. (2001).

Percolation through GCL or CCL barriers is typically estimated using Darcy's equation for saturated conditions or Richards' partial differential equation for unsaturated conditions (Richards, 1931).

2.5.2.4 Gas Containment

When there is a need for gas containment, GMs are generally the best barriers to gas. GCLs and CCLs also make very good gas barriers when they are at high degrees of saturation and do not contain major secondary structures, such as desiccation cracks extending through the GCL or CCL.

2.5.2.5 Differential Settlement

Differential settlement is usually quantified in terms of the magnitude of differential settlement (Δ) that occurs over a distance (b), yielding angular distortion, Δ/b (Gilbert and Murphy, 1987), as shown in Figure 2-13. Angular distortion may damage barriers because distortion produces tensile strains, and tensile strains can cause barrier materials to fail if the strains are excessive. Tensile strains are generated by the material elongation associated with geometric distortion. A relationship between angular distortion and tensile strain is shown in Figure 2-13.

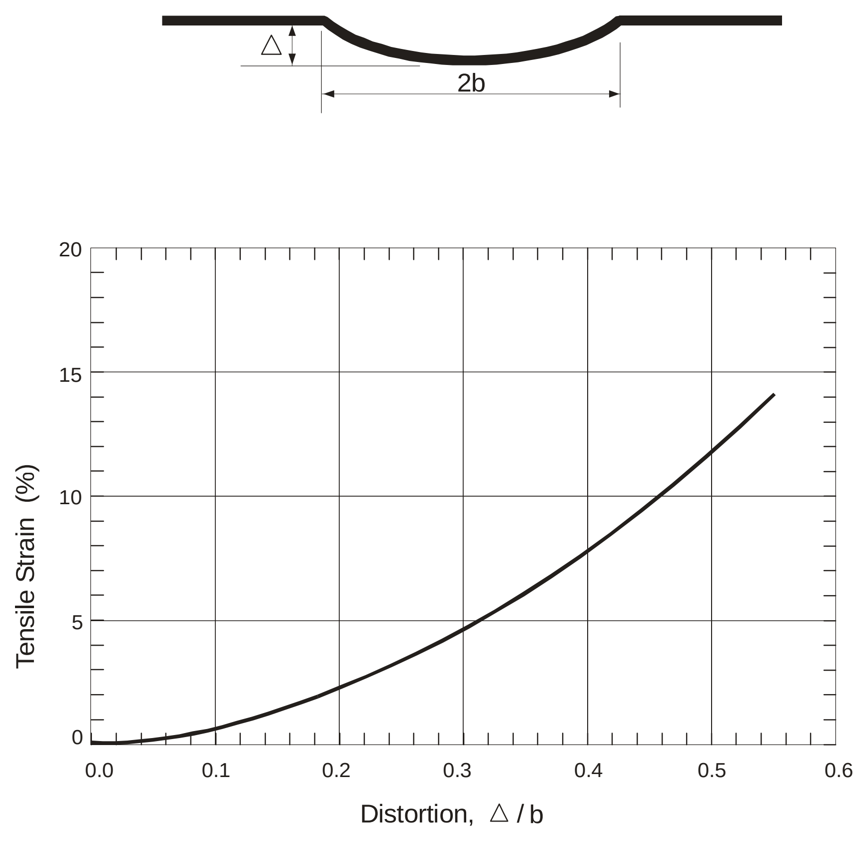


Figure 2-13. Theoretical Relationship Between Tensile Strain and Angular Distortion (modified from Gilbert and Murphy, 1987).

Procedures for estimating total and differential settlements are discussed in Chapter 6.4. Frequently, the estimates of Δ/b that are used for design are based primarily on experience and observations. The magnitude of Δ/b that is expected is highly site dependent and is a function of variables such as type of waste, age of waste, details of waste placement, and thickness of the cover system. The impact of settlements on hydraulic barriers is discussed in detail in Section 6.5.

The selected barrier materials should be able to accommodate the anticipated settlements. Axi-symmetric, out-of-plane tests on various GMs have resulted in the stress-strain curves shown in Figure 2-14. The ability of the different GMs to accommodate differential settlement is lowest for chlorosulfonated polyethylene-reinforced (CSPE-R) and HDPE and highest for VLDPE, LLDPE, and PVC. As previously mentioned, VLDPE and LLDPE are both in the VFPE category. If significant differential settlement is anticipated, as with cover system barriers over MSW, the use of GMs that can accommodate high out-of-plane, or axisymmetric, deformations should be considered.

Test results published by Koerner et al. (1996) and LaGatta et al. (1997) indicate that reinforced GCLs can withstand tensile strains of 5 to 16%, depending on product. Care should be taken to ensure an adequate overlap width, since, under elongating conditions, slippage may occur along overlaps.

CCLs can accommodate little tensile elongation. As described in Section 6.5, CCLs will typically exhibit tensile failure at extensional strains of 0.5% or less.

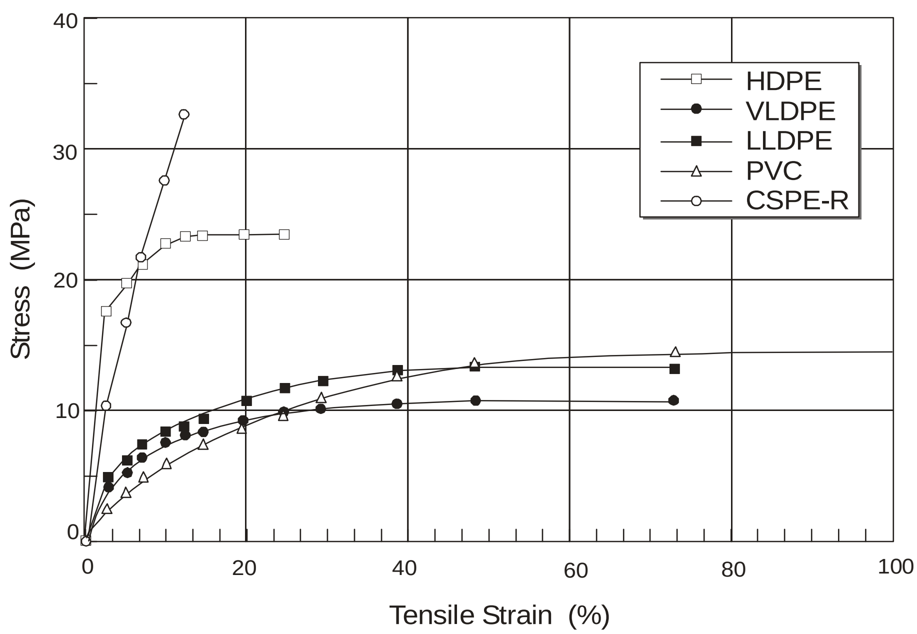


Figure 2-14. Stress-Strain Behavior of Common GM Materials Subjected to Axi-Symmetric, Out-of-Plane Tensile Strain (modified from Koerner et. al., 1990).

2.5.2.6 Wet-Dry Cycles

The potential for wet-dry cycles to affect the integrity of CCLs and, to a lesser extent, GCLs, should be considered whenever these materials are used as hydraulic barriers. Water balance analyses, such as those described in Chapter 4, can be helpful, but judgment should play an important role in the evaluation process. If damage to a CCL or GCL is anticipated, the normal solution is to use a composite GM/CCL or GM/GCL hydraulic barrier overlain by a protection layer.

Cyclic wetting and drying can have a significant impact on the hydraulic conductivity of CCLs under low confining pressures. As drying progresses, shrinkage occurs and reaches a limit at which cracking can occur. This cracking, caused by desiccation, occurs in block form, and gradually progresses deeper into the CCL until a pathway of water migration becomes available. Besides drying as a result of ET, CCLs may also lose moisture to materials (e.g., a dry soil foundation layer) beneath them.

Both soil dry density and soil water content affect the vulnerability of the soil to desiccation cracking (Albrecht and Benson, 2001). Highly plastic clays undergo large shrinkage when dried; clayey sands undergo little shrinkage. A given CCL material experiences less shrinkage when it is compacted at its optimum moisture content and with a high compactive effort as compared to the shrinkage of the same soil compacted to wetter or less dense conditions. Shrinkage and cracking can occur in CCLs as a result of water content changes of only 2 to 5 percentage points. Moisture content variations of this magnitude are inevitable in the top 1 to 2 m of soil at most sites. With the reintroduction of water, swelling occurs and the cracks start to close. However, the degree to which the cracks swell shut is highly dependent on overburden pressure (Boynton and Daniel, 1985). At overburden stresses of less than 40 to 100 kPa, cracks do not fully close, even after the soil is soaked. The overburden stress on CCLs in cover systems is typically less than 25 kPa. Thus, in cover systems, the remnants of desiccation cracks are likely to remain, causing the hydraulic conductivity to increase over its as-constructed value.

Experience has shown that severe desiccation can occur to depths of up to 1 m, and possibly deeper (Montgomery and Parsons, 1989, 1990; Corser and Cranston, 1991; Corser et al., 1992; Melchior et al., 1994; Melchior, 1997a,b; Maine Bureau of Remediation and Waste Management, 1997; and Khire et al., 1997, 1999). The information that is available on desiccation spans a period of field observation of approximately five years. Over longer periods, the depth of impacts associated with wet-dry cycling could extend even deeper. It is recommended that at least 1.2 m of cover soil, and possibly more, be used to protect the CCL (assuming that it is not overlain by a GM) from desiccation cracking. Even greater thicknesses (e.g., 1.5 m) may be necessary in certain cases.

Depending on the chemistry of the permeating water, GCLs may or may not be vulnerable to permanent damage from desiccation. When permeated with water containing little salts, GCLs are less vulnerable than CCLs to permanent damage from desiccation, because of the swelling and self-healing capability of bentonite (Boardman and Daniel, 1996; Lin and Benson, 2000). Data published by Shan and Daniel (1991), Boardman and Daniel (1996), and Lin and Benson (2000) indicate that, under this condition, GCLs can withstand at least five cycles of wetting and drying without a significant increase in long-term hydraulic conductivity. However, if the permeant contains cations that may exchange with the sodium in the GCL bentonite, the barrier will loose some capability to swell and recover from desiccation over time. GCLs have been damaged for this reason in at least several field installations (Melchior, 1997; James et al., 1997).

Though GCLs may have significant swelling and self-healing capability following wet-dry cycles, it is not recommended that these barriers be exposed to these cycles. There is concern that the GCLs may lose their self-healing capability over time due to cation exchange. This is especially a concern at sites in semi-arid and arid climates, since barriers may become saturated in the winter months and very dry in the summer months. Pore water in these environments also tends to have higher salt concentrations than that in more humid climates.

The best approach for protection of a CCL or GCL from desiccation is to place a GM over the barrier, and then cover the GM with soil.

2.5.2.7 Freeze-Thaw Cycles

The potential for freeze-thaw of the hydraulic barrier should be evaluated, as discussed in Section 2.3.2.2.2. If the hydraulic barrier is located below the maximum depth of frost penetration, then the barrier is usually assumed to be adequately protected from long-term frost damage. If the barrier is within the zone of frost penetration, then the impacts of frost upon the barrier materials should be considered.

Frost is generally believed to have no effect on GMs (Comer et al., 1995). This is only true, however, if the GM is buried such that stresses induced by thermal contraction do not cause tensile failure of a GM. An exposed GM (i.e., an exposed GM cover system) will undergo much larger temperature fluctuations than one buried beneath a thick layer of cover soil.

Laboratory data (Hewitt and Daniel, 1997) as well as field data (Erickson et al., 1994; Kraus et al., 1997) suggest that GCLs can withstand multiple cycles of freeze-thaw with little or no adverse effect on the thawed hydraulic conductivity of the GCL. However, the GCL test data available at this time are relatively short-term. In addition, there is the potential for GCLs to become damaged if they desiccate under freezing conditions and then rehydrate with water containing exchangeable cations. If desiccation/rehydration of GCLs is a concern, suitable approaches for GCL protection are to place the GCL beneath a sufficiently thick soil layer or to cover the GCL with a GM.

Freezing temperatures can cause desiccation and freeze-thaw cracking in CCLs, resulting in barriers with increased permeability to water and gas. Desiccation cracking occurs as water is drawn from a CCL and towards a freezing front. Freeze-thaw cracking occurs as the ice lenses form in the CCL. Available information indicates that CCLs will not maintain a hydraulic conductivity of 1×10^{-9} m/s or less if subjected to freeze-thaw at the level of overburden stress normally encountered in cover systems. Instead, the CCL hydraulic conductivity will increase by one to two orders of magnitude (Othman et al., 1994). The exception to this appears to be for compacted soil-bentonite CCLs (Wong and Haug, 1991), which do not appear to be vulnerable to damage from freeze-thaw action. If CCL damage by frost action is a concern, suitable approaches for CCL protection are to place the barrier beneath a sufficiently thick soil layer or to cover the CCL with a GM and then a soil layer.

2.5.2.8 *Shear Strength*

Measurement of the shear strength parameters of different barrier materials is discussed in some detail in Section 6.2.4. Specific issues relevant to barrier strength are discussed in this section.

GMs can have a low interface shear strength when placed adjacent to certain materials, such as GNs or GTs. For some interfaces (e.g., GM/GT), the shear strength can be significantly enhanced by using a textured GM. There are a number of manufacturing methods available to provide such texturing:

- co-extrusion for blown film manufacturing;
- impingement for flat die manufacturing;
- lamination for flat die manufacturing; and
- structuring via a heated calendar for flat die manufacturing.

Perhaps the single most important design issue for GCLs that are placed in cover systems is slope stability. When GCLs are installed on slopes, instability can occur by at least four different mechanisms: (1) slippage at the interface between the upper surface of the GCL and overlying material; (2) shearing within the GCL; (3) slippage at the interface between the lower surface of the GCL and the underlying material; and (4) a combination of the first three mechanisms. The first and third mechanisms are termed “interface” failures, and the second one is termed an “internal” failure. Laboratory test methods to evaluate the shear strength of GCLs are discussed in Section 6.2.4. Specific testing issues for GCLs are discussed below.

The response of GCLs to shearing stresses depends on the hydration conditions. Wet bentonite is far weaker than dry bentonite and, therefore, the internal shear strength of hydrated GCLs can be much lower than that of dry GCLs. An example is shown in Figure 2-15 for an unreinforced GCL. If the GCL is expected to become hydrated by absorbing moisture from subgrade soils or by other mechanisms, the shearing tests are normally performed on hydrated GCLs. It is important to realize that the bentonite does not have to be completely saturated to be weakened from hydration; the bentonite need only absorb significant moisture from the subgrade soil to have the low shear strength of hydrated bentonite (Figure 2-16).

Reinforcement can significantly increase the internal shear strength of GCLs. As shown in Figure 2-15, the peak failure envelope for internal shear of reinforced GCLs is much higher than the peak failure envelope for unreinforced GCLs, but the residual strengths for reinforced and unreinforced GCLs are about the same because at residual conditions, the internal reinforcement has been broken.

Slippage may occur at the interface between a GCL and adjacent materials. Because GCLs may be manufactured from woven or nonwoven GTs, and from smooth or textured GMs, a wide range of interface shear responses may be observed. Further, GCLs may interface with a wide range of soil and geosynthetic materials. No general statements can be made about the actual shear strength of interfaces: there are so many permutations possible that each specific interface should be evaluated through interface shear testing.

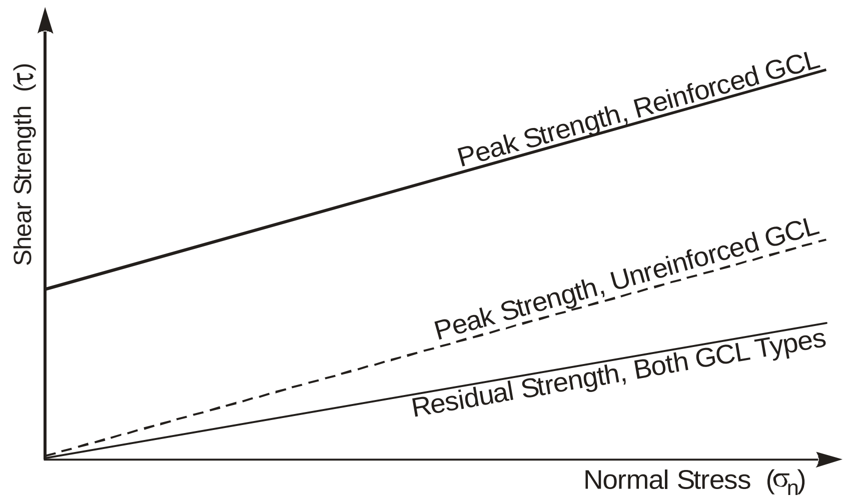


Figure 2-15. Comparison of Shear Strengths for Internally Reinforced GCLs and Unreinforced GCLs.

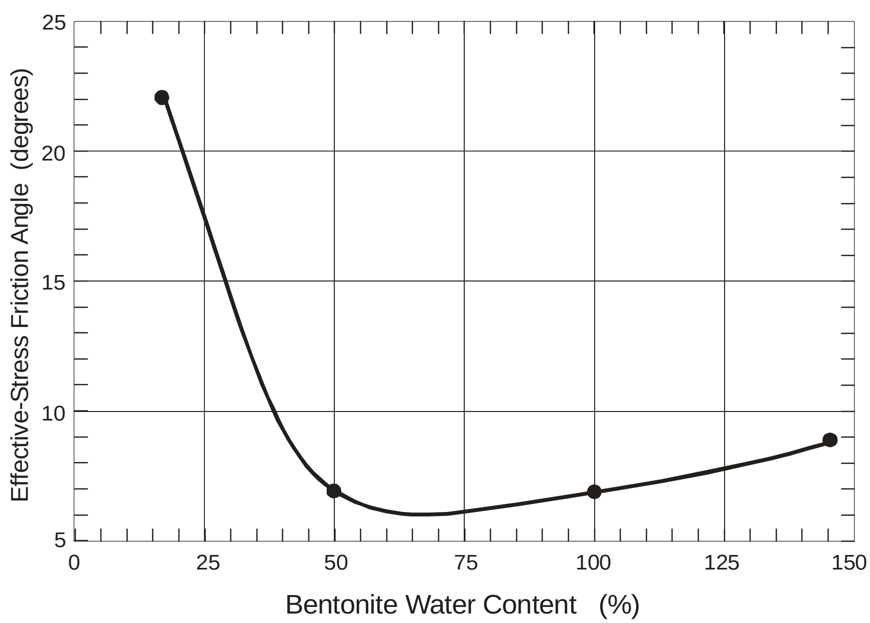


Figure 2-16. Effect of Bentonite Water Content on Shear Strength of an Unreinforced GCL (modified from Daniel et al, 1993).

Experience has shown that certain design situations involving GCLs installed on slopes warrant particular attention:

- Cover system (i.e., low normal stress and no seepage forces) slopes that are inclined at 6H:1V or flatter will be stable with a FS of 1.5 or more with respect to unreinforced GCL internal shear strength and interfaces with the GCL. Steeper slopes may also be stable but require careful testing and analysis.
- Those GCLs with woven slit-film GTs on one or both surfaces should be carefully evaluated to be sure that hydrated bentonite does not extrude and lubricate the adjacent material interface (upper and/or lower) and cause a reduction in interface shear strength compared to the shear strength in the absence of extrusion.
- Designs that rely on the dry shear strength of GCLs for stability should assure that the GCLs will be fully and completely protected against hydration. This is usually possible only by having GMs on both surfaces of the GCL and, in addition, having construction and deployment conditions in the field that do not allow the GCL to absorb moisture.
- The internal shear strengths of needlepunched and stitch-bonded GCLs appear to be adequate to achieve internal stability of the GCLs on cover system slopes as steep as 2H:1V with a FS of 1.5 or more. However, interface shear strengths for these types of GCLs at cover system normal stresses will often be less than the internal shear strength and at a 2H:1V slope it is likely that the cover system will be unstable or only marginally stable.
- For cover systems with soils and textured GMs having interfaces with internally-reinforced GCLs, slopes as steep as 3H:1V can be constructed and remain stable at a FS of 1.5 or more (in the absence of seepage forces), but actual stability depends on the particular materials used.
- Woven GTs generally have lower interface shear strength with materials such as soil or other geosynthetics than non-woven GTs. If high interface shear strength is required with a GT-encased GCL, a GCL with non-woven GTs on both surfaces is usually required. Many times the critical interface will be between a GCL and overlying GM. In this situation, high interface shear strength is usually achieved by installing a nonwoven GT component of the GCL with a textured GM. The fibers of the non-woven GT become entangled with the ridges on the textured GM, creating what some have described as the "Velcro effect" in which high adhesion is developed. However, under large deformations along the interface, a polishing of the materials may occur, and the residual strength may be much lower than the peak strength. Clearly, the shearing response of GCL interfaces can be very complex and requires careful testing and engineering.

The shear strength of a CCL, and particularly a GM/CCL interface, can be critical to the stability of a cover system. Low hydraulic conductivity is most easily achieved by adding water to the clay and compacting it wet of its optimum water content. However, the conditions that tend to result in a low CCL hydraulic conductivity also tend to cause low interface shear strength. The selection of appropriate water content-density parameters is usually a compromise between the need for low hydraulic conductivity and the need for adequate shear strength. The design engineer should not focus solely on achieving low CCL hydraulic conductivity to the extent that inadequate attention is given to the shear strength of the CCL and CCL interfaces with other materials.

2.5.2.9 Accidental or Intentional Puncture

The potential for accidental (due to construction and operational activities) or intentional breach of the hydraulic barrier should be considered in the design of cover systems. With respect to this issue, the thinness of both GMs and GCLs is a disadvantage in contrast to the typical thickness of CCLs. In evaluating GCLs, however, the sealing potential of bentonite should be considered. This is not the case for GMs. Thus CCL, GM/GCL, or GM/CCL hydraulic barriers are superior to GM barriers alone from the standpoint of resistance to puncture.

2.5.2.10 Anticipated Lifetime

The anticipated lifetime of the barrier material should be considered in relation to the required design lifetime of the cover system. In this regard, reference should be made to Section 1.2.6 of this document, where a distinction is made between the minimum post-closure period and the design life goal of a cover system. The anticipated lifetimes of the different hydraulic barrier materials are discussed below.

2.5.2.10.1 GMs

For GMs, aging involves a gradual transition from a ductile material to a brittle material. As embrittlement occurs, the GM does not disappear; rather settlement, deformation, seismic vibration, etc. can cause a brittle cracking, signifying the end of the material's functional life.

The service life of any GM component of the cover system is dependent on the specific material used and how well the material is protected. While the degradation mechanisms leading to GM embrittlement are many, the most severe ones are eliminated by the timely protection of the GM after installation with cover soil or other materials. For example, the potential for polymer degradation by ultraviolet light and elevated temperature is essentially eliminated by placement of cover soil over the GM. Furthermore, the potential for chemical degradation of a cover system hydraulic barrier may not be an issue since the cover system is located above the waste. The possible exception to this is for wastes that generate gases or vapors that may bring volatile chemicals at high enough concentrations to the underside of the GM. The primary mechanism of degradation of a GM hydraulic barrier in a cover system is oxidation of the polymers causing embrittlement over a long time period.

Conceptually, the oxidation of GMs can be considered in three distinct stages. These stages are designated as: (i) depletion time of antioxidants; (ii) induction time to the onset of polymer degradation; and (iii) degradation of the polymer to decrease some properties to a defined level (e.g., 50% of its original value). The purpose of antioxidants in a GM formulation is to prevent polymer degradation during processing and to prevent polymer oxidation reactions from taking place during the first stage of service life. However, there is only a limited amount of antioxidant in any formulation. Hence, the lifetime for this stage is limited to the specific amount of antioxidant used. Once the antioxidant is depleted, oxygen or other strong oxidizing agents will begin to attack the polymer, leading to the induction time stage and subsequently to the degradation of performance properties. The duration of the antioxidant depletion stage also depends on the type of selected antioxidant. Many different antioxidants are commercially available, and depletion time will vary from formulation to formulation. Proper selection of antioxidants is known to contribute greatly to the overall lifetime of the GM. For example, Hsuan and Koerner (1996) reported an antioxidant depletion time of about 130 years at 25°C for an HDPE GM formulation with approximate 0.5% antioxidant package. The testing was conducted for a simulated landfill environment with the GM placed on a layer of dry sand, covered with sand and then 0.3 m of water, and subjected to a compressive stress of 260 kPa. Note that this antioxidant depletion time is for HDPE, which is considered to be the most stable of polymers being used in GMs. Research is ongoing for GMs using time-temperature superposition procedures followed by Arrhenius modeling (Hsuan and Koerner, 1998; Hsuan and Koerner, 2002). The most extensive service life data currently available are for HDPE GMs. Hsuan and Koerner are currently evaluating the antioxidant depletion time for other polymers in a like manner.

In properly formulated GMs, oxidation does not begin to occur until after the depletion of the antioxidant. Oxidation of the polymer occurs only very slowly in a buried soil environment. The initial stage of oxygen absorption is called the induction stage. It is the time period in which there is no measurable change in the physical-mechanical properties of the GM. The reason for this is related to the concentration of hydroperoxide, as described below. The first step of oxidation (after depletion of the antioxidants) is the formation of free radicals. The free radicals subsequently react with oxygen and start chain reactions. The free radicals are highly reactive in that they cause chain scission of the polymer backbone, which gradually results in the embrittlement of the material. In the induction stage, little hydroperoxide is present and, when formed, it does not decompose. As a result, accelerated oxidation reactions do not occur. As oxidation propagates slowly, additional hydroperoxide molecules are formed. Once the concentration of hydroperoxide reaches a critical level, decomposition of the hydroperoxide begins and accelerated chain reactions start. This signifies the end of the induction period (Rapoport and Zaikov, 1986). This also indicates that the concentration of hydroperoxide has a major effect on the duration of the induction period.

The duration of the induction stage for HDPE can be estimated from data for plastic pipes and testing conducted on HDPE waste exhumed from a landfill (Hsuan and Koerner, 2002). Viebke et al. (1994) presented aging data for unstabilized medium density polyethylene pipes that were tested with pressurized water inside and circulating air outside and at temperatures ranging from 70º to 105°C. They found the activation energy of oxidation in the induction period to be 80 KJ/mol. Using their experimental values, an induction time for medium density polyethylene of 12 years was extrapolated at a typical in-service temperature of 25°C. This value is consistent with the approximately 20-year induction time estimated for 25-year old HDPE water and milk bottles exhumed from a landfill. Milk and water bottles are one of a few commercial HDPE products that do not contain antioxidants because of their limited shelf life. The exhumed bottle materials were considered to show no signs of degradation since their yield stress, yield strain, and modulus values had not changed significantly from those measured for new milk and water bottles. However, there was a decrease of approximately 30% in the break strength and break elongation values, signifying that the induction stage was essentially completed and degradation had begun.

The end of the induction stage signifies the onset of relatively rapid oxidation. This is the third, and final, stage in GM degradation. Oxidation proceeds more rapidly because the free radicals increase significantly via the decomposition of hydroperoxide. One of the free radicals is an alkyl radical, which represents polymer chains that contain a free radical. In the early stage of acceleration, cross-linking occurs in these alkyl radicals due to oxygen deficiency. The physical and mechanical properties of the material subsequently respond to such molecular changes. The most noticeable change is in the melt index, since it relates to the molecular weight of the polymer. In this stage, a lower melt index value is detected. In contrast, the mechanical properties do not seem to be very sensitive to cross-linking. The tensile properties (stress, strain and modulus) generally remain unchanged or are undetectable. As time proceeds further, and oxygen continues to be available, the reactions of alkyl radicals change to chain scission. This causes a reduction in molecular weight. In this stage, the physical and mechanical properties of the material change according to the extent of the chain scission. The melt index value reverses from the previous low value to a value higher than the original starting value signifying a decrease in molecular weight. As for tensile properties, break stress and break strain decrease. Tensile modulus and yield stress increase and yield strain decreases, although to a lesser extent. Eventually the GM material becomes brittle in that the tensile properties change significantly and engineering performance is compromised, as described previously. This signifies the end of the so-called service life of the GM.

Although arbitrary, researchers have assumed that the end of service life of a GM material occurs when the relevant engineering properties reduce to 50% of the initial values. This is commonly referred to as the half-lifetime, or simply the half-life. The specific property could be yield stress, yield strain, or modulus of HDPE or the comparable break properties of resins that do not show a pronounced yield point. It should be noted that even at its half-life the GM still exists and can function albeit at a decreased performance level. Using the previously mentioned Viebke et al. (1994) aging data, the half-life of unstabilized polyethylene has been estimated to be approximately 440 years at an in-service temperature of 25°C (Hsuan and Koerner, 2002).

Considering the three stages of GM oxidation, the anticipated service life for commercially-available HDPE GMs will be measured in terms of at least several hundred years. Other types of GMs, particularly those with greater amorphous phase material, may have different service lives from that for HDPE GMs. Great care should be used in specifying GM materials to require products that, through polymer type, additive (e.g., antioxidant) packages, physical robustness, etc., are capable of achieving as long a service life as possible.

2.5.2.10.2 GCLs

Little information currently exists on the service life of GCLs. Adequately protected and absent of external degradation mechanisms, the service life of bentonite is indefinitely long. However, long-term bentonite degradation is a concern if there is potential for cation exchange. In addition, both durability and chemical compatibility are issues with respect to the reinforcing fibers or yarns of GCLs placed on sideslopes. While the EPA test plots described by Daniel (2002) and summarized in Section 7.4.5 go far to show the validity of such GCL reinforcement, the performance of this reinforcement over a 30 or 100-year time frame is unknown.

2.5.2.10.3 CCLs

For CCLs the anticipated service life is also difficult to assess, generally not from the perspective of the soil particles themselves, but for the necessary association of the soil particles with water. Clearly, the soil particles of a CCL will last for geologic time. However, if the CCL barrier material should desiccate or suffer freeze-thaw cycling, its hydraulic conductivity will be compromised. If a CCL is protected from freeze-thaw and other environmental effects, and not subjected to excessive differential settlements, its anticipated service life is indefinitely long (Mitchell and Jaber, 1990).

The lifetime of a CCL is clearly material and site specific. Factors that can impact the service life of CCLs are summarized in Table 2-8.

Table 2-8. Factors affecting the anticipated service life of CCLs.

Factors Promoting a Longer CCL Service Life	Factors Leading to a Shorter CCL Service Life
Use of clayey sand or soil-bentonite mixture	Use of highly plastic clay
Placement and compaction of soil at a relatively low water content (e.g., on line of optimums)	Placement and compaction of soil at a relatively high water content (e.g., much wetter than line of optimums)
Placement of CCL beneath 1 to 2 m or more of cover soil	Placement of CCL beneath less than 1 m of cover soil
Protection against desiccation provided by a GM or other type of vapor barrier	No GM or other vapor barrier provided
Climate with high rainfall year-round and light to moderate drought periods of short duration	Climate with highly variable rainfall and with prolonged droughts occasionally occurring
Cool climate that minimizes ET	Climate with periods of year with warm temperature and high ET or periods with freezing temperatures

2.5.3 Composite Hydraulic Barriers

A cover system with a GM/GCL, GM/CCL, or GM/GCL/CCL composite barrier allows significantly less percolation compared to the same cover system with a GM, GCL, or CCL barrier alone (see Section 2.5.2.3). The GM component provides protection to the underlying GCL or CCL. The GM prevents penetration of plant roots and burrowing animals into the GCL or CCL in most applications. The GM also protects the GCL or CCL from desiccation. The GCL or CCL, in turn, serves to reduce the rate of leakage through occasional imperfections in the GM.

2.5.3.1 Prompt Placement of Overlying Materials

An interesting aspect of construction of a GM/CCL composite is that the work is generally performed by two separate contracting organizations. The CCL is usually constructed by an earthwork contractor and the GM is often installed by a geosynthetics installer. They rarely are the same organizations. Thus, timing and coordination can be a challenge. To protect the CCL from desiccation, freezing, and other stressors, the GM should be placed over the CCL as soon as possible after the final lift of CCL is placed and accepted. In turn, after the GM in installed, overlying layers (soil and geosynthetics) should be placed as quickly as reasonably possible. However, all too often, days, weeks or even months pass after completion of the CCL and before GM placement, and a similar time lag can occur with respect to the placement of overlying materials. During this gap in construction activity, the CCL must be protected. This is difficult since the CCL can desiccate even if left exposed for only a few days. For short-term protection, the completed CCL should be covered by a 0.15 to 0.3 m or even thicker layer of clayey soil that is periodically moistened and then stripped away just prior to placement of the GM.

With a GM/GCL composite liner, the GCL also should be covered with a GM as soon as possible after installation. For GCLs, the biggest concern is that of pre-mature hydration.

A particular problem with GM/CCL composite liners is desiccation of the CCL when the GM has been placed and left exposed (not covered with soil). Data reported by Bowders et al. (1997) show that the exposed GM component can heat and cause desiccation of underlying clay soils over a period of a few weeks. Desiccation occurred more rapidly with black-surfaced GMs than with white-surfaced GMs since white-GMs reflect radiant heat, which decreases their surface temperature. To minimize the potential for CCL desiccation, it is recommended that the GM be covered as quickly as reasonably possible, which typically will mean that it not be left exposed for more than several days to a few weeks prior to covering with soils. Consideration should also be given to using light colored GMs.

If a GM/GCL composite barrier is used, the GM should also be covered as quickly as reasonably possible, not so much over concern related to desiccation of the GCL, but, rather, over concern related to the need to apply overburden pressure to the GCL to prevent bentonite extrusion.

2.5.3.2 Intimate Contact

Regarding intimate contact of a GM with an underlying CCL, the surface of the CCL should be smooth rolled with a steel-drummed roller before the GM is placed, and the incidence of wrinkles, or waves, in the GM should be minimized. Wrinkles form in the GM after initial placement and subsequent heating during the day. At night, as the temperature declines, the GM contracts, and the wrinkles are reduced (provided too much slack is not installed in the seamed system). Wrinkles are more pronounced in the stiffer and thicker GMs (e.g., HDPE), but wrinkles occur in all types of GMs because their expansion/contraction characteristics are largely the same (Koerner, 1998). The issue with wrinkles is not that they form when the GM heats and expands, but, rather, that as cover soils are placed on the GM the wrinkles may be trapped, reducing contact between the GM and the underlying material. The trapped wrinkles may also fold over, inducing stresses in the GM.

To limit the trapping of wrinkles, cover soils should not be placed over GMs when excessive wrinkles are present. Thus, cover soil placement should occur from daybreak until a time when daytime heating causes wrinkles to develop. Cover soil placement can also be performed at night. If night placement occurs, however, special precautions are needed to assure worker safety, and intensified CQA monitoring should be conducted in recognition of the low light conditions.

To reduce wrinkle formation, white-surfaced GMs may be considered. White-surfaced GMs reflect more radiant heat than black-surfaced GMs, and, thus maintain a lower temperature than black-surfaced GMs. Consequently, white-surfaced GMs experience less thermal expansion, such that wrinkle heights are reduced by approximately one-half (Koerner and Koerner, 1995). Since sunlight exposure is less of a factor with white-surfaced GMs, backfilling can continue longer into the day for this GM type than for black-surfaced GMs.

On long sideslopes, it may be preferable to use textured GM rather than smooth GM to decrease the size of GM wrinkles that develop, especially near the slope toe. Giroud (1994) has shown analytically that GM wrinkles are shorter and spaced closer together when the shear strength between the GM and the underlying material is increased. Therefore, based on analysis, the use of textured, rather than smooth, GM decreases the potential for large wrinkles to form.

For GM/GCL and GM/GCL/CCL composite barriers, lateral transmission of liquid in the upper GT of the GCL has been evaluated by Harpur et al. (1994) and found to be of little concern. Apparently, as the bentonite hydrates it fills in, or extrudes through, the voids of the GT, greatly decreasing the transmissivity of the GT adjacent to the GM. This, however, gives concern in another respect. That is the possibility of decreasing the shear strength of the GM/GCL interface. Proper direct shear testing and slope stability analyses are required when this type of composite barrier is on steep sideslopes.

2.5.4 Construction

The manufacture, installation, QC, and CQA of GMs and GCLs and the construction, QC, and CQA of CCLs are discussed in detail by Daniel and Koerner (1993, 1995). That detailed discussion is not repeated herein.

In brief, GM and GCL hydraulic barriers are manufactured in panels of certain widths and lengths. GM panels are connected by seaming using thermal processes (extrusion or fusion seaming) for HDPE, VFPE, PVC, fPP, or fPP-R GMs or chemical processes (chemical fusion or adhesive seaming) for fPP, fPP-R, and PVC GMs.

GCL panels are connected by overlapping. Often, dry powdered or granular bentonite is placed within the overlap, and this practice is recommended. For GM-supported GCLs, the GM is welded in the field. Most specifications for GCL installation require that the GCL be covered before it becomes hydrated, and this practice is also recommended. It is common practice not to deploy more GCL than can be covered before a rainstorm could develop.

CCLs are constructed by processing a soil and then compacting it with a certain applied energy to a specified range of moisture contents and dry densities. The selection of moisture contents and dry densities for construction specifications should not be done arbitrarily but, rather, should be based on the results of laboratory hydraulic conductivity tests performed on samples of the proposed soil material. The resulting compaction criteria may then be narrowed based on other engineering considerations, such as shear strength and shrinkage potential. The recommended procedure is described by Daniel and Benson (1990), and Daniel and Koerner (1993,1995) and has more recently been updated by Benson et al. (1999). The approach described by Daniel and Wu (1993) is recommended for establishing appropriate moisture content-density criteria that will ensure both low as-built hydraulic conductivity and good resistance to desiccation cracking.

Heavy, footed compactors with large feet that fully penetrate a loose lift of soil are ideal. Rollers with feet that fully penetrate a loose lift of soil pack the base of a new lift into the surface of the previously-compacted lift, which helps to bond lifts together. The long feet also help to break down and remold clods of soil over the full thickness of a lift. Recommended compactor specifications include a minimum mass of 18,000 kg and minimum foot length of 180 to 230 mm (but the foot should have a length no smaller than the thickness of a loose lift). However, in many landfill cover systems it is simply not possible to use such heavy compactors because the foundation (underlain by waste at shallow depth) may not be adequate to support the weight of the equipment. Lighter-than-ideal equipment will need to be used in such cases. To compensate for the light weight, it may be necessary to use thinner lifts and more passes of the compactor. When a gas collection layer is overlain by a CCL, the first lift of the CCL is sometimes compacted with a somewhat thicker lift thickness so that the feet of the compactor don’t penetrate though the CCL and damage the underlying materials. Alternatively, the first lift of the CCL is sometimes compacted to its specified maximum thickness with compactors having shorter feet, rubber-tired equipment, or other equipment. This first lift is generally required to meet compaction criteria, but may not be required to meet a permeability criterion (i.e., laboratory or field permeability testing of the first lift of CCL may not be required).

Soil-bentonite liners can often be compacted with rubber-tired or smooth-drum rollers. Soil-bentonite mixtures do not develop clods, and densification of the soil is often the primary objective with soil-bentonite liners. However, rollers with fully-penetrating feet may be effective in bonding soil-bentonite lifts.

After compaction of a lift, the soil should be protected from desiccation and freezing. Desiccation can be minimized in several ways: the lift can be temporarily covered with a sheet of plastic (but one should be careful that the plastic does not heat excessively which can lead to drying of the clay), the surface can be smooth-rolled to form a relatively impermeable layer at the surface, or the soil can be periodically moistened. For temporary protection against freezing, the CCL lift can be covered with a layer of clayey soil. Protection of a completed CCL was discussed in Section 2.5.3.2.

2.5.5 Maintenance

Maintenance is discussed in Chapter 9. Since the hydraulic barrier is overlain by the surface, protection, and drainage layers, hydraulic barrier maintenance is generally not needed unless the cover soils and drainage layer are breached due to erosion or there are problems with slope instability.

2.5.6 Monitoring

Monitoring is discussed in Chapter 8. If the cover system water balance is being assessed, the moisture content or matric potential at the top and bottom of the hydraulic barrier may be monitored. Percolation through the hydraulic barrier may also be monitored.

2.6 Gas Collection Layer

A gas collection layer may be necessary beneath a cover system hydraulic barrier if the underlying wastes generate gases or emit volatile constituents. The primary function of the gas collection layer is to convey gas to some outlet (e.g., passive gas vents, active gas wells). Collection of gases beneath a barrier can enhance cover system slope stability (see Section 6.2.2.2 and 7.7) and reduce the potential for gas emissions and lateral migration.

2.6.1 General Issues

For wastes that generate gases or emit volatiles, some type of gas management system is required. Passive systems that rely on periodic gas vents typically require a gas collection layer to prevent the buildup of gas pressures in the waste and beneath the hydraulic barrier. Depending on gas generation rates, extraction well spacing, the presence or absence of horizontal gas trenches, the air permeability of the waste, and other factors, a gas collection layer may or may not be needed when using active gas extraction systems. However, a continuous gas collection layer tapped periodically by relatively shallow vent pipes is the recommended approach for many situations.

For MSW landfills, which may generate significant quantities of gas, control of gas beneath cover systems with a GM, GCL, or composite barrier is especially important. If gas is not properly managed, the gas may migrate through the subsurface (as opposed to venting to the atmosphere), causing potential safety hazards in enclosed areas, on adjacent properties, etc. Subsurface gas migration may also lead to adverse groundwater quality impacts due to diffusion of volatile constituents from the gas phase to groundwater. Moreover, uncontrolled gas buildup beneath a GM, GCL, or composite barrier will produce uplift pressure that will either cause GM bubbles (or “whales”) to occur, displacing the cover soil and appearing at the surface (Figure 7-23), or cause a decrease in the normal stress between the GM or GCL and the underlying material. The whales can cause excessive deformations in the cover system components. The authors are aware of at several cases where an HDPE GM was deformed past its yield strain when a whale developed. At several facilities, the latter effect (i.e., decrease in normal stress) led to slippage of the GM and overlying cover materials creating high tensile stresses evidenced by compression ridges in the cover soil and folding of the GM at the slope toe and tension cracks in the cover soil near the slope crest. One example of a cover system stability problem caused by gas pressures is described in Section 7.7. Briefly, gas generated in a MSW landfill uplifted the GM barrier of a cover system and resulted in the GM and overlying materials moving downslope over a GT. Though the landfill had vertical gas extraction wells, the upper portion of the wells was not perforated. As a consequence, gas accumulated beneath the cover system, generating uplift pressures on the underside of the GM.

Gas collection layers should be designed to provide free-flow of gas to outlets. Methods for calculating the maximum flow rate in a gas collection layer are presented in Section 5.3. The allowable flow rate of a gas collection layer can be calculated as described in Section 2.6.2.3. Outlet design is discussed in Section 2.6.2.4.

The need for a soil or GT filter between the gas collection layer and overlying hydraulic barrier should be evaluated. For example, a GT is often used between a CCL and a granular or GN gas collection layer to prevent CCL material from being pushed into the gas collection layer during construction and retain the CCL particles should percolation occur. In this application, the GT is serving as a separator and a filter. A GT filter may also be required between a GCL and a gas collection layer to prevent downward extrusion of hydrated bentonite. The design of soil and GT filters is presented in Section 4.7.

2.6.2 Elements of Design

Important questions that typically need to be addressed when considering the design of the gas collection layer include:

- What materials are available to construct the gas collection layer?
- What thickness of gas collection layer material is needed?
- What is the maximum design flow rate and the allowable flow rate in the drainage layer?
- How should gas collection layer transitions and outlets be designed?
- How should the gas collection layer be constructed?
- What type and frequency of maintenance should be employed?
- What type and frequency of monitoring should be employed?

2.6.2.1 Materials

Like drainage layers (see Section 2.4.2.1), gas collection layers may be constructed of granular materials or geosynthetics. The material used should have adequate gas conductivity to minimize the build up of gas pressures beneath the barrier and adequate gas transmissivity to convey the design gas flow rate.

2.6.2.1.1 Granular Materials

Granular gas collection materials are normally composed of relatively clean sand or gravel. When a granular material is used, a separation or protection layer (typically a GT) may be needed between the granular material and the overlying barrier.

Specifications for granular materials often require:

- no more than 5% (dry-weight basis) of material passing the No. 200 sieve;
- a maximum particle size on the order of 25 to 50 mm;
- a GT cushion may be required between the GM and granular material to protect the GM from damage (e.g., deep scratches, puncture);
- restrictions on gradation, stated in terms of allowable percentages for specified sieve sizes (these restrictions may exist for various purposes);

- restrictions on the angularity of the material, if the material will interface with geosynthetics, which are vulnerable to puncture by large, sharp objects (or, alternatively, a GT cushion may be employed);
- that no deleterious material be present; and
- a minimum hydraulic or gas conductivity.

Gas conductivity of granular material is occasionally measured directly in the laboratory using techniques such as those described by Scanlon et al. (1999). However, more often it is estimated from the soil hydraulic conductivity as:

$$k_g = k\left(\frac{\rho_g \mu_w}{\rho_w \mu_g}\right) \qquad \text{(Eq. 2.24)}$$

where: k_g = gas conductivity (m/s); k = hydraulic conductivity (m/s); ρ_g = gas density (kg/m^3); ρ_w = water density (kg/m^3); μ_g = gas viscosity (kg/m/s); and μ_w = water viscosity (kg/m/s). Laboratory hydraulic conductivity testing of granular materials is discussed in Section 2.4.2.1.1. Gas conductivities are typically 20 times less than hydraulic conductivities because gas density is approximately three orders of magnitude less than water density and gas viscosity is approximately 50 times less than water viscosity. Because the gas permeability of a material decreases as its pore space becomes filled with water, gas collection layers should be designed to remain relatively dry and should be installed in a relatively dry state.

2.6.2.1.2 Geosynthetics

A range of geosynthetics, such as those described in Section 2.4.2.1.2, can be used for the gas collection layer. Like granular gas collection layers, a geosynthetic gas collection layer should meet filter criteria with the overlying hydraulic barrier. Furthermore, if a GM hydraulic barrier overlies a GN or core gas collection layer, a GT may be required between the collection layer and GM to provide higher interface friction on steep sideslopes and, possibly, reduce deformation-related intrusion of the GM into the collection layer and/or protect the GM from puncture or other damage by the collection layer.

Specifications for geosynthetic gas collection layers often require:

- resin and additive requirements;
- minimum thickness;
- minimum mass per unit area;
- specified density;
- minimum air transmissivity at a specified normal stress and gradient;
- minimum strength requirements to survive installation;
- if the gas collection material is a GN or core, inclusion of a GT above the material, if necessary, to increase interface friction, reduce deformation-related intrusion of an

overlying hydraulic barrier into the material and/or protect the hydraulic barrier from puncture or other damage by the drain; and

- if the gas collection material is a GN or core, inclusion of a GT filter below the material.

Gas transmissivity of geosynthetics is occasionally measured directly in the laboratory (e.g., Koerner (1997) presents data for needlepunched nonwoven GTs), but is more often estimated from the geosynthetic hydraulic transmissivity using Eq. 2.23 with the gas transmissivity, θ_g ($m^3/s/m$), substituted for k_g and the hydraulic transmissivity, θ_h ($m^3/s/m$), substituted for k.

Because the gas transmissivity of a material decreases as its pore space becomes filled with water, gas collection layers should be designed to remain relatively dry and should be installed in a relatively dry state.

2.6.2.2 Thickness of Granular Layers

The recommended minimum thickness of a granular gas collection layer is usually 0.3 m. This allows sufficient thickness for ease of construction. With extremely careful control of thickness, it is possible to construct even thinner granular gas collection layers (down to a thickness of about 0.15 m), but granular gas collection layers thinner than 0.3 m are not very common.

2.6.2.3 Required Flow Capacity

Similar to a drainage layer, a gas collection layer, either granular material or geosynthetic, can be designed using Eq. 2.21. Methods for calculating the maximum flow rate are presented in Section 5.3. FS values should be selected considering the uncertainties in the various design variables and the consequences of failure.

For all types of gas collection layer materials, the required hydraulic properties are evaluated considering the material properties measured in the laboratory and reduction factors that consider the potential for long-term clogging, deformation, etc. Eqs. 2.22 and 2.23 for drainage layer materials can be used with Eq. 2.24 for this purpose.

2.6.3 Gas Collection Layer Outlets

As previously discussed, gas or vapors collected in the gas collection layer should be conveyed to an outlet, which is typically a vertical riser pipe or vent. Since each outlet requires penetration of the hydraulic barrier, the number of outlets should be limited. Ideally, outlets should be located at high points within the cover system, although this is not always possible. Connections between gas outlets and the hydraulic barrier should be carefully designed to prevent water infiltration through and around the gas outlets and to accommodate differential settlements between the outlets and the barrier. The authors are aware of connections that were damaged due to differential settlement. For example, as described in Section 7.5, cover system GM boots around the gas well penetrations at a MSW landfill were not designed to accommodate settlement of the waste, which would cause downward displacement of the GM barrier relative to the wells. Within about one year after construction, 0.3 to 0.9 of differential settlement had occurred and the GM boots had torn from the GM barrier. The problem was resolved by replacing the gas extraction well boots with new expandable boots that could elongate up to 0.3 m and could also be periodically moved down the well.

2.6.4 Construction

The construction, QC, and CQA of granular gas collection layers and the manufacture, installation, QC, and CQA of geosynthetic gas collection layers are discussed in detail by Daniel and Koerner (1993, 1995).

In brief, granular material is usually loosely dumped from a truck and spread with a low-ground pressure bulldozer. Low-ground pressure equipment is used to minimize the generation of fines. Granular gas collection layers are generally not compacted.

Geosynthetic drainage layers are manufactured in panels of certain widths and lengths. The panels are placed in the field and connected by overlapping, seaming, tying, interlocking, or other means.

When a gas collection layer is overlain by a CCL, the first lift of the CCL is sometimes compacted with a thicker lift thickness so that the feet of the compactor don't penetrate though the CCL and damage the underlying materials. Alternatively, the first lift of the CCL is sometimes compacted to its specified maximum thickness with compactors having shorter feet, rubber-tired equipment, or other equipment. This first lift is generally required to meet compaction criteria, but may not be required to meet a permeability criterion (i.e., laboratory or field permeability testing of the first lift of CCL may not be required).

2.6.5 Maintenance

Maintenance is discussed in Chapter 9. Since the gas collection layer is overlain by the surface, protection, and drainage layers and the hydraulic barrier, gas collection layer maintenance is generally not needed unless there are problems with slope instability.

2.6.6 Monitoring

Monitoring is discussed in Chapter 8. Depending on the design of the gas collection system, the flow rates and chemistry of gas removed from the gas collection layer may be monitored.

2.7 Foundation Layer

The foundation layer is the lowermost component of the cover system. The primary functions of the foundation layer are to provide grade control for cover system construction, adequate bearing capacity for overlying layers, a firm subgrade for compaction of overlying layers, and a smooth surface for installation of any overlying geosynthetics. In some applications, the foundation layer may be designed to attenuate the potential effects of waste differential settlements on the cover system components (e.g., the foundation layer may be required to have a certain thickness). If the foundation layer material is granular, the layer may also serve as a gas collection layer.

2.7.1 General Issues

Waste receives its final mechanical compactive effort during placement of the foundation layer. To minimize post-construction settlement, and especially differential settlement, of the cover system, the foundation layer should be heavily proofrolled with large compactors. However, even a large compactor will not compact waste below a depth of about 1 to 2 m.

To compact the waste to greater depths, as may be required when warehouses or other structures are constructed on a cover system, the foundation subgrade may be proofrolled before the foundation layer is placed or preload fill or deep dynamic compaction may be used. A detailed description of the dynamic compaction method is presented by Mayne et al. (1984). With deep dynamic compaction, a large weight (usually a concrete block) is dropped from a height of many meters transmitting high energy to the ground surface. The impact of the weight compacts the underlying materials and collapses voids, causing deformation in both vertical and horizontal directions. Dynamic compaction is carried out in several passes, with the weight dropped in a predetermined grid pattern during each pass. The resulting craters are eventually filled with soil and the surface is proofrolled.

The depth of influence of the technique depends on the physical and dynamic properties of the material to be compacted, the location of the groundwater table, and other factors. As a general rule, the depth of influence for soils (not necessarily solid waste) can be estimated from the following empirical equation:

$$D_i = \alpha \, (W\,H)^{0.5} \qquad \text{(Eq. 2.25)}$$

where: α = empirical constant between 0.3 to 1 $(\text{m/tonne})^{0.5}$, with the specific value depending on soil grain size distribution and degree of saturation; D_i = depth of influence (m); W = mass of the falling weight (tonne); and H = height of the falling weight (m). It has been estimated that for soil densification, the densification is substantial down to a depth equal to about $D_i/2$ (Mayne et al., 1984), beyond which it decreases.

2.7.2 Elements of Design

Important questions that typically need to be addressed when considering the design of the foundation layer include:

- What materials are available to construct the foundation layer?
- What thickness of foundation layer material is needed?
- How should the foundation layer be constructed?
- What type and frequency of maintenance should be employed?
- What type and frequency of monitoring should be employed?

2.7.2.1 Materials

Materials most often used for the foundation layer include on-site or locally available soils. For landfills, daily or intermediate cover soil already in place is sometimes used for all or a portion of the foundation layer. In a few situations, waste material can be used to construct the foundation layer. If constructed of granular material, the foundation layer may also serve as a gas collection layer.

2.7.2.2 Thickness

The thickness of the foundation layer is selected based on site-specific criteria. The minimum thickness of a foundation layer is usually 0.3 m. When the foundation layer is designed to attenuate the waste differential settlements, it may be several meters to more thick.

2.7.3 Construction

The foundation layer may be placed and compacted using procedures for structural fill or may have no specific compaction criteria. At a minimum, the foundation layer is generally heavily proofrolled with large compactors, as described in Section 2.7.1. As many load repetitions as practical may be used so that stresses are felt as deeply as possible in the waste mass.

2.7.4 Maintenance

Maintenance is discussed in Chapter 9. Since the foundation layer is overlain by the other cover system components, foundation layer maintenance is generally not needed unless there are problems with slope instability.

2.7.5 Monitoring

Monitoring is discussed in Chapter 8. If the cover system water balance is being assessed, the foundation layer moisture content or matric potential may be monitored. Percolation through the foundation layer may also be monitored.

2.8 Examples of Cover Systems for Different Applications

Cover systems can be constructed with a wide variety of configurations of soil and geosynthetic layers to satisfy project-specific design criteria. A few examples used on specific projects are presented below. Additional examples of cover system cross sections can be found in Koerner and Daniel (1997).

Figure 2-17 illustrates two different hydraulic-barrier type of covers systems for a MSW landfill, one with a CCL hydraulic barrier and the other with a GM/CCL composite hydraulic barrier. For either example, a GCL can be considered as an alternate to the CCL. The choice of the underlying soil material, CCL or GCL, is controlled primarily by the how these materials respond to the anticipated differential settlements, wet-dry cycles, freeze-thaw cycles, and shear stresses and economics. The mechanical and hydraulic properties of CCLs and GCLs were discussed previously in Section 2.5. Soil thicknesses for this type of cover system will vary based on project-specific conditions.

Figure 2-18 presents an ET-barrier type of cover system for a MSW landfill in an arid setting. Design of the ET-barrier type of cover system is discussed in Section 3.2. Cover systems constructed at arid sites often require surface layers that are more resistant to erosion than vegetated topsoil. As discussed in Section 2.2.2.2, gravel-soil mixtures, gravel veneers, riprap, and other materials may be used as surface layer material for this purpose. MSW landfills constructed in arid environments may need a gas collection layer beneath the ET barrier depending on the gas generation rates in the landfill and the efficiency of any gas collection system. Soil thicknesses will vary based on project-specific conditions.

Figure 2-19 presents the cover system for a low-level radioactive waste landfill with a minimum design life of 200 years. The cover system for a low-level radioactive waste disposal facility is typically designed with a higher level of protection than cover systems for MSW and hazardous waste landfills. For the cover system in Figure 2-19, the protection layer includes a thick biointrusion layer to minimize the potential for exposure of animals and plants to waste. It also incorporates a GM/GCL/CCL composite hydraulic barrier. As for cover systems over MSW and HW landfills, soil thicknesses will vary based on project-specific conditions.

Figure 2-20 shows the lightweight cover system used as part of the remediation of an uncontrolled dumpsite containing HW. The site is in a marsh. The low bearing capacity of the foundation soil and waste at the site necessitate the use of this type of cover system. As described in Section 6.6, if the waste to be covered is a quasi-liquid (e.g., a sludge), the design of the cover system is often different. In such cases, the waste strength is increased (by physical solidification, dewatering, or other means), the cover system is reinforced, and/or a lightweight cover soil that includes a GM or a GCL is used. Geotechnical design consideration for cover systems on soft waste materials are discussed in more detail in Section 6.6.

Figure 2-21 illustrates "floating covers" for liquid or sludge waste impoundments. While GM floating covers placed over impoundments are rarely considered "cover systems", they often remain in place for many years and, in effect, may be designed to function as cover systems. For this reason, liquid waste impoundment covers are mentioned here. Liquid wastes may be covered with a GM to reduce emissions of volatile waste constituents, meet personnel safety requirements, and satisfy aesthetic requirements. The dimensions of the GM are proportioned when the impoundment is empty, if there is any possibility that draining of the impoundment may occur. To keep the central portion of the cover quasi-stable, expanded polystyrene (EPS) floats may be attached to the underside of the GM in a pattern that creates a stiffened central portion (Gerber, 1984). The slack is accumulated on the sides of the impoundment where it is accommodated by an arrangement of parallel floats with a sand tube welded to the upper side of the GM (Figure 2-21(a)). When the trough that is created by the floats and sand tube fills with rainwater, the water can be pumped from the GM surface. An alternative to this type of slack accommodating system is the tensioned-membrane approach illustrated in Figure 2-21(b). Here the GM is configured with tensioned lines such that weights in adjacent steel stanchion posts move up or down as the liquid level falls or rises. For the cases illustrated in Figure 2-21, wind loads can induce significant stresses, and GM edge and connection stresses are very high. Because of this, Koerner (1998) recommends that GM covers meet the minimum strength values given in Table 2-7 for a very high degree of installation survivability. Furthermore, since the GMs are continuously exposed to the environment, they require excellent resistance to ultraviolet degradation. Favored in view of these two requirements are fPP-R, CSPE-R, and ethylene interpolymer alloy-reinforced (EIA-R) GMs.

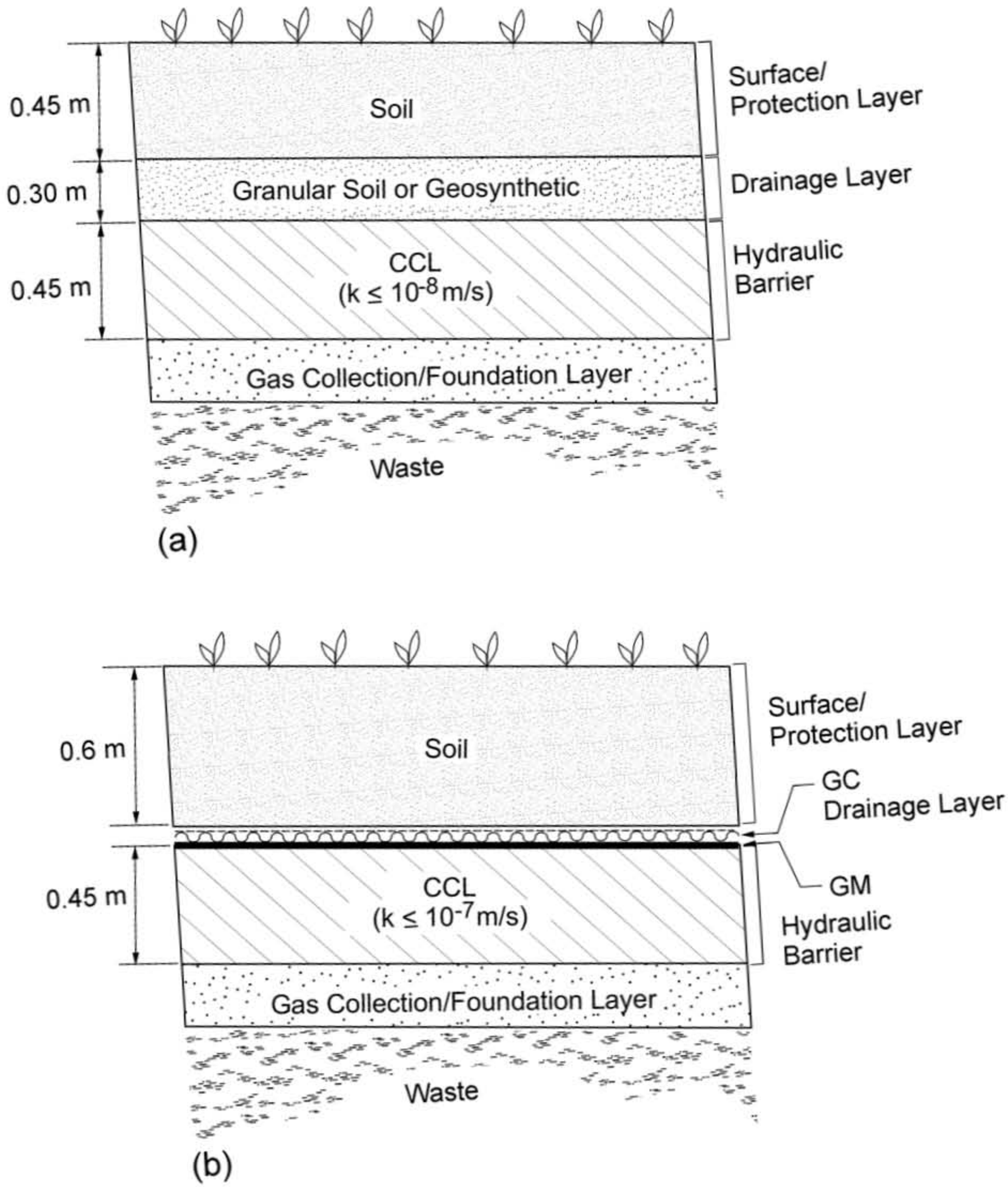


Figure 2-17. Examples of Hydraulic Barrier-Type of Cover Systems for MSW Landfills: (a) Cover System with CCL Hydraulic Barrier; (b) Cover System with GM/GCL Composite Hydraulic Barrier.

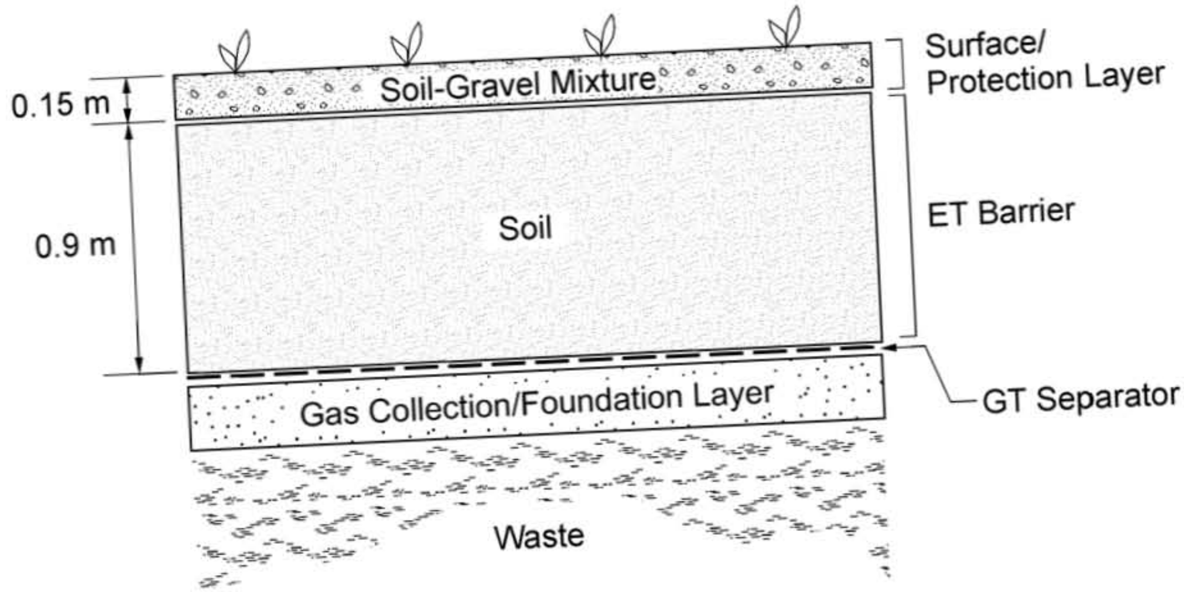


Figure 2-18. Example of ET Barrier-Type of Cover System for MSW Landfills.

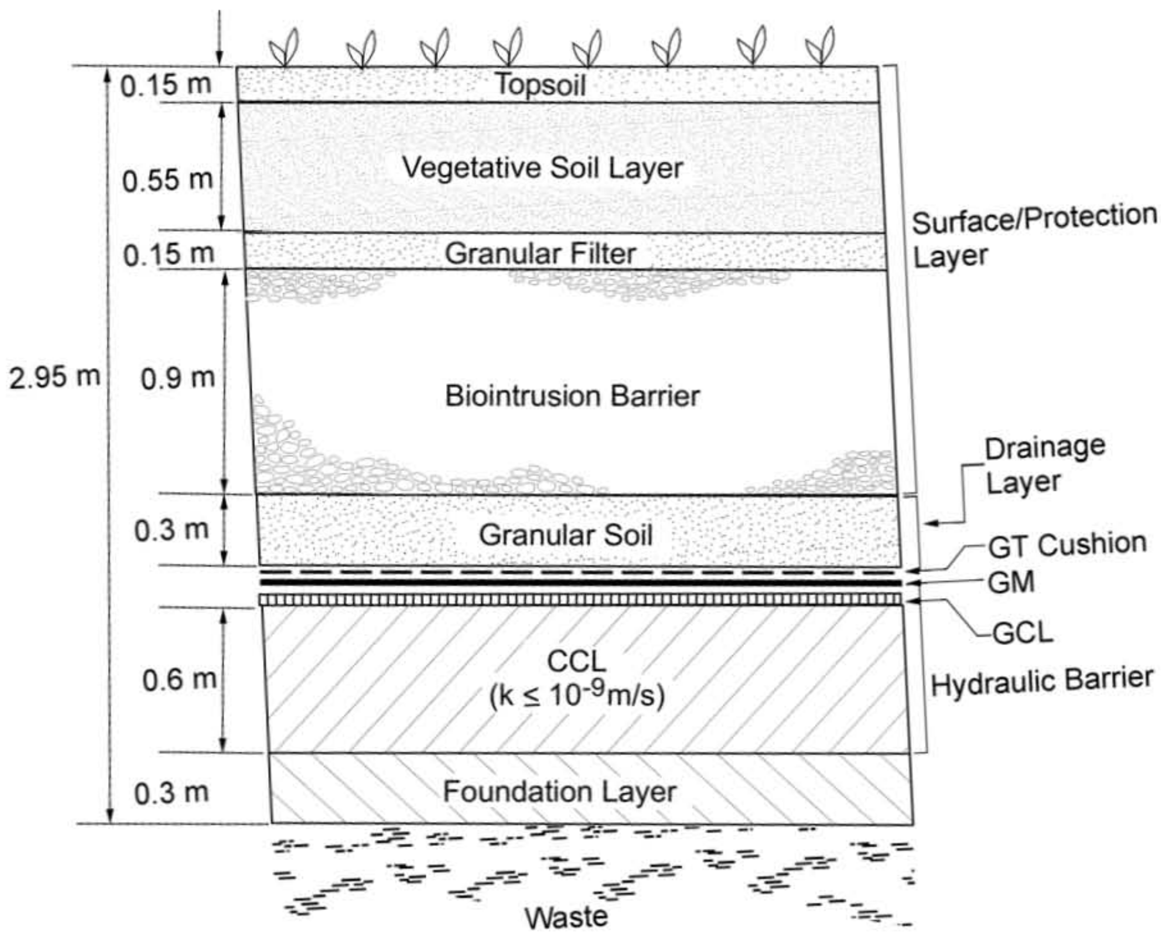


Figure 2-19. Example of Cover System for a Low-Level Radioactive Waste Landfill.

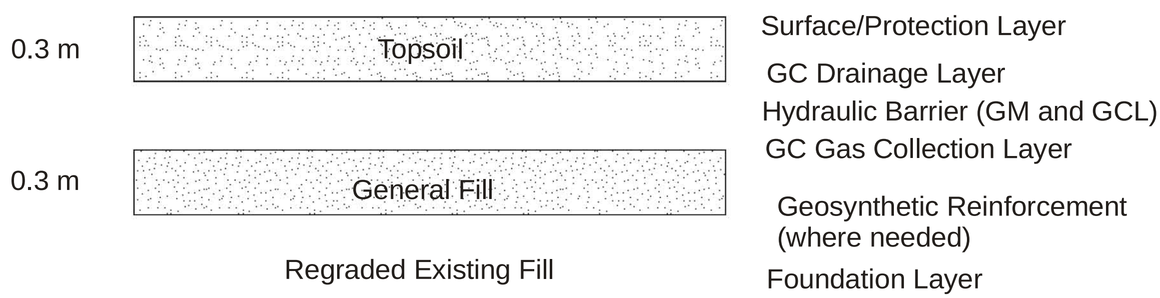


Figure 2-20. Example of Lightweight Cover System for a HW Remediation Site.

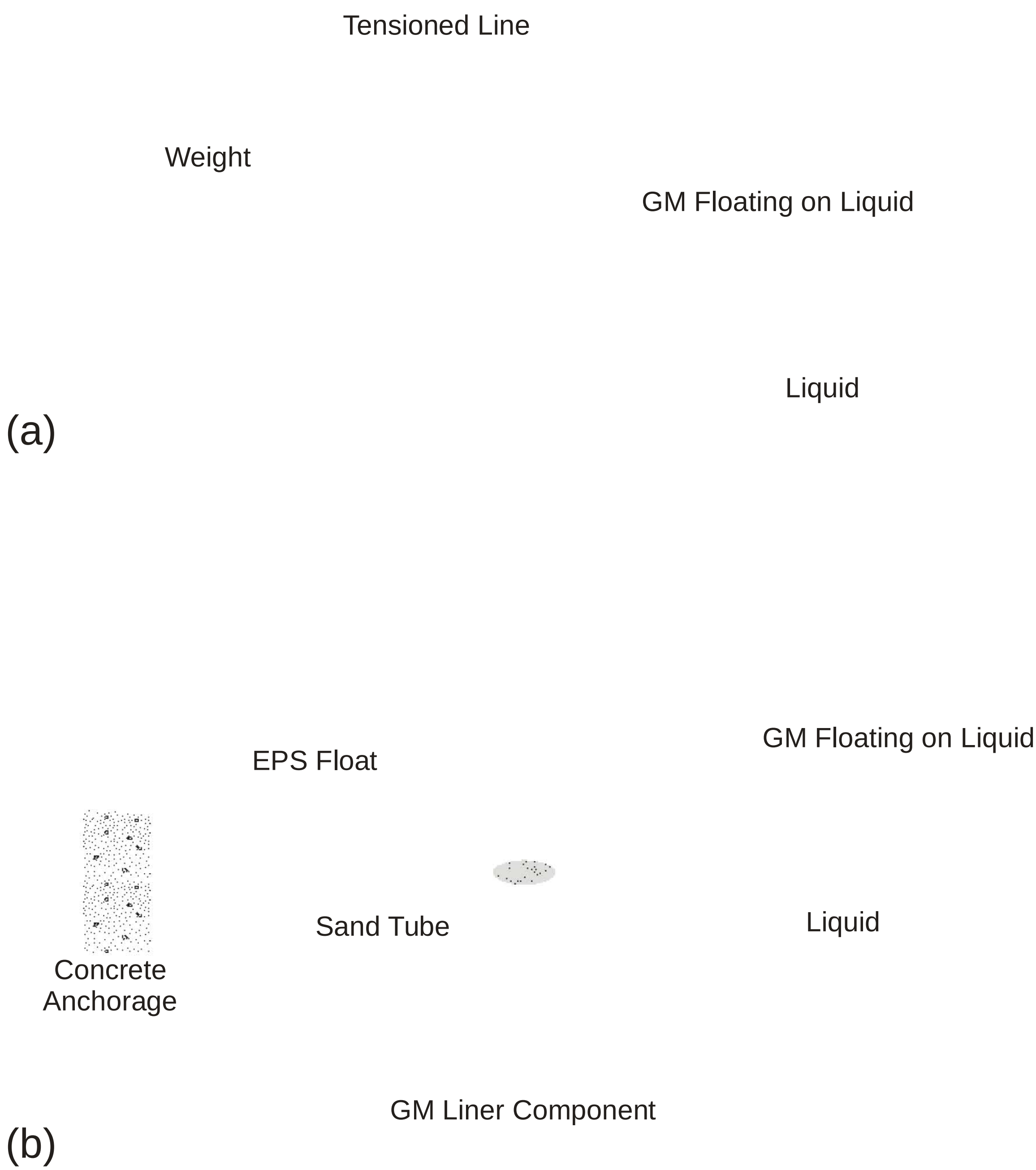


Figure 2-21. Examples of Floating "Cover System" for HW Impoundments: (a) GM with Tensioned Lines; and (b) GM with Floats and Sand Tubes.

Chapter 3
Alternative Design Concepts and Materials

3.1 Introduction

As previously mentioned in Section 1.3, RCRA and CERCLA regulatory requirements provide flexibility for innovation and alternatives in cover system design. The regulatory mechanism for approval of an alternative design or material typically includes a demonstration of technical equivalence. The alternative must perform in a manner that is equivalent or superior to the design or material it replaces. Depending on the function of the proposed cover system alternative, the demonstration of technical equivalence may include an evaluation of water percolation through the cover system, gas emission rate, erosion potential, and/or long-term performance (e.g., ability to accommodate foundation settlements, service life). Some of the alternative design concepts and materials discussed in this chapter have met this equivalency criterion on a project-specific basis and have been employed in cover systems for a limited number of landfills and contamination source areas.

The two alternative cover system design concepts discussed in this chapter (with a performance goal of preventing precipitation from percolating through the cover system) are based on either: (i) the evapotranspiration (ET) barrier principle; or (ii) the capillary barrier principle. Cover systems with an ET or capillary barrier are generally best suited for semi-arid and arid climates with minimal snowpack, and capitalize on the naturally occurring low precipitation rates and high potential evapotranspiration (PET) rates in these climates. Arid sites generally receive less than 250 mm of annual rainfall with evaporation exceeding rainfall and sparse vegetation, and semi-arid sites have a mean annual precipitation between 250 and 500 mm and are typically vegetated with grasses (Lincoln et al., 1982). The extent of arid and semi-arid lands in the U.S. is shown in Figure 3-1. In wetter climates, these alternative cover system design concepts are generally not as effective as designs with hydraulic barriers since the fine-grained soil layers used to store infiltrating water in the alternative designs would have to be relatively thick to provide adequate water storage capacity, and water migrating into the lower regions of these soil layers may not be easily removed by ET. The alternative design concepts differ from designs with hydraulic barriers alone in that they are intended to emphasize the following:

- unsaturated hydraulic conductivities of the soil components;
- low hydraulic conductivity of fine-grained soil layer(s), even at high degrees of soil saturation;
- relatively high water storage capacity of fine-grained soil layer(s) with eventual removal of stored water primarily by ET;
- increased transpiration through the use of diverse native vegetative; and
- ease of construction and/or substantial cost savings through the use of locally-available materials.

