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The following are attachments to the prefiled answers of Ian Magruder and Scott M. Payne.

ATTACHMENT 1

DIVISION OF WATER QUALITY
DWQ

Michael F. Easley, Governor

William G. Ross Jr., Secretary
North Carolina Department of Environment and Natural Resources

Alan W. Klimek, P.E. Director
Division of Water Quality

May 31, 2007

MEMORANDUM

To: Aquifer Protection Section Central Office
Aquifer Protection Section Regional Supervisors
Construction Grants and Loans Section
Interested Parties

From: Ted L. Bush, Jr., Chief
Aquifer Protection Section

TB

Subject: Groundwater Modeling Policy

In response to the need for consistent evaluation of land based utilization and disposal sites as well as other subsurface investigations, the Aquifer Protection Section has adopted the subject policy dated May 31, 2007, to be utilized by both consultants preparing applications and Division review staff. The subject policy reflects recent changes in the non-discharge rules with the adoption of Subchapter 02T. This policy provides additional detail to the requirements in Subchapter 02T. In addition this policy will assist with the preparation and review of other subsurface investigations needed for reports submitted for Division review.

All permit applications and other site reports shall be reviewed in accordance with the attached document for any application received on or after August 1, 2007. For any application received prior to that time, staff should review the application for adherence to the policy and discuss with the applicant and/or their consultants to encourage consistency with the policy.

Groundwater Modeling Policy

May 31, 2007

NCDENR Division of Water Quality – Aquifer Protection Section

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Introduction

The phrase "*predictive calculations or modeling*", or variations of this phrase, appears several times in the North Carolina Administrative Code Section 15A NCAC 02L and 15A NCAC 02T regulations. This Division of Water Quality (Division) policy provides guidance regarding this phrase as it pertains to these regulations, and is not intended as a stand alone step-by-step manual for conducting groundwater modeling.

(1) Purpose of policy

The dual purpose of this policy is to: (a) provide guidance to investigators in selecting and using appropriate groundwater models for both permitted sites and incident investigations; and (b) provide guidance for regulators to use in evaluating the adequacy of groundwater modeling results submitted by investigators.

(2) Basis of technical approach

This policy is based on the following guides published by the American Society for Testing and Materials (ASTM, available at www.astm.org):

D 5447-93	Application of a Ground-Water Flow Model to a Site-Specific Problem
D 5609-94	Defining Boundary Conditions in Ground-Water Flow Modeling
D 5610-94	Defining Initial Conditions in Ground-Water Flow Modeling
D 5611-94	Conducting a Sensitivity Analysis for a Ground-Water Flow Model Application
D 5490-93	Comparing Ground-Water Flow Model Simulations to Site-Specific Information
E 978-92	Evaluating Mathematical Models for the Environmental Fate of Chemicals

and also on the following other sources:

"Groundwater Modeling Guidance", by Richard J. Mandle, Groundwater Modeling Program, Michigan Department of Environmental Quality, 2002.

"Groundwater Modeling for Hydrogeologic Characterization", by the California Environmental Protection Agency, July, 1995.

Ground-Water Modeling, by James W. Mercer and Charles R. Faust, National Water Well Association, 1981.

A Review of Field-Scale Physical Solute Transport Processes in Saturated and Unsaturated Porous Media, by L.W. Gelher, A. Mantoglou, C. Welty, and K.R. Rehfeldt, Electric Power Research Institute (EPRI) EA-4190, Palo Alto, CA., 1985.

Contaminant Hydrogeology, by C.W. Fetter, Macmillan Publishing Co., New York, 1993.

Groundwater Transport: Handbook of Mathematical Models, by I. Javandel, C. Doughty, and C.F. Tsang, American Geophysical Union Water Resources Monograph 10, Washington D.C., 1984.

Other references were also used in specific areas of the guidelines. See the complete list of references at the end of this document.

(3) Particular groundwater flow or transport model chosen by investigator

By its nature groundwater modeling must be site specific, and site characterization (as documented by the report documenting the hydrogeologic evaluation) must precede selection of an appropriate groundwater model. The Division requires that any model used on a Division-regulated project: (1) be thoroughly documented in readily accessible published format; (2) be peer-reviewed in the scientific literature [includes appropriate government publications and reviews published by or in cooperation with the International Ground Water Modeling Center]; and (3) be appropriate to the site under investigation, as determined by these guidelines. If there is uncertainty whether the use of a particular model will be accepted, contact the Aquifer Protection Section (Central Office) at (919) 733-3221.

(4) Types of predictive calculations or modeling

There are three types of predictive calculations or modeling described by the Division in these guidelines: (a) groundwater mounding calculations, (b) groundwater contaminant transport calculations where the groundwater standards are to be protected, and (c) groundwater contaminant transport calculations to surface water bodies where the surface water quality standards are to be protected. These three types of predictive calculations or modeling are performed for the following regulatory purposes:

(a) <u>Groundwater mounding calculations</u>

The permitted disposal and utilization systems in Subchapter 02T have a design criteria of maintaining a one-foot minimum vertical separation between the seasonal high water table and the ground surface. The intention of this regulation is to maintain a minimum of one foot of vertical separation between the applied waste (ground surface if applied on the ground) and the altered or mounded seasonal high groundwater table in order to facilitate soil remediation effects of the applied waste. The “altered or mounded” term is meant to signify the additive or compounded effects of the disposal activity onto the ambient seasonal high groundwater table. This mounding effect onto the seasonal high groundwater table is usually obtained via predictive calculations or modeling methods (often called groundwater mounding analysis). In addition, this analysis may be used to determine the effects of groundwater lowering or mounding on surface water bodies (e.g., wetlands, steams, etc.).

(b) <u>Groundwater contaminant transport calculations applied to investigating and/or maintaining groundwater standards</u>

It is the intention of Subchapter 02L and 02T regulations to maintain and protect the groundwater quality of the state. With this goal, the purpose of the predictive calculations or modeling is to document that the activity in question will not result in contravention of groundwater standards at a specified receptor or location, or at the assigned compliance boundary for a permitted facility. In the particular case regarding treatment and disposal of soil containing petroleum products, the purpose of the predictive calculations or modeling is to document that the disposal activity will not result in the contravention of groundwater standards, in addition to other environmental standards (e.g., surface water).

(c) <u>Groundwater contaminant transport calculations applied to evaluating potential impact to surface waters</u>

Because one of the intentions of the Subchapter 02T regulation is to not allow any violations of surface water standards, the Division may require that an evaluation be made to determine the potential impact of the waste disposal activity or release onto the surface waters. Predictive calculations or modeling methods may be required for the following facilities that have or propose a non-discharge disposal activity and there is reasonable concern that surface waters may be adversely impacted by the subject non-discharge waste disposal activity:

(i) any facility treating industrial waste,
(ii) any facility with a design flow of over 25,000 gpd, or
(iii) any facility utilizing a high-rate disposal system.

This evaluation would be conducted using a standard hydrogeologic investigation in combination with predictive calculations or modeling to determine the potential impact to surface waters. The evaluation would be mainly concerned with the potential impact of waste nutrients (nitrogen and phosphorus) onto the surface water body, but also could consider other surface water quality standards as described in 15A NCAC 02B .0200 at the direction of the Division. The evaluation would predict the resultant impact in terms of total pounds/day of contaminant to potentially discharge into the surface water body of concern. The Division may require this “groundwater to surface water” potential nutrient impact to be evaluated whenever surface waters or groundwater lowering ditches or drains are located inside the facility’s compliance boundary, or otherwise reasonably deemed to be “at risk” by the Division.

Section I: Groundwater Modeling Process

The groundwater modeling process involves the following steps:

(1) define study objectives
(2) develop an initial conceptual model and data collection plan
(3) collect required data
(4) refinement of conceptual model
(5) select a computer model
(6) construct a groundwater model
(7) calibrate groundwater flow and transport model
(8) use models for predictive simulations
(9) conduct sensitivity analysis of calibrated models and predictive simulations
(10) perform mass balance calculations
(11) performance monitoring and model refinement

In general, the groundwater modeling process is a direct outgrowth of the hydrogeologic investigation. Most of the data required by the groundwater modeling process should be acquired in the hydrogeologic investigation and documented. The overall purpose of the hydrogeologic investigation is to support a demonstration as to whether or not the groundwater standards can be met. These predictions are accomplished via predictive calculations or modeling.

(1) Define study objectives

In this critical first step, complete and detailed objectives of the modeling effort are specified. These objectives will dictate the level of detail and accuracy required in model simulation. These objectives should:

- Adequately address any regulatory requirements. These requirements will typically be:
  - ensuring that the groundwater standards will be maintained at the facility's compliance boundary or specific property location,
  - ensuring that the contaminant plume will not adversely affect a known or potential receptor,
  - estimating the flow and loading to surface water discharge areas;
  - estimating the zone of influence around an infiltration gallery to ensure a closed loop infiltration and recovery groundwater system (for infiltration galleries), or
  - ensuring that the one-foot water table separation rule can be maintained.

- Identify constituents and processes to be modeled and acceptable model assumptions to be made.

- Provide acceptable tolerances for model calibration.

The study objectives as defined above should be documented in writing. And a description should be provided with regards to how the model/predictive calculations will address the study objectives.

(2) Develop an initial conceptual model and data collection plan

A conceptual model of groundwater flow is an interpretation or working description of the characteristics and dynamics of the physical hydrogeologic system. This is also a critical step in the modeling process, for if the investigator incorrectly conceptualizes the hydrogeologic environment, then groundwater model results likewise will be incorrect and will produce invalid predictions. The purpose of the conceptual model is to document regional and site-specific hydrogeologic data into a set of assumptions and concepts that can be evaluated quantitatively with the numeric or analytic models used for analysis and prediction.. Consult the Division's *Hydrogeologic Investigation and Reporting Policy* for further guidance on developing a conceptual model and on performing a hydrogeologic investigation.

An initial conceptual model should be developed from available regional and local studies and information, and initial site visits before significant site-specific data collection efforts are undertaken. This step is necessary to assure that adequate types and quantities of data are collected to adequately define the conceptual model and to constrain the numerical model or calculation basis. The initial conceptual model generally undergoes refinement or modification as a result of the data collection process, and may be further modified as a result of sensitivity analyses with the quantitative model.

The conceptual model and quantitative models derived from it should adhere to the principle of parsimony. That is, the simplest model that adequately describes the operation of the hydrogeologic system for the expected analysis conditions is always preferred over more complex models.

For the Mountain and Piedmont regions of North Carolina, the conceptual model of the occurrence and movement of groundwater described by LeGrand is a good starting point. For coastal plain applications, the model described by Giese, et al provides an initial model framework.

The conceptual model should include a written description of the following:

(1) Topography and drainage
(2) Hydrostratigraphic units:
  a. Lateral and vertical boundaries
  b. Relationship to other units
  c. Hydraulic and transport properties within each unit
(3) Boundary and Initial Conditions for Flow and Transport
(4) Time Domain to be used for analyses (steady or transient)
(5) Sources and sinks for water to enter or leave the modeled system
  a. Recharge and evapotranspiration
  b. Wells and springs
  c. Connected surface water bodies
  d. Topographic and manmade drains

(3) Collect required data

Data should be collected in accordance with the procedures outlined in the Division's *Hydrogeologic Investigation and Reporting Policy* document.

An important component of the data collection process is the documentation of data variability, uncertainty, and deficiencies, and a compilation of the uncertainties recorded for each of the other components.

Groundwater models should not be used as a substitute for site-specific measurements of field data. Rather, the site-specific measurements should be used to constrain the modeling by providing data for model calibration, measurements of hydrostratigraphic unit geometries and properties, as well as sources and sinks to be modeled.

(4) Refinement of conceptual model

Analysis of collected field data may support the initially developed conceptual model. Or, the analysis can result in a refinement of the initially developed conceptual model.

At this stage the investigator should address the adequacy of the data collection effort. The collection of additional appropriate site data may be required in order to further refine/confirm the conceptual model.

(5) Select a computer model

A computer model is a set of one or more mathematical algorithms that simulate the characteristics of a physical hydrogeologic system. The computer model selected should be appropriate for the conceptual model developed. Modeling objectives should provide guidance on the complexity of model required. In general, the simplest model should be used that adequately matches the conceptual model. If the problem can be conceptualized in two dimensions, then a three-dimensional model is unnecessary. When selecting an appropriate groundwater model for a particular application, it is important to consider the amount and quality of data available. Do not use a complex, multi-dimensional groundwater model if there is not sufficient on-site data in addition to adequate knowledge of outer hydraulic boundaries, sources and sinks. However, there may be situations in which a fully developed three-dimensional numerical model is required, such as multi-aquifer groundwater flow/transport problems or multi-layer models incorporating multiple soil horizons (where such complex models result from complex conceptual models, and necessitate greater detailed hydrogeologic data collection).

When selecting a groundwater computer model, the user should consider the track record of the model. The Division's Aquifer Protection Section requires that any model used on a Division-regulated project: (1) be thoroughly documented in readily accessible published format; (2) be peer-reviewed in the scientific literature [includes appropriate government publications and reviews published by or in cooperation with the International Ground Water Modeling Center]; and (3) be appropriate to the site under investigation, as determined by this policy.

(6) Construct a groundwater model

Model construction is the process of transforming the conceptual model into mathematical form. For numerical models, this process usually involves translating the conceptual model into a discretized flow domain, identifying discrete periods of time for analysis or annually-averaged conditions, and compiling input parameters for the groundwater computer model, including initial and boundary conditions and hydraulic properties. For semi-analytical models, the process is similar, except no spatial discretizing is required. For analytical models, again no spatial discretizing is required, but care must be taken to ensure that the pre-set boundary conditions for a particular analytical solution adequately match the site in question, and that the assumed groundwater flow field is adequate for the site.

(a) <u>Flow sources and sinks</u>

Sources and sinks influence groundwater flow patterns, and their effects should be documented for inclusion in the selected model. Common sources and sinks that should be identified include: pumping or injection wells, precipitation and evapotranspiration (or net groundwater recharge described below), drains, leakage across confining layers, and flow to or from surface water bodies. Descriptions of sources and sinks should include rates and temporal (seasonal and otherwise) variability. Development of a water budget is usually helpful to quantify the contributions of sources and sinks.

Net groundwater recharge (or simply recharge) refers to the portion of precipitation that infiltrates the soil and enters into the surficial groundwater aquifer, and is a key parameter in all groundwater flow models. Recharge can be quantified in two general ways: by either performing stream hydrograph baseflow separation on a regional (basinwide or sub-basinwide) scale, or evaluating detailed site-specific soil infiltration/evapotranspiration and surface runoff estimates and performing a site-specific water balance to estimate recharge. Obtaining accurate estimates of recharge is difficult without extensive regional and/or site-specific evaluation, and usually published recharge estimates are used. With any groundwater flow model, there is always a direct correlation between net groundwater recharge and the aquifer bulk transmissivity, which is usually evident during the model calibration process (see Section I (7) Calibrate groundwater flow and transport model below) and easily seen by running sensitivity analyses. Therefore, selecting the appropriate net recharge is usually balanced with selecting the appropriate aquifer bulk transmissivity.

At times it can be advantageous to model total precipitation (P) into the groundwater flow model, and model evapotranspiration (Et) out of the model, with the net groundwater recharge (R) being estimated as $R = P - Et$. A benefit of this methodology is that seasonal changes in recharge (R) can be easily modeled using long-term averaged precipitation (P) and standard evapotranspiration (Et) models. When modeling recharge via the P – Et methodology, special care should be taken to check the model water balance output to ensure that the model calculated recharge ($R = P - Et$) is within an acceptable or reasonable range.

(b) Boundary conditions and extent of model

The physical size or extent of the model (length, width and depth of model) often has a large bearing on the flow sources and sinks that need to be included into the model in addition to the types of boundary conditions included in the model. In general, the groundwater flow and transport model should have as many physical boundaries (such as rivers, lakes, ocean) as possible in order to adequately simulate the regional groundwater flow conditions at the particular site of interest. These types of physical boundaries can generally be considered specified head, specified flux, or head-dependent flux boundaries. Other good physical boundaries to model would be ridge lines or hilltops, which can usually be considered no-flow boundaries for the surficial aquifer. However, these types of features may not be no-flow boundaries for deeper confined aquifers.

In general, model boundaries should be located far enough away to minimize their direct influence on the study area. To accomplish this and help lead the investigator toward using real surrounding physical boundaries in the model, the physical size (i.e., each horizontal dimension) of the model should be at least four (4) times larger than the largest dimension of the facility’s land application system or other source of contamination. This general rule can also be applied to the area impacted by small point source contamination sources such as might be encountered in incident investigations. For example, if a proposed spray irrigation facility has a spray field that is 1000 feet by 500 feet, then in general the model should be at least 4000 feet in length and breadth, or may need to be larger if appropriate in order bring in a physical boundary, such as a neighboring river, into the model. Exceptions to this rule may be if a constant head boundary (river, stream, etc.) or other boundaries are close to the site being modeled. When in doubt as to how large to make the extent of the model, it should be made larger in order to take into account neighboring physical boundaries.

Caution should be exercised when modeling groundwater-lowering ditches, which should not be modeled as constant head boundaries. In general these features should be modeled as a head-dependent drain boundary, where the drain elevation is the elevation of the lowest topographical elevation in the ditch or drain pipe. In many situations, these drain elevations provide important controls on the configuration of the water table and the depth to water beneath land application units. Additionally, groundwater-lowering ditches may necessitate an investigation into the potential impact to surface waters.

(c) Regional groundwater gradient

It is important that the predictive groundwater model accurately reflect the regional groundwater gradient as measured in the field. Failure to do so will generally result in incorrect groundwater mounding calculations and incorrect groundwater contaminant transport calculations. It is important to realize that if the model boundary conditions are correctly established, and if the sources and sinks (which include groundwater recharge and leakage across confining layers) are modeled correctly, then the model-predicted regional groundwater gradients will reflect field-measured groundwater gradients. If the model-predicted groundwater gradients are too high or too low or in the wrong direction, then this generally indicates that the model boundary conditions are incorrect and/or the model sources and sinks are incorrect.

It is often the case that groundwater gradients vary seasonally, varying in magnitude and direction. This again is generally a result of seasonal changes in physical boundary conditions, such as changing river water level; and seasonal changes in sources and sinks, such as changing groundwater recharge and evapotranspiration rates from winter to summer. Whether or not the groundwater model needs to take into account these seasonal groundwater gradient changes depends on the problem being solved and the time-scale of the problem. For groundwater mounding problems, the worst-case scenario will be in the winter and early spring when the seasonal groundwater table is at its maximum elevation. For groundwater contaminant transport problems, generally the time-scale of interest is measured in years because of generally slow-moving groundwater. For time scales measured in years or decades of years, seasonal fluctuations of groundwater gradient tend to average out, and modeling yearly averaged gradients is appropriate.

(d) Hydraulic properties

Hydraulic properties include the transmissive and storage characteristics of the aquifer system, such as transmissivity, hydraulic conductivity, storativity, and specific yield. They also include the leakage coefficients of stream, lake and riverbeds. Field and laboratory measurements of these properties should be documented, compared to accepted ranges for the medium under investigation, and uncertainty associated with the property measurements estimated. An assessment of heterogeneity and anisotropy over the aquifer domain for each property should be made, particularly in the Piedmont and Mountain regions of the State. See the Division document entitled *Performance and Analysis of Aquifer Slug Tests and Pumping Tests Policy* for details related to conducting aquifer tests.

(e) Parameters used in transport models

Groundwater transport models require certain additional hydraulic and chemical parameters, these being effective porosity, longitudinal and transverse dispersion coefficients (or dispersivity), chemical retardation factor, and chemical biodegradation decay rate. See Appendix A for details on how to estimate these parameters.

(f) Groundwater to surface water models

If the conceptual model involves a groundwater discharge to a surface water body, then consideration needs to be given to how the surface water body will be modeled. Generally, the surface water body can either be modeled as a constant head boundary where the yearly-average surface water elevation is used as the constant head, or as a head-dependent boundary where the surface water elevation is allowed to vary dependent on groundwater baseflow and upstream conditions. The evaluation should predict the resultant contaminant impact in terms of total pounds/day of contaminant to potentially discharge into the surface water body of concern, once the groundwater contaminant plume has reached steady-state conditions. See Section I (7) Calibrate groundwater flow and transport model (below) for more details on transport analyses.

(g) Contaminant source concentration used in transport models

Groundwater transport models require a source concentration or mass flux to be designated for the source of the contaminant plume to be modeled. In some situations, uncertainty in the timing, magnitude, and mass of chemical sources may contribute the largest uncertainty in predictions using transport models. In the case of a groundwater remediation system being modeled, the measured groundwater contaminant concentration in the source area actually measured in the field may be used. These sources should be considered constant and continual unless it can be documented that virtually all of the source mass has been removed.

In the case of a land application system (spray irrigation of treated wastewater, for example), the correct source groundwater contaminant concentration to be used in a transport model may be difficult to determine because of the uncertainty of chemical removal/uptake in the cover crop and shallow soil horizons. There are two cases to consider:

- If the cover crop and shallow soils are deemed to have no removal capacity for a particular contaminant chemical of concern, then the treated wastewater effluent chemical concentration should be used as the source concentration of contaminant flux into the groundwater system.
- If the cover crop and shallow soils are deemed to have a certain removal capacity for the particular contaminant, then the Division will allow 50% removal of the Realistic Yield Expectation (R.Y.E., as documented by NRCS, NCSU, etc.) to be used in calculating the resultant contaminant

concentration assumed to leach into the groundwater system (see Appendix A for a detailed discussion).

(h) Data deficiencies and uncertainty

A final component of the conceptual model is the documentation of data deficiencies, a compilation of the uncertainties recorded for each of the other components, and an acknowledgment of any alternative conceptual models that could be developed from the available data. This last component of the conceptual model is an important step, for it forces the investigator to quantitatively address the adequacy of the data collection effort. If high uncertainty is associated with the conceptual model, then an elaborate and costly modeling effort may not be justified.

(7) Calibrate groundwater flow and transport model

A reliable groundwater flow model must be able to simulate the observed movement of groundwater and/or concentrations of contaminants. Typically, a groundwater flow model is calibrated by comparing model output, such as a water level or head and discharge to surface water, with actual measured values. When groundwater flow calibration is involved, the modeling results should include (1) an evaluation of the calibration process, and (2) the resultant calibrated groundwater/potentiometric surface(s) with posted head residuals at individual observation wells. Residual statistics should be evaluated and reported.

Model calibrations are normally conducted with the flow model in "steady-state" mode, where all the model parameters are fixed and do not vary with time. Typically, annual averaged groundwater levels are used or approximated. However, in certain situations it may be necessary for the investigator to also calibrate a transient (or dynamic) flow model. In this situation, the model output for various time steps is compared to the observed values, such as water levels that vary monthly, seasonally, or during the course of a pumping test.

In addition to model calibration using individual observation wells, the gradient (magnitude and direction) of the model-predicted groundwater table/potentiometric surfaces(s) should reflect the field-measured gradient across the modeled site. It is possible, for example, that part of the modeled potentiometric surface appears accurate, but another part of the potentiometric surface is obviously wrong, either in magnitude or direction. Using such a model to predict contaminant transport may lead to serious errors.

It is important to realize that even though good groundwater flow calibration may be achieved, this does not imply that the model is "correct" in its representation of the actual hydrogeologic processes of the modeled site. Often times the groundwater flow model calibration process leads to the investigator realizing more site/subsurface information is necessary to improve either (a) the overall model calibration, and/or (b) the overall model water budget. Once the investigator is satisfied with a particular model calibration, the overall model water budget should be checked to ensure that a reasonable groundwater recharge value is being used for the particular site being modeled. For example, a groundwater flow model being used for a particular site in the interior coastal plain may appear to calibrate well, but if the resultant water budget shows that the net groundwater recharge is 30 inches/year when a net groundwater recharge of about 10 – 12 inches/year is more generally accepted, then the overall model should be re-evaluated to determine the source of error. In this particular case, it may likely be determined that the overall aquifer transmissivity was set too high, which led the investigator to adjust the net groundwater recharge too high in order to maintain adequate calibration. It may also imply that the overall hydrogeologic framework is incorrect or not complete.

Numerical groundwater contaminant transport models require that the groundwater flow field first be evaluated. Therefore, a numerical transport model calibration is really a two-step process. In Step #1 the groundwater flow model is calibrated, and then the flow field calculated by the flow model is used in the contaminant transport model. Step #2 involves calibration of the groundwater transport model to historic data on contaminant concentrations and degradation rates. Groundwater transport model calibration will require a minimum of two discreet sampling events from an appropriate time interval from the site. However, calibrating a groundwater transport model using to too few sampling events, or between time intervals that are relatively short, can lead to serious errors in predictive calculations.

(8) Use models for predictive simulations

The main purpose of a modeling effort is to generate a representative groundwater flow and/or transport model that will make accurate predictions based on an altered environment. Predictive simulations may either be run when using a model in "steady-state" mode or in "transient" mode. In steady-state mode, all the model parameters are fixed and do not vary with time, whereas in transient mode certain parameters such as rainfall, evapotranspiration, pumping rates, etc., are varied seasonally (typically) to generate a seasonal groundwater table variation. Predictive simulations will generally be of two forms: (a) groundwater flow and mounding simulations, and/or (b) groundwater transport simulations.

(a) <u>Groundwater flow and mounding predictive simulations</u>

Typically, predictive groundwater flow models are run in steady-state mode, when dynamic equilibrium is achieved. Transient groundwater flow models are run when multiple time periods are simulated. If the flow model is being run to predict a groundwater mound height generated by some type of land application system (such as an infiltration gallery or spray irrigation system), then the model is typically run for 200 - 360 days, or whenever the groundwater mound height appears to stabilize (or 720 days, whichever comes first). However, the simulation period should be a year or less, as the seasonal groundwater table/mound fluctuation is typically cyclical.

(b) <u>Groundwater transport simulations</u>

Groundwater transport models are typically run with the flow model in steady-state mode using average annual conditions. Because the time span of groundwater contaminant travel is usually measured in years, over the span of multiple years the seasonal groundwater flow variations are generally averaged out, and thus performing transport models with a transient groundwater flow model is generally not required.

A transport model should be run until the contaminant plume has reached steady-state (or near steady-state) conditions. Assuming the source of the contaminant flux remains constant (or near constant), at some point in time the shape of the plume will reach a maximum size and the shape of the plume will remain relatively fixed for future times. For larger discharging land application systems, steady-state conditions may not be reached for decades, especially if deeper semi-confined aquifers are involved in the groundwater flow and transport process.

(9) Conduct sensitivity analysis of calibrated models and predictive simulations

Sensitivity analysis involves varying values for a property, such as hydraulic conductivity, and observing the effect on model calculated head or concentration values. Usually there is a certain amount of uncertainty with regard to the actual aquifer hydraulic conductivity or transmissivity values to be used in the model. Thus, sensitivity analysis is particularly helpful in quantifying the uncertainty associated with model-predicted future or altered site conditions. By varying the hydraulic conductivity or transmissivity, or other potentially sensitive parameters, over the range of potentially expected values, the range of resultant groundwater elevations or concentrations will be generated. The investigator can then determine expected head or concentration results with a range of uncertainty associated with it.

The sensitivity analysis should identify a range of values for each sensitive input parameter. Data collected from on-site testing will help constrain the range of values for sensitive parameters. On-site data should be used whenever possible in the model domain. For poorly constrained parameters, use the most conservative input value(s) that can reasonably be expected to occur for the particular model application. For example, if the groundwater model is used to predict mounding conditions in response to irrigation, and the sensitivity analysis indicates that the model is sensitive to changes in transmissivity values, the transmissivity value(s) used in the model must be those which, within a reasonable range for the given hydrogeologic conditions, would result in the highest mounding of water levels. If a groundwater contaminant transport model is being used to predict the maximum distance that a contaminant may be expected to travel, sensitive input values used must be those which, within a reasonable range for given hydrogeologic conditions and chemical properties, would result in the furthest distance traveled for the modeled constituent.

The results of the sensitivity analyses should include a table showing the sensitive parameters and their ranges, and figures showing the resulting variations in modeled parameters using the two value endpoints (highest and lowest value) for each sensitive parameter. Additional field characterization may be required to obtain data for model input parameters that are determined to be relatively sensitive.

(10) Perform mass balance calculations

Water and mass balance model outputs (or calculations) should be shown describing all flow and transport fluxes. All source and sink terms should be shown and the net results should balance within reasonable margins for error (less than 0.5%).

For model calibration, if net groundwater recharge (R) is being modeled as total precipitation (P) into the model minus evapotranspiration (Et) out of the model ($R = P - Et$), then the water balance output should show the total P into the model, and the total Et out of the model, and an evaluation of the net groundwater recharge (R) into the model should be made. For groundwater mounding model simulations, the water balance output should show the total additional flux of water added into the model above the net groundwater recharge (R).

For groundwater transport models where a particular contaminant is reaching a receptor such as a stream or a well, the mass balance should show the total mass of contaminant (as a function of time) reaching the receptor, the total mass removed from the model domain at the receptor, and the total contaminant mass introduced into the model domain at the source.

(11) Performance Monitoring and Model Refinement

Groundwater models can be useful tools to simulate hydrogeologic conditions and contaminant concentrations over time. Models are most useful when used as working "tools" that are refined and improved when more information on site hydrogeologic conditions becomes available. As more site data becomes available, the groundwater model should be checked against this data and the model may need to be refined in order to more accurately predict future conditions. Additional wells or monitoring points may be required during the performance-monitoring period if the performance monitoring data indicates an inadequate monitoring network.

When required, a Performance Monitoring Report should be submitted to the Division's Aquifer Protection Section on an annual basis, or at a time interval agreed upon by the Section and the Responsible Party, which will contain the predicted model outputs compared with data obtained during the performance monitoring period. If there is a significant discrepancy between the predicted model output and the performance monitoring data, the groundwater model should be refined in order to more closely match actual field conditions.

Section II: Reporting Modeling Results

Results from groundwater modeling efforts must be adequately documented. Such documentation must provide regulators sufficient information to determine the adequacy of the model and supporting data, and validity of the modeling results. The major reporting elements shown below must be included in the model report submission. This format is a modified format taken from ASTM D 5447. The value of this format is that it standardizes criteria that should be considered in any modeling effort. The detail provided within the format should reflect the investment that has gone into the modeling effort.

The Division may request that groundwater computer model data inputs and outputs be provided in electronic form in order to allow staff to evaluate the model using the actual model.

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7.1 Model Assumptions/Limitations

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Appendix A

Dispersion, Chemical Retardation Factor, and Chemical Biodegradation Decay Rate Parameters used in Transport Models

Groundwater transport models typically require certain additional hydraulic properties and chemical properties, these being (a) longitudinal and transverse dispersion coefficients (or dispersivity), (b) chemical retardation factor, and (c) chemical biodegradation decay rate.

(a) Dispersion measures the natural spreading of a contaminant plume during migration. Site specific dispersion parameter values are difficult to measure without extensive field investigations, but fortunately researchers have developed methods of estimating them using simple formulas. The dispersion coefficient depends not only on the variability of the local hydraulic conductivity at the site, but also on the scale of the problem (i.e., the distance from the original plume site to the groundwater receptor or endpoint of travel). See Fetter (1993) or Gelhar (1985) for more discussions regarding these issues.

Longitudinal dispersion is a measure of the contaminant plume spreading in the direction (parallel) of groundwater flow. Transverse dispersion is a measure of the contaminant plume spreading perpendicular to the direction of groundwater flow. The longitudinal dispersion coefficient can be estimated as follows:

$$\mathbf{D_L = 0.1\ L\ v}$$

where: $\mathbf{D_L}$ = longitudinal dispersion coefficient in feet^2/day
$\mathbf{L}$ = distance in feet from the original plume site to the groundwater receptor of interest or endpoint of travel
$\mathbf{v}$ = average groundwater velocity in feet/day

The transverse dispersion coefficient ($\mathbf{D_T}$) can be estimated from the relation $\mathbf{D_L/D_T}$ = 6 to 20, depending on site conditions (Fetter, 1993), but a ratio of $\mathbf{D_L/D_T}$ of 10 is probably good for typical cases.

Often groundwater transport models will use the longitudinal and transverse dispersivity parameter instead of the longitudinal and transverse dispersion coefficient. The relationship between the two parameters is as follows:

$$\mathbf{D_L = \alpha_L v + D^*}$$
$$\mathbf{D_T = \alpha_T v + D^*}$$

where $\boldsymbol{\alpha_L}$ is longitudinal dispersivity in feet, $\boldsymbol{\alpha_T}$ is transverse dispersivity in feet, $\mathbf{v}$ is average groundwater velocity in feet/day, and $\mathbf{D^*}$ is effective diffusion coefficient, which is related to the diffusion due to concentration gradients. For typical groundwater transport problems, $\boldsymbol{\alpha_L}$ **v** and $\boldsymbol{\alpha_T}$ **v** are numerically much larger than $\mathbf{D^*}$, and thus $\mathbf{D^*}$ can often be ignored (See Fetter, 1993. However, in some transport problems **v** can be numerically very low, such as leakage through a liner problem, and $\mathbf{D^*}$ term will dominate, implying that the main dispersion mechanism is diffusion via concentration gradients.) In this case:

$$\mathbf{D_L} / \mathbf{D_T} = \boldsymbol{\alpha_L} / \boldsymbol{\alpha_T} = 10 \quad \text{(typically)}$$

(b) Chemical retardation factor (unitless number) is the measure of the relative migration velocity of the chemical (contamination) compared to water. For inorganic constituents (such as cations, anions, including NO_3, Cl) and fecal coliform, the retardation factor is normally set to 1. For organic chemicals, the retardation factor (**R**) should be based on the following formula:

$$\mathbf{R = 1 + \rho K_{oc} f_{oc} / n}$$

where:
- $\boldsymbol{\rho}$ = aquifer bulk density in g/cm^3, default = 1.8 g/cm^3
- $\mathbf{K_{oc}}$ = organic carbon-water partition coefficient in L/kg
- $\mathbf{f_{oc}}$ = aquifer organic carbon fraction (unitless), default = 0.001
- $\mathbf{n}$ = aquifer effective porosity (unitless)

The organic carbon-water partition coefficient ($\mathbf{K_{oc}}$) is readily available for most organic chemicals from various sources. Values other than the default values for aquifer bulk density ($\boldsymbol{\rho}$) and organic carbon fraction ($\mathbf{f_{oc}}$) should be documented.

(c) The chemical biodegradation decay rate measures the rate at which a contaminant is attenuated due to biological activity in the subsurface. Setting the decay rate to zero implies no biodegradation. Many groundwater transport models that allow biodegradation assume a first-order decay rate. Typically, these models will either require the decay rate in units of either 1/days or 1/years. Some models, however, may require the decay rate to be entered in terms of a half-life (or lifetime). The relationship between half-life ($\boldsymbol{\tau}$) and first-order decay ($\mathbf{k}$) rate is:

$$\boldsymbol{\tau} = \mathbf{0.693 / k}$$

where if $\boldsymbol{\tau}$ is in days, then $\mathbf{k}$ is in 1/days, or if $\boldsymbol{\tau}$ is in years, then $\mathbf{k}$ is in 1/years, etc. If a non-zero biodegradation rate is used in a transport model, evidence needs to be presented to justify its use.

With regards to Nitrate (NO_3) transport and decay in groundwater, there is evidence that Nitrate may decay (NO_3 denitrifying in a riparian buffer zone, for example) via a zero-order decay rate (see Nelson et. al., 1995), or via Michaelis-Menton kinetics that leads to a first-order decay for smaller concentrations and a shifting to a zero-order decay for larger concentrations (see Maag et. al, 1997). In this special case of NO_3 denitrifying (and other contaminants that may be similar), special care should be taken when modeling NO_3 removal with solely a first-order decay model, as over-prediction of the NO_3 removal rate could potentially occur.

Appendix B

Example Calculation of Nitrate Source Concentration to be used in a Groundwater Transport Model for Land Application Systems Utilizing a Cover Crop

Groundwater transport models require a source concentration or mass flux to be designated for the source of the contaminant plume to be modeled.

If the cover crop and shallow soils are deemed to have a certain removal capacity for the particular contaminant, then the Division will allow 50% removal of the Realistic Yield Expectation (R.Y.E., as documented by NRCS, NCSU (see http://www.soil.ncsu.edu/nmp/ncnmwg/yields/), site specific yield records, etc.) to be used in calculating the resultant contaminant concentration assumed to leach into the groundwater system (see North Carolina Cooperative Extension Service, 1990).

> *Note: This 50% R.Y.E. limit with regards to the cover crop is only for the purposes of calculating a potential "conservative" resultant contaminant concentration assumed to leach into the groundwater system, and do not imply that the cover crop will not remove the full R.Y.E. However, studies have shown that certain chemicals of interest (nitrogen, for example) typically do not accumulate in the soil, and are readily leached downward through the cover crop root zone into the surficial groundwater aquifer, especially when the crop is not in its growing season. Therefore, this 50% rule is meant to be conservative in order to guard against potential contaminant impact to groundwaters and surface waters of the State.*

This calculation should be done according to the following example.