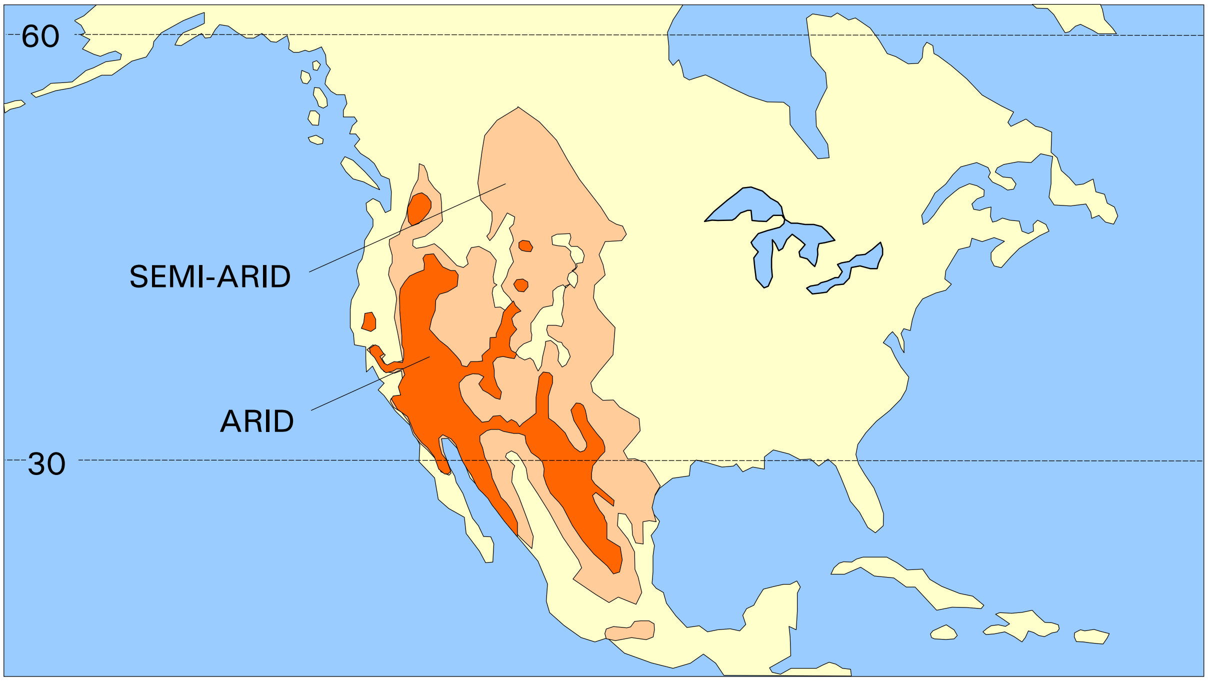


Figure 3-1. Semi-Arid and Arid Areas in the U.S. (modified from Meigs, 1953).

Because the soil layers in the alternative designs are relatively dry, they often have moderate to high gas permeabilities and, therefore, may not provide an effective barrier to gases, if any, generated within the landfill or contamination source area. It is important that the potential for gas generation and the need to collect and manage gases be considered when developing an alternative cover system design. If gas generation may occur, the collection, transmission, and, potentially, treatment of these gases should be considered. If the facility is a MSW landfill subject to EPA's gas collection and treatment regulations or if gas emissions through the cover system are a concern, the facility should incorporate appropriate gas containment components. The effect of seasonal freezing of near surface soils on lateral and downward gas migration also needs to be addressed.

In some areas in the southwest, regulatory agencies are promoting the use of alternative cover system designs to EPA performance criteria and guidance for MSW landfills. There is a concern that the CCL component of a GM/CCL composite barrier in a cover system may desiccate and crack over time, especially in semi-arid and arid climates (EPA, 1989; EPA, 1991; Suter et al., 1993), providing little value to the cover system. As an example, in southern California, regulators are currently allowing use of cover systems with ET barriers to close MSW landfills constructed without a Subtitle D liner system. The cross section of an ET barrier cover system constructed at such a landfill is shown in Figure 3-2.

The design of ET and capillary barriers is discussed in more detail below. Additional design and construction considerations for these cover systems are presented in "*Technical and Regulatory Guidance for Design, Installation, and Monitoring of Alternative Final Covers"* (ITRC, 2003), and in "*Evapotranspiration Landfill Cover Systems Fact Sheet*" (EPA, 2003). These designs should be carefully reviewed by a person knowledgeable and experienced in unsaturated soil moisture modeling and the design of such cover systems. Because there are uncertainties in the design assumptions and methods and field performance data for alternative cover system designs are limited, EPA is presenting a conservative design approach herein. Furthermore, EPA recommends that field monitoring of these cover systems be conducted to verify that the design assumptions and methods are appropriate. With these data, design procedures may be refined for a given geographic area. This is already occurring in southern California, where a more unified approach to the modeling and field monitoring of ET barriers is evolving.

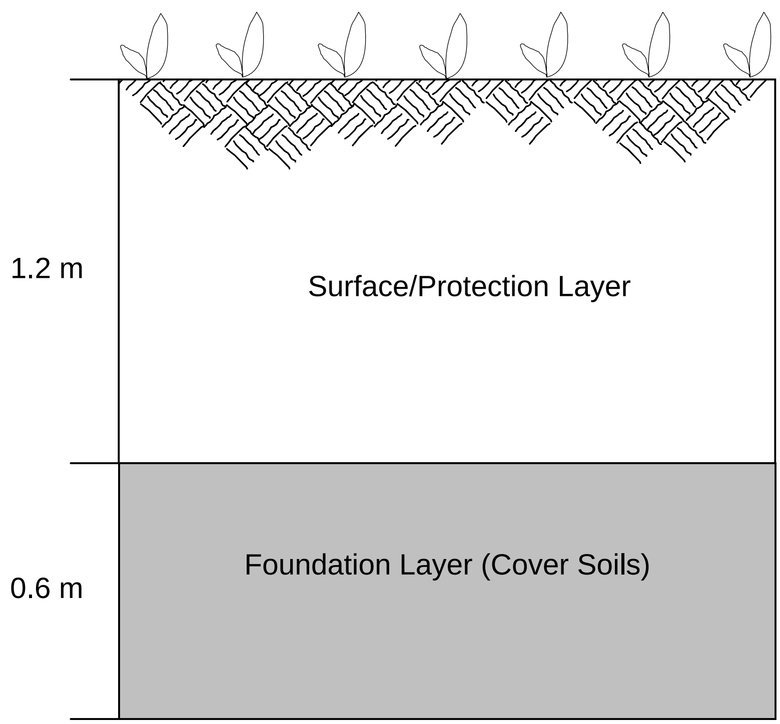


Figure 3-2. Cross Section of ET Cover System Used for a MSW Landfill in Southern California.

Chapter 3 also discusses emerging alternative materials that can be used in lieu of the various materials traditionally used in cover systems and described in Chapter 2. The considered alternative materials are geofoam, shredded tires, sprayed elastomers, and paper mill sludges.

3.2 ET Barrier Design

3.2.1 Overview

As discussed in Section 1.1.2 and illustrated in Figure 1-4, ET barriers consist of a thick layer of relatively fine-grained soil. The barrier may be overlain by a topsoil layer or surface treatment to promote vegetative growth and reduce the potential for erosion by water or wind. Soil types used for construction of ET barriers include fine-grained soils such as silty sands, silts, and clayey silts. In general, the greater the percentage of fines in a soil, the greater the water storage capacity and thus the thinner the barrier required to store a given amount of water. As discussed in Section 2.3.2.2.3, soils with a large fraction of clay are typically not used due to the potential for desiccation cracking of the clay. Cracks provide preferential pathways for infiltrating water to bypass the clay matrix and thereby bypass storage. In addition, there is somewhat less available water for plants in clays than in silty soils (Figure 2-11).

Previous research has shown that a simple ET barrier can be effective at limiting percolation and erosion, particularly in dry environments (Nyhan et al., 1990; Hauser et al., 1994; Nyhan et al., 1997; Dwyer, 1998; Dwyer, 2001). The thickness of the barrier is selected, based on the barrier soil's water storage capacity (Eq. 2.5) to retain infiltrating water until it can be removed by ET. Saturated flow in the near surface, when it does occur, is primarily downward as the hydraulic gradient is largely due to gravitational potential differences. Water movement deeper in the soil profile generally occurs under an unsaturated condition. Under this condition, the hydraulic gradient is comprised of a gravitational potential component (acting downward) and a matric potential component (which can act either upward or downward) (see Eq. 4.11). Matric potential gradients can be many orders of magnitude greater than the gravitational potential gradient. Water flows in response to the total potential gradient. Since the total potential gradient is the sum of the matric potential gradient, gravitational potential gradient, and other gradient components (e.g., solute potential gradient) which are generally less significant and are not considered in this guidance document, both upward and downward water movement is possible in the unsaturated soil of an ET barrier.

As previously mentioned, ET provides the mechanism to remove stored water from the ET barrier. Evaporation of water from the soil surface decreases the soil water content and, thus, matric potential in the upper portion of the barrier. This results in an upward matric potential gradient and upward flow. Plant transpiration also relies upon water potential gradients (matric and osmotic) to remove water from the ET barrier. Figure 3-3 shows a typical variation in water potential in the soil-plant-atmosphere system. In arid climates, the total water potential difference between soil moisture and atmospheric humidity can exceed 100 MPa (10,000 m of water) (Hillel, 1998). The largest portion of this overall potential difference occurs between the leaves and the atmosphere. The larger the soil-plant-atmospheric potential gradient, the more effective is the ET barrier. For this reason, well-vegetated ET barriers can be very effective in semi-arid and arid regions. These regions are characterized by large potential evapotranspiration (PET) compared to precipitation.

PET is an index that essentially represents the atmospheric "demand" for water. PET can be calculated using a form of Penman's equation (Penman, 1948). The total calculated PET for Tucson, Arizona from January 1987 through December 1999 was 25.71 m while the actual precipitation during this period was only 3.61 m (http://ag.arizona.edu/azmet/). This equates to a greater than 7:1 PET to precipitation ratio (i.e., there is a much greater demand for water by the atmosphere and plants than can be supplied to the soil by precipitation). A monthly comparison of PET versus precipitation for 1999 is shown graphically in Figure 3-4.

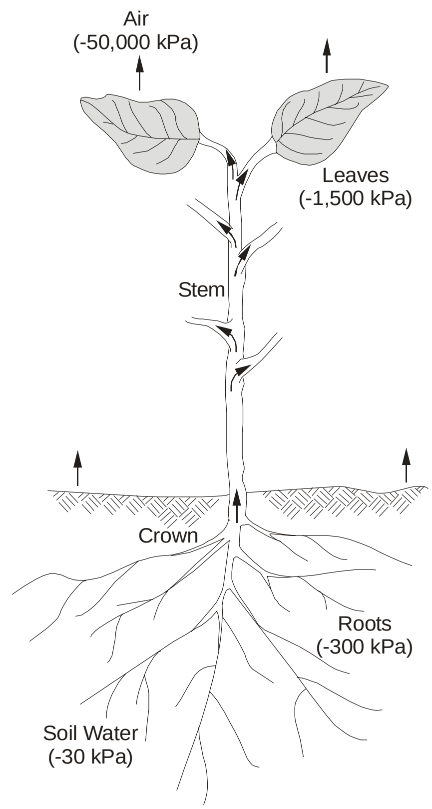


Figure 3-3. Typical Soil-Plant-Atmosphere Water Potential Variation (modified from Hillel, 1998).

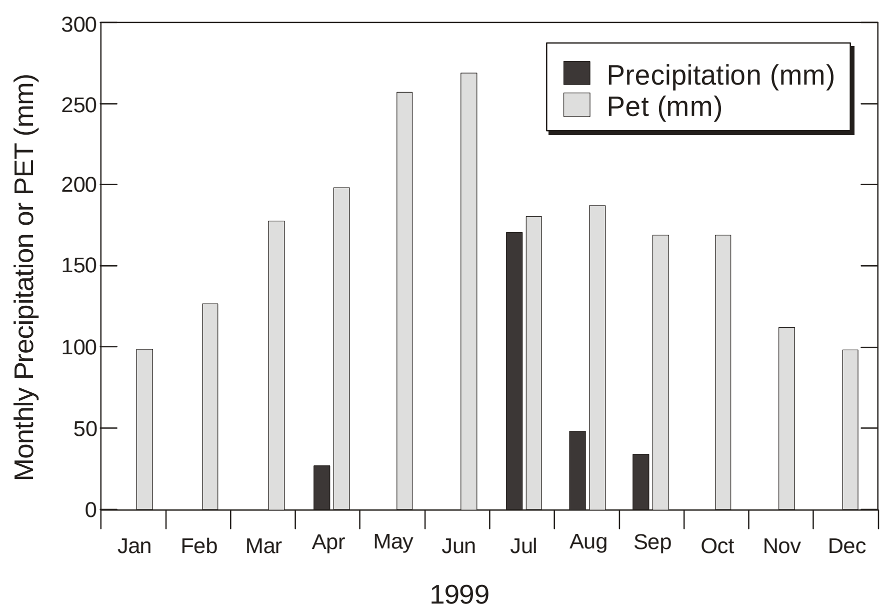


Figure 3-4. Monthly Precipitation and PET in 1999 for Tucson, Arizona.

3.2.2 General Issues

A number of the same general issues that were mentioned in Sections 2.2.1 and 2.3.1 for surface and protection layers, respectively, also apply to ET barriers. Important issues are water storage capacity and erosion potential, since excessive erosion can cause the cover to be ineffective.

3.2.3 Elements of Design

Important questions to be addressed when designing an ET barrier include the following:

- What materials should be used to construct the barrier?
- How thick should the barrier be to store the required amount of water?
- Are materials uniform and have appropriate placement methods been determined to minimize preferential pathways for percolation?
- What surface treatments should be applied to control erosion?
- Which plants should be established to promote transpiration and stabilize the cover surface?
- How and at what frequency should the barrier be maintained?
- What type and frequency of monitoring should be employed?

3.2.4 Design Concept

The ET barrier design concept can be summarized in the following steps:

1. Identify the critical infiltration event(s) that may result in percolation. This generally involves identifying the design precipitation event or series of events. Khire et al. (2000) recommend that the meteorological record for the site be reviewed to define critical time periods where PET less precipitation is near zero or negative. This condition should normally occur outside the growing season (Khire et al., 2000).
2. Calculate the depth of water that must be stored in the ET barrier based on the design infiltration event(s). For simplicity, it can be assumed that the barrier must hold all of the precipitation occurring during the critical infiltration event(s), i.e., there is no runoff or ET (Khire et al., 2000).
3. Characterize the unsaturated hydraulic properties of the considered fine-grained barrier soil and calculate its water storage capacity using Eq. 2.5.
4. Calculate the minimum soil thickness required for the fine-grained soil as described in Section 3.2.5.
5. Establish the vegetation (seed mix) to be used and any surface treatment (i.e., gravel veneer, gravel admixture, soil nutrient supplements) to be employed. Cover system vegetation is discussed in Sections 1.6.6 and 2.2.3. Surface treatments are described in Section 2.2.2.2.
6. Assess the need for optional layers (i.e., gas vent layer, biointrusion layer). Optional layers are described in Chapter 2.

7. Establish the adequacy of the design based on:

   - predictive computer modeling (Section 3.4.2),
   - field data to evaluate short-term performance (Section 3.4.3), and
   - natural analogs to predict long-term performance (Section 3.4.4).

3.2.5 Soil Thickness

An estimate of the required thickness of the ET barrier can be made based on the required depth of water to be stored in the soil and the water storage capacity of the soil. The design strategy for an ET barrier is to ensure that the storage capacity is sufficient to store the "worst-case" infiltration quantity resulting from the critical infiltration event(s), with an appropriate factor of safety, until the infiltration can be removed via ET.

As discussed in Section 2.3.2.2.7, the depth of water, H_w, that can be stored in a soil layer is the product of the water storage capacity, θ_{sc}, of the soil and the layer thickness, H_s. The storage capacity, in turn, is a function of the soil's field capacity and permanent wilting point. Representative values of θ_{sc} for different soil textures were presented in Table 2-6.

In dry environments, plants commonly reduce the water content of a near-surface soil to the permanent wilting point during every growing season (Anderson et al., 1993), making the soil's entire storage capacity available for subsequent precipitation when ET is low and plants are dormant. Thus, one potential scenario of the required amount of infiltration that an ET barrier has to store annually is the total precipitation input during the dormant period(s). Another scenario might be that created by spring snowmelt or summer thunderstorms. Both of these design scenarios should be considered.

ET-barrier type cover systems located in temperate climates have been vegetated with perennial, fast-growing, and deep-rooted hybrid poplar trees (Licht et al., 2001). Hybrid poplar trees have been used for phytoremediation and have been considered for cover system applications (i.e., phytocaps) because they exhibit relatively high water uptake rates (e.g., 810 to 1,070 mm/yr for tree plantations) and growth rates (e.g., 1 to 3 m/yr), develop deep root systems (2 to 3 m deep), are easily propagated, and can be planted economically. Two cover systems with ET cover systems vegetated with hybrid poplars are being monitored under the Alternative Cover Assessment Program (ACAP), which is discussed in Section 3.4.3.

Generally, there is a need to incorporate a factor of safety into the design of an alternative barrier to help offset some of the uncertainties associated with weather, in-place soil properties, and vegetation growth. Reasonable values for these parameters should be used and a factor of safety should be applied, at a minimum, to the required amount of water to be stored. Since there are few field performance data available for alternative cover systems, EPA believes that the minimum thickness of an ET barrier should be the larger of 1.25 H_s (i.e., a factor of safety of 1.25 applied to the calculated cover soil layer thickness) and 0.9 m. This factor of safety and minimum thickness not only account for uncertainities in precipitation, modeling, and material properties, but also allow for the possibility of long-term erosion of the surface soil. This level of conservatism may be reduced somewhat when the performance of the alternative barrier is modeled using an unsaturated flow code and site-specific parameters, if the cover system is monitored (see Chapter 8), or if a GM is used beneath the ET barrier. The latter case may apply when an ET/GM composite barrier is used in lieu of a GM/CCL composite barrier.

As an example, during 1987 to 1999 Tucson, Arizona received from about 5.1 to 236.0 mm of precipitation annually during December and January, when plants are typically dormant. The average precipitation during this time period was 58.2 mm. Dividing the worst-case precipitation value of 236.0 mm by a storage capacity of 0.15 for a silty loam soil yields a required ET barrier thickness of 1.7 m. Applying a factor of safety of 1.25 to this thickness yields a design thickness of 2.125 m. The above calculation method is simple, but conservative, and doesn't take into account runoff or evaporation. When the above scenario was simulated using an unsaturated flow model with historical weather data and assuming the silty loam soil was initially at its wilting point, the required barrier thickness to limit percolation to less than 0.5 mm/yr during the simulation period was calculated to be approximately 0.8 m. Applying a factor of safety of 1.25 to this thickness yields a design thickness of 1.0 m.

3.3 Capillary Barrier Design

3.3.1 Overview

As discussed in Section 1.1.2 and illustrated in Figure 1-5, capillary barriers consist of one or more layers of finer-grained soil overlying one or more layers of coarser-grained soil. Like the ET barrier, a capillary barrier may have a topsoil layer or surface treatment to promote vegetative growth and reduce the potential for erosion. The finer-grained soil in a capillary barrier has similar characteristics to the fine-grained soil used to construct an ET barrier: it is generally a silty soil, as described in Section 3.2.1. Soil types used for construction of the coarser-grained component range from coarse sand to cobbles.

The capillary barrier design concept relies on the differences in pore size distribution between the upper finer-grained soil and the lower coarser-grained soil to promote retention of water in the finer-grained soil under unsaturated flow conditions, as long as the contrast in unsaturated properties (e.g., soil-moisture characteristics and unsaturated hydraulic conductivities) of the two soils is sufficiently large. This can be explained as follows: at a given matric potential, a coarser-grained soil tends to have a much lower water content than a finer-grained soil. The hydraulic conductivities of unsaturated soils decrease exponentially with decreasing water content because flow paths through thin films of water coating the soil particles in dry soil are extremely tortuous. Thus, dry gravel is actually much less permeable to water than moist silty sand. If the soils remain unsaturated, the finer-grained soil tends to retain nearly all the soil water and the underlying layer serves as a barrier due to its dryness. The matric potential in the finer-grained soil layer typically must approach a value near zero (i.e., saturated conditions) before any appreciable flow occurs into the coarser-grained layer (Figure 1-5).

In contrast to ET barriers, which experience primarily vertical water flow, the primary direction of water flow (i.e., vertical or lateral) in capillary barriers depends on whether or not the capillary barrier is sloped. The water balance for non-sloped capillary barriers is similar to that for ET barriers. Thus, water is removed from the finer-grained soil component of a non-sloped capillary barrier by ET or percolation (breakthrough) into the coarser-grained soil layer. For sloping capillary barriers (most common scenario), lateral diversion of infiltrating water provides an additional means of removing soil water from the finer-grained soil layer. Lateral diversion is essentially gravity-driven unsaturated drainage within the finer-grained layer. Because the water content in the finer-grained layer is usually greatest near its interface with the underlying coarser-grained soil layer, and the hydraulic conductivity of an unsaturated soil increases with increasing water content, lateral diversion is concentrated near this interface. Laterally diverted water causes the water content in the finer-grained soil to increase in the downdip direction. The diversion length is the distance that water is diverted along the interface between the soil layers before there is appreciable breakthrough into the coarser-grained layer. To avoid significant breakthrough, the cover system slope length should be less than the diversion length (Figure 3-5). Therefore, if a capillary barrier is sloped, the two-dimensional (lateral and vertical) effects of soil-water movement must be taken into account in design of the barrier.

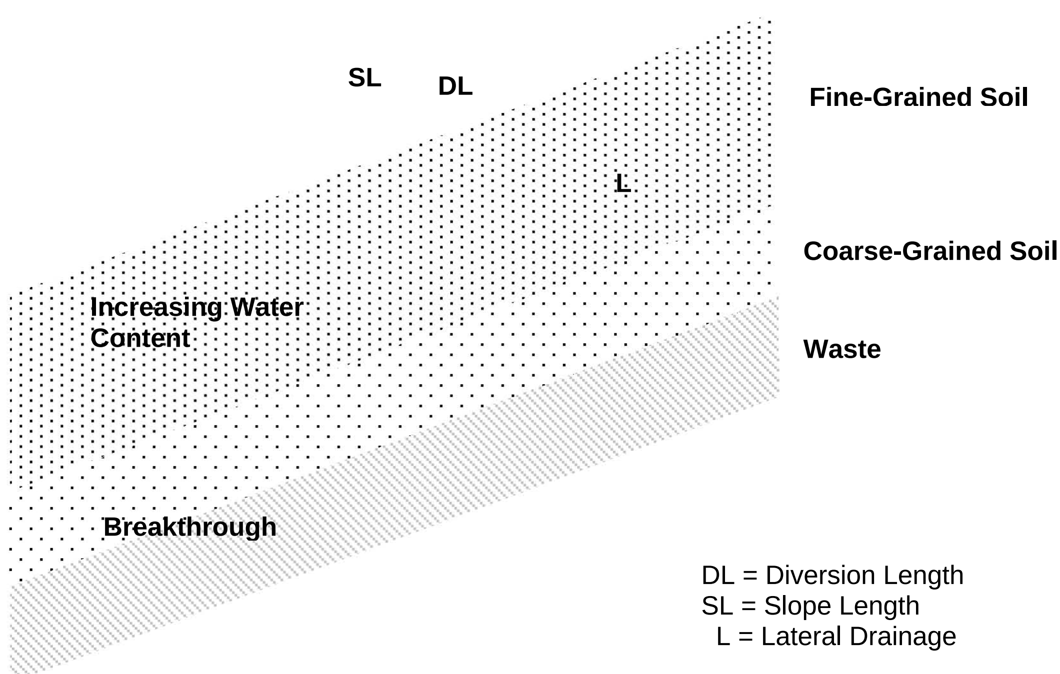


Figure 3-5. Problem Where Diversion Length is Less than Cover Slope Length on a Capillary Barrier.

Some advantages of incorporating a capillary barrier rather than an ET barrier alone in a cover system include:

- The finer-grained soil layer of a capillary barrier stores more water than a comparable layer without the capillary break (i.e., a free-draining layer). Compared to an ET barrier, the additional storage capacity either serves to reduce overall percolation, or reduce the total thickness required for the finer-grained soil to yield the same degree of percolation inhibition.
- The additional water stored within a capillary barrier tends to encourage the establishment and development of the surface vegetation. The increased vegetative cover, in turn, removes more soil water due to greater ET. Furthermore, plants serve an important function in reducing surface erosion.
- In addition to providing the capillary break, the coarser-grained layer of the capillary barrier can serve as a biointrusion barrier and/or possibly a gas collection layer if small amounts of gas are generated. (If gas emissions through the cover system are a concern, gas containment components should be incorporated into the cover system design.)

Potential disadvantages of a capillary barrier compared to an ET barrier include the need to specify and construct two different material types, the potential difficulties in constructing the interface between the different materials (to form the capillary break), and minimizing differential settlement.

3.3.2 General Issues

A number of the same general issues that were mentioned in Sections 2.2.1 and 2.3.1 for surface and protection layers, respectively, also apply to the capillary barrier. Important issues are water storage capacity and erosion potential, since excessive erosion can cause the cover to be ineffective. In addition, it is particularly important to construct smooth and unmixed interfaces between adjacent soil layers, as discussed in Section 3.5.2. Good CQA/CQC of these interfaces is essential.

Two issues specific to capillary barriers were described by Koerner and Daniel (1997) and are as follows: (i) the finer-grained soil must not be allowed to migrate over time into the underlying coarser-grained soil; and (ii) over periods of extremely high precipitation, the capillary barrier may cease to function, at least temporarily, as the coarser-grained soil becomes moist and more permeable than the finer-grained soil. The former issue is discussed in more detail in Section 3.3.6. The latter issue is addressed by incorporating an appropriate factor of safety in design, as discussed in Section 3.3.4.

3.3.3 Elements of Design

Important questions to be addressed when designing a capillary barrier include the following:

- How should the barrier be sloped?
- What materials should be used to construct the barrier?

- How thick should the different layers be to store the required amount of water, wick away infiltrating water, and create a capillary break?
- What surface treatments should be applied to control erosion?
- Which plants should be established to promote transpiration and stabilize the cover surface?
- How and at what frequency should the barrier be maintained?
- What type and frequency of monitoring should be employed?

3.3.4 Design Concept

The design concept for the finer-grained soil component of the capillary barrier is essentially the same as that presented for the ET barrier in Sections 3.2.4 and 3.2.5. The required minimum thickness, however, can be less for a non-sloped capillary barrier than for an ET barrier. In general, the capillary barrier increases the apparent field capacity of the finer-grained soil component, thereby increasing the water storage capacity of this component. Consequently, the finer-grained soil layer in a capillary barrier may not need to be as thick as the same layer used alone in an ET barrier. In fact, the non-sloped capillary barrier may be preferred if the finer-grained soil layer is required to be relatively thick. If this layer is too thick, all of the stored water may not be removed by subsequent ET.

The apparent field capacity, θ_{afc}, of the finer-grained soil component of a capillary barrier can be estimated using a measured or modeled water content at which drainage from the capillary barrier occurs (Stormont and Morris, 1998). This water content is greater than the soil's field capacity due to the effects of the capillary break and can be calculated as:

$$\theta_{afc} = 1/L\int_0^L \theta\left(z + h_z^*\right)dz \qquad \text{(Eq. 3.1)}$$

where: θ = volumetric water content; L = thickness of the finer-grained soil layer; z = distance above the finer-coarser interface; and h_z^* = minimum head at which flow into the coarser-grained layer first occurs.

The texture of the finer-grained soil is important in determining the additional water storage capacity achieved with a capillary barrier. Stormont (1996) described a field-scale (14 m^2 surface area) water balance experiment conducted to measure the water storage capacity of a capillary barrier. The barrier was comprised of a 900-mm thick layer of silty sand placed over uniform gravel (0.6 mm). The barrier was installed at a 10% grade. The water content in the finer layer, measured as added water, was increased at a constant rate of about 10 mm/day. Breakthrough into the coarser layer was detected by collecting water that drained from the coarser layer. The volumetric water content in the finer-grained layer at breakthrough was about 0.40 near its interface with the coarser-grained layer. Stormont (1996) estimated the total amount of water stored in the capillary barrier at breakthrough by integrating the measured water content over the thickness of the finer-grained layer. Expressed as a normalized quantity with respect to area (volume of water divided by surface area), the capillary barrier stored 285 mm of water at breakthrough, which corresponds to an average apparent field capacity of approximately 0.32. The storage capacity of the capillary barrier can be compared to that estimated for a simple ET barrier. Without the capillary break, water will drain approximately to the soil's field capacity. The field capacity for the same soil (silty sand) can be estimated at 0.19, based on the data for representative soils presented in Table 2-6. By integrating this water content over the same 900 mm thickness, the silty sand in an ET barrier configuration would be expected to store about 170 mm of water before drainage commenced. Thus, an additional 115 mm of water storage was gained by the capillary break for the same cover soil thickness. In other words, a simple ET barrier would need to be about 1510 mm thick to store the same amount of water as 900 mm of the same soil in the considered capillary barrier configuration.

The texture of the coarser-grained soil is also important in assessing the water storage capacity of a capillary barrier (Khire et al., 2000). For example, if the coarser soil becomes more broadly graded, h_z^* in Eqn. 3-1 will decrease and θ_{afc} will decrease. In contrast, if coarser soil becomes more uniformly graded or if the average particle size of the coarser soil is reduced, h_z^* will increase and θ_{afc} will increase.

The design of a sloped capillary barrier also includes the selection of the slope gradient and the distance between lateral drainage outlets to minimize the percolation of water through the coarser-grained soil. These parameters can be assessed using a two-dimensional or three-dimensional unsaturated flow computer model, such as HYDRUS-2D or VS2D-T. These models are briefly described in Chapter 4. In general, layer thickness, diversion length, and slope gradient requirements depend on climatological information for the specific site (e.g., precipitation, temperature, humidity) and the characteristics of the soils used in the cover (e.g., water storage capacity, hydraulic conductivity, texture). Other factors that should be taken into consideration include slope stability, vegetation characteristics, and potential for desiccation (Dwyer, 1997).

The lateral diversion capacity of the finer-grained layer is dependent in large part on the hydraulic conductivity of the layer. In general, the hydraulic conductivities of silts and loams are too low to permit appreciable lateral diversion. Field tests of capillary barriers with homogeneous finer-grained layers indicate that the effective diversion lengths are less than 10 m (Nyhan et al., 1990; Hakonson et al., 1994; Stormont, 1995; Stormont, 1996; Nyhan et al., 1997). These short diversion lengths are a consequence of the relatively low hydraulic conductivity of the finer-grained soils compared to the infiltration rate during stressful periods when the soil is relatively wet (e.g., spring snowmelt). Thus, soils that are often preferred as a rooting medium and for their water storage capacity (e.g., loams, silts) may not be conductive enough to substantially divert soil water laterally.

Utilizing "transport layers" or "unsaturated drainage layers" within the finer-grained layer (Stormont, 1995) that allow water to drain laterally and outlet (e.g., in a swale) can increase the diversion capacity of capillary barriers. Transport layers are one or more relatively conductive layer(s) that drain water laterally within the cover's finer soil layer while remaining unsaturated. Because soil water tends to accumulate near the interface between the finer and coarser layers and unsaturated hydraulic conductivity increases with water content, a transport layer near the interface is most effective in laterally diverting water. An effective transport layer, for example, could consist of a 300-mm thick relatively fine-grained, uniform sand that has a relatively high hydraulic conductivity under moderate to high matric potentials. The lateral diversion afforded by a transport layer complements the water storage function of the overlying soil, expanding the conditions and climate for which a capillary barrier could be effective.

3.3.5 Coarser-Grained Soil Layer

The primary function of the coarser layer is to form a capillary break, but it may also serve as a biointrusion barrier or, possibly, a gas collection layer.

Capillary break - The movement of water from the overlying finer-grained layer into the underlying coarser-grained layer is controlled by the water entry potential of the coarser-grained layer. The water entry potential is the potential associated with the movement of water into the smallest pores that form a continuous network. Water will not move from an overlying moist layer into an initially dry underlying layer at potentials less than the water entry potential suction of the underlying layer. Using a coarser-grained soil with a higher water entry potential delays the movement of water from the finer-grained soil layer into the coarser layer, permitting more water to be stored in the finer layer near the interface (Figure 1-5). The suction head corresponding to the water entry potential can be roughly approximated by the height of capillary rise within a soil (Hillel and Baker, 1988). Thus, the water entry potential is expected to be high for a uniform coarse-grained soil and decrease as the amount of fines in the soil increase.

Biointrusion Barrier - As discussed in Sections 2.3.2.2.4 and 2.3.2.2.5, plants and animals penetrating the cover system can create conduits for water to move downward into the waste, and may even transport waste to the surface. Plant roots will generally not grow in soils with water contents below the wilting point. Because coarse materials drain to low water contents, typically below the wilting point, they can serve as barriers to root penetration. To be effective as a root barrier, fines must be kept out of the coarse soil layer. This suggests that the particle-size of the coarse layer material either has to be fine enough such that the overlying fines do not penetrate into it, or an intermediate layer or a geotextile (GT) must be used to retain the overlying soil, as discussed in Section 3.3.6. One design approach deterring animal invasion is to use cobble-size particles that are too heavy for the animals to displace, as discussed in Section 2.3.2.2.5. Another approach is to use a dry, cohesionless uniform material that does not form a stable burrow or tunnel.

Gas Collection Layer - For wastes that produce gas, it may be necessary to collect, transmit, and potentially treat this gas as it is emitted from the buried waste. The coarser layer of the capillary barrier may potentially be used for gas collection and transmission. If the facility is a landfill subject to EPA's gas collection and treatment regulations or if gas emissions through the cover system are a concern, the cover system should incorporate a gas collection system. While these alternative designs may be adequate for hydraulic control, they should generally not be used without gas containment components at MSW landfill sites where landfill gas collection and treatment is required.

3.3.6 Internal Stability

In general, the greater the contrast in texture or particle-size distribution of the fine and coarse-grained soil components of a capillary barrier, the greater the effectiveness of the capillary break (Stormont, 1997). There is concern, however, that finer soil particles will move into the pores of the coarser soil, degrading the interface and reducing the effectiveness of the capillary break. The conventional approach for evaluating the internal stability of the capillary barrier is to ensure the soils satisfy a soil retention criterion. The retention criterion establishes the relationship of grain sizes of adjacent materials necessary for the coarser material to retain the finer material. The retention criteria for soil and geotextile filters are discussed in Section 4.7.

From conventional filter criteria, interface stability is favored by soils having similar particle-size distributions, apparently in conflict with maximizing the effectiveness of a capillary break. Conventional criteria, however, have been developed using high hydraulic gradients for applications such as dams. In contrast, capillary barriers would only rarely, if ever, experience positive pore pressures, and the associated hydraulic gradients would be small. Furthermore, capillary barriers will be subjected to cycles of wetting and drying in response to climatic conditions. Thus, interface stability should be considered under dry conditions, as well as, under relatively small positive water pressures. The biggest risk to internal stability of a capillary barrier may occur during barrier construction. For example, vibratory compaction could cause a large number of fine particles to move into the coarser layer.

Koerner and Daniel (1997) recommend that a GT separator be considered at the capillary barrier interface. They indicate that for extremely long service times (e.g., hundreds of years) fiberglass GTs have been considered for this application. It is noted, however, that with a GT at the capillary barrier interface, the capillary break may occur between the finer-grained soil and GT rather than between the finer- and coarser-grained soils (Stormont et al., 1997). This effect reduces the water storage capacity of the finer-grained soil. The GT could also function as a lateral drainage layer. If it is necessary to use a GT separator, the effects (reduced water storage capacity and lateral drainage) associated with use of the GT should be considered and addressed in the final capillary barrier design.

3.4 Alternate Design Performance Evaluation

3.4.1 Introduction

The preceding sections highlighted how the water storage and lateral diversion characteristics of ET and capillary barriers are affected by factors such as soil type and thickness and slope of the interface. In addition to the influence of material properties and configuration, the "stress" provided by the climate will have a major impact on the performance of these types of barriers. To accommodate these factors into the development of designs and estimating the performance of ET and capillary barriers, numerical simulations can be used. However, numerical simulations have two challenging aspects that must be addressed to enable reasonable representation of actual field conditions. First, for near-surface applications it is necessary to account for the effect of time- and climate-dependent processes, including precipitation, soil water evaporation, and plant transpiration. The second aspect, specific to capillary barriers, is that water movement within the near-surface soils and near the interface is transient, unsaturated flow involving materials of widely varying properties. Accuracy and stability of numerical solutions involving these types of flow behavior can be difficult to achieve.

As previously discussed in Section 1.2.3, EPA recommends that a cover system be designed to minimize percolation to prevent the bathtub effect, with a specific value selected based on the nature of the contained waste, the hydrogeological vulnerability of the site, and other factors. The Agency considers this performance criterion to apply over a considered performance period (e.g., maximum rate over at least a 30-year post-closure simulation).

Numerical modeling should be used to design a cover system that meets this performance criterion. Natural analogs may be used to help predict long-term cover performance, and field monitoring may be required, depending on site-specific percolation criteria.

3.4.2 Numerical Modeling

Computer numerical simulations can be used to predict the water balance performance of a cover system. Computer simulations are only as good as the input data provided and the system modeled. Much of the difficulty comes in obtaining good and accurate input data to correctly predict a cover system's water balance performance. It is advised that a realistic set of input parameters be developed for the simulations based on measurements from the actual soil to be used (at the anticipated installed density and moisture content), values from the literature, and expert opinion. Generally, input properties include unsaturated soil properties (i.e., moisture characteristic curves - matric potential versus moisture content) and hydraulic conductivity. There are a number of practitioners who believe that even a near perfect set of input data and a well-designed computer model will still not yield reliable results. Because of this limitation, it may be prudent in critical applications to not rely solely on the results of one set of computer model predictions and/or to use a larger factor of safety. It is suggested that for critical applications, two different computer models be employed and the results of the simulations compared.

The EPA HELP computer model (Schroeder et al., 1994a,b) is at present the industry standard for conducting water balance analyses for conventional hydraulic-barrier cover systems. This model is discussed in more detail in Section 4.2.3.2. Field applications of the model are discussed in Section 4.3. The HELP model incorporates a number of simplifying assumptions and does not solve the unsaturated flow equations. Thus, it is not considered particularly good for evaluating ET barriers and it is not recommended for evaluating capillary barriers. Unfortunately, there are no public-domain water balance models currently available that are as user friendly as HELP and that properly model unsaturated flow within the cover system soil layers.

A model that may be used for the analysis of ET and capillary barriers is UNSAT-H (Fayer and Jones, 1990), a one-dimensional finite-difference computer program to solve for water and heat flow in soils. This model is discussed in more detail in Section 4.2.8. Field applications of the model are discussed in Section 4.3. The UNSAT-H code solves Richard's partial differential equation (Richards, 1931) and can be used to simulate the water balance for evapotranspirative or non-sloped capillary barriers. However, the vegetation options in the model were developed for the DOE Hanford site near Richland, Washington and may not be applicable to other areas of the country. The model user either assumes that: (i) the vegetation is similar to cheatgrass; or (ii) vegetation quantity is based on a daily leaf area indices input by the user. The vegetation is required to start germinating from a seed before Julian day 91 or after day 273 and to stop transpiring between Julian days 151 to 243. In some areas of the southwest, Tucson, Arizona, for instance, relatively high precipitation and plant transpiration is still occurring after Julian day 243.

Other models that may be considered and that are discussed in this guidance document are LEACHM (Section 4.2.3.3.), SoilCover (Section 4.2.3.5), and, for sloping capillary barriers, HYDRUS-2D (Section 4.2.3.6). All of the models have their specific advantages and disadvantages, some of which are listed in Table 4-1.

3.4.3 Performance Monitoring

Because of design and construction quality control uncertainties, performance monitoring is recommended for alternative covers. Field performance data provide perhaps the most reliable information for assessing whether cover systems are performing as designed. It is recommended that a project specific monitoring system be utilized to monitor the performance of an ET or capillary barrier throughout the life of the cover system. As an example, a lysimeter used by the ACAP program for monitoring landfill cover performance is shown in Figure 3-6. Additional performance monitoring techniques are discussed in more detail in Chapter 8.

Examples of performance monitoring of alternative cover systems are highlighted below:

- **Albuquerque, New Mexico.** The Alternative Landfill Cover Demonstration (ALCD) is a large-scale field test at Sandia National Laboratories located on Kirtland Air Force Base in Albuquerque, New Mexico (Dwyer 1997, Dwyer 1998, Dwyer 2001). Six landfill cover profiles are installed with automated retrieval of water balance data (runoff, lateral drainage, percolation, soil moisture changes within the covers, and precipitation). The covers are periodically stress tested by adding precipitation to the covers through sprinkler systems to simulate worst case infiltration events at various locations in arid and semi-arid climates. Four alternative covers (ET Cover, 2 different Capillary Barrier Designs, and a cover featuring a GCL) are installed next to two prescriptive covers (RCRA Subtitle D - similar to Figure 1-6(a) and RCRA Subtitle C - similar to Figure 1-7) for direct water balance performance comparison. The project's intent is to compare and document the performance of alternative landfill cover technologies of various costs and complexities for interim stabilization and/or final closure of landfills in arid and semi-arid environments. The test covers are constructed side-by-side for comparison based on their performance, cost and ease of construction. The ALCD is not intended to showcase any one particular cover system. The focus of this project is to provide the necessary tools; i.e., cost, construction and performance data so that design engineers can support less expensive, regulatory acceptable alternatives to conventional cover designs. This project has been extensively reviewed by regulators from across the country as well as by panels from the National Academy of Science and the Department of Energy. Results from this project have shown properly designed alternative covers such as ET Covers and Capillary Barriers are as good as or better than their prescriptive counterparts. Results from this demonstration have been used by a number of regulatory agencies to approve permits for the use of an alternative landfill cover in lieu of a prescriptive cover (Dwyer 2001).

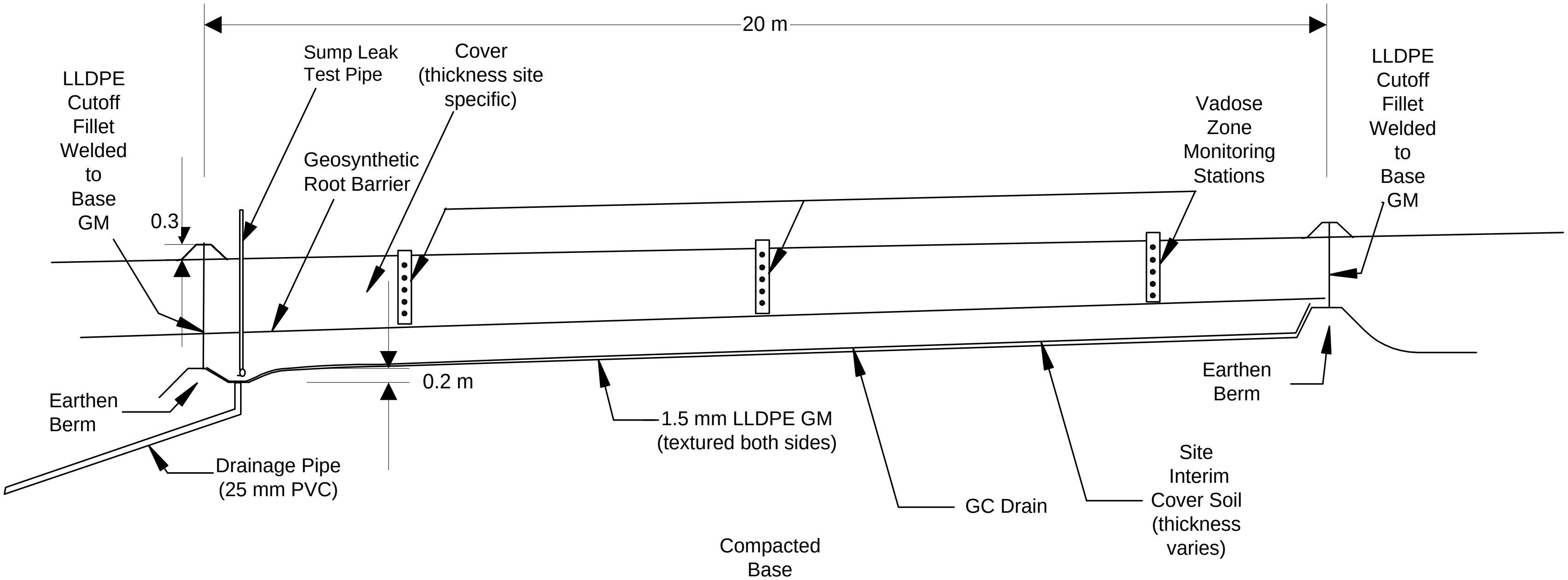
- **EPA's Alternative Cover Assessment Program (ACAP):** (http://www.acap.dri.edu/)

- **Sierra Blanca, Texas** (http://www.beg.utexas.edu/environqlty/vadose/index.htm)

3.4.4 Natural Analogs

Conventional engineering approaches for designing landfill covers often fail to fully consider ecological processes. Natural ecosystems effective at capturing and or redistributing materials in the environment have evolved over millions of years. Consequently, when contaminants are introduced into the environment, ecosystem processes begin to influence the distribution and transport of these materials, just as they influence the distribution and transport of nutrients that occur naturally in ecosystems (Hakonson et al., 1992). As described in Section 1.5.6, as the ecological status of the cover changes, so will performance factors such as water infiltration, water retention, ET, soil erosion, gas diffusion, and biointrusion (Caldwell and Reith, 1993). An important objective for an effective cover system is to design it so that subsequent ecological change will enhance and preserve system performance. Consideration of natural analogs can enhance a cover system design by disclosing what properties are effective in a given environment or what processes may lead to possible modes of failure. These factors can in turn be avoided during the design and construction phases. Natural analog studies provide clues from past environments as to possible long-term changes in engineered covers. Analog studies involve the use of logical analogy to investigate natural and archaeological occurrences of materials, conditions, or processes that are similar to those known or predicted to occur in some part of the engineered cover system (Waugh, 1995).

One possible analog might be observed by trenching adjacent to the site in an undisturbed area and measuring the depth of plant roots (Dwyer et al 1999). This will reveal the general depth of infiltration. Another method for assessing the average long-term depth of water penetration (or infiltration depth) is to trench adjacent to the site in an undisturbed area to observe the depth of calcium carbonate ($CaCO_3$) deposits or formation of a caliche layer. Soils in semiarid and arid regions commonly have carbonate-rich horizons at some depth below the surface. The position of the $CaCO_3$ bearing horizon is therefore, related to depth of leaching, which, in turn, is related to climate (Birkeland, 1984).



Notes:
1. Base shall be approved fill (fines > 30%) compacted to >95% of max. dry unit weight based on ASTM D 698 and dry of optimum water content.
2. Smooth base before placing GM. Eliminate all ridges, depressions, etc. > 25 mm in height. Remove all stones, etc. larger than 10 mm.
3. Place GM in early mornining and ensure good contact with all surfaces. No gaps shall exist between base and GM.
4. Vertical cutoff sheets shall be fillet welded to base GM.
5. GC drain is GN with non-woven GT heat bonded to both sides. Install using rub sheet.
6. Interim cover soil shall be placed on GC drain from the edges. No equipment can travel on GM or GC drain. Once spread in 450 mm loose lift, compact to >85% of max. dry unit weight based on ASTM D 698.
7. Vadose zone monitoring stations instrumented with TDRs to measure soil water content and heat dissipation sensors to monitor soil water pressure and temperature.

Figure 3-6. Test Plot Design Used at ACAP Sites.

The origin of carbonate horizons involves carbonate-bicarbonate equilibria (Birkeland, 1984), as shown by the following reactions:

$$\underset{g}{CO_2} + \underset{l}{H_2O}$$
$$\downarrow$$
$$\underset{s}{CaCO_3} + \underset{aq}{H_2CO_3} \leftrightarrow \underset{aq}{Ca^{2+}} + \underset{aq}{2HCO_3^-}$$

Carbon dioxide partial pressures in soil air are 10 to more than 100 times that in the atmosphere; this decreases the pH, which, in turn, increases $CaCO_3$ solubility. The partial pressure of CO_2 is high as a result of CO_2 produced by root and microorganism respiration and organic matter decomposition. Thus, one would expect the highest CO_2 partial pressure to be associated with the A horizon located near the surface, with values diminishing down to the base of the zone of roots. In arid and semi-arid regions, the quantity of water leaching through the soil is also generally greater near the surface than at depth. Thus, as the water moves vertically through the soil, the Ca^+ and HCO_3^- content might increase to the point of saturation after which further dissolution of $CaCO_3$ is not possible. Combining the effects of high CO_2 partial pressure and downward-percolating water, the formation of $CaCO_3$-rich horizons may be understood as follows. In the upper zone of the soil, Ca^{2+} may already be present or may be derived by weathering of calcium-bearing minerals. Due to plant growth and biological activity, CO_2 partial pressure is high and forms HCO_3^- upon contact with water. Water leaching through the profile carries Ca^{2+} and HCO_3^- downward in the profile. Precipitation of $CaCO_3$ to form a caliche horizon takes place by a combination of decreasing CO_2 partial pressure below the zone of rooting and major biological activity and the progressive increase in Ca^{2+} and HCO^{3-} concentrations with depth in the soil solution as the water percolates downward and water is lost by evapotranspiration. The position (depth) of the $CaCO_3$ bearing horizon is therefore related to depth of leaching, which, in turn, is related to the climate.

As more alternative cover systems are installed and demonstrate successful performance, confidence for their use at other sites will grow. A number of experiments and field-scale demonstrations throughout the country are currently producing field data to document the short-term performance of alternative cover technologies (Dwyer, 1997; Dwyer, 1999; Dwyer, 2001; Benson, 1997). As with any emerging technology, longer-term performance data are lacking. Natural analogs can be used to deduce how a system may perform over a longer period (Waugh, 1995). Computer modeling can be used to predict long-term performance and compare alternative designs (Khire, 1995; Morris and Stormont, 1997a, b). Until long-term performance data have been obtained, the combination of computer model predictions, field data, and natural analog studies forms the basis for evaluating long-term alternative cover system designs.

3.5 Construction of Alternative Designs

ET covers may often be easier to build and require a lesser amount of quality assurance (QA)/QC during construction than conventional designs with hydraulic barriers. This is due to the fact that the ET cover may only involve placement of two soil types, a topsoil layer and the relatively fine-grained ET barrier, and no geosynthetics or soils that must be compacted to meet strict hydraulic conductivity criteria. The complexity of construction of a capillary barrier increases with the number of layers in the system, including layers for soil water storage, internal drainage, biointrusion resistance, and/or gas transmission.

Specific construction and maintenance considerations for alternative cover system designs are discussed below.

3.5.1 Compaction Requirements

CCL hydraulic barriers in conventional cover system designs are compacted to attain a very low saturated hydraulic conductivity. As discussed in Section 2.5.4 of this document, this generally requires compacting the soil lifts 'wet of optimum' to remold the soil and produce high soil densities. Compacting the soil wet of optimum increases the potential for desiccation cracking and reduces the initial water storage capacity since the CCL is generally at a degree of saturation of at least 85%.

The alternative cover system designs outlined in this chapter are designed to function under unsaturated conditions; consequently obtaining very low saturated hydraulic conductivity is not a priority. Because a very low initial saturated hydraulic conductivity is not the objective when placing finer-textured soils in an alternative cover system, compaction "dry of optimum" is usually desired to reduce the potential for desiccation cracking. This compaction alternative also allows for additional initial water storage capacity and a structure that is less restrictive to plant roots. Compaction density requirements for the finer-grained soils should be based on consideration of the water content-unsaturated hydraulic conductivity relationship for the soil, erosion resistance, and plant rooting requirements. Generally, compaction for the ET barrier is performed in an attempt to mimic the naturally occurring in-situ soil density for a particular borrow material. Ideally, target densities for constructed ET cover soils should be within +/- 5% of the in-situ borrow soil density. In addition, this target in-situ soil density should be used for any subsequent laboratory testing and for input parameters in computer water balance models. It should be noted that unsaturated soil properties and saturated hydraulic conductivity are very sensitive to the soil's density. Uniformity of compaction is critical.

3.5.2 Capillary Barrier Soil Interfaces

During the emplacement of a capillary barrier, special care must be taken during the placement and compaction of the first lift of fine-grained soil on the underlying uncompacted coarse-grained soil. The interface between these two materials should remain smooth and continuous and the materials should not be mixed together.

Heavy compaction, especially if a vibratory compactor is used, should be avoided as finer soil may migrate into the coarser layer. Conversely, a lack of compaction will leave the finer soil near the interface in a loose condition. This finer soil could be more prone to internal erosion under the action of seepage forces should gravity-driven water percolation develop at the interface. Small wide-tracked bulldozers have been used to construct this interface. The steel tracks help distribute the weight of the bulldozer over a greater surface area, thus reducing its contact pressure. Kneading compaction is not recommended for the first lift of fine-grained soil; rather a smooth drum roller should be used. This will help minimize the potential for mixing of fine and coarse soils at their interface. The design process for capillary barriers should include an evaluation of appropriate procedures for soil compaction.

3.6 Maintenance and Monitoring of Alternative Designs

3.6.1 Maintenance

Maintenance is discussed in Chapter 9. The most important maintenance activities for the alternative designs involve maintaining the intended vegetative cover and the erosion control measures, repairing erosion gullies, surface depressions caused by localized settlement, surface cracks, and, as an associated activity, maintaining and repairing surface-water management structures.

Maintaining the surface layer and repairing cracks and erosion gullies in alternative cover systems is generally even more critical than maintaining the surface and protection layers in conventional cover systems that have a drainage layer and a GM barrier. A crack in an alternative cover system may allow short circuiting of water through the cover system and impair cover system performance. If differential settlement of an ET barrier occurs, the barrier can simply be repaired by applying more soil to the surface to bring the cover system back to its original grade. For a capillary barrier, the repair is more complex. The finer-grained soil first should be excavated to expose the coarser-grained soil, and the depression in the coarser-grained soil should be filled with the coarser soil so that the interface between the finer and coarser-grained soils is brought back in-line with that adjacent to it. The finer-grained soil at the repair location should then be blended in with (e.g., stair-stepped into) the surrounding finer-grained soil to reduce the potential for preferential pathways for infiltrating water.

3.6.2 Monitoring

Monitoring is discussed in Chapter 9. Alternative cover systems should be monitored to identify problems with excessive erosion, excessive differential settlement, excessive cracking, or slope instability, assess the health of the vegetative cover, and evaluate gas emissions, if gases are a concern. If the cover system water balance is being assessed, the soil moisture content or matric potential, percolation through the cover system, and surface-water runoff may also be monitored.

3.7 Alternative Materials

3.7.1 Geofoam

As described by Horvath (1995a), geofoam refers to any manufactured material created by some internal expansion process that results in a material with a texture of numerous, closed, gas-filled cells. The cell walls are solid, although generally relatively thin and permeable to gases. Currently, the most common geofoam material is expanded polystyrene (EPS), a white foam that is also used for non-geofoam applications, like beverage cups and packaging. It is noted that EPS, along with extruded polystyrene (XPS), another geofoam material, are both referred to by ASTM as rigid cellular polystrene (RCPS) in below-grade applications (Horvath, 1995a). This lightweight material of a density between 10 and 20 kg/m^3 has unique engineering properties. White (1995) presents the following data as typical of EPS:

- water absorption is very low, e.g., 2% (maximum) by volume;
- low temperatures, under-water or wet environments, and exposure to freeze-thaw cycling do not adversely impact mechanical properties;
- EPS is a very efficient thermal insulator (because it is approximately 98 to 99% gas by volume), and this feature has been capitalized upon in several landfill applications; and
- the mechanical properties of elastic modulus, Poisson's ratio, and compressive strength are readily assessed by either static or cyclic loading tests.

According to Horvath (1995a), the only concern with using EPS and XPS geofoams is that they may degrade when in contact with certain chemicals (i.e., petroleum hydrocarbons and, possibly the plasticizer in PVC GMs.

Geofoam has been used above the drainage layer and barrier of a cover system for insulation and because of its lightweight properties (Gasper, 1990). It has also been used as a spray for daily landfill cover (Gasper, 1990), beneath a GM as a smooth protection layer over steep slopes in an abandoned quarry (Horvath, 1995b), and to promote methane and radon gas venting (White, 1995 b).

3.7.2 Shredded Tires

Scrap automobile and truck tires represent a large quantity of waste material that can be used in select construction, operations, and closure applications for waste containment facilities. In the U.S., an estimated 280 million scrap tires are generated annually. When cut into pieces, typically ranging from 50 to 300 mm in length, shredded tires may be used in cover systems as the gas collection layer, the drainage layer, the protection layer, or a component of the foundation layer (GeoSyntec Consultants, 1998a,b,c).

Modern tires are composed of a combination of natural rubber and synthetic rubber elastomers derived from oil and gas. Multiple carbon blacks, extender oils, waxes, antioxidants, and other materials are added to enhance performance characteristics and manufacturing efficiency. Tires contain a bundle of high tensile strength steel wires surrounded by rubber that forms the bead of a tire to provide a firm contact with the rim. The individual wires that compose this bundle can be up to 3 mm in diameter and are relatively stiff. Most tires also contain steel belt wire in the tread and sidewall areas. This wire is much smaller diameter than bead wire and is therefore more flexible.

Metal wires protruding from tire shreds may scratch or puncture GMs and GCLs used in a cover system. Therefore, whenever tire shreds are used in a cover system, careful consideration should be given to the design of adequate protection (e.g., a geotextile or a soil layer between the tire chips and GM) for the cover system geosynthetics. To minimize the potential for bead wires to puncture a GM or GCL, the bead wire protrusions from the tire shreds should be limited (to less than 10 mm for example) and a GT or soil cushion layer should be considered. Project-specific laboratory or field testing is recommended. Tire shreds containing bead wire should not be placed in contact with geosynthetics: either the bead wire needs to be removed or a soil layer needs to be placed between the tire chips and the geosynthetics. Belt wire can also be problematic. The results of a field test program (GeoSyntec, 1998b) show that belt wires in direct contact with a GM can create some minor damage (i.e., indentations, scratches, dents). To reduce the potential for GM damage by protruding or loose belt wire, the GM should be separated from the tire chips by a GT or soil layer. The wires exposed at the cut edges of tire shreds can also be a hazard to personnel walking on the shreds, and can puncture the tires of vehicles trafficking over them. Track mounted or steel-wheeled equipment should be used when practical to mitigate the latter problem.

The exposed metal in tire shreds may also leach metals when exposed to water; however, with exceptions of iron and manganese, the metal concentrations are anticipated to be below their primary or secondary drinking water standards (Duffy, 1996; Humphrey et al., 1997).
Tire shreds are combustible at temperatures above 322 ˚C. Combustion generally requires an external ignition source (e.g., lightning), although there have been several fires in tire-shred fills used for highway embankment fills that seem to be associated with spontaneous combustion due to self-heating. Humphrey (1996) describes three fires that occurred during 1995 in tire shred fills that were at least 6 m deep. Two of these fills are located in Washington, and one is located in Colorado. Humphrey gave several potential mechanisms for ignition of the tire shreds, with the most likely mechanism being oxidation of exposed steel wires. To reduce the potential for future tire fires, Humphrey recommends minimizing the amount of steel belt exposed at the cut edges of tire shreds, minimizing the amount of crumb rubber in the shred material, covering the shreds with at least 1.2 m of soil to limit contact of the shreds with oxygen, not placing organic materials (e.g., topsoil) directly over the shreds, and preventing contact between the shreds and fertilizer. These recommendations may be appropriate for relatively deep fills, but appear to be very conservative for applications where tire shreds are used in a cover system drainage layer or gas collection layer.

Physical characteristics of tire shreds are dependent upon the shred size (gradation), uniformity, exposed wire content, and whether the shreds have been mixed with soils. Compared to natural materials (i.e., sands and gravels) typically used as drainage layer materials, tire shreds have a much larger size. If the tire shreds are used as a drainage or gas collection material, soil or GT filter or separation layers are often required between the shreds and the adjacent materials.

Based on data complied from Ahmed (1993), Humphrey et al. (1993), and Cecich et al. (1996), loosely dumped tire shreds typically exhibit dry densities between 4.0 and 4.8 kN/m^3; the density of compacted tire shreds is typically between 5.5 and 7.5 kN/m^3.

Tire shreds are relatively compressible. Laboratory tests on compacted tire shreds less than 75 mm in length indicate that tire shreds may exhibit vertical strains of up to approximately 20% under low vertical stresses up to approximately 25 kPa (Ahmed, 1993; Nickels, 1995). Tire shred compressibility under the anticipated overburden stress should be accounted for when specifying the minimum thickness of the as-compacted tire shred layer. Because they are so compressible, construction of CCLs over tire shreds may be difficult. GeoSyntec Consultants (1998a) showed that construction of a CCL directly over 300 mm of foundation soil underlain by 300 mm of tire shreds resulted in the development of numerous cracks in the CCL as the tire shreds compressed. Such a relatively thin soil layer over the tire chips made it difficult to obtain the required compaction density in the overlying CCL. Additionally, the weight of a sheepsfoot roller or similar equipment used for compaction of a CCL could cause deflections of the tire shreds in the foundation layer that would be large enough to introduce cracks into the CCL, thereby increasing its hydraulic conductivity. When the foundation layer was modified to a 450-mm thick soil layer over a 150-mm thick tire shred layer, the foundation was adequate for construction of a CCL with a hydraulic conductivity of 1 x 10^{-6} cm/s or less. These results are dependent on the size of the tire shreds and the thickness of the tire shred layer. All other things being equal, smaller shred size and a thinner shred layer will provide more constructible conditions than if these parameters were reversed. A field test program may be considered when assessing the feasibility of constructing a CCL on top of a tire chip layer. The compressibility of tire shreds may also preclude placement of GM directly over a tire shred layer. This is mostly a problem during construction when construction equipment imposes stresses on the GM. For example, the deformation imposed by a low-ground pressure dozer spreading a 0.3-m thick soil layer over the GM may be sufficient to tear welded seams. Moreover, the compressibility of the shreds directly under the GM may complicate placement of the GM itself (i.e., it may be difficult to unroll the GM and the weight of field personnel may cause deformations that are sufficient to complicate field welding). In the absence of a field test program to investigate this issue, GeoSyntec Consultants (1998b) has recommended that at least 0.3 m of soils be placed over the tire shreds to allow construction of the GM and overlying soil layers.

When comprising a gas collection or drainage layer, tire shreds must be able to provide the required flow capacity under the applied normal stress. This is typically not a problem given the relatively low stresses in cover system applications; however, at higher normal stresses, tire shred compression and hydraulic conductivity reduction may be significant. Various tests have indicated the hydraulic conductivity of 12 to 75 mm long tire shreds to be on the order of 0.006 to 0.79 m/s (Edil et al., 1992; Glade et al., 1993; Duffy, 1996) under relatively low normal stresses. The lower end of this range corresponds to smaller tire shreds. High variability in hydraulic conductivity values are due to differences in shred size, initial density, hydraulic gradients, and confining pressures under study conditions.

Available published data on the shear strength of tire shreds indicate a wide range of shear strength properties for tire shreds and tire shred/soil mixtures. The data are from varying test types and test conditions. Humphrey et al. (1993) present data from large-scale direct shear tests conducted on tire chips with three different gradations. At normal stresses ranging from 14 to 68 kPa, the reported failure envelopes (i.e., friction angle and cohesion intercept) ranged from 19˚ and 11.5 kPa to 26˚ and 4.3kPa. At the lower end of the normal stress range (i.e., 14 to 17 kPa), these measured shear strengths yield equivalent secant friction angles of 38 to 45˚.

3.7.3 Sprayed Elastomers

Although sprayed elastomers, such as polyurethane and polyurea, have been used for waterproofing secondary containment systems, concrete water tanks, tunnels, roofs, and other structures, there has been limited application of these materials to waste containment or remediation sites. Sprayed elastomers could potentially function as gas and/or hydraulic barriers in cover systems at these sites. These materials are typically easier and faster to apply than other cover system barriers materials. Sprayed elastomer barriers have fewer seams than GM barriers. However, these materials have not yet been used in a full-scale cover system application, and the installation quality control and quality assurance procedures for such an application are still being developed.

An elastomer barrier can be installed by heating an elastomer, pressurizing it, and spraying it onto a surface. The material can be applied directly to a prepared soil subgrade. However, it may be difficult to achieve a continuous barrier with a uniform finish using this installation practice, especially if the subgrade surface has cracks. Therefore, in a cover system barrier application it may be more appropriate to spray the elastomer onto a lightweight nonwoven heatbonded GT placed without wrinkles or folds on a soil subgrade.

Laboratory testing has been conducted on factory-sprayed and field-sprayed polyurea elastomer samples. Factory-sprayed samples were obtained from the material supplier, and field-sprayed samples were collected from a 30 m x 30 m test plot installed in 1993 at a landfill in Michigan. As described by Miller et al. (1997), the test plot included subplots with elastomer sprayed over a prepared soil subgrade with some cracks, over a moist prepared soil subgrade with less cracks, over a lightweight nonwoven heatbonded GT placed on a prepared soil subgrade, and over a woven GT placed on a prepared soil subgrade. Half of the sprayed area on each subplot was covered with approximately 150 mm of soil and the other half was left exposed. The results of mechanical and hydraulic tests conducted on the factory-sprayed elastomer samples and interface direct shear tests conducted on the field-sprayed elastomer samples are presented by Cheng et al. (1994). According to Miller et al. (1997), the elastomer sprayed over the nonwoven GT appeared to provide the best barrier installation. Field samples have been removed from this barrier at periodic intervals to assess long-term performance. No significant degradation or deterioration in the mechanical or hydraulic properties of the barrier samples has been observed (Miller et al., 1997).

It should be recognized that sprayed elastomers have not yet been used in a full-scale cover system application. While this type of application holds some promise, additional research and development is necessary.

3.7.4 Paper Mill Sludges

Paper mill sludges have been shown to have properties similar to those of clays and, as a consequence, have been used as the hydraulic barrier material for some landfill cover systems in, at least, Maine, Wisconsin, and Massachusetts (Zimmie and Moo-Young, 1995). From the limited engineering properties data available for paper mill sludges, the properties vary considerably among the sludges depending on the manufacturing process, water content, organic content, sludge age, degree of consolidation, and other factors. Since the sludges are degradable, their properties are time dependent. The degradation processes also generate gases, which must be managed.

Zimmie and Moo-Young (1995) performed laboratory tests to evaluate the water content, organic content, specific gravity, permeability, compaction, consolidation, and strength characteristics of seven paper mill sludges of various ages. They found that the sludges had a high initial water content ranging from 150 to 268%, an initial solids content of 27 to 40%, and an initial hydraulic conductivity ranging from about 5×10^{-10} to 5×10^{-8} m/s, and behaved similarly to highly organic soil.

Zimmie and Moo-Young (1995) also performed laboratory tests on six undisturbed samples of a sludge used as the cover system barrier material for a MSW landfill in Massachusetts. Three samples of the sludge were obtained shortly after construction and the other three samples were taken at 9, 18, and 24 months after construction. The results of the laboratory tests on these samples indicated that the water content and hydraulic conductivity of the sludge decreased somewhat over time, presumably as the sludge consolidated and biodegraded (i.e., it mineralized to become more like a kaolin clay).

Moo-Young and Zimmie (1996) evaluated how freeze-thaw affects the hydraulic conductivity of paper mill sludges through a series of laboratory tests on sludge samples and by monitoring the depth of frost penetration in the sludge barrier for the previously-mentioned MSW landfill in Massachusetts. Based on the results of their laboratory tests, performed over a range of water contents, if a sludge barrier is subjected to freezing and thawing cycles, the hydraulic conductivity of the sludge may increase by one to two orders of magnitude. Over the several year field study, the frost layer had not penetrated into the sludge barrier due to the protection provided by the overlying soil layers and the high water content of the sludge.

When using paper mill sludge in a cover system application, the chemical characteristics of the sludge need to be considered. Water percolating through the sludge may mobilize volatile organic compounds and heavy metals contained in the sludge. To keep certain chemicals from leaving the site (e.g., as runoff), paper mill sludge may be required to be the barrier or be located below the barrier. Depending on its chemical properties, it may not be suitable for use as a protection layer.

Chapter 4
Hydraulic Analysis and Design

4.1 Introduction

This chapter provides information on select topics related to cover system hydraulic analysis and design. The specific topics discussed in this chapter are:

- characteristics of selected water balance models (Section 4.2);
- evaluation of the water balance models (Section 4.3);
- recommendations for application of the water balance models (Section 4.4);
- design of drainage layers (Section 4.5);
- design of slope transitions (Section 4.6); and
- design of filter layers (Section 4.7).

4.2 Characteristics of Water Balance Models

4.2.1 Overview

As described in Section 1.2.5, with EPA's liquids management strategy, a primary function of a cover system is to limit post-closure leachate generation by minimizing or preventing, for all practical purposes, percolation of water into the waste. A water balance analysis is used to predict the quantity of this percolation. In addition to estimating percolation, water balance analyses of cover systems are used to:

- develop an understanding of how the various cover system components will function and identify which water routing mechanisms are most important;
- compare the performance of different cover system designs; and
- define the performance criteria for various cover system components (e.g., required storage capacity of surface and protection soil layers, required flow capacity of drainage layer) so that these components can be designed.

This section of the guidance document describes the water balance concept and presents several water balance analysis methods commonly used for cover systems.

4.2.2 Water Balance Concept

In a water balance analysis, water is routed into and out of a system using a series of calculations that require conservation of water mass. The potential pathways for water movement into and out of a cover system are illustrated in Figure 4-1. A cover system water balance is expressed in terms of water inflows and outflows and storage changes for a unit area of the system over some arbitrary time interval as:

$$P = R + ET + \Delta W_{surface} + \Delta W_{soil} + L + PERC + \Delta W_{foliage} \quad \text{(Eq. 4.1)}$$

where: P = precipitation (mm/day); R = runoff (mm/day); ET = evapotranspiration (mm/day); $\Delta W_{surface}$ = change in water storage at surface (mm/day); $\Delta W_{foliage}$ = change in water storage on plant foliage (mm/day); ΔW_{soil} = change in water storage in cover system soil (mm/day); L = lateral drainage (mm/day); and PERC = percolation through the cover system (mm/day). Water is input to the cover system as precipitation in the form of rain or snow and lost from the cover system by runoff, ET, lateral drainage, and percolation. Water also is stored on the cover system as ponded water or snow, on plant foliage, and in cover system soils by capillary action. Eq. 4.1 is cast above using a time interval of one day; the equation could be developed using any other time unit.

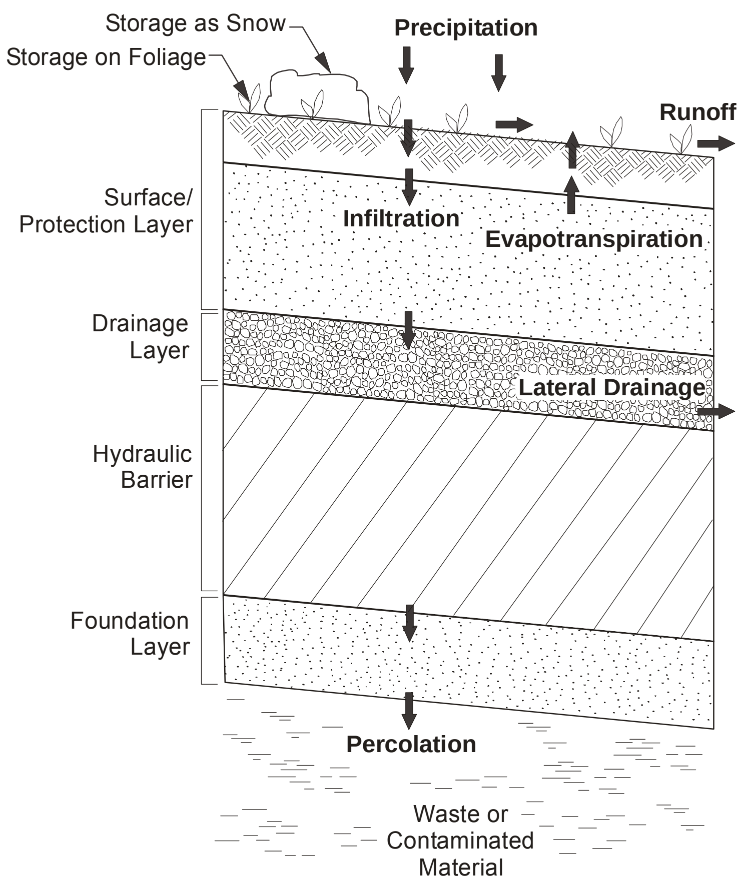


Figure 4-1. Water movement and storage in cover system.

Storage of water in soil coupled with removal of water by ET are the most important mechanisms for limiting percolation of infiltration. For most cover systems, infiltration is primarily removed from the cover system by ET. Flow from lateral drainage layers is typically a much smaller component of the water balance than is ET. It should be remembered, however, that while the internal drainage layer is typically of secondary importance to the overall cover system water balance, it is of prime importance to cover system slope stability (see Chapter 6 of this document). If even a relatively small amount of potential lateral flow is left undrained in a cover system, hydraulic heads can build up over the hydraulic barrier, leading to destabilizing seepage forces on cover system slopes.

Though Eq. 4.1 appears simple, the components of the water balance are dependent on many factors, are difficult to quantify, and are interdependent. It can be especially difficult to quantify percolation in arid and semi-arid environments where almost all precipitation is consumed by ET. Unlike in wetter climates where actual ET may approach the magnitude of potential evaotranspiration (PET) (i.e., the process is energy limited), in drier climates actual ET is generally much smaller than PET due to the lack of available water. ET is more difficult to accurately estimate under water limiting conditions. Because the magnitude of percolation in drier climates is so much smaller than the magnitudes of ET and precipitation, relatively small errors in estimated ET can result in relatively large errors in estimated percolation. Due to the difficulty in performing accurate analytical water balances, field water balances have occasionally been performed using cover system test plots to better assess the water balances components (e.g., the ACAP program, as described in Section 3.4.3). For example, field water balances have been performed for alternative cover systems without GM barriers and for cover systems at low level radioactive waste containment and disposal sites. Examples where field methods have been used to investigate one or more components of a cover system water balance include Cartwright et al., 1988; Nyhan et al., 1990; Anderson et al, 1993; Gee et al., 1994; Limbach et al, 1994; Melchior et al., 1994; Waugh et al., 1994; Dwyer, 1995; Khire, 1995; Sackschewsky et al., 1995; Schultz et al, 1995; Paige et al., 1996; Anderson, 1997; Gee et al., 1997; Karr et al., 1997; Khire et al., 1997; Laundré, 1997; Melchior, 1997a,b; Morris and Stormont, 1997; Nyhan et al, 1997; Ward and Gee, 1997; Dwyer, 1998; Khire et al., 1999; Dwyer, 2001; and Scanlon et al., 2002.

Water balance calculations are performed for time intervals that may be shorter than one hour or longer than a year. The time interval to use is dependent on the purpose of the water balance analysis. Guidance on the time interval to use for design is given subsequently.

4.2.3 Water Balance Methods

A variety of water balance methods are available to evaluate and design cover systems. They range in complexity from relatively simple empirical correlations to sophisticated computer-based finite difference and finite element mechanistic models. This guidance document describes the following water balance analysis methods: (i) simplified manual method; (ii) Hydrologic Evaluation of Landfill Performance (HELP) model; (iii) Leachate Estimation and Chemistry Model (LEACHM); (iv) UNSAT-H model: (v) SoilCover model; and (vi) HYDRUS-2D model. These are all well-documented manual methods or computer codes that consider the significant water balance processes (e.g., precipitation, runoff, and ET) and that have been used previously for cover system water balance analyses. All of the models except HYDRUS-2D are in the public domain. The characteristics of these models are compared in Table 4.1.

4.2.3.1 Simplified Manual Method

Koerner and Daniel (1997) present an updated version of the simplified method for performing manual or computer spreadsheet water balance calculations for cover systems. Their method is based on the previous work of Thornthwaite and Mather (1955, 1957), Fenn et al. (1975), and Kmet (1982). In this previous work, only monthly time steps were considered. Historically, simplified water balances using monthly time steps were used for cover system analysis and design. The computer code MBALANCE (Scharch, 1985), based on the simplified manual method with a monthly time step, was developed for landfill cover systems by Wisconsin Department of Natural Resources. This model was used in simulations that were compared to field water balances (Lane et al., 1992). Koerner and Daniel (1997) extended the method to consider a variable time step (e.g., daily, weekly, or monthly) to be selected based on the purpose of the analysis. A spreadsheet developed by Koerner and Daniel (1997) to evaluate monthly percolation through a cover system is shown in Table 4-2. The table is readily adaptable to PC-based spreadsheet computations and can be easily modified to accommodate daily or hourly time steps. Guidance on, and an example of, the use of Table 4-2 are presented in Koerner and Daniel (1997). The equation numbers given in the table are from that reference. The remainder of this section addresses several important aspects of the simplified manual method.

In the simplified manual method, it is assumed that no water is stored at the surface or intercepted by plants (i.e., $\Delta W_{surface} = \Delta W_{foliage} = 0$). For this set of assumptions, the following relationships are defined for a time interval taken as one day:

$$P = I + R \qquad \text{(Eq. 4.2)}$$

$$I = ET + \Delta W_{soil} + PERC^* \qquad \text{(Eq. 4.3)}$$

where: I = infiltration into cover soil (mm/day); and PERC* = percolation through cover soil (mm/day); and other terms are as defined previously.

In the simplified manual method, precipitation is partitioned into runoff and infiltration (Eq. 4.2). Runoff is calculated as a fraction of precipitation using the rational formula and a runoff coefficient appropriate for the cover system soil type and slope. According to Fenn et al. (1975), the rational formula will, in most cases, underestimate the quantity of cover system runoff.

From Eq. 4.3, water infiltrating the cover soil is partitioned into ET, soil water storage, and percolation through the cover soil. In the simplified manual method, ET is calculated as a function of PET, infiltration, and initial moisture content of the soil. PET is calculated using an empirical method developed by Thornthwaite and Mather (1955). If more water infiltrates the cover system than can potentially evapotranspire, the excess water will first be distributed within the root zone until the soil moisture content is at field capacity. The remaining water will be routed as percolation through the cover soil. If ET is greater than infiltration, then stored water will be lost from the cover soil root zone until the soil moisture content is at wilting point.

Table 4-1. Comparison of select water balance models.

Model	Reference	Calculation Scheme	Advantages	Disadvantages	Appropriate Use
Simplified Manual Method	Koerner and Daniel (1997)	Simplified empirical and mechanistic equations	Easy to perform Few data requirements Any time step Considers lateral drainage	Numerous simplifying assumptions must be made Steady-state conditions are assumed Essentially all calculations are uncoupled Cannot be used for unsaturated flow	Instructional tool for design of hydraulic barriers Check of computer simulations Parametric evaluations Calculation of peak lateral drainage from cover system
HELP	Schroeder et al. (1994a, 1994b) for EPA	Quasi 2-D water-routing model with multiple uncoupled subroutines Simplified empirical and mechanistic equations Simplified unsaturated flow model with unit hydraulic gradient	Widely accepted Used to design hydraulic barriers Easy to run simulations Default database of climatic, soils, and vegetation data Considers lateral drainage	Does not solve unsaturated flow equations Demonstrated overprediction of percolation in many cases Limited to daily climatic data	Design of hydraulic barriers Regulatory compliance demonstrations Parametric evaluations Calculation of peak lateral drainage from cover system
LEACHM	Hutson and Wagenet (1992) for Cornell University	Finite difference model with unsaturated flow model based on Richards' partial differential equation User specified boundary conditions	Mechanistic model Solves unsaturated flow equation May give a better estimate of ET in arid climates than other models	Maximum soil profile depth of 2 m Does not consider lateral drainage	Design of ET and capillary barriers (no lateral flow) Parametric evaluations Unsaturated flow analysis
UNSAT-H	Fayer and Jones (1990) for Pacific Northwest Laboratory	Finite difference model with unsaturated flow model based on Richards' partial differential equation User specified boundary conditions	Mechanistic model Solves unsaturated flow equation Flexibility in definition of unsaturated hydraulic conductivity-head-moisture content relationships	High computational demands Unsuitable for parametric evaluation Does not consider lateral drainage	Performance assessment of ET and capillary barriers (no lateral flow) Calibration with field data prior to making long-term predictions Unsaturated flow analysis

Table 4-1. Comparison of select water balance models (continued).

Model	Reference	Calculation Scheme	Advantages	Disadvantages	Appropriate Use
SoilCover	SoilCover (2000)	Finite element model with unsaturated flow model based on Richards' partial differential equation User specified boundary conditions	Mechanistic model Solves unsaturated flow equation Calculates actual evaporation based on matric suction at soil surface Easy to create input files with spreadsheet user interface	Limited boundary condition options High computational demands Maximum of 8 soil layers Maximum of 100 nodes Does not consider lateral drainage Requires temperature input	Performance assessment of ET and capillary barriers (no lateral flow) Unsaturated flow analysis
Hydrus 2-D	Šimůnek et al. (1999) for U.S. Salinity Laboratory	Two-dimensional finite element model with unsaturated flow model based on Richards' partial differential equation User specified boundary conditions	Mechanistic model Solves unsaturated flow equation Flexibility in definition of unsaturated hydraulic conductivity-head-moisture content relationships Considers lateral flow and anisotropy Inverse estimation of hydraulic properties from measured data Considers spatial heterogeneity	High computational demands Does not include vapor flow Does not calculate PET from climatic data Not in public domain	Performance assessment of ET and capillary barriers with lateral flow Calibration with field data prior to making long-term predictions Unsaturated flow analysis

Table 4-2. Example spreadsheet for simplified manual water balance method (Koerner and Daniel, 1997).

Row	Parameter	Reference	January	February	March	April	May	June	July	August	September	October	November	December	Total
A	Avg. Monthly Temp, °C	Input Data													
B	Monthly Heat Index (H_m)	Eq. 4.7													
C	Unadjusted Daily PET (UPET), mm	Eqs. 4.8 and 4.9													
D	Possible Monthly Duration of Sunlight (N)	Table 4.3 or 4.4													
E	PET, mm	PET = UPET - N													
F	Precipitation (P), mm	Input Data													
G	Runoff Coefficient (C)	See Table 4.1													
H	Runoff (R), mm	R = P – C													
I	Infiltration (IN), mm	IN = P – R													
J	IN – PET, mm														
K	Accumulated Water Loss (WL), mm	$WL = \sum(IN - PET)_{<0}$													
L	Water Stored (WS), mm	Section 4.3.1.12													
M	Change in Water Storage (CWS), mm	Section 4.3.1.13													
N	Actual ET (AET), mm	Eq. 4.16													
O	Percolation (PERC), mm	Eq. 4.18													
P	Check (CK), mm	Eq. 4.19													
Q	Percolation Rate (FLUX), m/s	Eq. 4.20													

If water does not flow laterally through an internal drainage layer, percolation through the hydraulic barrier is equal to percolation through the cover soil (i.e., PERC* = PERC). Conversely, if lateral flow occurs:

$$\text{PERC}^* = \text{PERC} + \text{L} \qquad \text{(Eq. 4.4)}$$

where all terms are as defined previously. In the simplified manual method, Eq. 4.4 is solved iteratively since both PERC and L are a function of hydraulic head.

Assuming steady-state conditions, the maximum flow in the internal drainage layer is calculated as:

$$q_m = \frac{\ell \text{L}}{8.64 \text{ x } 10^7} = \frac{\ell\left(\text{PERC}^* - \text{PERC}\right)}{8.64 \text{ x } 10^7} \qquad \text{(Eq. 4.5)}$$

where: q_m = maximum flow rate in drainage layer per unit width perpendicular to the direction of flow (m^3/s/m);λ = slope length (m); and other terms are as defined previously. The hydraulic transmissivity of the drainage layer must be adequate to accommodate this flow. The flow capacity of drainage layers was discussed in 2.4.2.3. Hydraulic design of drainage layers is discussed subsequently in Sections 4.5 and 4.6.

Koerner and Daniel (1997) recommend that the hydraulic requirements of a cover system drainage layer be evaluated based on a single storm event. They conservatively suggest that, for design, the cover soil above the drainage layer be assumed to be saturated and that percolation through the cover soil be set equal to infiltration into the cover soil (i.e., ET = 0 and $\Delta W_{soil} = 0$). For these conditions:

$$\text{PERC}^* = \text{P} - \text{R} \qquad \text{(Eq. 4.6)}$$

where all terms are as defined previously. Applying the rational formula to the calculation of R leads to:

$$\text{PERC}^* = \text{P } (1 - C_r) \qquad \text{(Eq. 4.7)}$$

where: C_r = runoff coefficient (dimensionless) obtained from Table 4-3 or project-specific information.

Eq. 4.7 was developed assuming that: (i) the cover soil is at field capacity before the storm begins; (ii) there is no ET during the storm; and (iii) the cover soil is sufficiently permeable to accept the calculated infiltration. To account for this last condition, Koerner and Daniel (1997) suggest that PERC* calculated with Eq. 4.7 be adjusted in accordance with Thiel and Stewart (1993) using Eq. 4.8a or 4.8b, depending on a comparison of the rate at which water becomes available for infiltration to the saturated hydraulic conductivity of the cover soil.

Table 4-3. Runoff coefficients (from Fenn et al., 1975) suggested by Koerner and Daniel (1997) for simplified manual water balance calculations.

Soil Description	Slope	Runoff coefficient
Sandy Soil	Flat (≤ 2%)	0.05 - 0.10
Sandy Soil	Average (2 - 7%)	0.10 - 0.15
Sandy soil	Steep (≥ 7%)	0.15 - 0.20
Clayey Soil	Flat (≤ 2%)	0.13 - 0.17
Clayey Soil	Average (2 - 7%)	0.18 - 0.22
Clayey Soil	Steep (≥ 7%)	0.25 - 0.35

$$\text{PERC*} = P(1 - C_r) \quad \text{when} \quad k_{cs} \geq P(1 - C_r) \qquad \text{(Eq. 4.8a)}$$

$$\text{PERC*} = k_{cs} \quad \text{when} \quad k_{cs} \geq P(1 - C_r) \qquad \text{(Eq. 4.8b)}$$

where: k_{cs} = the cover soil saturated hydraulic conductivity in the same units as P. Eq. 4.8 can be used to develop a conservative estimate of peak flow into a lateral drainage layer during a single storm event, a capability available in only one (i.e., HYDRUS-2D) of the other water balance models considered in this chapter.

In the simplified manual method, percolation through CCL or GCL barriers is calculated using Darcy's equation, which describes the flow of fluids through porous media. Percolation through GM and composite liners is calculated by Koerner and Daniel using the leakage rate equations developed by Giroud and Bonaparte (1989a,b). Hydraulic head is an input parameter to these equations. It is suggested that the maximum hydraulic head calculated on a monthly basis (h_m as derived subsequently) be used to calculate leakage rates through hydraulic barriers.

Input data needs for the simplified manual method are minimal. Only precipitation and mean temperature data are required. Koerner and Daniel (1997) provide guidance for selecting all other parameters (e.g., runoff coefficient, root zone depth, and soil water storage capacity). The advantages of the method are its simplicity, ability to use a variable time step, and ability to calculate lateral flows in cover system drainage layers. The main disadvantages of the method are the steady-state nature of all calculations and the numerous simplifying assumptions. Nonetheless when appropriately used, the simplified manual method presents an acceptable approach to the design of hydraulic barrier type final cover systems. The method is in no way adequate as a simulation or predictive tool, nor is it applicable to the analysis or design of capillary barriers or ET barriers.

4.2.3.2 HELP

The HELP computer code was developed by the U.S. Army Corps of Engineers Waterways Experiment Station (WES) for EPA to enable design engineers to compare the relative hydraulic performance of alternative waste containment system designs (Schroeder et al., 1994a, 1994b). Increasingly, HELP is being used to calculate percolation rates through cover systems and peak hydraulic heads in cover systems for slope stability analyses. HELP has been updated extensively since its inception. At the time of this writing, HELP Version 3.07 is the most recent revision. The documentation for HELP by Schroeder et al. (1994a, 1994b) can be purchased from the National Technical Information Service ((800) 553-6847), downloaded from the USEPA website at http://www.epa.gov/cincl/, or downloaded from the WES website at http://www.wes.army.mil/el/elmodels. The most recent version of the code can be downloaded from the WES website. Additional guidance on using HELP to evaluate landfill hydrologic performance can be found in EPA (1991). Users should use the most current version of the HELP model at the time the analysis is to be performed. Users should also recognize that conclusions drawn from studies using older versions of the model may not be the same as the conclusions that would be drawn using the most current version of the model.

The HELP model simulates hydrologic processes for landfills by performing sequential water balance calculations using a quasi-2-D, gradually varying approach. According to Peyton and Schroeder (1993), the model is considered quasi 2-D because it considers only vertical flow in all layers except lateral drainage layers, where flow can be vertical or lateral. The model is considered gradually varying because the simulation moves through time with the water balance processes being considered steady over each time step. A conceptualization of the HELP model is presented in Figure 4-2. The model can be used to separately evaluate each subprofile shown in Figure 4-2, including the complete cover system profile.

The hydrologic processes considered in the model include precipitation, surface-water storage (i.e., storage as snow), interception of precipitation by foliage, surface-water evaporation, runoff, snow melt, infiltration, plant transpiration, soil water evaporation, soil water storage, vertical flow (saturated and unsaturated) through non-barrier soil layers, vertical percolation (saturated) through soil barriers, vertical percolation (saturated) through GM and GM/soil composite barriers, and lateral drainage (saturated). Five main routines are used in the HELP model to estimate runoff, ET, vertical drainage to barriers, vertical percolation through soil barriers, and lateral or vertical flow (saturated) through lateral drainage layers. Several other routines interact with the main routines to generate daily precipitation, temperature, and solar radiation values and to simulate snow accumulation and melt, vegetative growth, interception, and vertical percolation through GM and GM/soil composite barriers.

Runoff in the HELP model is computed using the runoff curve number method of the USDA SCS) (SCS, 1985). (Note that the Soil Conservation Service (SCS) is now the Natural Resources Conservation Service (NRCS).) The method empirically correlates total runoff with total rainfall based on daily rainfall records, vegetation type, soil type, antecedent moisture conditions (level of soil moisture prior to rainfall), and other factors. The method does not consider the time

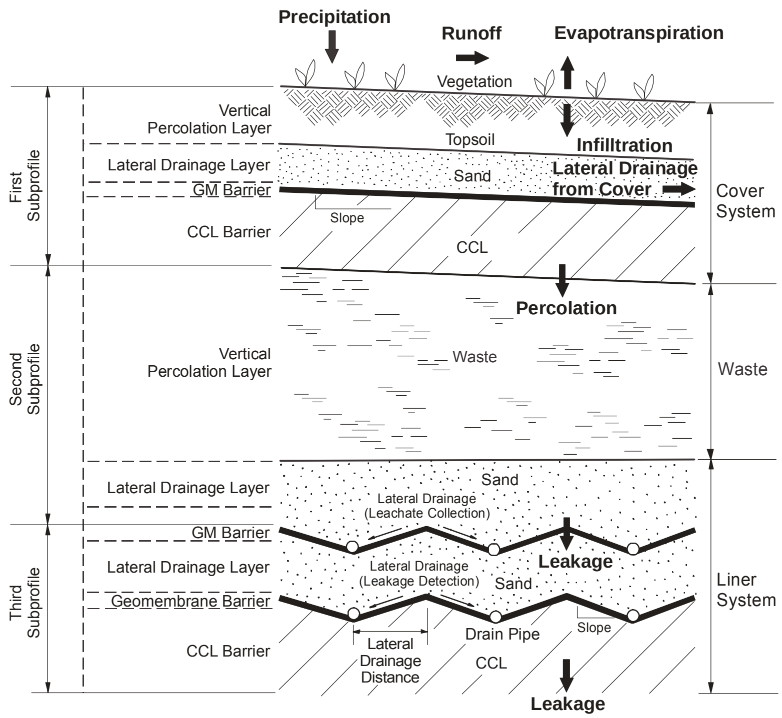


Figure 4-2. Conceptualization of HELP water balance model (from Schroeder et al., 1994a).

distribution of rainfall intensity and, therefore, does not give accurate estimates of runoff volumes for individual storm events. The daily runoff is calculated in the model as:

$$R = \frac{(P - 0.2S_r)^2}{(P + 0.8S_r)} \qquad \text{(Eq. 4.9)}$$

where: S_r = retention parameter (mm/day) dependent on SCS curve number; and R and P are as defined previously. The SCS curve number is a function of soil texture, vegetation quality, and cover system slope length and inclination. Schroeder et al. (1994a) indicate that long-term cumulative runoff should be independent of rainfall duration and intensity, since over a long simulation period a variety of precipitation events will occur. However, McBean et al. (1995) state that use of daily rainfall averages effectively decreases storm intensity (because the duration of most storms is less than 24 hours), resulting in a simulation having an overprediction of infiltration and underprediction of runoff.

ET is computed in HELP by a two-stage modified Penman energy balance method developed by Ritchie (1972). This method uses the PET concept as the basis for prediction of surface and soil water evaporation and plant transpiration. The PET demand is first met by evaporation of water or snow on foliage or on the ground, then soil water evaporation, and finally plant transpiration. ET is assumed to occur within the evaporative zone depth specified by the user and is not allowed to occur within or below a barrier. Also, the soil water content is not allowed to decrease below the wilting point, which is defined in the model as the volumetric water content at a matric potential of -1.5 MPa. Due to these controls, ET may be underestimated in arid climates. Growth and decay of surface vegetation is modeled using an algorithm taken from the Simulator for Water Resources in Rural Basins (SWRRB) model (Arnold et al., 1989).

Vertical drainage for cover soil (i.e., topsoil and protection) layers for both saturated and unsaturated flow conditions is computed using Darcy's equation. HELP assumes that soil pressure head is constant within a vertical percolation layer. Changes in either positive or negative pressure head cannot be modeled. The hydraulic gradient is due to change in elevation head only and is thus equal to 1.0. The HELP model does, however, define an unsaturated hydraulic conductivity to use with the unit hydraulic gradient for calculating unsaturated flow rates. The unsaturated hydraulic conductivity, k_u (m/s), is obtained in the HELP model using Campbell's equation (1974):

$$k_u = k_s\left[(\theta - \theta_r)/(\theta_s - \theta_r)\right]^{3+2/\lambda} \qquad \text{(Eq. 4.10)}$$

where: k_s = saturated hydraulic conductivity of soil layer (m/s); θ = volumetric water content of soil layer (dimensionless); θ_s = volumetric water content of soil layer at saturation (dimensionless); θ_r = residual volumetric water content, typically in the range of 0.01 to 0.10 (dimensionless); and λ = pore size distribution index (dimensionless), calculated as described in Schroeder et al. (1994a,b). As a result of the hybrid formulation given above, the HELP model cannot be used to simulate the physics of water movement through an unsaturated soil layer.

Lateral drainage below a cover soil layer is modeled by an analytical approximation to the steady-state solution of the Boussinesq equation. The peak daily head in a drainage layer is calculated using an equation formulated by McEnroe (1993). Vertical percolation through low-permeability soil hydraulic barriers is evaluated in HELP using Darcy's equation assuming saturated conditions. Vertical percolation through GMs and GM/soil composite barriers is evaluated based on the work of Giroud and Bonaparte (1989a,b) and Giroud et al. (1992a).

The daily water balance is calculated in the HELP model by a linking process, starting with a surface water balance, then ET in the subsurface, and finally subsurface water routing from the surface downward one soil layer at a time. The routing procedure uses a time step that can range from 30 minutes to six hours. However, only daily, monthly, annual, and long-term average output data are reported.

The HELP model requires daily and general climatic data, material properties data for the landfill components being modeled, and landfill design data. One of the strengths of the HELP model is its climatic and material property default data option. Required daily weather data are precipitation, mean temperature, and total global solar radiation. Daily precipitation may be input manually, selected from a historical database (e.g., 1974-1977 data in the HELP database, NOAA Tape, or Climatedata™ files), or generated stochastically using a weather generation model developed by the U.S. Department of Agriculture-Agricultural Research Service (USDA-ARS) (Richardson and Wright, 1984) with simulation parameters available for 139 U.S. cities. It should be noted that the historic precipitation data in the database for 1974-1977 are often not used because they are for an unusually dry time period in certain parts of the U.S. Other daily climatologic data are generated stochastically using the USDA-ARS routine. Required general weather data include average annual wind speed and latitude. Default general weather data for 183 U.S. cities are used by the model. The material properties of each layer being modeled are either selected from the HELP model database of default material properties or are specified by the model user. Landfill design data, including landfill general information and layer configuration, are user specified.

Due to its method of calculating downward flux and its limiting of upward flux (i.e., no upward flux within or below a barrier), version 3.07 of the HELP model is not considered a particularly accurate simulation model for cover systems located in arid areas where the subtleties of unsaturated moisture movement can dominate the water balance. As will be discussed, there are other water balance models that better simulate the physics of water movement in arid environments.

4.2.3.3 LEACHM Model

LEACHM (Hutson and Wagenet, 1992) is a one-dimensional finite difference code that is finding increasing use in the western United States, particularly California, for design and performance analysis of cover systems with ET barriers. LEACHM was originally developed to simulate the effects of agricultural management alternatives on the movement of water and chemicals in a shallow soil profile (i.e., to a maximum depth of 2 m). Only the hydrologic component of the model will be discussed further. The code and model documentation may be obtained from the Department of Soil, Crop & Atmospheric Sciences at Cornell University, Ithaca, New York.

The LEACHM model considers precipitation, runoff, ET, soil water storage, and percolation in the water balance. Infiltration of water into the soil profile and vertical drainage are simulated using a finite difference solution to Richards' partial differential equation (Richards, 1931). This equation is obtained by combining the differential form of Darcy's equation for unsteady vertical flow with the one-dimensional differential form of the conservation of mass equation:

$$\frac{\partial\theta}{\partial t} = \frac{\partial}{\partial z}\left[k_u(\theta)\frac{\partial\psi(\theta)}{\partial z} - 1\right] - S(z,t) \qquad \text{(Eq. 4.11)}$$

where: ψ = matric potential (negative) due to capillary suction forces (N/m^2); θ = soil volumetric water content (dimensionless); k_u = unsaturated hydraulic conductivity (m/s); z = vertical coordinate, positive downward (m); t = time (s); and S(z,t) = sink term representing uptake by transpiration (s^{-1}).

Unsaturated soil hydraulic conductivity in LEACHM is calculated using the Campbell (1974) relationship. Precipitation in excess of the infiltration capacity of the soil is shed as runoff. Evaporation and transpiration are modeled separately based on the methods of Childs and Hanks (1975). With this method, the potential evaporation and transpiration are first estimated based on the pan evaporation rate, a pan factor, and a crop cover fraction. The actual evaporation is then calculated as the lesser of the potential evaporation and the possible evaporation calculated using Richards' equation and the selected boundary condition. Any remaining PET demand is applied to transpiration. However, transpiration is not allowed if the matric potential head of the soil is less than –1.5 MPa, the potential assumed to correspond approximately to the soil wilting point.

LEACHM requires that climatic data, soil properties, vegetation data, and initial and boundary conditions be input. Unlike the HELP model, there are no default data; the user must specify each input parameter. Required weather data are precipitation magnitude, rate, and start time, minimum and maximum air temperatures, and pan evaporation rate. The precipitation option allows rainfall data for short, intense storms to be input. Thus, LEACHM may be used to estimate the head of water in the cover system due to a design storm. In the absence of pan evaporation rate data, the rate can be calculated by LEACHM using the Linacre equation (Hutson and Wagenet, 1992) with site-specific data (i.e., latitude, elevation, temperature, and precipitation). Required soil data are bulk dry density, initial moisture content, saturated hydraulic conductivity, and soil water retention curve. If a soil water retention curve is not available, LEACHM contains a routine to compute fitting parameters for Campbell's soil-water retention curve from the particle size distribution, bulk density, and organic matter content of the soil. However, there is considerable uncertainty in the use of the regression equations to compute these parameters. Vegetation data to be input are root depth and distribution, plant growth options (i.e., constant vegetation or growing vegetation), wilting point, minimum root potential, maximum ratio of actual to potential transpiration, root resistance, and plant growth timeline (e.g., germination, emergence, maturity, etc.). Very little guidance is provided in the LEACHM model documentation on selection of values for the various input parameters.

To set up the finite difference grid used by LEACHM, the soil profile is divided into a number of horizontal layers of equal thickness with nodes at the center of each layer. Soil properties are then specified for each layer. Two additional nodes are required for boundary conditions, one above the ground surface and one below the profile being modeled. The upper boundary condition can be changed with time by adjusting the head to simulate ponded or non-ponded infiltration, evaporation, or zero flux. The lower boundary condition can be selected as a fixed water table, free drainage (or unit gradient), zero flux, or lysimeter boundary. The initial condition is specified by assigning an initial head or water content to each node in the finite-difference nodal grid. Simulation output includes cumulative infiltration, evaporation, transpiration, and percolation at select times.

4.2.3.4 UNSAT-H

UNSAT-H is a one-dimensional finite-difference water balance model developed at Pacific Northwest Laboratory (Fayer and Jones, 1990) to assess the water dynamics of waste disposal facilities at the U.S. Department of Energy (DOE) Hanford site. The model also simulates soil heat flow and nonisothermal vapor flow. Vapor flow can be an important transport mechanism in near surface soils at arid sites. The UNSAT-H model was derived from the UNSAT model of Gupta et al. (1978) and has retained many of the same routines. At the time of this writing, Version 3.0 of UNSAT-H was the most current. The code can be obtained from the Energy Science and Technology Software Center, Department of Energy, Oak Ridge, Tennessee.

The UNSAT-H model considers precipitation, runoff, ET, soil water storage, and percolation in the water balance. Like the LEACHM model, infiltration of water into, and vertical movement of moisture in, the soil profile is governed in the UNSAT-H model by a finite difference solution to Richards' partial differential equation. However, the unsaturated soil hydraulic conductivity term in the UNSAT-H model is calculated using polynomials, Haverkamp functions, Brooks-Corey functions, or van Genuchten functions rather than the Campbell equation. Precipitation in excess of the infiltration capacity of the soil is shed as runoff. Evaporation and transpiration are considered separately.

Evaporation in the UNSAT-H model is calculated using one of two approaches: (i) an integrated form of Fick's law of diffusion that considers the flow of heat to and from the soil surface, the flow of water from the subsurface to the soil surface, and the transfer of water vapor from the soil surface to the atmosphere; or (ii) a Penman-type equation that is a modification of the diffusion equation and is dependent on net radiation and soil heat flux rather than on soil-surface temperature. Transpiration is calculated using a method based on leaf-area index or cheatgrass data and is limited by PET.

The UNSAT-H model requires that climatic data, soil properties, vegetation data, and initial and boundary conditions be input. There are no default data; the user must specify each input parameter. Required data for the meteorological data option are daily precipitation, daily maximum and minimum air temperatures, daily solar radiation, average daily dew point, and average daily wind speed. Daily precipitation and PET may be input instead of daily meteorological data. The precipitation option allows rainfall data for short, intense storms to be input. Required soil data are fitting parameters for the soil water characteristic functions and the unsaturated hydraulic conductivity functions. An option for including hysteresis is available. Vegetation data to be input include root depth, leaf area index, growing season, and percent bare area. Very little guidance is provided in the UNSAT-H model documentation on selection of values for the various input parameters.

The finite difference grid used by UNSAT-H is set up in a manner similar to that for LEACHM. The soil profile is divided into a number of horizontal layers with nodes located at the center of each layer. Two additional nodes, one above the ground surface and one below the profile being modeled, are used to set boundary conditions. The upper boundary condition can be changed with time by adjusting the head to simulate ponded or non-ponded infiltration, evaporation, or zero flux. The lower boundary condition can be selected as a fixed water table, free drainage (or unit gradient), zero flux, or specified flux boundary. The initial condition is specified by assigning an initial head or water content to each node in the finite-difference nodal grid. Simulation output includes infiltration, evaporation, transpiration, and percolation at hourly or daily intervals.

4.2.3.5 SoilCover

SoilCover model was developed in 1990 at the University of Saskatchewan for the analysis of the flow of water and heat between the atmosphere and the soil surface, particularly for land based disposal systems. Since then the model has been modified by Geo-Analysis 2000 Ltd., Saskatoon, Canada to include oxygen diffusion, an enhanced vegetation routine, freeze/thaw considerations, and soil property function revisions. SoilCover Version 5.2 was the most recent release at the time of this writing. The code and accompanying user's manual is available for download from http://www.members.shaw.ca/geo2000/page12.html.

SoilCover uses a finite-element method to solve the one-dimensional heat and mass transfer partial differential equations derived by Wilson (1990). The mass transfer equation is obtained by combining the differential forms of Darcy's law and Fick's law for unsteady vertical flow with the one-dimensional differential form of the conservation of mass equation. Both liquid flow and nonisothermal vapor flow are incorporated into the model. There is no option for isothermal vapor flow, nor is there an option for shutting off vapor flow altogether like is available with UNSAT-H. The unsaturated hydraulic conductivity function in the SoilCover model may be either user-defined (i.e., tabulated data) or predicted based on a Fredlund-Xing curve (Fredlund and Xing, 1994) fit to the water content versus matric potential data. The method used to predict the unsaturated hydraulic conductivity function was developed by Fredlund et al. (1994), and, according to SoilCover (2000), is especially well-suited for modeling fine-grained soils. Precipitation, runoff, ET, soil water storage, and percolation are included in the water balance.

SoilCover calculates evaporation using a modified Penman equation developed by Wilson (1990):

$$E_v = \frac{\Gamma R_n + \nu\left[0.35(1+0.15U_a)P_a\left(\frac{1}{h_a}-\frac{1}{h_r}\right)\right]}{\Gamma + \frac{\nu}{h_r}} \qquad \text{(Eq. 4.12)}$$

where: E_v = vertical evaporative flux (mm/day); Γ = slope of the saturation vapor pressure versus temperature curve at the mean temperature of the air (dimensionless); R_n = net radiant energy available at the surface (mm/day); ν = psychrometric constant (dimensionless); U_a= wind speed (km/hr); P_a = vapor pressure in the air above the evaporating surface (Pa); h_a = relative humidity of the air (dimensionless); and h_r = relative humidity at the soil surface (dimensionless). The model also offers the option of calculating evaporation based on user-input PET, in which case it uses the following equation:

$$E_v = PET\left[\frac{(h_r - h_a)}{(1-h_a)}\right] \qquad \text{(Eq. 4.13)}$$

where all terms are as defined previously. Unlike the other models described in this report, SoilCover calculates evaporation as a sink term directly from the surface relative humidity, which is a function of the matric suction and the temperature at the soil surface. The developers of SoilCover claim this method of calculating evaporation is a strength of the model.

Runoff is calculated as any precipitation that cannot infiltrate. Transpiration is calculated by applying fluxes at nodes in the root zone. Plant water stress and canopy shading effects are also considered by SoilCover.

The SoilCover model requires that climatic data, soil properties, vegetation data, and initial and boundary conditions be input Required climatic data include daily maximum and minimum air temperature, daily net radiation, daily maximum and minimum relative humidity, and daily wind speed. If the option for entering daily PET is chosen, then daily net radiation and wind speed are not required. Precipitation is entered on a daily basis as a constant flux top boundary condition, but intensity may be accounted for by constraining the precipitation between specified hours. Climatic data input is relatively easy because of the SoilCover's Microsoft Excel user interface. Daily data may be copied from a spreadsheet source and pasted directly into SoilCover.

Properties for up to eight soils may be entered. Required soil properties are porosity, specific gravity, saturated hydraulic conductivity, and coefficient of volume change. In addition, up to 20 water content vs. suction data points may be input. SoilCover then fits the Fredlund-Xing (1994) soil-water characteristic function to the data points. The unsaturated hydraulic conductivity function, the thermal conductivity function, and the volumetric specific heat function can then be generated using the fit soil-water characteristic function. The user may also choose to enter tabulated data for these functions. Very little guidance is provided in the SoilCover user's manual on selection of values for the various input parameters, however a short list of coefficients of volume change for typical soils is provided. Required input parameters for vegetation include growing season start and stop day, moisture wilting and limiting points, daily depths to top and bottom of roots, and selection of either poor, good, or excellent grass quality

The bottom boundary condition may be specified as either constant pressure or constant water content. There is no option for constant flux, constant gradient, or seepage face lower boundary conditions. The sparse lower boundary condition options necessitate that the user pay very close attention to the bottom boundary fluxes throughout the duration of the simulation to ensure that a realistic boundary is being modeled. For many landfill cover simulations, including a coarse-grained soil beneath the soil profile and adjusting the value of the bottom boundary condition is necessary to avoid "wicking" water from the boundary condition itself. If a gravel layer is added below the profile, percolation results may be obtained by utilizing the SoilCover option of cumulating fluxes at a selected internal node. The bottom temperature boundary condition must also be specified on a daily basis.

The finite element mesh is generated by SoilCover from input depths and thickness of the soil layers. Maximum and minimum node spacing for each layer must be specified along with the node spacing expansion factor. Only 100 nodes are permitted, so spacing and expansion factors may need to be adjusted. Initial conditions (either water content or suction) are also assigned to each node based on the initial conditions input for the top and bottom of each layer. SoilCover linearly interpolates the initial conditions. However, the assigned initial conditions may be overwritten by the user after the mesh has been generated. Simulation output includes infiltration, evaporation, transpiration, and percolation at daily intervals.

4.2.3.6 HYDRUS-2D

HYDRUS-2D is a two-dimensional unsaturated flow model developed at the U.S. Salinity Laboratory (Šimůnek et al., 1999). The model also simulates heat flow and solute transport. The current model is an extension of the earlier unsaturated flow codes SWMS_2D and CHAIN_2D. At the time of this writing version 2.02 of HYDRUS-2D was the most current. The model may be purchased from the International Ground Water Modeling Center, Colorado School of Mines, Golden, Colorado or http://www.Mines.EDU/research/igwmc/software/igwmcsoft/. The documentation and a free demo version of HYDRUS-2D may be downloaded from http://www.ussl.ars.usda.gov/models/hydrus2d/htm.

HYDRUS-2D uses a finite element method to solve Richards' equation in a plane oriented either vertically or horizontally. The two-dimensional domain may take on any geometric shape. Because the model is two-dimensional, lateral flow and anisotropy may be simulated. A sink term is included in Richards' equation for removal of water via plant transpiration. Vapor flow cannot be simulated. The model has an option for allowing soil properties to be temperature dependent, and it also allows hysteresis and spatial variability through a scaling transformation. The unsaturated hydraulic conductivity is calculated by either a Brooks-Corey, van Genuchten-Mualem, or modified van Genuchten method. Precipitation, runoff, ET, soil water storage, and percolation are included in the water balance.

Precipitation and potential evaporation are the only climatic inputs required. HYDRUS-2D does not have an option for internally calculating potential evaporation, so the user must use another model or method to generate data to input. Vegetation parameters required include the heads between which transpiration occurs and also the heads between which transpiration is optimal. A menu containing a variety of properties for plants is available. The distribution of roots must also be specified. Input required for soil properties includes saturated hydraulic conductivity and fitting parameters from the selected soil-water retention function. A menu of soil properties is available. In addition, van Genuchten properties can be predicted by inputting the percentage of sand, silt and clay, density, field capacity, and/or wilting point water content. HYDRUS-2D also has the option for inverse estimation of soil hydraulic properties from measured flow data.

The two-dimensional profile is created through a pre-processing module called Meshgen2D within the HYDRUS-2D graphical user interface. After the domain geometry is defined, Meshgen2D assists in generating the finite element mesh.

Boundary conditions may be specified flux, specified pressure head, unit gradient, atmospheric, seepage face, or deep drainage. Precipitation and potential evaporation are specified using the atmospheric option, which allows the boundary condition at the soil surface to change from either prescribed flux or prescribed head. The user inputs the upper and lower limits of head for which the prescribed flux boundary operates. Therefore, evaporation and precipitation will proceed at the potential rate until the soil surface dries or wets to a specified head. Once below the specified head, the boundary changes to a prescribed head boundary condition, and evaporation is limited by the ability of water to flow to the surface. If the surface becomes saturated during precipitation, excess precipitation is removed as runoff. The seepage face option allows water to exit the domain when the soil adjacent to the boundary becomes saturated. Deep drainage provides an option for a variable flux depending on the level of the groundwater table. Initial conditions may be specified as either water contents or pressure heads.

The HYDRUS-2D post-processor allows a variety of options for viewing output. Results can be displayed graphically, including an animation of changes in pressure head or water content through time. Cross-sections plotting pressure head or water content vs. depth or length may be taken from the profile at any time of the simulation. Other output options include viewing the instantaneous or cumulative water boundary fluxes over time, run time information, graphical display of soil hydraulic properties, or converting output to ASCII format.

4.3 Evaluation of Water Balance Models

4.3.1 Overview

A number of researchers have performed field studies and analytical assessments to evaluate the HELP, LEACHM,UNSAT-H, SoilCover, and HYDRUS-2D models (Thompson and Tyler, 1984; Peters et al., 1986; Barnes and Rodgers, 1988; Peyton and Schroeder, 1988; Nyhan, 1989; Wilson, 1990; Nichols, 1991; Udoh, 1991; Fayer et al., 1992; Lane et al., 1992; Benson et al., 1993; Peyton and Schroeder, 1993; Martian, 1994; Tratch, 1994; Fleenor and King, 1995; Khire, 1995; Khire et al., 1997; Webb et al., 1997; Zornberg and Caldwell, 1998; Scanlon et al., 2002). These studies were used to either simulate field or laboratory water balance data or to investigate trends and magnitudes of the different water balance components (i.e., infiltration, runoff, etc.). The conclusions of these studies are not always in general agreement. For example, some studies found that a certain model overpredicted or underpredicted infiltration or percolation in a certain climate, whereas, other studies using the same model concluded just the opposite. In many of the comparisons between measured and calculated water balances, site-specific field data were used in the water balance predictions. However, in the current state of practice for the majority of projects, measurement of site-specific parameters required for the models, such as soil field capacity, wilting point, and evaporation depth or rooting depth, is not performed. Thus, the model user is left to depend on default data, which may lead to an inaccurate representation of a site. At present, these hydrologic models should be used carefully to ensure a conservative and reasonable basis for design. As a true predictive tool, the value of the models is limited unless site-specific calibrations are performed. The results of a few of the more significant field studies are presented below.

4.3.2 Lysimeters at DOE Hanford Site

Fayer et al. (1992) compared field water balances for eight unvegetated lysimeters at DOE's Hanford site to water balances simulated using the UNSAT-H, Version 2 model. The Hanford site is located about 35 km northwest of Richland, Washington, in the northern cold desert of the Columbia Basin. Average annual rainfall at the site is only 162 mm and average potential evaporation is 1,600 mm (Gee et al., 1994). On average, over 70% of precipitation falls during October through April. The soil profile in the lysimeters and the simplified profile used for simulations are shown in Figure 4-3. The uppermost soil in the lysimeters is a silt loam material. The soil profile in the lysimeters is intended to simulate a capillary barrier.

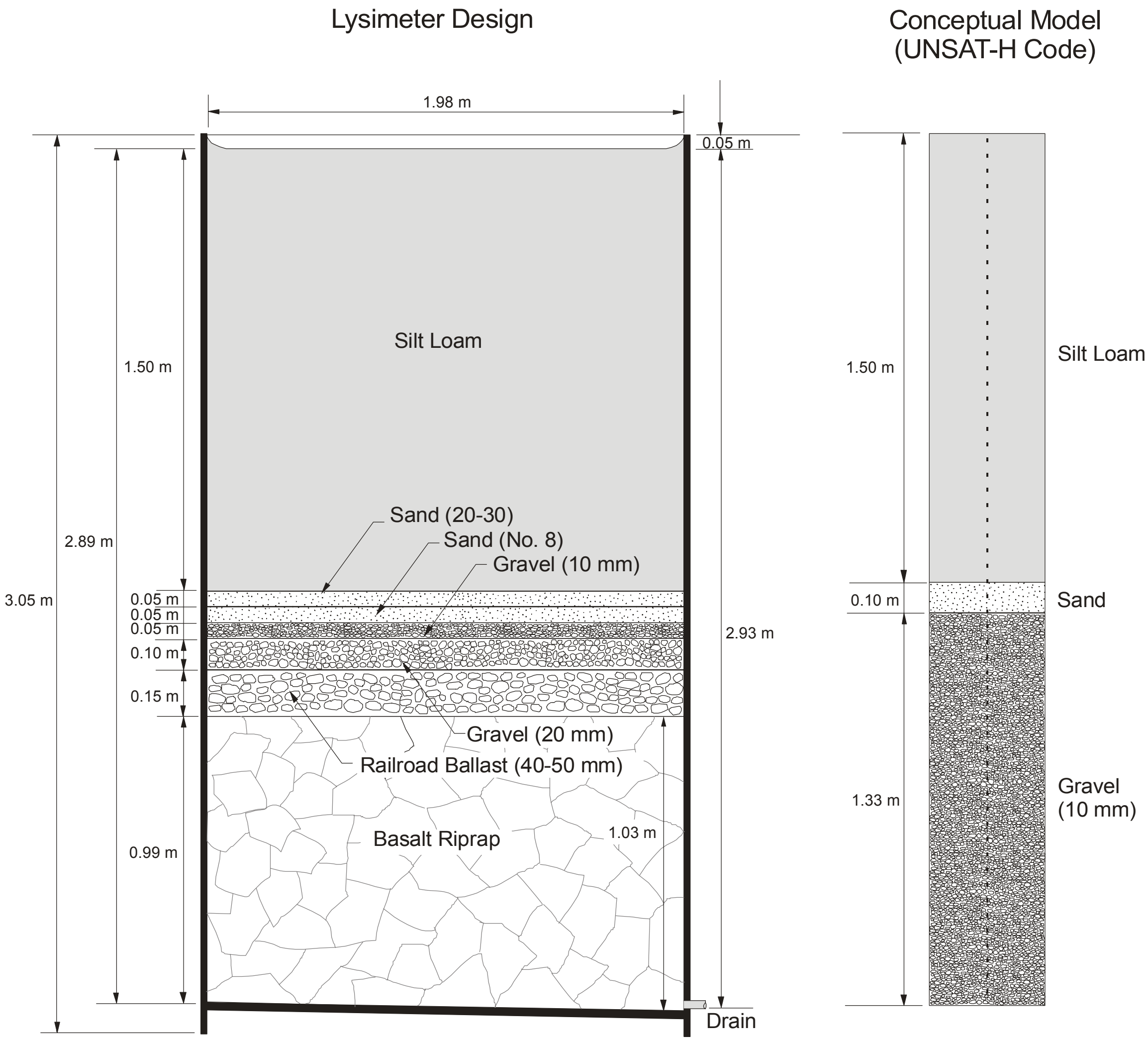


Figure 4-3. Lysimeter design and conceptual model used to compare measured and simulated water balance for DOE Hanford site (from Fayer et al., 1992).

Of the eight lysimeters constructed by Fayer et al. (1992), six were drainage lysimeters and two were weighing lysimeters. The drainage lysimeters comprised two replicates of three precipitation treatments: (i) ambient; (ii) two times the average annual precipitation; and (iii) breakthrough (i.e., water added until drainage occurred). The weighing lysimeters served as additional replicates, with one of the lysimeters receiving the normal precipitation and the other receiving two times the average annual precipitation. Soil water content and percolation data were collected for the lysimeters from November 1987 to April 1989.

The field water balances for the lysimeters were compared to water balance simulations performed using UNSAT-H. The simulations were performed with actual weather data from a nearby meteorological station, measured soil properties data for the silt loam, and assumed properties for the sand and gravel layers beneath the silt loam. The lower boundary of the drainage lysimeters was modeled as a unit gradient and the lower boundary of the weighing lysimeters was represented as a zero-flux condition. The upper boundary condition was allowed to vary depending on climatic conditions.

Measured and simulated water contents for the drainage lysimeters under the three precipitation conditions are shown in Figures 4-4 to 4-6. Measurable percolation only drained from the lysimeters with the "breakthrough" precipitation treatment. In general, the simulated soil water profiles showed reasonable agreement with measured water contents. However, UNSAT-H tended to underestimate somewhat the amount of soil water storage during the spring and overestimate the amount of soil water storage during the winter. Fayer et al. (1992) attributed this discrepancy primarily to the underestimation of evaporation in the winter and the overestimation of evaporation in the summer. This effect is also apparent in the plot of measured and simulated soil water storage in Figure 4-7(a). By decreasing evaporation, increasing the saturated hydraulic conductivity of the silt loam, and adding a snow cover, simulated soil water storage shows better agreement with measured soil water storage (Figure 4-7 (b)).

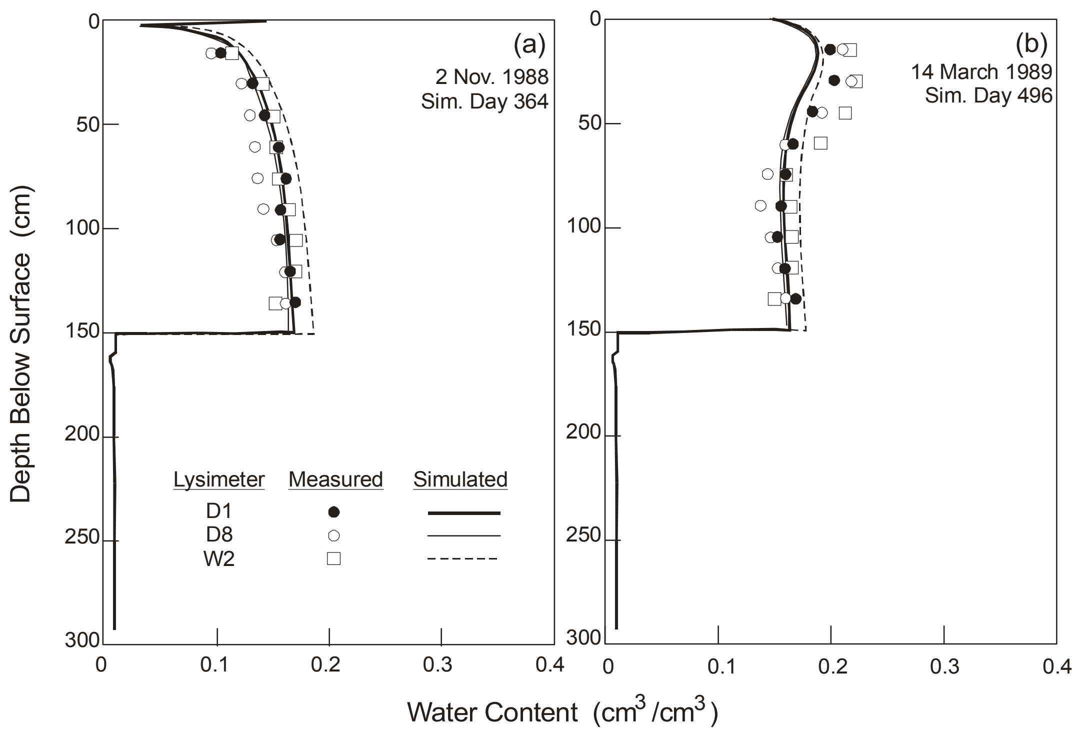


Figure 4-4. Measured and simulated (UNSAT-H) water contents for the ambient precipitation treatment at DOE Hanford lysimeters (from Fayer et al., 1992).

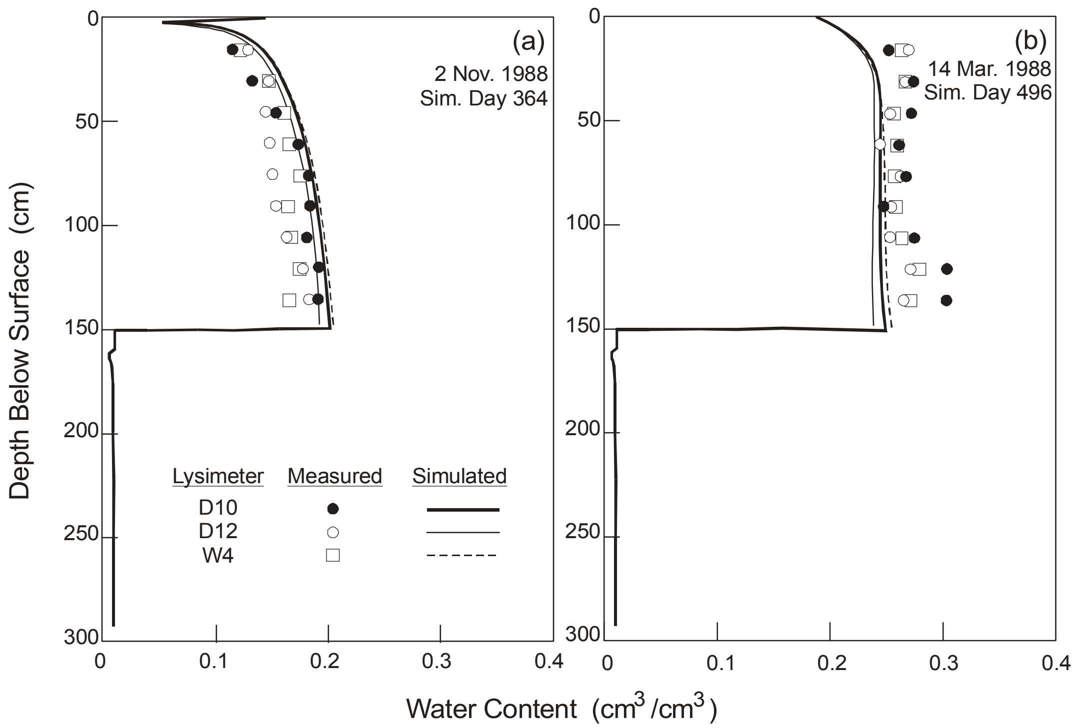


Figure 4-5. Measured and simulated (UNSAT-H) water contents for the 2x average precipitation treatment at DOE Hanford lysimeters (from Fayer et al., 1992).

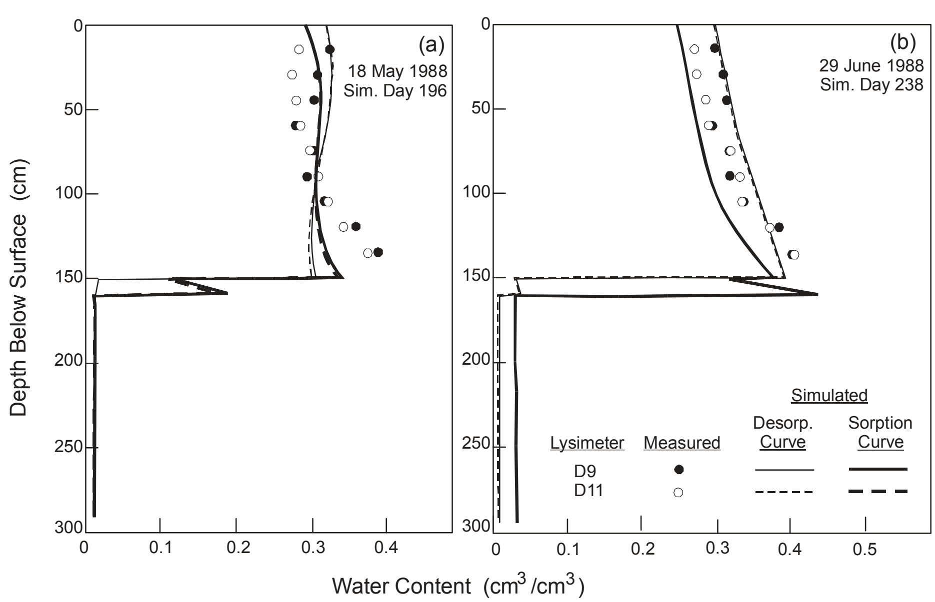


Figure 4-6. Measured and simulated (UNSAT-H) water contents for the breakthrough precipitation treatment at DOE Hanford lysimeters (from Fayer et al., 1992).

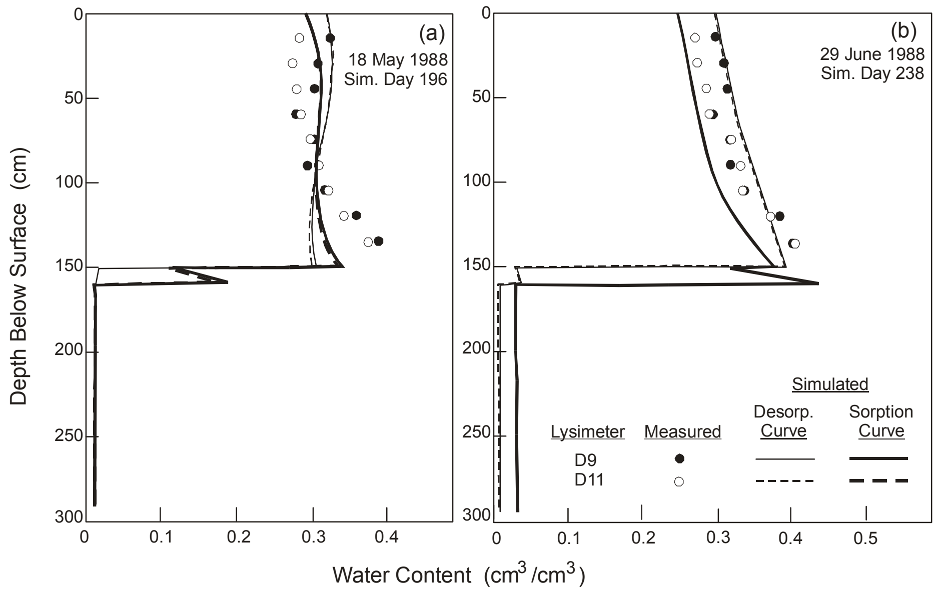


Figure 4-7. Measured and simulated (UNSAT-H) water storage for the 2x average precipitation treatment at DOE Hanford lysimeters: (a) initial simulation; (b) simulation with improved calibration (from Fayer et al., 1992).

4.3.3 Test Plots at Hill Air Force Base

Paige et al. (1996) described calibrating Version 2 of the HELP model to field measurements from two cover system test plots constructed at Hill Air Force Base (Hill AFB), in Layton, Utah and monitored for a four-year period. The calibrated models were then used to simulate the long-term performance of the cover systems. One test plot had an ET-type soil cover ("control soil cover") consisting of a 0.9-m thick sandy loam topsoil layer. The other test plot had a cover system consisting of the following components, from top to bottom: 1.2-m thick sandy loam topsoil layer; 0.3-m thick sand lateral drainage layer; and 0.6-m thick CCL. Both cover systems were constructed over a 0.3-m thick gravel layer with lysimeters so that percolation could be monitored. Cross sections of the cover systems are shown in Figure 4-8. After construction, the plots were vegetated with native grasses. Water balance data measured over the four-year monitoring period include precipitation, lateral flow in the sand drainage layer, percolation, soil moisture content, and runoff.

Using the HELP model default values for the ET-type cover, HELP overpredicted annual ET by approximately 30% and underpredicted annual percolation by approximately 95%. For the hydraulic barrier-type cover, ET was overpredicted by 48%, runoff was overpredicted by 150%, and lateral drainage was underpredicted by 97% when the HELP model was run with default values. The HELP model was subsequently calibrated to the field water balances primarily by modifying the soil properties of the cover systems (e.g., saturated hydraulic conductivity, soil water storage capacity). The measured and calibrated values of the water balances for the ET-type cover system and the hydraulic barrier-type cover system are shown in Tables 4-4 and 4-5, respectively. As can be seen from these tables, even with the site-specific calibration, significant differences between field and simulated water balance components occurred. In particular, for the ET cover system, correlation between measured and predicted percolation was not good.

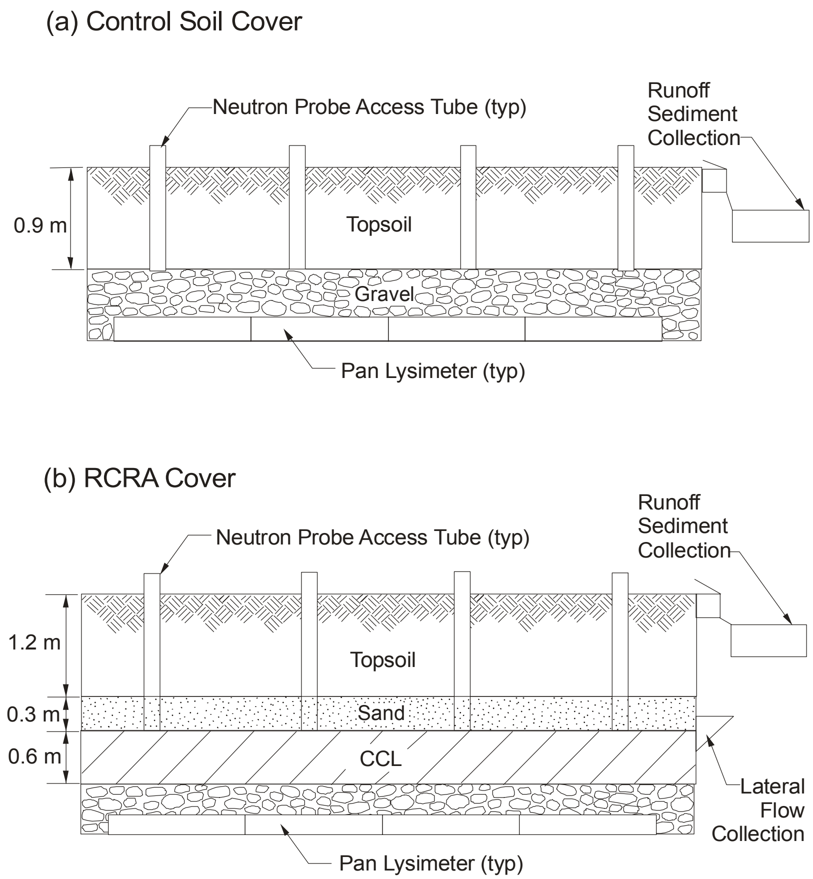


Figure 4-8. Hill Air Force Base test plots: (a) ET-type cover system; and (b) hydraulic barrier-type soil cover system (from Paige et al., 1996).

Table 4-4. Difference between measured annual values and HELP simulation values for the control ET-type cover system at Hill AFB (modified from Paige et al., 1996). Results obtained using input parameters calibrated from site water balance data.

Water Balance Variable	Measured		HELP Predicted		Difference	
	(cm)	(% meas. precip.)	(cm)	(% pred. precip.)	(cm)	(% meas. precip.)
1991						
Precip.	53.72	100.00	53.70	100.00	-	-
Runoff	1.50	2.79	1.14	2.14	0.36	0.67
Perc.	9.09	16.93	17.09	31.84	-8.00	-14.90
ET	34.70	64.58	34.64	64.53	0.06	0.11
Soil water[1]	8.43	15.70	0.81	1.49	7.62	14.19
1992						
Precip.	39.09	100.00	39.26	100.00	-	-
Runoff	0.10	0.26	0.25	0.63	-0.15	-0.38
Perc.	5.79	14.81	10.84	27.62	-5.05	-12.92
ET	33.30	85.18	28.47	72.50	4.83	12.36
Soil water	-0.10	-0.26	-0.30	-0.75	0.20	0.51
1993						
Precip.	41.78	100.00	41.85	100.00	-	-
Runoff	0.25	0.61	0.61	1.49	-0.36	-0.86
Perc.	23.80	56.96	10.49	25.08	13.31	31.86
ET	30.66	73.37	32.18	76.88	-1.52	-3.64
Soil water	12.93	-30.94	-1.44	-3.44	14.37	34.39

[1] Change in soil water storage.

Table 4-5. Difference between measured annual values and HELP simulation values for the control soil cover system at Hill AFB (modified from Paige et al., 1996). Results obtained using input parameters calibrated from site water balance data.

Water Balance Variable	Measured		HELP Predicted		Difference	
	(cm)	(% meas. precip.)	(cm)	(% pred. precip.)	(cm)	(% meas. precip.)
1991						
Precip.	53.72	100.00	53.70	100.00	-	-
Runoff	1.14	2.13	0.84	1.57	0.30	0.56
Lat. Drain	19.00	35.37	17.25	32.11	1.75	3.26
Perc.	0.00	0.00	0.28	0.51	-0.28	-0.52
ET	24.59	45.77	34.36	63.98	-9.77	-18.19
Soil water[1]	8.99	16.73	0.99	1.84	8.00	14.89
1992						
Precip.	39.09	100.00	39.26	100.00	-	-
Runoff	0.05	0.13	0.13	0.32	-0.08	-0.20
Lat. Drain	6.70	17.15	11.23	28.60	-4.53	-11.59
Perc.	0.00	0.00	0.28	0.69	-0.28	-0.72
ET	30.12	77.06	27.86	70.99	2.26	5.78
Soil water	2.21	5.65	-0.22	-0.59	2.43	6.22
1993						
Precip.	41.78	100.00	41.85	100.00	-	
Runoff	0.71	1.70	0.43	1.02	0.28	0.67
Lat. Drain	23.32	55.80	11.10	26.53	12.22	29.25
Perc.	0.00	0.00	0.28	0.64	-0.28	-0.67
ET	27.94	66.87	31.80	75.96	-3.86	-9.24
Soil water	-10.18	-24.37	-1.73	-4.16	-11.91	-28.51

[1] Change in soil water storage.

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

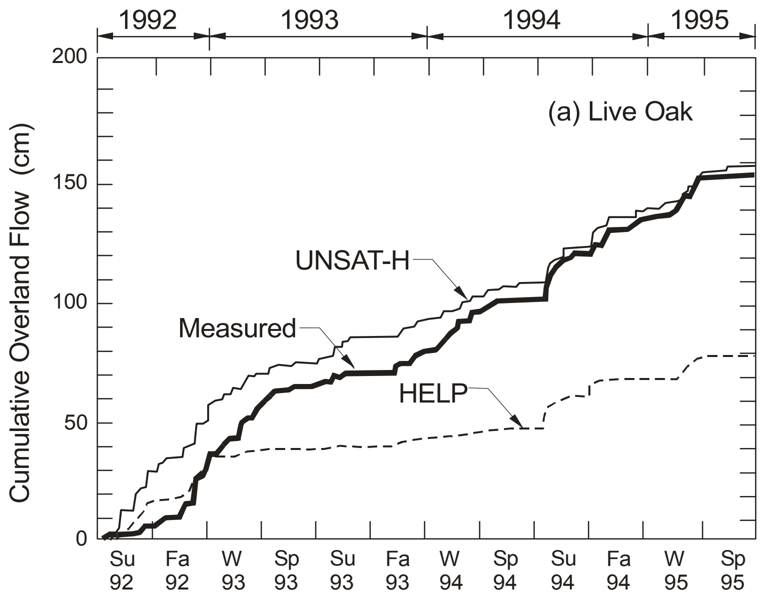
Of all the available studies, the one reported by Lane (1992), Khire (1995), and Khire et al. (1997, 1999) is perhaps most interesting because of the scope and practical applicability of the study to cover system analysis and design. The study involves field water balance evaluations for three 30 m x 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 0.6-m thick CCL overlain by a 0.15-m thick vegetated 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 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 2002. Runoff and percolation is collected in tanks and measured, while soil moisture content is measured by time domain reflectrometry.

Khire (1995) and Khire et al. (1997) used their test plot data to assess the predictive capabilities of the HELP and UNSAT-H models. The models were assessed by comparing model predictions to measured hydrologic data for the three cover system configurations. The predictions were performed using climatic data and laboratory-measured soil properties. Input parameters that were not measured were estimated from published information. The input parameters for this study were better defined than for most actual design projects. The UNSAT-H predictions were conducted with a unit gradient lower boundary condition and a specified flux upper boundary condition. Khire (1995) and Khire et al. (1997) drew the following conclusions from their study:

- Properly simulating runoff is essential because the fraction of precipitation that is not shed enters the cover system and may ultimately become percolation. Throughout most of the monitoring period, HELP underpredicted runoff for the humid Live Oak site (Figure 4-9) and overpredicted runoff for the semi-arid Wenatchee site with a CCL (Figure 4-10). Overall, HELP underpredicted runoff by 740 mm (≈ 90%) for the Live Oak site and overpredicted it by 30 mm (≈ 30%) for the Wenatchee site. Cumulative runoff predictions made using UNSAT-H were reasonably accurate for the Live Oak site (i.e., less than 3% error); however, season-to-season differences in runoff amounts were significant. For the Wenatchee site, UNSAT-H underpredicted runoff by 50 mm (≈ 270%) for the plot with a CCL and predicted no runoff for the plot with a capillary barrier. The underpredictions resulted in more water entering the soil in the simulations than in the field. This resulted in higher soil water storage in the simulations than in the field.
- Although HELP predicted ET fairly accurately for the Live Oak site, it was underpredicted by only 70 mm (≈ 4%), an accurate prediction of ET was not expected given that more water entered soil due to the underprediction of runoff. Instead, an overprediction of ET was expected unless the PET demand had already been met.

UNSAT-H underpredicted ET for the Live Oak site by 300 mm (≈ 15%). Examination of the water-balance equation indicates that underpredicting runoff and fairly accurately predicting ET, or vice versa, results in an overprediction of soil water storage and/or percolation. Both HELP and UNSAT-H overestimated ET at the Wenatchee sites by about 20 to 165 mm (≈ 20 to 40%).

- HELP somewhat captured the trends in percolation at the Live Oak site, but overpredicted total percolation by more than 700 mm (≈ 300%) (Figure 4-11).



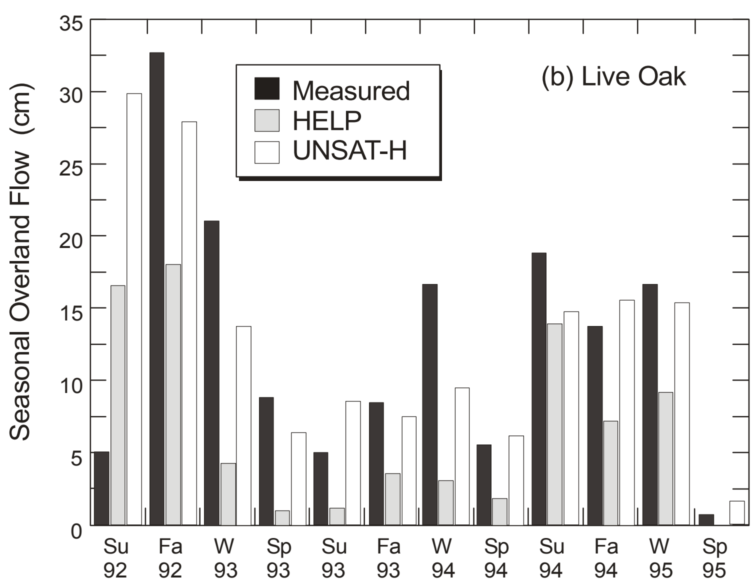


Figure 4-9. Measured and predicted cover system runoff at Live Oak site: (a) cumulative; and (b) seasonal (from Khire, 1995).

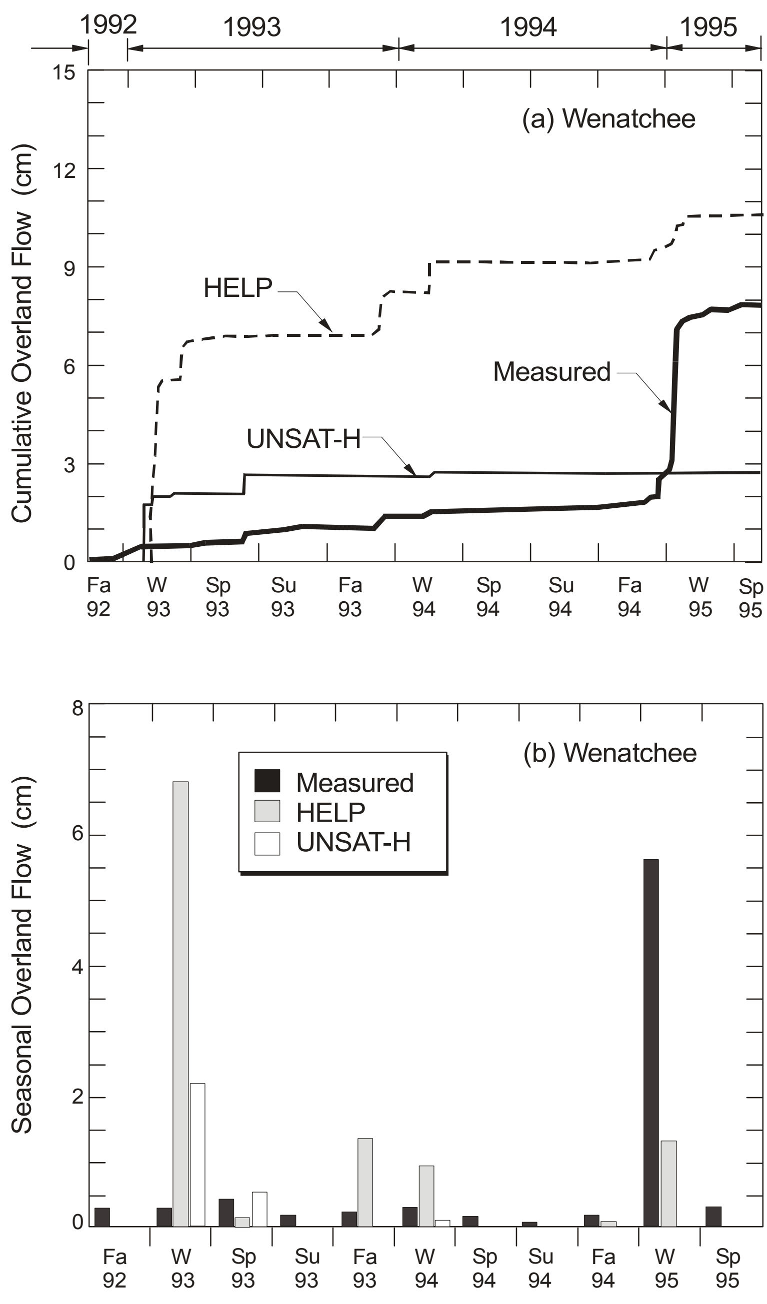


Figure 4-10. Measured and predicted runoff for hydraulic barrier-type cover system at Wenatchee site: (a) cumulative; and (b) seasonal (from Khire, 1995).

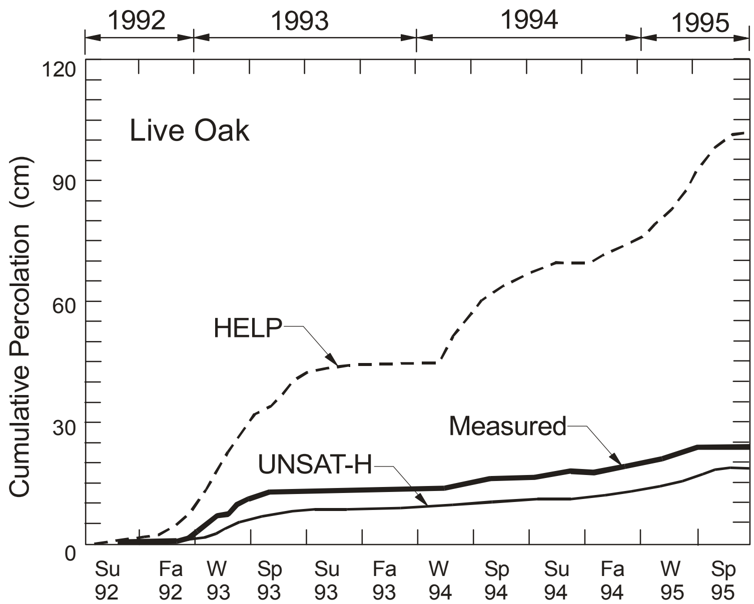


Figure 4-11. Measured and predicted cover system percolation at Live Oak site (from Khire, 1995).

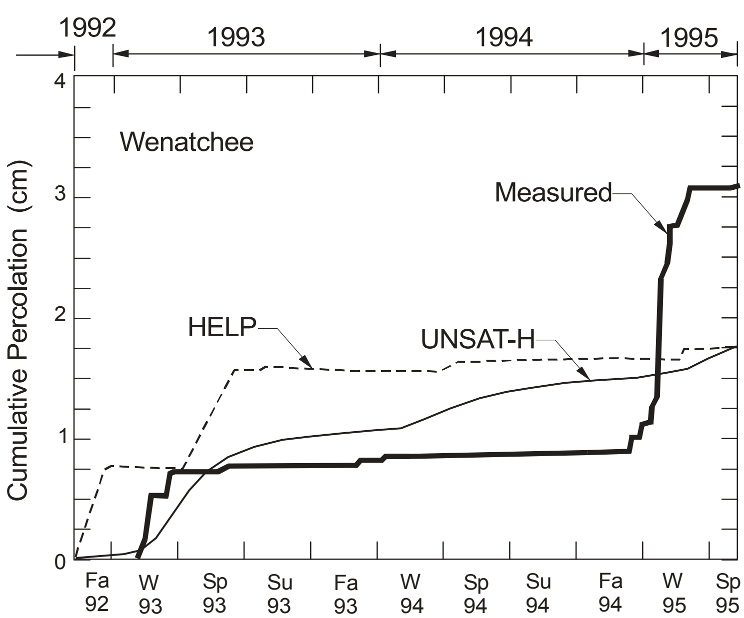


Figure 4-12. Measured and predicted percolation for hydraulic barrier-type cover system at Wenatchee site (from Khire, 1995).

One reason why percolation was overpredicted is that there was additional water in the soil caused by the underprediction of runoff. Another factor that contributed to the overprediction of percolation is the unit hydraulic gradient used by HELP to model unsaturated vertical flow. HELP assumes that water in the soil flows vertically downward under a unit hydraulic gradient (i.e., hydraulic gradient = 1). Khire (1995) and Khire et al. (1997) indicate that the hydraulic gradient in the field rarely equaled "1" and, for most of the time, was oriented vertically upward. UNSAT-H underpredicted percolation for the Live Oak site only slightly, by about 60 mm. Both HELP and UNSAT-H underpredicted percolation for the Wenatchee site with a CCL barrier (Figure 4-12). However, at least part of this difference is believed to have been caused by the preferential flow of water and snow melt through cracks and animal burrows in the winter of 1995. Prior to that time, both models had overpredicted percolation. UNSAT-H significantly overpredicted percolation for the Wenatchee site with a capillary barrier (Figure 4-13). One reason why percolation was overpredicted by over 90 mm (≈ 2,000%) is that there was additional water in the soil caused by the underprediction of runoff.

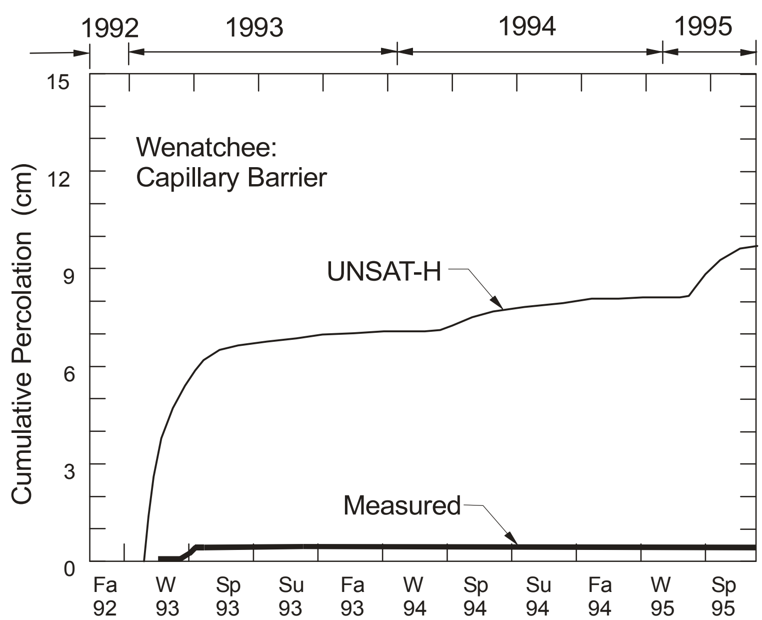


Figure 4-13. Measured and predicted percolation for capillary barrier-type cover system at Wenatchee site (from Khire, 1995).