A certain municipal wastewater treatment plant uses spray irrigation to land apply its treated wastewater. The WWTP has a design flow of 50,000 GPD (0.05 MGD), and the investigator is concerned about meeting the NO_3 (nitrate) groundwater standard of 10 mg/l N at the compliance boundary. The WWTP sprays onto a 15 acre dedicated field where the cover crop is fescue grass on Goldsboro soils. The WWTP achieves the following average effluent limits with regard to nitrogen species:

[Ammonia-N]	=	8 mg/l
[NO_3-N + NO_2-N]	=	10 mg/l
[TKN]	=	15 mg/l

Total nitrogen in the wastewater effluent is thus [TKN] + [NO_3-N + NO_2-N] = 25 mg/l (ppm) N. The total pounds/year of N applied to the spray fields is:

Total pounds N/year applied = (25 ppm N) x (0.05 MGD) x (8.34 lbs/gallon) x (365 days/year)

= 3,805 lbs N/year

Calculate the Cover crop R.Y.E. Uptake:

According to the NC State University Realistic Yield Expectations (R.Y.E.) for Soils in North Carolina (see NCSU, 2000), fescue planted on Goldsboro soil series will yield 4.0 dry tons of hay/acre/year.

According to the NRCS Conservation Practice , Standard Nutrient Management Code 590 document (see NRCS, 1998), Nitrogen Fertilization Rate for fescue is 40 - 50 lbs N/ton hay (use 50 lbs N/ton hay).

Combining the above two figures, 15 acres of fescue will consume:

R.Y.E. = (4.0 dry tons/acre/year) x (50 lbs N/ton hay) x (15 acres)
= (200 lbs N/acre/year) x (15 acres)
= 3,000 lbs N/year

The Division will allow 50% uptake of the R.Y.E. for the purposes of calculating contaminant concentrations leaching to the underlying groundwater system:

50% of R.Y.E = 0.5 x (3,000 lbs N/year) = 1,500 lbs N/year

Resultant pounds N/year assumed to leach into groundwater system = 3,805 lbs N/year - 50% of R.Y.E.
= 3,805 lbs N/year - 1,500 lbs N/year
= 2,305 lbs N/year

Resultant chemical conc. of flux leaching into the groundwater system = 2,305 lbs/year / (0.05 MGD x 8.34 x 365)
= 15.1 mg/l N

which is assumed to all convert (oxidize) to NO_3-N by the time the contaminant is in the groundwater system.

Thus, for the purposes of building the groundwater flow and transport model, the investigator would apply 50,000 GPD onto the 15 acres at a concentration of 15.1 mg/l NO_3-N and assume that all the contaminant flux recharges into the groundwater system.

From the above analysis, it is clear that if the total nutrients in the effluent is less than or equal to 50% of R.Y.E., then all the effluent nutrient is assumed to be taken up by the cover crop, and there is no need to perform any groundwater contaminant transport analysis for the nutrients involved.

ATTACHMENT 2

Appendix A – Model Review Checklist

Review Questions	*Yes/No*	*Comment*
1. Planning		
1.1 Are the project objectives stated?		
1.2 Are the model objectives stated?		
1.3 Is it clear how the model will contribute to meeting the project objectives?		
1.4 Is a groundwater model the best option to address the project and model objectives?		
1.5 Is the target model confidence level classification stated and justified?		
1.6 Are the planned limitations and exclusions of the model stated?		
2. Conceptualisation		
2.1 Has a literature review been completed including examination of prior investigations?		
2.2 Is the aquifer system adequately described?		
2.2.1 Hydrostratigraphy including aquifer type (porous, fractured rock ...)		
2.2.2 Lateral extent, boundaries and significant internal features such as faults and regional folds		
2.2.3 Aquifer geometry including layer elevations and thicknesses		
2.2.4 Confined or unconfined flow and the variation of these conditions in space and time		
2.3 Have data on groundwater stresses been collected and analysed?		
2.3.1 Recharge from rainfall, irrigation, floods, lakes		
2.3.2 River or lake stage heights		
2.3.3 Groundwater usage (pumping, returns, etc.)		
2.3.4 Evapotranspiration		
2.3.5 Other		
2.4 Have groundwater level observations been collected and analysed?		
2.4.1 Selection of representative bore hydrographs		
2.4.2 Comparison of hydrographs		
2.4.3 Effect of stresses on hydrographs		
2.4.4 Water table maps / piezometric surfaces		
2.4.5 If relevant, are density and barometric effects taken into account in the interpretation of groundwater head and flow data?		
2.5 Have flow observations been collected and analysed?		
2.5.1 Baseflow in rivers		
2.5.2 Discharge in springs		
2.5.3 Location of diffuse discharge areas		
2.6 Is the measurement error or data uncertainty reported?		
2.6.1 Measurement error for directly measured quantities (e.g. piezometric level, concentration, flows)		
2.6.2 Spatial variability / heterogeneity of parameters		
2.6.3 Interpolation algorithm(s) and uncertainty of gridded data		

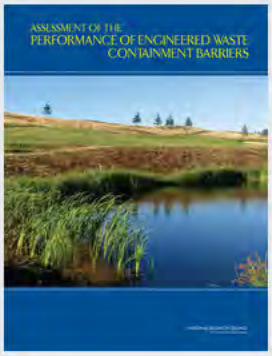
Review Questions	*Yes/No*	*Comment*
2.7 Have consistent data units and geometric datum been used?		
2.8 Is there a clear description of the conceptual model?		
2.8.1 Is there a graphical representation of the conceptual model?		
2.8.2 Is the conceptual model based on all available, relevant data?		
2.9 Is the conceptual model consistent with the model objectives and target model confidence level classification?		
2.9.1 Are the relevant processes identified?		
2.9.2 Is justification provided for omission or simplification of processes?		
2.10 Have alternative conceptual models been investigated?		
3 Design and construction		
3.1 Is the design consistent with the conceptual model?		
3.2 Is the choice of numerical method and software appropriate?		
3.2.1 Are the numerical and discretisation methods appropriate?		
3.2.2 Is the software reputable?		
3.2.3 Is the software included in the archive or are references to the software provided?		
3.3 Are the spatial domain and discretisation appropriate?		
3.3.1 1D / 2D / 3D		
3.3.2 Lateral extent		
3.3.3 Layer geometry		
3.3.4 Is the horizontal discretisation appropriate for the objectives, problem setting, conceptual model and target confidence level classification?		
3.3.5 Is the vertical discretisation appropriate? Are aquitards divided in multiple layers to model time lags of propagation of responses in the vertical direction?		
3.4 Are the temporal domain and discretisation appropriate?		
3.4.1 Steady state or transient		
3.4.2 Stress periods		
3.4.3 Time steps		
3.5 Are the boundary conditions plausible and sufficiently unrestrictive?		
3.5.1 Is the implementation of boundary conditions consistent with the conceptual model?		
3.5.2 Are the boundary conditions chosen to have a minimal impact on key model outcomes? How is this ascertained?		
3.5.3 Is the calculation of diffuse recharge consistent with model objectives and confidence level?		
3.5.4 Are lateral boundaries time-invariant?		
3.6 Are the initial conditions appropriate?		
3.6.1 Are the initial heads based on interpolation or on groundwater modelling?		
3.6.2 Is the effect of initial conditions on key model outcomes assessed?		
3.6.3 How is the initial concentration of solutes obtained (when relevant)?		
3.7 Is the numerical solution of the model adequate?		

Review Questions	*Yes/No*	*Comment*
3.7.1 Solution method / solver		
3.7.2 Convergence criteria		
3.7.3 Numerical precision		
4 Calibration and sensitivity		
4.1 Are all available types of observations used for calibration?		
4.1.1 Groundwater head data		
4.1.2 Flux observations		
4.1.3 Other: environmental tracers, gradients, age, temperature, concentrations, etc.		
4.2 Does the calibration methodology conform to best practice?		
4.2.1 Parameterisation		
4.2.2 Objective function		
4.2.3 Identifiability of parameters		
4.2.4 Which methodology is used for model calibration?		
4.3 Is a sensitivity of key model outcomes assessed against:		
4.3.1 Parameters		
4.3.2 Boundary conditions		
4.3.3 Initial conditions		
4.3.4 Stresses		
4.4 Have the calibration results been adequately reported?		
4.4.1 Are there graphs showing modelled and observed hydrographs at an appropriate scale?		
4.4.2 Is it clear whether observed or assumed vertical head gradients have been replicated by the model?		
4.4.3 Are calibration statistics reported and illustrated in a reasonable manner?		
4.5 Are multiple methods of plotting calibration results used to highlight goodness of fit robustly? Is the model sufficiently calibrated?		
4.5.1 Spatially		
4.5.2 Temporally		
4.6 Are the calibrated parameters plausible?		
4.7 Are the water volumes and fluxes in the water balance realistic?		
4.8 has the model been verified?		
5 Prediction		
5.1 Are the model predictions designed in a manner that meets the model objectives?		
5.2 Is predictive uncertainty acknowledged and addressed?		
5.3 Are the assumed climatic stresses appropriate?		
5.4 Is a null scenario defined?		
5.5 Are the scenarios defined in accordance with the model objectives and confidence level classification?		
5.5.1 Are the pumping stresses similar in magnitude to those of the calibrated model? If not is there reference made to the associated reduction in model confidence?		
5.5.2 Are well losses accounted for when estimating maximum pumping rates per well?		
5.5.3 Is the temporal scale of the predictions commensurate with the calibrated model? If not is there reference made to		

Review Questions	*Yes/No*	*Comment*
the associated reduction in model confidence?		
5.5.4 Are the assumed stresses and time scale appropriate for the stated objectives?		
5.6 Do the prediction results meet the stated objectives?		
5.7 Are the components of the predicted mass balance realistic?		
5.7.1 Are the pumping rates assigned in the input files equal to the modelled pumping rates?		
5.7.2 Does predicted seepage to or from a river exceed measured or expected river flow?		
5.7.3 Are there any anomalous boundary fluxes due to superposition of head dependent sinks (e.g. evapotranspiration) on head dependent boundary cells (Type 1 or 3 boundary conditions)?		
5.7.4 Is diffuse recharge from rainfall smaller than rainfall?		
5.7.5 Are model storage changes dominated by anomalous head increases in isolated cells that receive recharge?		
5.8 Has particle tracking been considered as an alternative to solute transport modelling?		
6 Uncertainty		
6.1 Is some qualitative or quantitative measure of uncertainty associated with the prediction reported together with the prediction?		
6.2 Is the model with minimum prediction error variance chosen for each prediction?		
6.3 Are the sources of uncertainty discussed?		
6.3.1 Measurement of uncertainty of observations and parameters		
6.3.2 Structural or model uncertainty		
6.4 Is the approach to estimation of uncertainty described and appropriate?		
6.5 Are there useful depictions of uncertainty?		
7 Solute Transport		
7.1 Have all available data on the solute distributions, sources and transport processes been collected and analysed?		
7.2 Has the appropriate extent of the model domain been delineated and are the adopted solute concentration boundaries defensible?		
7.3 Is the choice of numerical method and software appropriate?		
7.4 Is the grid design and resolution adequate, and has the effect of the discretisation on the model outcomes been systematically evaluated?		
7.5 Is there sufficient basis for the description and parameterisation of the solute transport processes?		
7.6 Are the solver and its parameters appropriate for the problem under consideration?		
7.7 Has the relative importance of advection, dispersion and diffusion been assessed?		
7.8 Has an assessment been made of the need to consider variable density conditions?		
7.9 Is the initial solute concentration distribution sufficiently well-known for transient problems, and consistent with the initial conditions for head/pressure?		
7.10 Is the initial solute concentration distribution stable and		

Review Questions	*Yes/No*	*Comment*
in equilibrium with the solute boundary conditions and stresses?		
7.11 Is the calibration based on meaningful metrics?		
7.12 Has the effect of spatial and temporal discretisation and solution method taken into account in the sensitivity analysis?		
7.13 Has the effect of flow parameters on solute concentration predictions been evaluated, or have solute concentrations been used to constrain flow parameters?		
7.14 Does the uncertainty analysis consider the effect of solute transport parameter uncertainty, grid design and solver selection/settings?		
7.15 Does the report address the role of geologic heterogeneity on solute concentration distributions?		
8 Surface water – groundwater interaction		
8.1 Is the conceptualisation of surface water–groundwater interaction in accordance with the model objectives?		
8.2 Is the implementation of surface water– groundwater interaction appropriate?		
8.3 Is the groundwater model coupled with a surface water model?		
8.3.1 Is the adopted approach appropriate?		
8.3.2 Have appropriate time steps and stress periods been adopted?		
8.3.3 Are the interface fluxes consistent between the groundwater and surface water models?		

ATTACHMENT 3



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Committee to Assess the Performance of Engineered Barriers

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Preface

Engineered barriers to isolate potentially harmful waste from humans and ecosystems have been used for over 35 years, and much has been written about them and their constituent components. However, few reports have provided an overall assessment of the performance of engineered barrier systems. The last broad assessment was conducted in 1995 (Rumer and Mitchell, 1995). Since that time, new materials and sensor technologies have been introduced and models to predict contaminant transport have improved.

At the request of program managers at the Environmental Protection Agency, Nuclear Regulatory Commission, National Science Foundation, and Department of Energy, the National Research Council established a committee to assess the effectiveness of surface and subsurface engineered barriers over the long term. The Committee to Assess the Performance of Engineered Barriers comprised academics and practitioners who collectively possessed expertise covering the science and technology of waste containment system regulations, analyses, design, construction, operations, maintenance, monitoring, and performance evaluation.

The study was guided by recognition that a defensible assessment of the long-term performance of engineered waste barriers must take into account the materials acting both individually and as part of a composite containment system, the type of waste contained, and performance indicators such as leakage rates, contaminant concentrations, and the condition of system components, all as a function of time and location. Information on these and other aspects of barrier systems was gleaned from the literature, briefings at committee meetings and field trips, discussions with colleagues, and the knowledge and experience of committee members. The committee met four times between October 2005 and August 2006 and visited four engineered barrier facilities: the McColl Superfund Site and the Puente Hills Landfill in southern California and the Love Canal treatment facility and the Model City Landfill in New York.

The committee thanks the following individuals for briefing the committee, hosting field trips, or providing background materials: Edmond Bourke, Rachel Detwiler, Brian Downie, Richard Fragaszy, John Hino, Ron Johnson, Jack Keener, Walter Kovalick, Kai Kuo, J. Michael Kuperberg, Kelly Madalinski, Don McLeod, Thomas Nicholson, Scott Parkhill, Jacob Philip, David Rothbart, Brian Sadowski, and Greg Zayatz. Special thanks go to Stephen Hammond and the New York State Department of Environmental Conservation, who provided data and information on the effectiveness of the state's modern engineered barrier systems. Finally, the committee extends its thanks and appreciation to Anne Linn, who served so ably and cheerfully as study director. Without her organizational and writing skills, knowledge, enthusiasm, and ability to keep the committee focused and on track, completion of this study would not have been possible.

James K. Mitchell, *Chair*

Acknowledgments

This report has been reviewed in draft form by individuals chosen for their diverse perspectives and technical expertise, in accordance with procedures approved by the National Research Council's Report Review Committee. The purpose of this independent review is to provide candid and critical comments that will assist the institution in making its published report as sound as possible and to ensure that the report meets institutional standards for objectivity, evidence, and responsiveness to the study charge. The review comments and draft manuscript remain confidential to protect the integrity of the deliberative process. We wish to thank the following individuals for their review of this report:

Craig H. Benson, University of Wisconsin, Madison
Rudolph Bonaparte, Geosyntec Consultants, Atlanta, Georgia
Jeffrey C. Evans, Bucknell University, Lewisburg, Pennsylvania
M. James Hendry, University of Saskatchewan, Saskatoon
Susan Hubbard, Lawrence Berkeley National Laboratory, Berkeley, California
Stephan Jefferis, University of Surrey, Guildford, United Kingdom
Robert M. Koerner, Drexel University, Philadelphia, Pennsylvania
Kenneth A. Snyder, National Institute of Standards and Technology, Gaithersburg, Maryland

Although the reviewers listed above have provided many constructive comments and suggestions, they were not asked to endorse the conclusions or recommendations, nor did they see the final draft of the report before its release. The review of this report was overseen by William L. Fisher, The University of Texas at Austin. Appointed by the National Research Council, he was responsible for making certain that an independent examination of the report was carried out in accordance with institutional procedures and that all review comments were carefully considered. Responsibility for the final content of this report rests entirely with the authoring committee and the institution.

Contents

Summary

Modern waste containment systems rely on surface and subsurface engineered barriers to contain hazardous and toxic waste, to prevent the offsite flow of contaminants, and/or to render waste less harmful to humans and ecosystems for tens to hundreds or thousands of years, depending on the type of waste, local conditions (e.g., geological setting, climate, land use), and regulations. The barriers may be at the bottom, top (cover), and/or sides (lateral barriers or walls) of the waste containment system, and they usually employ a variety of materials and mechanisms (e.g., liquid extraction) to control contaminant transport. Barriers are made of natural (e.g., soil, clay) and/or synthetic materials, such as polymeric materials (e.g., geomembranes, geosynthetic clay liners), usually arranged in layers.

Engineered barrier systems are monitored for effectiveness and proper functioning. Specified parameters that are observed or measured at the time of construction (e.g., hydraulic conductivity) provide an indication of whether the components of the systems will function as designed. Common measures of the effectiveness of a barrier system include the rate of release of contaminants from the barrier system and/or the detection of concentrations of contaminants beyond the boundaries of the barrier that exceed specified allowable maximum values. Design, initial performance, and monitoring criteria for these waste containment systems are governed by federal and state environmental regulations initially put in place beginning in the mid-1970s.

At the request of the Environmental Protection Agency (EPA), Department of Energy (DOE), National Science Foundation (NSF), and Nuclear Regulatory Commission (USNRC), the National Academies Committee to Assess the Performance of Engineered Barriers was established to provide a technical assessment of the available information on engineered barrier performance over time. The committee was charged with the following tasks:

1. Identify engineered barrier systems used for surface and subsurface waste containment.
2. Describe how performance is defined, predicted, measured, and monitored.
3. Present information on field performance of engineered barrier systems.
4. Evaluate the information on field performance.
5. Assess methodologies and capabilities for predicting and monitoring performance and for assessing risk.
6. Identify information needed to fill the knowledge gaps.