4.4 Recommendations for Application of Water Balance Models

The specific water balance analysis method and input parameters to use for analysis and design of a cover system should be selected based on the purpose of the analysis and project-specific factors such as climate, type of cover (i.e., hydraulic barrier, ET barrier, or capillary barrier), and cover system components. Given the inconsistencies in water balance analysis results (e.g., the models sometimes overpredict and sometimes underpredict the various components of the water balance), uncertainties in soil properties and long-term barrier integrity (e.g., CCL hydraulic conductivity may increase over time if the CCL is not adequately protected), and other factors, significant engineering judgment must be applied when performing a water balance analysis for a specific site. The following general recommendations are made regarding the use of water balance methods for the design of cover systems:

- Percolation rates through cover systems with GM, GM/CCL, or GM/GCL hydraulic barriers should be very low when these barriers are properly constructed due to the effectiveness of these barrier types in preventing water migration through the barrier. Both the simplified manual method and the HELP model are well suited to performing analyses to demonstrate the effectiveness of these type of barriers in minimizing percolation.
- Estimated percolation rates through hydraulic barriers layers containing GMs for various categories of annual rainfall were provided by Gross et al. (1997) (Table 4-6). These estimates can be used by design engineers as a check of percolation rates calculated on a project-specific basis using either the simplified manual method or the HELP model. Percolation rates were calculated by Gross et al. (1997) using the HELP model with synthetic rainfall data generated by the model for several different cities in each rainfall category and the following ranges of input parameters: (i) fair grass vegetation; (ii) sandy loam and silty clay loam topsoil; (iii) 5 and 20% cover system slopes; (iv) coarse sand and GN internal drainage layers; and (v) 10-year synthetic weather records.

Table 4-6. Percolation Rates through Cover Systems with Barriers Incorporating GMs Estimated Using the HELP Model (from Gross et al., 1997).

Average Annual Rainfall (mm)	Average Percolation Rates (mm/yr)	
	GM Barrier	GM/CCL or GM/GCL Barrier
100-300	0-0.05	0-0.005
300-600	0.002-0.3	0.0002-0.03
600-800	0.1-1	0.01-0.1
800-1,000	0.3-2	0.03-0.2
1,000-1,600	1-5	0.1-0.5

- Either the simplified manual method or the HELP model can be used for the design of internal drainage layers underlain by hydraulic barriers containing a GM. A discussion of the design storm to use with each method is given below.

- Neither the simplified manual method nor HELP are capable of serving as a water balance predictive tool using estimated or default input data. The HELP model has limited capability as a predictive tool when calibrated using site-specific data.

- Any of the water balance analysis methods may be used for evaluating percolation through cover systems with CCL or GCL hydraulic barriers. While methods that incorporate unsaturated flow models are potentially more accurate than methods where saturated conditions are assumed for flow through the hydraulic barrier, the latter methods (i.e., simplified manual method and HELP model) are easier to use. These latter methods are likely to overpredict actual percolation rates for humid sites.

- For capillary-barrier and ET-barrier cover systems, a water balance analysis method that can correctly model unsaturated flow is preferred. Thus, LEACHM, UNSAT-H, SoilCover, or HYDRUS-2D is preferable to the HELP model for evaluation of these types of systems.

- For cover systems in any climate that rely on enhanced ET to minimize percolation, methods that correctly model unsaturated flow and that allow different vegetation scenarios to be input, such as LEACHM, UNSAT-H, SoilCover, or HYDRUS-2D, are preferred.

- Reference should be made to the available technical literature for the best available information on the tendencies of the various water balance models to either underpredict or overpredict the various components of the water balance for both wet and arid climatic conditions. This information should be considered in interpreting the results of project-specific water balance analyses.

- Reference should be made to the technical literature for new models that may be developed in the future with enhanced capabilities for the performance of cover system water balance analyses. All of the available models have their strengths and weaknesses. There remains room for improvement of the models and their specific applications

- Due to the difficulty in performing accurate analytical water balances, field water balances should be performed, whenever possible, to verify the analytical results. This is especially the case for alternative cover systems.

- An important input parameter in the design of cover system internal drainage layers for hydraulic barrier cover systems is rainfall intensity and duration. As previously discussed, the HELP model is limited to using daily rainfall data, and this does not capture short-term intense peaks in storm events. Koerner and Daniel (1997) have suggested that hourly rainfall data be considered along with the simplified manual method to calculate percolation through the cover soil into the internal drainage layer (PERC*). They presented an example calculation of the sensitivity of PERC* to the use of monthly, daily, or hourly precipitation data. The example assumes a site near Austin, Texas, with a 200-m long 3H:1V slope and a surface runoff coefficient of 0.4. The results of their analysis were as follows:

  - PERC* = 0.011 mm/hr, using the simplified manual method with the average monthly temperature, duration of sunlight, and precipitation data from Austin;
  - PERC* = 1.3 mm/hr using the HELP model with historical daily precipitation data from 1974-1977 for San Antonio and all other climatic data generated for Austin; and
  - PERC* = 50 mm/hr using Eq. 4.7 with the probable maximum 6-hr precipitation event for the project vicinity (i.e., 500 mm).

- Koerner and Daniel (1997) noted that the calculated peak flow rate based on hourly storm data is more than one order of magnitude larger than the calculated peak flow based on daily precipitation values. Because of this, they recommended that hourly precipitation data be considered to conservatively calculate peak flow rates into the drainage layer and to determine if the drainage layer has adequate capacity to transmit the peak flow during extreme storm events.

For this guidance document, PERC* was calculated for the same example as above using the HELP model with climatic data generated synthetically for Austin for a 20-year simulation period. For the authors' simulation, PERC* = 3.1 mm/hr. This calculated PERC* is about 2.5 times larger than the value obtained by Koerner and Daniel (1997) of 1.3 mm/hr using the historical weather data for 1974-1977 for San Antonio. This result reinforces the comment made previously in this chapter that the HELP precipitation database for the period 1974-1977 reflects unusually dry weather for certain parts of the U.S. More generally, short-duration rainfall records may not contain wet weather cycles or intense storm events that control design. Also, as Koerner and Daniel (1997) noted, the rate of infiltration into a cover system soil will be limited by the hydraulic conductivity of the cover soil materials. If it is assumed in the above example that the cover soil has a saturated hydraulic conductivity of 1×10^{-6} m/s, then from Eq. 4.8, the maximum possible rate of infiltration into the cover for a non-ponded surface condition is 3.6 mm/hr , approximately the rate of percolation calculated with the HELP model and daily rainfall data generated synthetically for Austin, Texas, (i.e., 3.1 mm/hr). Thus, for typical cover systems with low to moderately permeable surface and protection layers, it will often be adequate to use the HELP model and a synthetic rainfall record with a sufficiently long simulation period (e.g., 20 years) to calculate lateral drainage and hydraulic head. Alternatively, Eq. 4.8b can be used directly to obtain a conservative value of PERC* for design.

4.5 Design of Drainage Layers

4.5.1 Simplified Manual Method

The required hydraulic properties of the cover system drainage layer are a function of the expected peak rate of percolation into the drainage layer (PERC* in Sections 4.2 and 4.3), the length of the cover system slope, the inclination of the cover system slope, and other factors.

Assuming no change in water storage in the drainage layer material, lateral flow in that layer is equal to percolation through the cover soil into the layer (PERC*) minus percolation through the hydraulic barrier (PERC). From Eq. 4.4:

$$L = PERC^* - PERC \qquad \text{(Eq. 4.14)}$$

where all terms are as defined previously. Assuming steady-state conditions, the maximum flow in the drainage layer is given by Eq. 4.5, repeated here:

$$q_m = \frac{\ell L}{8.64 \text{ x } 10^7} = \frac{\ell \left(PERC^* - PERC\right)}{8.64 \text{ x } 10^7} \qquad \text{(Eq. 4.5)}$$

where: q_m = maximum flow rate in drainage layer per unit width perpendicular to the direction of flow ($m^3/s/m$); ℓ = slope length (m); and other terms are as defined previously. For design of drainage layers, PERC can be conservatively assumed to be zero: that is, all percolation through the cover soil (PERC*) is assumed to become lateral flow in the drainage layer. For this case:

$$q_m = \frac{\ell \left(PERC^*\right)}{8.64 \text{ x } 10^7} \qquad \text{(Eq. 4.15)}$$

The hydraulic transmissivity of the drainage layer must be adequate to accommodate this flow. In the simplified manual method, the DuPuit-Forcheimer assumptions are used along with the further assumption that the line of seepage is parallel to the cover system slope to calculate the required drainage layer hydraulic transmissivity. For these assumptions, the hydraulic gradient is constant and equal to the sine of the slope angle:

$$i = (\sin\beta) \qquad \text{(Eq. 4.16)}$$

where β = slope angle (degrees). The required hydraulic transmissivity of the drainage layer is then obtained using Darcy's equation and the known values of q_m and i:

$$\theta_h = (q_m/i)\ FS \qquad \text{(Eq. 4.17)}$$

Substituting Eqs. 4.15 and 4.16 into Eq. 4.17 results in:

$$\theta_h = \frac{\ell\, PERC^*}{8.64 \text{ x } 10^7 \sin\beta} FS \qquad \text{(Eq. 4.18)}$$

where: θ_h = required hydraulic transmissivity of drainage layer (m^2/s); FS = factor of safety (dimensionless); and other terms are as defined previously. As previously discussed in Section 2.4.2.3, a minimum FS value of 2 is recommended for cases where the uncertainty in input parameters is low and the consequences of failure are small. For many situations, a larger FS may be appropriate. Koerner and Daniel (1997) have recommended using a FS value of at least 5 to 10 to account for uncertainities in the hydraulic conditions.

The maximum hydraulic head in the drainage layer for the assumptions given previously is:

$$h_m = \frac{q_m}{k_d \tan\beta} \qquad \text{(Eq. 4.19)}$$

where: h_m = maximum hydraulic head (m); k_d = hydraulic conductivity of drainage layer (m/s); and q_m is as defined previously. The maximum hydraulic head for this set of assumptions occurs at the base of the slope. The required thickness (measured perpendicular to the slope) of the internal drainage layer is obtained from the equation:

$$t_m = (h_m / \cos\beta)\ FS = \theta/k \qquad \text{(Eq. 4.20)}$$

where: t_m = the required thickness of the internal drainage layer (m); and other terms are as defined previously. The actual thickness of the internal drainage layer must be larger than t_m in order for pressure head not to build up in the layer. The definition of the thickness, head, and depth of flow on a slope is shown in Figure 4-14.

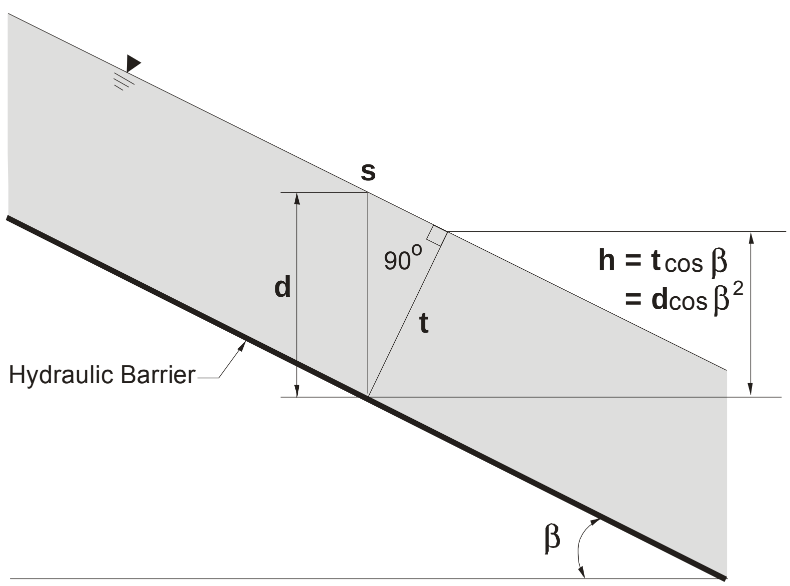


Figure 4-14. Definition of liquid depth (d), thickness (t), and hydraulic head (h), above a hydraulic barrier.

4.5.2 Refinement to Simplified Manual Method

For a sloping drainage layer receiving a constant rate of percolation (PERC*), flow in the layer is not actually parallel to the slope as assumed in the previous subsection. Rather, as the hydraulic head builds up on the slope, the phreatic surface takes on a curved shape. Figure 4-15 illustrates this condition for a cover system slope with a toe drain. For this condition, the hydraulic gradient is not constant but varies along the slope length.

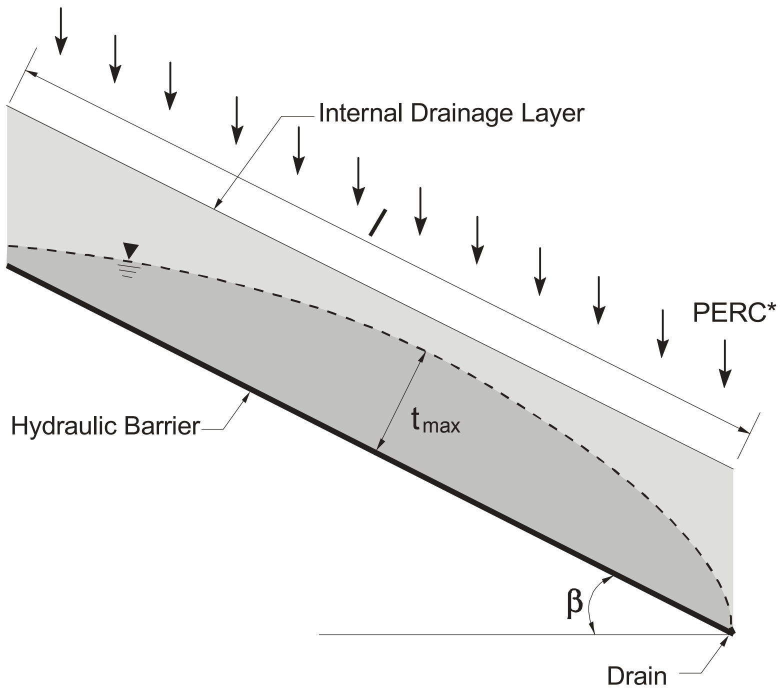


Figure 4-15. Hydraulic head distribution on a cover system slope with a toe drain.

An improved estimate of maximum hydraulic head in the internal drainage layer that takes account of the varying hydraulic gradient (while maintaining use of the DuPuit-Forcheimer assumptions) can be obtained using the equations from Giroud et al. (1992b) and Giroud and Houlihan (1995):

$$h_m = (j\ell\cos\beta/2)\left[\left(\tan^2\beta + \frac{4\ \mathrm{PERC}^*}{k\cos\beta}\right)^{1/2} - \tan\beta\right] \qquad \text{(Eq. 4.21)}$$

where all terms are as defined previously, and j is given by Eq. 4.21:

$$j = 1 - 0.12\exp\left[-\left(\log(8\lambda/5)^{5/8}\right)^2\right] \qquad \text{(Eq. 4.22)}$$

where:

$$\lambda = \frac{\text{PERC}^*}{k \tan\beta \sin\beta} \qquad \text{(Eq. 4.23)}$$

It is noted that Eq. 4.20 tends to the simplified solution of Eq. 4.18 when PERC*/k tends towards zero and/or β is very large. Values of average hydraulic head, h_{avg} (m), for a given value of h_m can be obtained from Figure 4-16. For the case of (PERC*/k cosβ) < 0.25 $\tan^2\beta$:

$$h_{avg} = \frac{\text{PERC}^* \ell}{2k \sin\beta \cos\beta} \qquad \text{(Eq. 4.24)}$$

It is suggested that for design of internal drainage layer, h_m be used from single storm event analyses to size the drainage layer. In contrast, it is suggested that h_{avg} be used to calculate long-term PERC values. For the simplified manual method, PERC* to calculate h_m should be derived using hourly water balance calculations for the design storm (limited by Eq. 4.8 as previously discussed) and PERC* to calculate h_{avg} should be derived using monthly water balance calculations.

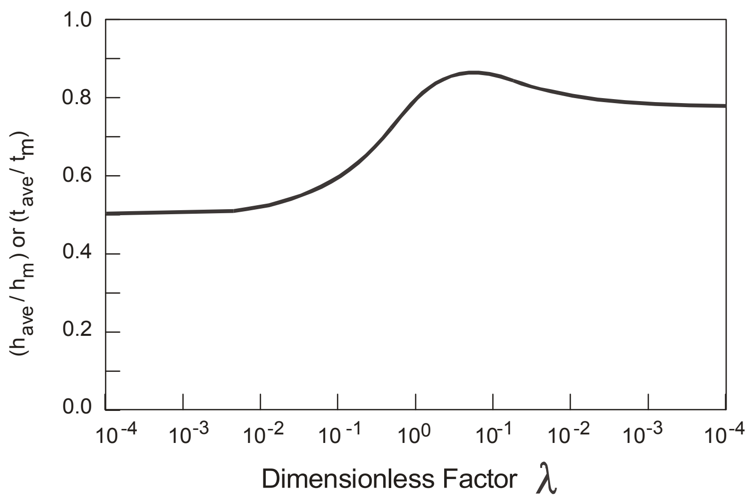


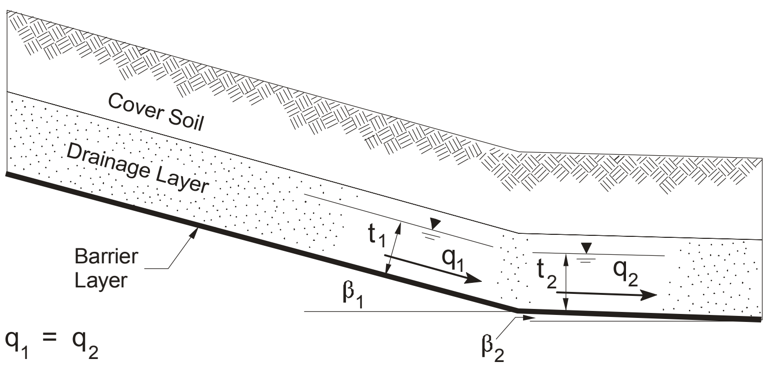
Figure 4-16. Dimensionless factor for calculating (h_{ave}/h_m) for internal drainage layers. (from Giroud and Houlihan, 1995).

4.5.3 HELP Model

In the HELP model, lateral drainage in internal drainage layers is modeled by an analytical approximation to the steady-state solution of the Boussinesq equation (Darcy's equation coupled with the continuity equation), employing the Dupuit-Forcheimer assumptions. Hydraulic heads calculated for internal drainage layers in the HELP model are similar to those that would be calculated using the equations presented by Giroud and Houlihan (1995) for equal values of PERC*. Based on the example calculation in Section 4.4 of this document, the HELP model can be used directly for calculating lateral flow and hydraulic heads in cover system internal drainage layers. However, in using the model, the user should select a weather data generating option that produces extreme wet weather periods for the project site. Use of the 1974-1977 HELP model internal weather database will not typically be adequate.

4.6 Design of Slope Transitions

Design of internal drainage layers at benches and other slope transitions 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 (q) across the slope transition must not decrease. Flow capacity for laminar flow parallel to a slope is equal to the hydraulic gradient multiplied by the hydraulic transmissivity of the drainage layer material. This design requirement is illustrated in Figure 4-17.



$$q_1 = q_2$$

$$T_1 \sin \beta_1 = T_2 \sin \beta_2$$

$$k_1 t_1 \sin \beta_1 = k_2 t_2 \sin \beta_2$$

Figure 4-17. Continuity of flow across a slope transition for laminar porous media condition.

For many conventional cover system designs, the hydraulic gradient on the flatter part of the slope transition will be about one order of magnitude lower than the hydraulic gradient on the steeper part. For example, the gradient of a 3H:1V slope is 0.32, whereas the gradient reduces to 0.03 for a 3% slope inclination typical of a cover system bench. For this condition, to prevent backup of flow and build-up of hydraulic head for drainage layer flowing full, the hydraulic transmissivity of the drainage layer on a cover system bench or slope transition will need to be about one order of magnitude larger than that of the drainage layer on the sideslope.

More generally, based on Figure 4-17, the slope transition should be designed such that:

$$\theta_{h2} > \theta_{h1} (\sin\beta_1 / \sin\beta_2) \qquad \text{(Eq. 4.25)}$$

where all terms are as defined previously. The subscript 1 refers to the portion of the drainage layer on the steeper upslope side of the transition, and the subscript 2 refers to the drainage layer on the flatter downslope side of the transition (Figure 4-17). Eq. 4.25 can be used directly to analyze and design geosynthetic drainage layers for which hydraulic transmissivities are either known or measured in the laboratory. For granular drainage materials where materials are typically specified in terms of a required hydraulic conductivity and thickness, Eq. 4.25 is recast as:

$$k_2 \geq k_1 \, (t_{m1}/t_{m2}) \, (\sin\beta_1/\sin\beta_2) \qquad \text{(Eq. 4.26)}$$

where all terms are as defined previously. For Eq. 4.25 to be valid, t_{m1} and t_{m2} must be less than the total thickness of the drainage layer.

The concept of having a larger internal drainage layer hydraulic transmissivity (or hydraulic conductivity) on a slope bench compared to the adjacent upslope portion of the cover is illustrated in Figure 4-18(a). This approach is conveniently achieved with geosynthetic drainage layers; it is more difficult to implement with granular drainage materials because it requires very coarse-grained materials on the benches or slope transitions while meeting filter criteria at the interface between drainage materials. Other options for designing benches and slope transitions are shown in Figures 4-18(b), (c), and (d). These include:

- installing a perforated pipe within the slope transition to convey water to outlet pipes (Figure 4-18(b)); this approach is technically acceptable, but there can be a problem with the pipes freezing and plugging; also, it is essential that the pipes remain open and not be plugged or damaged by maintenance personnel; in addition, the discharge from the pipes may tend to erode soil beneath the pipes, and the surface should be adequately protected to prevent excessive erosion;
- installing a perforated pipe within the slope transition to convey water to a downdrain or downchute; this has the advantage of keeping the piping below the surface, where it can be protected from freezing; because the surface of the bench is normally sloped to provide surface drainage, the perforated pipe can follow the slope of the bench and provide gravity drainage to the outlet point; the outlet must still be protected and cannot be obstructed or clogged; and
- allowing the drainage layer to daylight on the bench. The bench must be suitably protected to prevent erosion; also, the outlet cannot freeze, which makes this approach questionable in northern climates.

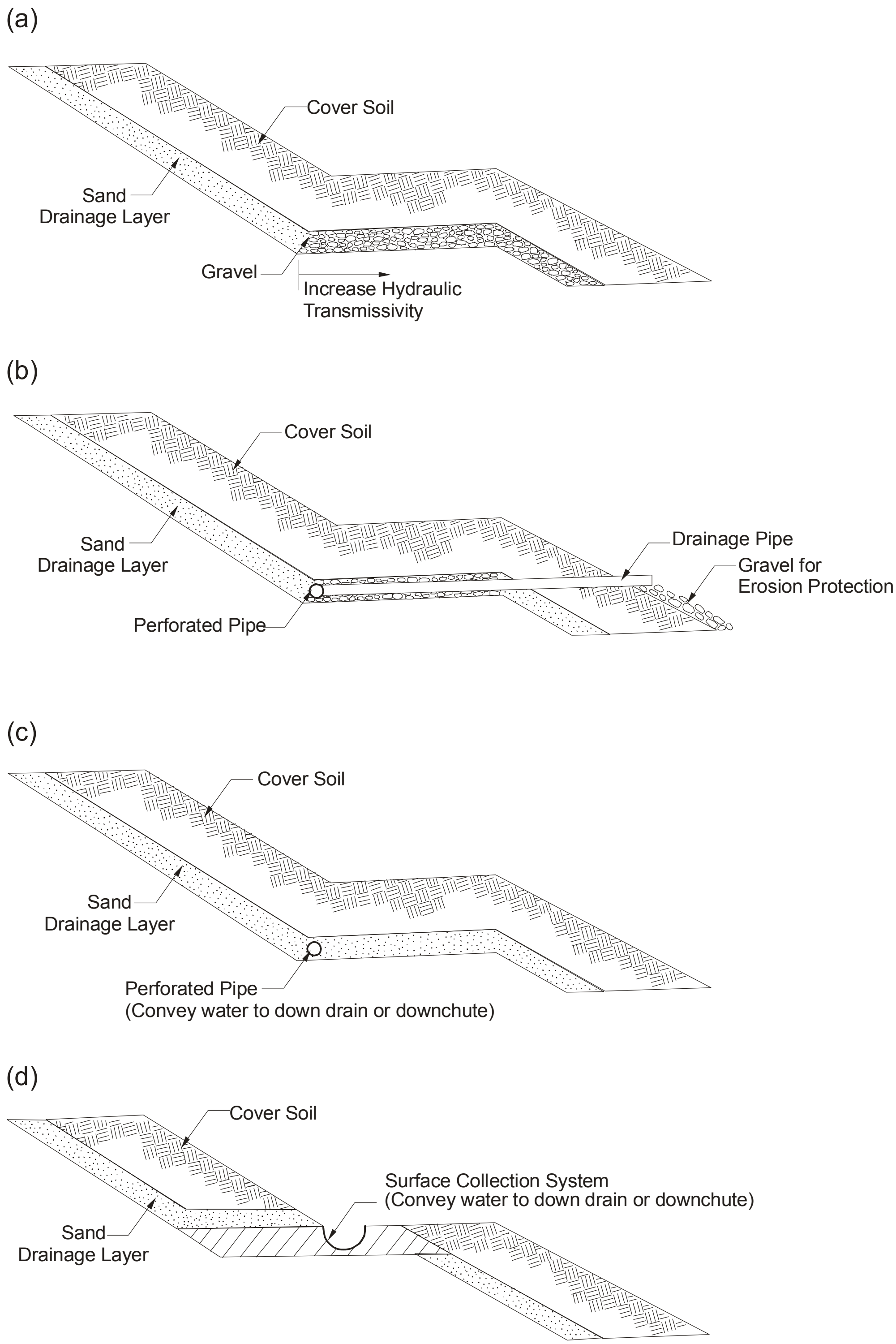


Figure 4-18. Design options for cover system slope transitions.

4.7. Design of Filter Layers

4.7.1 Overview

To prevent clogging of internal drainage layers, it is often necessary to install a granular or GT filter layer directly over the drainage layer material. Several of the cover system slope stability problems described in Chapter 7.4 of this document were due, at least in part, to inadequate filter layer design. The function of the filter in cover system applications is to limit the migration of fines from the overlying cover soil into the internal drainage layer, while allowing unimpeded percolation from the cover soil into the drainage layer. If the drainage material is a granular soil, the filter material may be either soil or GT. If the drainage material is itself a geosynthetic, the filter layer will also need to be a GT.

Filter criteria establish the relationship of grain sizes necessary to retain adjacent materials and prevent clogging of a drainage layer, while allowing unimpeded percolation. Criteria for the design soil and GT filter layers are discussed below.

4.7.2 Soil Filters

Soil filters usually consist of fine to medium sands when placed over coarse sand or fine gravel drainage layers. The filter particle size distribution must be carefully selected. Fortunately, there is a considerable body of information available to use in selecting a filter particle size distribution (Koerner and Daniel, 1997). Typically, the criteria described in Cedergren (1989) are used. To prevent piping from the overlying cover soil into the filter layer, and from the filter into the drainage layer, these criteria require, respectively:

$$D_{15} \text{ (filter)}/D_{85} \text{ (cover soil)} < 4 \text{ to } 5 \qquad \text{(Eq. 4.27)}$$

and:

$$D_{15} \text{ (drainage layer)}/D_{85} \text{ (filter)} < 4 \text{ to } 5 \qquad \text{(Eq. 4.28)}$$

To maintain adequate permeability of the filter layer and drainage layer, these criteria require, respectively:

$$D_{15} \text{ (filter)}/D_{15} \text{ (cover soil)} > 4 \text{ to } 5 \qquad \text{(Eq. 4.29)}$$

and:

$$D_{15} \text{ (drainage layer)}/D_{15} \text{ (filter)} > 4 \text{ to } 5 \qquad \text{(Eq. 4.30)}$$

where: D_{85} = particle size at which 85% by dry weight of the soil particles are smaller (mm); and D_{15} = particle size at which 15% by dry weight of the soil particles are smaller (mm). The criteria should be satisfied for all layers or media in the drainage system, including cover soil, filter material, and drainage material.

4.7.3 GT Filters

A GT must be installed over a GN or drainage core when the overlying material is to be a cover soil. The primary function of the GT in this application is as a filter layer. As with soil filter layers, GT filters must allow percolation from the cover soil to pass unimpeded into the drainage layer while retaining the cover soil and limiting the migration of particles from the cover soil.

As with soil filters, the design of GT filters involves a two-step process: first to assess permeability (or permittivity) and second to evaluate soil retention (or apparent opening size).

The first step in design of a GT filter is to establish the GT permittivity (Ψ) requirements. The usual formulation involves expressing the minimum allowable GT permittivity (Ψ_{min}) as a multiple of the required permittivity (Ψ_{req}) to maintain flow continuity from the cover soil, as follows:

$$\Psi_{min} = FS\Psi_{req} \qquad \text{(Eq. 4.31)}$$

and:

$$\Psi = \frac{k_n}{t} \qquad \text{(Eq. 4.32)}$$

where: Ψ = GT permittivity (s^{-1}); k_n = GT cross-plane hydraulic conductivity (m/s); and t = thickness of GT at a specified normal pressure (m). A minimum FS of 5 to 10 is recommended.

The testing of a GT for permittivity is conceptually similar to the testing of granular soils for permeability. In the U.S., the testing is usually performed using the permittivity test, ASTM D 4491. Alternatively, some design engineers prefer to work directly with permeability and require the GT's hydraulic conductivity to be some multiple of the adjacent soil's hydraulic conductivity (e.g., 5 to 10, or higher).

The second step of the design of a GT filter is intended to assure adequate retention of the cover soil. There are several methods available for establishing the soil retention requirements of GT filters. Most of the available approaches, as applied to a cover system, involve a comparison of the cover soil particle size characteristics to the 95% opening size of the GT (i.e., defined as 0_{95} of the GT). The 0_{95} is the approximate largest soil particle size that can pass through the GT. Various test methods are used to estimate 0_{95}: (i) in the U.S., wet sieving is used and the value thus obtained is called the apparent opening size (AOS), ASTM D 4751; (ii) in Canada and some European countries, hydrodynamic sieving is used and the value thus obtained is called the filtration opening size (FOS); and (iii) in other European countries, wet sieving is used.

The simplest of the available design methods involves a comparison of the GT AOS to standard soil particle sizes as follows (Koerner, 1998):

- for soil with $\leq$ 50% passing the No. 200 sieve (0.074 mm): $0_{95} < 0.59$ mm (i.e., AOS of the GT $\geq$ No. 30 sieve); and
- for soil with > 50% passing the No. 200 sieve: $0_{95} < 0.33$ mm (i.e., AOS of the GT $\geq$ No. 50 sieve).

Alternatively, a series of direct comparisons of GT opening size (0_{95}, 0_{5o}, or 0_{15}) can be made to some soil particle size to be retained (D_{90}, D_{85} or D_{15}). The numeric value depends on the GT type, soil type, flow regime, etc. For example, Carroll (1983) recommends the following relationship:

$$0_{95} < (2 \text{ or } 3)D_{85} \qquad \text{(Eq. 4.33)}$$

where: D_{85} = particle size at which 85% by dry weight of the soil particles are smaller (mm); and 0_{95} = the 95% opening size of the GT (mm). As shown by Giroud (1992, 1996), Eq. 4.33 should only be used if the coefficient of uniformity of the soil to be protected is less than four. General procedures, applicable for all values of the coefficient of uniformity of the soil to be protected, are available: see Giroud (1982), Lafleur et al. (1989), and Luettich et al. (1992).

Occasionally, a drainage layer is placed directly against a GCL. For GT-encased GCLs, the GT components may not be adequate to prevent migration of bentonite into the drainage layer. The required filter criteria for this condition are under study, and the manufacturer's and technical literature should be consulted. One study indicated that a 350 g/m^2 nonwoven, needlepunched GT provided adequate protection from bentonite migration for all GCLs investigated (Estornell and Daniel, 1992).

Chapter 5
Gas Emission Analysis and Collection System Design

5.1 Introduction

This chapter provides information on select topics related to cover system gas emission analysis and collection system design. The specific topics discussed in this chapter are:

- mechanisms of gas generation and emission (Section 5.2);
- characteristics of selected gas emissions models (Section 5.3); and
- design of gas collection systems (Section 5.4).

5.2 Mechanisms of Gas Generation and Emissions

5.2.1 Overview

Landfill gas (LFG) is the byproduct of anaerobic decomposition of the organic material placed in a landfill during its active life. Landfill gas emissions may create hazardous situations. The nature and extent of the hazard depends on the emission rate, toxic constituent concentration, and the relative concentration of the flammable components. Human health and the environment may be adversely affected because of its potential to: A) create flammable/explosive conditions within enclosed spaces; B) contain a mixture of toxic and hazardous air pollutants (HAPs); C) contain large quantities of greenhouse gases (carbon dioxide, methane, nitrous oxide); D) contain volatile organic compounds (VOC) that are precursors to the formation of ozone and smog; and E) have an odiferous smell that is objectionable to many people. The literature indicates that LFG may contain more than 100 non-methane organic compounds (NMOC's) (EPA 1997 a and b). Over thirty of the NMOC's are classified as HAPs (EPA 1997 a and b). Landfills are listed as a source in EPA's Urban Air Toxic Strategy and have been identified for residual risk evaluation.

The LFG emission rate through the cover system of a landfill is dependent on the gas generation rate, whether the facility has a liner system, site hydrogeology, characteristics of the cover system, and characteristics of any gas control system. The Landfill Gas Emissions Model (LandGEM) estimates landfill gas emissions based on the age of the landfill, the quantity of waste placed within it, waste acceptance rate, and other site specific information. LandGEM uses a first-order decomposition rate equation to make the estimates (Thorneloe, 1999). A personal computer-based version of LandGEM can be downloaded from EPA's website at http://www.epa.gov/ttn/catc/products.html#software. A user's manual is also available on this website. The software has various sets of defaults values that can be adjusted by the user. One set is for those sites where the CAA requirements are determined to be applicable and appropriate. The other set is typically used for emission inventories and is less conservative than the CAA defaults. Site-specific data can also be used if available. EPA is also developing GUIDANCE FOR EVALUATING LANDFILL GAS EMISSIONS FROM CLOSED OR ABANDONED FACILITIES under EPA Contract number 68-c-00-186 Task Order Number 3.

This guidance when published will provide procedures and a set of tools for evaluating the nature, extent, risks and hazards associated with LFG emissions to ambient air, LFG subsurface vapor migration due to landfill gas pressure gradients, and subsurface vapor intrusion into buildings. Figure 5.1 provides a flow diagram of the guidance.

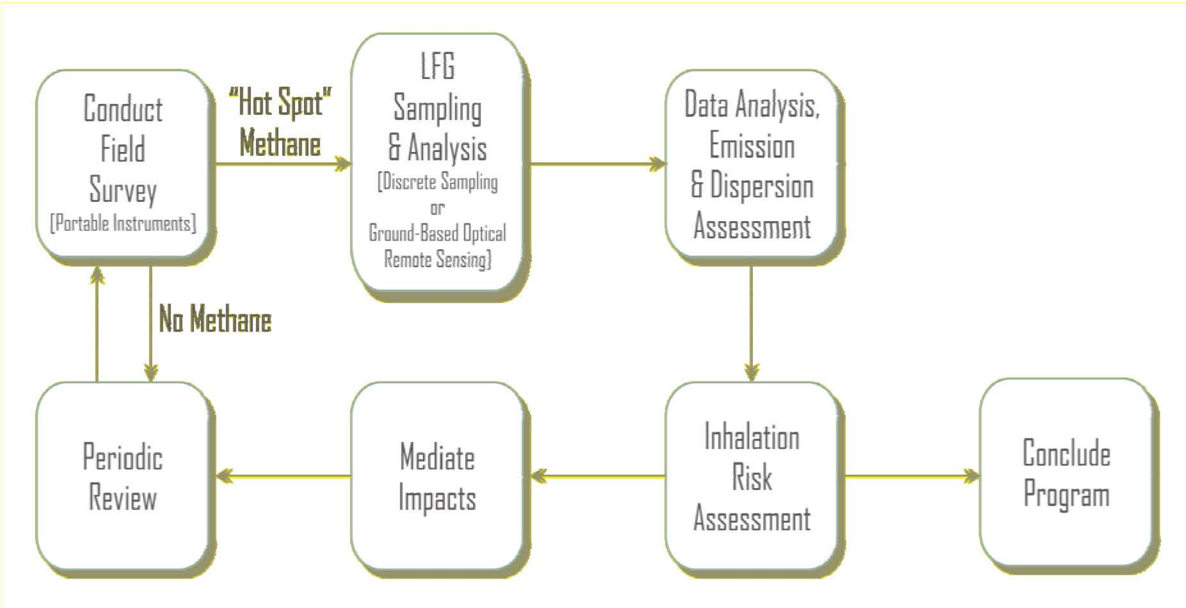


Figure 5-1. Flow Diagram of Guidance for Evaluating Air Pathway at Older, Closed Landfills

MSW landfills constructed or operated after October 9, 10993 are governed by the RCRA Subtitle D regulations. These regulations establish siting restrictions, and design, operating, and monitoring standards that are designed to minimize the potential for environmental damage. Additionally, rules and regulations implementing the Clean Air Act (CAA) establish Maximum Achievable Control Technology (MACT) standards that are applicable to landfills that exceed size and age threshold. The CAA regulations require landfill gas collection and control systems to be installed at landfills that (1) contain at least 2.5 million megagrams (Mg) or 2.5 million cubic meters of waste and (2) emit 50 Mg per year or more of NMOCs (EPA, 1998). EPA's emission guidelines (EGs) apply to existing landfills that were in operation from November 8, 1987 to May 30, 1991. EPA's new source and performance standards (NSPS) apply to any existing landfill constructed on or after May 30, 1991 or which undergo changes in design capacities on or after May 30, 1991. To help evaluate performance of the gas collection and control system, the NSPS/EGs (60 CFR §753) require that:

"Each owner or operator of a MSW landfill gas collection and control system used to comply with the provisions of Sec. 60.752(b)(2)(ii) of this subpart shall…(d) Operate the collection system so that the methane concentration is less than 500 parts per million above background at the surface of the landfill. To determine if this level is exceeded, the owner or operator shall conduct surface testing around the perimeter of the collection area along a pattern that traverses the landfill at 30 meter intervals and where visual observations indicate elevated

concentrations of landfill gas, such as distressed vegetation and cracks or seeps in the cover….”

For further information on the requirements of these regulations and available technical documents and fact sheets, refer to http://www.epa.gov/ttnatw01/landfill/landflpg.html.

Most hazardous waste landfills do not include significant quantities of garbage and other biodegradable materials. Hence the ability of a hazardous waste landfill to generate LFG is severely limited by the lack of carbon, nutrients, and moisture. Additionally, the Land Disposal Restriction (LDR) rules (see 40 CFR 268.7) have required generators of hazardous waste to meet the best demonstrated available technology (BDAT) standards since 1986. The LDR rules require generators to meet BDAT standards before any hazardous waste is placed in a landfill. The BDAT standards are designed to substantially diminish the toxicity of the waste or substantially reduce the likelihood of migration of hazardous constituents from the waste. The BDAT standards are waste code and media specific but in all cases the maximum allowable constituent concentration prior to disposal is measured in the parts per million or less range. Hazardous waste landfills are also required to install covers that are hydraulically impermeable. The existence of a hydraulically impermeable geomembrane (see Section 1.2.2) also limits uncontrolled LFG emissions from these types of facilities. There are, however, no regulatory requirements regarding LFG emissions from hazardous waste landfills. In contrast, hazardous waste disposed prior to 1986 may have included highly concentrated materials. Additionally, prior to 1980 most landfills were closed with a cover system consisting primarily of earthen materials that were permeable by design. . Emission of particulates from waste containment facilities and remediation sites is also a concern for the Agency. However, this type of emission is not addressed in this guidance document.

5.2.2 MSW Landfill Gas Generation

The anaerobic decomposition of MSW produces two principal gases, methane (CH_4) and carbon dioxide (CO_2), and much smaller quantities of other gases, including nitrogen, oxygen, sulfides, ammonia, and other constituents, and trace amounts of a variety of NMOCs, typically including vinyl chloride, ethylbenzene, toluene, and benzene (Tchobanoglous, 1993; EPA, 1997a, 1997b). The typical constituents found in MSW landfill gas and their concentrations are listed in Table 5-1. Though it is not included in Table 5-1, landfill gas is also typically saturated with water vapor at levels of 1 to 5% by volume. Typical concentrations of NMOCs in landfill gas are presented in Table 5-2.

The methane in MSW landfill gas can accumulate in enclosed or confined spaces (typically near the perimeter of the landfill, but in some cases at considerable distances from the perimeter) in concentrations that are odorous, asphyxiating, toxic, corrosive, flammable, or even explosive. Besides methane, other gas constituents, such as hydrogen gas, are also explosive, and certain gas constituents, such as hydrogen sulfide gas, are also toxic at a certain concentrations. Landfills have been identified as the source of nearly 30 HAPs, including the constituents listed in Table 5-2. A more comprehensive list of the different NMOCs and HAPs in landfill gas is contained in EPA's AP-42 which provides guidance for estimating landfill gas emissions (EPA

1997a and b). The source of these HAPs is primarily household and small quantity generator hazardous wastes, including paints, solvents, pesticides, and adhesives.

Because of the hazards posed by MSW landfill gas, care must be taken in handling the gas. CAA regulations establish requirements for MSW landfill gas collection and control at certain facilities, as described in Section 1.4.

Table 5-1. Typical landfill gas constituents (from Tchobanoglous et al., 1993).

Constituent	Percent by Volume
Methane	40-60
Carbon dioxide	40-60
Nitrogen	2-5
Oxygen	0.1-1
Ammonia	0.1-1
Sulfides, disulfides, mercaptans, etc.	0-0.2
Hydrogen	0-0.2
Carbon monoxide	0-0.2
Trace constituents	0.01-0.6

Table 5-2. Typical concentrations of NMOCs in gas from 25 landfills in southern California (from Pierce et al., 1998).

Trace Gas Constituent	Average Constituent Concentrations (ppm by volume)	
	Range	Mean
Benzene	0.432-21.8	2.76
Chlorobenzene	0.054-5.24	0.606
1,1-Dichloroethane	0.10-15.9	2.39
1,2-Dichloroethane	0.01-3.74	0.29
Methylene Chloride	0.10-56.3	10.5
Tetrachloroethene	0.30-28.2	3.24
Tetrachloromethane	0.001-0.413	0.046
Toluene	8.37-67.7	28.3
1,1,1-Trichloroethane	0.012-8.28	0.715
Trichloroethene	0.293-13.6	1.60
Vinyl Chloride	0.277-16.8	1.99

Landfill gas generation is often considered to occur in five sequential phases, as shown in

Figure 5-1. During Phase I, the initial adjustment phase, waste placement starts, and the waste begins to accumulate moisture. Microbes in the waste begin to acclimatize to the landfill environment. With plenty of substrate and nutrients available, aerobic microbes start to degrade the waste, producing water, carbon dioxide, organic acids, and inorganic minerals. The aerobic decomposition is sustained by the oxygen trapped within the waste mass. Because Phase I is relatively short lived and involves aerobic decomposition, it is sometimes combined with Phase II and referred to as the "aerobic phase".

During Phase II, the transition phase, oxygen trapped within the landfill is depleted and the landfill transitions from an aerobic to anaerobic environment. Since the amount of trapped oxygen is limited, this stage is also relatively short lived (i.e., a few days to a few months). As oxygen is depleted, a trend for reducing conditions is established, with a shifting of electron acceptors from oxygen to nitrates and sulfates. Reduction of these latter molecules, often produces nitrogen gas and hydrogen sulfide gas. In addition, the carbon dioxide level begins to increase, causing the formation of carbonic acid and a decrease in the leachate pH to the acidic range. Waste temperatures are hottest during this phase, reaching 54 to 71ºC.

In Phase III, the acid phase, waste is degraded anaerobically. The waste first undergoes hydrolysis, where larger organic molecules are converted into shorter, soluble molecules and hydrogen gas is produced. Acidogenic bacteria then convert the hydrolyzed compounds into volatile organic acids (VOAs). The acids, in turn, cause the pH to drop (e.g., to 5.5 to 6.5) in

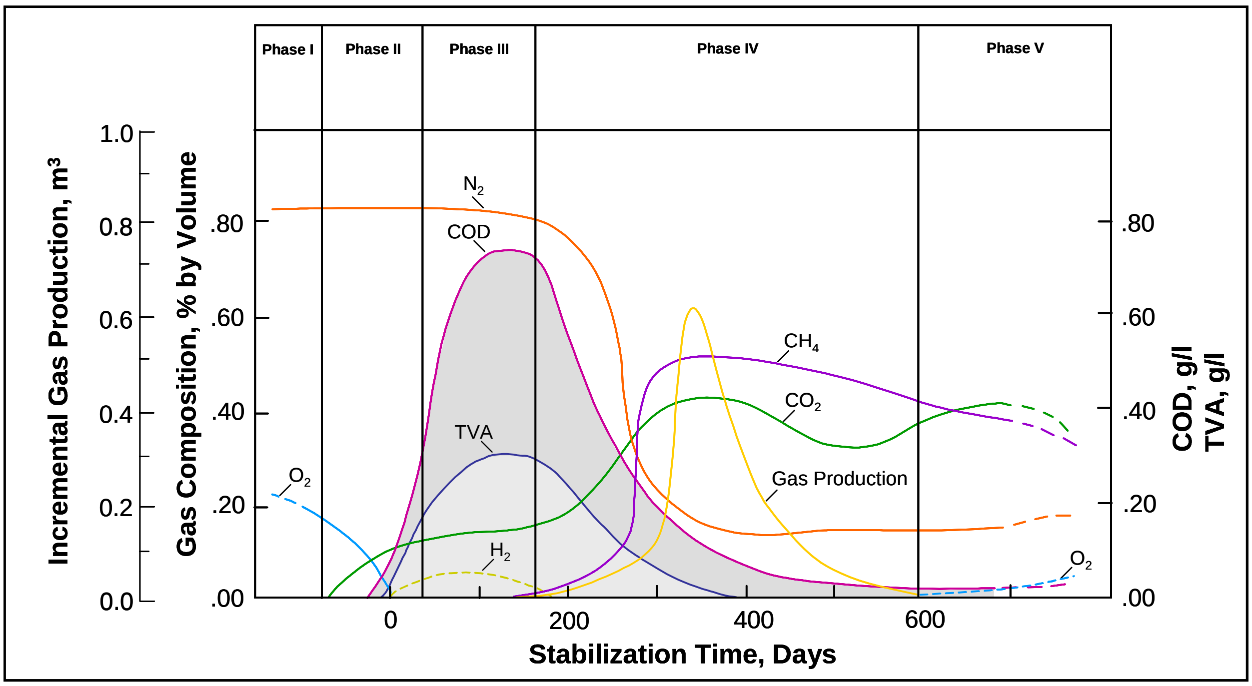


Figure 5-2. Generalized Phases in the Generation of Landfill Gases (modified from Kreith, 1994).

turn causing heavy metals concentrations to rise in the leachate. Viable biomass growth associated with the acidogenic bacteria and the rapid consumption of substrate and nutrients are the predominant features of this phase. The primary gas formed during this stage is carbon dioxide.

In Phase IV, the methane fermentation phase, the VOAs and hydrogen gas produced by the acidogens are converted into methane by methanogenic bacteria. Both acid production and methane fermentation occur during this phase; however, methane fermentation predominates. The highest landfill gas generation rates occur during this phase. As the VOAs are utilized, the pH of the leachate increases to more neutral values (e.g., 6.8 to 8) and heavy metals concentrations decrease. Sulfates and nitrates are reduced to sulfides and ammonia. Gas temperatures have dropped by this phase to about 38 to 54ºC. Gas production probably begins to drop off at the lower end of this temperature range. As described by Hutric and Soni (1997), a study of an experimental MSW digestor showed that gas generation rates peaked at two temperatures: about 40 ºC, when mesophilic bacteria are present, and between 55 and 60ºC, when thermophilic bacteria are present. At temperatures below 40 ºC, gas generation rates decrease rapidly with decreasing temperature.

By Phase V, the maturation phase, the landfill has matured and the readily biodegradable material has been stabilized (i.e., converted to methane or carbon dioxide). Biodegradation is limited by lack of readily degradable substrate and nutrients, so biological activity slows. The landfill gas production rate, consequently, also decreases. Both carbon dioxide and methane gases are produced, but at much lower rates than in Phase IV. Towards the latter part of this phase, the landfill may become aerobic, with oxidizing conditions, and small amounts of oxygen and nitrogen gases may be present.

Since landfills are heterogeneous and all waste is not placed at the same time, the stages described above typically occur concurrently in different areas and depths of an active or recently closed landfill. The dichotomy between stages is often masked when a landfill is active and new waste is being added to old. After a landfill closes, the landfill tends to move into Phase IV, with the newer waste just keeping the landfill at this phase for a longer time period.

The rate of waste degradation is controlled by the amount and type of degradable materials in the waste, waste temperature, waste moisture content, and other factors. Food waste may degrade about five times faster than yard waste, fifteen times faster than paper, and fifty times faster than wood or leather. Degradation is enhanced (the reaction rates increase) by the initial temperature increase caused by the heat released from aerobic degradation. The temperature falls over time, however, as the waste loses heat to its surroundings. In deeper landfills, this heat is better retained and degradation occurs faster than in shallower landfills. Water is generated in the aerobic biodegradation process and required for the anaerobic biodegradation process. In addition, water movement through a landfill helps to mix the enzymes, bacteria, and substrate. The subsistence moisture level required by methanogenic bacteria is very low. This is why gas generation occurs even in the driest of landfills (McBean et al., 1995).

Although moisture content is thought to be an important factor in landfill gas emissions, there is much variability in the level of emissions from site to site. Typically, emissions in more arid regions are thought to occur over a longer period of time than sites in more temperate climates. For those sites operated as a wet landfill where leachate has been added or there are other liquid additions, emissions occur over a much faster rate and there can be a high level of fugitive gas emissions depending upon how liquid is added to the site.

5.2.3 Gas Emissions

Water and gas flows occur simultaneously in a waste containment facility or contamination source area as a dynamically-coupled process. As described by Berglund (1998), flow of gas and water within a landfill can be conceptualized as a trickle bed. The liquid phase trickles over the waste particles and the gas phase migrates in the remaining pore space. At the present, it is not possible to effectively integrate all the biochemical reaction and multi-phase transport mechanisms into one model. Instead, the processes must be uncoupled and discussed separately. These processes are: (i) percolation of water through a cover system, which was discussed in Chapter 4; (ii) waste degradation and gas production, which was discussed in Section 5.2.2; and (iii) gas emissions through a cover system, which is discussed in this section.

Gas flow within and through the cover system of a waste containment facility or remediation site is mainly pressure driven at gas pressures above about 3kPa, but also responds to temperature, density, and concentration gradients. Pressures generated by MSW gas may be on the order of 2.5 to 7.5 kPa for younger landfills located in temperate climates to 0.5 kPa for older landfills located in arid climates. As gas pressures increase in a waste mass, the gas travels along the path of least resistance. The final disposition of the generated gas depends on the engineered controls (e.g., containment systems, gas management system). Gas may be stored in the waste, migrate through a liner or barrier (if they exist) and into available air space in the surrounding subsurface, emitted through the cover into the atmosphere, or collected and treated by a gas management system, if one exists.

Gas emissions may be affected by the gas pressures within a landfill, barometric pressure, moisture content and gas permeability of the soil components of the cover system, chemical diffusion rate through a GM barrier component of the cover system, advective flow rate through any holes in the GM barrier component, and other factors. Barometric pressure is a function of atmospheric pressure and changes in weather. It responds diurnally to atmospheric tides with a high in the early morning hours and a low in the afternoon hours. It also responds to changes in the high and low-pressure systems related to weather conditions. Because gas emissions are typically pressure driven (i.e., convection rather than diffusion is typically the primary transport mechanism), gas emissions generally follow the reverse trend of barometric pressures, due to the effect on pressure gradients. When barometric pressures are highest, gas emissions are lowest and vice versa. For landfills with active gas collection systems, a change in barometric pressure should have less impact on gas emissions than occurs for landfills without these systems.

Even if landfill gases move from the landfill, through the cover system, and into the atmosphere, some of the gases may be consumed by microbes in cover system soils. The extent of oxidation is a function of how well the cover is maintained. Often cracks in a cover can result in leaks for the gas to escape to the atmosphere (see Figure 5-2). Data are available documenting oxidation rates through soil covers. For example, relatively high methane oxidation rates of 45 g/m^2/d were observed in topsoil above a landfill in California (Whalen et al, 1990); methane concentrations in the air immediately above the topsoil were very low. Oxidation of methane by soil microbes has been demonstrated in controlled laboratory experiments (Knightley et al., 1995). The experiments showed that as the flux of methane into a soil layer decreased, a greater proportion of methane was oxidized. Clearly, however, widespread documentation of atmospheric methane emissions from soil-covered landfills shows that microbial methane degradation in cover soils is often not complete. If gas control is needed at a site, then an active or passive system needs to be installed. The cover material needs to be maintained to minimize any cracks that will allow a preferential flow path for fugitive emissions of landfill gas.

Figure 5-3. Surface Cracks at a Landfill

With respect to soils, the pores of the soil must be nearly saturated to prevent gas migration. Thus, it is not surprising that landfill gas is often detected at the surface of the soil cover systems. Landfill methane emissions measured at landfill sites and reported in the literature have ranged from about 0.003 to 3,000 g/m^2/d (Bogner and Scott, 1997). In general, the higher rates were associated with landfills that did not have gas recovery and that were covered with relatively more permeable and/or drier soils. For example, at the Olinda MSW Landfill in Southern California, which is covered by a sandy silt soil layer, measured emission rates were greater than 1,000 g/m^2/d prior to installation of a gas collection system. After a gas collection system was installed, measured gas flux rates were less than 10 g/m^2/d. The flux rates were still lower (less than 0.01 g/m^2/d) in the area of the landfill with a gas recovery system and covered with a clayey silt layer.

Characteristics of Selected Gas Emission Models

5.3.1 LandGEM Model for MSW

Gas emission rates for MSW landfills can be difficult to predict. The better the input data, the better the estimate. Landfill gas emissions vary over time. EPA recommends the use of a first-order decomposition rate equation to estimate annual emissions over a user-specified time period. EPA had developed an automated estimation tool for calculating landfill gas emissions. The is referred to as the Landfill Gas Emissions Model (LandGEM). It uses a Microsoft Excel interface and is used to estimate emission rates for total landfill gas, methane, carbon dioxide, nonmethane organic compounds, and individual air pollutants from municipal solid waste (MSW) landfills (EPA, 2005). The equation that is used in LandGEM is:

$$Q_{CH_4} = \sum_{i=1}^{n} \sum_{j=0.1}^{1} kL_o \left(\frac{M_i}{10} \right) e^{-kt_{ij}}$$

where,

Q_{CH4} = annual methane generation in the year of the calculation (m^3/year)
i = 1-year time increment
n = (year of the calculation) - (initial year of waste acceptance)
j = 0.1-year time increment
k = methane generation rate ($year^{-1}$)
L_o = potential methane generation capacity (m^3/Mg)
M_i = mass of waste accepted in the i^{th} year (Mg)
t_{ij} = age of the j^{th} section of waste mass M_i accepted in the i^{th} year (decimal years, e.g., 3.2 years)

LandGEM can use either site-specific data to estimate emissions, or, if no site-specific data are available, use default parameters. The model contains two sets of default parameters, CAA defaults and inventory defaults. The CAA defaults are based on federal regulations for MSW landfills laid out by the Clean Air Act (CAA) and can be used for determining whether a landfill is subject to the control requirements of these regulations. The inventory defaults are based on emission factors in EPA's *Compilation of Air Pollutant Emission Factors* (*AP-42*) and can be used to generate emission estimates for use in emission inventories and air permits in the absence of site-specific test data. (EPA , Chapter 2.4, 1997) The software can be downloaded from EPA's web site (http://www.epa.gov/ttn/catc/products.html#software). Figure 5.4 provides a screen capture of the user interface.

Figure 5.4 Screen-Capture for EPA's LandGEM Computer Software

The methane generation rate constant, k, is a function of waste moisture content, pH, and temperature, and nutrient availability to methanogens. In a study by EPA, the value of k for MSW landfills was estimated to range from 0.003 to 0.21/yr (EPA, 1998) based on field test data and the results of theoretical models using field test data. For landfills at arid and semi-arid sites (defined by EPA as sites with less than 640 mm of precipitation per year), EPA's "best estimate" of k is 0.02/yr (EPA, 1997a). The methane generation potential, L_0, is a function of waste composition. EPA found values for L_0 ranging from 6.2 to 270 m^3/Mg based on theoretical modeling and field test data for a number of landfills (EPA, 1998). EPA's "best estimate" of this parameter is 100 m^3/Mg (EPA, 1997a) and was obtained from empirical data from operating landfills using gas extraction data.. Murphy (1998) indicated that the gas generation rates predicted with the EPA model showed reasonable correspondence to field data from landfills in arid settings when the default parameters were changed to k = 0.005/yr and L_0 = 16 m^3/Mg. For most landfills, the EPA parameters can be used with the EPA model to develop initial estimates of gas generation rates. These parameters can then be adjusted as data on gas flow rates or emissions are collected over time. Hutric and Soni (1997) describe how this data fitting may be carried out.

From chemical analysis of landfill gas samples collected at landfills, EPA also developed "best estimates" of NMOCs and air pollutants for landfills with and without co-disposal of hazardous waste. EPA's best estimate of the NMOC concentration as hexane is 2,420 ppmv for landfills that did not have co-disposal of hazardous waste (EPA, 1998). The measured NMOC concentration reported for the 23 landfill considered by EPA ranged from 240 to 14,300 ppmv (EPA, 1998). As described by Repa (1994), at the sites in the EPA study, the compounds most frequently detected in landfill gas included benzene, tetrachloroethene, toluene, trichlorofluoro-methane, trichloroethane, and vinyl chloride. The compounds detected at the highest average concentration included ethylbenzene, methylene chloride, propane, and xylenes. Effort is underway to update EPA's landfill gas emission factors. A Cooperative Research and Development Agreement with the Environmental Research and Education Foundation is providing cofunding with EPA's Office of Research and Development for conducting field tests. These data will help in providing more up-to-date landfill gas emission factors. Once these are released, LandGEM will be updated to included the updated emission factors.

LandGEM does not account for gas storage within the waste mass, nor subsurface gas migration, which for old unlined landfills can be a significant migration pathway. However, it is considered a reliable tool in helping to quantify potential gas emissions. If more reliable estimates are needed, then emission measurements can be conducted using open-path technology (Modrak et al., 2004 and 2005a and b). Gas extraction rates have been estimated to be 10 to 60% of the total gas generated (Augenstein et al., 1997). However, collection efficiency is never precisely known and, if used in the gas generation equation, must be an assumed value, Hutric and Soni (1997).   EPA reports a gas collection efficiency range of 60 to 85%, with an average of 75 percent most commonly assumed. (EPA, Chapter 2.4, 1997)

5.3.2 Diffusion Model for Emissions of Organic Vapors

For hazardous waste landfills and waste piles, if any volatiles are left in the waste when it reaches the facilities, they are rapidly emitted from the surface of exposed waste. After a cover system is placed over the waste, emissions of organic vapors occur by diffusion, convection by barometric pumping, and gas venting.

The model EPA has used for hazardous waste landfills and waste piles assumes diffusion of volatiles from the waste surface though the cover system (and neglects convective flow due to changes in barometric pressure) (EPA, 1992). Volatiles in the waste are assumed to be in equilibrium with air in void spaces of the waste. When the organic vapors reach the surface of the cover system, they are assumed to be removed by wind (i.e., the constituent concentration at the cover system surface is assumed to be zero).

5.4 Design of Gas Collection Systems

Gas collection systems are typically designed as part of passive gas management systems or active gas extraction systems utilizing negative pressure systems. Passive systems are primarily effective at controlling convective flow (due to pressure and density gradients) and have limited success controlling diffusive flow. Active systems are effective in controlling both types of flow. Active systems are preferred when a significant amount of gas is being generated, and these systems are required for facilities of certain sizes to reduce the amount of gas constituents released to the atmosphere. Design of gas collection systems can be based on calculated gas generation rates or vapor emission rates or from the results of field tests (e.g., pump tests).

Some design engineers collect and vent or extract MSW landfill gas with vertical, perforated collection wells (typically 1 to 3 wells per hectare) without a continuous gas collection layer beneath the hydraulic barrier component of the cover system. This approach can be justified if the waste itself is sufficiently permeable to gas, if the gas wells are relatively closely spaced, or, at arid sites, where gas is generated relatively slowly. With gas wells, the gas moves within the waste to the perforations in the pipe and then flows or is drawn out of the system. Another approach to venting or extracting gas from a landfill involves installing a continuous gas collection layer beneath the cover system barrier. With this type of system, shallow gas venting or extraction pipes will tie into the gas collection layer. Gas collection trenches with periodic vent or extraction pipes represents a third approach to gas collection beneath the cover system. Also, a combination of these three gas venting/extraction systems can be used. For active systems, additional components may include a vacuum blower system, a manifold to connect multiple wells, off-gas treatment (e.g., enclosed flare, gas-to-energy system, carbon adsorption), condensate holding tank, and monitoring and control equipment.

In any case (deep wells penetrating the waste, a continuous gas collection layer, beneath the barrier layer, and/or collection trenches) the system outlets are typically plastic pipes extending up through the cover system. Gas flow through the pipes can be either passive (vented to the atmosphere or flared) or active (collected through a header using a blower system to create a small vacuum). Without a gas management system, gas pressure will build up in the landfill. Note that with a GM in the cover system and relatively small cover soil thicknesses, gas pressure can cause GM uplift. Even if the GM is not physically lifted, positive gas pressure beneath the GM can lower the effective stress at the interface between the GM and underlying material (e.g., GCL), thereby reducing interface shear strength and potentially contributing to a slope failure. At several landfill facilities, this latter effect had led to slippage of the GM and overlying cover materials (Bonaparte et al., 2002) creating high tensile stresses as evidenced by compression ridges in the cover soil and folding of the GM at the slope toe and tension cracks in the cover soil near the slope crest.

Based on the above, all of the three types of gas collection systems require careful design considerations:

- if gas removal is by deep wells, the uppermost pipe perforations should be effective in capturing gas in the upper layers of waste;
- if gas removal is by a gas collection layer beneath the GM and vents, the gas collection layer should be designed with adequate long-term transmissivity; and
- if gas is removed by horizontal collection trenches, some of the trenches should be placed in close proximity to the bottom of the cover system to prevent gas accumulation and uplift pressure on the cover system GM.

In general, gas collection systems should be designed with a minimal number of penetrations through the cover system, as each penetration is a potential location for preferential flow (i.e., short-circuiting of gas through the cover system).

For passive systems, a maximum of one well per acre should be included initially (EPA, 1991). If monitoring of the vents reveals excessively high gas concentrations, then additional wells can be installed.

In addition to the above, as gases are collected, condensate usually forms because the temperature at the surface is often less than the temperature of the gas. Gas collection systems often include condensate traps and piping that directs condensate to some collection point (e.g., back into a MSW landfill).

Chapter 6
Geotechnical Analysis And Design

6.1 Introduction

This chapter provides information on select topics related to cover system geotechnical analysis and design. The specific topics discussed in this chapter are:

- static slope stability (Section 6.2);
- seismic slope stability and deformation (Section 6.3);
- settlement (Section 6.4);
- steep slopes (Section 6.5); and
- soft waste materials (Section 6.6).

6.2 Static Slope Stability

6.2.1 Overview

Slope stability is a critical issue in the design of cover systems. Slopes on landfills, waste piles, and other waste containment structures are sometimes quite steep. Sideslope inclinations can range from flatter than 5H:1V (11.3°) to steeper than 2H:1V (26.6°). For example, cover systems have been constructed over waste slopes steeper than 1.5H:1V (33.7°) as part of the remediation of old dumps. This section of the guidance document addresses issues associated with the static slope stability of cover system components. Both internal and interface downslope sliding of one or more components are considered. Failure surfaces that extend into the waste are not addressed herein, but should be considered in slope stability analyses. Special stability issues associated with cover system sideslopes steeper than about 2.5H:1V and cover systems installed over soft waste materials are discussed in Sections 6.5 and 6.6, respectively.

The frequency of occurrence of cover system stability problems has been high. More than a dozen case studies of past problems of this nature are described by Gross et al. (2002) and briefly discussed in Section 7.4 of this guidance document. One example of a cover system stability problem is shown in Figure 6-1. The photographs in this figure show a topsoil surface/protection layer that has slid downslope over a reinforced GCL barrier in a cover system that did not contain an internal drainage layer. Figure 6-2 shows another example, this one involving a topsoil surface/protection layer and underlying sand drainage layer that has slid over a textured HDPE GM barrier. In this case, the sand drainage layer (specified hydraulic conductivity of 1×10^{-5} m/s) had inadequate flow capacity and the drainage layer outlets were constricted. With GMs, GCLs, CCLs, GTs, and GCs commonly used in a variety of cover system configurations, the stability of potential low shear strength materials and interfaces must be considered for most

Figure 6-1. Example of Cover System Slope Stability Problem. The Topsoil Surface/Protection Layer Slid Downslope Over the Reinforced GCL Barrier.

Figure 6-2. Example of Cover System Slope Stability Problem. The Topsoil Surface/ Protection Layer and Underlying Sand Drainage Layer Slid Downslope Over the Textured HDPE GM Barrier.

designs. Significantly, past failures have involved sliding along each of the geosynthetic interfaces listed in Table 6-1.

Table 6.1 Interfaces upon which cover system components have undergone sliding.

- Topsoil surface/protection layer sliding on:
  - GT
  - GM
  - GCL
  - CCL
- Sand drainage layer sliding on:
  - GT
  - GM
  - GCL
  - CCL
- GN drainage layer sliding on GM
- GC drainage layer sliding on GM
- GT sliding on GM
- GM sliding on:
  - GT
  - GCL
  - CCL
- GCL sliding on:
  - CCL
  - prepared subgrade

6.2.2 Limit Equilibrium Analyses

6.2.2.1 Overview

The simplest limit equilibrium (LE) formulation to analyze the slope stability of cover systems assumes infinite slope conditions and neglects the stabilizing influences of passive soil resisting forces at the toe of the slope, any true cohesion/adhesion in cover system materials and interfaces, and tension in the geosynthetic layers. More sophisticated LE formulations account for these factors. Both the infinite slope and more sophisticated LE formulations are discussed below. In all of the closed-form, two-dimensional LE solutions, force equilibrium is satisfied in the directions normal and parallel to the slope, but moment equilibrium is ignored.

6.2.2.2 Infinite Slope

For cover system geometries where the cover soil thickness is constant, infinite slope equations provide a simple and conservative basis for design. Equations can be formulated in terms of: (i) total unit weights of the cover system materials and boundary water pressures; or (ii) buoyant unit weights and body seepage (or drag) forces. In keeping with the approach of Giroud et al. (1995a), equations are formulated herein using buoyant unit weights and seepage forces.

Body seepage forces occur in cover systems when water infiltrating the cover system develops a significant flow component in the downslope as opposed to vertical downward direction. This occurs, for example, when infiltration is blocked by a hydraulic barrier. If the rate of infiltration is sufficient, hydraulic head will build up above the barrier layer and induce downslope flow. Downslope flow of water has a destabilizing effect on the cover system. The seepage force per unit volume on soil particles in the direction of laminar flow is expressed as:

$$f_w = \gamma_w i \qquad \text{(Eq. 6.1)}$$

where: f_w = seepage force per unit volume (N/m^3); γ_w = unit weight of water (N/m^3); and i = hydraulic gradient (dimensionless). The concept of a seepage force, F_w (N) (acting parallel to the slope), and buoyant unit weight, W_b (N) (acting vertically), in an infinite soil slope underlain by a hydraulic barrier is illustrated in Figure 6-3. For a 1-m thick cover system at a 3H:1V (18.4°) slope and with water flowing in the entire soil thickness, the water induces a downslope body seepage force of 3 kPa.

If there is no water flow in an infinite slope, the slope stability factor of safety is given by (Giroud et al., 1995a):

$$FS = \frac{\tan\phi_i}{\tan\beta} + \frac{a_i}{\gamma_t t \sin\beta} \qquad \text{(Eq. 6.2)}$$

where: FS = factor of safety (dimensionless); ϕ_i = angle of internal or interface friction for the critical potential slip surface (degrees); a_i = adhesion (for an interface) or cohesion (for internal strength) for the critical potential slip surface (Pa); β = slope angle (degrees); γ_t = total unit weight of material above the critical potential slip surface (N/m^3); and t = thickness of material above the critical potential slip surface (m). Use of this equation assumes that there is a unique critical potential slip surface in the cover system. For the case of no adhesion or cohesion (a_i = 0), Eq. 6.2 reduces to the classical solution:

$$FS = \tan\phi_i / \tan\beta \qquad \text{(Eq. 6.3)}$$

For a hydraulic barrier system, two conditions need to be considered: (i) stability above the hydraulic barrier; and (ii) stability below the hydraulic barrier. These two conditions must be considered because effective stresses above and below a non-porous hydraulic barrier, such as a GM, are different. The infinite slope factor of safety for "full flow" (t_w = thickness of water flow parallel to the slope = t in Figure 6-3) parallel to the slope along an internal or interface slip surface <u>above</u> the hydraulic barrier is (Giroud et al., 1995a):

$$FS_A = \frac{\gamma_b}{\gamma_{sat}} \frac{\tan\phi_a}{\tan\beta} + \frac{a_a}{\gamma_{sat}\, t \sin\beta} \qquad \text{(Eq. 6.4)}$$

where: FS_A = factor of safety for critical potential slip surface above the hydraulic barrier (dimensionless); ϕ_a = angle of internal or interface friction for the critical potential slip surface above the hydraulic barrier (degrees); a_a = cohesion (for internal strength) or adhesion (for an interface) for the critical potential slip surface above the hydraulic barrier (Pa); γ_b = average buoyant unit weight of material above the critical potential slip surface (N/m^3); and γ_{sat} = average saturated unit weight of material above the critical potential slip surface (N/m^3); and all other terms are as defined previously. The buoyant unit weight, γ_b, is equal to total unit weight, γ_t, minus the unit weight of water, γ_w.

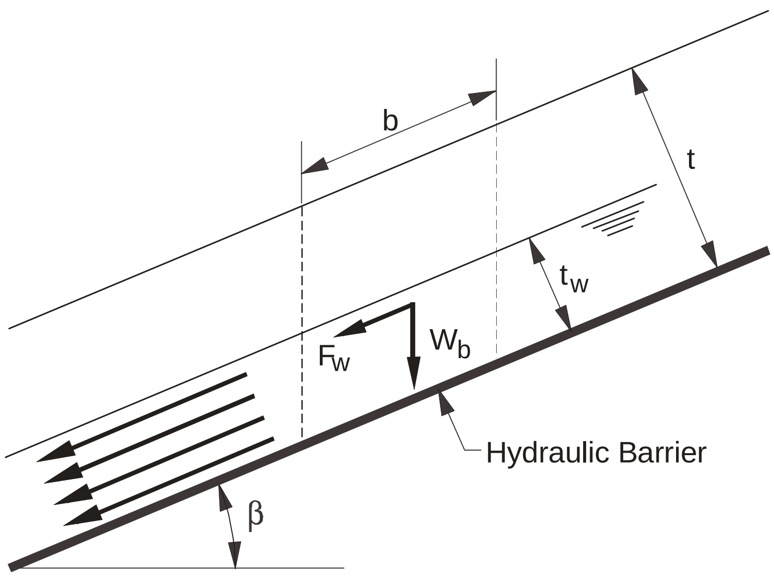


Figure 6-3. Seepage Force and Buoyant Unit Weight for a Soil Layer Overlying a Hydraulic Barrier on an Infinite Slope (modified from Giroud et al, 1995a).

This factor of safety can be compared with the factor of safety expressed by Eq. 6.2 for the case of no water flow. The comparison shows that for typical soils:

- $FS_{A\ full\ flow} / FS_{no\ flow} \approx 0.5$ if $a_i = 0$; and
- $FS_{A\ full\ flow} / FS_{no\ flow} \approx 0.9$ if $\phi_i = 0$.

Based on these results, for slip surfaces located above the hydraulic barrier, the factor of safety can decrease by a factor of two due to water flow parallel to the slope if shearing resistance is generated primarily through friction.

The factor of safety ratios presented above are based on the assumption that the shear strength properties, ϕ_a and a_a, are not influenced by the presence of water. If the presence of water reduces the magnitudes of these parameters, the effects noted in the above comparison would be even more substantial.

The infinite slope factor of safety for "full flow" parallel to the slope along an internal or interface slip surface below a non-porous hydraulic barrier is given by (Giroud et al., 1995a):

$$FS_B = \frac{\tan\phi_b}{\tan\beta} + \frac{a_b}{\gamma_{sat}\, t \sin\beta} \qquad \text{(Eq. 6.5)}$$

where: FS_B = factor of safety for critical potential slip surface below the hydraulic barrier (dimensionless); ϕ_b and a_b are the internal or interface shear strength parameters for the critical potential slip surface below the hydraulic barrier; and all other terms are as defined previously. It should be noted that the shear strength parameters ϕ_b and a_b, used in Eq. 6.5, will typically be different than the parameters ϕ_a and a_a, used in Eq. 6.4, as the interfaces in the two equations are different. This factor of safety is to be compared with the factor of safety expressed by Eq. 6.2 for the case of no water flow. The comparison shows that for typical soils:

- $FS_{B\ \text{full flow}} / FS_{\text{no flow}} = 1$ if $a_i = 0$; and
- $FS_{A\ \text{full flow}} / FS_{\text{no flow}} \approx 0.9$ if $\phi_i = 0$.

Based on these results, the factor of safety along critical potential slip surfaces below the hydraulic barrier is only affected to a relatively minor degree by water flow above the hydraulic barrier.

The final infinite slope case to be considered is for "partial-depth" flow ($t_w < t$ in Figure 6-3) parallel to the slope. The appropriate equations are (Giroud et al., 1995a):

$$FS_A = \left(\frac{\gamma_t (t - t_w) + \gamma_b t_w}{\gamma_t (t - t_w) + \gamma_{sat} t_w}\right) \frac{\tan\phi_a}{\tan\beta} + \frac{a_a / \sin\beta}{\gamma_t (t - t_w) + \gamma_{sat} t_w} \qquad \text{(Eq. 6.6)}$$

and

$$FS_B = \frac{\tan\phi_b}{\tan\beta} + \frac{a_b / \sin\beta}{\gamma_t (t - t_w) + \gamma_{sat} t_w} \qquad \text{(Eq. 6.7)}$$

where: t_w = thickness of water flow parallel to the slope (m), as defined in Figure 6-3, and all other terms are as defined previously.

Based on the foregoing equations, the effect of water flow on the stability of a cover system is much greater if the slip surface is above the hydraulic barrier than if it is below the hydraulic barrier. The reasons for this can be summarized as follows:

- The main effect of water flowing downslope within a cover system slope is the significant decrease in the effective normal stress above the hydraulic barrier.
- Other effects of water flowing downslope within a cover system are a slight increase in the effective normal stress below the hydraulic barrier layer and a slight increase in the shear stress above and below the hydraulic barrier.
- As a result of the changes in effective normal stress, the frictional component of shear strength decreases significantly above the hydraulic barrier but decreases only slightly below the hydraulic barrier.
- As a result of the changes in shear strength and the slight increase in shear stress, the factor of safety is significantly affected above the hydraulic barrier and only mildly affected below the hydraulic barrier.

It can also be inferred from the above assessment that waste-generated gases beneath a cover system effect the stability of the interface between a non-porous hydraulic barrier and an underlying material by decreasing the frictional component of shear strength along the interface while the shear stress along the interface remains unchanged. This is one reason why gases may need to be collected and controlled via a gas collection layer, gas wells, or other means. One example of a cover system stability problem caused by gas pressures is described in Section 7.7. Briefly, gas generated in a MSW landfill uplifted the GM barrier of a cover system and resulted in the GM and overlying materials moving downslope over a GT. Though the landfill had vertical gas extraction wells, the upper portions of the wells were not perforated. As a consequence, gas accumulated beneath the cover system, generating uplift pressures on the underside of the GM.