This report focuses on engineered barriers designed to contain municipal solid waste, other nonhazardous solid and liquid waste, hazardous and toxic wastes, and low-level radioactive wastes. The primary questions addressed are: How well are these engineered barrier systems working? How long are they likely to work effectively? Because engineered barrier systems constructed in compliance with current regulatory requirements have been operating for only a few decades at most, the assessment necessarily focuses on short- and medium-term performance. Predictions of long-term performance must be based on models and extrapolation of data and observations obtained over shorter periods of time. In this report, performance periods are defined as follows:

- short term: the period until completion of construction of the barrier component,
- medium term: the operating period of the waste unit, and
- long term: the postclosure period.

Based on as much as 20 years of observations, the committee concluded that most engineered waste containment barrier systems that have been designed, constructed, operated, and maintained in accordance with current statutory regulations and requirements have thus far provided environmental protection at or above specified levels. Extrapolations of long-term performance can be made from existing data and models, but they will have high uncertainties until field data are accumulated for longer periods, perhaps 100 years or more. We will never have all the long-term observations and data that we would like.

Long-term containment is difficult and requires high-quality engineering. Few significant failures have occurred and, in general, repair or limited reconstruction has been possible. Given that development of optimal designs for lifetimes of thousands of years is likely to be both infeasible and prohibitively expensive, designs that allow for recovery, repair, and/or replacement are to be encouraged. Findings and recommendations on specific barrier components, systems, and models are described below.

MONITORING BARRIER PERFORMANCE

Because most waste containment systems are buried, their component systems are usually monitored indirectly. Direct monitoring of the integrity of barrier system components is generally limited to an end-of-construction assessment of the component. Modern construction quality assurance procedures have, in general, been effective in ensuring the integrity of barrier components in the short term.

The primary (top) liner in a double-liner system is perhaps the only type of engineered barrier system in which postconstruction integrity is routinely monitored directly. Liquids collected in the leak detection layer sandwiched between the primary and secondary (bottom) liners provide a direct assessment of the performance of the primary liner system. The postconstruction integrity of caps (covers) can be monitored by exhumation and testing of cap material. In situ moisture content monitoring of soil layers within and beneath containment system covers (caps) can provide an indirect measure of cap performance.

The performance of engineered barriers and barrier systems should be monitored with a variety of techniques and in a variety of media (surface water, groundwater, air, and soil). Geophysical techniques offer promise for cost-effective, long-term, indirect monitoring of barrier systems. For example, electrical resistivity and electromagnetic surveys may detect gross defects that facilitate concentrated flow through vertical barriers. Tomographic imaging and seismic velocity surveys may detect changes in physical properties caused by vertical barrier degradation. Multispectral imaging can show changes in vegetation and in water content and temperature in near-surface soils caused by problems with caps and vertical barriers. Interferometric synthetic aperture radar, light detection and ranging, and other airborne/satellite techniques can resolve centimeter-scale deformations caused by local or global instability or barrier performance problems. However, to date, these technologies have yielded little data that can be used to quantitatively and reliably monitor barrier systems. Development of these tools for long-term monitoring purposes is an area of ongoing research.

PERFORMANCE OF BARRIER SYSTEM COMPONENTS

Common barrier system components include earthen barriers (e.g., clay liners), geomembranes, geosynthetic clay liners, granular and geosynthetic drainage layers, evapotranspirative barriers, vertical barriers, and asphalt concrete barriers. Most of the information available is on components used in covers and liners; hence, these are covered in more detail in this report than components used in vertical barriers. Available data indicate that compacted clay layers generally perform effectively as components within barrier systems as long as good construction and/or operational practices are followed. However, secondary permeability may develop in unprotected clay liners and covers as a result of wetting and drying, freezing and thawing, and deformation processes. Diffusion can be a significant contributor to the total migration of chemical contaminants through well-constructed, low-permeability earthen barriers. High temperatures near the barrier and reactions between migrating chemicals and the earthen materials (especially bentonite) used for the barrier have the potential to increase the hydraulic conductivity above the usual target of $<1 \times 10^{-9}$ m/s over the medium and long terms. Additional monitoring will be required to determine whether compacted clay and composite barriers effectively halt volatile organic compound migration in the long term.

Geomembranes installed following strict construction quality assurance protocols exhibit significantly fewer leaks and perform better than those installed without such requirements. Defective materials or seams and physical damage caused during construction can all degrade short-term performance. Over the medium and long terms, geomembrane performance may be reduced by punctures caused by increased overburden pressure, material degradation, and high temperatures. The estimated service lives of geomembranes decrease from 1,000 years at 10°C to only about 15 years at 60°C. Geomembranes appear to offer little, if any, resistance to the migration of several types of volatile organic compounds. This lack of resistance can be a short-term problem if a geomembrane is used as the sole barrier, or a medium- or long-term problem if the barrier system is comprised of more than one barrier material or type.

The use of defective materials and/or separation of overlapped panels will decrease the short-term effectiveness of geosynthetic clay liners. Hydraulic conductivity may increase if the liner is exposed to relatively strong liquids (e.g., high ionic strength chemicals) and is a performance concern over all timescales. Medium- and long-term concerns for geosynthetic clay liners include the effects of desiccation and local and global slope instability. Chemical transport through individual geosynthetic clay liners can be a problem when holes are too large to permit self-healing (e.g., through swelling of bentonite) or when the liner is the sole barrier component and is susceptible to diffusion.

Granular drainage layers are important barrier components for reducing leachate head on liners and covers because they enhance stability and cut off advective and diffusive transport. Their short-term performance may be degraded by inadequate discharge capacity and clogging. Over the medium and long terms, granular drainage layers can become clogged as a result of soil infiltration, biological activity, and chemical precipitation. Geosynthetic drainage layers are susceptible to similar problems. Installation damage and inadequate capacity degrade their short-term performance, and clogging caused by soil infiltration, soil and geosynthetics penetration, creep of the geonet core, biological activity, and mineral precipitation degrade their medium- and long-term performance.

Evapotranspirative barriers are now beginning to be used in capacitive cover systems. Only a few years of data are available, but they suggest that evapotranspirative barriers can be an effective alternative to compacted clay or composite covers in arid and semiarid climates. Significantly more data over much longer time frames and/or studies of natural analogs that have functioned for hundreds or thousands of years are required to make a reliable prediction of the long-term performance of evapotranspirative barriers.

The short-term performance of vertical cutoff walls is primarily affected by the quality of construction. Construction defects that can compromise wall performance include gaps in the wall caused by poor mixing or defective material and high-permeability zones caused by caving or trapping of low-quality material at joints between panels. Chemical incompatibility, desiccation above the water table, and cracking caused by various mechanisms all adversely affect the medium- and long-term performance of vertical cutoff walls. Defective materials, cracking, and degradation are also performance concerns for asphalt cement barriers.

CONTAINMENT SYSTEM PERFORMANCE

Although existing data suggest that modern containment systems are performing well thus far, they have not been in existence long enough to allow a direct assessment of long-term performance. Likewise, models appear to be capable of predicting long-term performance, although relatively few field data exist to verify the models. Models that predict the long-term performance of containment systems depend on predictions of the long-term integrity of containment system elements. Thus, maintaining the integrity of containment system elements over the active life of the wastes they contain appears to be required to assure satisfactory long-term performance of engineered barrier systems. Moreover, redundant design appears to enable the waste containment system to serve as an effective barrier to contaminant transport, even if the performance of an individual component degrades with time.

Available data show that liners constructed following rigorous construction quality assurance guidelines provide protection against offsite contaminant leakage. Composite liners composed of either compacted clay and geomembranes or geosynthetic clay layers and geomembranes provide better protection than any single component acting alone. Reliable predictions of leakage rates through composite liners should take into account holes in geosynthetic wrinkles and elevated leachate head. Cover systems are effective at isolating waste, as long as periodic maintenance is performed. Vertical waste containment barriers have not been monitored sufficiently to draw conclusions about their field performance or the accuracy of predictions of the transport of contaminants through them.

RECOMMENDATIONS

Data Collection and Distribution

A systematic approach to data collection and reporting that targets the most important data and makes those data readily accessible would greatly facilitate periodic assessments of long-term performance. Key types of data that should be collected are listed in Table 6.1. Key parameters that should be monitored include groundwater quality in the saturated zone at the down-gradient edge of the containment facility, gas emissions in the vadose zone around the site if the waste has potential for generating harmful gases (e.g., methane), leachate head acting on the liner inside the containment facility, temperature on geomembrane liners, and the quality and quantity of leachate being generated by the facility.

Recommendation 1: Monitoring programs for new facilities should include provisions for collecting data needed to assess the long-term performance of engineered barriers, and operators of existing facilities should collect these data to the extent practical using in-place monitoring systems.

The performance of many engineered barriers is monitored indirectly, usually through evidence of contaminant migration outside the waste containment system. The absence of direct monitoring data introduces uncertainties about how well the individual elements of the overall containment system are working.

Recommendation 2: Regulatory agencies should develop guidelines to increase direct monitoring of barrier systems and their components, and NSF should sponsor research for the development of new cost-effective monitoring techniques, especially for assessing the effectiveness of vertical barriers, for this purpose.

Assessing or predicting the performance of engineered barriers is made more difficult because the necessary data and observational information do not exist, are hard to find, are incomplete, or have not been analyzed. The effort to compile and evaluate these data is considerable, but there is enough new information on field performance, material behavior, and monitoring and modeling capabilities to make an assessment of performance worthwhile about every 5 to 10 years. More frequent assessments may be required based on previous monitoring data and performance assessment models.

Recommendation 3: Federal agencies responsible for engineered barrier systems should commission and fund assessments of performance on a regular basis. Given the rate at which performance data and knowledge of waste behavior, contaminant transport, and monitoring accumulate, the interval at which these assessments should take place is probably on the order of once every 5 to 10 years. The results of the assessment should be placed in the public domain in a form that is readily accessible.

Much data used to predict performance come from laboratory experiments, models, and field-constructed prototype barrier systems (e.g., test pads). Although useful for understanding material properties and behavior, these data are no substitute for performance data collected in the field from operating containment systems. An overall comprehensive assessment of performance requires long-term monitoring and analysis of data from different types of waste containment systems constructed from a variety of components and located in different climate regimes.

Recommendation 4: EPA, USNRC, NSF, and DOE should establish a set of observatories at operational containment facilities to assess the long-term performance of waste containment systems at field scale. The program would involve building one or more field facilities, monitoring the site, and analyzing and archiving the data. New sites could be created or adjustments could be made to existing observatories when promising new and innovative concepts and materials become available.

Models

Analytical and numerical models are relied on to predict contaminant transport, containment effectiveness, degradation of materials, and changes in behavior over time, even though some models have shortcomings (e.g., they do not account for advection-dispersion processes; they are used in applications for which they were not designed).

Recommendation 5: Regulatory agencies (e.g., EPA, DOE, USNRC) and research sponsors (e.g., NSF) should support the validation, calibration, and improvement of models to predict the behavior of containment system components and the composite system over long periods of time. These models should be validated and calibrated using the results of field observations and measurements.

Monitoring Periods

The optimum time for monitoring varies with the facility, type of waste, climate, and the observed performance. Yet funding is often not available to continue monitoring until the site no longer poses risk to human health and the environment, and no national policy exists to assure that such funding will be available.

Recommendation 6: EPA should develop financial assurance mechanisms to ensure that funding is available for monitoring and care for as long as the waste poses a threat to human health and the environment.

Performance Criteria

Performance criteria are needed that account for both barrier performance and impacts to public health and safety that extend beyond the barrier system.

Recommendation 7: EPA and USNRC should develop guidance for the practical implementation of performance-based criteria for assessment of containment system performance as an alternative to prescriptive designs.

1

Introduction

"Municipal" waste dumps have existed in the western world since at least 500 B.C.,[1] but it was not until landfills began to be built according to established guidelines that the concept of engineered containment systems was born. By that definition, the first engineered barriers were built in the 1960s, after the U.S. Public Health Service and the American Society of Civil Engineers published guides recommending that waste be compacted and covered with a new layer of soil each day to guard against rodents and odors.[2]

Waste disposal systems have since become increasingly sophisticated, driven by advances in research and engineering practice, the generation of increasing amounts of hazardous and toxic wastes, and new requirements in state and federal regulations. Modern waste disposal systems and barriers to isolate subsurface contaminants are engineered to provide safe long-term containment of municipal solid waste, other nonhazardous solid and liquid waste (e.g., industrial waste), hazardous and toxic wastes, and low-level radioactive waste. Engineered barriers are used to contain this waste, to prevent the movement of contaminants offsite, to minimize infiltration of surface and groundwater into the waste, and/or to render waste less harmful to people and ecosystems for tens to hundreds or thousands of years, depending on the type of waste and local circumstances. Design and initial performance criteria and monitoring requirements for these containment systems are governed by federal and state environmental regulations.

There are approximately 4,000 landfills in the United States (Bonaparte et al., 2002) and about 7,800 contaminated sites awaiting corrective actions and cleanup.[3] Corrective actions have already been completed for approximately 328 sites under the Resource Conservation and Recovery Act (RCRA; personal communication from Tom Rinehart, Chief, RCRA Corrective Action Branch, Environmental Protection Agency [EPA], on September 26, 2006) and many more sites under state jurisdiction or owner or operator control. An assessment of the performance of permitted modern landfills and contaminated-site corrective actions is timely now that some of these engineered barrier systems are approaching the end of their postclosure monitoring periods (commonly 30 years).

The Committee to Assess the Performance of Engineered Barriers was established to provide a technical assessment of the available information on engineered barrier performance over time. The charge to the committee is given in Box 1.1.

The two primary questions addressed in this study are: How well are engineered barrier systems working? How

BOX 1.1 Committee Charge

In order to develop an improved framework for assessing the effectiveness of surface and subsurface engineered barriers over the long term, an ad hoc committee will complete the following tasks:

1. Identify engineered barrier systems used for surface and subsurface waste containment.
2. Describe how performance is defined, predicted, measured, and monitored.
3. Present information on field performance of engineered barrier systems.
4. Evaluate the information on field performance.
5. Assess methodologies and capabilities for predicting and monitoring performance and for assessing risk.
6. Identify information needed to fill the knowledge gaps.

[1] The first documented waste dump was built in Athens. See <http://www.epa.gov/epaoswer/non-hw/muncpl/timeline_alt.htm>.

[2] <http://www.epa.gov/epaoswer/non-hw/muncpl/timeline_alt.htm>.

[3] These include sites regulated under RCRA; the Comprehensive Environmental Restoration, Compensation, and Liability Act; the Uranium Mill Tailings Radiation Control Act; and the Low-Level Waste Policy Act. See <http://web.em.doe.gov/bemr96/umtra.html> and Environmental Protection Agency Superfund, Radiation, and Wastes pages at <http://www.epa.gov>.

long are they likely to work effectively? The answers to these questions were derived from analysis of available data and case history information, input from federal agency managers and outside experts, information provided by waste containment facility operators and technical personnel, and the research and practical experience of members of the committee.

This report focuses on engineered barriers that were designed to contain municipal solid waste, other nonhazardous solid and liquid waste, hazardous and toxic wastes, and low-level radioactive wastes. Barriers constructed for both waste landfills and corrective action containment and cleanup of contaminated sites were considered. Barrier systems that were intended to treat rather than isolate waste (e.g., reactive barriers that transform contaminants into nonhazardous substances) were not considered.

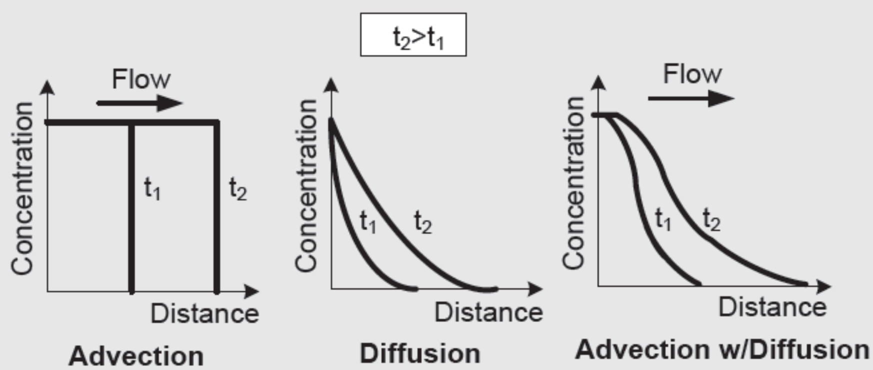
Many barrier systems are intended to function for tens to thousands of years. Because most regulated sites have been in operation less than 30 years, it is not possible to make unequivocal conclusions about the actual long-term performance of these systems. Consequently, the focus of this report is on performance over periods of several tens of years—the operating, maintenance, and monitoring period for many waste collection systems (Box 1.2)—and on information that provides useful insights about future behavior. Partly because of the very long times (on the order of 1 million years) required to isolate high-level radioactive waste, and partly because no examples exist in the United

BOX 1.2 Terms Used in This Report

Contaminant: Any solid, liquid, or gas resulting directly or indirectly from human activities that may cause an adverse effect on human health and/or the environment.

Modes of contaminant transport:

- Advective transport of contaminants driven by hydraulic or pneumatic gradients
- Diffusive transport of contaminants driven by chemical gradients
- Coupled flows of contaminants driven by thermal and electrical gradients
- Mechanical transport of contaminants by processes such as wind or water erosion



Flux: A measure of the amount of contaminant flow through the ground or a two-dimensional barrier. Flux is usually expressed as a volume or mass passing though a unit cross-section area per unit of time.

Hydraulic conductivity: The rate at which a liquid flows through soil under a unit hydraulic gradient. The hydraulic gradient is the amount of fluid total head loss divided by the flow distance over which it is lost.

Periods of performance (as used in this report):

- Short term: the period until completion of construction of the barrier component
- Medium term: the operating period of the waste unit
- Long term: the postclosure period

States, engineered barriers for high-level radioactive waste were not evaluated.

1.1 WHAT ARE ENGINEERED BARRIERS?

Engineered barrier systems for surface and subsurface waste containment comprise components designed to contain, control, and retard the migration of contaminants toward and within the subsurface and to prevent surface water from infiltrating into the waste or contaminated ground. These component systems may include a low-permeability bottom, side walls, and a cover, as shown schematically in Figure 1.1. Barrier materials typically include natural or modified soil; cementitious, bituminous, and synthetic materials; aggregates; and reactive media, usually arranged in layers. Figure 1.2 shows a geomembrane cover system constructed to prevent ingress of surface water into contaminated ground. The component systems and the materials from which they are constructed are the product of extensive materials research, theoretical analysis, laboratory testing, field measurements, and construction considerations.