6.2.2.3 Slope of Finite Length

Equations for the LE evaluation of sloping geosynthetic-soil layered systems (such as a cover system) for a slope of finite length have been presented by Giroud and Beech (1989), EPA (1991), Koerner and Hwu (1991), McKelvey and Deutsch (1991), Bourdeau et al. (1993), Druschel and Underwood (1993), Giroud et al. (1995a,b), Soong and Koerner (1997), Koerner and Daniel (1997), and Koerner and Soong (1998), among others. The most detailed treatments of the subject have been presented by Koerner and coworkers and Giroud et al. (1995a,b). Giroud et al. (1995b) have shown that compared with the method they present, the method utilized by Koerner and coworkers is more rigorous, but somewhat more complicated to use because it requires solution of quadratic equations. The formulation by Giroud et al. (1995a,b) involves an approximation that allows expression of the factor of safety as a closed-form algebraic equation where each term in the equation has a distinct physical meaning and is sufficiently accurate for practical purposes. The simpler formulation is presented below, but either method is acceptable when properly applied.

The two-part wedge considered by Giroud et al. (1995a,b) is illustrated in Figure 6-4. For this condition, the slope stability factor of safety for a slope with constant soil thickness above the critical potential slip surface and for the case of no water flow ($t_w = 0$ in Figure 6-4) is given by:

$$FS = \frac{\tan\phi_i}{\tan\beta} + \frac{a_i/\sin\beta}{\gamma_t\, t} + \frac{t}{h}\left(\frac{\sin\phi_s}{\sin(2\beta)\cos(\beta+\phi_s)}\right) + \frac{c_s}{\gamma_t h}\left(\frac{\cos\phi_s}{\sin\beta\cos(\beta+\phi_s)}\right) + \frac{T/h}{\gamma_t t} \qquad \text{(Eq. 6.8)}$$

where: ϕ_s = angle of internal friction for the soil material (i.e., protection layer and/or granular drainage layer) above the critical potential slip surface (degrees); c_s = cohesion of soil material above the critical potential slip surface (Pa); h = height of slope (m), as defined in Figure 6-4; T = geosynthetic tension above the potential slip surface (N/m); and all other terms are as defined previously.

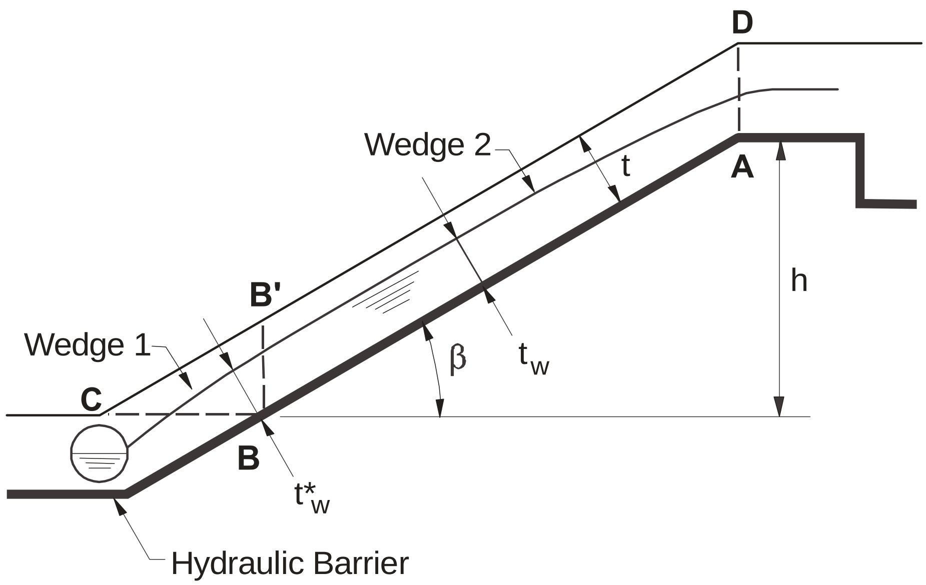


Figure 6-4. Definition of Two-Part Wedge and Flow Thickness for the Case of a Slope of Finite Height (modified from Giroud et al., 1995a).

Eq. 6.8 consists of five terms, each of which has physical significance. The significance of each term is as follows:

- The first term quantifies the contribution of the frictional component of the critical interface or internal shear strength to stability (i.e., the frictional component along line segment AB in Figure 6-4).
- The second term quantifies the contribution of the adhesion component of the critical interface or internal shear strength to stability (i.e., the adhesion component along line segment AB in Figure 6-4).
- The third and fourth terms quantify the contribution of the toe buttressing effect, which results from the shear strength of the soil located at the toe of the slope above the slip surface (i.e., the soil shear strength along line segment BC in Figure 6-4). Both terms depend on the soil internal friction angle, whereas only the fourth term depends on the soil cohesion.
- The fifth term quantifies the contribution to the factor of safety of any tension in the geosynthetics located above the slip surface (which may include one or more

geosynthetics specifically used as reinforcement).

The case of partial-depth and full-depth flow of water for a slope of finite height was addressed by Giroud et al. (1995a). For the case of a slope of uniform thickness above the critical potential slip surface, the factor of safety above the hydraulic barrier may be calculated using the following equation:

$$\begin{aligned} FS_A = & \left(\frac{\gamma_t(t-t_w)+\gamma_b t_w}{\gamma_t(t-t_w)+\gamma_{sat} t_w}\right)\frac{\tan\phi_a}{\tan\beta} + \frac{a_a/\sin\beta}{\gamma_t(t-t_w)+\gamma_{sat} t_w} \\ & + \left(\frac{\gamma_t(t-t_w^*)+\gamma_b t_w^*}{\gamma_t(t-t_w)+\gamma_{sat} t_w}\right)\left(\frac{t}{h}\right)\left(\frac{\sin\phi_s}{\sin(2\beta)\cos(\beta+\phi_s)}\right) \\ & + \left(\frac{c_s t/h}{\gamma_t(t-t_w)+\gamma_{sat} t_w}\right)\left(\frac{\cos\phi_s}{\sin\beta\cos(\beta+\phi_s)}\right) + \frac{T/h}{\gamma_t(t-t_w)+\gamma_{sat} t_w} \end{aligned} \qquad \text{(Eq. 6.9)}$$

where: t^*_w = thickness of water in Wedge 1 (m), as defined in Figure 6-4; and all other terms are as defined previously. For potential slip surfaces below a non-porous hydraulic barrier:

$$\begin{aligned} FS_B = & \frac{\tan\phi_b}{\tan\beta} + \frac{a_b/\sin\beta}{\gamma_t(t-t_w)+\gamma_{sat} t_w} + \left(\frac{\gamma_t(t-t_w^*)+\gamma_b t_w^*}{\gamma_t(t-t_w)+\gamma_{sat} t_w}\right)\left(\frac{t}{h}\right)\left(\frac{\sin\phi_s}{\sin(2\beta)\cos(\beta+\phi_s)}\right) \\ & + \left(\frac{c_s t/h}{\gamma_t(t-t_w)+\gamma_{sat} t_w}\right)\left(\frac{\cos\phi_s}{\sin\beta\cos(\beta+\phi_s)}\right) + \frac{T/h}{\gamma_t(t-t_w)+\gamma_{sat} t_w} \end{aligned} \qquad \text{(Eq. 6.10)}$$

When there is full flow of water in Wedge 1 ($t^*_w = t$) as well as in Wedge 2 ($t_w = t$), Eq. 6.9 gives the following equation for the factor of safety for a critical potential slip surface above the hydraulic barrier:

$$\begin{aligned} FS_A = & \frac{\gamma_b}{\gamma_{sat}}\left(\frac{\tan\phi_a}{\tan\beta}\right) + \frac{a_a}{\gamma_{sat} t\sin\beta} + \frac{\gamma_b}{\gamma_{sat}}\left(\frac{t}{h}\right)\left(\frac{\sin\phi_s}{\sin(2\beta)\cos(\beta+\phi_s)}\right) \\ & + \frac{c_s}{\gamma_{sat} h}\left(\frac{\cos\phi_s}{\sin\beta\cos(\beta+\phi_s)}\right) + \frac{T/h}{\gamma_{sat} t} \end{aligned} \qquad \text{(Eq. 6.11)}$$

and Eq. 6.10 reduces to:

$$\begin{aligned} FS_B = & \frac{\tan\phi_b}{\tan\beta} + \frac{a_b}{\gamma_{sat} t\sin\beta} + \frac{\gamma_b}{\gamma_{sat}}\left(\frac{t}{h}\right)\left(\frac{\sin\phi_s}{\sin(2\beta)\cos(\beta+\phi_s)}\right) \\ & + \frac{c_s}{\gamma_{sat} h}\left(\frac{\cos\phi_s}{\sin\beta\cos(\beta+\phi_s)}\right) + \frac{T/h}{\gamma_{sat} t} \end{aligned} \qquad \text{(Eq. 6.12)}$$

where all terms are as defined previously.

Another case sometimes encountered is that of a tapered cover soil thickness, as illustrated in Figure 6-5. For this geometry, the factor of safety for the case of no water flow is given by the equation:

$$FS = \frac{\tan\phi_i}{\tan\beta} + \frac{t_b}{t_{avg}}\left[\frac{a_i}{\gamma_t t_b \sin\beta} + \frac{t_b}{h}\left(\frac{\sin\phi_s}{\sin(2\beta)\cos(\beta+\phi_s)}\right) + \frac{c_s}{\gamma_t h}\left(\frac{\cos\phi_s}{\sin\beta\cos(\beta+\phi_s)}\right) + \frac{T/h}{\gamma_t t_b}\right] \quad \text{(Eq. 6.13)}$$

where:

$$t_{avg} = (t_a + t_b)/2 \quad \text{(Eq. 6.14)}$$

t_{avg} = average thickness of soil layer between points A and B, which are defined in Figure 6-5 (m); t_a = thickness of soil layer at point A (m), as defined in Figure 6-5; t_b = thickness of soil layer at point B (m), as defined in Figure 6-5; and all other terms are as defined previously.

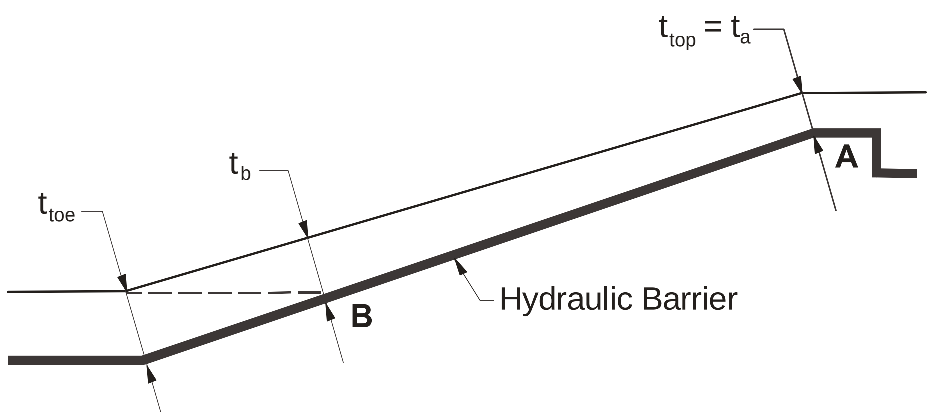


Figure 6-5. Definition of Slope with a Tapered Soil Layer (from Giroud et al., 1995b).

Eq. 6.13 can also be used to calculate the factor of safety for a partly tapered slope of height h, illustrated in Figure 6-6, by calculating an average soil thickness for the entire slope using the equation:

$$t_{avg} = \frac{t_a}{2}\left(1 + \frac{h_u}{h}\right) + \frac{t_b}{2}\left(1 - \frac{h_u}{h}\right) \quad \text{(Eq. 6.15)}$$

where: h_u = height of slope above the slope grade break (m), as illustrated in Figure 6-6, and all other terms are as defined previously.

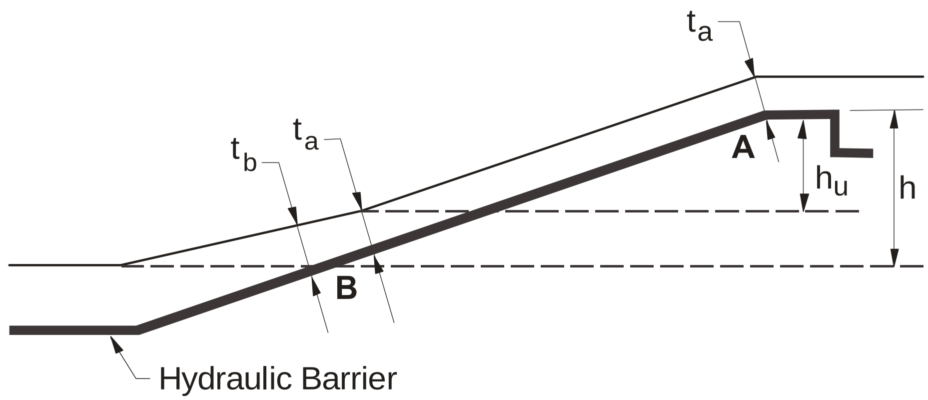


Figure 6-6. Definition of Slope with a Partly Tapered Soil Layer (from Giroud et al., 1995b).

The equations presented above provide closed-form solutions to a variety of cover system slope stability situations. Some situations are too complex, however, to address using closed-form solutions and are more easily evaluated using commercially available two-dimensional slope stability computer software (e.g., PCSTABL5M (Achilleous, 1988), UTEXAS4 (Wright, 1999), XSTABL (Sharma, 1994), or SLOPE/W (available from Geo-Slope Int. Ltd., Alberta, Canada)). Available software has the advantage over closed-form solutions in that it can be applied to non-uniform slope, soil cover, and hydraulic head conditions, and can incorporate a pseudo-static seismic coefficient for use in seismic stability evaluations.

It is noted that the above two-dimensional LE methods are based on a plane-strain condition and do not consider the shear resistance along the two sides of the slide mass that parallel the direction of movement. A two-dimensional analysis, however, is considered appropriate for cover system design because it yields a conservative estimate of the slope stability factor of safety for design geometries encountered in cover systems. The degree of conservatism decreases as the cover system geometry approaches a two-dimensional configuration (i.e., the ratio of cover system slope width to slope length increases). For the majority of cover system geometries, the incremental increase in stability calculated by considering three-dimensional effects will be negligibly small.

The LE method is useful for evaluating cover system stability under most conditions but is subject to several limitations. With the LE method, material and interface shearing resistances are assumed to be independent of displacement. For geosynthetic materials and interfaces, however, mobilized shearing resistance is not constant but increases with increasing displacement to a peak value. For many materials and interfaces, the shear resistance decreases with increasing displacement after reaching the peak, and ultimately reaches a “residual” value (Figure 6-7). This behavior is sometimes referred to as “strain-softening.” In using the LE method, judgment must be applied to the selection of shear strength values for strain-softening materials (i.e., peak, residual, or some other value). The LE method is similarly limited with respect to tension forces in cover system geosynthetic components and, therefore, cannot be used to estimate the magnitude and distribution of stresses and deformations in these components. These limitations of the LE method can be overcome by using another slope stability evaluation method, stress-deformation analyses discussed below in Section 6.2.3.

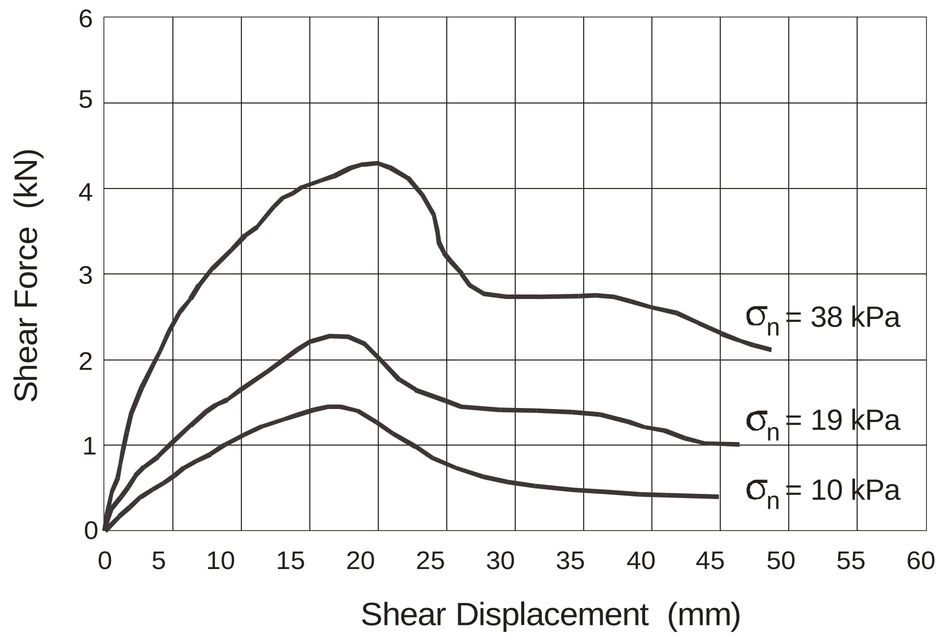


Figure 6-7. Results of Direct Shear Test on a GCL Illustrating Peak and Large-Displacement Shearing Resistances at Different Normal Stresses (σ_n).

As a final comment on the LE equations presented in this section, the equations incorporate terms to account for material internal cohesion or interface adhesion and for geosynthetic tension. Caution should be taken when selecting cohesion or adhesion and geosynthetic tension values for design. As suggested by Koerner and Daniel (1997), cohesion and adhesion values should be used only when there is clear physical justification. From the analysis results presented previously, characterization of internal or interface shearing resistance by a cohesion or adhesion term instead of a friction angle will greatly affect the results of slope stability analyses when hydraulic heads are present in the cover system. In general, geosynthetic tension should not be in the equations unless the design includes a geosynthetic reinforcement layer. Other types of geosynthetics, such as GMs, GNs, GCs, etc., are not designed to permanently transmit tensile loads, are potentially subject to significant tensile creep, and typically have a low tensile modulus (which means that the geosynthetic must elongate to generate tension). Even if geosynthetic reinforcement is used, it should only be relied upon for the tensile force that it can generate at a specified acceptable level of deformation. This acceptable level of deformation must be selected considering the overall performance of all system components.

6.2.3 Stress-Deformation Analyses

Stress-deformation analysis methods may be used for cover system design when the limitations of LE methods are considered significant. The primary advantage of stress-deformation methods is their ability to account for the stress-strain response of materials and interfaces and, therefore, to predict the distribution of stresses and strains within the cover system components, particularly geosynthetic components. Stress-deformation methods can also account for the effects of construction sequencing. The primary disadvantage of stress-deformation methods is the relatively large effort required to obtain material stress-deformation relationships and perform the calculations compared to the effort required with LE methods.

Several studies have been published on the application of stress-deformation methods to cover systems. For example, Long et al. (1993, 1994) described a finite difference model (GEOSTRES) that considers stress equilibrium and strain compatibility. GEOSTRES uses inelastic, non-linear springs to model the shear resistance-displacement behavior at each interface and to model the axial load-displacement behavior within each component. Wilson-Fahmy and Koerner (1992, 1993) adopted a two-dimensional finite element model to account for stress equilibrium and deformation compatibility in stability analyses of soil-geosynthetic systems on slopes.

6.2.4 Shear Strength Parameters

It is recommended that laboratory testing using project-specific materials, coupled with testing procedures and conditions representative of the anticipated field application, be performed to establish design shear strength parameters on a project by project basis. Sabatini et al. (2001) have shown that for a given factor of safety, designs based on project-specific laboratory testing programs are more reliable and less prone to slope instability than designs that utilize shear strength parameters obtained from more general sources, such as databases or the published technical literature.

The various methods used for laboratory shear strength testing of soils are well known and are fully described in a number of geotechnical textbooks and laboratory guides (Lambe, 1951; Holtz and Kovacs, 1981; Bardet, 1997). The most commonly used methods for laboratory shear strength testing of soils are the triaxial compression test and direct shear test.

Currently there are several types of laboratory devices available for the evaluation of shear strength of geosynthetic materials and interfaces. These laboratory devices include:

- large-scale (300 mm x 300 mm) direct shear box specified by ASTM D 5321;
- conventional (50 to 100 mm square or circular) direct shear box with testing generally following ASTM D 3080;
- torsional shear device (ASTM standard under development);
- tilt table; and
- large-displacement shear box.

A summary of the advantages and disadvantages of the first four devices is presented in Table 6-2. Shallenberger and Filz (1996) described the capabilities and limitations of the large-displacement shear box. More recently, Marr (2001) discussed the attributes of test equipment and methods used to evaluate the shear strength of geosynthetic materials and interfaces.

Most project-specific laboratory testing being performed presently uses the ASTM standard 300 mm x 300 mm direct shear box. The large scale of this box is advantageous due to the structure of many geosynthetics, which requires a large test specimen to achieve a representative size of material for testing.

Table 6-2. Summary of advantages and disadvantages associated with test devices for measuring interface shear strength (modified from Gilbert et al., 1995).

Test Device	Advantages	Disadvantages
Large-scale direct shear box	Industry standard Large scale Large displacement Minimal boundary effects	Machine friction Load eccentricity Limited continuous displacement Limited normal stress Expensive
Conventional direct shear box	Large experience base with soil Large normal stress Inexpensive	Machine friction Load eccentricity Small scale Limited displacement Boundary effects
Torsional shear device	Unlimited continuous displacement	Machine friction Anisotropic shearing Small scale Expensive
Tilt table	Minimal machine effects Minimal boundary effects Inexpensive	Small experience base Limited continuous displacement Limited normal stress No post-peak behavior Large effort to prepare sample

Project-specific shear strength testing programs are designed to simulate the anticipated field conditions by selecting appropriate testing procedures and conditions. These include the soil compaction conditions (i.e., water content and density), soil consolidation stress and time, wetting conditions for the materials and interfaces, range of applied normal stresses, direction of shear for geosynthetic interfaces, and shear displacement rate and magnitude. The potential effects of many of these testing conditions on measured interface shear strength parameters are reported in the literature (e.g., Martin et al., 1984; Saxena and Wong, 1984; Bonaparte et al., 1985; Williams and Houlihan, 1987; Seed et al., 1988; Giroud et al, 1990; Seed and Boulanger, 1991; Swan et al., 1991; Pasqualini et al., 1993; Stark and Poeppel, 1994; Bemben and Schulze, 1995; Gilbert et al., 1995; Nataraj et al., 1995; Bonaparte et al., 1996; Gilbert et al., 1996; Shallenberger and Filz, 1996; Stark and Eid, 1996; Stark et al., 1996; Dove et al., 1997; Eid and Stark, 1997; Sharma et al., 1997; Daniel et al. 1998; De and Zimme, 1998; Fox et al., 1998; Sabatini et al. 1998; Snow et al., 1998; Li and Gilbert, 1999; Breitenbach and Swan, 1999). Particular attention should be given to the following:

- Testing should be performed with materials and boundary conditions representative of the anticipated field conditions.
- Soils used in the tests should be compacted to representative field conditions. The compaction moisture content for CCLs used in a direct shear testing program should be near the upper limit of acceptable moisture content and near the lower limit of dry unit weight allowed by the construction specification.
- For GM/CCL interface shear tests, a variety of opinions exist with regard to the application of additional moisture to the interface just prior to assembly of the test

specimen. Options include not adding moisture, lightly or moderately "spritzing" water onto the CCL, or submerging the assembled sample. The rationale for any of these techniques is to simulate suspected installation (e.g., rainfall or moisture conditioning) or post-installation (e.g., condensation collecting at the interface or consolidation-induced water movement to the interface) increases in CCL moisture content at the interface. Counterbalancing these potential mechanisms for moisture content increase at the interface is the effect of thermal gradients typically induced in CCLs beneath GMs prior to covering the GMs with soil. The thermal gradients tends to induce water vapor migration away from the hotter interface and into the underlying cooler soil (Bowders et al., 1997a). Another factor to consider is the post-compaction thixotropic effect identified by Shallenberger and Filz (1996), wherein residual interface shear strengths were found to increase with "curing time" after sample preparation. Given all of these factors, the design engineer must give careful consideration as to the application of additional moisture to the interface just prior to assembly of the test specimen.

- Hydration (soaking) times for GCL samples should be adequate to achieve minimum strength. Daniel and Scranton (1996) showed that hydration times of 24 hours were sufficient for small, 64-mm diameter samples. Koerner and Daniel (1997) noted, however, that complete hydration of relatively large (300 mm x 300 mm) direct shear tests samples takes longer than traditionally required for hydration of soils in relatively small direct shear boxes. Gilbert et al. (1996) reported hydration times, as determined by cessation of GCL swelling under constant normal stress, for reinforced GCLs of up to 25 days. However, Gilbert et al. (1996) used deionized water as the permeating liquid (which increases swell potential), and Daniel et al. (1993) showed that full hydration is not necessary to achieve minimum shear strength. Given this information, an acceptable approach to GCL hydration is to monitor vertical deformation of the GCL and continue to hydrate until these deformations have ceased under the applied normal stress (see discussion of normal stress below). When this procedure cannot be performed, a minimum hydration time of 72 hours is recommended for GCLs to be tested in a 300 mm x 300 mm direct shear box. It should be remembered that without adequate hydration time, the measured GCL strength may be larger than the fully hydrated strength.

- Testing conditions must adequately reflect the field consolidation conditions of the GCL or CCL components. GCLs hydrated as indicated above will be fully consolidated under the normal stress applied during hydration. Consolidation requirements for CCLs may be established using ASTM D 3080. Specimen consolidation times of 48 hours or more may be required for some CCL materials. For both GCLs and CCLs, the normal stress applied during hydration should be equal to the normal stress applied by the cover system in the field, if the full thickness of overlying cover materials is to be placed quickly. Alternatively, a more conservative approach would be to apply a normal stress during hydration equal to only a portion of the overburden stress (e.g., one-third or one-half) that will exist once the cover system is fully constructed. In this latter approach, after hydrating the GCLs, they should be consolidated at the normal stress associated with the full weight of the overlying cover system layers. Under the low normal stresses associated with most cover systems, GCLs will typically swell during hydration.

- ASTM D 5321 and ASTM D 6243 recommend that tests be performed at a minimum of three normal stresses, with each test conducted on a new test specimen. The three selected normal stresses should bracket the normal stress applied by the cover system to the material or interface being tested. This is important because many of the materials used in cover systems exhibit a non-linear relationship between internal or interface shear resistance and normal stress. For cover systems, the applicable range of normal stresses will typically be in the range of about 5 to 40 kPa. Uniformity of normal stress over the entire test specimen must be maintained during hydration, consolidation and shearing so as to avoid stress concentrations.

- Shear displacement rates should be selected considering the type of slope stability analysis to be performed and the types of potentially critical materials or interfaces to be tested. For geosynthetic/geosynthetic interfaces (excluding GCLs), the maximum rate allowed by ASTM D 5321 of 0.08 mm/s will generally be acceptable. For long-term stability conditions where the potentially-critical material or interface includes a CCL or GCL component, the shear displacement rate should be as slow as reasonably achievable; the default shear displacement rate of 0.017 mm/s given in ASTM D 5317 is too fast to achieve drained shearing conditions for CCLs and GCLs. Procedures for estimating shear rates to obtain fully-drained conditions for CCLs are given in ASTM D 3080. Procedures and data for estimating shear rates to obtain fully-drained conditions for GCLs are given in ASTM D 6243. It is noted, however, that it may not be necessary to achieve fully-drained test conditions to obtain test results suitable for long-term analyses. Available data suggest that for design purposes, a shear displacement rate of not more than 0.0005 mm/s will produce test results appropriate for use in slope stability analyses involving GCL materials and interfaces. In contrast, for the evaluation of seismic stability, shear displacement rates should be as fast as reasonably achievable. For both conditions, testing should be performed using samples fully-consolidated under the applied normal stresses.
- Tests should be carried out to a shear deformation adequate to evaluate both the peak and large-displacement shear resistance of the material or interface being tested. Many geosynthetic/geosynthetic and soil/geosynthetic interfaces exhibit very significant post-peak reductions in shear strength (Figure 6-7). ASTM D 5321 states that one should "*run the test until the applied shear force remains constant with increasing displacement.*" To achieve a large-displacement shear condition (defined as a relatively-flat, post-peak shear stress versus shear displacement line) in a direct shear test, shear displacements of 50 mm or more may be necessary. It should also be noted that this large-displacement shear strength is close to, but typically not as low as, the absolute minimum (i.e., residual) shear strength of the material or interface. Residual shear resistances may not occur until shear displacements reach 200 mm, or more. Torsional ring shear testing (Stark and Poeppel, 1994) can be used to evaluate residual shear strengths for soils and geosynthetics for which representative samples can be produced for the small size and torsional shearing mode of this type of test. Alternatively, large-displacement shear box testing (Shallenberger and Filz,1996) can be used to evaluate residual shear strengths for larger-size test specimens in a linear displacement mode. For most practical design applications, true residual strength can be estimated to an acceptable degree of accuracy as 90 to 95% of the large-displacement strength obtained from a 300 mm x 300 mm direct shear test.
- Multi-component cover systems may have more than one potentially-critical slip surface.

The shear test program for a project may need to consider several materials or interfaces.

- Some materials exhibit significant manufacturing variability. For example, the degree of texturing on GMs and the amount of internal-reinforcing in needlepunched GCLs has been observed to vary significantly from lot to lot. This variability should be considered both in design and in the selection of project QC/QA protocols.
- Test results can be interpreted in terms of a secant friction angle that varies as a function of normal stress, or by a tangent friction angle and apparent cohesion (or adhesion for interface strength) applicable to the range of considered normal stresses. Both of these approaches are illustrated in Figure 6-8. For cover system applications, internal and interface shear strength parameters should be defined in terms of a secant friction angle for cases where hydraulic heads could develop in the cover soil. Since the apparent cohesion or adhesion may not be a true material or interface property, the use of this parameter with high heads (relative to the total normal stress) could lead to an over-prediction of the true slope stability factor of safety. Also, as previously mentioned, Koerner and Daniel (1997) suggested that cohesion and adhesion values should be used only when there is clear physical justification.

All of the foregoing factors should be considered in designing a laboratory shear testing program to evaluate internal and interface shear strengths and in using the results of the program in slope stability analyses.

Several other factors may affect long-term shear strength properties of the cover system materials and interfaces. For example, in cold regions, freeze-thaw may reduce the shear strength of cover system CCLs and CCL/geosynthetic interfaces. Research has shown that many CCLs undergo significant change in soil fabric and reduction in shear strength as a result of freeze-thaw cycling (e.g., Nagasawa and Umeda, 1985; Othman et al., 1994). A case study illustrating how this problem contributed to slope failure for a landfill cover system in Ohio is presented in Section 7.4.3. In addition, both heating and cooling result in soil moisture migration, which can cause changes in material and interface, shear strengths (e.g., Daniel et al., 1993). Furthermore, long-term creep may also be significant, particularly in geosynthetic components. No consistent standard of practice presently exists for directly addressing the potential effects of all of these factors on cover system stability. These factors may be indirectly accounted for through the use of higher minimum acceptable factors of safety, when appropriate, or through placement of a greater thickness of cover soil above the critical layers for thermal insulation and isolation from environmental factors.

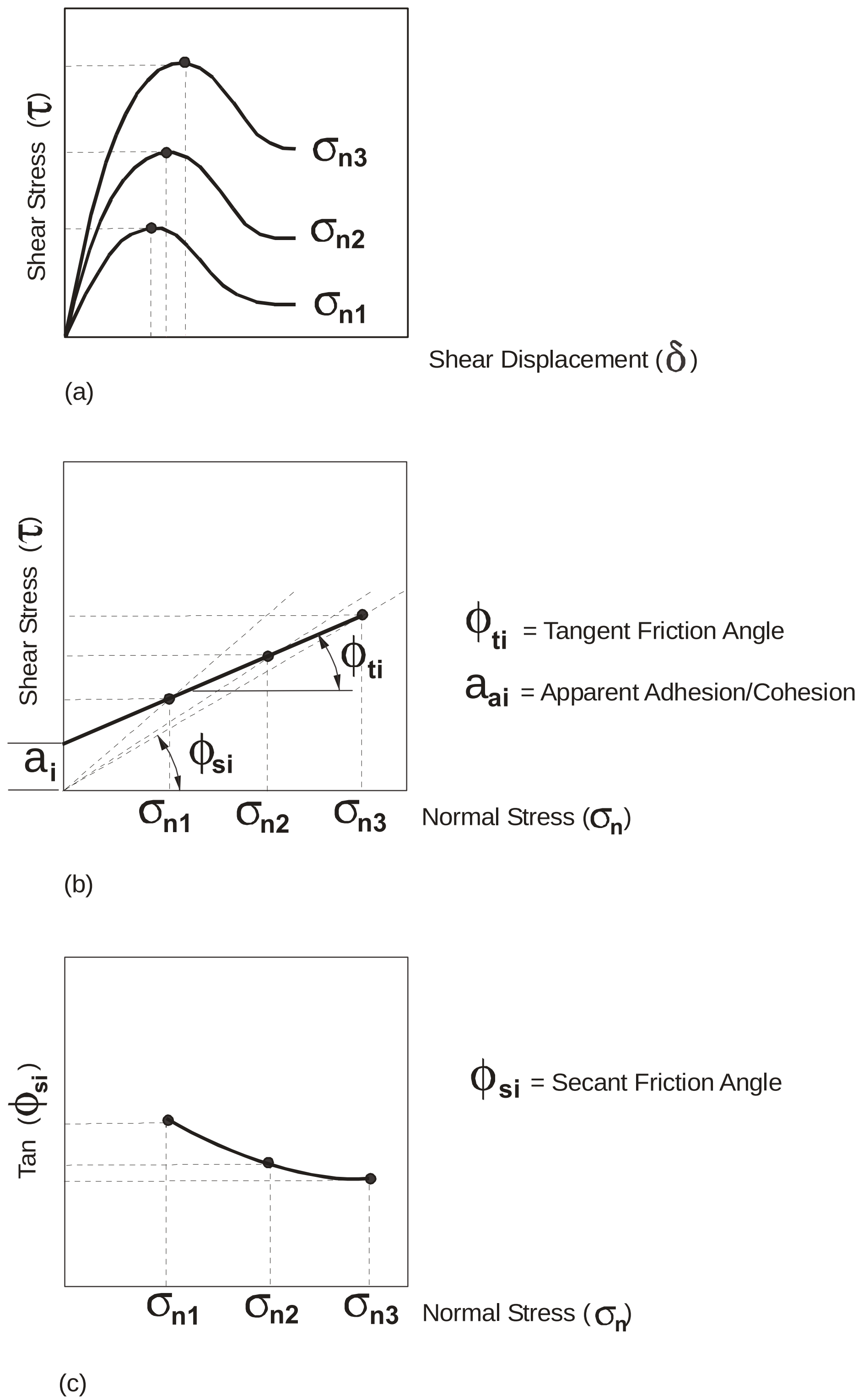


Figure 6-8. Interpretation of Cover System Interface or Internal Shear Test: (a) Test Results for Peak Strengths; (b) Tangent Friction Angle, ϕ_{ti}, and Apparent Adhesion or Cohesion, a_{ai}; and (c) Secant Friction Angle, ϕ_{si}. Similar Interpretations are Applied to Large-Displacement and Residual Conditions.

6.2.5 Construction Considerations

The placement of soil over a slope with underlying low shear strength materials or interfaces will induce shear stresses that can reduce slope stability. These shear stresses result from the operation of construction equipment on the slope, the weight of the soil, and, if the soil is pushed down the slope, from the moving soil itself. Construction-induced stresses have been investigated by McKelvey and Deutsch (1991) and Koerner and Daniel (1997). These references present closed-form LE equations that can be used to evaluate the effect of construction equipment operation on cover system stability. The clear recommendation that comes out of these investigations is that cover soils should be placed over low shear strength materials and interfaces from the bottom of the slope upward and not from the top of the slope downward (Figure 6-9).

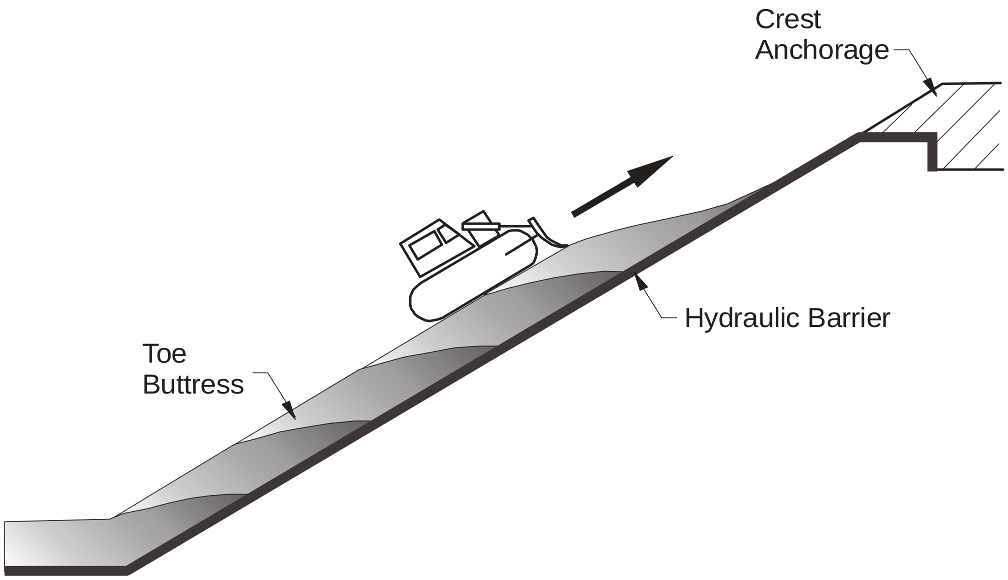
The following comments are provided with respect to placement of soil materials in cover systems:

- By placing cover soils from the bottom of the slope upward, a passive, stabilizing soil wedge is established at the toe of slope prior to placement of soil higher on the slope. The operation of construction equipment over this lower wedge tends to compact and strengthen the wedge.
- Relatively small, wide-track dozers (i.e. low-ground pressure dozers) are recommended for placing the soil cover material. This type of equipment limits both the dynamic force imparted to the slope during acceleration and braking and the tractive force applied through the dozer tracks.
- Downslope dynamic forces can be limited further by limiting the dozer speed on the slope and by instructing the dozer operator to avoid hard breaking, particularly when backing downslope.

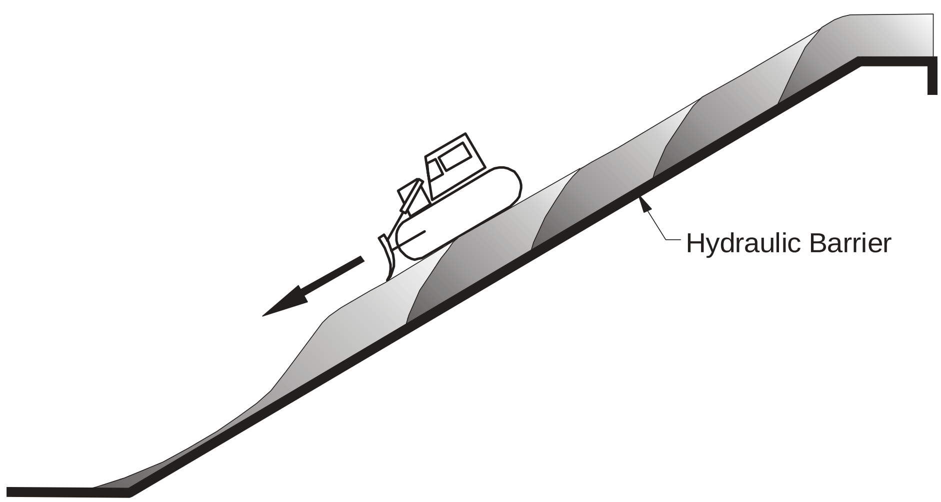
By application of the construction procedures described above, construction-induced impacts to the stability of a cover system slope (designed to conventional slope stability factors of safety described next in Section 6.2.6) are minor. For other conditions (e.g., lower factors of safety than recommended in Section 6.2.6, placement of soil from the top of slope downward, use of large construction equipment) construction-stage stability should be checked using the procedures described by McKelvey and Deutsch (1991) or Koerner and Daniel (1997).

6.2.6 Factors of Safety

LE analysis methods provide a calculated slope stability factor of safety (FS). Minimum acceptable FS values for cover systems depend on project-specific conditions and uncertainties. For example, when cover systems include strain-softening materials or interfaces, differing minimum factors of safety are often applied to peak strength analyses and analyses based on large-displacement or residual strength. Other criteria may also influence selection of a minimum acceptable FS, including regulatory requirements, reliability of laboratory test methods, similarity between laboratory testing conditions and field conditions, completeness of laboratory test data, uncertainty with respect to other design input parameters (e.g., unit weights, hydraulic heads, geometry), and consequences of slope failure.



(a)



(b)

Figure 6-9. Cover Soils Should be Placed Over Low Shear Strength Materials and Interfaces from the Bottom of the Slope Upward (a) and not from the Top of the Slope Downward (b).

Previous agency guidance on selecting slope stability factors of safety was given in EPA (1988). The FS values given in the 1988 document were meant to apply to excavation and embankment (soil) slopes used in the construction of landfills and surface impoundments. As the reported FS values represent general guidance, however, they have sometimes been cited as criteria for the design of cover systems. The values from EPA (1988) are given in Table 6-3 below.

Table 6-3. Previously-recommended minimum FS values (modified from EPA, 1988).

Consequences of Slope Failure	Uncertainty of Strength Measurements	
	Small[1]	*Large*[2]
No imminent danger to human life or major environmental impact if slope fails	1.25	1.5
Imminent danger to human life or major environmental impact if slope fails	1.5	2.0 or greater

[1] The uncertainty of the strength measurements is smallest when the soil conditions are uniform and high quality strength test data provide a consistent, complete, and logical picture of the strength characteristics.
[2] The uncertainty of the strength measurements is greatest when the soil conditions are complex and when available strength data do not provide a consistent, complete, or logical picture of the strength characteristics.

Duncan (1992), in a state-of-the-art paper on slope stability of soils, provided the following discussion on the selection of an appropriate factor of safety:

> "*Criteria for acceptable values of safety factor should be established with two important considerations in mind. These are, (1) what is the degree of uncertainty involved in evaluating the conditions and shear strengths for analysis, and (2) what are the possible consequences of failure? When the uncertainty and the consequences of failure are both small, it is acceptable to use small factors of safety, on the order of 1.3 or even smaller in some circumstances. When the uncertainties or the consequences of failure increase, larger factors of safety are necessary. Large uncertainties coupled with large consequences of failure represent an unacceptable condition, no matter what the calculated value of the factor of safety. Typical minimum acceptable values of factor of safety are about 1.3 for end-of-construction and multi-stage loading, 1.5 for normal long term loading conditions, and 1.0 to 1.2 for rapid drawdown, in cases where rapid drawdown represents an improbable or infrequent loading condition.*"

While the guidance was developed by Duncan for soil slopes, the philosophy on FS selection is directly applicable to the design of cover systems for waste containment applications and is generally consistent with Table 6-3.

More recently, Koerner and Soong (1998) presented recommendations on FS selection that incorporate a similar philosophy and that are specific to cover systems. The first step in the approach suggested by Koerner and Soong (1998) is to qualitatively classify the project as critical or non-critical and temporary or permanent. This qualitative classification is adapted from Bonaparte and Berg (1987), who suggested its use for geosynthetic reinforcement in highway applications. With this classification system, a critical application is one in which the consequences of failure include a potential for personal injury, significant property damage, or significant environmental release of contaminants. In contrast, a non-critical classification would apply to a cover system of limited extent that could be readily repaired (e.g., a monolithic soil cover) and for which the consequences of failure are minor. Table 6-4 presents the qualitative classification system proposed by Koerner and Soong (1998). The classification system is used to assign a ranking (low, moderate, or high) to the cover system so that the appropriate FS value can be selected.

Table 6-4. Qualitative classification for cover system applications (Koerner and Soong (1998)).

Concern	Duration	
	Temporary	*Permanent*
Noncritical	Low	Moderate
Critical	Moderate	High

Based on the cover system rankings in Table 6-4, Koerner and Soong (1998) recommended the minimum static slope stability FS values for cover systems given in Table 6-5. Koerner and Song (1998) suggested lower FS values for non-hazardous (principally MSW) landfills as compared to hazardous waste landfills due to the differences in waste characteristics and the larger magnitude of post-closure settlements for MSW landfills compared to hazardous waste landfills (which results in an appreciable flattening of the MSW cover system slopes with time). For Table 6-5, Koerner and Soong (1998) considered remediation waste piles to primarily consist of low-hazard materials such as construction and demolition wastes and mine wastes. Abandoned dumps on the other hand were considered to include CERCLA remediation sites and other sites containing potentially-hazardous wastes or unknown wastes. Hence, abandoned dumps were considered to pose a higher hazard than either non-hazardous waste landfills or remediated waste piles.

Liu et al. (1997) have suggested that factors of safety for design of cover systems be selected by a multi-step process that involves:

- estimating the mean value, standard deviation, coefficient of variation, and correlation coefficient (between parameters) for each variable in the slope stability analysis;
- calculating failure probabilities and correlating these probabilities to the LE factor of safety for the potential ranges in parameter values; and
- defining an acceptable probability of failure based on the cost and consequences of failure.

Table 6-5. Minimum FS values for static slope stability of cover systems recommended by Koerner and Soong (1998).

	Type of Waste			
Ranking	*Remediated waste piles*	*Non-hazardous waste landfills*	*Abandoned dumps*	*Hazardous waste landfills*
Low	1.2	1.3	1.4	1.4
Moderate	1.3	1.4	1.5	1.5
High	1.4	1.5	1.6	1.6

The approach described by Liu et el. (1997) provides a rational, probability-based approach to designing safe cover system slopes. It is recognized, however, that not all engineers are comfortable with probabilistic approaches and that the standard of practice is to use deterministic methods to establish factors of safety. At a minimum, however, Liu et al. (1997) provides a useful framework for the design engineer to systematically consider the areas of uncertainty and consequences of failure in the project.

A number of technical publications have addressed the issue of FS selection as it relates to the use of peak versus large-displacement (or residual) internal or interface shear strength where the large displacement strength of the material or interface is smaller than the peak shear strength (Byrne, 1994; Stark and Poeppel, 1994; and Bonaparte et al., 1996).

The foregoing discussion should make it clear that there is no single value of FS applicable to all situations. Selection of a FS value for a particular project is a key design decision that should be the responsibility of the design engineer. Based on the foregoing discussion, the following general guidance is given. This guidance applies specifically to cover systems, where the geometry is well defined and the mass being analyzed consists entirely of manufactured or constructed materials placed under controlled conditions. These minimum FS recommendations may not be appropriate for other applications.

- A minimum acceptable factor of safety (FS_{min}) for static stability analyses of 1.5 will often be appropriate for permanent cover system applications where the design is based on peak internal and interface shear strengths conservatively established using project-specific interface direct shear tests, two-dimensional limit equilibrium slope stability analyses, and appropriate consideration of the potential for internal hydraulic head build-up during the representative design storm events. This FS_{min} is applied to normal operating conditions (e.g., no seismic loading or live loading).
- A smaller or larger FS_{min} may be considered based on an evaluation of: (i) consequences of cover system failure; and (ii) uncertainty associated with each design parameter.
- If the cover system contains geosynthetic materials that exhibit strain-softening internal or interface shear strengths, FS_{min} for large-displacement conditions should also be checked. A FS_{min} of 1.2 is suggested where large-displacement shear strengths have been conservatively established using project-specific interface direct shear tests conducted in accordance with ASTM D 5321. For purposes of this evaluation, 50 to 75 mm of displacement, coupled with the observation that the shear stress-displacement plot is essentially flat at the end of the test, is considered to satisfy the large-displacement condition. If true residual shear strengths are obtained using either a torsional-ring or large-displacement shear apparatus, FS_{min} values as low as 1.15 may be considered.
- Cover system designs should be checked for low-probability extreme loading conditions. These conditions need to be identified on a case-by-case basis, but may include extreme storm events, live loads, or earthquakes. Design for earthquakes is addressed subsequently. Values of FS_{min} for extreme loading conditions may, in general, be lower than those associated with the representative design conditions as described above. FS_{min} values for these conditions should be selected on a case-by-case basis.

If FS_{min} cannot be achieved for a given set of conditions, there are a variety of measures that can be taken to increase its value. Examples of these measures are listed in Table 6-6.

Table 6-6. Engineering measures to increase cover system slope stability factor of safety.

Use cover system materials that have higher internal or interface shear strengths, as available
Provide for a flatter cover system slope by initially placing waste to a flatter slope (for new facilities) or waste excavation (for existing facilities) (Figure 6-10)
Shorten the slope length through the use of benches or berms
Use perimeter retaining walls or buttresses to achieve a flatter cover system slope angle (Figure 6-11)
Improve cover system internal drainage if hydraulic head buildup is predicted to occur
Utilize geosynthetic reinforcement, but only within the limitations of this approach described in this chapter

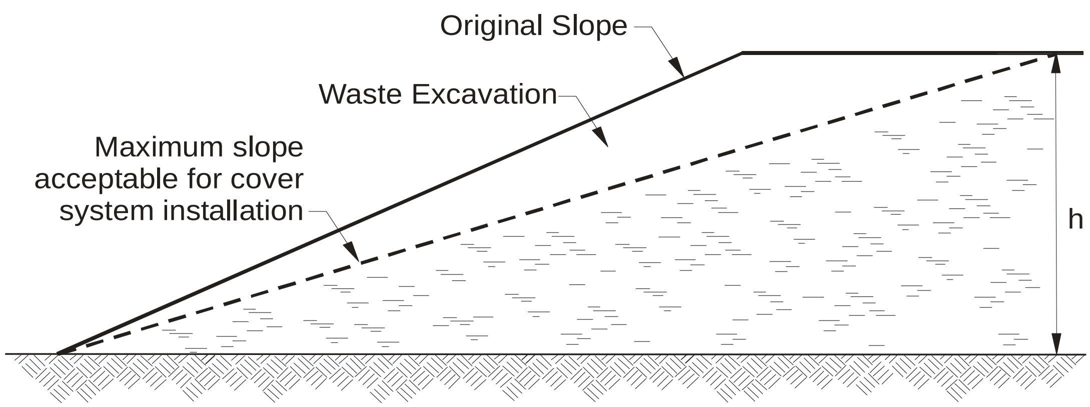


Figure 6-10. Waste Excavation Approach for Constructing Cover Systems Over Steep Waste Slopes.

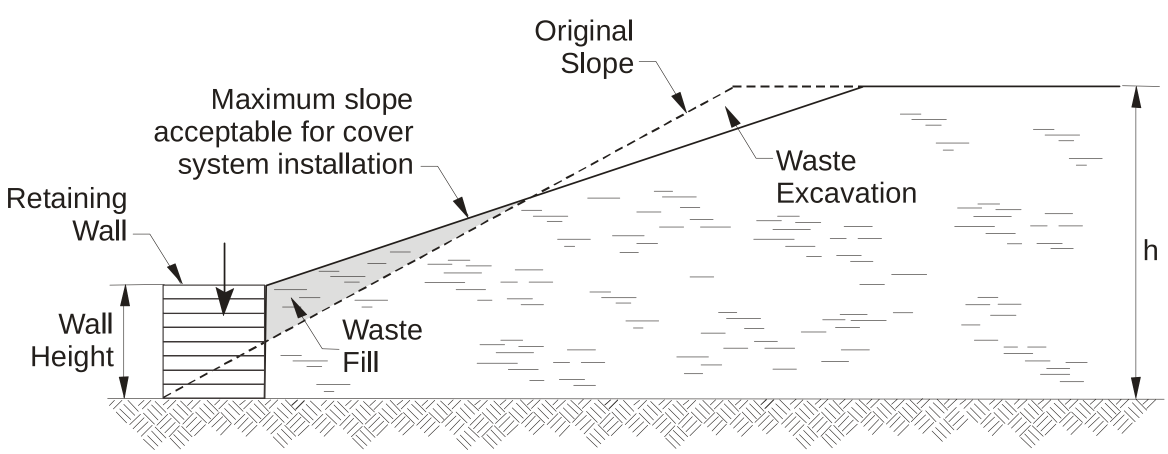


Figure 6-11. Buttress Approach for Constructing Cover Systems Over Steep Waste Slopes Without the Need for Significant Waste Excavation.

6.3 Seismic Slope Stability and Deformation

6.3.1 Overview

The cover system for a landfill or other waste containment unit or for a remediation site may be subject to damage as a result of strong ground accelerations that can accompany an earthquake. Impacts may involve either excessive seismic displacement of one or more of the cover system components or complete instability of the cover system. For most situations, peak seismic accelerations in a cover system will be larger than in the surrounding free field, due to amplification of the ground movements by the underlying waste.

State and federal regulations have various requirements with respect to the evaluation of the potential impact of seismically-induced ground motions on cover systems. EPA regulations for hazardous waste landfills (40 CFR §264 and §265) are silent with respect to seismic design and performance criteria. EPA seismic regulations for MSW landfills, contained in 40 CFR §258.14, require that "*all containment structures, including liners, leachate collection systems and surface water control systems, are designed to resist the maximum horizontal acceleration in lithified earth material for the site*" if the landfill is located in a "seismic impact zone". EPA defines a seismic impact zone as "*an area with a ten percent or greater probability that the maximum horizontal acceleration in lithified earth material, expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10g in 250 years*." EPA further elaborates that the "*maximum horizontal acceleration in lithified earth material means the maximum expected horizontal acceleration depicted on a seismic hazard map, with a 90 percent or greater probability that the acceleration will not be exceeded in 250 years, or the maximum expected horizontal acceleration based on a site-specific seismic risk assessment.*" While this regulation does not explicitly mention cover systems, EPA considers the cover system to be part of a landfill "containment structure" and therefore covered by the regulation. However, the agency recognizes that although difficult and potentially costly, cover systems can be repaired if damaged. In contrast, landfill bottom liner systems generally cannot be repaired once covered with waste. As a consequence, the agency believes that seismic performance criteria (e.g., acceptable FS or magnitude of permanent seismic deformation) applicable to cover systems may not always need to be as stringent as those applied to landfill bottom liner systems.

The EPA guidance document entitled "*RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities*" (Richardson et al., 1995) presents available information and analysis methods to evaluate the seismic performance of landfills. The information contained in this guidance document is consistent with and includes new information that has become available after publication of the EPA document listed above.
Evaluation of the seismic stability of a cover system involves four steps, each of which can be performed using either conservative, simplified approaches, or more complex, detailed analyses. These four steps, which are discussed in more detail below, are as follows:

1. conduct a seismic hazard evaluation to estimate peak horizontal bedrock accelerations for a site and representative causative earthquake events to associate with that acceleration (Section 6.3.2);
2. perform a seismic response analysis to evaluate peak horizontal accelerations at the ground surface or in the waste mass cover system due to the causative earthquake events

(Section 6.3.3);

3. select dynamic shear strength properties for cover system materials and interfaces to use in seismic slope stability and/or deformation analyses (Section 6.3.4); and
4. perform seismic slope stability and deformation analyses (Section 6.3.5).

6.3.2 Seismic Hazard Evaluation

The objective of a seismic hazard evaluation is to characterize the design earthquake with respect to the parameters required for engineering analysis (e.g., magnitude, style of faulting, site-to-source distance, peak ground acceleration, and spectral accelerations). The peak horizontal bedrock acceleration at a project site may be estimated using seismic hazard probability maps or site-specific seismic hazard assessments. The most commonly used maps in the U.S. are those developed by the U.S. Geological Survey (USGS) depicting peak and spectral horizontal bedrock accelerations with 10, 5, and 2% probabilities of exceedance in 50 years (corresponding, respectively, to a 90% probability of not being exceeded in 50, 100, and 250 years). These maps, which can be downloaded from the USGS website (http://geohazards.cr.usgs.gov/eq/index.html), are periodically updated to reflect recent developments in the field of seismology. Background information on the development of these maps is provided by Frankel et al. (1996). Figure 6-12 presents the U.S. national map for peak horizontal acceleration in bedrock with a 90% probability of not being exceeded in 250 years. A map for California and Nevada is presented in Figure 6-13. These maps are included in this guidance document because the probability-recurrence relationship for these maps corresponds to the EPA regulatory criterion for seismic design of MSW landfills.

Seismic hazard maps like those of USGS discussed above usually present the estimated free-field peak horizontal acceleration for a hypothetical bedrock outcrop on level ground at a particular location. If bedrock is not present at or near the ground surface, the peak acceleration may need to be modified to account for local site conditions. The presence of a waste mass will further modify the earthquake ground motions, as discussed subsequently. The primary difficulty associated with using seismic probability maps is that the maps by themselves do not provide information on the magnitude, site-to-source distance, or duration of the earthquake associated with the map acceleration values. For some types of seismic analyses, information on these variables is necessary. Because they are probabilistically derived, the acceleration values provided on such maps are composed of contributions of earthquakes of many different magnitudes from several to many different seismic sources. Each source may be associated with a different site-to-source distance and each magnitude-distance combination with a different duration. The USGS website has recently made available information on the distribution of earthquake magnitudes and site-to-source distances associated with the bedrock accelerations obtained directly from the USGS seismic hazard maps. Using this feature, the peak bedrock acceleration for a given site with a 2% probability of exceedance in 50 years (90% probability of not being exceeded in 250 years) is deaggregated by earthquake magnitude and site-to-source distance. Deaggregated spectral accelerations are also provided for spectral periods of 0.2, 0.3, and 1 second. USGS currently provides deaggregated data for 64 central and eastern U.S. cities and 56 western U.S. cities. As an example of the information available at the USGS website,

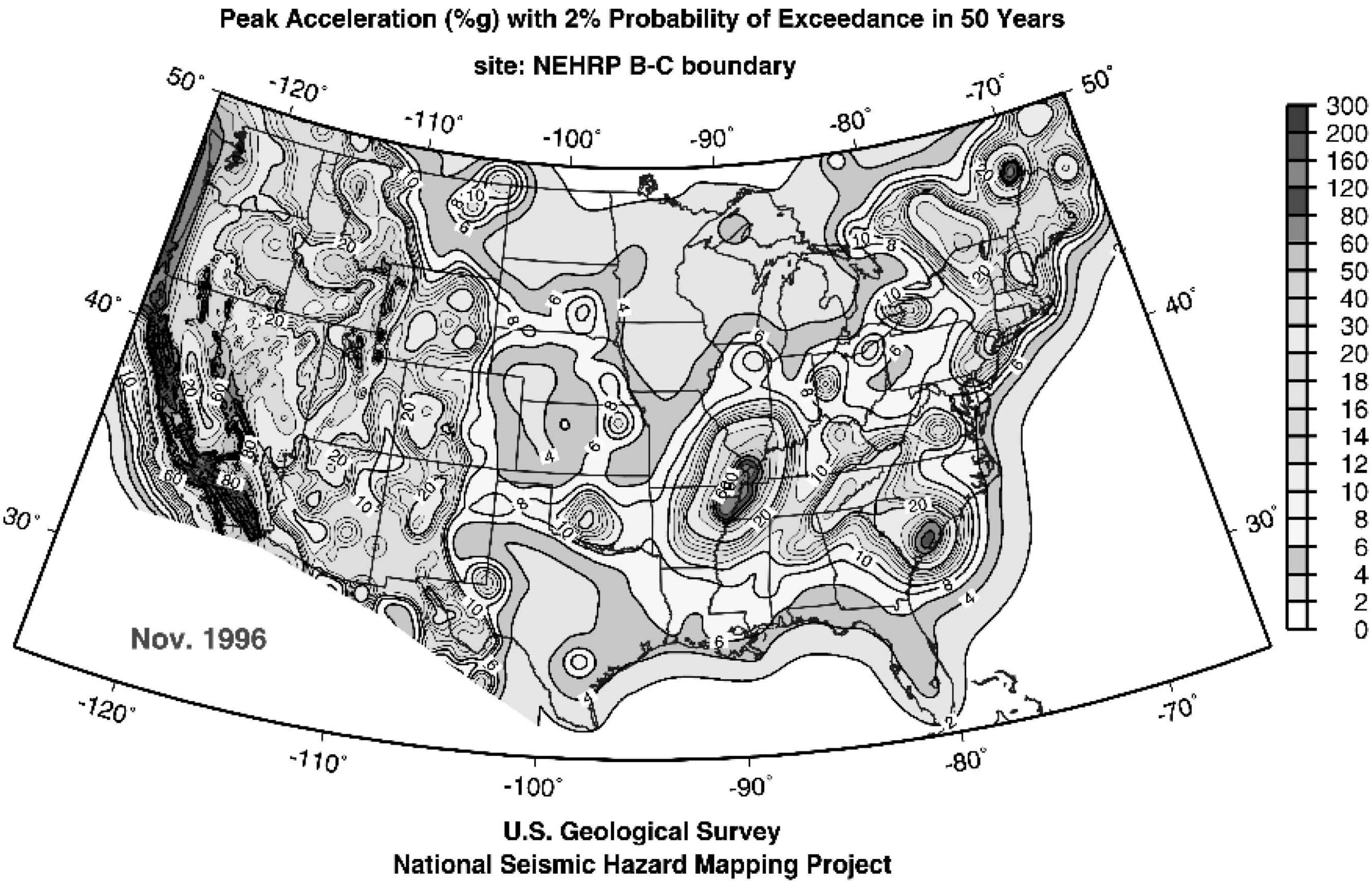


Figure 6-12. Seismic Hazard Probability Map for the U.S. for Peak Horizontal Acceleration in Bedrock with a 90% Probability of not Being Exceeded in 250 Years (downloaded from http://geohazards.cr.usgs.gov/eq/).

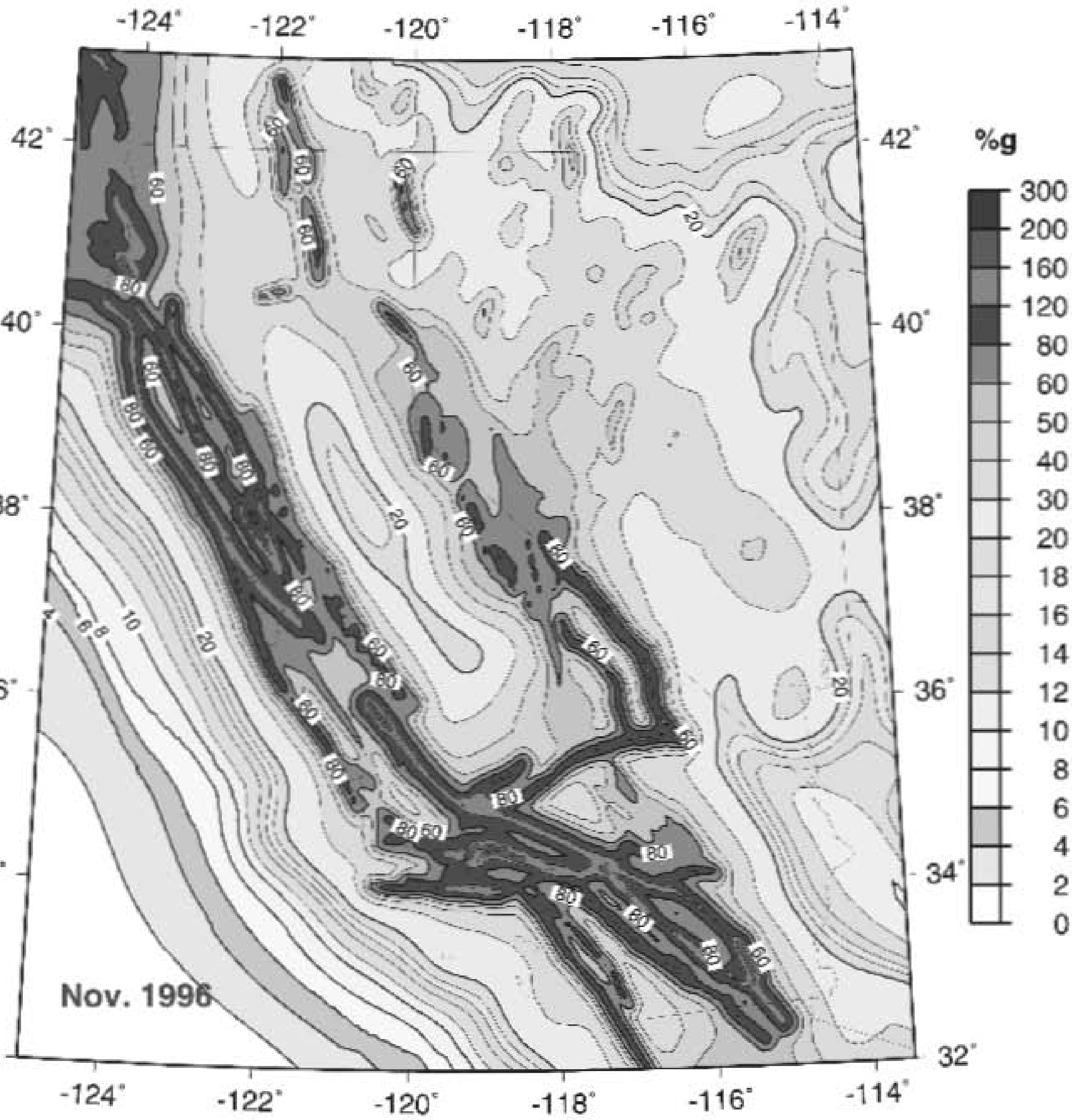


Figure 6-13. Seismic Hazard Probability Map for California and Nevada for Peak Horizontal Acceleration in Bedrock with a 90% Probability of not Being Exceeded in 250 Years (downloaded from http://geohazards.cr.usgs.gov/eq/).

deaggregated bedrock acceleration and site-to-source distance data for Evansville, Indiana are presented in Table 6-7.

In using deaggregated data, such as given in Table 6-7, the engineer should identify the earthquake magnitude and distance combination that encompasses about two-thirds of the seismic hazard. For example, if more than two-thirds of the seismic hazard for a given site is from a small magnitude, near-source earthquake, then seismic analyses should be performed using input variables (e.g., strong motion records) appropriate for this type of earthquake event. In some cases, more than one combination of earthquake magnitude and source distance may need to be considered. The values in Table 6-7 for Evansville illustrate such a case: a significant portion (≈ 40%) of the seismic hazard for Evansville is derived from earthquakes less than 25 km from the site with magnitudes between 5 and 6 (though the magnitude of some of the local events contributing to the seismic hazard may be as great as 7.5). However, over 20% of the seismic hazard is from a distant earthquake more than 150 km from the site with a magnitude of 8.0. Therefore, for some projects in Evansville, the impact of both local and distant events may warrant consideration.

Table 6-7. Deaggregated peak horizontal bedrock accelerations as a percentage of the aggregated peak probabilistic acceleration of 0.328 g for Evansville, Indiana, for a 2% probability of exceedance in 50 years (modified from USGS website).

Hypocentral Distance (km)	Earthquake Magnitude						
	5.0	*5.5*	*6.0*	*6.5*	*7.0*	*7.5*	*8.0*
25	13.888	13.372	9.591	5.787	2.447	1.584	0.000
50	1.507	3.176	4.624	5.054	2.818	2.440	0.000
75	0.093	0.349	0.927	1.737	1.449	1.757	0.000
100	0.011	0.060	0.247	0.670	0.591	0.938	0.000
125	0.002	0.017	0.095	0.348	0.250	0.488	0.000
150	0.000	0.004	0.030	0.143	0.114	0.269	0.000
175	0.000	0.001	0.008	0.050	0.051	0.146	15.718
200	0.000	0.000	0.002	0.019	0.023	0.083	6.904
225	0.000	0.000	0.001	0.007	0.010	0.042	0.000
250	0.000	0.000	0.000	0.003	0.004	0.020	0.000
275	0.000	0.000	0.000	0.001	0.002	0.012	0.000
300	0.000	0.000	0.000	0.000	0.001	0.006	0.000
325	0.000	0.000	0.000	0.000	0.000	0.003	0.000
350	0.000	0.000	0.000	0.000	0.000	0.002	0.000
375	0.000	0.000	0.000	0.000	0.000	0.002	0.000
400	0.000	0.000	0.000	0.000	0.000	0.001	0.000
425	0.000	0.000	0.000	0.000	0.000	0.001	0.000

As a means to reduce uncertainty and increase accuracy, site-specific seismic hazard analyses may be preferred to seismic hazard maps for assessing the seismic hazard to critical structures in regions of high seismic activity. A site-specific seismic hazard analysis involves:

- identification of the seismic sources capable of strong ground motions at the project site;
- evaluation of the seismic potential for each capable source; and
- evaluation of the intensity of the design ground motions at the project site.

Site-specific seismic hazard analyses may be performed using either a deterministic or probabilistic approach. Detailed discussion of this topic is beyond the scope of this chapter. The reader is referred to Reiter (1990), Krinitzsky et al. (1993), Richardson et al. (1995), and Kramer (1996). An example of a site-specific seismic hazard analysis applied to a landfill site (including cover system) in California is given in Kavazanjian et al. (1995a).

6.3.3 Seismic Response Analysis

6.3.3.1 Introduction

The seismic hazard assessment as discussed above provides an estimate of peak horizontal accelerations in bedrock for a given site along with information on the causative earthquake event(s). A response analysis is used to estimate the seismically-induced motions (e.g., acceleration, velocities, and/or displacements) at the ground surface or in the waste mass cover system. Response analyses are needed because soil layers and waste modify the bedrock motions, sometimes in a manner that can significantly increase damage potential.

6.3.3.2 Material Properties Selection

The first step in the seismic response analyses is to characterize the soil and waste material properties needed to perform the analysis. For equivalent linear analyses of vertically-propagating shear waves (the most common type of seismic response analysis performed for geotechnical and waste management applications), these properties include total unit weight, dynamic shear modulus, and damping ratio for each material through which the waves propagate. Kramer (1996) provided an extensive review of the available technical literature on the selection of soil and rock properties for response analyses. Guidance on selecting MSW waste properties can be found in Sharma et al. (1990), Fassett et al. (1994), Richardson et al. (1995), Kavazanjian et al. (1995b), Kavazanjian and Matasovic (1995), and Matasovic and Kavazanjian (1998).

Shear modulus reduction factor and damping ratio curves for the Operating Industries Inc. (OII) site, a large inactive MSW and IW landfill in Monterey Park, California, were developed independently by Idriss et al. (1995), Augello et al. (1998), and Matasovic and Kavazanjian (1998). The curves proposed by these three sets of investigators are shown in Figure 6-14. These curves represent the most reliable information currently available for use in estimating strain-dependent dynamic shear modulus reduction factors, G/G_{max} (dimensionless), and damping ratios for MSW and other solid waste materials for use in equivalent linear response analyses. The strain-dependent damping ratio is obtained directly from Figure 6-14. The dynamic shear modulus, G (Pa), is obtained by multiplying the shear modulus reduction value from Figure 6-14 by the maximum small-strain dynamic shear modulus, G_{max} (Pa), which can be calculated using the equation:

$$G_{max} = \frac{\gamma_{t,waste} v_{s,waste}^2}{g} \qquad \text{(Eq. 6.16)}$$

where: $v_{s,\,waste}$ = shear wave velocity of waste (m/s); $\gamma_{t,\,waste}$ = total unit weight of waste (N/m^3); and g = acceleration of gravity (m/s^2). The small-strain shear modulus is ideally obtained from project-specific field testing. For landfills, this type of testing may be performed with the non-

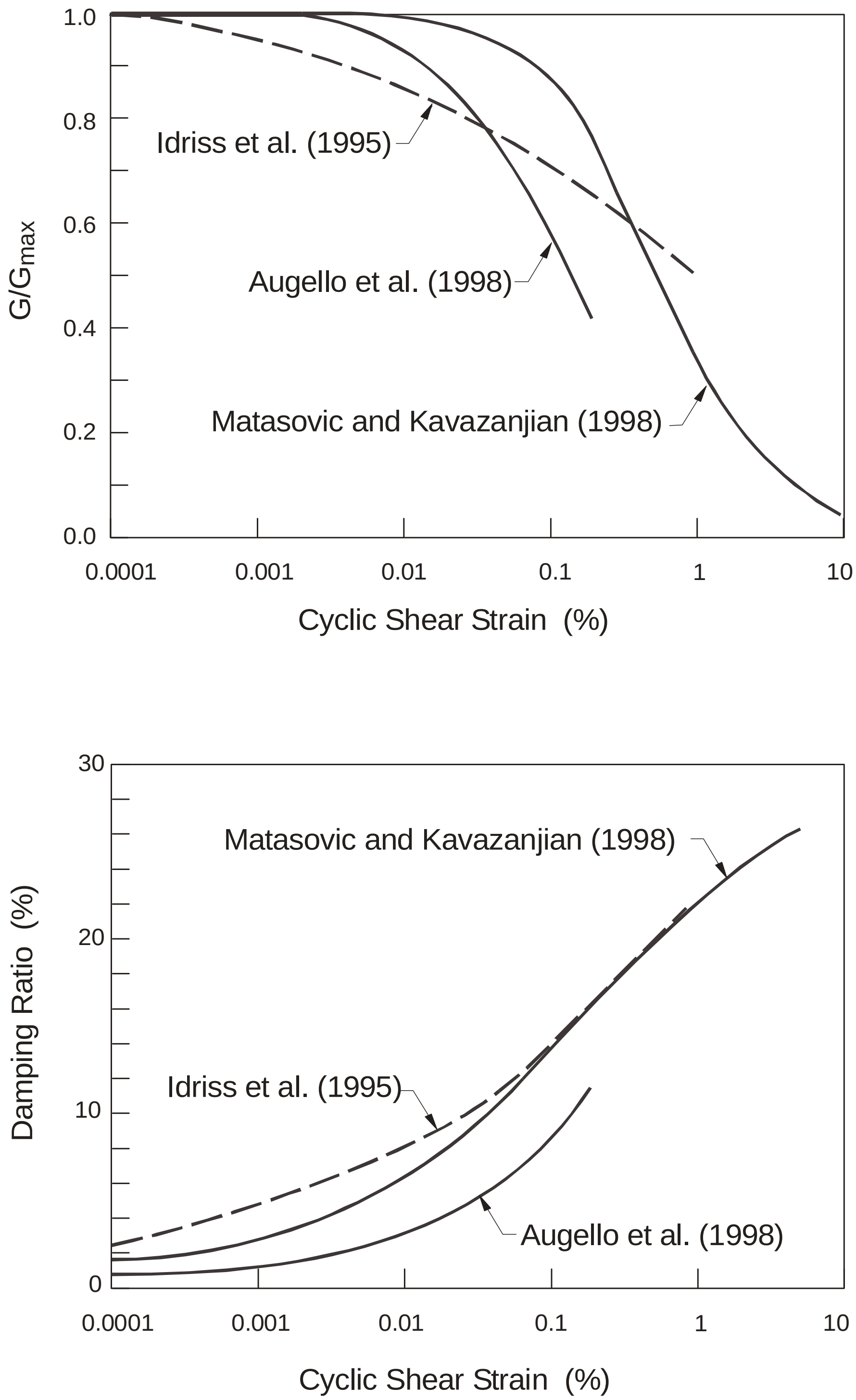


Figure 6-14. Estimated Shear Modulus Reduction Factor and Damping Ratio Curves for the OII Landfill, California (modified from Idriss et al.,1995; Augello et al, 1998; and Matasovic and Kavazanjian, 1998).

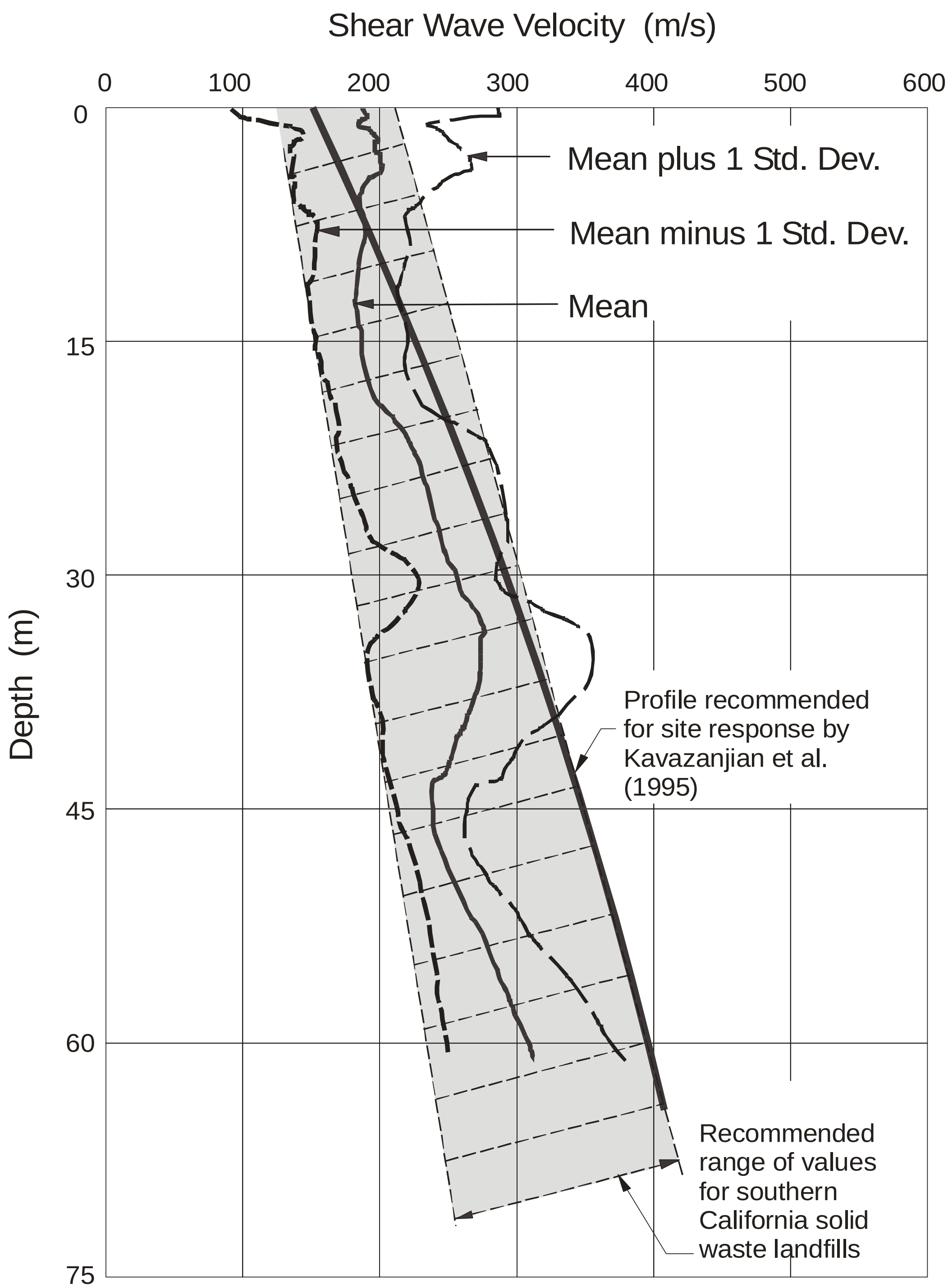


Figure 6-15. Shear Wave Velocities for Southern California Solid Waste Landfills (modified from Kavazanjian et al., 1996).

intrusive spectral analysis of surface waves (SASW) technique (Kavazanjian et al. 1994, 1996). Intrusive downhole or cross-hole geophysical testing techniques may also be used. In the absence of project-specific testing, the data for southern California landfills from Kavazanjian et al. (1996), presented in Figure 6-15, can be used. It is noted that results obtained from a limited amount of SASW testing of MSW landfills in the eastern U.S. (unpublished) suggests that shear wave velocities for waste in these facilities may be lower, on average, than shear wave velocities for waste in relatively dry southern California landfills. In the absence of better information, the lower portion of the recommended range of shear wave velocities shown in Figure 6-15 can be used for MSW landfills located in the eastern U.S and other temperate to wet climates.

6.3.3.3 Simplified Response Analysis

Simplified approaches to seismic response analyses involve the empirical correlation of peak horizontal waste mass or cover system acceleration, as applicable, to peak bedrock acceleration. Correlations of this type were first used in geotechnical engineering to relate peak ground accelerations at a site with a soil profile overlying bedrock to peak bedrock accelerations at the same site (e.g., Seed and Idriss (1982) and Idriss (1990)). More recently, Bray et al. (1995), Kavazanjian and Matasovic (1995), Singh and Sun (1995), Bray and Rathje (1998), and Matasovic et al. (1998) have extended this type of relationship to solid waste landfills.

Matasovic et al. (1998) compared estimated horizontal bedrock accelerations to recorded peak horizontal accelerations at the OII site. Table 6-8, taken from Matasovic et al. (1998), presents peak acceleration values (average of two horizontal components) recorded at the top deck of the OII landfill versus the estimated peak horizontal bedrock accelerations for the site. Based on these results, Matasovic et al. (1998) concluded that peak horizontal bedrock accelerations from both near-field and far-field earthquakes up to at least 0.15 g can be significantly amplified by solid waste landfills. They suggested that, based on the OII data, the curve developed by Harder (1991) for the upper-bound amplification of seismic accelerations in earth dams, shown in Figure 6-16, provides a conservative upper bound for amplification of peak accelerations in solid waste landfills. They also suggested that the relationship of Idriss (1990) for soft soil sites, shown in Figure 6-16, provides a reasonable representation of average amplification potential of solid waste landfills.

Table 6-8. Earthquake parameters, corresponding peak horizontal bedrock acceleration estimates, and peak horizontal accelerations recorded on the top of the OII Landfill, California (modified from Matasovic et al., 1998).

Earthquake	Moment Magnitude	Style of Faulting	Site-to-Source Distance (km)	Estimated Peak Bedrock Acceleration (g)	Peak Acceleration at Top Deck (g)
Pasadena (3 Dec 88)	5.0	Strike-Slip	13	0.075	0.105
Malibu (19 Jan 89)	5.0	Thrust	50	0.018	0.009
Joshua Tree (23 Apr 92)	6.1	Strike-Slip	163	0.006	0.017
Landers (28 Jun 92)	7.3	Strike-Slip	140	0.032	0.085
Big Bear (28 Jun 92)	6.4	Strike-Slip	119	0.015	0.049
Mojave Desert (11 Jul 92)	5.5	Strike-Slip	131	0.004	0.012
Northridge (17 Jan 94)	6.7	Thrust	43	0.104	0.230

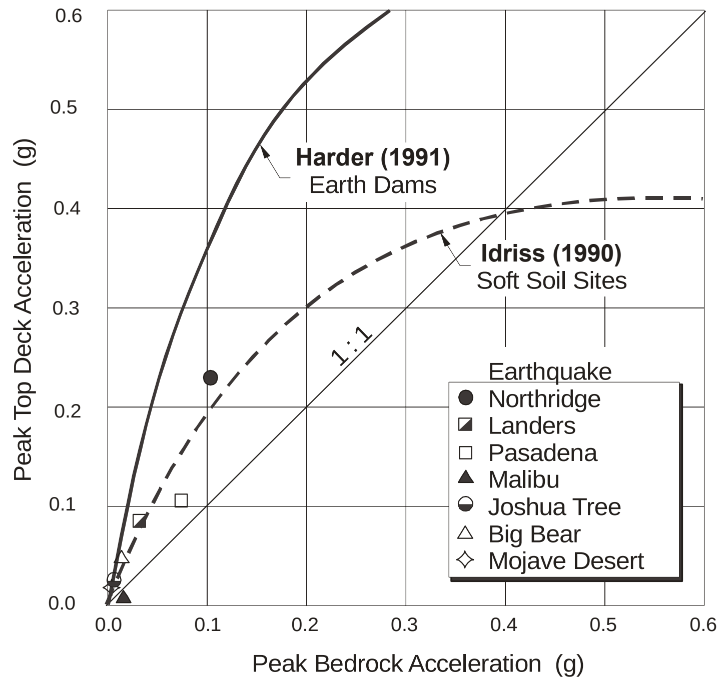


Figure 6-16. OII Data from Matasovic et al. (1998), Harder (1991) Curve for Upper-Bound Amplification of Seismic Acceleration in Earth Dams, and Idriss (1990) Curve for Soft Soil Sites.

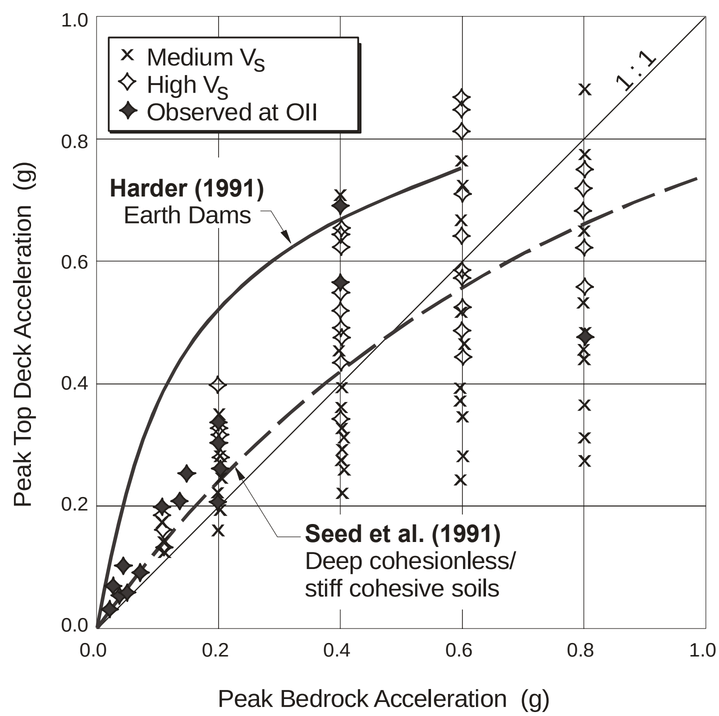


Figure 6-17. Results of Parametric Study Comparing Calculated Peak Horizontal Acceleration for OII Landfill Top Deck and Peak Bedrock Acceleration (modified from Bray and Rathje, 1998).

Independent of Matasovic et al. (1998), Bray and Rathje (1998) used the non-linear one-dimensional dynamic response analysis D-MOD (Matasovic, 1993; Matasovic and Vucetic, 1995) to perform parametric analyses of landfill response for a range of waste properties, waste heights, site conditions, and bedrock ground motions. The results of their parametric evaluation for cover systems are given in Figure 6-17. This figure presents a plot of peak horizontal acceleration at the landfill top deck versus peak horizontal bedrock acceleration. Bray and Rathje (1998) also compared their results to the Harder (1991) curve and the Seed et al. (1991) curve for stiff soil sites. Inspection of Figure 6-17 shows that the Harder (1991) curve provides a conservative upper bound to the calculated cover system accelerations. Bray and Rathje noted that the large amount of scatter in their parametric analysis results is due in large part to the sensitivity of the results to the input ground motion (i.e., variability among earthquakes). Variability in the assumed foundation conditions and waste profile also influenced the results. These findings are significant and engineers should consider this sensitivity when performing and interpreting the results of seismic response analyses.

Until more data become available, the Harder (1991) curve is conservatively recommended as a conservative upper-bound amplification of ground motions for simplified seismic site response of solid waste cover systems. Knowing the peak horizontal bedrock acceleration (from a USGS seismic hazard map, other map, or site-specific analysis), the Harder (1991) curve can be used to estimate an upper-bound peak horizontal acceleration at the top deck of the landfill. Should the Harder (1991) curve result in excessive cover system accelerations, detailed seismic response analyses can be conducted to assess whether a lower value of peak acceleration can be used on a project-specific basis. Site-specific seismic response analyses should also be used for any site where the average shear wave velocity in the upper 30 m of the foundation is less than 120 m/s (i.e., soft soil sites).

6.3.3.4 Analytical and Numerical Response Analyses

A one or two-dimensional seismic site response analyses may be performed for sites where significant cover system accelerations are anticipated or it is necessary to obtain a better estimate of seismically-induced motions in the cover system than can be obtained with the simplified approach. These analyses are also recommended for sites with soft soil foundations and for critical facilities or facilities with special features. Such projects include those in regions with the potential for very large earthquakes, where waste thicknesses are relatively large, or where cover system material or interface shear strengths are particularly low. The site response analysis is performed considering both the foundation soils and waste mass.

The computer program SHAKE, originally developed by Seed and coworkers (Schnabel et al., 1972) and updated by Idriss and Sun (1992), is perhaps the most commonly used computer program for one-dimensional seismic site response analysis. The SHAKE model idealizes the soil (and waste mass) profile as a system of homogeneous, visco-elastic sublayers of infinite horizontal extent. The response of this system is calculated considering vertically propagating shear waves. An equivalent linear analysis accounts for the strain-dependent non-linearity of soil and waste stiffness and damping using an iterative procedure to obtain modulus and damping values that are compatible with the equivalent uniform strain induced in each sublayer. At the outset, a set of properties (shear modulus, damping ratio, and total unit weight) is assigned to each sublayer of the soil or waste deposit. The analysis is conducted using these properties, and the shear strain induced in each sublayer is calculated. The shear modulus and damping ratio for each sublayer are then modified based on the applicable relationship relating these two properties and shear strain (see Figure 6-14).

Basic input to SHAKE includes the soil and waste profile, soil and waste properties, and selected earthquake acceleration time history. Soil and waste properties include the shear wave velocity (v_s) or maximum (small-strain) dynamic shear modulus (G_{max}) and total unit weight (γ_t) for each soil layer plus shear modulus reduction and damping ratio curves for each soil and waste material.

Computer programs are available for equivalent-linear and truly non-linear two and three-dimensional seismic site response analyses. A discussion of these more sophisticated models is provided by Kramer (1996). These models are only occasionally used in cover system design practice. Application of these models to the evaluation of cover system earthquake response often may result in lower-intensity seismically induced cover system motions than obtained using the one-dimensional SHAKE analysis. Use of non-linear analysis methods is recommended when the peak horizontal bedrock acceleration exceeds 0.4 g (Kavazanjian and Matasovic,1995; Bray and Rathje, 1998). Examples of the use of these more sophisticated models are presented by Idriss et al. (1995), Augello et al. (1998), and Matasovic and Kavazanjian (1998), who used the two-dimensional finite element program QUAD4M (Hudson et al., 1994) to evaluate the seismic response of the OII landfill, and Kavazanjian and Matasovic (1995), Bray and Rathje (1998),and Matasovic et al. (1998), who used the one-dimensional non-linear program D-MOD (Matasovic, 1993) to evaluate landfill seismic response. These more sophisticated models should only be applied by experienced geotechnical earthquake engineers, as their application is quite complex.

To perform seismic site response analyses and/or to perform permanent seismic deformation analyses, discussed subsequently, it is necessary to select earthquake acceleration-time histories as an input parameter to the analyses. Acceleration-time histories can be developed either by selecting a representative instrumental (accelerogram) record from the available catalog of records obtained during previous earthquakes or by synthesizing an artificial accelerogram. Acceleration-time histories should be selected for each seismic source having a potentially controlling influence on a site. Both near-field and far-field earthquake events should be considered. A higher magnitude far-field event with sufficient energy near the fundamental period of a solid waste mass may be more damaging to an overlying cover system than a near-field event characterized by a higher peak horizontal bedrock acceleration, higher frequency motions, and a shorter duration. For analysis, each acceleration-time history is scaled to the peak horizontal bedrock acceleration for the site. Selection of a time history from the available catalog of time histories is, in general, a preferable approach as opposed to synthesizing a time history. However, due to limitations in the catalog of available records, it is not always possible to find a representative time history from the catalog, particularly for sites in the eastern U.S. Richardson et al. (1995) and Kramer (1996) provide guidance on the selection of acceleration-time histories for use in seismic analyses.

6.3.4 Dynamic Shear Strength

The dynamic shear strengths of the components and interfaces of a cover system must be estimated to perform seismic slope stability and/or deformation analyses. These estimates are typically based on static or cyclic undrained shear strength tests. Shaking-table laboratory test results and observed earthquake performance of cover system components and interfaces are also used to develop information on cover system performance in earthquakes. Information on the cyclic shear strength of soils used in cover systems can be obtained from the geotechnical earthquake engineering literature (e.g., Kramer, 1996; Kavazanjian et al., 1997; and Lai et al., 1998). Shear strengths of CCLs, dry GCLs, and unsaturated granular soils typically used in cover systems are not significantly degraded by seismic loading, and cyclic shear strength is assumed to equal static shear strength. For hydrated unreinforced GCLs, this may not be the case, depending on the anticipated stress level and number of cycles of loading (Lai et al., 1998). The limited available data on the cyclic shear strength of interfaces involving geosynthetics (Kavazanjian et al., 1991; Yegian and Lahalf, 1992, Augello et al., 1995; Yegian et al., 1995; and Chaney et al., 1997) suggest that cyclic shear strengths of geosynthetics can be approximated using the results of static shear strength tests.

6.3.5 Seismic Stability and Deformation Analysis

6.3.5.1 Overview

The static LE slope stability analysis methods discussed previously in this document may be adapted for use in the seismic stability evaluation of cover systems. This adaptation can be achieved using a number of different approaches, of which the following three represent the current state of practice: (i) the pseudo-static factor of safety method; (ii) the modified pseudo-static factor of safety method; and (iii) the permanent seismic deformation method. These three approaches are discussed below.

6.3.5.2 Psuedo-Static Factor of Safety Method

Due to its simplicity, the psuedo-static factor of safety method remains the most common method of analysis used in practice for seismic design of cover systems. With this approach, the factor of safety for the cover system is calculated using a LE analysis that incorporates a specified seismic coefficient that is applied as a horizontal body force to the potential slide mass. The factor of safety obtained for the calculation is compared to a minimum acceptable factor of safety to determine the adequacy of the design. The seismic coefficient equals the fraction of the weight of the potential failure mass that is applied as a horizontal force to the centroid of the mass in a pseudo-static limit equilibrium stability analysis.

For the case of an infinite slope with no water flow, the pseudo-static factor of safety is given by:

$$FS = \frac{(\cos\beta - k_h \sin\beta)\tan\phi_i}{(\sin\beta + k_h \cos\beta)} + \frac{a_i}{\gamma_t t(\sin\beta + k_h \cos\beta)} \quad \text{(Eq. 6.17)}$$

where: k_h = pseudo-static seismic coefficient (dimensionless); and all other terms are as defined previously.

For the case of a slope of finite length, the pseudo-static factor of safety can be calculated, for the case of no water flow, using the approximate solution for sliding of the two-part wedge shown in Figure 6-4 (Matasovic et al., 2002):

$$FS = \frac{A}{B}\tan\phi_i + \frac{a_i}{B\gamma_t t} + \frac{t}{2h}\left(\frac{\sin\beta\tan\phi_s}{1-(B/A)\tan\phi_s}\right)\left(\frac{1+k_h^{\ 2}}{AB}\right)^2 + \frac{c_s}{\gamma_t h}\left(\frac{\sin\beta}{1-(B/A)\tan\phi_s}\right)\left(\frac{1+k_h^{\ 2}}{AB}\right) + \frac{T/h\sin\beta}{B\gamma_t t} \qquad \text{(Eq. 6.18)}$$

where A is a dimensionless parameter, given by:

$$A = \cos\beta - k_h \sin\beta$$

and B is also a dimensionless parameter, given by:

$$B = \sin\beta + k_h \cos\beta$$

and all other terms were defined previously. Note that if $\phi_s = 0$, $c_s = 0$ and T = 0, Eq. 6-18 reduces to Eq. 6-17.

The case of a slope of finite length has also been addressed by Koerner and Daniel (1997), who provide a solution that requires the solving of a quadratic equation. For more complicated geometries and slope conditions, design calculations are more easily performed using one of the LE slope stability computer programs described previously. It is common in performing seismic stability analyses of cover systems to assume no water flow in the slope. The rationale for this assumption is that the probability of occurrence of both a design-level earthquake event and a design-level storm event at the same time is extremely low.

The main drawback of the pseudo-static factor of safety approach lies in the difficulty in relating the value of the seismic coefficient to the characteristics of the design earthquake. Use of the peak acceleration at the top of the waste mass as the seismic coefficient, coupled with a psuedo-static factor of safety of 1.0, results in a very conservative design basis. This result would imply no displacement of the cover system during the design earthquake, not even for the milli-seconds during which the peak accelerations are applied. A seismic coefficient smaller than that corresponding to the peak ground acceleration is sometimes used, but the magnitude of cover system displacement in this case is unknown.

6.3.5.3 Modified Pseudo-Static Factor of Safety Method

The problem of selecting an appropriate seismic coefficient for the pseudo-static approach can be addressed by implicitly considering the potential for seismically-induced deformations. Based on Hynes and Franklin (1984), Richardson et al. (1995) suggested that seismically-induced displacements in a slope will be less than 0.3 m if the yield acceleration, $k_y g$ (m/s^2), defined as the horizontal acceleration producing a pseudo-static factor of safety of 1.0, is no less than 50%

of the peak horizontal acceleration of the slope (i.e., cover system). This result represents an upper-bound value for the seismic deformations calculated by Hynes and Franklin (1984) using almost 400 earthquake strong motion records. The value of $k_y g$ required to produce 0.3 m of permanent seismic displacement drops to about 15% of the peak horizontal acceleration if the mean plus one standard deviation curve is considered rather than the upper-bound curve. Other values of $k_y g$ can be derived from Figure 6-18.

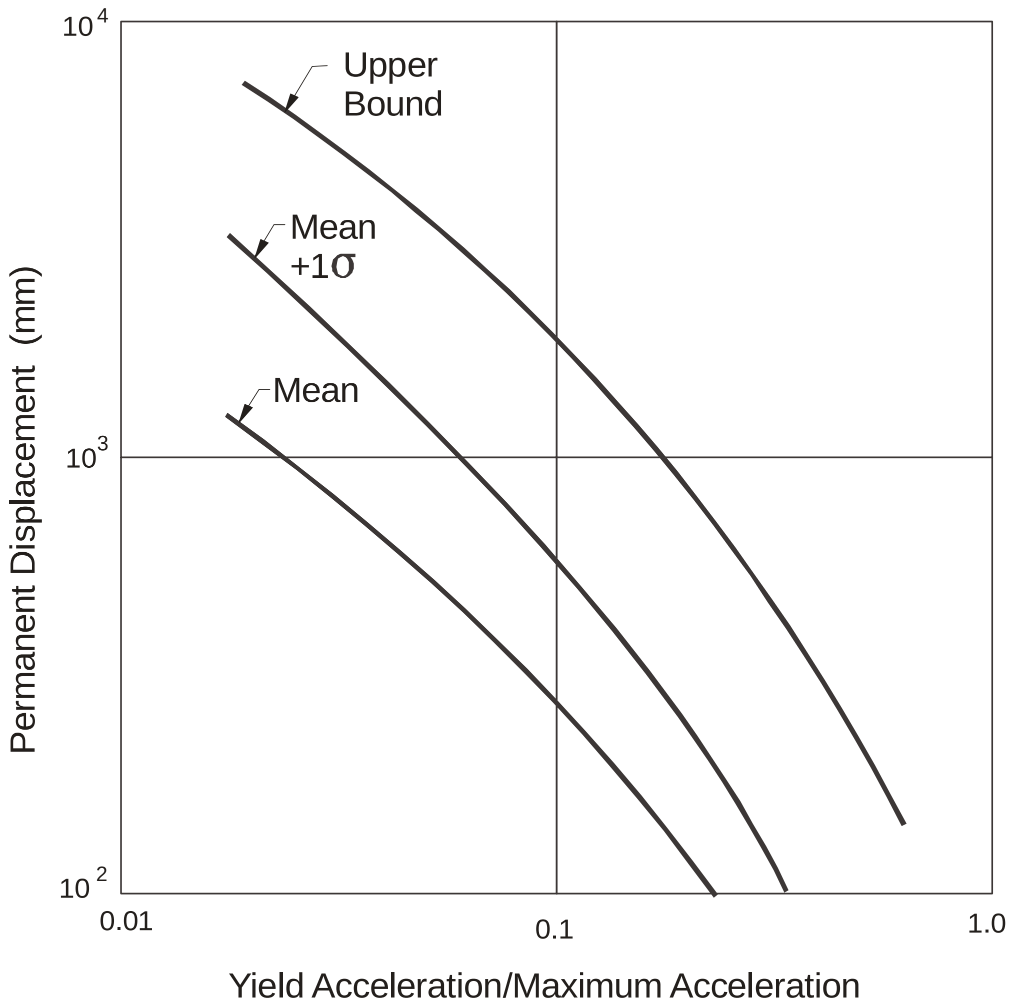


Figure 6-18. Hynes and Franklin (1984) Permanent Seismic Displacement Chart (modified from Richardson et al., 1995).

Kavazanjian (1998) presented a refined procedure for deriving a displacement-based seismic coefficient value specifically for the design of cover systems for solid waste landfills. Seismic coefficient values for specified levels of permanent seismic displacement are calculated by multiplying a ratio, obtained from Table 6-9, by the peak horizontal acceleration of the cover system obtained using the Harder (1991) curve shown in Figure 6-14. Kavazanjian has suggested that for earthquakes of magnitude less than or equal to 6.5 within 10 km of the project site, and for any earthquake of magnitude less than or equal to 5.5, the mean ratios in Table 6-9 be used. Kavazanjian further recommended that for earthquakes of magnitude greater than 6.5, and for earthquakes between magnitude 5.5 and 6.5 that are more than 10 km from the project site, the mean plus one standard deviation ratios in Table 6-9 be used. Kavazanjian (1998) recommended that seismic coefficients derived using Table 6-9 should be used with a factor of safety of 1.0. It is cautioned that the use of peak shear strength parameters with this approach is unconservative. Shear strength values should be selected considering the displacement value from Table 6-9 associated with the chosen seismic coefficient. It is noted that this simplified approach is not recommended for soft soil sites; soft soil sites should be evaluated using a site-specific seismic response analysis and permanent seismic displacement analysis with acceleration-time histories selected as previously described in this chapter.

Table 6-9. Ratio of yield acceleration, $k_y g$, to peak acceleration of cover system as a function of calculated permanent seismic displacement (based on Hynes and Franklin (1984) curves shown in Figure 6-15). Note: σ = standard deviation.

Calculated Displacement (mm)	Mean Ratio	Mean + 1σ Ratio
100	0.23	0.35
150	0.17	0.27
300	0.08	0.17
500	0.05	0.11
1,000	0.03	0.06

6.3.5.4 Permanent Seismic Deformation Method

With the permanent seismic deformation method, cumulative permanent seismic deformations are calculated on the basis of that portion of the acceleration-time history of the cover system that exceeds $k_y g$. For the infinite slope case, k_y is calculated using Eq. 6.17 and FS = 1.0. For the case of a finite length slope with uniform soil thickness above the critical potential slip surface, k_y is calculated using Eq. 6.18 and FS = 1.0. For more complex cases, k_y is calculated using a LE slope stability computer program.

The actual calculation of permanent seismic displacement is usually performed using Newmark's "sliding block on a plane" method of analysis (Newmark, 1965). In a Newmark analysis, acceleration pulses (in the earthquake acceleration-time history) exceeding $k_y g$ are double-integrated to calculate the accumulated "permanent" seismic displacement (Figure 6-19). Theoretically, this calculated permanent displacement is a rigid body displacement that accumulates everywhere along the critical potential slip surface. Typically, only the horizontal component of the earthquake acceleration-time history is considered in the analysis. The acceleration-time history of the cover system used in the analysis is obtained from a seismic response analysis. With this approach, the response analysis is "decoupled" from the computation of permanent displacement (i.e., seismic response is calculated assuming no slip displacement between the cover system and landfill, and cover system displacement is calculated using the results of the seismic response analysis (Bray and Rathje, 1998). The decoupled approach is generally conservative for cover system displacement analyses.

Several commercially-available, PC-based computer programs exist to perform Newmark analyses (Houston et al., 1987; Yan et al., 1996). These models assume a constant value of k_y. Recognizing that most geosynthetic materials and interfaces exhibit strain-softening shear behavior, Matasovic et al. (1997) proposed a modification to the standard Newmark procedure

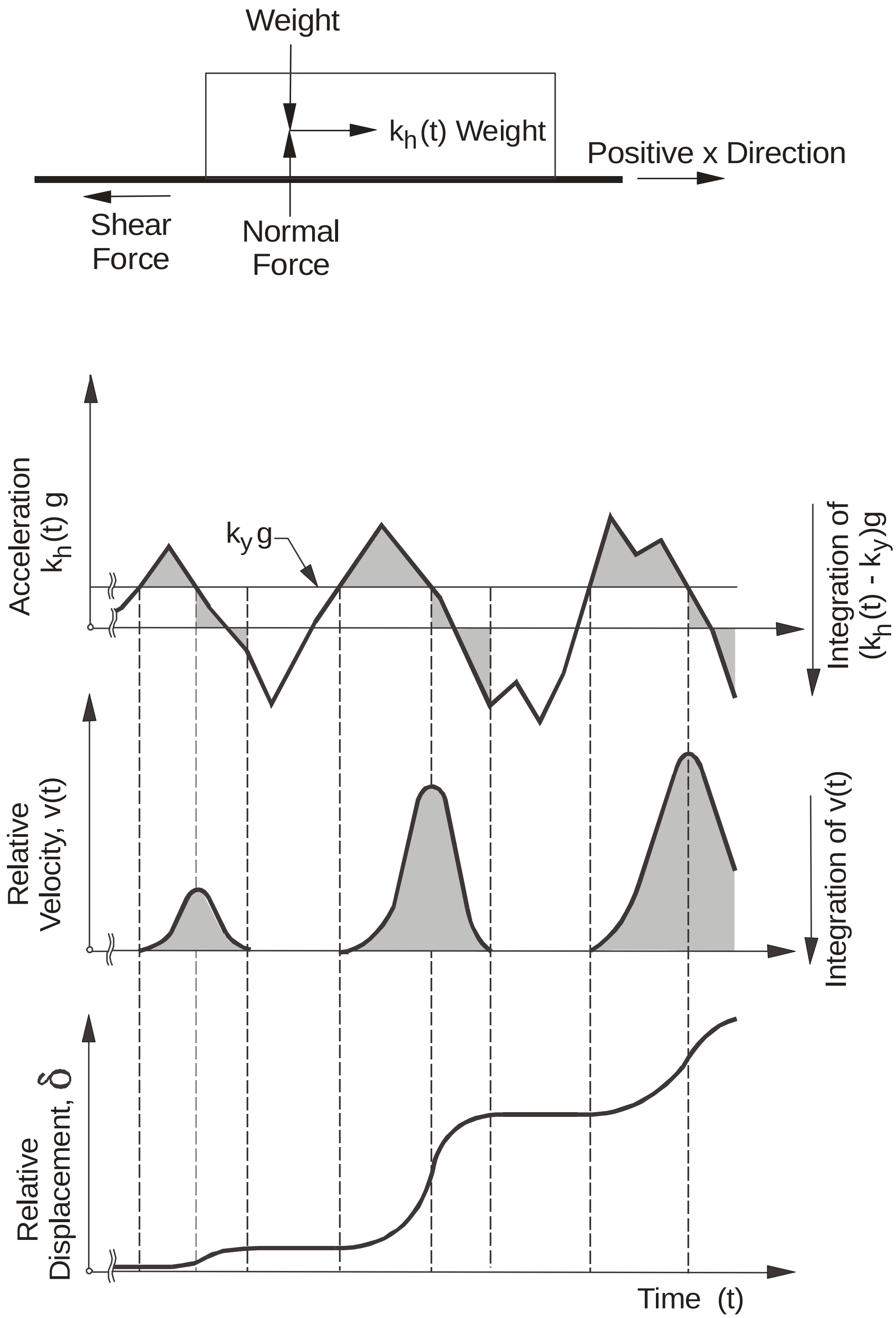


Figure 6-19. Basic Elements of Classical Newmark Sliding-Block Analysis with Constant Yield Acceleration.

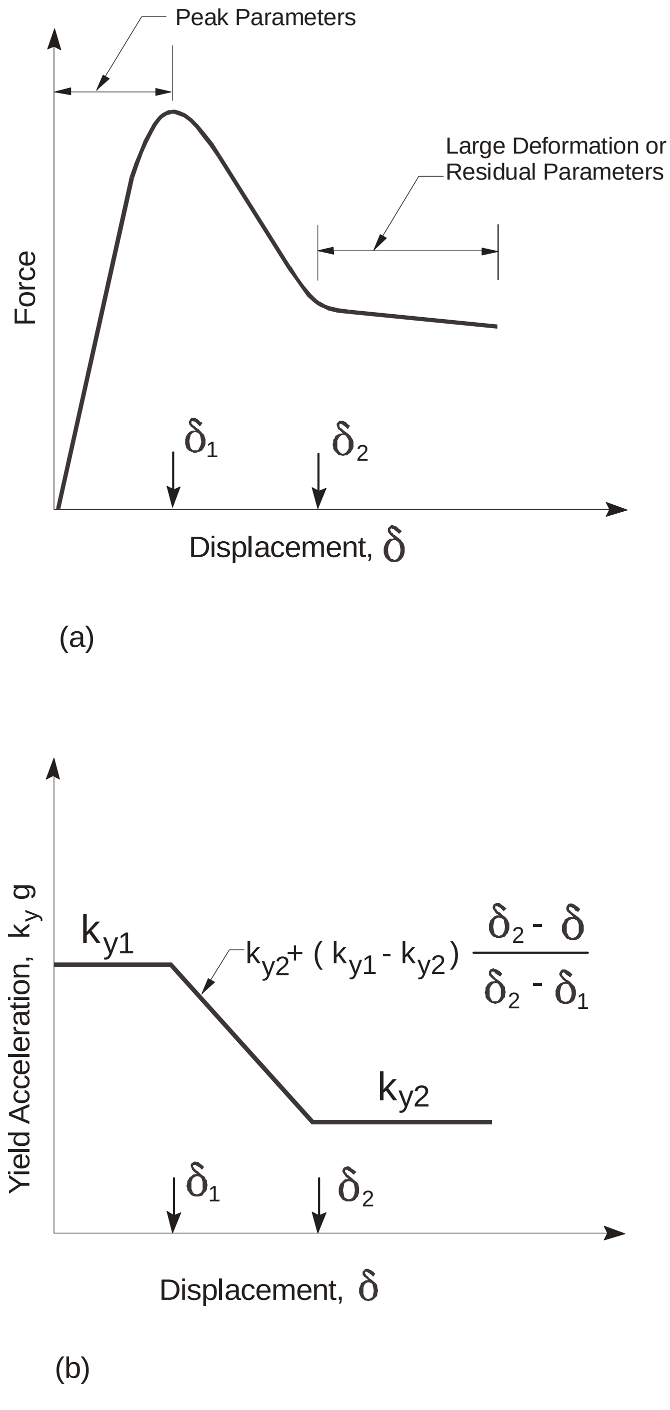


Figure 6-20. Yield Acceleration Degradation Model (modified from Matasovic et al., 1997). $k_y = k_{y1}$ and is Based on Peak Strength Parameters at Displacements up to the Displacement at Peak Strength (δ_1). $k_y = k_{y2}$ and is Based on Residual Strength Parameters at Displacements Greater than the Displacement at Residual (or Large-Displacement) Strength (δ_2). At Displacements Between δ_1 and δ_2, k_y Varies Linearly Between k_{y1} and k_{y2}.

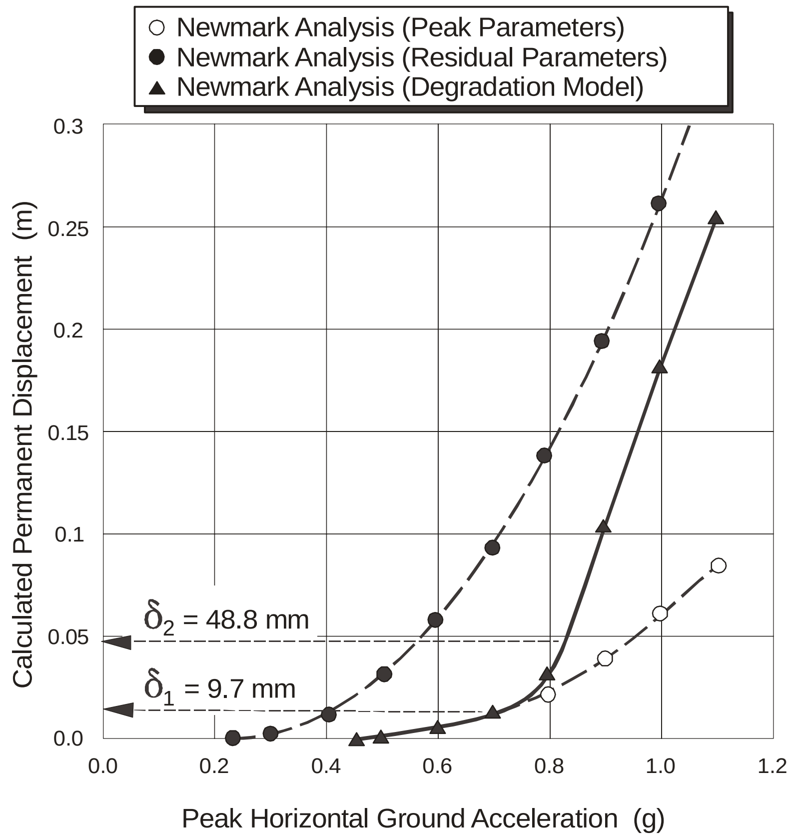


Figure 6-21. Results of Newmark Seismic Deformation Analysis for Constant and Degrading Yield Acceleration at a Normal Stress of 20.7 kPa (modified from Matasovic et al., 1997). δ_1 and δ_2 are as Defined in Figure 6-20.

specifically for cover systems incorporating geosynthetic interfaces. The modified version incorporates a linear k_y degradation model to account for strain-softening materials and interfaces (Figure 6-20). Matasovic et al. (1997) demonstrated the sensitivity of the calculated permanent seismic deformation for a typical GT/CCL interface and three differing assumptions regarding k_y: (i) constant, based on peak interface shear strength parameters; (ii) constant, based on residual (or large-displacement) interface shear strength parameters; and (iii) degrading, in accordance with Figure 6-20. Figure 6-21 presents typical calculation results from Matasovic et al. (1997) for the post-peak strain-softening exhibited by a GT/CCL interface. The sensitivity of the calculation results to the k_y assumption is evident.

6.3.5.5 Seismic Deformation Performance Criteria

In the current state-of-practice for design of cover systems, it is common to require permanent seismic deformations calculated using a conservative, Newmark-type approach to be less than 150 to 300 mm (Seed and Bonaparte, 1992; Anderson and Kavazanjian, 1995). Smaller values are sometimes considered if the potential slip surface underlies a non-ductile critical component, such as a HDPE GM. Larger deformations are sometimes considered if the potential slip surface is above all non-ductile critical components. Inherent in the selection of an allowable displacement value is an understanding that the calculation methodology is conservative, and actual earthquake-induced deformations would be less than calculated. In this regard, some engineers prefer to view the calculated seismic displacement as a performance index as opposed to a true prediction of actual deformations.

In applying seismic performance criteria to cover systems, several factors can be considered that do not typically apply to liner systems. First, the condition of a cover system can be readily observed after an earthquake through a post-earthquake inspection program. Second, the potential adverse impacts associated with excessive deformation of a cover system will involve tearing of geosynthetics, cracking of soils, disruption of gas management systems, and disruption of surface-water management systems. The risk of personal injury or environmental impact resulting from these types of problems will typically be small. The damage to cover systems caused by seismic displacement is typically repairable, although some at considerable cost and effort. For these reasons, it may be acceptable in some cases to consider calculated permanent displacements that are near the upper limit of the current state-of-practice for cover system applications.

Kavazanjian (1998) proposed two criteria for the seismic design of cover systems: (i) design without damage; and (ii) design accepting some limited damage to the cover system, but without "harmful discharge." For the "no damage" criterion, Kavazanjian suggested that a calculated permanent seismic displacement of up to 300 mm is acceptable for simplified analyses which use upper bound displacement curves from generic Newmark displacement charts (e.g., Hynes and Franklin, 1984), residual shear strengths, and/or simplified seismic response analyses. Kavazanjian further suggested that a calculated permanent seismic displacement of up to 150 mm represents an acceptable "no damage" criterion in cases where more sophisticated analyses are used to calculate the permanent seismic displacement using project-specific seismic response and formal Newmark displacement analyses.

For the case of "no harmful discharge", Kavazanjian (1998) suggested that a permanent deformation criterion of up to 1 m may be acceptable. With respect to this criterion, Kavazanjian (1998) states:

> "*When designing a cover system to withstand* [an earthquake] *without discharge, provisions are needed to mitigate potential hazards associated with discharge of leachate and/or gas from disrupted conveyance systems (e.g., use of automatic shut-off valves, secondary containment, and/or articulated seismic joints) and facilitate post-earthquake repair of damage.*"
>
> "*Multiple penetrations through geomembrane cover elements for gas and leachate collection or other purposes may limit allowable displacement to less than 1 m on an economic basis due to the cost of repair. However, if the anticipated displacement is above the geomembrane, there are not penetrations through the geomembrane on slopes, and benches provide sufficient capacity to retain cover soil that sloughs from above, the allowable seismic displacement of a geosynthetic landfill cover system may be unlimited, provided the owner is prepared to repair and/or replace the protective soil cover and drainage layer (if any) on top of the geomembrane after a severe earthquake.*"

The choice of a "no damage" or "no harmful discharge" design criterion will need to be made on a case-by-case basis by the design engineer, facility owner, and regulatory agency. Obvious factors that should be considered in choosing a criterion are: (i) potential impacts of large displacements; (ii) type of waste being covered; (iii) cost to repair cover system damage; and (iv) level of assurance that personnel and funds will be available for post-earthquake inspections and repairs after the earthquake occurs.

6.4 Settlement

6.4.1 Mechanisms of Settlement

Cover systems may be subject to settlements resulting from a variety of mechanisms. For purposes of evaluating cover system performance, settlements can be considered to have one of three sources (see Figure 6-22): (i) settlement of foundation soil; (ii) settlement due to overall waste mass compressibility; and (iii) settlement due to localized mechanisms in the waste.

Angular distortion or differential settlement may: (i) induce unacceptable tensile stress and strain in one or more cover system components, which can lead to component tearing or cracking; or (ii) cause cover system slopes to change or reverse grade which, in turn, can affect the performance of the cover system drainage layer and/or gas collection layer.

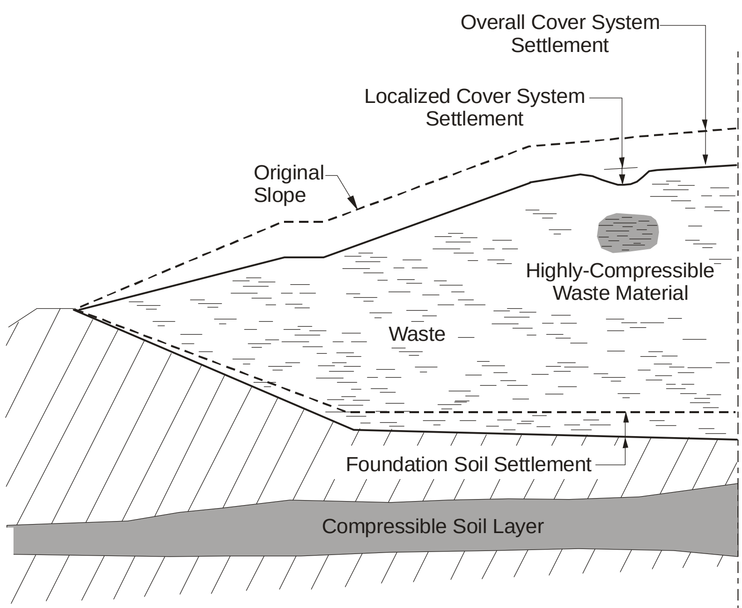


Figure 6-22. Sources of Cover System Settlement (modified from Othman et al., 1995).

6.4.2 Settlement of Foundation Soils

Impacts of foundation settlement on the performance of a cover system are usually not significant. Occasionally, situations arise where foundation settlements are of sufficient magnitude to affect the cover system design. For example, if the waste mass is underlain by a thick layer of soft clay, consolidation settlements can be large. Both primary settlement and long-term secondary settlement should be considered. Calculations are performed using equations from conventional geotechnical engineering practice (e.g., Holtz and Kovacs, 1981;

Lambe and Whitman, 1969) and a timeframe at least equal to the active life and post-closure care period of the facility.

6.4.3 Overall Waste Compression

Overall waste mass compressibility results in area-wide waste mass settlement. The potential for waste settlement is highly dependent on the type of waste. Relative to most other wastes, MSW is very compressible, due to both its initial compressibility when placed and the additional compressibility induced by the biodegradation of the organic component of the MSW. This latter component creates a significant time dependency to waste settlement. Other types of facilities that can undergo large settlement include impoundments containing high water content industrial sludges (typically inorganic). Materials such as mine waste, ash and slag, construction and demolition waste, and soil waste have relatively lower settlement potential. The following discussion of overall waste settlement focuses primarily on the settlement potential of MSW waste and other highly compressible waste material. The evaluation of ash, soil waste, or other low-compressibility inorganic waste is typically performed using equations for conventional geotechnical engineering practice (e.g., Holtz and Kovacs, 1981, Lambe and Whitman, 1969).

MSW waste compression results from complex factors including (Sowers, 1973; Edil et al., 1990; Sharma and Lewis, 1994):

- mechanical compression due to self-weight and surface loads;
- raveling (i.e., movement of fines into larger voids);
- physiochemical changes, including corrosion, oxidation, and combustion; and
- biochemical decomposition under aerobic and anaerobic processes.

The magnitude and rate of MSW settlement are controlled by many factors, among which are the waste fill height, organic content, age, moisture content, degree of compaction, and temperature. Figure 6-23 presents data from Edgers et al. (1992), König et al. (1996), and Spikula (1996), which shows that MSW landfills can settle from about 5 to 20% (and even up to 30%) of the initial landfill thickness (measured from the time the landfill first reached final grade).

A number of methods have been proposed for evaluating the short-term and long-term compression of waste. Three settlement models that have been adopted from geotechnical engineering and applied to waste are: (i) one-dimensional compression model; (ii) power creep model; and (iii) Gibson and Lo model (Gibson and Lo, 1961). A discussion of the latter two models is presented in Edil et al. (1990), and they are not discussed further herein. Presently, there is little experience in applying these last two models, and their applicability to the prediction of long-term settlements is not well demonstrated.

Conventional one-dimensional compression models have been widely used to estimate waste settlements (Sowers, 1973; Yen and Scanlon, 1975; Rao et al., 1977; Burlingame, 1985; Landva and Clark, 1990; Morris and Woods, 1990; Fassett et al., 1994; Stulgis et al., 1995). However, it is often assumed that primary self-weight settlement is complete prior to installation of the cover system. Thus, it is assumed that calculated primary settlements do not directly influence cover system performance.

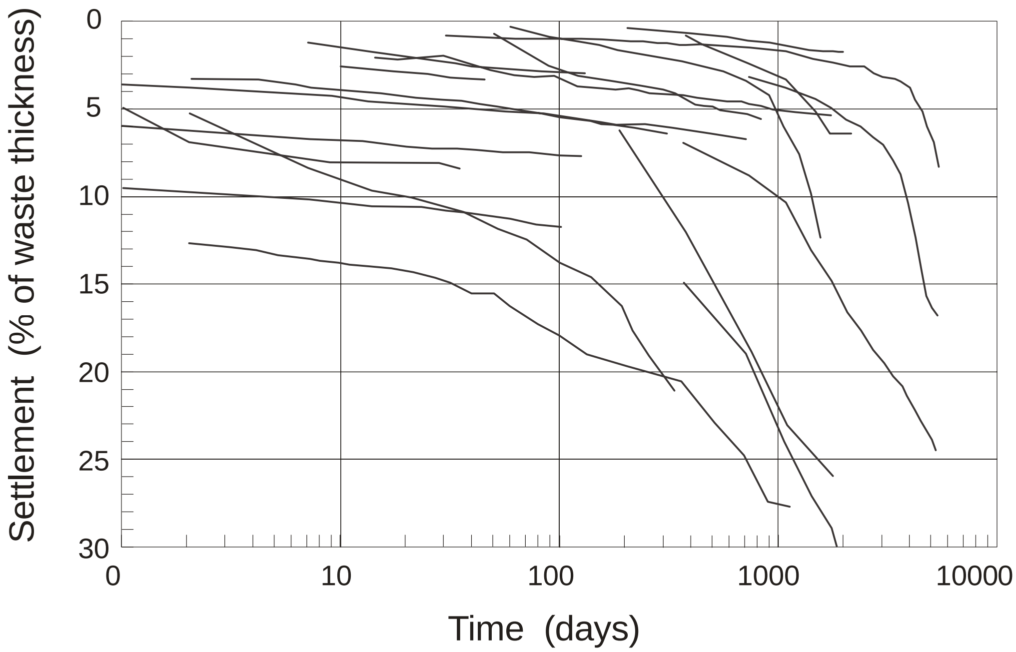


Figure 6-23. Total Settlement Data from Edgers et al. (1992), König et al. (1996), and Spikula (1996) for MSW Landfills, Measured from the Time the Landfill Reached Final Grade.

Cover system performance will be affected, however, by secondary waste settlement. The secondary waste settlement, ΔH_s (m), that occurs between times t_1 and t_2 is calculated with the one-dimensional model using an equation of the form:

$$\Delta H_s = C_{\alpha\varepsilon} H_1 \log \frac{t_2}{t_1} \qquad \text{(Eq. 6.19)}$$

where: $C_{\alpha\varepsilon}$ = modified secondary compression index (dimensionless); H_1 = height of waste at time t_1 (m); t_1 = starting time for the period of secondary compression (s); and $t_2 = t_1$ plus the time duration of secondary compression (s). Use of Eq. 6.19 implies that the magnitude of secondary settlement is independent of the applied stress. A modified form of Eq. 6.19 has been suggested by Bjarngard and Edgers (1990) and Stuglis et al. (1995) to account for a variable value of $C_{\alpha\varepsilon}$ between "intermediate" and "long-term" secondary compression times. Their equation is formulated herein as:

$$\Delta H_s = C_{\alpha\varepsilon 1} H_1 \log \frac{t_2}{t_1} + C_{\alpha\varepsilon 2} H_2 \log \frac{t_3}{t_2} \qquad \text{(Eq. 6.20)}$$

where: $C_{\alpha\varepsilon 1}$ = modified secondary compression index during the intermediate secondary compression period (dimensionless); $C_{\alpha\varepsilon 2}$ = modified secondary compression index during the long-term secondary compression period (dimensionless); H_1 = height of waste at time t_1 (m); H_2 = height of waste at time t_2 (m); t_1 = starting time for the period of secondary compression (s); $t_2 = t_1$ plus the time duration of intermediate secondary compression (s); and $t_3 = t_2$ plus the time duration of long-term secondary compression (s). Inspection of Figure 6-23 suggests that for some MSW materials, $C_{\alpha\varepsilon}$ is more or less constant during the period for which data exist, while for other facilities, a variable $C_{\alpha\varepsilon}$ could be used to better fit the data.

The reader should be aware that the choice of a value of $C_{\alpha\varepsilon}$ cannot be made without consideration of the normalization term t_1. For a given $C_{\alpha\varepsilon}$, the calculated value of ΔH_s will be significantly affected by the choice of t_1. Ideally, $C_{\alpha\varepsilon}$ and t_1 should be selected by empirically fitting Eq. 6.19 or Eq. 6.20 to field settlement data. In the absence of this type of correlation, it is suggested that t_1 be taken as the time period between when waste reaches final grade and when the cover system is installed over the waste.

Since $C_{\alpha\varepsilon}$ and t_1 are empirically derived, ΔH_s is assumed to be independent of applied effective stress, and the primary purpose of calculating ΔH_s herein is to assess potential impacts to the performance of the cover system, it is not necessary to subdivide the waste mass into a series of horizontal layers for purposes of calculating ΔH_s. With this approach, calculations are typically performed for increasing time intervals after closure to obtain a relationship between cover system settlement and elapsed time since closure.

Values of $C_{\alpha\varepsilon}$ for MSW reported in the technical literature have generally been in the range of 0.01 to 0.1 (Sowers, 1973; NAVFAC, 1983; Burlingame, 1985; Landva and Clark, 1990; Fassett et al., 1994; Stulgis et al., 1995). Given the empirical nature of $C_{\alpha\varepsilon}$ and t_1, it is interesting to compare calculated values of $(\Delta H_s/H_1)$ obtained using Eq. 6.19 to the range of observed time-dependent post-closure settlements for MSW (Figure 6-20). For the comparison, the waste mass is considered as a single unit with an average t_1 value of 100 days (approximately 3 months). Table 6-10 presents calculated values of $\Delta H_s/H_1$ (in percent) for values of $C_{\alpha\varepsilon}$ ranging from 0.01 to 0.1 and post-closure times of 100, 1,000 and 10,000 days (to which 100 days are added to obtain t_2 values).

Table 6-10. Results of parametric study of calculated post-closure secondary settlements (ΔH_s) as a percentage of initial landfill height (H_1).

$C_{\alpha\varepsilon}$	$(t_2 - t_1)$ (days after closure)		
	100	*1,000*	*10,000*
0.01	0.30	1.0	2.0
0.03	0.90	3.1	6.0
0.06	1.8	6.2	12.0
0.10	3.0	10.4	20.0

Based on the calculated values in Table 6-10, $C_{\alpha\varepsilon}$ values less than about 0.03, coupled with t_1 values of 100 days, are too small to model MSW time-dependent settlements. Careful review of the source references used to develop Figure 6-23 suggests that $C_{\alpha\varepsilon}$ values in the range of 0.04 to 0.08, coupled with t_1 values of about 100 days, provide a reasonable modeling of the settlement trends for modern MSW landfills that are typically filled fairly quickly and compacted using heavy trash compactors. Larger values of $C_{\alpha\varepsilon}$, in the range of 0.08 to 0.12, coupled with t_1 = 100 days, are needed to model the settlement trends in some of the older landfills in the source database. These landfills may have been filled with more variable waste placed under conditions less controlled than for modern landfills. Larger values of $C_{\alpha\varepsilon}$ would also be expected for landfills undergoing leachate recirculation or otherwise managed to increase biological activity and methane production in the waste mass.

If t_1 is assumed to be 30 days rather than 100 days, calculated $\Delta H_s/H_1$ values at 10,000 days would be about 25% larger than given in Table 6-10. Thus, if t_1 = 30 days is assumed, $C_{\alpha\varepsilon}$ values should be reduced about 25% from the recommended ranges given above. This calculation exercise clearly points out the sensitivity of calculated $\Delta H_s/H_1$ values to the magnitude of t_1.

6.4.4 Differential Settlement Due to Localized Mechanisms

Localized settlements, in the form of depressions, can develop within the first several years after cover system installation over MSW. These types of localized occurrences appear to be more common in older waste fills where a number of factors may contribute to the problem, including: (i) little initial waste compaction; (ii) variable waste characteristics; (iii) placement of sludges in the waste fill; and/or (iv) poor surface-water control leading to ponding of water on the waste. Localized differential settlement can lead to excessive stress or strain in cover system components (Gilbert and Murphy, 1987). Localized differential settlement of waste is generally attributed to one or more of several mechanisms, namely: (i) deterioration and collapse of objects (e.g., drums) in the waste; (ii) settlement associated with a highly-compressible zone of waste; and (iii) migration or raveling of waste particles into underlying voids.

Typically, analyses to evaluate impacts of localized differential settlement on the cover system are not performed as part of cover system design. However, in a few situations it may be necessary to evaluate potential effects of localized areas of high waste compressibility on cover system performance (e.g., cover systems for old dumps where the composition of waste is unknown or there is reason to believe that significant local waste heterogeneity may exist (due to any of the factors described above)). Several analysis methods are available for use in evaluating the potential effects of localized settlements on cover system performance. None of the methods have been field calibrated to any significant degree and selection of input parameters to the analyses is based primarily on engineering judgment. The analysis methods include:

- the application of mine subsidence models to the prediction of waste differential settlements (Murphy and Gilbert, 1985);
- an approach based on the uncoupled combined use of the tensioned membrane and soil arching theories for analyzing the stresses and strains in geosynthetics (such as geosynthetic layers within a cover system) that lose foundation support after construction due to development of a foundation void or depression (Giroud et al., 1990); Poorooshasb (1991) used a somewhat different analytical approach to address a similar problem;
- a boundary element formulation to model deformations around a collapsing void within an existing waste mass (Jang and Montero, 1993);
- two-dimensional finite element analyses to evaluate the response of a waste mass containing compressible zones (Carey et al., 1993); and
- the displacement method of Sagaseta (1987) to evaluate the response of a cover system over a waste mass containing localized compressible zones (Othman et al., 1995)

In the situation where differential settlement is likely to occur and the localized depressions cannot be eliminated, the choices are (i) continuously grading and maintaining the site; (ii) installing a thick buffer soil or waste prior to cover system construction; or (iii) installing geosynthetic reinforcement beneath the cover system. One or more of the analysis methods described above can be used to design soil buffer or geosynthetic (geogrid or high strength GT) support systems. The critical design parameters in any such analysis are the locations and dimensions of the anticipated localized depression or void. Since it is generally not possible to predict where such a feature will occur, any buffer soil or reinforcement layer, if used, will typically need to be installed over the entire waste mass.

6.4.5 Impacts of Settlement on Cover Systems

In design, settlement profiles accounting for the various settlement mechanisms are developed to evaluate potential impacts to the cover system. The evaluation usually considers: (i) post-settlement cover system grades; (ii) potential for depressions and ponding in the cover system; and (iii) stresses and strains in cover system components. Post-settlement grades should be adequate to shed runoff, prevent ponding, and prevent excessive stress or strain in cover system components, particularly the CCL, GCL, and GM hydraulic barriers.

Tensile strains causing cracking in compacted clays have been evaluated by Leonards and Narain (1963); Ajaz and Parry (1975a,b, 1976); Gaind and Char (1983); Chandhari and Char (1985); Jessberger and Stone (1991); and Lozano and Aughenbaugh (1995). Based on these studies, compacted clays tested under unconfined or low confinement conditions exhibit relatively brittle behavior and reach failure at axial extensional strains of 0.02 to 4%, with most compacted clays exhibiting failure at extensional strains of 0.5% or less. The studies also showed that the magnitude of tensile strain causing cracking increases with increasing percentage of fines and water content, and with increasing confining stress.

LaGatta et al. (1997) evaluated the impact of differential settlement on the hydraulic conductivity of GCLs overlain by a 0.6-m thick layer of pea gravel. The GCLs were tested either dry or hydrated and either intact or with a 230 mm overlap. The overlapped GCLs were tested across the overlap. The angular distortions (see Figure 2-13) of the upper surface of the GCLs were monitored and used to calculate tensile strain. The results of their evaluation indicate that intact and overlapped samples of needlepunched GCLs can withstand angular distortions of 0.35 to 0.6, equivalent to tensile strains of 5 to 16%, while maintaining a saturated hydraulic conductivity of 1×10^{-9} m/s or less. Stitch-bonded GCL samples were found to achieve the same hydraulic conductivity criterion up to an angular distortion of 0.35, equivalent to a tensile strain of 5%. For GT-encased, non-reinforced GCL samples, the maximum allowable angular distortion was much less, only 0.1, which is equivalent to a tensile strain of about 1%. This type of GCL, which is no longer available, had an open weave GT on its lower surface. The GT provided essentially no support to the GCL and allowed bentonite to migrate downslope within the depressed area and experience significant swelling. At the end of the test, the thickness of hydrated bentonite was approximately 5 mm on the sides of the depression and 50 mm on the floor of the depression. GCLs samples consisting of bentonite adhered to a GM maintained an equivalent hydraulic conductivity of 1×10^{-9} m/s or less when subjected to angular distortions of up to 0.8, producing a maximum tensile strain of almost 30%. Migration of bentonite was not observed because the GM component of the GCL blocked most of the flow. Within the test area, the GCL was only hydrated along the part of the overlap.

The tensile behavior of GMs varies depending on the polymer type, stress-strain characteristics, susceptibility to stress cracking, temperature, and other factors. The present state-of-practice for the design of strain-softening GM barriers (e.g., polyethylene GMs) is to limit the allowable GM tensile stress (or strain) to the short-term yield value divided by a factor of safety. The allowable tensile stress (or strain) for GMs exhibiting strain-hardening behavior (e.g., PVC GMs) is based on the short-term failure (break) value divided by a factor of safety. It should also be remembered that GMs are designed to be barriers, not tensile inclusions (as is geosynthetic reinforcement, for example). The long-term stress-strain, creep, and brittle fracture behavior of these materials under stress is not well understood. To the extent possible, applications should be designed to minimize tensile stresses and elongations in GMs.

The authors recommend that when it is necessary to specify allowable geosynthetic tensile stresses and strains, the yield stress and strain of the GM material be determined in a wide-width tension test (for plane deformation) or axisymmetric tension test (for spherical deformation) and that the factor of safety used to calculate the allowable values be at least five. The factor of safety should be applied to the yield values for strain-softening GMs and to the failure (break) values for strain-hardening materials. This recommendation should be conservative for virtually all types of commercially-available GMs used in cover system applications. If a higher value of allowable tensile stress or strain is desired, the design engineer must demonstrate that the product to be specified can sustain the allowable values without long-term creep, brittle rupture, or other type of long-term problem. This demonstration must also apply to GM seams.

6.5 Steep Slopes

6.5.1 Introduction

Occasionally, in the closure of old, existing landfills, it is necessary to address the issue of steep existing waste slopes. One option is to cut the slope back to a shallower grade by excavation and then relocate the excavated waste either on-site or off-site at another landfill (Figure 6-10). The advantage of this approach is that it increases the stability of the waste mass and results in a final slope inclination within the "conventional" range for cover systems. Disadvantages associated with waste excavation and relocation include construction-related instability, health and safety concerns associated with exposing the waste, leachate generation, nuisance (e.g., odor) issues, waste characterization necessary for on-site or off-site disposal of the excavated waste, and cost.

Several alternative approaches exist for constructing cover systems over steep waste slopes without need for waste excavation, or at least with very limited waste excavation. These alternatives include the use of: (i) a waste buttress coupled with a conventional slope cover system (Figure 6-11); or (ii) a steep slope cover system. Both of these alternatives are illustrated below, primarily in the form of case studies illustrating their use.

6.5.2 Waste Buttress

Two examples of the use of a waste buttress in the closure of old, existing landfills are presented below in order to illustrate the application of this technology.

Cargill and Olen (1997, 1998) describe the closure of an inactive hazardous waste landfill located on Long Island, New York. The landfill covers approximately 19 ha and has waste slopes extending up to 42 m above the surrounding ground surface. The steepness of the existing waste slopes, with inclinations up to 1H:2V and an average inclination steeper than 2H:1V, prevented the use of a conventionally-designed cover system. Regrading of the landfill to achieve slopes on which a conventional cover system could be constructed was not feasible due to limits on the final landfill height and lack of alternate landfilling locations for the excavated waste. For this project, the cover system design for the steeper slope sections incorporated a

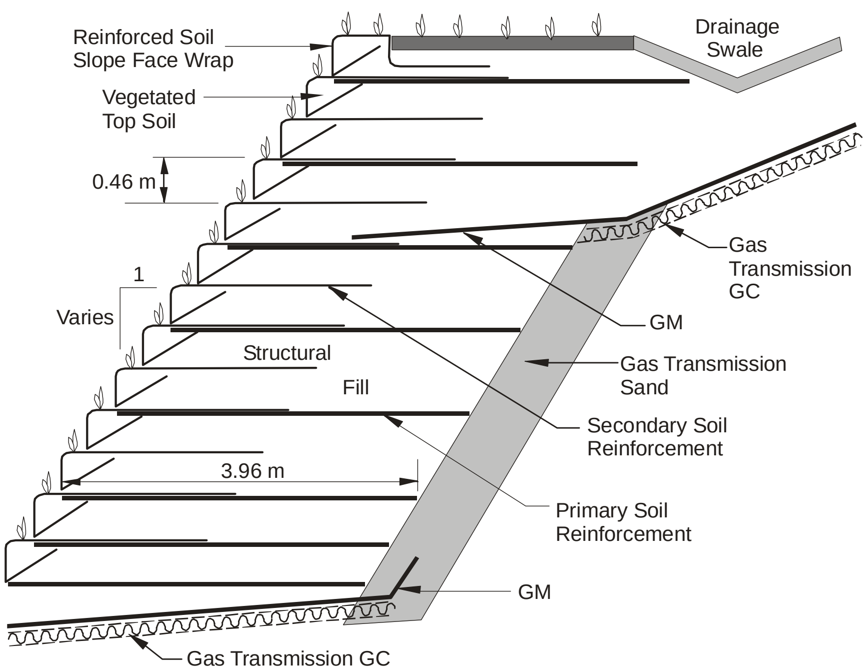


Figure 6-24. Detail of Reinforced Soil Slope Cover System Used on Steeper Slope Sections of a Hazardous Waste Landfill Cover System (modified from Cargill and Olen, 1997, 1998).

shingled GM within a geogrid-reinforced waste buttress (Figure 6-24). Approximately 4,300 linear m of slope buttress was constructed at heights up to 6.1 m. A cross section of this buttressed cover system is shown in Figure 6-25, and photographs of the system during construction and after completion are shown in Figures 6-26 and 6-27.

Slope stability is a major factor in the design of a cover system such as that described above. Three broad types of stability conditions must be considered for this type of closure. The first condition involves internal and interface stability of the components of the conventional portion of the cover system. The stability of these conventional components are evaluated using the procedures described in Sections 6.2 and 6.3 of this document.

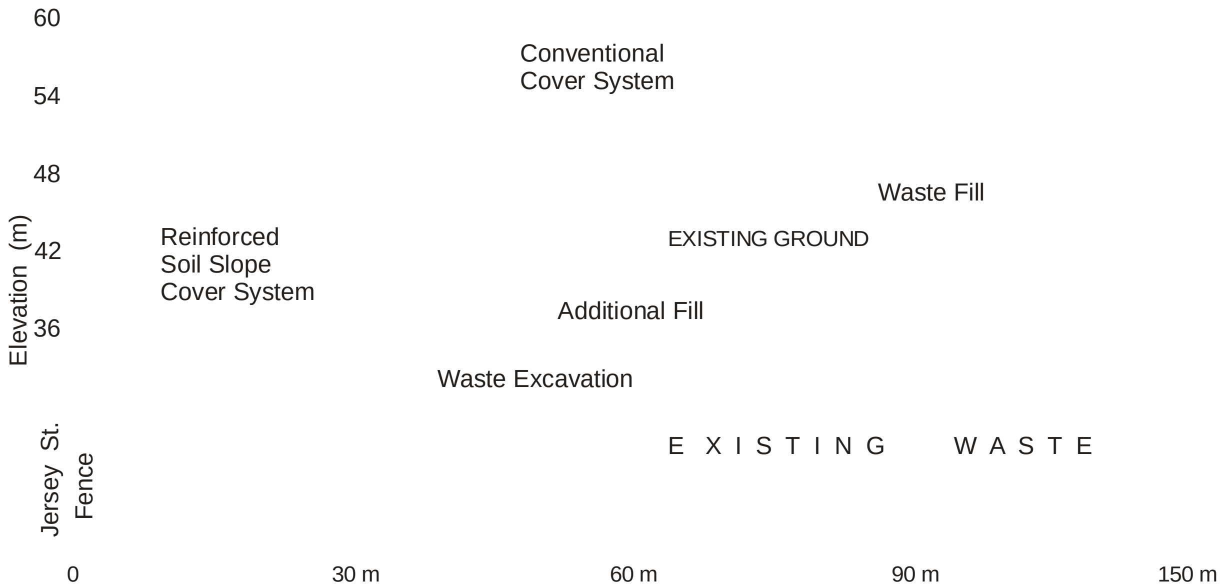


Figure 6-25. Reinforced Soil Slope Cover System on Steeper Slope Sections and Conventional Cover System on Shallower Slope Sections of a Hazardous Waste Landfill (modified from Cargill and Olen, 1997, 1998).

Figure 6-26. Construction of Reinforced Soil Slope Cover System on Steeper Slope Sections of a Hazardous Waste Landfill.

Figure 6-27. Constructed Reinforced Soil Slope Cover System for a Hazardous Waste Landfill.

The second condition involves the internal stability of the waste buttress. Many different types of earth retaining structures, including crib walls, mechanically stabilized earth (MSE) walls, and reinforced soil slopes, can be used in this application. If the structure is to be founded on firm ground, it can be fairly rigid, such as a precast concrete bin wall. However, if the structure is to be founded on waste, it must be flexible and able to undergo significant settlement and distortion while maintaining functionality. Geosynthetic-reinforced MSE walls and slopes with flexible facing elements meet these criteria. Cargill and Olen (1997, 1998) utilized geogrid-reinforced soil to form the buttress component of the cover system and a flexible facing (Figure 6-28). Procedures for design of earth retaining structures and evaluation of the internal stability of these structures can be found in a series of documents published by the U.S. Department of Transportation Federal Highway Administration (FHWA) (i.e., Holtz et al., 1995; Elias and Christopher, 1996; Sabatini et al.,1997) and in geosynthetic textbooks (Koerner, 1998).

The third stability condition that must be considered is the global stability of the buttress, waste mass, and landfill foundation. Global stability is typically evaluated using classical two-dimensional, LE slope stability analysis methods (e.g., Bishop, 1955; Spencer, 1967; Morgenstern and Price, 1965), as coded in the previously-mentioned commercially-available, PC-based computer programs (see Section 6.2.2.3). A critical aspect in the evaluation of global stability is the establishment of shear strength and unit weight parameters for soil and waste materials, and liquid heads (e.g., leachate heads in the waste and/or groundwater heads in the

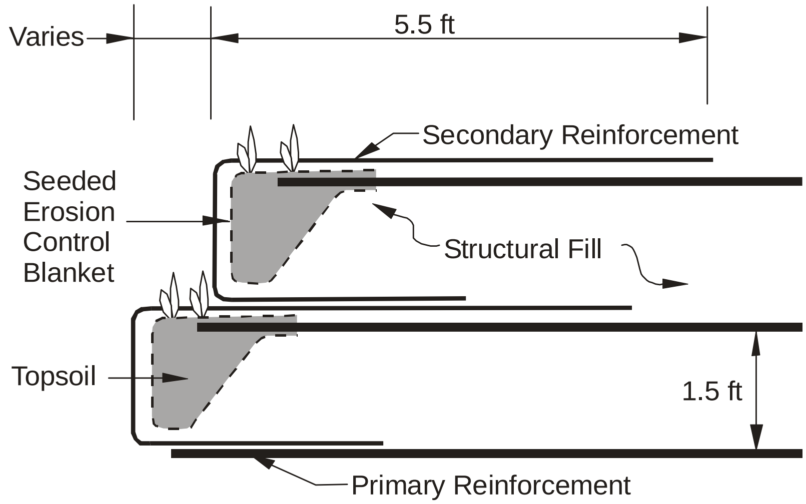


Figure 6-28. Flexible Facing Used with Reinforced Soil Slope Cover System for a Hazardous Waste Landfill (modified from Cargill and Olen, 1997; 1998).

foundation). Shear strengths for soil materials can be established using project-specific geotechnical site investigations and laboratory testing programs. For waste, shear strength and unit weight parameters can be established from information in the technical literature (e.g., Landva and Clark, 1990; Fassett et al., 1994; and Kavazanjian et al., 1995b) or through the use of project-specific field and laboratory test programs. For liner system materials and interfaces, shear strength parameters can be established using a laboratory testing program that includes consideration of the relevant testing procedures discussed for cover systems in Section 6.2.4. If the potential slip surface passes through a strain-softening soft soil foundation or liner system interface, the shear strength values selected for the waste, foundation, and/or liner system must be based on strain compatibility between the various materials along the potential slip surface. For example, the shear strain necessary to develop the peak shear strength of MSW may correspond to post-peak (e.g., residual) shear strength of a soft soil foundation material.

For another project, Graves et al. (1998) discussed the use of a pre-cast concrete crib wall 915 m long and up to 9 m high as part of an upgraded closure activity and flood protection for a 20-ha unlined sanitary landfill in Cuyahoga County, Ohio. In the years after landfill closure, an adjacent creek caused erosion and undermining of the landfill, creating localized near-vertical waste slopes (and overall waste slopes of about 2H:1V) and causing concerns about overall stability of the landfill. Flattening of the slope to allow installation of a conventional final cover system and achieve adequate slope stability factors of safety would have required relocation of approximately 700,000 m^3 of waste. Through use of a crib wall buttress at the toe of the landfill, the required waste excavation volume was reduced to 280,000 m^3. Design details and construction photographs for this project are illustrated in Figures 6-29 through 6-32.

The case study presented above utilized a pre-cast concrete crib wall as a gravity buttress to support waste slopes. Other types of wall systems could also be used for this application. Reference should be made to Sabatini et al. (1997) for an inventory of available wall types and typical wall unit costs. An emerging technology that holds promise for future use in landfill stabilization projects involves the use of geofiber reinforcement. Geofibers consist of relatively

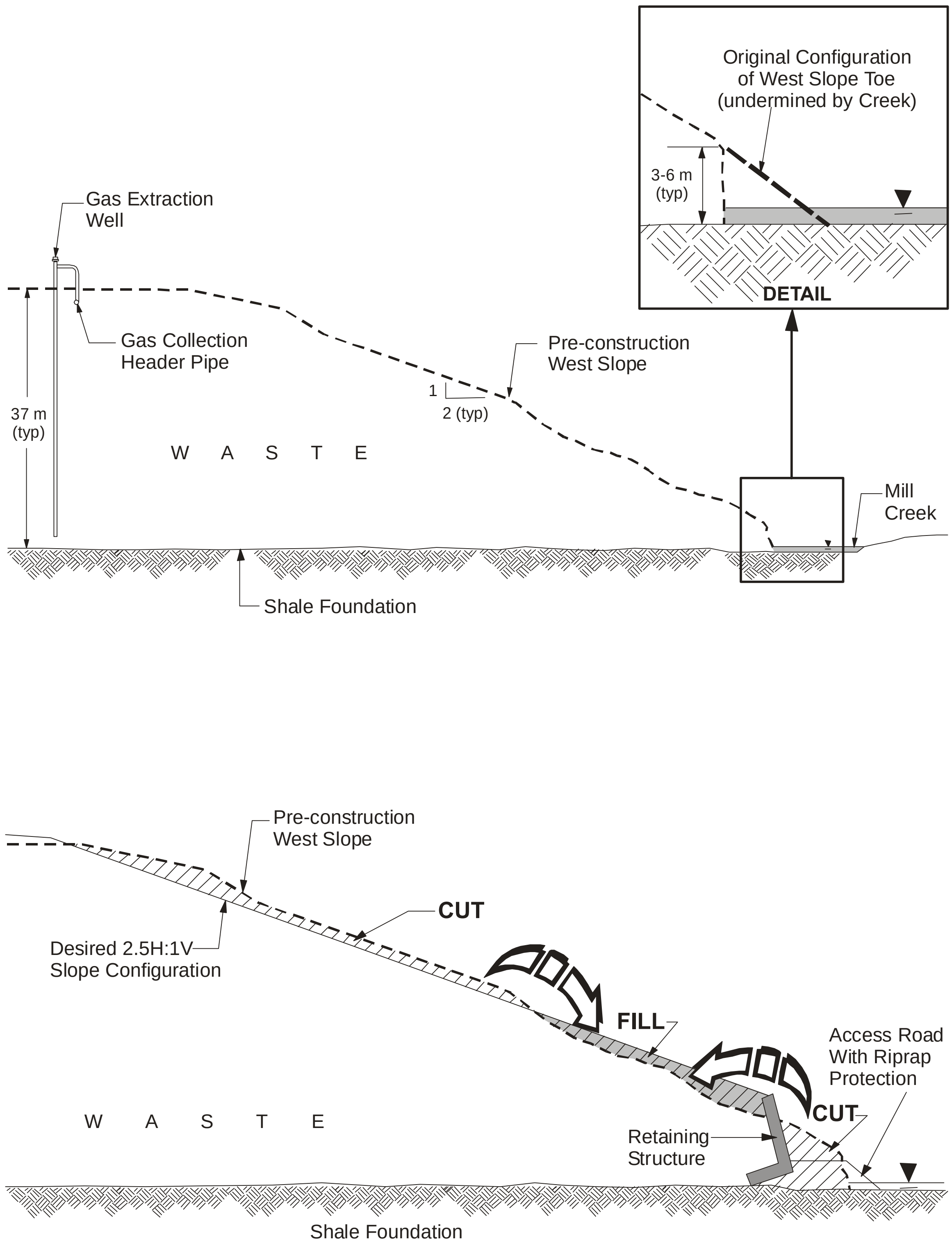


Figure 6-29. Waste Buttress Reduced Waste Excavation Volumes Required for an Upgraded Closure Activity at a Sanitary Landfill (modified from Graves et al., 1998).