Engineered barrier systems can be used to control migration of both liquids and gases (Box 1.2). Most engineered barriers are designed to control advective contaminant transport (i.e., movement of dissolved and suspended materials within flowing fluids) and to intercept and contain the flow of contaminants. In addition, barriers are designed to promote contaminant retention by mechanisms such as sorption, and they may also hinder diffusive contaminant transport. The potential for direct contact by humans or other organisms may also influence the design and effectiveness of engineered barriers.

Engineered barrier system components include bottom barriers, covers, and lateral barriers or walls. These components may comprise a single element with a single mechanism to control contaminants, such as a low-permeability vertical barrier wall to control advective transport. Usually, however, liners and covers contain several different component layers, each with a specific role. An example is a bottom barrier composite liner system that employs a high-permeability drainage layer to extract contaminated liquids (leachate) and/or gas overlying a low-permeability barrier to resist advective and diffusive transport of liquids and gas.

No engineered barrier system can be relied on to completely halt the transport of contaminants, but the rate of release of contaminants to the environment can be minimized. Consequently, most engineered barrier systems are monitored for effectiveness and proper functioning.

Waste repositories and containment systems are a unique and difficult class of structures because (1) small component

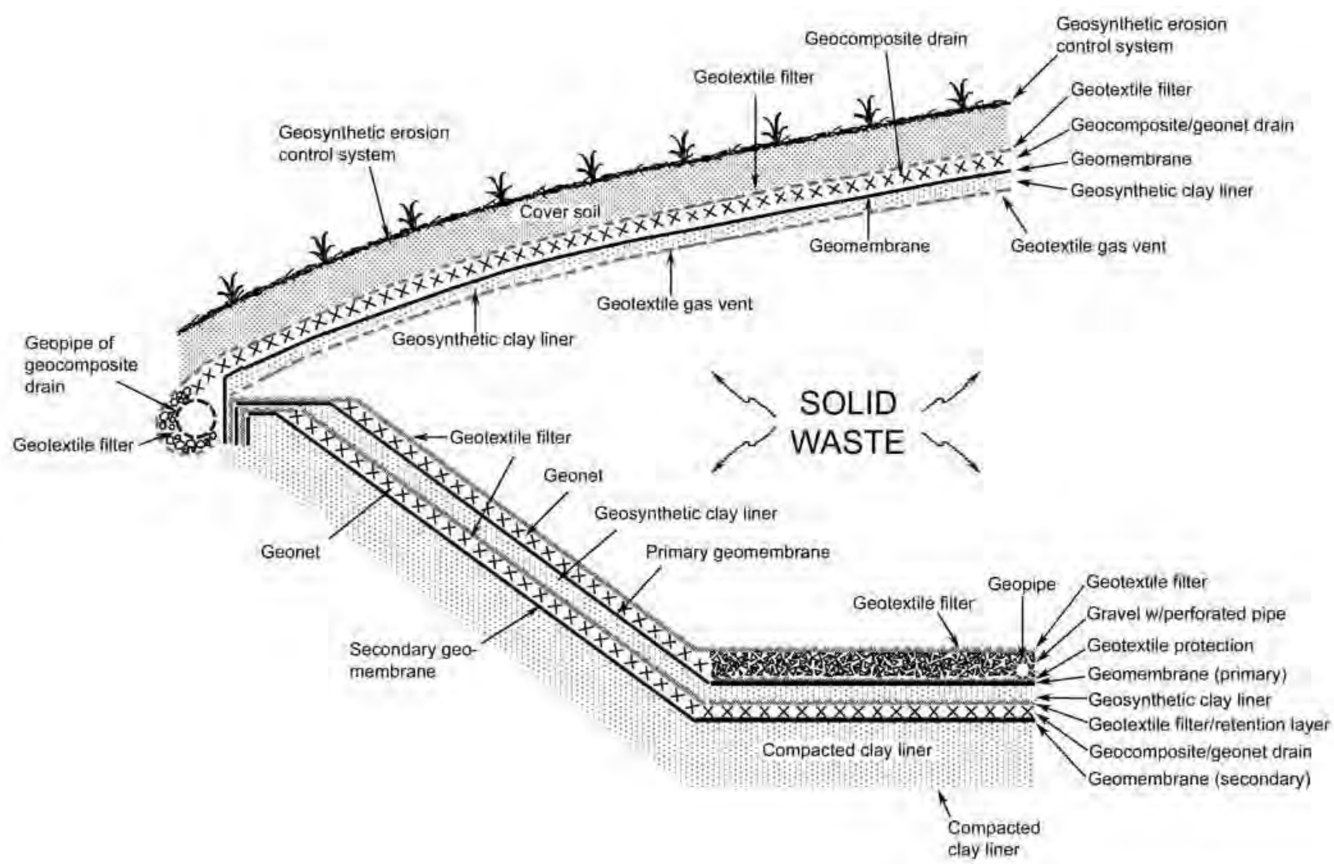


FIGURE 1.1 Schematic drawing of a municipal solid waste landfill. SOURCE: Bonaparte et al. (2002).

FIGURE 1.2 Geomembrane cover placed over contaminated ground as part of a RCRA Corrective Action at Love Canal in New York state in 1989. SOURCE: Scott Parkhill, Miller Springs Remediation Management, Inc.

failures or areas of damage can compromise the integrity of the entire system and (2) the operational lives are so long (up to thousands of years). Although an extensive body of literature exists on the performance of some barrier components, especially liners and covers, information on the performance of other components, especially vertical barriers, is sparse.

1.2 WHAT IS PERFORMANCE?

The performance of barrier systems can be defined in relation to governmental regulations (e.g., prescriptive requirements, performance standards, environmental impacts). A common definition of good performance is whether the components and systems function as designed in terms of specified measured or observed parameters. The rate of release of contaminants from a barrier system, if known, and/or the detection of excessive contaminant concentrations beyond barrier boundaries (i.e., at a specified point of compliance) are the most commonly used measures of performance. Performance can be defined in terms of the magnitude of the flux of contaminants, liquids, or gases at a given time; the cumulative or average magnitude of the flux over a period of time; or the length of time to reach a given magnitude of flux. Prescriptive criteria for the composition of the barrier itself and performance criteria for system components can also be used to define barrier performance. Examples of component prescriptive criteria include the following:

- the hydraulic conductivity of a clay liner is less than a specified maximum allowable value and the thickness of the liner is greater than a specified minimum required value;
- the hydraulic conductivity and thickness (or transmissivity) of a drainage layer are greater than specified minimum allowable values; and
- the thickness of a geomembrane or a vegetated erosion control layer is greater than a specified minimum required value.

Examples of system performance criteria include the following:

- the head in a drainage layer is less than a specified maximum value or
- the flow rate into a leak detection system is less than a specified maximum value (the action leakage rate).

Other definitions of engineered barrier performance consider both the barrier system and the interactions between the barrier system and the environment (see Appendix A). An example measure of this broader definition is that water, air, and soil quality pose an acceptable risk to human health and the environment, where risk is expressed as an incremental lifetime cancer risk for humans, a hazard quotient for humans or ecological systems, a toxicity unit for ecological systems, or a line of evidence for ecological systems. Such measures require subjective analysis beyond the scope of this report; consequently, this report focuses on the performance of the barrier system itself.

1.3 ORGANIZATION OF THE REPORT

This report provides a framework for assessing the effectiveness of engineered barrier systems and their surface and subsurface components over periods extending beyond several tens of years, although this necessarily requires extrapolation of data and observations obtained over shorter time periods. The intended audience includes government agencies responsible for developing regulations and guidelines to ensure compliance with regulations to protect human health and the environment, scientists and engineers interested in identifying areas of uncertainty and carrying out additional study, individuals concerned with the long-term safe containment and remediation of contaminated sites, and managers of engineered disposal facilities.

The report is organized into six chapters. Chapter 2 provides an overview of engineered barrier systems, including waste types, barrier materials and their roles as components of waste containment systems, and systems for isolating and containing different types of waste. Statutory requirements and methods for monitoring performance are summarized in Chapter 3. Details on how key parameters (e.g., hydraulic head) are measured and used appear in Appendix B, and construction quality assurance techniques for monitoring barrier element integrity appear in Appendix C. Chapter 4 summarizes observations and predictions of the performance of barrier system components, based on field data, laboratory studies, and modeling. Additional information on predicting impacts on human health and the environment is given in Appendix A. The overall field performance of engineered barrier systems, including case histories, is presented and evaluated for completeness and reliability in Chapter 5. Finally, Chapter 6 identifies information gaps and research needs and presents recommendations for both assessing and enhancing the long-term performance of engineered barrier systems.

2

Overview of Engineered Barrier Systems

This chapter presents an overview of engineered barrier systems, including the types of wastes that are contained by barrier systems, the regulations that govern barrier systems, and the variety of components and configurations that comprise barrier systems. There are close relationships between waste types, regulations, and barrier systems, which ultimately drive how well the barrier systems will perform. The chapter concludes with a description of the life cycle of landfills.

2.1 WASTE CLASSIFICATION AND DISPOSAL REQUIREMENTS

Surface and subsurface barriers are mandated for containment of regulated solid and liquid wastes. Federal, state, and local regulations define regulated solid wastes and how such wastes are characterized, treated, stored, and disposed of. The first step in classifying a material as a regulated solid waste is to determine whether the material is "inherently waste like" (40 CFR 261.2(a)(2)(iii)). Once a material is determined to be a solid waste or waste by-product (e.g., contaminated soil and groundwater, gaseous emissions), it is classified by composition, source, or location. Wastes classified by composition include solid waste, hazardous waste, liquid waste, radioactive waste, mill tailings, and infectious waste. Sources of waste include residential, commercial, and institutional activity; industrial enterprises; farming and ranching; mining; dredging; nuclear power and nuclear defense; and medical activity. The various means used to classify wastes in the United States are summarized in Sharma and Reddy (2004).

Disposal requirements for regulated waste depend on how the waste is classified. Thus, regulatory requirements for disposal and containment vary. The configurations and monitoring systems for engineered barriers are to a large extent determined by the regulatory requirements. Major federal statutes governing waste containment systems are summarized in Table 2.1.

All states and tribal authorities must conform with federal regulations setting minimum standards for waste disposal, containment, and management of solid wastes. Typically, when two different types of waste are mixed together or when one type of waste is derived from another type, the waste type with the more stringent regulatory requirements takes precedence. However, solid waste that would normally be regulated because of its composition might be exempt from regulation because of its source. Examples of exempted waste include agricultural waste from farming and ranching activities, wastes generated from mining and hydrocarbon production, dredging spoil, very low-level radioactive wastes from industrial plants and medical facilities, and infectious wastes from medical activities. Most federal regulations have state analogs that are often more restrictive than the federal regulations.

2.2 ENGINEERED BARRIER SYSTEMS

Engineered barrier systems for containing waste can be categorized by functional mechanism (resistance, capacitance, extraction, and injection) or by orientation (covers, bottom barriers, and lateral barriers). The general characteristics of the two classification systems and the components of barrier systems (e.g., liners, liquid collection layers) are described below.

2.2.1 Functional Mechanisms

Engineered barriers employ a variety of functional mechanisms to contain waste. Resistance is probably the most common functional mechanism, and resistive barriers are used in bottom barrier, cover, and lateral barrier systems. An example is a soil or geomembrane liner in a bottom barrier (Figure 2.1). Resistive barriers contain waste by their inherent resistance to advective and/or diffusive transport of contaminants through them. An effective resistive barrier slows migration to the point where physical processes such

TABLE 2.1 Major U.S. Federal Statutes Governing Waste Classification and Containment

Waste Classification	Description	Federal Statute
Municipal solid waste	Focuses on residential and commercial refuse such as food, paper, glass, plastic, textile, grass, wood, and metal	Resource Conservation and Recovery Act (RCRA), as amended (42 USC 6901 et seq.) Subtitle D
Hazardous waste	Focuses on hazardous waste such as refining and manufacturing by-products, paint, solvents, pesticides, and ashes; hazardous wastes must be stabilized before disposal in a containment system	RCRA, as amended (42 USC 6901 et seq.) Subtitle C
Wastes associated with cleanup of abandoned hazardous waste sites	Applies to hazardous contaminated soils and liquids removed from the ground or generated from treatment	Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) (1986), as amended (42 USC 9601 et seq.)
Low-level radioactive and mixed waste from production of nuclear power (Nuclear Regulatory Commission) and weapons (Department of Energy)	Requires each state to provide disposal capacity for commercial low-level radioactive waste, such as protective clothing, filters, containers, tools, laboratory equipment, and piping	Low-Level Radioactive Waste Policy Act as amended in 1985 (42 USC 2021b et seq.)
Uranium tailings and other contaminated materials at uranium mill processing sites and adjacent properties	Applies to mining waste rock, tailings from ore processing, industrial waste, and waste water	Uranium Mill Tailings Radiation Control Act of 1978 (42 USC. 2022 et seq.)

as dilution can decrease downstream contaminant fluxes to levels that meet regulatory standards. While not the main purpose of a resistive barrier system, some resistive barriers contain reactive materials that transform contaminants as they pass through the barrier.

Capacitive barriers function by retaining contaminants or contaminant transport media by sorption processes or retention in pore spaces. An example is a layer in a cover barrier that stores water and supports vegetative growth on the cover (Figure 2.1). A capacitive barrier retains the contaminants and transport media by processes such as adsorption, redox reactions, and/or precipitation in pore spaces. A capacitive barrier that works by retention alone will merely delay the eventual breakthrough of contaminants unless the retention capacity exceeds the contaminant mass. Therefore, capacitive barriers usually require a supplemental mechanism if they are to be effective over the long term. Such mechanisms include (1) resistance to transport (described above); (2) reactive treatment, in which the contaminant reacts with barrier media and is transformed to a harmless substance; and (3) gradient reversal, in which the stored contaminant or transport media is released from the barrier back in the direction of its origin (e.g., release of stored water from soil covers by evapotranspiration). The use of zeolites in barriers for radioactive waste is an example of a capacitive barrier that also relies on other mechanisms to reduce contaminant transport: the zeolites retain the radioactive isotopes while radioactive decay processes reduce their potential impacts.

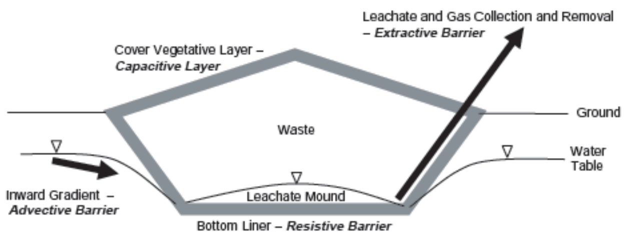


FIGURE 2.1 Schematic illustration of functional mechanisms for engineered barriers.

Advective barriers rely on advective flow to control the migration of contaminant transport media. In an advective barrier, gradients are introduced to generate a flow counter to the indigenous direction of contaminant transport. An example is a groundwater pumping system surrounding the waste that creates an inward gradient so that liquid flows into and not out of the system (Figure 2.1). Advective barriers also include pneumatic barriers, where a suction pressure is applied to control landfill gas transport.

Extractive barriers are used in conjunction with advective transport to remove contaminated liquid or gas for treatment and/or disposal. Examples are leachate and gas collection and removal systems that remove contaminated liquid and gas from inside the barrier system (Figure 2.1). Extractive barriers include extraction wells, trenches, and blanket leachate collector systems.

2.2.2 Barrier System Orientations

Final covers are the most common type of engineered barrier system. They are used for both engineered containment systems and nonengineered contaminated sites and dumps to keep waste and contaminants in and to keep potentially infiltrating water out. Most are resistive barriers, and many incorporate capacitive and extractive barriers to control surface water infiltration (Table 2.2). Enhanced capacitive covers, also referred to as alternative covers because they can

TABLE 2.2 Orientation, Components, and Functional Mechanisms of Engineered Barrier Systems

Barrier Orientation	Typical Components	Functional Mechanism
Final covers	Low-permeability soil layers, geosynthetic clay liners, geomembranes, ACC and PCC layers	Resistance
	Vegetated evapotranspirative soil layers	Capacitance
	Blanket drainage layers, gas vents	Extraction
Bottom barriers	Low-permeability soil liners, geomembranes, geosynthetic clay liners, ACC liners, PCC floors	Resistance
	Compacted soil attenuation layers	Capacitance
	Blanket leachate collection layers, hydraulic control layers	Extraction and/or injection
Lateral barriers	Soil-bentonite and cement-bentonite walls, PCC walls, soil-cement and jet-grouted walls, sheet pile walls, vertical geomembranes	Resistance
	Low-permeability “treatment” walls	Capacitance
	Vertical wells and trenches	Extraction and/or injection

NOTE: ACC = asphalt cement concrete; PCC = Portland cement concrete.

be substituted (with regulatory approval) for cover systems prescribed by regulation (i.e., composite or compacted clay covers), rely on evaporation and transpiration to control infiltrating surface water. They are becoming more common in arid or semiarid regions where potential evapotranspiration significantly exceeds actual precipitation (Shackelford, 2005).

Bottom barrier systems are used in landfills, surface impoundments, and other containment systems to hold waste and contaminants and to facilitate collection and removal of contaminated liquids. Basal liner systems for landfills are the most common bottom barrier system. Landfill basal liner systems are typically composed of several different components, including protection layers, liquid collection and removal systems, resistive barrier layers (liners), and diffusion attenuation layers. They generally employ a combination of mechanisms, including extraction, resistance, capacitance, and sometimes advection and reaction. Extractive bottom barriers (e.g., horizontal leachate collection wells) are also occasionally installed as remediation systems for contaminated sites. Resistive bottom barriers have also been proposed as remediation measures, but they are not commonly used because of practical difficulties associated with installing a continuous barrier beneath an existing contaminated site.

Lateral barrier systems are used to enclose wastes and provide barriers to groundwater flow or to facilitate removal of groundwater. They include side slope liner systems for landfills, vertical barrier walls, interceptor and extraction trenches, vertical extraction well systems, and vertical advective barriers. Side slope liner systems for landfills work in much the same way as landfill basal liner systems and rely on similar mechanisms and employ similar components. Vertical barrier wall systems are generally resistive barriers but may also include extractive components (e.g., extraction wells inside the barrier) or capacitive/reactive and advective mechanisms to supplement their resistance to contaminant transport. Extractive and injection lateral barrier systems, including vertical wells and trenches filled with granular materials, are used to control advective transport of liquid and gas in the subsurface. Extractive barriers, including vertical walls and extraction trenches, are usually accompanied by treatment and/or disposal of the extracted liquid and/or gas. The most common type of injection barrier is a vertical well into which air is injected to control the migration of gas in the vadose zone. Vertical liquid injection trenches can be used to create hydraulic gradients counter to the direction of contaminant transport. Injection barriers are often used in conjunction with some type of source control (e.g., gas or groundwater extraction) to limit the operating period of these systems. Vertical barrier walls and extraction trenches are commonly an integral part of systems used to isolate and contain waste and contaminated ground at previously uncontrolled disposal sites, many of which have been designated for remediation under the Superfund program.

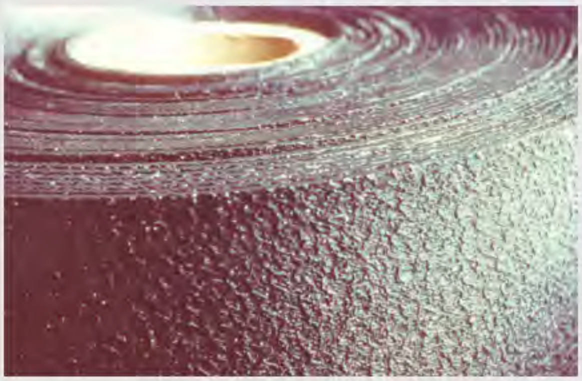
2.2.3 Barrier Components

Common components of engineered barrier systems are listed in Table 2.2. These components can be classified broadly as soils, aggregates, cementitious and bituminous materials, and geosynthetics. Geomembrane liners (Box 2.1) and low-permeability soil liners, which both serve as resistive barriers, are common components in engineered barriers and

BOX 2.1 Geosynthetic Barrier System Components

Over approximately the past 20 years, the use of components manufactured from synthetic polymeric materials, termed geosynthetics, has become commonplace in geotechnical engineering. Geosynthetics are often supplied in rolls 4.6 m wide and 30 to 60 m long (Koerner, 2005). Common geosynthetic materials employed in engineered containment systems include geomembranes, geotextiles, geonets, and geosynthetic clay liners.

Geomembranes: Polymeric sheets, typically 0.57 to 2 mm thick, used as resistive barriers. They can be joined or seamed to adjacent sheets to cover large areas. High-density polyethylene is most commonly used for landfill bottom liners due to its superior chemical resistance. However, more flexible polymers, such as low-density polyethylene, polypropylene, and polyvinyl chloride are often used in final covers to accommodate settlement. Geomembranes may be smooth or textured to enhance the shear resistance of the interfaces between the overlying or underlying materials.



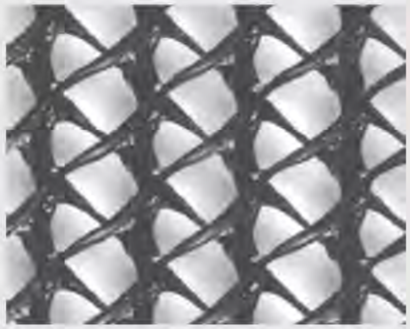
Textured geomembrane roll. SOURCE: GSE Lining Technology, Inc.

Geomembrane deployment on a slope. SOURCE: GSE Lining Technology, Inc.

Geotextiles: Woven and nonwoven fabrics used for physical protection (e.g., as a cushion to protect a geomembrane from puncture) and as filters, separators, and occasionally drainage media. They can be overlapped with adjacent sheets or joined by sewing. Nonwoven geotextiles, probably the most common type used in waste containment systems, generally have a mass per unit area between 150 and 750 g/m^2 (Koerner, 2005), although lighter and heavier geotextiles are available. Common materials include polyethylene and polypropylene.

Nonwoven geotextiles of varying mass per unit area. SOURCE: International Geosynthetics Society, Mini Lecture Series, Lecture 15.

Geonets: Grids or webs of extruded high-density polyethylene strands that facilitates in-plane transmission of liquids and/or gases. They can be overlapped on adjacent sheets or connected using synthetic mechanical ties spaced at intervals of several feet. Geonets are generally 5 to 9 mm thick and are available in two-layer (bi-planar) or three-layer (tri-planar) configurations or as a drainage geocomposite with a nonwoven filter geotextile heat bonded to one or both sides of the geonet (Koerner, 2005).



(*Left*) Bi-planar geonet; (*middle*) tri-planar geonet; (*right*) geocomposite drainage net. SOURCE: Tenax Corporation (left and middle); GSE Lining Technology, Inc. (right).

Geosynthetic Clay Liner: Granulated bentonite (containing sodium montmorillonite) that is most commonly either encapsulated between two geotextiles or glued to a carrier geomembrane. The bentonite layer is typically about 6 mm thick and has a mass per unit area of 3.6 to 4.3 kg/m^2 and a gravimetric water content of 10 to 20 percent at the time of manufacture (Koerner, 2005). When the bentonite is sandwiched between two geotextiles, at least one is a nonwoven fabric and the geosynthetic clay liner is generally reinforced by needle punching, stitch bonding, or heat welding the nonwoven fibers into the fabric on the other side.

Needle-punch reinforced geosynthetic clay liner.

are frequently placed together to form a composite liner. Soil liners, often referred to as compacted clay liners, are generally made from fine-grained silts and clays or admixtures of coarse-grained soils with bentonite. The soil is compacted in 150-mm-thick lifts with a kneading action to form layers that are typically between 0.3 and 0.9 m thick. Alternatives to low-permeability soil liners include geosynthetic clay liners and Portland cement concrete or asphalt cement concrete, in which a granular soil aggregate is bound with cement or asphalt into a low-permeability layer that is 0.15 to 0.3 m thick. Portland cement concrete layers are commonly used to store hazardous and radioactive wastes until the radioactivity has decayed to levels that meet waste disposal criteria (10 CFR 61), prior to transport to permanent waste disposal sites. Asphaltic cement concrete layers have been used in cover and liner systems for long-term containment of radioactive wastes and in more conventional landfills.

Resistive barrier components in vertical walls include soil bentonite, cement bentonite, soil-cement-bentonite, steel or polymeric sheet piles, and geomembrane panels. These materials may be placed in a trench in a continuous mass or in interlocking panels or overlapping columns. The depth of installation depends on the type of vertical wall and the method of installation (EPA, 1998): a maximum of approximately 30 m for jet-grouted and soil-cement column walls, 25 m for soil-bentonite and cement-bentonite walls excavated with a backhoe,[1] and 15 m for geomembrane panel and sheet pile walls.

Blanket drainage systems, including granular and geosynthetic drainage layers and horizontal and vertical wells,

[1]Much greater depths can be achieved with cable-hung or kelly-mounted grabs. For these depths, walls will be excavated in panels and backfilled with cement-bentonite-aggregate concrete.

are typically employed for liquid and gas removal (i.e., extractive control) and injection (i.e., for advective control). Capacitive barrier components are generally simple soil layers, sometimes enhanced by the addition of reactive or sorptive substances. Barrier components often include layers to protect against (1) mechanical distress or intrusion, such as puncture or tearing of resistive barriers; (2) erosion of final covers; and (3) clogging or mechanical penetration and disruption of drainage systems. Typical protective layers include soil, cobbles, or select waste mechanical buffers, and graded soil or geotextile filters and cushions.

2.2.4 Typical Engineered Barrier System Configurations for Landfills

Regulations often dictate the configuration of engineered barrier systems. Configurations that follow prescriptive minimum standards for bottom barriers and cover systems are illustrated in Figures 2.2 and 2.3, respectively. Figure 2.2a illustrates a bottom barrier system for municipal solid waste (MSW) landfills. The minimum prescriptive barrier system for an MSW landfill is a composite liner system composed of a 1- to 2-mm-thick geomembrane underlain by a 0.6-m-thick low-permeability soil layer and overlain by a 0.3-m-thick granular drainage layer with a minimum saturated hydraulic conductivity of 1×10^{-4} m/s. This layer comprises the leachate collection and removal system and is used to collect and remove the leachate from above the composite liner system. Barrier components that provide equivalent environmental protection to the prescribed components may be employed in practice. These include substitution of geosynthetic clay liners for low-permeability soil layers and substitution of geosynthetic drainage layers for granular drainage layers. Some existing MSW landfills have single-barrier layers with extraction (drainage) layers on top, but due to their potential for higher leakage potential in comparison with composite liners, they are not generally allowed for new construction. Some jurisdictions require double-liner systems for MSW landfills, with two separate resistive barrier layers overlain by extraction layers.

Figure 2.2b shows a bottom barrier system for hazardous waste landfills. The prescriptive minimum standard for the barrier system is generally a double liner with a single geomembrane primary liner over a drainage layer, over a geomembrane and low-permeability soil composite secondary liner system. A double composite liner system (i.e., a barrier system with two stacked composite liners) is used in many hazardous landfills to provide better protection against leakage than is achievable with a single composite liner. The extraction layer of the underlying composite liner is referred to as the leak detection system because monitoring of this layer provides a quantitative measurement of the leakage through the primary (upper) liner system.

Figure 2.3a shows a final cover system for MSW landfills according to RCRA Subtitle D. The prescriptive minimum barrier is a single 0.45-m-thick low-permeability soil barrier layer with a saturated hydraulic conductivity no greater than 1×10^{-5} cm/s, overlain by at least 0.15 m of vegetated cover soil for protection and underlain by a prepared foundation layer. However, Subtitle D also requires that the permeability of the final cover be less than that of any engineered bottom barrier or natural geological formation. This requirement is generally implemented by placing a geomembrane on top of the low-permeability soil layer in the cover of any landfill that has a geomembrane in the base liner system. Typical alternatives to the minimum prescriptive cover systems for MSW landfills include a 0.3-m-thick low-permeability soil barrier layer with a saturated hydraulic conductivity no greater than 1×10^{-6} cm/s (the California prescriptive minimum) or a geosynthetic clay liner or geomembrane used in lieu of the low-permeability soil layer. A drainage layer is often placed on top of the barrier layer and the overlying protection layer is frequently 0.3 m thick or more. In addition to these resistive barrier alternatives, capacitive evapotranspirative

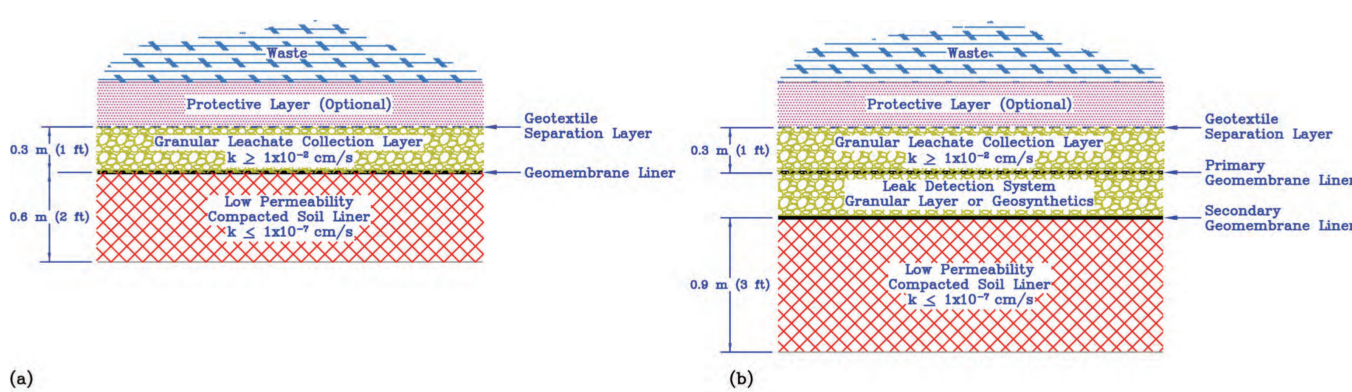


FIGURE 2.2 Prescriptive minimum bottom barrier system for (a) municipal solid waste landfills and (b) hazardous waste landfills under RCRA regulations in 40 CFR §264 and §258, respectively. The protective layer between the waste and the geotextile separation layer is typical but is not part of the prescriptive standard.

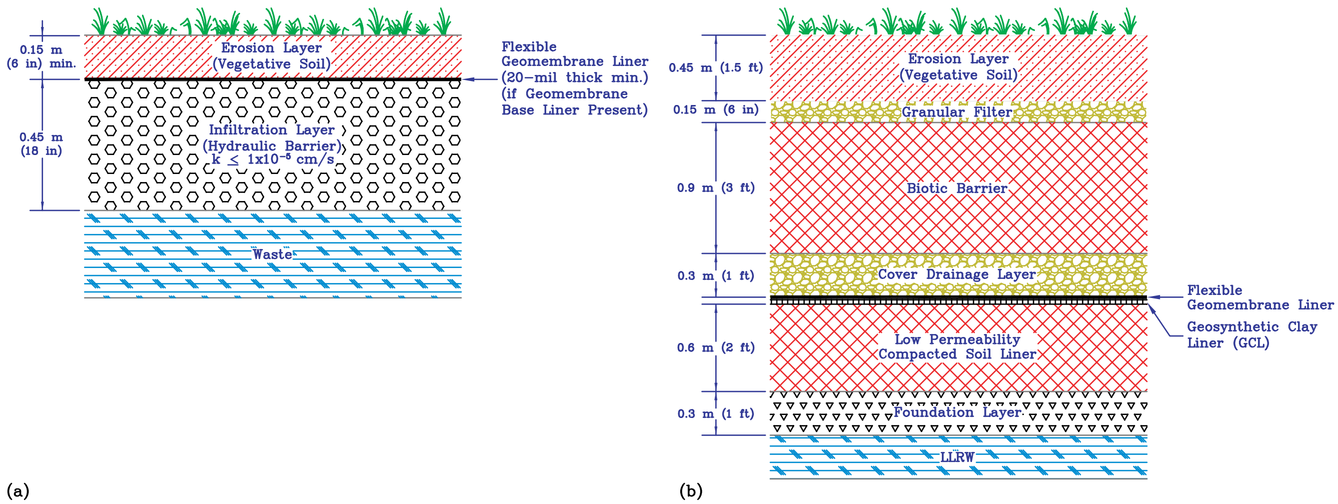


FIGURE 2.3 Prescriptive minimum final cover system for (a) municipal solid waste landfills and (b) hazardous waste and low-level radioactive waste landfills under RCRA 40 CFR §258.

final covers are increasingly being used as alternative cover systems in locations with arid and semiarid climates, where the performance of low-permeability soil barrier layers is in doubt because of the potential for desiccation cracking.

Figure 2.3b illustrates a final cover system for hazardous waste and low-level radioactive waste landfills. To limit infiltration to the greatest possible extent, a composite barrier is employed that generally consists of a geomembrane underlain by a 0.6-m-thick low-permeability soil layer with a saturated hydraulic conductivity no greater than 1×10^{-6} cm/s and overlain by a 0.3-m-thick granular drainage layer. A biotic barrier—a layer of rock or other mechanically resistant material—may be used to prevent inadvertent human and animal intrusion. The cover systems of low-level radioactive waste landfills may include thicker foundation and protective soil layers than those used in hazardous waste landfills.

The prescriptive final cover system for a site regulated under RCRA Subtitle C is a composite cover barrier with a biotic barrier, similar to that shown in Figure 2.3b. This type of final cover is sometimes referred to as a RCRA cap. RCRA caps may be used in conjunction with lateral barriers such as vertical barrier walls and extraction or injection trenches or wells to control lateral migration of contaminated groundwater or volatilized contaminants or lateral infiltration of groundwater. Although many uncontrolled waste sites are remediated under CERCLA or state-run programs and require a feasibility study to establish an appropriate cover system, RCRA caps are frequently employed at these sites.

2.2.5 Alternatives to Prescriptive Requirements

Prescriptive designs provide a good initial basis for landfill design and work well in many situations. However, landfills and other containment systems logically should be designed on a site-specific basis, taking into account the landfill size and nature of the waste, the operating environment (e.g., different designs might be needed for bioreactor landfills and conventional landfills; Rowe, 2005), and the local climate and hydrogeology. Alternative designs that require less engineering than prescriptive designs can be justified in certain circumstances. Under other circumstances, the prescriptive design may not be adequate and a higher level of engineering is required to provide adequate long-term environmental protection. It should be noted that simply meeting prescriptive standards does not relieve the design engineers of legal liability if the prescriptive design proves not to provide adequate environmental protection (see Estrin and Rowe, 1995, 1997, for a discussion of some of the legal issues related to design).

2.3 LIFE CYCLE OF A LANDFILL

Landfills are typically developed as a set of contiguous cells over a period of one or more decades. The life cycle of a cell begins with the initial construction of its base barrier system. Good construction quality control and quality assurance are critical at this stage to minimize subsequent leakage through the barrier system. The three stages following construction of the barrier system are (1) the initial period, (2) the active period, and (3) the postclosure period. During the first few months of landfilling (Stage 1), there is generally not enough waste in a cell to significantly control the flow of rainwater or snowmelt into the leachate collection system. The leachate collection system flow rates can be quite high (relative to later stages) after significant rainfall events. During the active period, waste is placed and covered with daily and intermediate layers of soil. Precipitation percolates through the waste and cover soils, which absorb and release some of the moisture at slower rates than the infiltration rate. Consequently, flow rates in the leachate collection system(s) decrease as the cells fill and eventually stabilize. At closure of the cell, a cover is placed, which will substantially reduce the percolation of moisture into the waste and further dampen the effects of rainfall events. As a result, flow rates in the leachate collection system(s) decline further.

These three stages are common to all landfills. Municipal solid waste landfills also include stages related to the decomposition of waste and biological processes in the leachate collection system. Hence, leachate composition varies with time. Periods of heavy precipitation can dilute the leachate, especially after new sections of the leachate collection system have been constructed. Transport of leachate in a waste mass can be affected by settlement of wastes caused by decomposition and mechanical stress. The residence time of leachate in the waste mass can increase and flow of leachate into a collection system can decrease due to increased waste density.