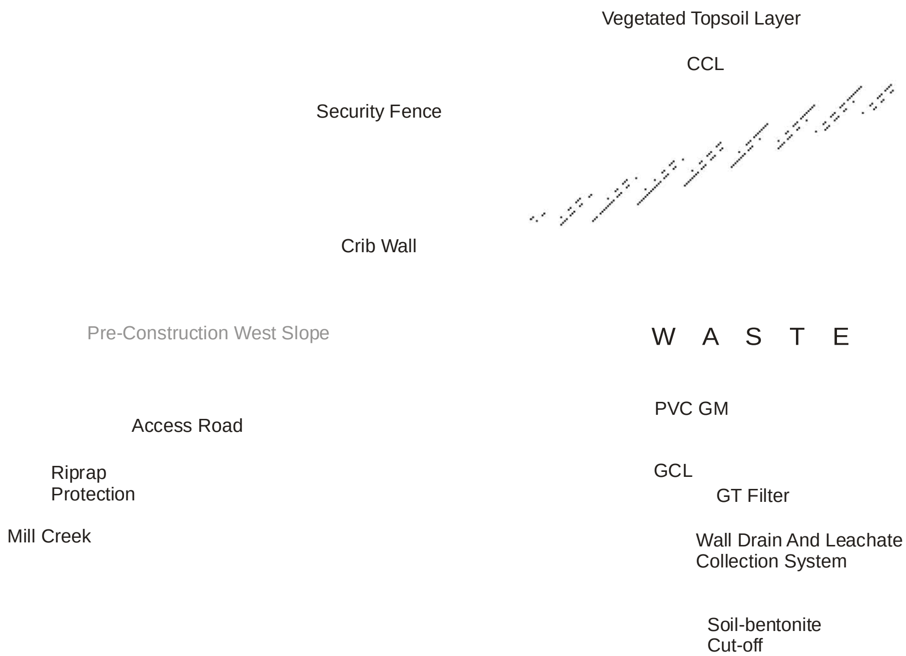


Figure 6-30. Pre-cast Concrete Crib Wall Used as Waste Buttress for a Sanitary Landfill (modified from Graves et al., 1998).

Figure 6-31. Construction of the Pre-cast Concrete Crib Wall Waste Buttress for a Sanitary Landfill.

Figure 6-32. Aerial View of Constructed Cover System with a Pre-cast Concrete Crib Wall Waste Buttress at a Sanitary Landfill.

small (e.g., 25 to 50 mm in typical length) polymeric inclusions, distributed throughout the reinforced soil mass. There are a variety of techniques for mixing the fibers into a soil fill, including pneumatic application and mechanical mixing. An example of the use of geofibers for a slope stabilization project is given in Gregory and Chill (1998).

6.5.3 Steep Cover System Slopes

Cover system slopes somewhat steeper than those conventionally used can be achieved through the careful selection of cover system components. Materials that can be used to increase the inclination of cover system slopes include:

- textured GMs or GMs manufactured from polymers that generate higher interface shear strengths compared to smooth GMs manufactured from HDPE;
- geosynthetic reinforcement installed parallel to the landfill slope in the internal drainage layer or protection layer and anchored at the crest of the slope;
- geosynthetic drainage layers;
- geofiber reinforcement of cover system soil layers;
- geocell (e.g., Figure 6-33) and geosynthetic erosion control materials (described in Section 2.2.5.3); and

- topsoil surface/protection soil layer with adequate cohesion to resist erosion, yet with adequate characteristics to support vegetative cover.

(a) Expanded

230 mm

6 m

230 mm

130 mm

(b) Collapsed

2.4 m

230 mm

3.4 m

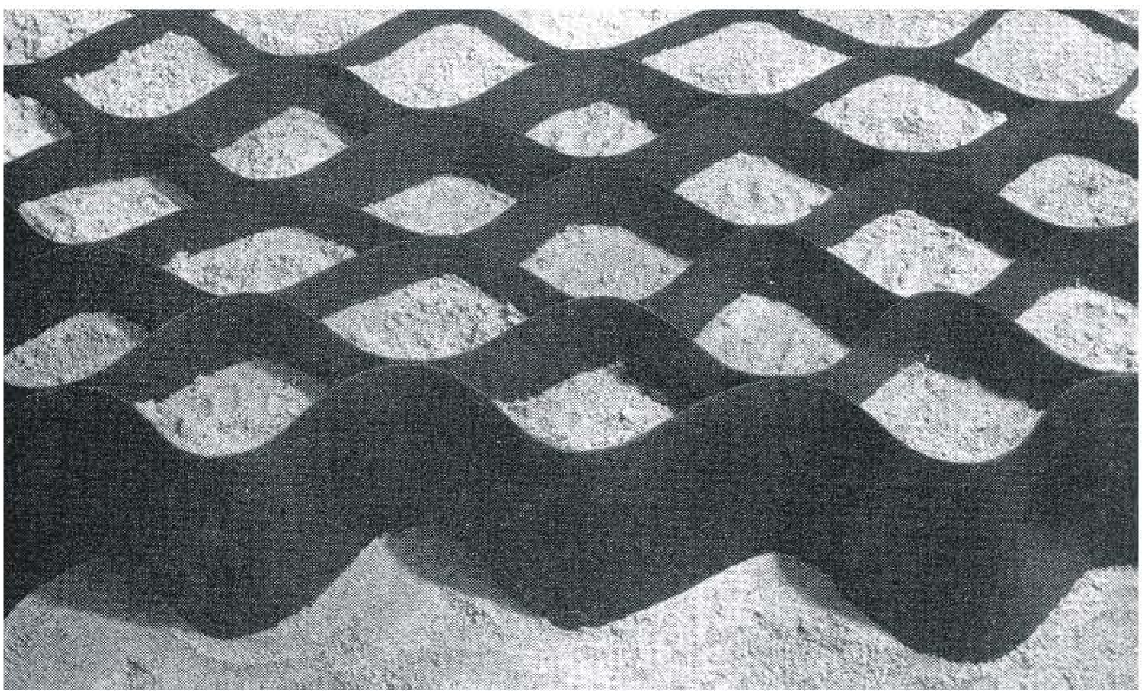


Figure 6-33. Geocells can be Used to Reinforce the Topsoil Surface/Protection Layer on Steep Cover System Slopes.

Using these materials, cover system slopes as steep as 2.5H:1V, and possibly slightly steeper can be constructed and maintained at a factor of safety at or near the target range discussed previously in this chapter. Even steeper slopes (e.g., 2H:1V, as demonstrated in the GCL test plot program described in Chapter 7 of this document) can be constructed, but factors of safety are likely to be lower than the target range given in herein. Moreover, long-term surface erosion problems should be anticipated when steep slopes are used. With steeper slopes, several other design aspects take on even greater importance than they might otherwise. For example, greater attention must be given to the selection of internal and interface shear strengths due to the greater potential for slope instability; thus, project-specific shear strength testing is essential. Also, seepage in a slope containing geosynthetic reinforcement can greatly reduce the effectiveness of the reinforcement. The stress-elongation characteristics of the various cover system components also become more important as slopes become steeper. Thus, the consideration of deformation compatibility of the cover system components is essential. It is possible, for example, that the elongation required to induce the design tensile force in a geosynthetic reinforcement layer is larger than the shear deformation needed to cause a GCL to exhibit large displacement rather than peak internal shear strengths characteristics. For this case, the design should be based on the large-displacement GCL shear strength and not the peak shear strength. Deformation compatibility can be evaluated as previously discussed in Section 6.2.3.

6.6 Soft Waste Materials

Another type of design issue sometimes encountered is the in-situ closure of impoundments or the capping of remediation source areas containing soft waste materials. These soft materials include high moisture content sludges, saturated process wastes, and saturated sediments or solid wastes. The common characteristic of these materials is that they have very low bearing capacity, which precludes using conventional techniques for cover system construction. These materials are also prone to large post-construction settlements that must be accounted for in design.

In general, if the undrained shear strength of the near surface waste is less than about 15 to 20 kPa, the waste may not be able to support a conventional cover system and the bearing capacity of the waste will be an important consideration in the design process. At undrained shear strengths below about 10 kPa, waste bearing capacity may become the controlling design criterion. Guidance on performing foundation bearing capacity analyses can be found in a number of textbooks, including Lambe and Whitman (1969) and Holtz and Kovacs (1981), and a number of more focused technical papers and reports, including Bonaparte and Christopher (1987), Humphrey and Rowe (1991), and Holtz et al. (1995). The three latter references provide information on the use of geosynthetics reinforcement to increase the bearing capacity of a soft foundation material (e.g., waste). The critical bearing capacity condition for a cover system over soft waste will typically occur during construction The critical condition is often associated with the timeframe during which the leading edge of construction is inducing relatively high shear stresses in the soft waste. Thus, construction equipment loads must be taken into account.

The engineering evaluation of a cover system over soft waste includes an assessment of overall bearing capacity, rotational stability, and lateral spreading. These potential failure mechanisms are illustrated in Figure 6-34. In addition to these stability evaluations, long-term settlement of

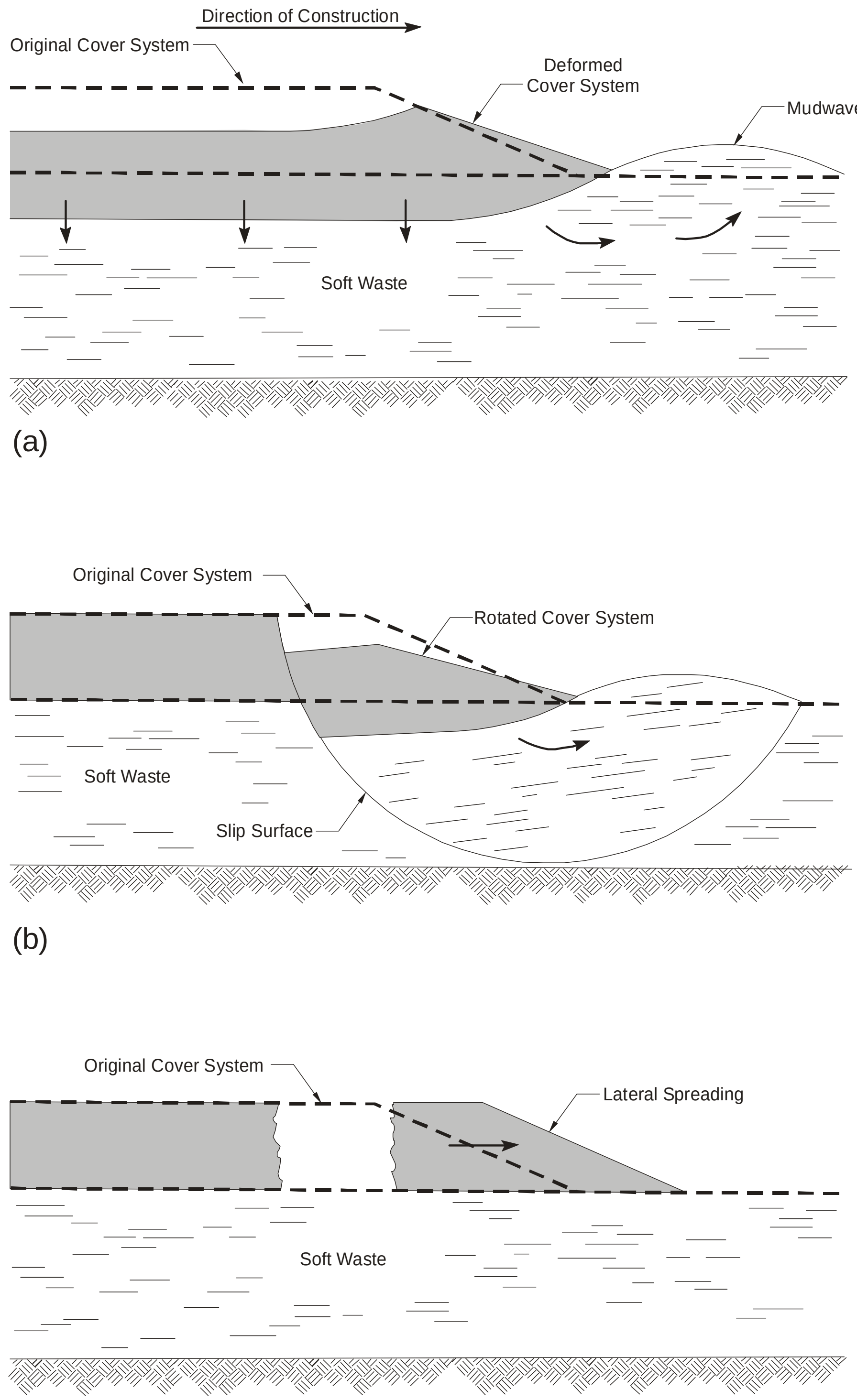


Figure 6-34. Three Potential Failure Mechanisms to be Considered for Cover Systems Constructed Over Soft Wastes: (a) Bearing Capacity Failure; (b) Rotational Failure; and (c) Lateral Spreading.

the cover system is estimated using classical geotechnical engineering calculation methods for soil or waste, as appropriate, as described previously in Section 6.4.

The design engineer has several available options when soft waste has inadequate shear strength to support the overlying cover system. These options include:

- strengthening of the waste by physical solidification;
- strengthening of the waste by preloading;
- strengthening of the waste by dewatering;
- strengthening of the waste by ET drying;
- supporting the cover system over the waste using reinforcement; and
- using lightweight cover system components.

Solidification is defined by EPA as a process in which materials are added to a waste to produce a solid to achieve one or more goals (Battelle, 1993). In the application being considered herein, the goals are to increase the waste's shear strength and decrease its compressibility. Agency guidance on waste solidification technology regulatory status, range of applicability and limitations, and use on a project-specific basis is given in the agency document (Battelle, 1993). Typical solidifying agents for this application include cement, cement kiln dust, lime, lime kiln dust, and fly ash. The final product may vary from a granular, soil-like material to a cohesive solid, depending on the properties of the solidifying agent and waste and the ratio of the solidifying agent to waste. One disadvantage of this approach is that the solidification process causes significant bulking (increase in volume) of the waste. In some cases, this increased volume can be used to advantage to build up the top elevations of what is initially a flat impoundment to achieve the sloping grades required for the cover system. As an example of a solidification project, Bodine and Trevino (1996) describe a case study where a cover system with a GM/CCL barrier was constructed over an oily sludge storage basin after the sludge was solidified in-situ using Portland cement. A 3.7-m diameter crane-mounted rotary auger was used to mix the cement with the sludge.

Strengthening of the waste by preloading involves spreading a layer of soil fill over the waste, then allowing the waste to consolidate under the weight of the fill. Additional layers of fill can be placed and the consolidation step repeated. Each consolidation step increases the undrained shear strength of the waste by about 20 to 25% of the applied vertical stress resulting from the weight of the fill. Disadvantages of this technique are that it is time consuming, due to the time required for waste consolidation, it involves multiple construction steps, and it requires significant amounts of fill. The time duration required for waste consolidation can be decreased through the use of wick drains. However, installation of wick drains into the soft waste may not be feasible due to access, settlement, and clogging issues. Vacuum consolidation can be considered as an alternative means to soil fill for applying a consolidation stress to the soft waste. However, as with wick drains, installation issues may render this technique unfeasible for some applications. Guidance on the design of preload systems can be found in Holtz and Kovacs (1981) and Ladd (1991).

Strengthening of the waste mass by dewatering involves the use of drainage trenches or other means to reduce the water level in the soft waste. A reduced water level has two benefits: (i) as the water level is pulled down, the effective stress in the waste, and hence the waste's strength, is increased; and (ii) evaporation from exposed waste above the water table tends to dry out the waste and increase its shear strength. In some cases, this surface drying can by itself lead to a stable crust upon which to build a cover system.

Strengthening of the waste by ET drying involves using high moisture uptake plant species to remove water from the waste by transpiration, thereby strengthening the near-surface waste. With ET drying, select plant species are planted or hydroseeded over the area to be strengthened. Because soft waste materials typically have a high moisture content, plants can readily access moisture in the waste matrix. As plants become established, two complementary benefits occur: (i) the waste surface dries and a strengthened crust develops; and (ii) the plant roots form a mat that reinforces and further strengthens the crust. Depending on the type of vegetation selected, the root mat may extend several inches (as in the case of grasses) or several feet (as in the case of certain tree species) into the waste. The success of the ET approach is dependent on the physical properties of the waste and the ability to keep the waste surface dewatered for the period of time required to establish healthy plant growth. The application of fertilizers or conditioning agents may be necessary to establish and sustain adequate plant growth. Simple greenhouse testing can be used to evaluate the potential effectiveness of ET drying. Pilot testing is recommended to quantify the amount of strengthening that can be achieved in a particular field application.

Geosynthetic reinforcement materials can be used to support cover systems over soft waste. This technique has found increasing use in recent years. With the approach, one or more layers of geogrid or GT reinforcement are placed over the soft waste, fill is placed on top of the reinforcement, and then the cover system is constructed on top of the fill. This technique has been used successfully with very soft waste materials (i.e., materials with undrained shear strengths below 10 kPa). Michalski et al. (1995) provide an example of the application of this technique to the closure of a 10-ha pickle liquor sludge lagoon in Pennsylvania. Closure of the lagoon required construction of a RCRA Subtitle C cover system, which was supported over the soft sludge by geogrid reinforcement. Guidance on the design of geosynthetic reinforcement systems can be found in the technical references cited previously in this subsection, and in Koerner (1998). While this technique has been shown to be effective, it is not without limitations. The technique does not reduce the inherent compressibility of the waste. Thus, when utilized with very soft materials, large total settlements of the cover system may occur. Also, when using this system over large areas, it may not be possible to build up the required final grades for the cover system. Even with a saw-tooth final grading plan, fill thicknesses can be significant to achieve cover system grades of at least several percent. The problem is exacerbated by the fact that the largest waste settlements will occur at the location where the fill thickness is greatest, which will tend to reduce cover system grades as the waste settles. The effects of settlements on cover system grades need to be considered in the design of geosynthetic reinforcement systems and may also be important for some, or all, of the other technologies described above.

The final option considered for construction of cover systems over soft waste materials is the use of lightweight cover system components. Examples of lightweight materials include, from the bottom of the cover system upward:

- geosynthetic reinforcement as a replacement for structural fill;
- lightweight structural fill as a replacement for structural fill; potential lightweight fill materials include slag, expanded clay and shale, vermiculite, tire chips, and geofoam; expanded clay and shale materials, manufactured by heating clay or shale in a kiln, are discussed by Bowders et al. (1997b); the geofoam class of geosynthetics is discussed in Section 3.7.1 of this guidance document;
- GC drainage layer or a thick needlepunched nonwoven GT as a replacement for a granular gas transmission layer;
- GCL as a replacement for a CCL; and
- GC drainage layer as a replacement for a granular drainage layer.

An example of a lightweight cover system designed and constructed as part of a CERCLA remedial action at a soft waste and soil site near Beaumont, Texas, is illustrated in Figure 6-35. Each of the options for constructing cover systems over soft waste materials have advantages and disadvantages that must be carefully evaluated for each project application. For most applications, several of the options will be used in combination to achieve the project design criteria.

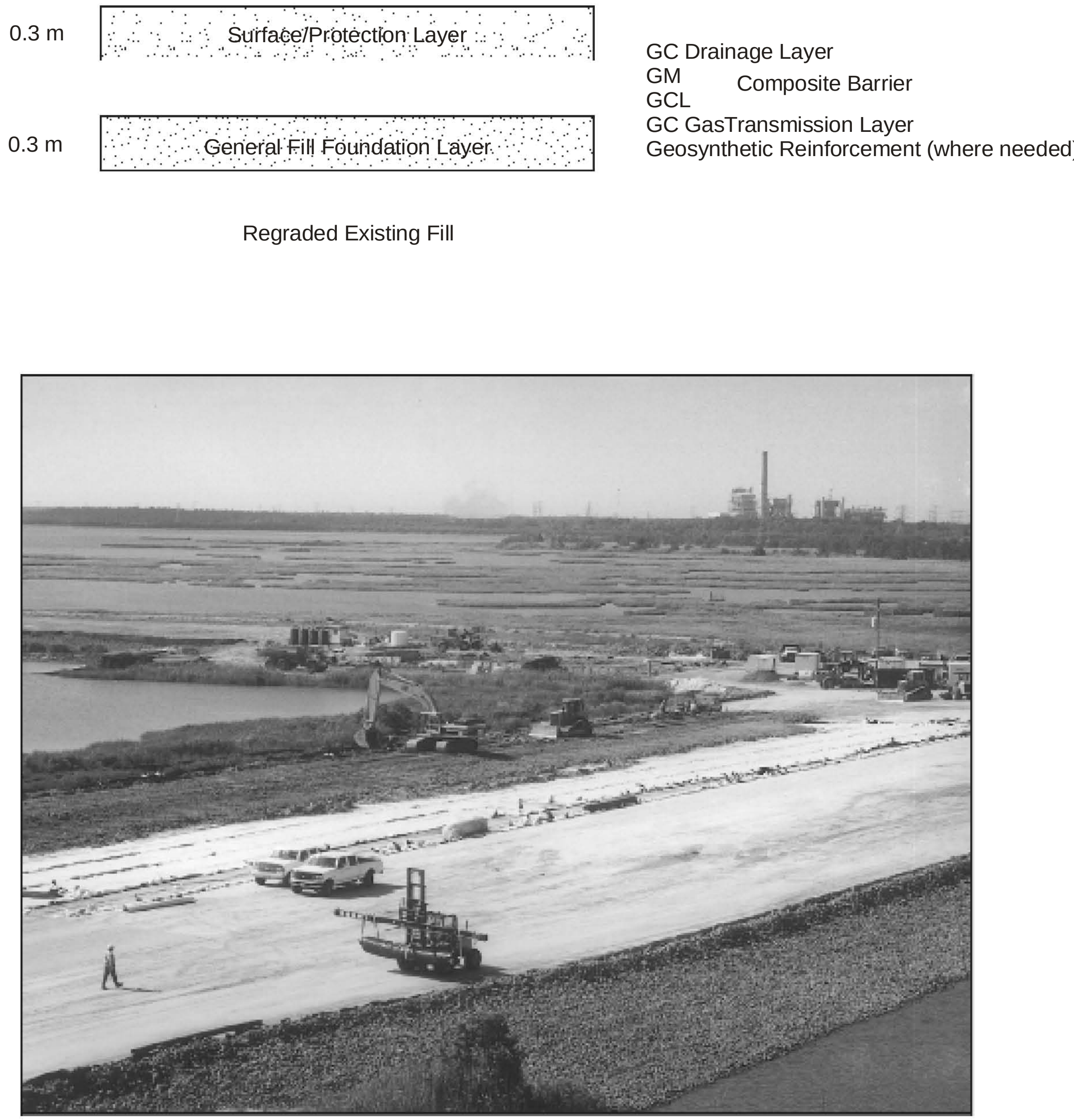


Figure 6-35. Example of a Lightweight Cover System Constructed Over Soft Waste and Soil at CERCLA site near Beaumont, Texas.

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

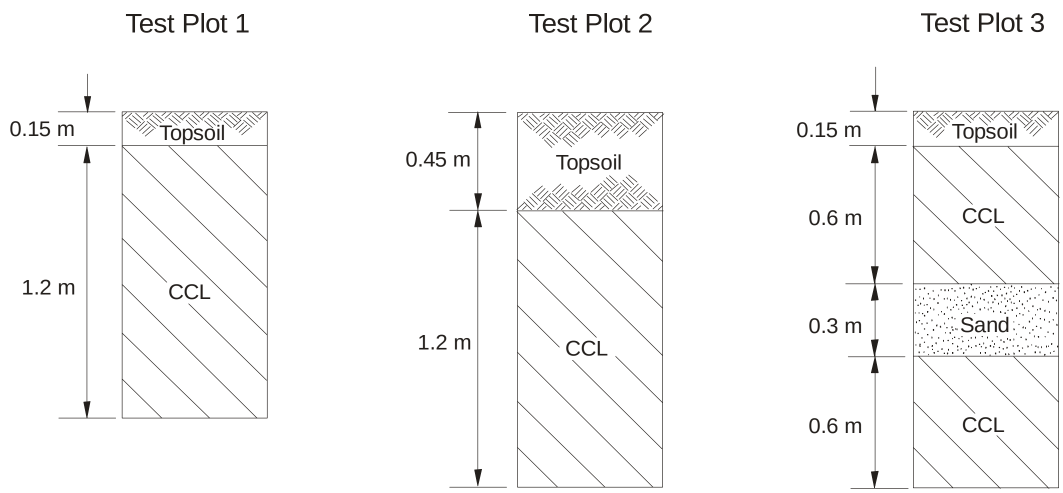


Figure 7-1. Cross Sections of Cover System Test Plots at a Landfill in Omega Hills, Wisconsin (modified from Montgomery and Parsons, 1989). Test Plot 1 is Representative of the Existing Landfill Cover System.

"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.

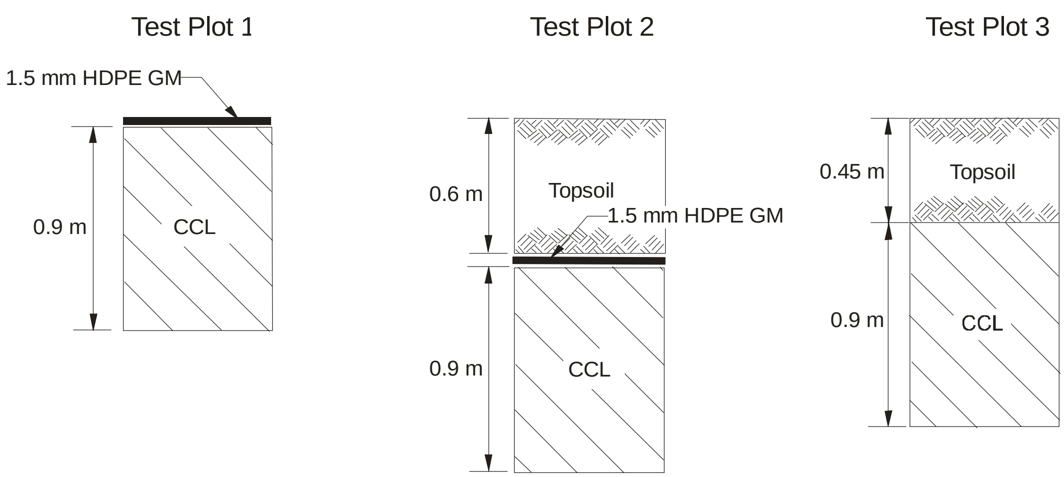


Figure 7-2. Cross Sections of Cover System Test Plots at a Landfill in Kettleman City, California (modified from Corser and Cranston, 1991).

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

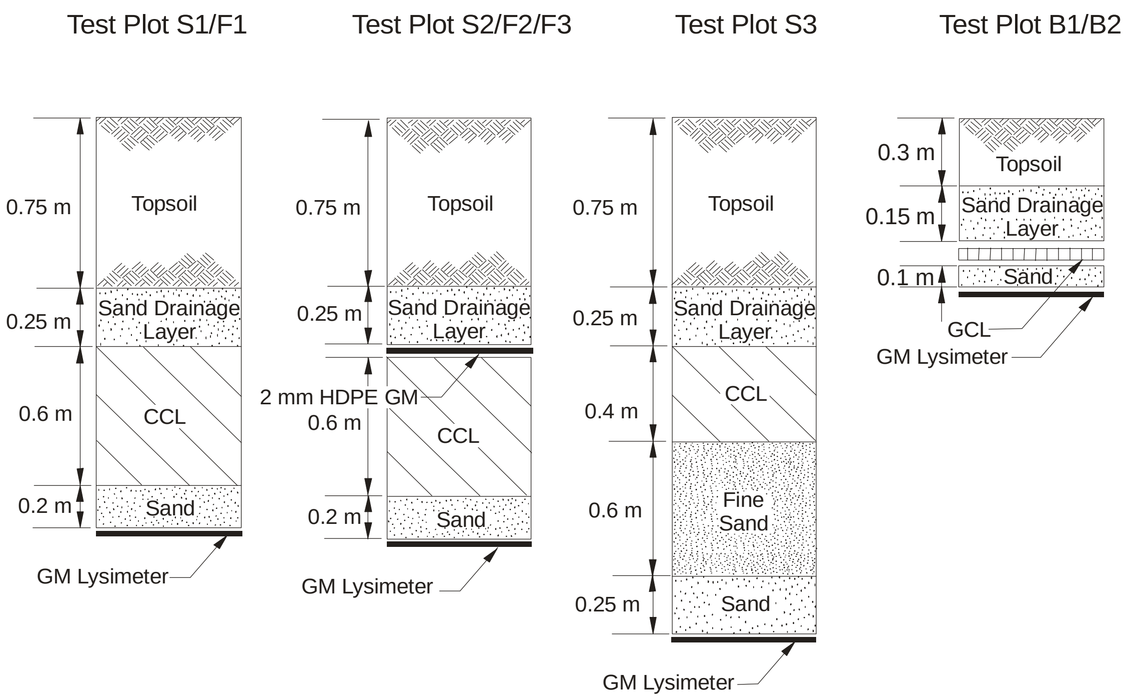


Figure 7-3. Cross Sections of Cover System Test Plots at a Landfill in Hamburg, Germany (modified from Melchior et al., 1994; Melchior, 1997a).

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 x 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

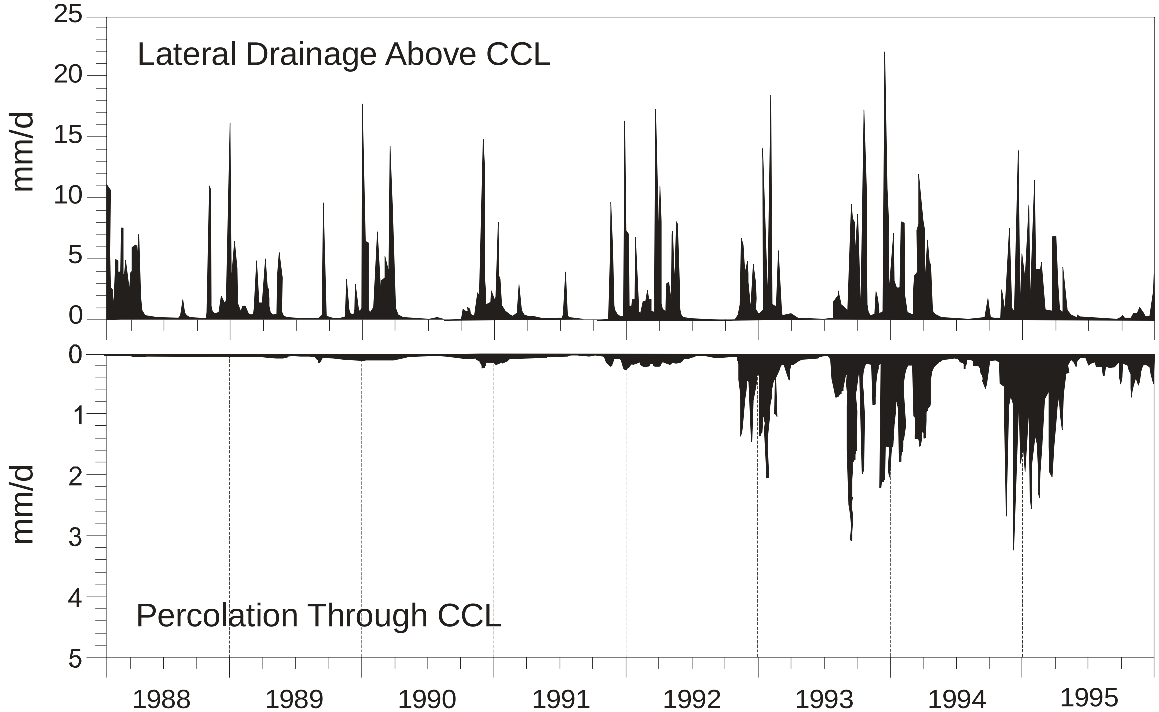


Figure 7-4. Summary of Field Data for Test Plot S1 with a CCL Barrier at a Landfill In Hamburg, Germany (modified from Melchior, 1997a).

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

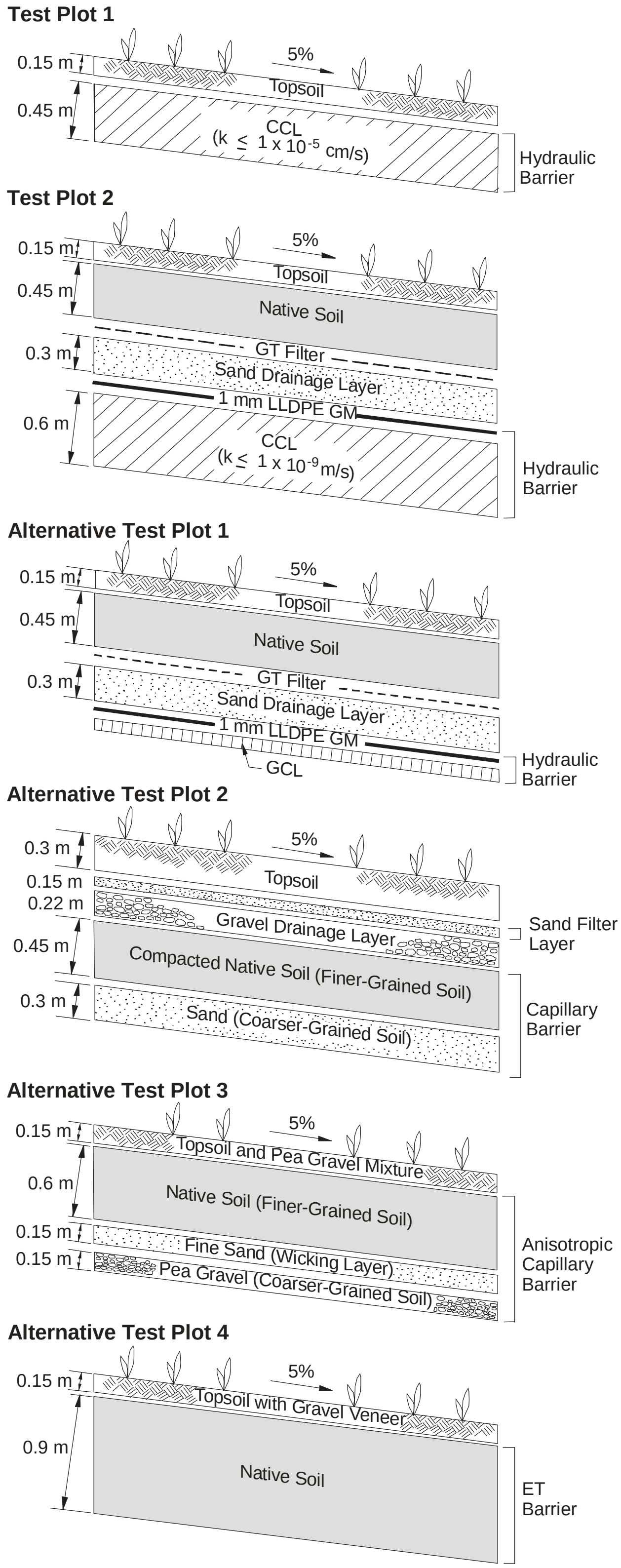


Figure 7-5. Cross Sections of Cover System Test Plots at Kirtland Air Force Base in Albuquerque, New Mexico (modified from Dwyer, 1997).

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 clean; and (ii) the area to be seamed must be dry.

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.

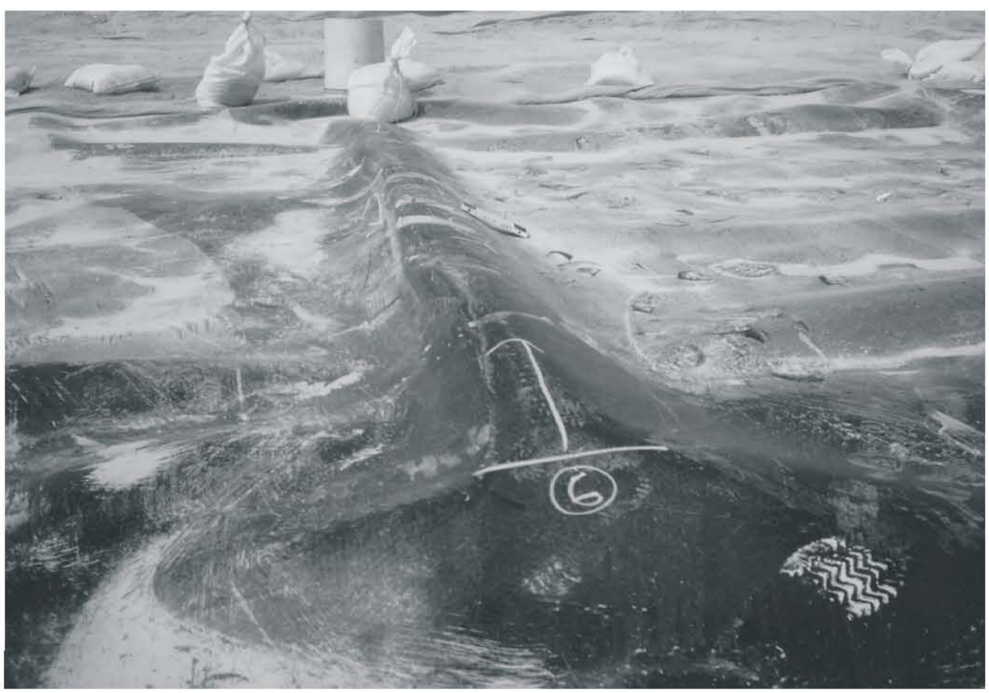


Figure 7-7. Wrinkles Developed in an HDPE GM and Folded Over During Placement of Cover Soil.

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 x 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.

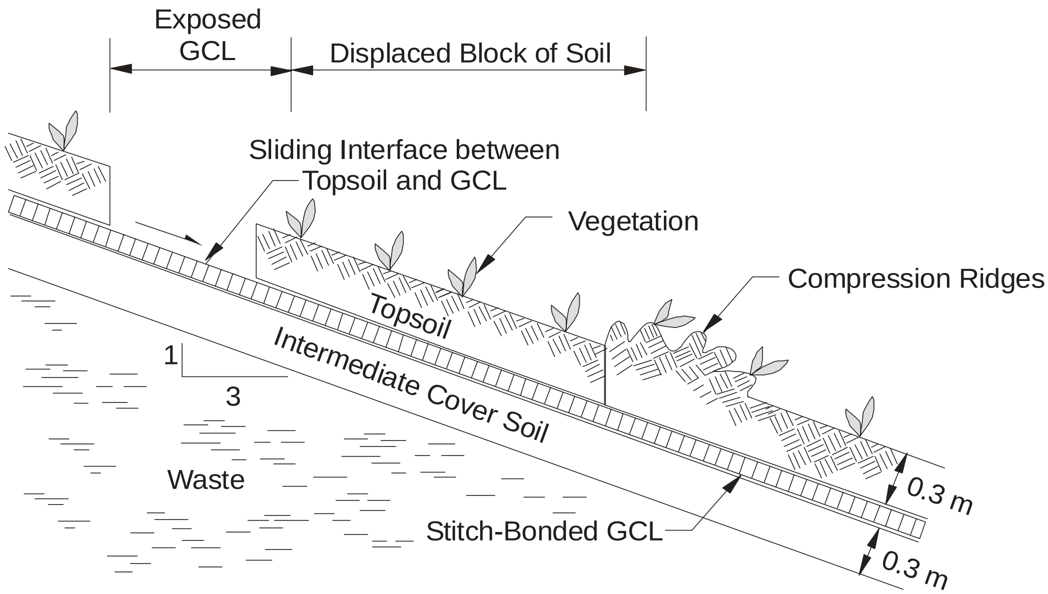


Figure 7-8. Observed Failure Mechanisms for Sliding of Soil Layer Over Stitch-Bonded GCL.

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:
    - o 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.
    - o 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.

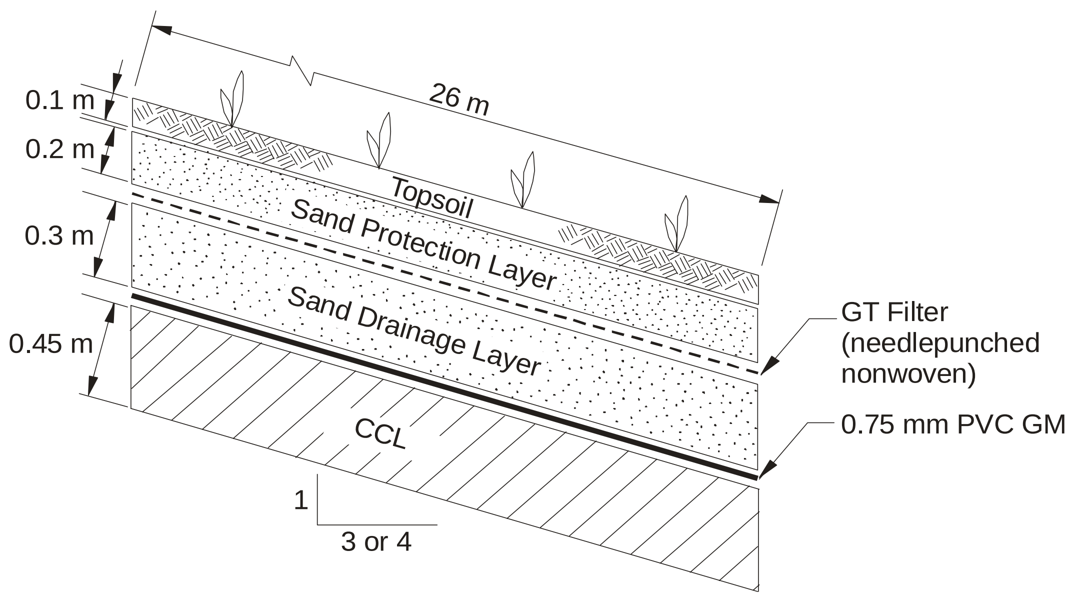


Figure 7-9. Cover System Cross Section for Northern Ohio Landfill that Underwent a Slope Failure after Thaw.

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.

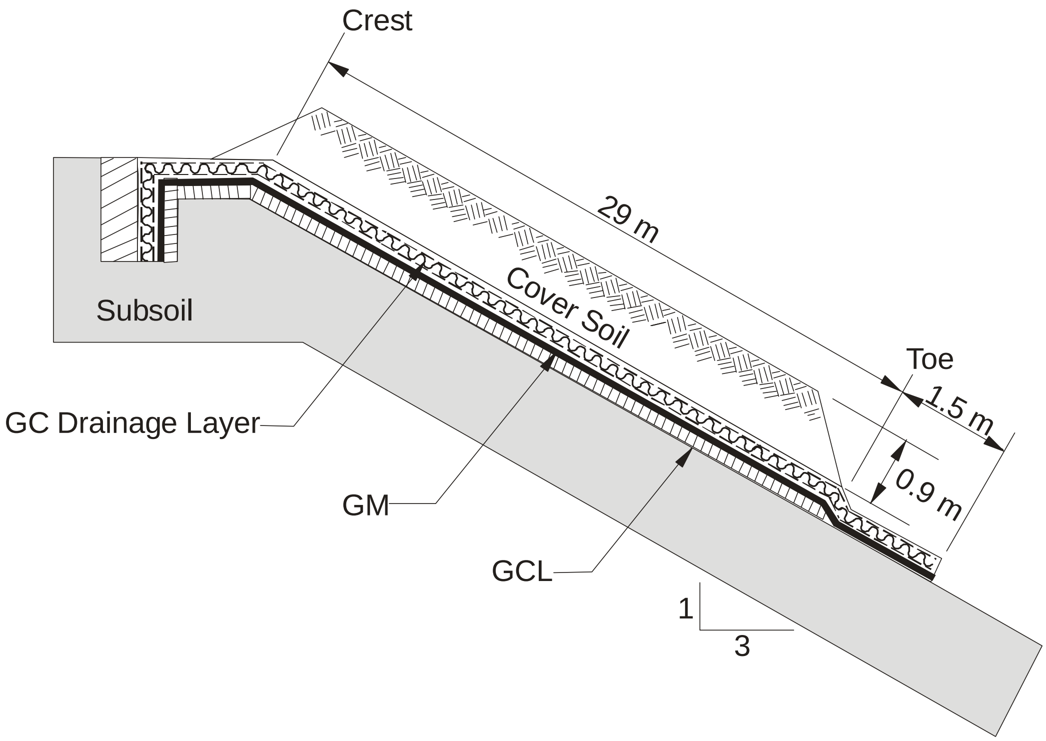


Figure 7-10. Typical Cross Section for 3H:1V Cover System Test Plot in Cincinnati, Ohio (modified from Carson et al., 1998).

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.

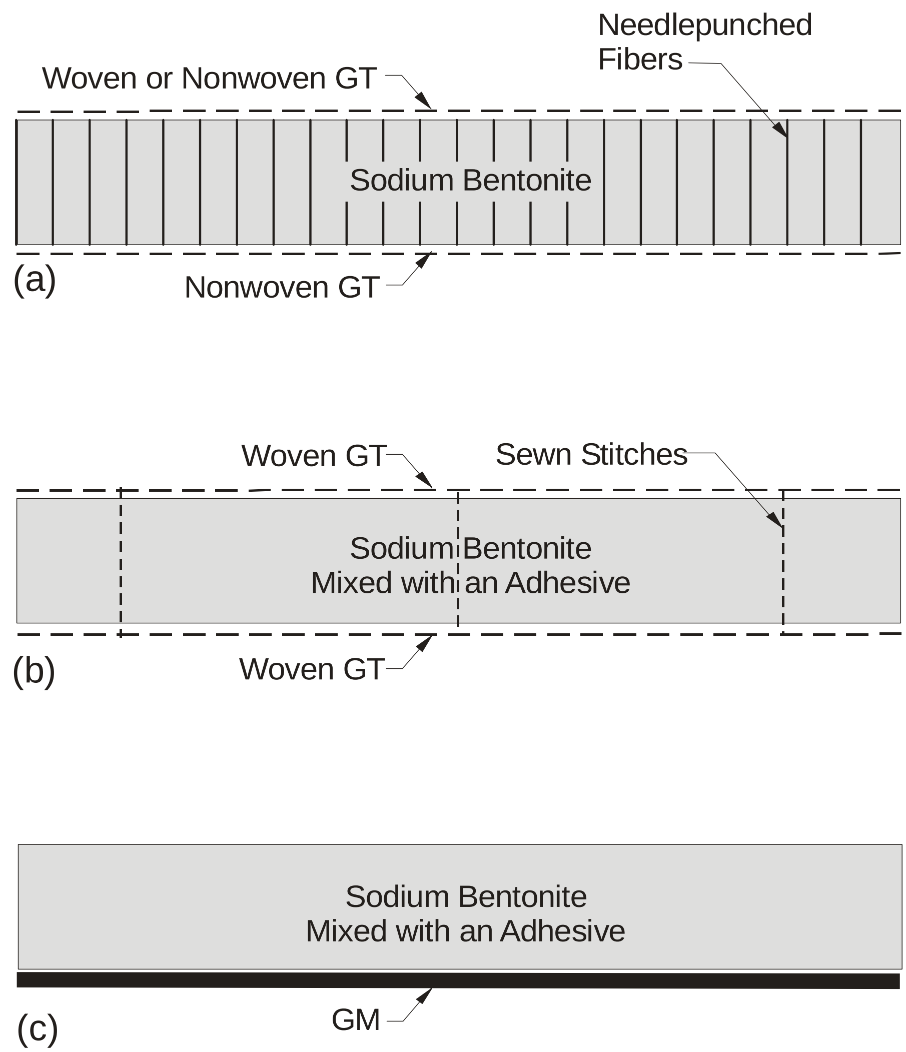


Figure 7-11. Schematics of GCLs Used in Cover System Test Plots in Cincinnati, Ohio: (a) Reinforced, GT-Encased, Needlepunched GCL (e.g., Bentofix and Bentomat); (b) Reinforced, GT-Encased, Stitch-Bonded GCL (e.g., Claymax); and (c) Unreinforced, GM-Supported GCL (e.g., Gundseal).

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.

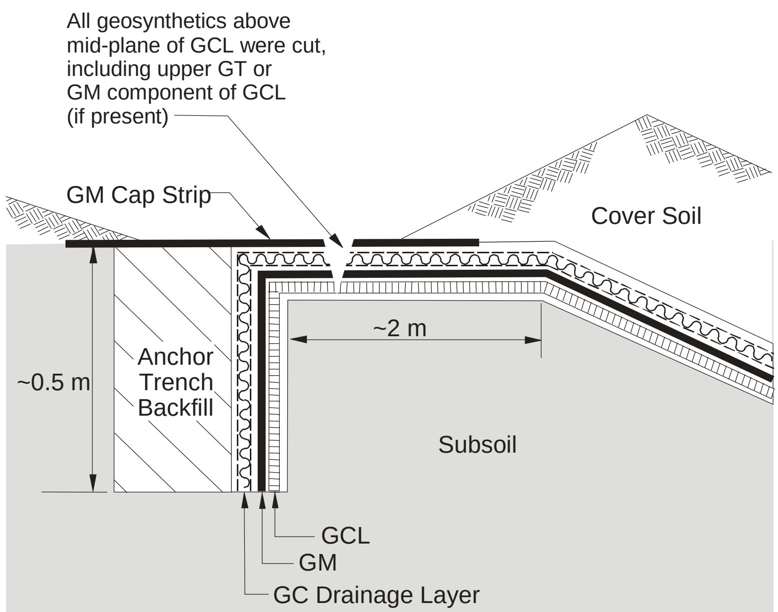


Figure 7-14. Cut in Anchor Trench Geosynthetics above Mid-Plane of GCL on Cincinnati, Ohio Test Plots.

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.

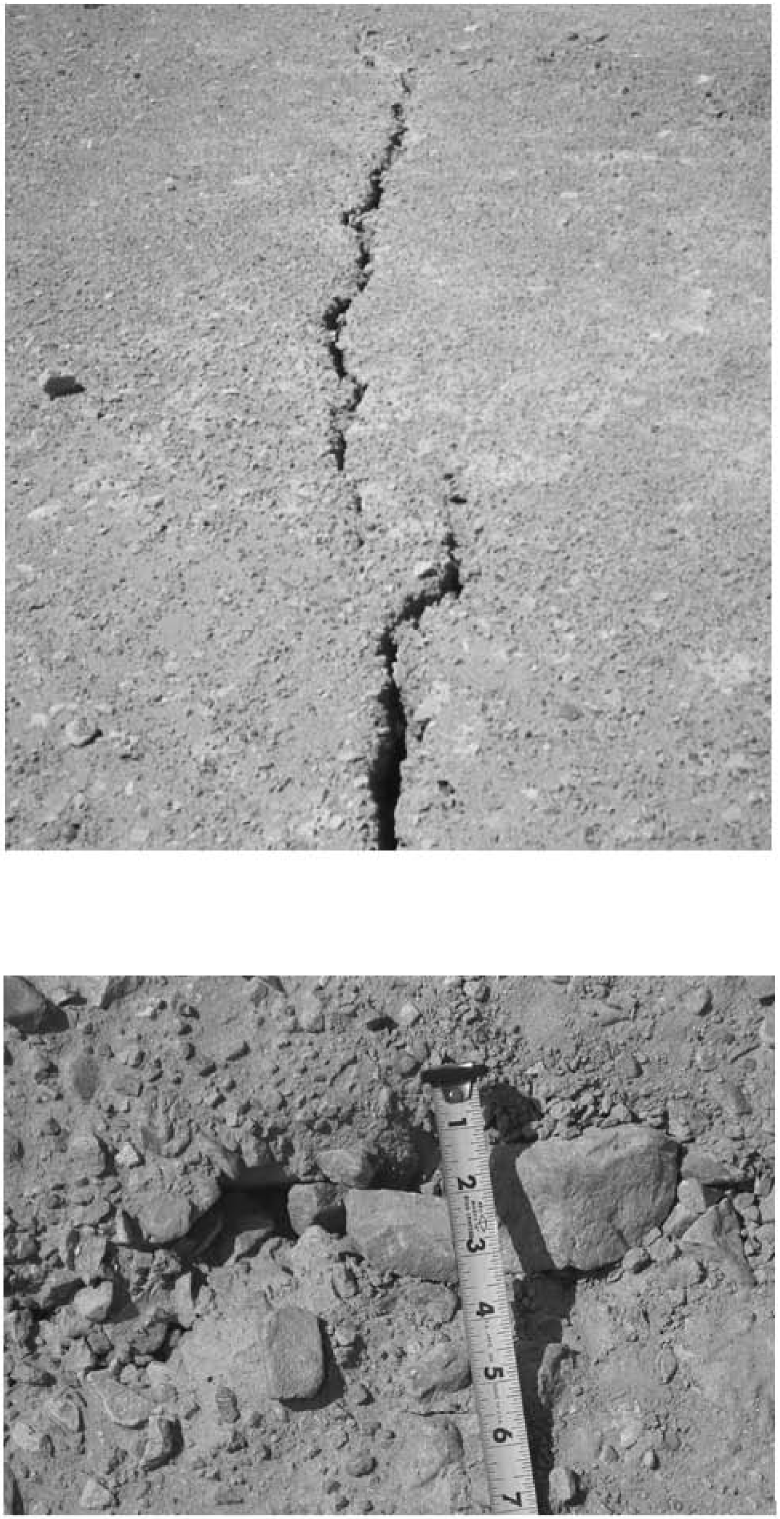


Figure 7-16. Surface Tension Cracks in Cover Soils from Differential Settlement of Underlying MSW.

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.

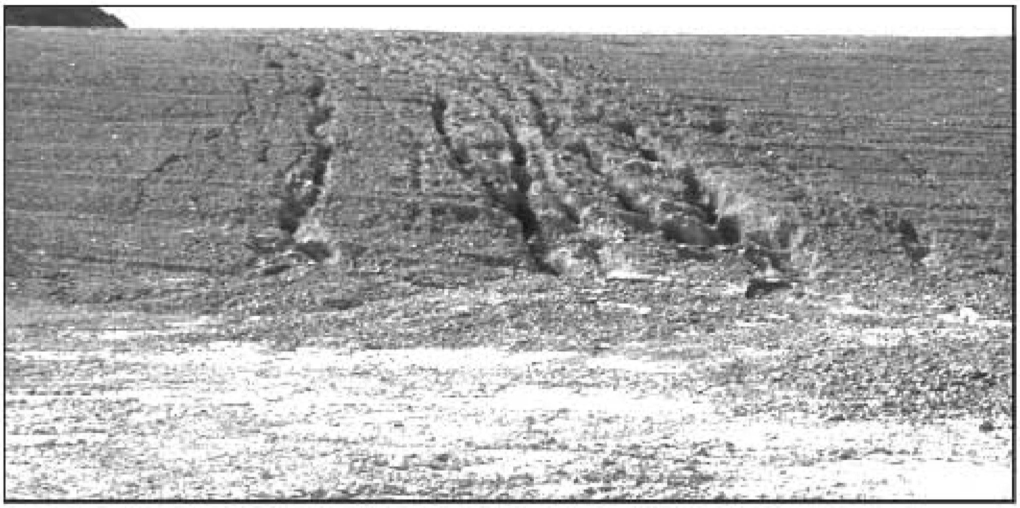
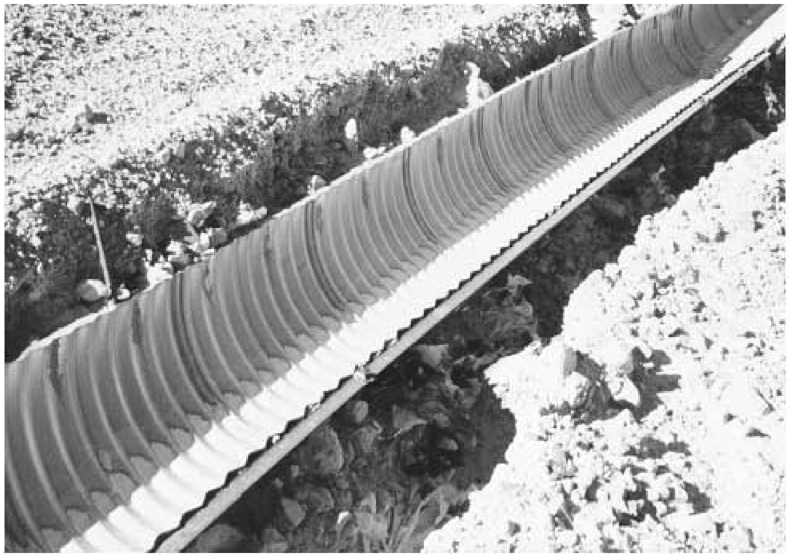


Figure 7-18. Gullies on a RCRA Subtitle D Cover System without Surface-Water Runoff Control System and Located in an Arid Setting.

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.



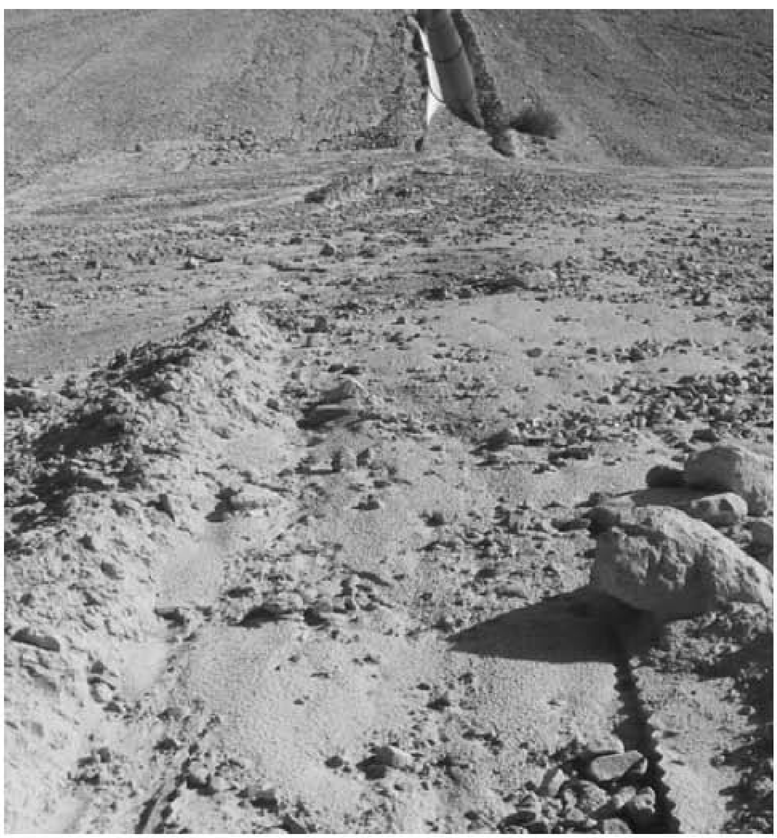


Figure 7-19. Failed Surface-Water Runoff Control System for Another Portion of the Closed MSW Landfill Located at an Arid Climate Site and Shown in Figure 7-18.

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.

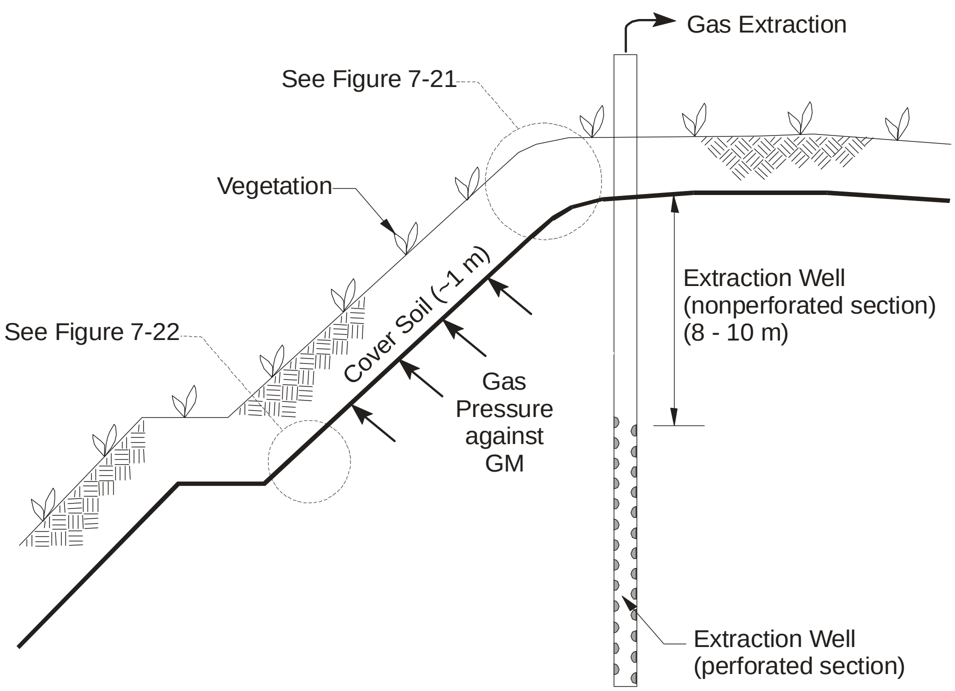


Figure 7-20. Gas Pressures Built Up Beneath Cover System of Closed MSW Landfill Because Upper Portion of Gas Extraction Wells Was Not Perforated.

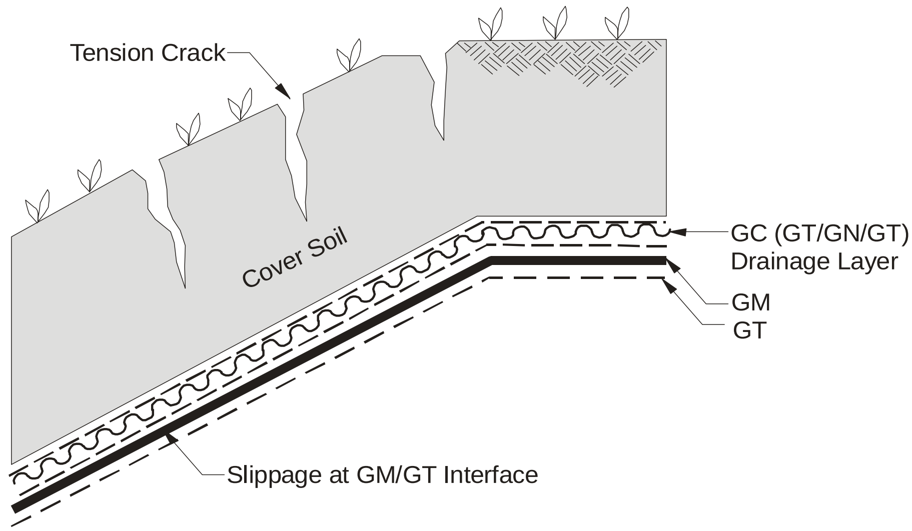


Figure 7-21. Gas Pressures Beneath Cover System GM Resulted in Slippage at GM/GT Interface with Straining of the GM and GC at the Slope Crest.

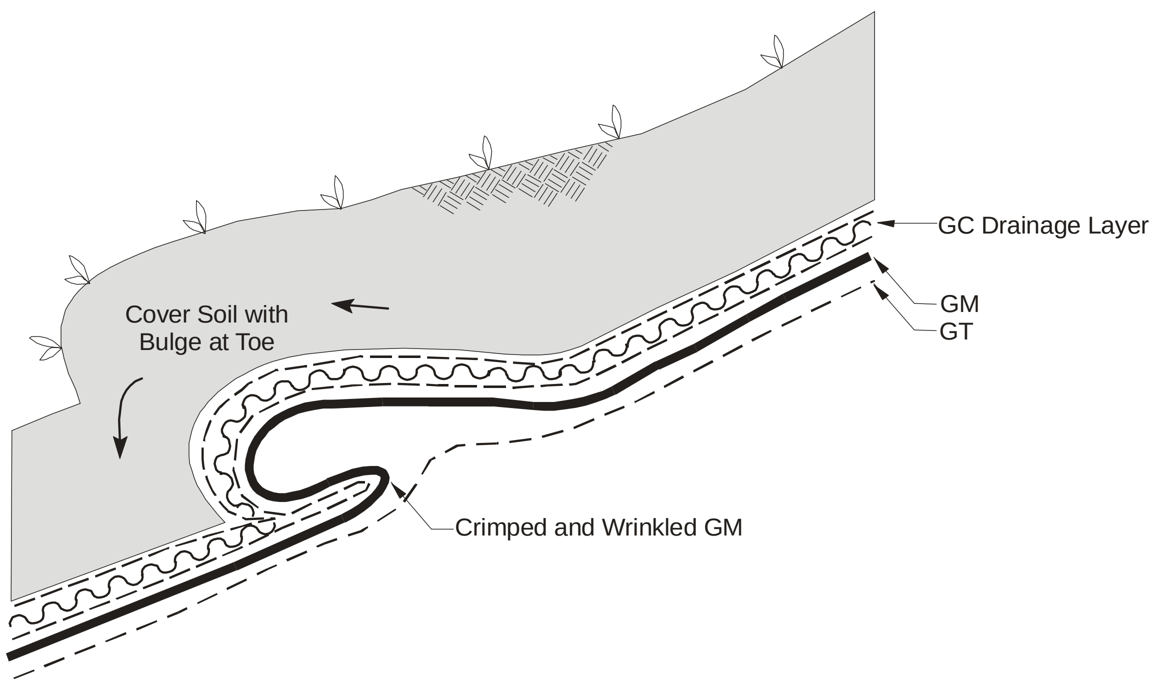


Figure 7-22. Slippage at GM/GT Interface Also Caused the GM and GC to Fold Over at the Slope Toe.

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.

Chapter 8
Performance Monitoring

8.1 Introduction

Performance monitoring of cover systems is necessary to both satisfy regulatory requirements and confirm the performance of a cover system. The feedback on the effectiveness of a cover system design can also improve future designs and performance predictions. As discussed by Kavazanjian (2000), development of performance monitoring data for geoenvironmental projects (such as cover systems) is often complicated by a number of factors:

- long time periods of interest;
- imperfect knowledge of phenomena and impacts, which is sometimes addressed by multi-parameter modeling, periodic review and updating of monitoring plans, and sensitivity studies;
- measurement of very small quantities or changes in a physical system, a factor that has resulted in the development of improved monitoring techniques and methods of statistical analysis and in the monitoring of surrogates; and
- difficulty in measuring parameters of interest, which is sometimes addressed by making indirect measurements (e.g., monitoring soil moisture rather than percolation through the cover system) or monitoring surrogates.

For MSW landfills and HW facilities, post-closure monitoring is required to assure that post-closure care needs are identified and addressed. Regulations for MSW landfills presented in 40 CFR 258.61(c) and regulations for HW facilities presented in 40 CFR 264.118 require facility owners or operators to prepare a written post-closure plan that includes a description of the performance monitoring activities and the frequency of such activities. The post-closure care period of 30 years given in RCRA regulations has generally been considered by EPA to be the minimum timeframe for performance monitoring and maintenance. EPA has the authority to designate a longer post-closure period under 258.61(b) if necessary for the continuing protection of human health and the environment. Requirements analogous to those given above for MSW landfills exist for HW disposal facilities in 40 CFR 264.117(a)(2)(ii). Also, the post-closure monitoring requirements for MSW and/or HW landfills will likely also be ARARs for any cover system that forms part of a CERCLA remediation. In addition, Five-Year Reviews may need to be performed, as described in "Comprehensive Five-Year Review Guidance" (June 2001) OSWER 9355.7-03B-P, EPA 540-R-01-007.

While performance monitoring is important for all facilities with a cover system, it is particularly such for closed facilities, such as old dumps and remediation sites, not underlain by engineered liner systems or leachate collection systems which themselves can be monitored. For these sites, percolation monitoring via a lysimeter (see Section 8.2.4) or soil moisture monitoring (see

Section 8.3) is recommended. Such monitoring is also recommended for alternative cover systems, including those with ET or capillary barriers (see Sections 3.2 and 3.3, respectively). Prior to implementing a monitoring program, it is important to establish the criteria (i.e., action levels) for acceptable performance. These criteria are typically developed on a project-specific basis and may consider the characteristics of the material being contained, human health and environmental risk, properties of the cover system components, hydrogeologic setting, and other factors. For example, as discussed in Section 1.2.3, EPA requires that a landfill cover system have a maximum percolation rate over the considered monitoring period to prevent the "bathtub" effect. Exceedance of site specific percolation criteria could trigger additional requirements for the landfill owner or operator. For example, the facility owner could perform an investigation of the higher than anticipated percolation rates, with the study including an assessment of the monitoring instrument accuracy and drift, condition of the in-place cover system components, anticipated performance based on modeling (maybe there was a significant weather event), and other tasks.

In a project-specific context, monitoring will provide the facility owner/operator, design engineer, regulators, and other stakeholders with the data necessary to evaluate whether project design criteria are being achieved. For the entire industry, additional data on the hydraulic and geotechnical performance of cover systems would be very beneficial to the development of improved materials, designs, construction procedures, and monitoring/maintenance procedures for these types of facilities. As previously noted in this guidance document, few data currently exist on the field hydraulic performance of cover systems and on their long-term structural integrity when subjected to total and differential settlements.

The types of monitoring systems addressed in this chapter are:

- infiltration monitoring systems (Section 8.2);
- soil moisture monitoring systems (Section 8.3);
- gas emissions monitoring systems (Section 8.4); and
- settlement monitoring systems (Section 8.5).

Other types of post-closure monitoring activities typically associated with waste containment facilities are not addressed herein. These include groundwater monitoring systems, landfill gas monitoring systems, and monitoring for physical conditions at the site, such as the condition of vegetative cover, erosion control structures, sediment control structures, leachate collection and removal system, landfill gas extraction system, etc. The condition of all of these latter systems and structures must be monitored during the post-closure period to assure adequate performance of the site in the long term and to comply with various regulatory requirements. These systems all require regular inspection and maintenance, topics which are addressed in Chapter 9.

8.2 Infiltration Monitoring

8.2.1 Overview

Infiltration monitoring can be performed indirectly, by monitoring leachate collection system flows for landfills containing such systems, or more directly, by monitoring the cover system internal drainage layer when one exists or by monitoring a lysimeter installed beneath the hydraulic barrier layer. Each of these techniques is described below.

8.2.2 Leachate Collection System Monitoring

Data on the quantity and composition of leachate generated within a landfill can provide significant insight into the performance of a cover system. In facilities underlain by a leachate collection system and composite liner, leachate flow data can be used as an indicator of cover system performance.

If a cover system is properly designed and installed, the rate of leachate flow into the leachate collection system will decrease with time, with the possible exception of cases where leachate recirculation is practiced. This trend is clearly seen in Figures 1-8 and 1-9. For a cover system designed and installed to prevent infiltration, the long-term leachate collection system flow rate would be expected to approach zero. If the cover system does not act as an effective hydraulic barrier, higher than anticipated long-term leachate flow rates might be observed.

The decrease in leachate collection system flow rate with time after closure can occur relatively rapidly (e.g., within a few months) in some cases or more slowly (e.g., over several years), depending on the type and thickness of waste, the waste's moisture content relative to its field capacity at the time of closure, and, to a lesser extent, the rate of waste degradation (for MSW). Evaluation of cover system performance during this transition period requires some judgment. Techniques that can be used to help in the evaluation include: (i) plotting leachate flow rates in the manner shown in Figures 1-8 and 1-9 to observe the time trend in flow rates; (ii) estimating the timeframe for residual drainage from the waste using Darcy's equation, the known thickness of the waste, an estimate of the unsaturated hydraulic conductivities of the waste and daily/interim cover materials, and an assumed hydraulic gradient equal to one; and (iii) looking for anomalies in the trend of leachate flow rate with time.

With respect to item (ii) above, timeframes for residual drainage calculated using Darcy's equation should be considered at best order-of magnitude estimates because of the difficulty in estimating an appropriate value for the unsaturated hydraulic conductivity of waste, the fact that the unsaturated hydraulic conductivity of waste is not constant but rather varies with matric potential, and the difficulty in accounting for such factors as channelized flow along preferential pathways in the waste and lateral flow at interfaces between waste and daily/interim cover layers. Based on experience, it appears that the use of Darcy's equation, coupled with published estimates for MSW waste permeability, will typically provide a conservative, overestimate for the timeframe for residual drainage of MSW. For example, for a 25-m thick MSW landfill with an assumed average unsaturated hydraulic conductivity of 1 x 10^{-7} m/s, the timeframe for residual drainage from the waste by gravity is 7.9 years.

With respect to item (iii) above, several different types of anomalies in flow rates can occur. If periodic increased leachate collection system flow rates are observed, the timing of the increases should be compared to the timing of precipitation events at the project site. A correlation between the two is potentially indicative of a breach in the cover system. The most common potential breach locations are around gas well penetrations through the cover system and at the edge of the cover system around the perimeter of the facility. If the increased flow rates were due to a breach and associated influx of precipitation, the concentrations of leachate constituents in the leachate collection system flow would also be expected to lower (i.e., the flow is more dilute) during the period of increased flow than during other periods. If the increased flow rates do not correlate with precipitation, other sources need to be investigated. Another potential source involves the release of a slug of leachate from the waste to the leachate collection system. If the source of the flow anomalies is slug flow along preferential pathways in the waste (as opposed to uniform, porous media-type flow), leachate constituents would be expected to be similar to those at earlier times. However, the constituent concentrations may potentially be lower during the anomaly than at earlier times if a significant amount of the leachable constituents have already been transported from the pathways. Another potential source of long-term leachate flow for some older landfills is groundwater infiltration, either from perched water zones or from a continuous zone of saturation that rises above the bottom of the facility. Indicators of groundwater infiltration include relatively dilute leachate chemistry, changes in leachate predominant ion chemistry, and correlation between leachate collection system flow rates and changes in groundwater levels at the site.

It should be noted that the observation of a reduction in leachate collection system flow rate with time after closure does not by itself prove that a cover system is functioning as designed. The observed reduction in flow rate after closure may be due to decreasing residual drainage from the waste, with percolation into the waste reduced from the pre-closure value, but still at a rate above the intended design value. A slow rate of percolation into the waste may not be reflected in leachate collection system flow rates for some period of time, due to available moisture absorption capacity of some or all of the solid waste mass.

In summary, monitoring of flow from the leachate collection system is extremely valuable from the standpoint of evaluating the performance of the entire waste containment facility. This type of performance monitoring also provides a valuable indication that the cover system is (or is not) functioning as designed. However, if the actual performance of the cover system must be quantified or definitively demonstrated, more direct monitoring methods will need to be used.

8.2.3 Drainage Layer Monitoring

As previously discussed in Section 1.5.3, conventional RCRA-type cover systems may require a drainage layer installed between an overlying protection layer and underlying hydraulic barrier (Figure 1-12), particularly on sideslopes. Drainage from this layer can be monitored: (i) as confirmation that the layer is functioning as intended; and (ii) to generate data on the water balance for those components of the cover system above the drainage layer. Flow from the drainage layer can only be quantified if the layer is designed to convey flow to not more than a few discrete discharge points. At the discharge points, the flow rate can be monitored using a flowmeter, tipping bucket, pore pressure transducer, or other means. If the drainage layer simply daylights at the edge of the cover system and discharges as sheet flow to the surrounding area or surface-water drainage structure, such as shown in Figure 2-5(a), quantitative monitoring will not be possible. The need for monitorable discharge points results in a trade-off because, while it is beneficial to collect monitoring data, construction of the discharge points may complicate the design for some projects where simply daylighting the drainage layer would otherwise suffice.