The processes that occur in municipal landfills are reasonably well understood (e.g., Reinhart and Townsend, 1998). Concentrations of inorganic contaminants (e.g., chloride), largely unaffected by biological and chemical interactions, increase in the leachate over periods of up to several decades (Rowe, 2005) but eventually level off and then decrease because of dilution. Contaminants like calcium and volatile fatty acids reach a peak concentration in the leachate much earlier than chloride and then decrease because of waste biodegradation processes and biological activity in the leachate collection system (Rowe, 2005). Biodegradation of volatile fatty acids produces landfill gases (mostly methane and carbon dioxide), increases the pH of the leachate, and causes a number of inorganic contaminants (e.g., calcium, heavy metals) to precipitate (Rittman et al., 1996; Jefferis and Bath, 1999). In addition, when leachate enters a leachate collection system that is at or near atmospheric pressure, there may also be a substantial decrease in pCO_2, which can lead to additional calcite precipitation in the collection system. The precipitates can clog leachate collection systems and also substantially reduce the concentrations observed in leachate pumped out of the landfill. Thus, the processes that occur in the leachate collection system have both advantages and disadvantages. On the negative side, they cause clogging, which reduces the hydraulic conductivity and hence, potentially, the flow. On the positive side, they treat ("clean up") the leachate by immobilizing a significant fraction of ions like calcium and some heavy metals and by reducing the concentrations of organic contaminants (e.g., volatile organic compounds, volatile fatty acids) in the collected leachate. As a consequence, the leachate that is collected is not necessarily representative of what may leak through liners. The concentration of contaminants available to leak or diffuse through the liner may be higher than what is observed in the collected leachate (Rowe, 2005).

Degradation of organic waste in MSW landfills generates gas and heat, which can increase the temperature both on the base liner and in the cover and adversely affect the barrier components (Rowe, 2005; Yesiller et al., 2005). Aerobic degradation of organic wastes produces primarily carbon dioxide, while anaerobic degradation produces 40 to 45 percent methane and traces of other volatile and semivolatile organic compounds. Aerobic degradation generates water, while anaerobic degradation consumes water. The gas generated by anaerobic degradation contains roughly similar amounts of methane and carbon dioxide, minor amounts of nitrogen and a few other compounds, and trace amounts of volatile organic compounds. Semivolatile compounds are not easily volatilized and thus are not present in meaningful concentrations. The gas may condense and form a contaminated liquid (e.g., an acidic liquid with small amounts of volatile organic compounds). Some can also dissolve in leachate. Waste placed in an MSW landfill generally degrades aerobically until the oxygen trapped in the pores is consumed; then anaerobic degradation begins. However, improperly operated landfill gas systems and poor daily cover practices can lead to the introduction of additional oxygen to the waste and sustained aerobic degradation. Sustained aerobic degradation produces higher temperatures than anaerobic degradation. Gas generation in MSW landfills, whether aerobic or anaerobic, generally increases steadily during the active life of the landfill, peaks at or near the time when the last waste is placed in the landfill, and then decreases with time during the closure period. How long significant quantities of gas are generated depends to a large extent on the local climate and operational practices. With sufficient liquid, anaerobic degradation may be essentially complete in decades. In an arid climate, on the other hand, anaerobic degradation may continue at a slow rate for hundreds of years. Therefore, a final cover that limits infiltration can actually extend the period over which decomposition processes are active in MSW landfills.

Occasionally, water or oxygen is intentionally introduced during the active life of MSW landfills to accelerate anaerobic or aerobic degradation, respectively. One of the objectives of these bioreactor landfills is to reduce the period during which the waste remains active by accelerating stabilization of the waste and reducing potential long-term environmental liability.[2] However, bioreactor landfills remain an experimental technology because many issues have not yet been resolved (e.g., the effect of extra heat on the service life of the liner, the effect of additional heat and leachate on the long-term performance of the leachate collection system).

MSW landfill temperatures typically increase gradually over time, then peak and ultimately decrease as degradation tapers off. The time-temperature history is linked to biological activity in the waste and hence in most cases to gas production. Both gas production and temperature can be increased by addition of moisture to the waste. Although organic matter may be degraded relatively quickly,

[2] Technical information on bioreactor facilities can be found on websites such as <http://www.epa.gov/epaoswer/non-hw/municipal/landfill/bioreactors.htm>, <http://www.wm.com/WM/environmental/Bioreactor/index.asp>, and <http://www.itrcweb.org>.

contaminants will likely continue to be released for decades to centuries because of their slow release through garbage bags and the slow degradation of waste such as cellulose. In addition, time-dependent sorption/desorption of contaminants from low-permeability soil layers in barriers may affect the release of contaminants to the environment. Overall, the period during which a large landfill will potentially release contaminants at unacceptably high levels may be on the order of hundreds of years for municipal solid wastes (Rowe et al., 2004). The threat can be mitigated through long-term cap maintenance to minimize the migration of liquid into the landfill. In the case of radioactive wastes, it may take hundreds to thousands of years (e.g., mill tailings, see NRC, 2002) before decay reduces radiation to preemplacement levels. Low-level radioactive waste landfills (e.g., Fernald, Ohio) are thus designed for a very long lifetime (i.e., 200 to 1,000 years; 40 CFR §192.02).

Hazardous waste landfills are generally not subject to the same long-term biological and chemical processes as MSW landfills because hazardous waste is generally stabilized chemically prior to disposal. However, the life span of a hazardous waste landfill is generally assumed to be on the order of a hundred years or more. Nonengineered hazardous waste dumps and contaminated soil and groundwater sites subject to corrective action may have life spans on the order of tens to hundreds of years, depending on the source and nature of the contaminants. Low-level radioactive waste landfills may have life spans on the order of hundreds of years up to a thousand years, depending on the rate of radioactive decay (GAO, 2005; NRC, 2006).

3

Monitoring Barrier Performance

Monitoring is an essential component of engineered barrier system design and operation. Preconstruction monitoring is required to develop a conceptual site model for barrier system design and analysis, to establish a baseline for evaluating the effectiveness of the engineered barrier system, and, in the case of a barrier system for preexisting contamination, to establish boundary conditions and geometric constraints for barrier system design. Postconstruction (long-term) monitoring is critical to ensure that barrier integrity is sound and that contaminants are not inadvertently released into the environment. Monitoring systems may observe both the physical conditions of the barrier and subgrade and the chemical environment surrounding the barriers. Information from monitoring of existing waste containment systems provides the basis for many of this report's conclusions on the long-term performance of engineered barriers.

This chapter summarizes statutory requirements for monitoring barrier system performance and reviews techniques that can be used to monitor the integrity of engineered barrier systems and their components.

3.1 STATUTORY REQUIREMENTS FOR MONITORING

Statutory requirements for monitoring systems are prescribed in accordance with the regulatory classification of the waste. Thus, monitoring requirements depend on whether the waste contained by the barrier system is regulated under the Resource Conservation and Recovery Act (RCRA; Subtitles C and D); the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA); the Uranium Mill Tailings Remedial Action (UMTRA); the Low Level Waste Policy Act; or another regulatory program. A common element among almost all statutory monitoring programs is an initial 30-year postclosure monitoring period. Another commonality is that regulatory programs may be delegated to state governments and tribal authorities with regulatory programs that conform to the minimum federal requirements. Environmental Protection Agency (EPA)-approved states and tribal authorities may have monitoring requirements that exceed the federal minimum standards. In approved states, both state and local governments are generally involved in overseeing monitoring programs.

RCRA Subtitles C and D prescribe minimum standards for monitoring hazardous waste treatment, storage, and disposal facilities and municipal solid waste (MSW) landfills, respectively. These standards require owners and operators to monitor and maintain activities to preserve the integrity of the disposal system. These responsibilities are governed by closure and postclosure monitoring plans certified by the EPA regional administrator or the director of an approved state or tribal authority. Monitoring plans describe procedures for obtaining the data necessary to maintain the integrity of the final closure, to maintain the operating leachate collection and leak detection systems (and gas monitoring system, if applicable), and to monitor groundwater quality. Financial assurance requirements are based on "projected costs for an entire post-closure period of thirty years" (EPA, 2003b). At the end of the initial 30-year postclosure period, monitoring and maintenance may have to continue if the lead regulatory agency determines that the waste still poses a threat to human health or the environment. In addition, Subtitle D allows the director of an approved state or tribal authority to increase or decrease the 30-year postclosure period if the owner/operator demonstrates that a different period is needed to protect human health and safety. However, there are no financial assurance requirements for monitoring beyond 30 years.

Engineered barriers constructed under RCRA corrective actions are implemented either as part of a permit or through an order of consent. Postclosure monitoring requirements are similar to those for CERCLA sites (described below). Another class of RCRA engineered barriers are those constructed for "past practices units," which are "closed" pre-RCRA facilities. These barriers are subject to separate monitoring requirements developed through a consent order.

CERCLA requires postclosure monitoring for 30 years or as long as the waste poses a threat to human health and the environment. CERCLA requirements include performance evaluations of the remedy, including monitoring systems, at 5-year intervals. The postclosure monitoring and evaluation periods can be modified if the closed CERCLA site is redeveloped for beneficial use following closure (e.g., development of a contaminated, or brownfield, site). In some cases, disposal cells are being built at these sites that comply with both CERCLA and RCRA regulations.

Closed low-level radioactive waste sites are subject to a 30-year observation and maintenance period, which may be shortened or lengthened, based on site-specific conditions, as described in 40 CFR §61.29. The disposal site must be "designed, used, operated, and closed to achieve long-term stability" (10 CFR §61.44). When closed, the licensee is responsible for postoperational surveillance and must maintain a monitoring system based on the operating history of the site.

UMTRA monitoring requirements appear to be tied to final ownership of the land, either the state or the Department of Energy (DOE). A remediated site may be transferred to a state subject to access by federal authorities or to tribes, or it may remain under DOE ownership. The Secretary of Interior takes ownership of removed radiation waste (42 USC §7914-7916).

No RCRA facilities have reached a 30-year lifetime, so performance can only be judged on short and medium timescales. It is unknown if the regulatory requirements will be reduced or extended at the majority of sites, although indefinite maintenance and monitoring periods are anticipated for many sites. Given the discretion written into the regulations, these types of decisions will likely vary by state.

3.2 CONTAINMENT SITE MONITORING SYSTEMS

Monitoring systems at waste containment sites may target a variety of media, including soil, groundwater, surface water, and air. Monitoring systems are often designed for the sole purpose of meeting the statutory requirements discussed above and are rarely designed to directly monitor barrier performance. Ideally, a monitoring system would do both. A well-conceived monitoring system is configured

- to provide information needed to assess barrier system performance and physical state (e.g., degradation),
- to provide information to assess the state of the waste mass to understand the progress of waste decomposition and stabilization,
- to monitor places where model scenarios predict contaminants are most likely to be released,
- to detect contaminant migration along unanticipated pathways,
- to provide early warning of a contaminant release and thus facilitate corrective action before migrating contaminants adversely impact human health and/or the environment, and
- to provide information to determine facility maintenance and rehabilitation needs.

3.2.1 Methods for Monitoring

Monitoring system measurements may be made in situ or on samples recovered from monitoring wells or probes. Monitoring devices may take point, area, or volume measurements. Well points for groundwater, subsurface gas sampling probes, and piezometers for measuring hydraulic head are examples of point measurements. Area measurements include blanket drainage layers behind barriers (e.g., leak detection layers in double-liner systems, pan lysimeters beneath sumps) and some geophysical measurements (e.g., ground-penetrating radar, vertical seismic profiling, electrical resistivity tomography). Volume measurements, such as the electrical measurements of resistivity and conductivity (e.g., capacitance probes, time domain reflectometry) and other types of geophysical measurements (e.g., gamma and neutron probes), gauge the properties of a characteristic volume of soil. Table 3.1 identifies common monitoring methods for contaminant migration at waste containment sites. Geophysical techniques are included in the table and subsequent discussion, although their use in monitoring engineered barriers has been limited for reasons discussed below. Appendix B provides a more detailed list of typical metrics used in monitoring, how they are measured, and their use in monitoring containment system performance.

3.2.2 Saturated Zone Monitoring Systems

Saturated zone (groundwater) monitoring systems are the most commonly employed method to evaluate barrier performance. Both the hydraulic potential (phreatic surface, hydraulic head) and the groundwater chemical composition in the pore water recovered from saturated soil beneath the phreatic surface are measured. Fixed groundwater monitoring systems include direct measurements made with wells, piezometers, or plate lysimeters, and indirect measurements made with electrical and other geophysical measurements. Groundwater monitoring sometimes includes one-time measurements made on samples recovered from push-in probes (e.g., cone penetrometers, hydropunch).

Geophysical methods can be used to monitor groundwater, but they are rarely used in regulatory compliance monitoring systems because techniques have not yet been developed that provide sufficiently quantitative and reliable data. They may, however, be employed in evaluation monitoring programs or in investigations for the development of corrective action programs (e.g., Meju, 2006; Slater and Binley, 2006). Measurements of electrical conductivity or resistivity and electromagnetic potential are sometimes used to establish the extent of the saturated zone and they

TABLE 3.1 Common Methods for Monitoring Water, Soil, and Air Media

Method	Measurement	Advantages	Disadvantages
Saturated Zone			
Wells	Phreatic surface; water samples	Simple; accurate determination of hydraulic head; allows sampling, which can be combined with laboratory analysis	Invasive; risks cross-contamination
Piezometers	Hydraulic head or pressure; samples (some designs)	Accurate determination of hydraulic head; allows sampling, which can be combined with laboratory analysis	Invasive; risks cross-contamination
Plate lysimeters	Liquid flux; samples (some designs)	Can allow sampling and combined laboratory analysis; long-term monitoring; applicable to relatively large volumes with low hydraulic conductivity	Slow; cannot distinguish effects of liner consolidation from induced flow; may alter boundary conditions driving flow
Geophysical: direct-current resistivity/ induced polarization, electromagnetic induction, transient electromagnetics, radio frequency magnetotellurics	Moisture content; leak detection; changes in conductivity; chemical composition; hydrochemical parameters; temperature	Non- or minimally invasive; applicable to large volumes/areas; can provide an indication of barrier integrity and performance; one- to four-dimensional and autonomous monitoring possible	Nonuniqueness; natural ambiguity in using single technique but results much improved by using multiple techniques, affected by a variety of factors (e.g., mineralogy, grain size and its distribution, temperature); accuracy and resolution decrease with depth depending on survey geometry; results often mixed
Self-potential	Leak detection	Noninvasive; applicable to large volumes/areas; can provide information on redox processes	Source mechanism usually uncertain; interpretation mostly qualitative, although two- and three-dimensional inversion methods are now possible
Push technology	Soil stratigraphy; samples (some designs); hydraulic head, pressure, temperature; detection of some chemicals	Rapid; inexpensive; nearly continuous profiling	Invasive; risks cross-contamination
Surface Water			
Grab samples	Laboratory characterization of chemical composition	Easy; inexpensive; laboratory characterization possible	Discreet samples; requires care to ensure representative sampling; sample degradation issues; sample transportation/chain of custody
Vadose Zone			
Tensiometers	Soil suction	Permanent installation; simple; robust	Invasive; risks cross-contamination; vacuum gauge calibration; can develop air leaks; ceramic cup may plug; limited to –1 atm by cavitation
Gas monitoring probes; borehole and well headspace monitoring; passive landfill vents	Methane; oxygen; carbon dioxide; hydrocarbons; nonmethane organic compounds	Permanent or temporary installation; simple; robust; can identify migration pathways	Discrete samples; seasonally variable; does not include emissions from other sources (e.g., composting); not quantitative
Flux box	Methane; oxygen; carbon dioxide; hydrocarbons; nonmethane organic compounds	Quantitative evaluation of gas transport out of covers	Seasonably variable; may alter flow boundary conditions
Lysimeters	Liquid flux; samples (some designs)	Can allow sampling and combined laboratory analysis; long-term monitoring; applicable to large volumes with low hydraulic conductivity	Slow; cannot distinguish effects of liner consolidation from induced flow

TABLE 3.1 continued

Method	Measurement	Advantages	Disadvantages
Electrical (TDR, capacitance gauges) and thermal probes	Soil moisture content	Water content; soil suction; liquid flux	Invasive; variations in moisture content must be related to flux analytically; suction measurement requires site-specific calibration
Neutron-neutron probe	Soil moisture content	Water content; soil suction; liquid flux	Point measurement; requires site-specific calibration
Nuclear magnetic resonance	Soil moisture content; porosity		Difficult to implement in the field
Electromagnetic induction (frequency and time domain, surface and borehole); electrical resistivity tomography (surface and borehole)	Moisture content; unsaturated flow; changes in electrical conductivity; salinity	Long-term spatial and temporal monitoring; applicable to large volumes/ areas; indication of barrier integrity; noninvasive; one- to four-dimensional and autonomous monitoring possible	Globally averaged values; require contrasting properties (e.g., fluid resistivity) to detect change; accuracy and resolution decrease with depth (surface configuration); nonuniqueness of inversion methods and thus ambiguity in interpretation (however, this may be improved by use of multiple techniques)
Ground-penetrating radar (surface and borehole)	Volumetric water content	Rapid; indication of barrier integrity; noninvasive; four-dimensional and autonomous monitoring possible	Limited depth penetration in conductive subsurface media (surface configuration)
Self potential	Leak detection; fluid flow	Inexpensive; noninvasive; spatial-temporal measurements possible	Interpretation mostly qualitative, although two- and three-dimensional inversion methods are now possible
Air			
Gas sampling (discrete)	Total VOCs; hydrogen sulfide; sulfur dioxide	Easy; inexpensive; quantitative measurements	Labor intensive; discreet samples; seasonally variable; does not include emissions from other sources (e.g., composting)
Air quality monitoring	Total hydrocarbons; particulates	Can use continuously or semicontinuously; in situ quantitative with techniques such as Fourier transform infrared spectroscopy or ultraviolet spectroscopy; large area of measurement; laboratory quantitative with techniques such as flux chambers	Does not include emissions from other sources (e.g., composting)

NOTES: TDR = time domain reflectometry; VOCs = volatile organic compounds.
SOURCE: McNeill (1980), O'Donnell et al. (1995), Pellerin et al. (1998), EPA (1998, 2003a, 2004), Daily and Ramirez (2000), DOE (2001), Bonaparte et al. (2002), Reedy and Scanlon (2002), Slater and Binley (2003, 2006), Haas et al. (2004), and Daniels et al. (2005).

can also sometimes be related to concentrations of inorganic constituents and soil moisture content (Meju, 2006). Electromagnetic measurements made from the ground surface can use different frequencies to achieve different depths of penetration and/or sensitivities. Time-lapsed seismic reflection, ground-penetrating radar, or electrical resistivity data may be used to track water and gas movement in the subsurface. Acoustic monitoring can be used to locate leaks of significant size and other areas of concentrated subsurface flow. Fiber-optic sensors, which have been used to study dynamic hydrologic processes (Selker et al., 2006), offer the potential for monitoring temperature changes in landfills, although this technology may be too expensive for this application. Other geophysical techniques measure turbidity (measured optically) and photoluminescence, which can be correlated to the presence and concentration of certain organic chemicals. Finally, the emergence of autonomous monitoring systems enable time-lapsed imaging of dynamic processes.[1] These areas may be fruitful for future research.