Currently, drainage layer monitoring is not routinely performed; it is usually only conducted for a cover system test plot as part of a water balance assessment.

8.2.4 Lysimeter Monitoring

Lysimeters have long been used for agricultural and hydrologic studies to collect deep drainage or percolation data or to estimate recharge. Lysimeters have been used to monitor percolation through cover systems with hydraulic, ET, and capillary barriers. The most common approach is to use a collection lysimeter, also called a pan lysimeter or drainage lysimeter. Other types of lysimeters, including monolithic lysimeters, weighing lysimeters, and suction lysimeters, have been used for various types of research studies, but not specifically for evaluation of installed cover systems. The principal advantage of collection lysimeters is that, when properly designed, they provide a direct measure of soil-water flux. Lysimeters perform best when they are installed during cover system construction. When installed after the fact, great care is needed to assure that the boundary conditions (e.g., vegetation and soil properties) above and adjacent to the lysimeter are similar to the characteristics found elsewhere in the cover system.

The use of a collection lysimeter for percolation monitoring of a cover system is illustrated in Figure 8-1. A lysimeter of the type shown in Figure 8-1 is constructed with hydraulic barrier (typically GM) beneath or within the soil profile to be monitored. The liner is shaped to contain percolation and is typically backfilled with a granular (sand or gravel) or geosynthetic drainage layer. A geosynthetic drainage material may be preferred to a granular drainage material because it has lower storage capacity and faster response time than most granular drainage materials. Also, when a granular drainage material is used, it can impact the boundary conditions of the cover systems and create the capillary barrier effect described in Chapter 1. Liquid collected in the lined lysimeter drains by gravity to a monitoring point, where the flow is collected and periodically measured with a pore pressure transducer, float and a pulse generator, tipping bucket, or other means. To date, few lysimeters have been installed beneath full-scale cover systems; instead, they have been installed beneath cover system test plots. However, this statistic

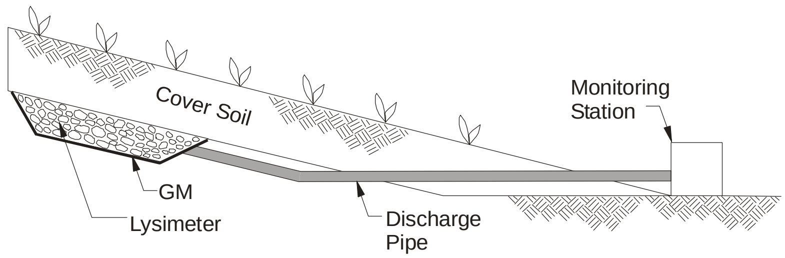


Figure 8-1. Example of a Collection Lysimeter Used to Monitor Percolation.

is changing since collection lysimeters are being installed beneath full-scale cover systems as part of the ACAP program, which was discussed previously in Section 3.4.3. Generally, the larger
the lysimeter, the more representative the monitoring results of the performance of the cover system as a whole.

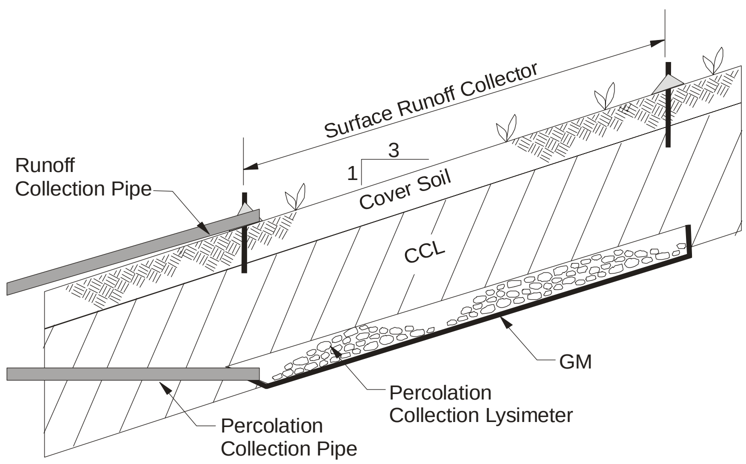


Figure 8-2. Collection Lysimeter and Runoff Collection Pipe Used to Monitor Percolation and Runoff at the Omega Hills, Wisconsin Test Plots Described in Section 7.2.1.

As described by Bonaparte et al. (2002), it appears that the best way to document the field performance of CCLs in cover systems is with the use of lysimeters installed at the base of the cover system. Five case studies reporting on the use of lysimeters to monitor percolation through cover systems (e.g., Dwyer, 1997, 1998, 2001; Melchior, 1997a,b and Melchior et al., 1994; Montgomery and Parsons, 1989, 1990; Nyhan et al., 1997; Paige et al., 1996) were described previously in Section 7.2. A sixth case study using a similar technique was described in Section 4.3.4 (Lane, 1992; Khire, 1995; Khire et al., 1997, 1999). The test plot and lysimeter set-up for the third and sixth case studies are illustrated in Figures 8-2 and 8-3, respectively. These figures also illustrate how surface runoff was monitored for the test plots. Other examples of the use of lysimeters in test plots are given by Webb et al. (1997) and Gee et al. (1997).

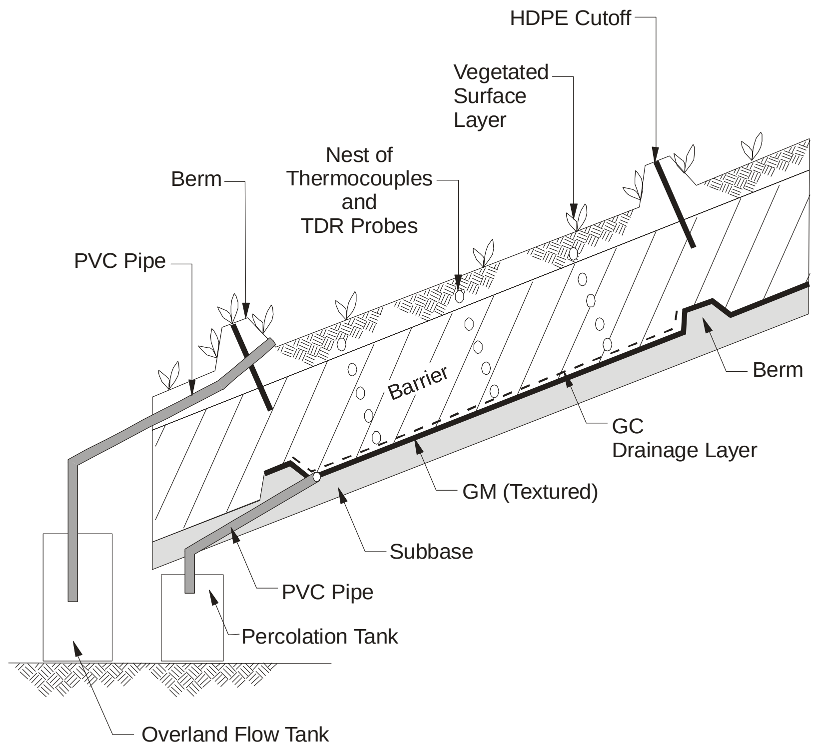


Figure 8-3. Collection Lysimeter and Runoff Collection Pipe Used to Monitor Percolation and Runoff at the Live Oak, Georgia and Wenatchee, Washington Test Plots Described in Section 7.2.4.

8.3 Soil Moisture and Matric Potential Monitoring

8.3.1 Overview

Soil moisture and matric potential measurements can be used to assess soil moisture or matric potential content at discrete locations, changes in cover system water storage, and vertical gradients in cover system soils. With careful calibration, the measured moisture contents can be converted to matric potentials, and vice-versa, through the use of an acceptable soil-moisture characteristic curve. Currently available techniques of assessing soil moisture content in cover systems include neutron probes, time domain reflectrometry (TDR) probes, and frequency domain reflectometry (FDR) probes. Methods of measuring soil matric potential include tensiometers, electrical resistance sensors, thermocouple psychrometers, and heat dissipation sensors. TDR and FDR probes have also been used to measure matric potential when they have been combined with a matrix material whose water retention function has previously been determined. With these modified sensors, the matrix material around the TDR or FDR probes comes into equilibrium with the surrounding soil, and the water content (and indirectly the matric potential) of the matrix material is measured with the probes. These modified probes will not be discussed further.

All of the soil moisture and matric potential monitoring methods listed above are non-destructive, in-direct techniques. With the exception of thermocouple psychrometers, good contact between the sensor and the soil (or borehole casing for neutron and FDR probes) is required to obtain accurate measurements. This is especially critical for sensors that measure matric potential and rely on good hydraulic contact with the soil to establish thermodynamic equilibrium. Good hydraulic contact may be hard to attain in very coarse soils, such as gravel, and in shrink-swell clays. Except for the neutron probe, all of the sensors can be fully automated.
Soil moisture and matric potential measurements may also be made directly on soil samples excavated from the cover system. While this latter method is reliable for determining soil moisture content or matric potential, it involves destructive sampling (i.e., damage) of the cover system soil and the inherent problem of sample variability associated with the destructive sampling protocol.

8.3.2 Neutron Probes

The neutron probe, when calibrated, can yield very good indirect measurement of soil moisture content. The probe is inserted into a cased access borehole, orientated in any direction, where readings are taken at various locations (Figure 8-4). The casing material is generally aluminum or PVC piping. The principle of operation is based upon the neutron thermalization process, wherein a radioactive source emits high-energy neutrons, with an energy of about 5 MeV, into the soil. These neutrons are then reduced to a lower energy state upon colliding with hydrogen atoms associated with soil water (Gardner, 1987). After an average of 19 collisions, the neutrons cease to lose further energy and are said to be "thermal" neutrons with an energy of approximately 0.025 MeV. Higher molecular weight elements, such as oxygen, also slow the neutrons, but far fewer collisions are required with hydrogen to slow the reaction to thermal energy levels. The source of the high-energy neutrons in most commercially available neutron probes is a radioactive americium and beryllium mix. The americium emits an alpha particle that bombards the beryllium atoms, which, in turn, emit a neutron. The fast neutrons are emitted approximately radially from the source and form a sphere around the source within which the neutrons are attenuated. The size of this spherical influence varies inversely with the moisture content. The sphere is about 0.7 m for dry soil and about 0.16 m for saturated soil; sphere diameter is unaffected by the strength of the radioactive source (Gardner, 1987). The number of pulses counted by the probe detector is proportional to the number of thermal neutrons encountered. A calibration curve can be developed to correlate count rate with soil moisture content.

While the probe's manufacturer usually supplies calibration curves, calibration for each application is recommended. A calibration involves taking multiple readings in a given soil against a range of gravimetrically-determined moisture contents. Soil heterogeneity and organic matter can have adverse affects on accuracy of neutron probe readings. Also, extraneous

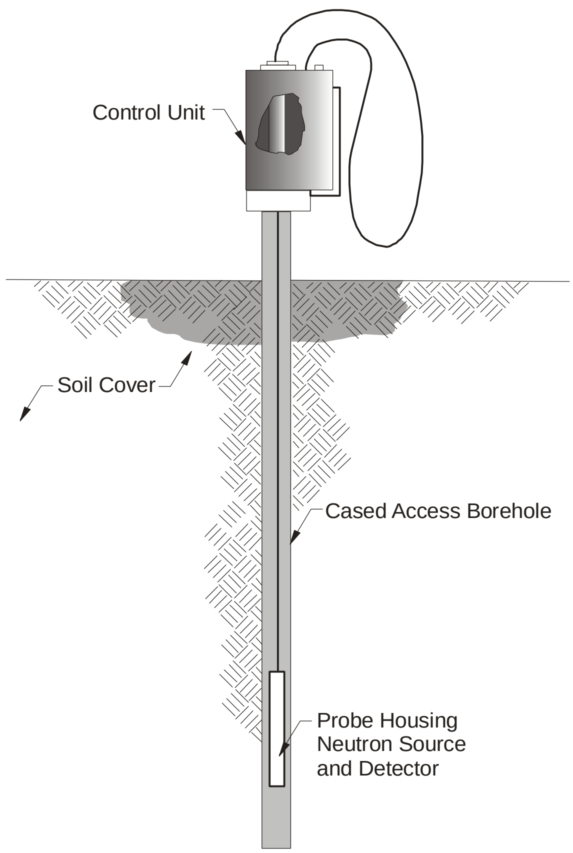


Figure 8-4. Neutron Probe Installed in a Vertical Cased Borehole.

hydrogen atoms not associated with water can also impact probe accuracy. Potential sources of the extraneous hydrogen atoms include hydrocarbons, methane gas, hydrous minerals (e.g., gypsum), hydrogen-bearing minerals (e.g., kaolinite, illite, and montmorillonite), and organic matter in the soil. Irregularities in the borehole casing or contact with the soil around the perimeter of the borehole can also produce error in moisture content values obtained. A disadvantage to the use of a neutron probe is the fact that a radioactive source is present, thereby posing a potential hazard for the operator as well as imposing difficulty in its use and maintenance (i.e., regulatory constrains). In addition, because of the regulatory constraints for using the radioactive source, this method cannot be automated.

In cover system monitoring applications, neutron probes are typically placed into access tubes in the cover system, and water content measurements are made at discrete locations at discrete time intervals. Neutron probes have been used to monitor soil moisture content in cover systems at a number of sites (e.g., Montgomery and Parsons, 1989, 1990; Nyhan et al., 1990; Fayer et al., 1992; Anderson et al., 1993; Schultz et al., 1995; Paige et al., 1996).

8.3.3 Time Domain Reflectometry

The process of sending electromagnetic pulses through a conductor and observing the reflected waveform is called time domain reflectometry (TDR). When monitoring soil moisture, TDR equipment generally consists of a cable tester or a specially designed commercial TDR unit, coaxial cable, and a stainless steel probe (Figure 8-5). The type of material surrounding the conductor (i.e, cable and probe) influences the waveform traveling down the conductor. The waveform is reflected differently when it reaches the start of the probe and the end of the probe.

Figure 8-5. TDR Probe and Coaxial Cable.

The time-of-travel along the conductor is dependent on the dielectric constant of the surrounding medium (e.g., the sheath around the cable or the soil around the probe). If the dielectric constant of the medium surrounding the conductor is high, the electronic signal propagates more slowly. Because the dielectric constant of water is much higher than most materials, a signal within a wet or moist medium propagates slower than in the same medium when dry. The dielectric constant of water is about 80, whereas the dielectric constant of dry soil is typically in the range of about 3 to 5. Ionic conductivity affects the amplitude of the signal but not the propagation time. Thus, soil moisture content around the probe can be assessed by a pre-determined correlation between time-of-travel along the probe (obtained from analysis of the reflected waveform) and soil moisture content. A generic calibration equation developed by Topp et al. (1980) is sometimes used. However, the probes should be calibrated for their specific application (e.g., soil texture and density and cable length) to yield accurate soil moisture measurements (Lopez and Dwyer, 1997).

The accuracy of TDR for soil moisture measurements is relatively good for many soil types and, according to Schofield et al. (1994), about the same as that for neutron attenuation. A disadvantage of TDR is the fact that accuracy decreases with increased cable length between the probe and the cable tester; generally a maximum range of about 60 m is recommended. In addition, soils with a high moisture content and a high electrical conductivity rapidly attenuate the electrical pulse before it is reflected back. If the attenuation is great enough there will be no return signal and the probe cannot be used. However, probes can be coated to reduce signal attenuation.

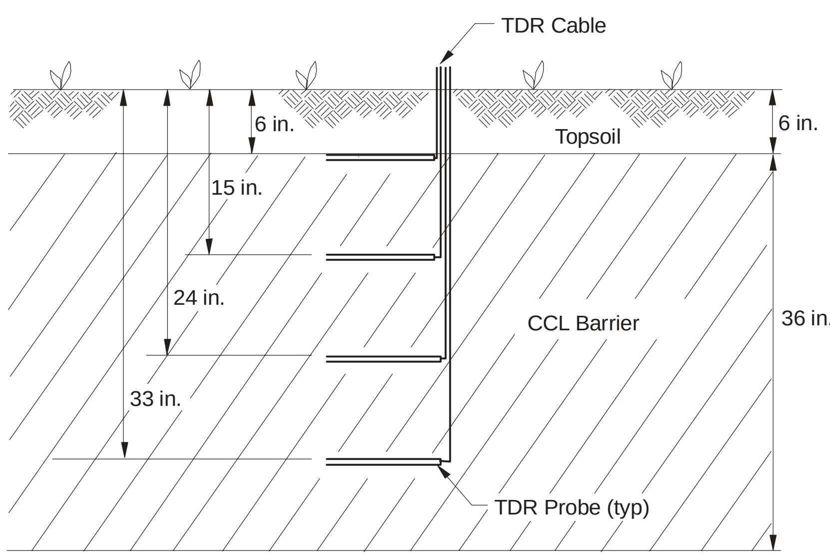


Figure 8-6. Example of Horizontally Orientated TDR Probes in a Cover System.

Probes may be installed during or after construction. They can be installed in any direction; however, when installed after construction, they are usually inserted vertically. When installed in this fashion, care should be taken to minimize the soil disturbance around the probe such that the probe fits snuggly in the soil. There have been cases where a space formed between the probe and soil during installation such that water was able to infiltrate into the space and short-circuit the cover system during heavy rainfall events. Consequently, the water content measurements at the probe were not representative of the surrounding cover system soils. Recent developments have attempted to minimize the cable length problem and reduce the cost of the TDR system. The latest development is a probe that does not require a cable tester or TDR unit but rather connects directly to a data logger. Calibration similar to the traditional TDR system is required for best results. The probe consists of two stainless steel rods connected to a printed circuit board. A five-conductor cable is connected to the circuit board to supply power, activate the probe, and monitor pulse output. The circuit board is potted in an epoxy block.

TDR has been used to monitor soil moisture content in cover systems at a number of sites (e.g., Dwyer, 1997, 1998, 2001; Kavazanjian, 2000; Khire, 1995; Khire et al., 1997,1999; Lane et al. 1992; Montgomery and Parsons, 1989, 1990; Nyhan et al., 1997). The use of TDR for soil moisture content monitoring is illustrated in Figure 8-6.

8.3.4 Frequency Domain Reflectometry

Frequency domain reflectometry (FDR) methods of soil moisture content measurements are also known as radio frequency (RF) capacitance techniques. These techniques actually measure soil capacitance. The probe contains a pair of electrodes and the soil serves as the dielectric medium completing a capacitance circuit comprising part of a feedback loop of a high frequency transistor oscillator. As high frequency radio waves (about 150 MHz) are pulsed through the capacitance circuitry, a natural resonant frequency dependent upon the soil capacitance is established. The soil capacitance is related to the dielectric constant by the geometry of the electric field established around the electrodes. Either the natural resonant frequency or the frequency shift between the emitted and received frequencies is recorded.

The FDR probe is often used in an access tube (cased borehole) similar to the neutron probe for measuring soil moisture content at various depths. In this application, it is important that the access tube be sized to provide a snug fit around the probe, thereby minimizing annular air gaps that greatly affect the travel of the electronic signal into the soil. Installation of the access tube also requires special attention to ensure complete soil contact with the casing since annular air gaps or soil cracks around the outside of the tube also produce erroneously-low readings.

Though the FDR probe manufacturer may provide calibration curves, it is important that the probe be calibrated with the site-specific soil. With proper calibration and use, the accuracy of the FDR method for measuring soil moisture content is good.

8.3.5 Tensiometers

A tensiometer measures soil matric potential values between 0 and approximately -90 kPa. The range of measurement is limited by the cavitation of water, which occurs at matric potentials less than -100 kPa. A tensiometer commonly consists of a high air entry, porous ceramic cup connected to a pressure measuring device through a rigid plastic tube (Figure 8-7). Plastic is the preferred material for the tube because of its non-corrosive nature and lower heat conduction properties. The tube is sealed at the top with a removable cap allowing the tensiometer to be filled with deaired water and accumulated air to be purged. A Bourdon gauge, manometer, or

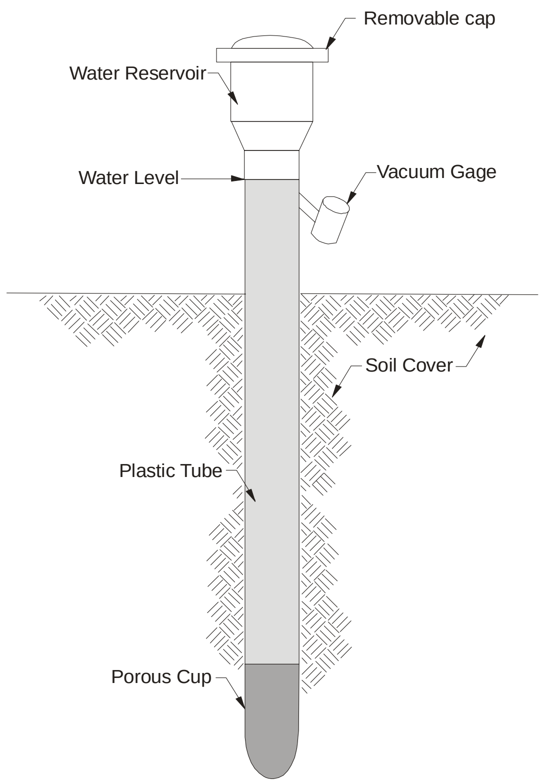


Figure 8-7. Tensiometer.

pressure transducer is attached to the upper portion of the water-filled tube to measure the negative pressure of the water in the tensiometer. The matric potential of the soil is equal to this negative pressure plus a pressure correction that accounts for the elevation potential of the water column in the tensiometer.

When the tensiometer is inserted into the soil, the soil absorbs water from the tensiometer and as this occurs the water pressure in the tensiometer decreases until the tensiometer fluid pressure is in equilibrium with soil water matric potential outside the cup. Tensiometers are limited to moist soils.

8.3.6 Electrical Resistance Sensors

Electrical resistance sensors have been used for over 60 years in agricultural applications (Bouyoucos and Mick, 1940). They consist of electrodes embedded in a gypsum, nylon, or fiberglass porous material that equilibrates with the surrounding soil. During equilibrium, water and solutes exchange between the sensor and the soil; therefore, the matric potential of the sensor is the same as that of the soil after equilibrium. Although electrical resistance varies primarily with water content, the equilibrium between the sensor and the soil is a matric potential rather than a water content equilibrium. These dual relationships result in a hysteretic relationship between the sensor's electrical resistance and matric potential. In practice, the sensors are more often calibrated to soil water content than to matric potential.

The electrodes in electrical resistance sensors have leads connected to a Wheatstone bridge to measure resistance. When the sensor is placed in firm contact with the soil, water flows into or out of the sensor until equilibrium is established. As the moisture content of the resistance block decreases, the electrical conductivity of the block decreases and the electrical resistivity of the block increases. Ohmmeters are used to measure resistance. The upper measurement range of the sensors is controlled by the air entry pressure of the sensor matrix material, and the lower limit depends on the range in smaller pore sizes of the sensor matrix. For gypsum blocks, the upper limit is approximately –30 kPa (Bourget et al., 1958) and the lower limit is approximately –1000 kPa (Tanner et al., 1952; Bourget et al., 1958). Additional discussion of gypsum blocks and fiberglass moisture sensors are given below.

Daniel et al. (1992) described gypsum blocks as prismatic or cylindrical blocks of gypsum that change electrical resistance when they change moisture content. The gypsum block is placed in the soil and the gypsum either takes in water from or gives up water to the surrounding soil until thermodynamic equilibrium is established. The electrical resistance of gypsum varies with moisture content: the higher the moisture content, the higher the electrical conductivity and, hence, the lower the electrical resistance. Because gypsum is partly soluble in water, it gives the sensor a buffering capacity that makes it insensitive to soil electrolyte concentrations less than about 300 ppm (2 mmhos/cm). However, for salt concentrations greater than 5,000 ppm in the surrounding soil, the electrolyte concentration in, and electrical resistance of, gypsum blocks can be affected. As a result of their solubility, gypsum blocks placed in wet soils tend to disintegrate. However, resins may be added to gypsum to improve their longevity. It has been reported that gypsum blocks may function for more than 5 years in dry soils but as little as 3 months in wet soils.

Daniel et al. (1992) report that fiberglass sensors work in much the same way as gypsum blocks; however, they don't have the buffering capacity that is provided by the dissolving gypsum. A porous fiberglass cloth is placed in the soil; the fiberglass gains or loses water until thermodynamic equilibrium is reached. A temperature-measuring probe may be a part of the unit.

Both gypsum blocks and fiberglass sensors were used to monitor the performance of the GCL test plots described previously in Section 7.4.5. The shapes and dimensions of the sensors used in the test plots are shown in Figure 8-8, and a typical placement of the sensors within a test plot cover system cross section is indicated in Figure 8-9. As indicated by Figure 8-9, the gypsum blocks were placed in the subgrade beneath the cover system geosynthetics. The fiberglass sensors were placed at the subgrade/GCL and GCL/GM interfaces.

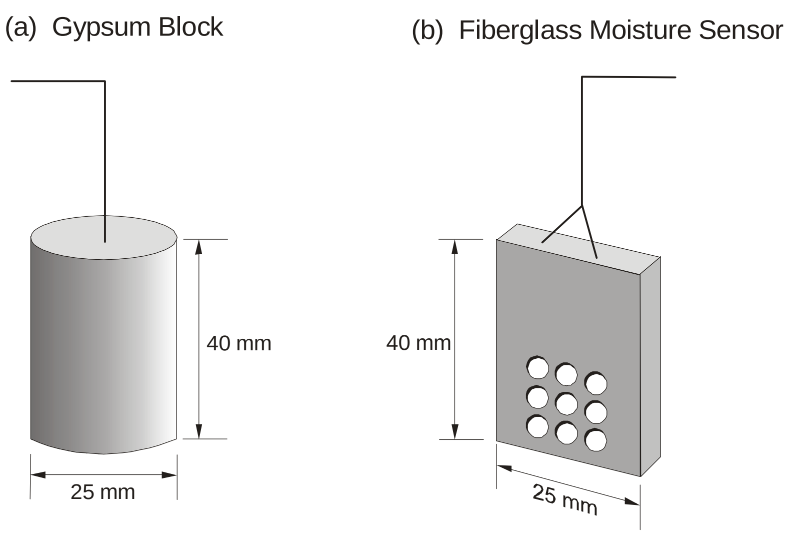


Figure 8-8. Dimensions of Electrical Resistivity Sensors Used in GCL Test Plot Described in Section 7.4.5.

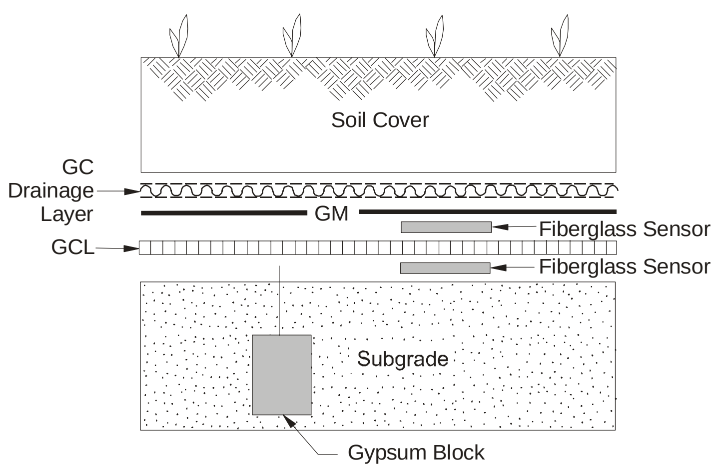


Figure 8-9. Layout of Electrical Resistivity Sensors Used in GCL Test Plot Described in Section 7.4.5.

8.3.7 Thermocouple Psychrometers

A psychrometer infers the matric potential of the liquid phase of a soil from measurements within the vapor phase that is in equilibrium with the sample. It measures the relative humidity within a soil system as the difference between a dry bulb (non-evaporating) temperature and a wet bulb (evaporating) temperature. The primary difficultly with this technique is that the relative humidity in the soil gas phase changes only a small amount within the typical range of interest. For example, at 25 °C, a water potential of –1.5 MPa (wilting point) corresponds with a relative humidity of about 0.99, and a water potential of –8 MPa (lower limit of extraction for many desert plants) corresponds with a relative humidity of 0.94. Thus, practically all measurements of interest to most cover system studies lie in a narrow relative humidity range between 0.94 and 1.0. Thermocouple psychrometers are typically used to monitor matric potentials in the range of –8MPa to –30 kPa.

The majority of psychrometers used in the field utilize the Spanner design. This design is composed of a thermocouple, a reference electrode, a heat sink, a protective porous ceramic bulb or wire mesh screen, and a recorder. The technique is based on measuring the temperature of a water droplet or wet surface using thermocouple junctions. Calibration curves are developed by immersing the unit in a series of sodium or potassium chloride solutions of known concentration (generally 0.1, 0.3, 0.5, 0.8, and 1.0 molar (Morrison, 1983)) at specified temperatures. The calibration curves are used to compute the in-situ soil-water potential from the measured field output voltage. Problems sometimes encountered with psychrometers are that their calibration can change over time due to corrosion (Daniel et al., 1981) and/or microbial growth on the thermocouple wires (Merrill and Rawlins, 1972).

8.3.8 Heat Dissipation Sensors

Heat dissipation sensors, also called thermal conductivity sensors (Fredlund, 1992) or matric potential sensors, rely on the relationship between the heat dissipation of a ceramic matrix in contact with soil and the matric potential of the soil. These sensors also have a relatively long history of use in agricultural studies. The sensor consists of a heater and a temperature sensor in a ceramic matrix (Figure 8-10). A current is applied to the heater and the temperature of the sensor is measured at certain time intervals, typically at 1 and 20 s after the initiation of heating. The change in temperature (i.e., the heat dissipation) is controlled by the water content of the ceramic matrix because water conducts heat much more readily than air (i.e., thermal conductivity increases with water content). The measured temperature increase represents the heat that is not dissipated. The temperature increase is calibrated to sensor matric potential.

The upper measurement range of the sensor is controlled by the air entry pressure of the sensor matrix material, which is generally about –10kPa. The lower limit is generally considered to be about –1 MPa (Reece, 1996). The sensitivity of the heat dissipation sensors decreases as soils dry below –1 MPa.

Heat dissipation sensors have been used to monitor soil matric potential in cover systems at a number of sites, including the ACAP test sites.

Figure 8-10. Heat Dissipation Sensor.

8.4 Gas Emissions Monitoring

Gas emissions measurements can be used to assess the performance of cover systems and gas control systems. Gas emissions are a common concern for MSW landfills or CERCLA sites that contain MSW. Landfill methane emissions measured at MSW landfill sites and reported in the literature have ranged from about 0.003 to 3,000 $g/m^2/d$ (Bogner and Scott, 1997). In general, the higher rates were associated with landfills that did not have gas recovery and that were covered with dry soils without a GM barrier. For example, at the Olinda MSW Landfill in Southern California, which is covered by a sandy silt soil layer, measured emission rates were greater than 1,000 $g/m^2/d$ prior to installation of a gas collection system. After a gas collection system was installed, measured gas flux rates were less than 10 $g/m^2/d$. The flux rates were still lower (less than 0.01 $g/m^2/d$) in the area of the landfill with a gas recovery system and covered with a clayey silt layer. Given this wide range of emissions, it is appropriate at many MSW sites and CERCLA sites that contain MSW to divide the sites into areas with different surface characteristics, moisture regimes, and gas control strategies and obtain order-of-magnitude estimates of fluxes from these areas for the purposes of assessing emissions. For HW landfills or waste piles that began operation after EPA passed its Land Disposal Restrictions, emissions are generally at much lower rates than recently filled MSW landfills and it may be possible to install a passive venting system.

Landfill gas emission rates can be measured indirectly or directly. Subsurface vertical methane gradients calculated using Fick's First Law (i.e., assuming diffusive transport only) and measured concentrations at gas probes at various depths have been used to estimate gas emissions. This indirect method typically results in higher estimated fluxes than those measured using a direct chamber technique (e.g. a flux chamber) (Rolston, 1986). However, the indirect method is often useful as an independent check on emission values obtained using a flux chamber (Bogner and Scott, 1997). The most common direct methods for monitoring landfill gas emissions are vent sampling and the flux chamber techniques. The most common means of evaluating gas emissions is by using indirect methods (i.e., back-calculating emissions from the source based on a measured concentration). One method of indirect monitoring (described in EPA, 1992) involves concentration profile sampling. The sampling device is placed at the cover system with sampling probes spaced at different intervals. The concentration, wind speed, and temperature are measured at each of the probe heights to generate profiles for each. This technique does not work when quiescent or unstable wind conditions exist, such as shifting of direction. The site must be relatively homogeneous; the technique will not work if emissions or waste composition vary with respect to locations. In all cases, gas sampling must be conducted over a period of time, since gas emission rates are not constants.

The transect technique, performed with a device that used both a vertical and horizontal array of sampling probes placed downwind of the source in the plume centerline, has also been used. Background measurements are also made upwind of the source to correct for the contribution from other sources. The device also has instruments to measure wind speed, wind direction, and temperature. The measured concentrations are spatially integrated and a Gaussian dispersion model is used to back-calculate the emission rate from the source that would be needed to give the measured concentration.

Instantaneous Surface Monitoring (ISM), Integrated Surface Sampling (ISS), and flux chamber techniques (Cooper and Bier, 1997; Lu and Kunz, 1981). Each of these methods is described below. For all of these methods, monitoring is generally not conducted within 72 hours following a precipitation event to allow the cover soil to drain (the trapped water in the soil impedes emissions). Gas emissions can also be measured by vent sampling, which requires the volumetric rate of flow be measured.

With ISM, a portable flame ionization detector (FID) is used to measure the instantaneous concentration of total organic compounds (TOCs) (as methane) along transects or grids established at the landfill surface. This method does not measure flux, but can be used to divide the site into areas with different emission rates. The specific emission rates of these areas can then be evaluated using a flux chamber.

ISS uses a grid-based method to collect samples of the surface gases. Within each grid square, a 8 to 10 L sample of gas is continuously collected from about 50 to 75 mm above the soil cover surface over 25 minutes. Thus, the method provides an average constituent concentration, but not flux, in a grid square. The gas samples can be analyzed in the field or laboratory.

Flux chamber methods have been used at landfill sites since at least the late 1970's (e.g., at the Fresh Kills Landfill, New York (Lu and Kunz, 1981). They involve enclosing a known volume of atmosphere above a known soil surface area and obtaining a direct, though spatially limited, measurement of emission rate. Flux chambers represent a compromise as they may influence flow fields, temperature, and concentrations at the soil/atmosphere interface. However, they have significant advantages if they are operated over short time periods and minimize disturbance. Also, unlike the ISM and ISS methods, the flux chamber method can be used to monitor emissions in high winds; the ISM and ISS methods should generally not be performed when the average wind speed exceeds 16 kph to avoid dilution of the emitted gas by air (Cooper and Bier, 1997). The sensitivity of the flux chamber method can be adjusted by varying the flux chamber volume. They are good for measurement over a 1 to 10 m^2 scale, but are typically less than 1 m^2 with a volume less than 20 L.

8.5 Settlement Monitoring

Post-closure settlement monitoring should consider both total and differential landfill settlements. In general, differential settlements are of most concern because they may induce unacceptable tensile stress and strain in one or more cover system components and they may cause cover system slopes to change or reverse grade. As previously discussed in Section 6.4, cover system settlement can be considered to have one of three sources: (i) settlement of foundation soil; (ii) settlement due to overall waste mass compressibility; and (iii) settlement due to localized mechanisms in the waste. When monitoring cover system settlements, the sources of the settlements are not differentiated; rather, the total settlement at any point due to all of these sources is measured. The measured settlements are then evaluated to assess the effect of the settlements on the cover system components and slopes. For example, most compacted clays exhibit failure at extensional strains of 0.5% or less, as discussed in Section 6.4.5.

Procedures for monitoring total settlements of the cover system surface include:

- aerial surveys, which are generally limited to a vertical accuracy of about 100 to 200 mm with good ground control, and are often more expensive than ground surveys depending on the size of the survey area; the accuracy of aerial surveys may be impacted by a number of things including time of day, angle of sun, and cloud and ground cover; they also require a certain amount of field surveying for ground-truthing and targeting;
- conventional instrumental ground surveys of settlement monuments installed on the cover system, which can achieve high precision (vertical accuracy to within less than 1 mm); and
- global positioning system (GPS) surveys performed using hand operated equipment; the precision and cost of GPS surveys are a function of the specific equipment used; if

significant vegetation is present, GPS may be less reliable since it may be difficult to receive satellite signals.

In addition to total settlements of the cover system surface, settlement of the components within the cover system are sometimes monitored. For example, the settlements of the cover system components for a low-level radioactive waste landfill are being monitored by settlement plates and ground penetrating radar (GPR). The settlement plates were installed above the drainage layer during cover system construction to verify that sufficient drainage layer slope is being maintained. The GPR targets were installed at different locations within the protection layer (Figure 8-11). Both the settlement plates and the GPR targets are periodically surveyed.

Figure 8-11. Placement of GPR Target on Top of Drainage Layer (and at the Bottom of the Protection Layer) During Cover System Construction.

Settlement monuments can be installed on cover system slopes to monitor for downslope creep or instability. This type of monitoring may not be necessary for cover systems designed to conventional factors of safety (as defined in Chapter 6 of this document). However, for situations where lower factors of safety are utilized, slope monitoring is advisable. Slope monitoring should also be considered for final cover systems in seismic impact zones where the cover system is designed to yield (undergo permanent seismic displacement) during the design earthquake event. Slope inclinometers can also be used to monitor for slope movements.

Differential settlement monitoring may be identified through aerial or ground survey techniques if the differential settlement feature is large enough to be captured by the resolution of the survey technique used. Area-wide surface depressions less than 300 to 600 mm in depth are unlikely to be identified through aerial survey. Likewise, highly localized raveling (fines moving into larger voids) or sinkhole features are likely to go undetected in aerial surveys. These same features would be missed in ground surveys where it would be unusual, for example, to install settlement monuments on a survey grid with a grid dimension smaller than about 30 m. The most reliable means for identifying localized differential settlements is to perform periodic visual surveys across the entire landfill surface. This type of survey should ideally be performed immediately after a rainstorm when puddles and ponded water would provide evidence of surface depressions. At the same time, the cover system can be inspected for evidence of other types of differential settlement features such as sinkholes, gullies, or raveling conditions. Also, experience indicates that contrasts develop in surface vegetation in and around depressions, since the cover soil in the depression tends to stay wetter than elsewhere. Thus, contrasts in cover vegetation color and health can be used to identify locations where surface depressions might exist.

As pointed out in EPA (1991), subsidence depressions should be remediated below the level of the hydraulic barrier to avoid long-term acceleration of the subsidence due to a “roof ponding” mechanism. Roof ponding refers to the common structural problem in flat roof systems where ponding water causes the roof rafters to deflect, thus allowing more water to pond, causing more deflection, and so on. This mechanism continues until the roof collapses. In addition, ponding above a portion of the hydraulic barrier increases the potential for percolation through the barrier within the ponded area. Remediation requires removing the cover system in the region of subsidence, backfilling the depression with fill, and then reconstructing the cover system in the repaired area. To minimize the potential for continuing settlement, the use of engineering measures such as geosynthetic reinforcement or separation layers, lightweight fill, vibratory compaction of backfill (to help fill ravel features and voids), etc. should be considered.

Chapter 9
Post-Closure Maintenance and Site End Use

9.1 Introduction

After a cover system has been constructed, it must be monitored and maintained for some timeframe (i.e., the post-closure period). As discussed in Sections 1.2.6 and 8.1, post closure maintenance must be conducted as long as the waste poses a threat to human health and the environment. The post-closure period of 30 years given in RCRA regulations has generally been considered by EPA to be the minimum timeframe for performance monitoring and maintenance for MSW and HW facilities. For CERCLA facilities, the minimum timeframe for cover system maintenance and monitoring is also often assumed to be 30 years, and the EPA is required to evaluate the performance of the cover system at least once every five years to assure that human health and the environment are being protected by the implemented remedy.

Regulatory requirements for post-closure maintenance of MSW landfill cover systems are contained in 40 CFR §258.61 (a)(1):

> "*(a) Following closure of each MSWLF unit, the owner or operator must conduct post-closure care. Post-closure care must be conducted for 30 years, except as provided under paragraph (b) of this section, and consist of at least the following:*
> *(1) Maintaining the integrity and effectiveness of any final cover, including making repairs to the cover as necessary to correct the effects of settlement, subsidence, erosion, or other events, and preventing run-on and run-off from eroding or otherwise damaging the final cover.*"

For MSW landfills, 40 CFR §258.61 (b) provides the following flexibility with respect to the length of the post-closure period:

> "*(b) The length of the post-closure care period may be:*
> *(1) Decreased by the Director of an approved State if the owner or operator demonstrates that the reduced period is sufficient to protect human health and the environment and this demonstration is approved by the Director of an approved State; or*
> *(2) Increased by the Director of an approved State if the Director of an approved State determines that the lengthened period is necessary to protect human health and the environment.*"

Analogous requirements for HW landfills are contained in 40 CFR §264.310 (b)(1) and (5). Regulations for MSW landfills presented in 40 CFR §258.61(c) and regulations for hazardous waste facilities presented in 40 CFR §264.118 require facility owners or operators to prepare a written post-closure plan that includes a description of the post-closure maintenance activities and the frequency of such activities. The purpose of these activities is to ensure the integrity of the cover system and functionality of any monitoring equipment. Maintenance activities include those conducted in response to observations made during periodic inspections and monitoring and scheduled routine activities, such as pump maintenance or replacement. An example of a post-closure inspection, monitoring and maintenance schedule is presented in Table 9-1. An example of a post-closure inspection form, used by the U.S. Army Corps of Engineers, is presented in Table 9-2. This table can be used to document the condition of a landfill cover and identify any required post-closure maintenance activities. In addition to regularly scheduled inspections, a thorough inspection of the cover system should be conducted after major storm events.

The maintenance (and monitoring) activities to be conducted at a closed waste containment facility or remediation site depend on the end use of the site. For example, as discussed in Section 9.3.5, when a mountain bike challenge course was constructed on top of a cover system, routine cover system maintenance included repairing ruts made by the bike tires. It is recommended that personnel conducting the maintenance activities be familiar with the function of the cover system, rather than only familiar with the site end use (e.g., sports facility). If maintenance is not correctly performed, cover system or monitoring system integrity may be impaired.

Table 9-1. Example of waste containment facility or remediation site monitoring and maintenance schedule.

Component	Inspection and Monitoring Frequency[1]	Methods[2]
Cover System Vegetation	Monthly	Visual
Cover System Erosion	Monthly and After Major Storms	Visual
Cover System Intrusion	Monthly	Visual
Cover System Subsidence	Quarterly	Visual
Cover System Slope Stability	Quarterly	Visual
Cover System Drainage Outlets	Quarterly	Visual
Cover System Grades (Survey)	Every 5 Years	Survey/GPS
Gas Extraction System	Monthly	System Check
Surface-Water Management System	Quarterly and After Major Storms	Visual
Leachate Collection and Removal System/ Leak Detection System	Monthly	System Check
Perimeter Security (fence, gate, locks)	Quarterly	Visual
Access Roads	Quarterly	Visual/RT/PC
Groundwater Monitoring System	Quarterly	System Check
Gas Monitoring System	Quarterly	System Check
Survey Monuments	Annually for First 5 Years, at 5 Year Intervals Thereafter	Survey
Post-Earthquake Condition of all Systems/Structures	After Earthquakes	All Above

[1]Frequency of inspection and monitoring may be reduced (or increased) based on observed conditions during the post-closure period.
[2]GPS = global positioning system; RT = rut depth for unpaved roads; and PC = pavement cracking for paved roads.

This chapter discusses cover system maintenance and site end use. Other types of post-closure maintenance activities typically associated with waste containment facilities or remediation sites are not addressed herein. These include maintenance of leachate collection and removal systems, leak detection systems, groundwater monitoring systems, and gas management and monitoring systems. The condition of these systems must be monitored during the post-closure period to assure adequate performance of the site in the long term and to comply with various regulatory requirements.

Table 9-2. Example of post-closure monitoring form used by U.S. Army Corps of Engineers for CERCLA sites.

Site Name:	**Date of Inspection:**
CERCLIS ID:	**Weather:**
State:	**Temperature:**
Corps Construction District:	**Corps Design District:**
EPA Region:	**Site Map: Attach**
Inspection Team: Attach Roster	**Note: Indicate the location of any deficiency noted below on the site map**
ITEM	**REMARKS**
COVER SYSTEM SURFACE	
1. SETTLEMENT (LOW SPOTS) Yes ( ) No ( ) Areal Extent: Depth:	
2. CRACKS Yes ( ) No ( ) Length: Width: Depth:	
3. EROSION Yes ( ) No ( ) Areal Extent: Depth:	
4. HOLES Yes ( ) No ( ) Areal Extent: Depth: Suspected Cause (Rodent or Other):	
5. VEGETATIVE COVER Yes ( ) No ( ) Grass: Yes No Condition: Trees/Shrubs Yes ( ) No ( ) Size:	
6. ARMORED COVER Yes ( ) No ( ) Material Type: Condition:	
7. BULGES Yes ( ) No ( ) Areal Extent: Height: Suspected Cause (gas pressure or other):	
8. WET AREAS Yes ( ) No ( ) Ponding: Yes ( ) No ( ) Areal Extent: Seeps: Yes ( ) No ( ) Areal Extent: Estimated Flow Rate: Soft Subgrade: Yes ( ) No ( ) Areal Extent:	
9. SLOPE INSTABILITY Yes ( ) No ( ) Slides: Yes ( ) No ( ) Areal Extent: Probable Slide Interface: Suspected Cause: Exposed Cover Components:	

Table 9-2. Example of post-closure monitoring form used by U.S. Army Corps of Engineers for CERCLA sites (cont).

BENCHES	
1. FLOW BYPASS BENCHES Yes ( ) No ( ) Description of problem:	
2. BENCH BREACHED Yes ( ) No ( ) Description of problem:	
3. BENCH OVERTOPPED Yes ( ) No ( ) Description of problem:	
LETDOWN CHANNELS	
1. SETTLEMENT Yes ( ) No ( ) Areal Extent: Depth:	
2. MATERIAL DEGRADATION Yes ( ) No ( ) Material Type: Areal Extent: Degree of Degradation:	
3. EROSION Yes ( ) No ( ) Areal Extent: Depth:	
4. UNDERCUTTING Yes ( ) No ( ) Areal Extent: Depth:	
5. OBSTRUCTIONS Yes ( ) No ( ) Type: Areal Extent: Size:	
6. SLOPE INSTABILITY Yes ( ) No ( ) Type: Areal Extent:	
COVER PENETRATIONS	
1. GAS VENTS Yes ( ) No ( ) Active ( ) Passive ( ) Functioning: Yes ( ) No ( ) Condition: Routinely Sampled: Yes ( ) No ( )	
2. GAS MONITORING PROBES Yes ( ) No ( ) Functioning: Yes ( ) No ( ) Condition: Routinely Sampled: Yes ( ) No ( )	
3. MONITORING WELLS Yes ( ) No ( ) Functioning: Yes ( ) No ( ) Condition: Routinely Sampled: Yes ( ) No ( )	
4. LEACHATE EXTRACTION WELLS Yes ( ) No ( ) Functioning: Yes ( ) No ( ) Condition: Routinely Sampled: Yes ( ) No ( )	
5. SETTLEMENT MONUMENTS Yes ( ) No ( ) Located: Yes ( ) No ( ) Condition: Routinely Surveyed: Yes ( ) No ( )	

Table 9-2. Example of post-closure monitoring form used by U.S. Army Corps of Engineers for CERCLA sites (cont).

COVER DRAINAGE LAYER	
1. OUTLET PIPES Yes ( ) No ( ) Functioning: Yes ( ) No ( ) Condition:	
2. OUTLET ROCK Yes ( ) No ( ) Functioning: Yes ( ) No ( ) Condition:	
DETENTION/SEDIMENTATION PONDS	
1. SILTATION Yes ( ) No ( ) Areal Extent: Depth:	
2. EROSION Yes ( ) No ( ) Areal Extent: Depth:	
3. OUTLET WORKS Yes ( ) No ( ) Functioning: Yes ( ) No ( ) Condition:	
4. Embankment Yes ( ) No ( ) Functioning: Yes No Condition:	
RETAINING WALLS	
1. DEFORMATIONS Yes ( ) No ( ) Horizontal Displacement: Vertical Displacement: Rotational Displacement:	
2. DEGRADATION Yes ( ) No ( ) Description of damage:	
VERTICAL BARRIER WALLS	
1. SETTLEMENT Yes ( ) No ( ) Areal Extent: Depth:	
2. PERFORMANCE MONITORING Yes ( ) No ( ) Type of Monitoring: Frequency: Evidence of Breaching: Yes ( ) No ( )	
GROUNDWATER SYSTEMS	
TYPE OF SYSTEM: Containment ( ) Treatment ( ) Functioning: Yes ( ) No ( ) Condition: Routinely Monitored: Yes ( ) No ( )	

Table 9-2. Example of post-closure monitoring form used by U.S. Army Corps of Engineers for CERCLA sites (cont).

PERIMETER DITCHES/OFF-SITE DISCHARGE	
1. SILTATION Yes ( ) No ( ) Areal Extent: Depth:	
2. VEGETATION GROWTH Yes ( ) No ( ) Areal Extent: Type:	
3. EROSION Yes ( ) No ( ) Areal Extent: Depth:	
4. DISCHARGE STRUCTURE Yes ( ) No ( ) Functioning: Yes No Condition:	
FENCING	
FENCING DAMAGE Yes ( ) No ( ) Description of damage:	
PERIMETER ROADS	
ROAD DAMAGE Yes ( ) No ( ) Description of damage:	
SITE ACCESS	
ACCESS RESTRICTIONS Yes ( ) No ( ) Description:	
GENERAL	
1. VANDALISM Yes ( ) No ( ) Description of damage:	
2. CHANGED SITE CONDITION Yes ( ) No ( ) Description:	
3. LAND USE CHANGE Yes ( ) No ( ) Description:	
INTERVIEWS	
1. INTERVIEW ON-SITE WORKERS Yes ( ) No ( ) Problems: Suggestions: Attach report:	
2. INTERVIEW NEIGHBORS Yes ( ) No ( ) Problems: Suggestions: Attach report:	
3. INTERVIEW LOCAL OFFICIALS Yes ( ) No ( ) Problems: Suggestions: Attach report:	

Table 9-2. Example of post-closure monitoring form used by U.S. Army Corps of Engineers for CERCLA sites (cont).

REVIEW DOCUMENTS	
1. GROUNDWATER MONITORING RECORDS Abnormalities: Yes ( ) No ( )	
2. GAS GENERATION RECORDS Abnormalities: Yes ( ) No ( )	
3. SETTLEMENT MONUMENT RECORDS Abnormalities: Yes ( ) No ( )	
4. OPERATION AND MAINTENANCE PLAN Plan in Place? Yes ( ) No ( ) Plan is Being Followed? Yes ( ) No ( ) Plan is Adequate? Yes ( ) No ( ) Optimization is Being Considered? Yes ( ) No ( ) Systems with Optimization Potential? Yes ( ) No( )	

9.2 Cover System Maintenance

9.2.1 Overview

There are a number of routine activities that should be conducted as part of a long-term cover system maintenance program. These activities can generally be divided into the following major categories:

- vegetation-related activities;
- erosion-related activities;
- subsidence-related activities;
- other surface layer performance related activities;
- drainage layer related maintenance;
- surface-water related activities; and
- monitoring system-related activities.

These maintenance categories, which are discussed in more detail below, are not all inclusive for a facility. For example, site access control must also be maintained. In addition, for facilities with gas control systems, there may be certain maintenance activities required under the CAA. Further, there are likely other site-specific categories that need to be considered for waste containment and remediation sites put to beneficial use.

9.2.2 Vegetation-Related Maintenance

Cover system vegetation maintenance may include periodic irrigation and fertilization, as least until vegetation is established, reseeding or replanting areas where vegetation has failed, cutting young trees before they get too large and their roots disturb the cover system components, and mowing. In virtually all cases, some degree of maintenance is necessary until the cover system reaches a state of equilibrium with its inherent environment. Maintenance of cover system vegetation is especially important for alternative cover systems that rely primarily on ET to limit percolation.

As discussed in Section 2.2.3, grasses on cover systems located in humid or temperate climates are usually mowed periodically to discourage the growth of deep-rooted plants, such as trees and certain shrubs. Deep-rooted plants are usually undesirable because their root systems could plug the drainage layer or penetrate and increase the hydraulic conductivity of the hydraulic barrier, if the barrier consists of only a CCL or GCL without an overlying GM. Trees can also create problems if they are blown over, uprooting large masses of soil and leaving a crater in the surface. Many shrub species are shallow-rooted, do not require trimming/cutting, and are sufficiently dense in their ground surface covering so as to prevent larger (deep-rooted) trees and bushes from germinating. Mowing on a regular basis is expensive, thus its avoidance by proper selection of shrub vegetation is an important design consideration.

9.2.3 Erosion-Related Maintenance

Cover system erosion, primarily by water, has been a problem for a number of cover systems, as discussed in Section 2.2.5.1. It is important that significantly eroded areas be repaired in a timely manner after they are observed to prevent progressive erosion and damage to cover system components. Furthermore, it is easier to repair erosion rills prior to their development into larger erosion gullies. As discussed in Section 2.2.5.2, rills can be removed by tilling the soil surface. Gullies, on the other hand, generally cannot be repaired this way. Instead they should be cut out and backfilled with soil that is blended into the adjacent soil.

9.2.4 Subsidence-Related Maintenance

As cover system settlement occurs, the surface grades of the cover system often decrease. If the grades decrease substantially (and more than considered for design), the flow of water within any cover system internal drainage layer and/or the flow of stormwater runoff may be impeded. Regrading of a cover system is difficult not only from soil availability and placement perspectives, but also from complications arising from pipes, piers, and other appurtenances extending through the cover system. For example, a MSW landfill with an active gas extraction system and leachate recirculation system may have numerous wells penetrating its cover system and surface piping extending across the cover system, thereby requiring relatively small construction equipment for maintenance regrading. Production rates with small equipment are low. Obviously, the surface vegetation must be replaced after maintenance grading, and, in the interval before vegetation is established, a temporary erosion control material may be necessary.

The cover system may also exhibit localized differential settlements that cause ponding of water and breaks in cover system piping. The existence of such depressions may lead to localized areas with increased rates of percolation through the cover system. Whenever differential settlement is visually observable, maintenance is necessary. If the cover system drainage layer, hydraulic barrier, or finer-soil-to-coarser-soil interface, in the case of a capillary barrier, has also subsided, the cover system will need to be reconstructed to bring the surface of these layers to grade. For a capillary barrier, this repair must be carefully constructed, as described in Section 3.6.1, to reduce the potential for preferential pathways for infiltrating water. Besides causing localized increases in percolation, cover system depressions also generate tensile strains in the cover system components. As discussed in Section 2.5.2.5, tensile strains can cause barrier materials to fail if the strains are excessive. Depending on the shape of the depression, and the resulting tensile strains, a barrier material may need to be replaced in the depressed area. In other words, bringing the surface of a CCL to grade in a depressed area will not be sufficient if the CCL has failed due to excessive tensile strains. Instead, the barrier would have to be repaired in some manner (e.g., by reconstructing the CCL or by bringing the CCL to grade and placing a GM over the repaired area).

In addition to the above, subsidence-related maintenance may include adjusting the boots around penetrations of the cover system barrier as the cover system settles.

9.2.5 Other Surface Layer Related Maintenance

To minimize percolation through the cover system, the integrity of the surface layer should be maintained. Significant cracks or holes in the surface layer should be repaired, especially for cover systems with ET or capillary barriers. The cracks may be caused by wet-dry cycles or may be an indication of slope instability. Holes may be caused by burrowing animals.

9.2.6 Drainage Layer Related Maintenance

Drainage layer maintenance generally consists of clearing outlets of any obstacles, such as debris, sediment or ice.

9.2.7 Maintenance of Surface-Water Management System

Maintenance of surface-water (i.e., stormwater) management systems is often required after significant storm events. Excess sediment or other obstacles in drainage channels should be removed, and damaged channel linings should be repaired. In areas where erosion has undercut drainage channels (see Figure 7-19), the channels should be reconstructed. It is important that these undercut areas are not just backfilled with soil if they are gully-like. As discussed above in Section 9.2.3, gullies have to be cut out and reconstructed. Otherwise it is easier for the gully to reform along the same flow path.

Drainage downchutes, outlets, energy dissipaters, and other areas where cover system stormwater flows concentrate or substantially change energy state often require regular maintenance and repair. These types of structures deserve careful attention during post-closure monitoring and need to be maintained in good operating condition. Gross et al. (2002) provide several examples of damage to these types of structures resulting from stormwater flows.

9.2.8 Maintenance of Cover Monitoring System

Maintenance of the cover system monitoring system may include period re-calibration of monitoring devices, replacement of batteries in data acquisition systems, and replacement of damaged or non-functioning monitoring system components.

9.3 Site End Use

9.3.1 Overview

Increasingly, beneficial post-closure land uses are being considered in the design of cover systems for waste containment facility closures and remediation sites. As of February 2001, more than 190 cleaned up CERCLA sites have been returned to productive use (EPA, 2001b). EPA's Superfund Redevelopment Initiative reflects the Agency's belief that contaminated sites should be cleaned up in a manner that is protective for reasonably anticipated future land use (EPA, 1999a; EPA, 2001a). EPA does not favor one type of reuse over another, as land use is a local decision. Further, the Agency believes that reuse should help to ensure proper maintenance of the remedy (or cover system for waste containment sites) while providing tangible benefits to key stakeholders, especially the surrounding community. The possible benefits of reuse include (EPA, 1999a):

- *"Positive economic impacts for communities living around the site including new employment opportunities, increased property values, and catalysts for additional redevelopment activities;*
- *Stakeholder acceptance of the municipal landfill presumptive remedy because of potential time and cost savings, and increased involvement in the restoration and redevelopment process;*
- *Enhanced day-to-day attention, potentially resulting in improved maintenance of remedy integrity and institutional controls; and*
- *Improved aesthetic quality of the area through discouragement of illegal waste disposal or trespassing on restricted portions of the site, as well as increased upkeep of the site by future site occupants."*

For CERCLA sites, EPA must balance this preference for future land use with other technical and legal provisions, including ARARs. Only if the remedy is anticipated to achieve cleanup levels that allow the site to be available for the reasonable anticipated future land use, will EPA support that reuse.

The reuse selected for a given site is a function of a number of factors, including the stakeholders, site features, environmental considerations, site ownership, land use considerations and environmental regulations, community input, and public initiatives. These factors are discussed in EPA (2001a). The three major categories of site end use that have been employed at waste containment facilities and remediation sites are: (i) ecological enhancement; (ii) recreational reuse; and (iii) industrial and commercial reuse (EPA, 1999a). Each of these categories is discussed in more detail below, and case histories illustrating these categories are presented. Additional detail is provided in EPA publications (available for download at the EPA website http://www.epa.gov/superfund/programs/recycle/newdocs.htm) on the recreational reuse (EPA, 2001b) and commercial reuse (EPA, 2002) of CERCLA sites. About half of the 190 CERCLA sites mentioned above that had been developed by February 2001 are being used for industrial or commercial purposes (EPA, 2002).

Whatever the type of end use, there are site design issues, such as settlement, gas management, and surface-water management, which are often common to many sites. In addition, some types of sites and end uses may have more issues than others. For example, when developing a former MSW landfill site as a retail shopping complex, there is extra concern about foundation settlement and gas migration to enclosed structures. If the site were developed as wildlife habitat, settlement and gas migration would likely not be as much a concern.

The selected end use can have a significant impact on cover system design. For example, if a site is to be used for a golf course or other facility with a vegetated surface layer that requires irrigation, the cover system may require an internal drainage layer and a barrier that includes a GM to control percolation through the cover system. It is important that the site end use be considered during the design phase of the cover system so that any special features needed to support the post-closure use can be incorporated into the cover system at that time. It can be significantly more expensive to retrofit a constructed cover system to support a specific site end use than to design the cover system to support the specific end use from the start. These end-use designs will have their own monitoring and maintenance requirements. Personnel maintaining the end-use facility should be aware of the maintenance requirements related to the prior disposition of the facility (i.e., waste containment facility or remediation site).

9.3.2 Ecological Reuse

Closed waste containment and remediation sites located in ecologically significant areas have been used as wildlife restoration areas or wetlands. Besides providing a nurturing environmental for plants and wildlife, wetlands filter sediments and contaminants from surface water and can absorb floodwaters, which reduces the flooding potential for lowlands.

9.3.3 Recreational Reuse

Closed MSW landfills are a natural fit for reuse as recreation areas because they typically have a large surface area, and the cover system can generally be contoured to meet the specifications for recreational facilities, such as ball fields or golf courses (EPA, 2001b). Recreational reuse has included trails for hiking, mountain biking, or horseback riding, camping facilities, picnic areas, parks, playgrounds, sledding areas, playgrounds, ball fields, and golf courses. In many cases, a site that will be developed for recreational purposes will support more than one type of recreational activity. For example, a site developed as a general use park may also accommodate sports fields, playgrounds, trails, or other recreational features. In other cases, recreation may be secondary to a primary use, such as a commercial development. Detailed information on the development of recreation facilities over waste containment facilities and remediation sites is presented in EPA (2001b) and is not repeated herein.

9.3.4 Industrial and Commercial Reuse

The beneficial use of closed sites is particularly attractive in areas where developable real estate is limited and expensive. In major urban areas, closed waste containment and remediation sites are increasingly viewed as offering potential for traditional urban developments, such as office parks and retail centers. In such settings, these facilities may not be suitable for ecological or recreational use. Industrial and commercial reuse has included parking lots, restaurants, retail shopping stores or complexes, office buildings, intermodal transportation facilities, port cargo handling facilities, and airports.

One impediment to the design of structures over closed waste containment facilities or remediation sites is that the underlying materials (waste or contaminated materials) may have much different properties than soil. The foundations for these structures should be carefully designed to be protective of the cover system and prevent structural damage. If the waste or contaminated material is anticipated to experience large settlements (e.g., as is typical for MSW), the use of shallow building foundations (e.g., spread footings, reinforced concrete mats, grid foundations with column footings tied together with a system of grade beams and usually an integrated concrete floor) is generally limited to small lightly loaded structures that can tolerate some differential settlements (Dunn, 1995). These shallow foundations are typically located above the cover system barrier layer and contain more reinforcing steel than is required for foundations on conventional sites. Structures on shallow foundations can also be designed to accommodate differential settlements by using tilt-up wall construction, where both the wall sections and the footings are broken up into discrete sections with control and leveling joints between them, by casting the slab in separate sections connected by cable linkages, or by other means (EPA, 2002).

If settlements are anticipated to be too high, site improvement techniques can be considered. Dunn (1995) offers these techniques for reducing the total settlement of structures constructed over MSW landfills:

- *allowing the MSW to reach an acceptable level of decomposition, either by delaying construction or enhancing decomposition …;*
- *supplemental compaction of the MSW, which is usually limited to relatively shallow MSW depths of no more than two or three meters;*
- *surcharging, with settlement monitoring;*
- *dynamic compaction; and*
- *grouting or fly-ash injection.*

If these techniques are unfeasible, deep foundations can be considered.

Heavier structures over waste materials may need to be supported on deep foundations, which are typically piles driven into competent supporting materials below the waste, though drilled piers are also sometimes used. Deep foundations may not be appropriate for sites with a liner system, with wastes that are difficult to drive or drill through, or that have an uncontaminated aquifer that could be impacted by the foundation construction (EPA, 2002). Where deep foundations penetrate the cover system, the penetrations need to be carefully designed to control infiltration and gas emissions. In some cases, structures on pile or pier foundations may settle less than the surrounding area, and gaps may form between the structure and adjacent features (e.g., roads, parking lots, etc.), potentially damaging structure entryways and utilities. Periodic maintenance of these structures may include site regrading, repair of entrances, and adjustment of utilities.

Shallow and deep foundations on waste containment or remediation sites are designed using standard geotechnical methods with consideration of settlement, bearing capacity of shallow foundations, capacity of deep foundations, and downdrag due to waste settlement. In addition to these geotechnical considerations, environmental factors, and especially gas migration, must be considered. Gas migration to enclosed structures is especially a concern with site reuse. Sites that are expected to produce significant amounts of gas may not be good candidates for industrial or commercial uses, unless the gas is well controlled. For this case, there are generally two systems for gas control: (i) a gas management system that is usually incorporated into the containment system; and (ii) a gas protection system for the structure that is usually independent of the gas management system. Gas protection techniques used for industrial and commercial facilities include (EPA, 2002):

- "*Construct floor slabs with convex bottoms to prevent methane from pooling below the structure.*
- *Place an impermeable (gas resistant) geomembrane or other hydraulic/gas barrier under the structure or within the building's floors. This is especially important for sites likely to experience settlement that may disrupt the cover.*
- *Engineer an air space below a structure to allow for gas detection and venting, as well as to facilitate inspection and maintenance of the cover.*
- *Place gas detectors in closed structures to warn of potential gas buildup.*
- *Install vent fans to remove methane buildup from the structure.*
- *Ensure that the design of utilities does not allow for gas migration along utility conduits. One approach is to attach utility service entrances to the outside wall of the structure so they do not penetrate the floor slab, which may create a pathway for gas entry."*

Additional detail on the development of commercial facilities over waste containment facilities and remediation sites is presented in EPA (2002) and is not repeated herein. Most of this information is also applicable to the development of industrial facilities over these sites.

9.3.5 Case Histories

Several published case histories of different site end uses for different types of facilities are presented below. Additional case histories are presented in several EPA publications (1999a, 2001b, 2002), which can be downloaded from the Agency's website at http://www.epa.gov/superfund/programs/recycle/product.htm. The website also include individual case histories of the reuse of some CERCLA sites.

Bowers Landfill

As described by EPA (1999a, 1999b), the 5-ha Bowers Landfill site was located in a former rock quarry within the Scioto River floodplain in central Ohio. Municipal, chemical, and industrial wastes were disposed in the landfill. Until the remedy was constructed the site was flooded an average of 29 days/yr, and contaminants from the site were carried to groundwater and the river. The remedy included removing surface debris and sediments, constructing a cover system that included a CCL barrier and gas collection system over the landfill, and creating 3-ha of wetlands between the landfill and the river. The wetlands not only provide a protective buffer between the landfill and river, but also provide habitat for numerous species of plants, birds, and other wildlife.

Figure 9-1. Constructed Wetlands at Bowers Landfill Site.

Three Landfills in Florida

Mackey (1996) presents case studies of different end uses that were implemented at three closed landfills in Florida. The first site, the Key Largo Landfill Facility, was developed as a nature preserve. This 6.0-ha facility is surrounded on three sides by the Florida Crocodile Refuge, which is maintained by the U.S. Fish and Wildlife Service (USFWS) and provides habitat for several endangered species. The cover system design consists of, from top to bottom:

- 0.15-m thick vegetated topsoil surface layer;
- 0.30-m thick limerock protection layer;
- GC drainage layer;
- 1.0-mm thick textured HDPE/VLDPE/HDPE GM; and
- 0.15-m thick compacted limerock foundation layer.

To enhance the value of the facility as a wildlife preserve, the cover system was vegetated with native plant species and feral cats and certain exotic plants were removed.

The second site, the 13.2-ha Sanlando Landfill Facility, was developed into a softball complex. During the selection of an end-use for the site, it was anticipated that a softball complex would get significant use because it would be located adjacent to a park already used by the community and there had been a large growth in population in the vicinity of the facility. Due to the numerous cover system penetrations that would be required to install poles for fencing and lights, piers for buildings constructed over the landfill, and utility conduits, the design engineers decided to use a CCL hydraulic barrier rather than a GM barrier over the majority of the facility. However, beneath buildings, a 0.5-mm thick PVC GM barrier was installed to reduce the potential for gas migration into the structures.

The third site, the Dyer Boulevard Landfill Facility, occupies 260 ha and includes three large disposal areas, one area containing MSW, one area containing construction and demolition waste (C&DW), and the remaining area containing mixed waste, waste and C&DW. This facility was developed into a multi-faceted sports and recreation complex that includes basketball courts, soccer fields, tennis courts, an observatory mound, picnic areas, canoe rentals, and multi-purpose trails for pedestrians, joggers, bikers, and horses. A specific design feature was incorporated into the cover system over the C&DW area. The end use of this area was a mountain bike challenge course. However, there was concern that the mountain bikes would cause rutting and erosion of the cover soils. To monitor the effect of the activity on the cover system and limit any significant impact, a GT reinforcement layer was placed beneath the mountain bike trails. The purpose of the GT is twofold: (i) to reduce rutting; and (ii) to alert maintenance personal that significant rutting has occurred (and that the cover system must be repaired).

Chisman Creek Superfund Site

As described by EPA (1999a, 1999c, 2001b), the 11-ha Chisman Creek Superfund site is located in York County, Virginia near Chisman Creek, a tributary of Chesapeake Bay. From 1957 to 1974, more than 500,000 tonnes of fly ash from a coal-fired power plant was deposited into abandoned sand and gravel pits on the site. The fly ash was not covered, and eventually resulted in groundwater, surface-water, and soil contamination. The remedy included constructing a cover system over the fly ash and installing a leachate collection and treatment system in the oldest and deepest pit. Because fly ash has low compressibility and doesn't generate gas, fly ash fills can be ideal sites for structures.

The site, plus some adjacent property, was developed into two sports parks, with two lighted softball fields, four soccer fields, restrooms, vending facilities, equipment storage facilities, and a parking area (Figure 9-2). The cover system was engineered to serve as a foundation for the park facilities and graded to accommodate park structures. The cover system design consists of, from top to bottom:

- 0.15-m thick vegetated topsoil surface/protection layer;
- 0.15-m thick sand drainage layer;
- 0.3-m thick CCL; and
- 0.3-m thick soil/ash mixture.

Figure 9-2. Softball Fields at Chisman Creek Superfund Site (from EPA, 1999c).

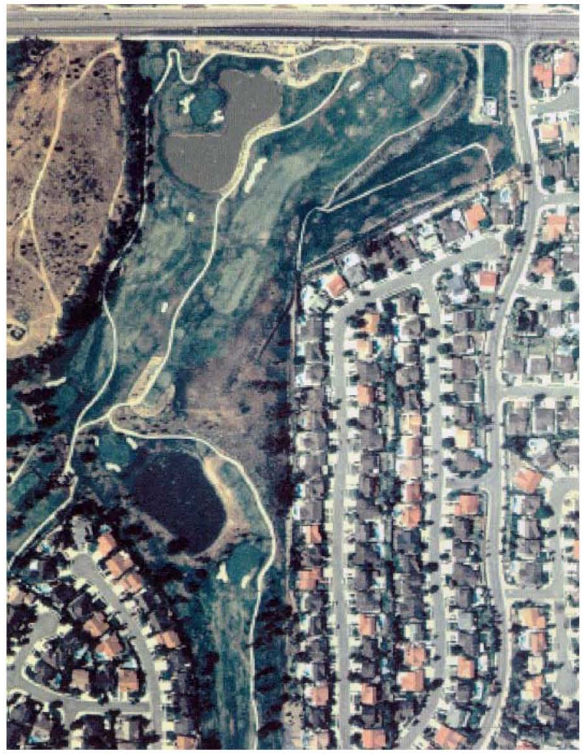
Precautions, such as placing underground utilities in oversized trenches filled with clean fill, were taken to protect future workers from coming into contact with the fly ash.

McColl Superfund Site

As described by Collins et al. (1998), the 8.8-ha McColl Superfund site is located in Fullerton, California and includes 12 unlined pits of sludges and other wastes from production of high-octane aviation fuel (Figure 9-3(a)). In the 1950s and 1960s, three pits were covered with diesel-oil based drilling mud and six pits were covered with soil to control odor and gaseous emissions. The site was placed on the NPL in 1982. The remedy for the site was designed around its end use as part of the Los Coyotes Golf Course and wildlife sanctuary (Figures 9-3(b) and (c)). The remedy includes a multi-component soil and geosynthetic cover system designed to control infiltration and emissions of thiophene compounds, retaining walls to stabilize steep slopes adjacent to the pits, and a soil-bentonite slurry wall. In areas that had been covered with soft drilling muds, a lightweight geocell-reinforced cover system was used. Beneath the golf course, the cover system was geogrid reinforced and included a cobble protection layer to minimize the potential for human intrusion. For both conditions, the cover system included an HDPE GM/GCL barrier underlain by a sand gas collection/foundation layer.

(a)

(b)



(c)

Figure 9-3. McColl Superfund Site: (a) Before Closure; (b,c) After Development as a Golf Course and Wildlife Sanctuary ((c) was downloaded from an EPA website at http://www.epa.gov/superfund/programs/recycle/briefs/ca_brief.htm).

Raymark Superfund Site

As discussed by EPA (1999a, 1999d, 2002), the 14-ha Raymark Superfund site is located in Fairfield County, Connecticut and was operated from 1919 to 1989 as a manufacturing facility for automotive parts and products. Waste generated during the assembly process was disposed in on-site lagoons. As these lagoons reached capacity, they were dredged and the dredged materials were used as fill for construction on over 70 local properties, including school playing fields, recreational parks, and commercial and residential properties. The dredged materials contained lead, asbestos, PCBs, dioxins, and 60 other hazardous substances, and subsequently contaminated soil and groundwater. The remedy for the contaminated properties consisted of relocating contaminated materials back to the Raymark Superfund site or constructing cover systems over them. On the Raymark property, buildings and structures within a 6-ha area were decontaminated and demolished, a groundwater pump-and-treat system was installed, and the on-site and off-site contaminated soils were collected and contained with a cover system. The cover system included GM/CCL hydraulic barrier and underlying sand gas collection layer. Between 0.9 and 3 m of clean fill were placed over the hydraulic barrier to facilitate site development and protect the barrier.

Figure 9-4. Conceptual Drawing of Future Shopping Center at Raymark Superfund Site (EPA, 1999d).

The proposed end-use for the Raymark Superfund site is a 3-ha retail shopping center (Figure 9-4). Prior to construction of the cover system, the site was improved to enhance its geotechnical properties. In-place soils and waste were stabilized using dynamic compaction or surcharging, and peat deposits were dewatered using wick drains. A 0.2-ha platform foundation for the shopping center has been constructed. The platform is supported by 277 30-m long piles that penetrate the cover system.

Denver Radium Superfund Site

As described by EPA (1999e, 2002), operable unit (OU) 9 of the Denver Radium Superfund site is a 7-ha property located in Denver, Colorado that was first used for industrial activities in 1886, with the construction of a smelter. The site was subsequently used for other industrial activities, including cyanide leaching, zinc milling, radium ore processing, minerals recovery, manufacturing and servicing of batteries, oil reclamation, and brick manufacturing. As property ownership, industrial activities, and land use changed, radioactive by-products were often left in place, used as fill or foundation material, or otherwise mishandled. At the time the site remedy was selected, the site soil was contaminated with radium-226, arsenic, zinc, and lead.

The remedy for OU 9 consisted of excavating radioactive materials found at the site and shipping them to an offsite disposal facility and relocating 13,000 m^3 of metals-contaminated soils to four unlined containment cells covered with asphaltic concrete. The primary risks to human health and the environment posed by the soils are related to the ingestion or inhalation of the metals. Since the metals in the soils are only slightly soluble, percolation of water through the soils is not likely to cause the metals to migrate. Thus, the cover systems for the four cells were designed to minimize contact with the waste, rather than to minimize percolation. The remedy was developed concurrently with the design of the site reuse. The site has been developed with a large retail store and parking lot (Figure 9-5). The uncontaminated spaces between the four containment cells were used for utility corridors, and the asphaltic concrete cover systems were incorporated into the parking lot. The store, itself, was constructed on uncontaminated soil.

Figure 9-5. Part of the Denver Radium Superfund Site was Developed with a Retail Store (EPA, 1999e).

Appendix A
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Chapter 1

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Chapter 7

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