3.2.3 Vadose Zone Monitoring Systems

Vadose zone monitoring systems measure hydraulic potential (soil suction), soil pore gas constituent concentrations, and the presence and chemical constituents of migrating

[1]See, for example, <http://geophysics.inel.gov/h2/hermes/pages/login.php>.

liquids above the phreatic surface. These techniques can also be used to monitor cover system performance and subsurface gas and leachate migration. Subsurface gas monitoring probes enable soil pore gas to be collected from a screened interval for subsequent compositional analysis. Head space (the space above the water surface in a well or borehole) monitoring can provide an indication of subsurface gas migration, either directly from the source (e.g., a landfill) or indirectly from off-gassing of groundwater contaminated by volatile and semivolatile organic compounds from the source. Atmospheric tracer tests using handheld instruments and flux chamber measurements can be used to determine the flux of emissions through the unsaturated soil in cover systems.

In situ moisture measurements can be made using electrical probes, including capacitance probes, which directly measure the electrical capacitance of a representative volume of soil; time domain reflectometry probes, which indirectly measure soil capacitance; and thermal probes, which measure the rate at which heat is dissipated by the soil. Capacitance, time domain reflectometry, and thermal probes must be calibrated using site-specific soil and may be sensitive to accumulation of salts and other changes in soil and pore water chemistry. Shallow electromagnetic techniques and ground-penetrating radar may be used to estimate soil moisture content and to track the movement of fluids through the vadose zone.

Lysimeters capture liquid migrating through the vadose zone and include suction lysimeters (sampling points from which migrating liquid is sucked from the soil), gypsum blocks (which absorb migrating liquid because of their affinity for water), and pan lysimeters (blanket drainage layers). Suction lysimeters and gypsum blocks capture relatively small samples of migrating liquid and can be bypassed by the migrating liquid if not properly configured. Pan lysimeters can capture large representative samples of migrating liquid for laboratory analysis. Pan lysimeters provide a direct measurement of percolation through the cover barrier element(s) subject to a combination of field and imposed boundary conditions. While some investigators maintain that pan lysimeter measurements are the best means available to assess cover system performance (Benson et al., 2001), other investigators have concerns about the impact of imposing a capillary break at the base of the cover and, for MSW landfills, the impact of obstructing heat and moisture flow from below on measured percolation. The net upward flow of heat and moisture in MSW landfills in arid and semiarid climates is discussed in Blight (2006), and the impact of this upward heat on lysimeter measurements is discussed in Kavazanjian et al. (2006a). Errors associated with lysimeters and other indirect methods to assess cover performance are discussed in Malusis and Benson (2006). Concerns about the impact of the capillary break at the base of pan lysimeters on the measured percolation are described in greater detail by Zornberg and McCartney (2005) and Kavazanjian et al. (2006a).

Indirect monitoring of vadose zone flux in cover systems can be carried out using moisture content measurements (Kavazanjian et al., 2006a). The primary limitation of this approach is that percolation must be calculated from laboratory measurements of the soil water characteristic curve, which relates moisture content to soil suction. Gee and Hillel (1988) suggest that the uncertainty associated with percolation calculated in this manner can lead to large uncertainties in the calculated liquid flux. The vadose zone is also monitored for gas transport of contaminants from waste sites. Generally, fixed gas monitoring probes placed at designated intervals are used to periodically collect vadose zone samples, although one-time soil gas probes (e.g., geoprobes) may be used in some situations. An active vadose zone gas monitoring program for a closed hazardous waste landfill site in California is described in Box 3.1.

3.2.4 Air Quality Monitoring

At sites where significant amounts of gas are generated, surface emissions sampling may be conducted using handheld instruments that measure the concentration of total volatile organic compounds or gases of concern (e.g., hydrogen sulfide, sulfur dioxide). A grid is generally laid out over the site, and sampling points within each grid square are chosen randomly. Where grid measurements are not feasible (e.g., on steep slopes), measurements taken with some overall minimum frequency and maximum spacing over a preestablished route may be employed. Air quality measurements may also be made at fixed sampling points to detect hydrocarbons, particulates, or other airborne substances. Other gas monitoring technologies include perfluorocarbon gas tracers and SEAtrace developed by Sandia National Laboratory, which measure the rate of migration of a gas tracer from the point of injection to a collection well (Sullivan et al., 1998). Gas tracers are injected on the inside of the barrier, and concentrations of perfluorocarbon gas tracers in the external monitoring wells are analyzed to determine whether there is a breach in the barrier (Pearlman, 1999). Finally, air quality monitoring is required at low-level nuclear waste disposal sites (e.g., see 40 CFR §192.02 for radon requirements). Tests for the presence of radioactive material in the air are conducted at both onsite and offsite locations.

3.2.5 Other Containment Monitoring Systems

Other monitoring systems used at landfills and contaminated soil and groundwater sites include surface water monitoring, deformation monitoring, and radioisotope monitoring systems. Surface water monitoring typically involves manually capturing samples of surface water runoff and streamflow at designated times and locations. In deformation monitoring, surface settlement is measured by survey or by photogrammetry methods. Radioisotope monitoring may be

BOX 3.1 Vadose Zone Monitoring for the McColl Superfund Site

This case history describes a vadose zone monitoring program for a closed hazardous waste site. The McColl Superfund site, located in Fullerton, California, was an 8.8-ha hazardous waste disposal site with 12 unlined pits, or sumps, containing approximately 60,150 m^3 of petroleum waste sludge generated from high-octane aviation fuel production. The waste sludge, disposed of during and just after Word War II, is highly acidic (pH < 1.0) and emits a strong, objectionable odor. During the 1950s and 1960s, three of the sumps were covered with diesel-oil-based drilling mud from petroleum production to control odors. In the late 1950s, six additional sumps were covered with soil. A golf course was subsequently developed over a portion of the site. Residential neighborhoods around the golf course followed in the 1960s. The site was initially brought to the attention of public health regulatory agencies as a result of odor and health complaints from nearby residents beginning in July 1978. By September 1982, EPA had added the site to its National Priorities List. Shortly after, the golf course owner abandoned a three-hole portion of the golf course due to waste "seeps" in the fairways.

The McColl site was remediated between May 1996 and August 1998 under a CERCLA consent degree. The components of the barrier system installed as part of site remediation are shown in Figure 3.1 and include:

- a RCRA-compliant multilayer geomembrane-geosynthetic clay liner cap placed over the sumps to control infiltration and the release of hazardous emissions,
- a gas collection system beneath the cap connected to an activated carbon absorption gas treatment system, and
- a soil-bentonite vertical barrier placed around the sumps and tied into the cap barrier layer to control inward migration of groundwater and outward migration of gasses.

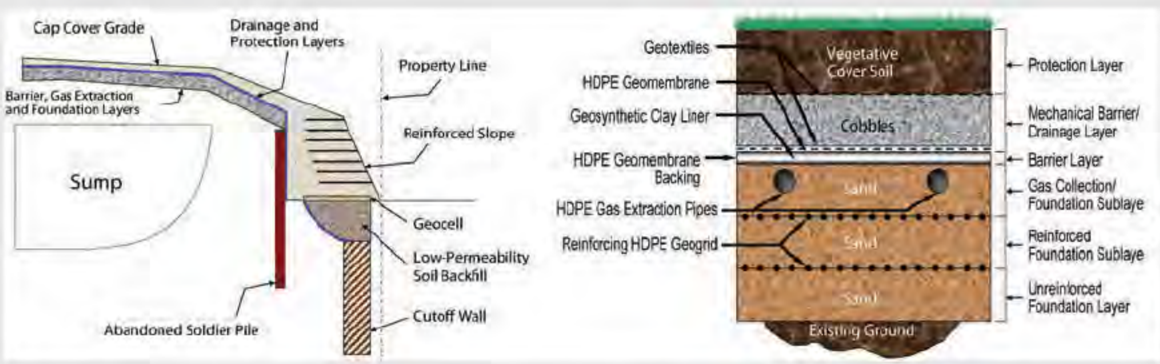


FIGURE 3.1 (*Left*) Tie-in between the cap and vertical barrier in the area immediately adjacent to the homes, where a reinforced earth berm was required. (*Right*) Detail of the cap section over the Los Coyotes sumps (the golf course area), where a bio-intrusion barrier and geogrid reinforcement were incorporated into the cap. NOTE: HDPE = high density polyethylene. SOURCE: Collins et al. (1998).

The primary function of the vertical barrier was to prevent lateral migration of volatile and semivolatile organic compounds and sulfur dioxide from the waste pits.

The monitoring system for the site included 12 pairs of vadose zone gas monitoring probes on either side of the barrier wall. The monitoring program also included flow rate and exhaust air quality monitoring for the carbon absorption unit that was connected to the gas collection layer beneath the composite cap shown in Figure 3.1; 20 groundwater monitoring wells; survey monuments on top of the cap; and visual inspections of the cap, surface water drainage system, subsurface drainage system outlets, gas collection system vents, groundwater monitoring wells, and perimeter fence at regular intervals ranging from daily to semiannually. Additional inspections are performed after extreme events, such as significant earthquakes (defined on the basis of magnitude and distance from the site) and major rainstorms.

The 12 sets of matched pairs of soil gas probes (pairs of probes set inside and outside the vertical barrier) were concentrated along the side of the site adjacent to residential development. Probes inside the barrier monitor soil pore gas pressure, and probes outside the barrier monitor the soil gas pore pressure and allow for sampling of the pore gas for analytical testing. The probe depths vary depending on the depth of the waste adjacent to the probes. The probes are monitored semiannually using handheld instruments for gas pressure, total volatile organic compounds, and sulfur dioxide. Differential gas pore pressure across the vertical barrier or detectable concentrations of volatile organic compounds or sulfur dioxide outside the wall trigger an evaluation monitoring program with likely additional sampling and testing. Since completion of the cap in November 1997, the evaluation monitoring program has not been triggered (i.e., there is no evidence of gas migration across the vertical barrier).

conducted in soil, ground- and surface water, and air at low-level radioactive waste facilities.

3.3 MONITORING OF BARRIER COMPONENTS

Engineered barrier components may be monitored for barrier integrity during and at the completion of construction (short-term monitoring) and for barrier performance after completion of construction (medium- and long-term monitoring). Monitoring of barrier integrity during and at the completion of construction generally involves direct measurement of barrier properties and performance. However, postconstruction monitoring of engineered barriers is usually based on monitoring of the environmental performance of the barrier system using the techniques discussed in Section 3.2. Direct monitoring of the integrity of barrier system elements in the medium and long term is relatively rare, although the use of embedded sensors, such as those being tested in the laboratory, pilot projects (Oh et al., 2003), and waste ponds (Frangos, 1997), may serve to facilitate more direct monitoring over the long term. Barrier components monitored in practice include compacted clay barriers, geomembrane barriers, leachate collection layers, vertical barriers, and geosynthetic clay liners.

3.3.1 Compacted Clay Barrier Monitoring

Direct monitoring of the short-term performance of compacted clay earthen barriers is based primarily on hydraulic conductivity measurements using in situ hydraulic conductivity tests (e.g., Daniel, 1989; Trautwein and Boutwell, 1994), such as sealed double-ring infiltrometer tests. These tests are routinely performed on separate but smaller prototypes of the barriers, typically referred to as “test pads,” which are constructed and evaluated prior to construction of the full-scale barrier as part of construction quality assurance (CQA; see Appendix C). Test pads are constructed using the same equipment and materials as the full-scale barrier layer and typically have the same thickness but shorter widths and lengths. The test pad may sometimes be incorporated into the full-scale barrier layer. Other construction criteria typically evaluated for the test pad include soil compaction criteria, such as gravimetric water content and dry density, lift thickness, and number of passes with the compaction equipment. The results of these tests are used to develop specifications and quality assurance requirements for construction of the full-scale barrier system. The CQA requirements (e.g., compaction data, laboratory tests on specimens recovered using Shelby tubes) provide an indirect assessment of the short-term integrity of the full-scale barrier system.

Direct postconstruction monitoring of compacted clay barrier layers is relatively rare. Indirect monitoring of barrier layers in cover systems often includes settlement monitoring because differential settlement is a major source of cracking and loss of integrity of clay barriers. Monitoring of gas concentrations at or near the ground surface also provides an indirect assessment of clay barrier integrity (i.e., of cracking) in covers of MSW landfills. Visual monitoring for cracks, ponded water after a storm (an indicator of nonuniform deformation), and distressed vegetation (and indicator of gas migration) may also provide an indirect assessment of clay barrier integrity in cover systems. Infrared and multispectral airborne and spaceborne monitoring of landfills where gas is being generated may also give an indirect assessment of cover barrier layer integrity, but these techniques have neither been investigated extensively nor employed in practice.

3.3.2 Geomembrane Barrier Monitoring

Current landfill construction practice includes extensive short-term monitoring of geomembrane integrity. During geomembrane installation, it is common practice to continuously observe installation, to nondestructively test all seams between geomembrane panels, and to periodically remove seam samples for destructive laboratory testing as part of CQA activities (see Appendix C). Furthermore, electrical leak detection methods are being used with increasing frequency to detect defects in geomembranes. These surveys are conducted either immediately after seaming (for covers) or following placement of the overlying leachate collection layer (for liners). These measures generally provide a high degree of confidence in the short-term integrity of a properly designed and constructed geomembrane barrier system. Analysis of 10 years of postconstruction leak detection surveys showed that a typical defect frequency rate for geomembrane liners constructed using strict CQA procedures was approximately 0.5 defects per hectare (Hruby and Barrie, 2003). This very low defect frequency corresponds to extremely high integrity for the geomembrane. (Furthermore, defects detected in these surveys are generally exposed and repaired, reducing the final postconstruction defect frequency to a minimal value.) In contrast, the average defect rate for geomembranes constructed without strict CQA was approximately 16 defects per hectare (Hruby and Barrie, 2003), approximately 30 times greater than for geomembranes with strict CQA, demonstrating both the importance and effectiveness of modern CQA procedures for geomembrane construction.

Direct postconstruction monitoring of geomembrane integrity is relatively rare. Because of limitations related to the thickness of soil or waste cover and the need for a conductive medium in the leak, electrical leak detection is generally useful as a CQA tool only for solid waste landfills. The primary measure of postconstruction integrity of geomembranes is measurement of the flow rate in the leak detection systems of double-liner systems. Measurements of volumetric seepage into the leak detection system provide a direct indication of the integrity (effectiveness) of the primary barrier system, which generally includes an overlying leachate collection layer and either a single geomembrane or a composite geomembrane/low-permeability soil barrier. In fact, the leak detection layer of a double-lined landfill provides continuous monitoring of advective flow through the primary liner system (assuming the leak detection layer is functioning properly). Leak detection system flows may contain not only advective leakage through the primary liner but also liquids from other sources, such as consolidation water from any compacted clay component of the primary liner and/or drainage of water that entered the system during construction (Bonaparte and Gross, 1990; Bonaparte et al., 2002). For landfills without double-liner systems, pan lysimeters are sometimes placed beneath the sump and in other strategic locations to monitor for leachate flow through the barrier system.

Leak detection layers and pan lysimeters enable samples to be collected for analytical testing of leachate constituents. This information can be useful in subsequent groundwater and vadose zone monitoring because it helps establish which constituents to measure. Although leak detection layers and pan lysimeters are advantageous for monitoring large areas of barriers, data collection is slow and it is difficult, if not impossible, to distinguish the effects of liner consolidation from induced flow in systems where the pan lysimeter underlies a compacted clay liner. Nonetheless, leak detection systems provide an effective means of monitoring the integrity of the overlying liner system. An example of how the leakage rate can be measured and interpreted to assess performance is provided in Box 3.2.

Temperature measurements are an essential requirement for evaluation of the potential long-term integrity of a geomembrane, since the service life of a geomembrane depends significantly on the temperature (Rowe, 2005). Furthermore, temperatures of multiple components can be used in heat and moisture transfer analysis. Time-temperature measurements are usually made on geosynthetic components of barrier systems or in the landfill mass itself for research purposes. Although temperatures have been measured in one landfill monitoring program (Rowe, 2005), routine monitoring of geomembrane temperatures is not yet a part of landfill engineering practice.

3.3.3 Leachate Collection Layer Monitoring

The performance of leachate collection layers is monitored in a variety of ways. Leachate head may be monitored in a collection sump or, in some cases, directly on the liner to evaluate the efficiency of the leachate collection and removal system. Measurements of the volumetric seepage of leachate into the collection and removal system can also be used to assess the efficiency of the system. These measures are somewhat indirect, as the leachate head and leachate volume depend on the leachate generation rate as well as the collection system efficiency. However, excessive head within the leachate collection layer and/or significant decreases in the leachate collection rate over time without any apparent external cause can indicate clogging of the leachate collection layer. Other methods that have been used to evaluate the performance of leachate collection layers include dye tracer tests, pumping tests, and video surveys. Dye tracer tests and pumping tests provide some indication of leachate collection layer continuity but may not give an overall assessment of the condition of the collection system. Video surveys generally indicate only the condition of the collection line being surveyed.

Chemical analysis of leachate samples is generally used to identify constituents of concern for vadose zone and groundwater monitoring beneath the liner. Although this approach identifies key organic or inorganic constituents, the potential for chemical transformation of leachate constituents between the leachate collection and removal system and the monitoring point must also be considered in establishing monitoring parameters. Chemical constituent concentrations in leachate may also be useful in evaluating the potential for diffusive transport across the liner and degradation of liner system components.

The leachate collection layer monitoring system and other monitoring systems used at a low-level radioactive waste repository are described in Box 3.3.

3.3.4 Vertical Barrier Monitoring

Vertical barriers for environmental protection commonly include slurry trenches, soil-mixed walls, and geomembranes dropped into trenches. Collection and extraction trenches may also be used as vertical barrier systems. Short-term monitoring of barrier effectiveness using a variety of CQA techniques, including sampling and testing of barrier materials and in situ testing of the barrier in slurry walls and soil cement walls (Appendix C), is standard practice. Although these methods generally do not ensure the same level of reliability as compacted clay layers or geomembranes, they can provide a high level of reliability for the short-term integrity of the as-constructed barrier. The long-term effectiveness of a vertical barrier is generally evaluated by monitoring the vadose and saturated zones down gradient of the barrier using the techniques described in Section 3.2. Physical sampling of soil-bentonite and soil-cement vertical barriers after construction is also possible but is generally done only after downstream monitoring indicates there may be a problem. The vertical barriers and monitoring systems for a large municipal solid waste landfill in California are described in Box 3.4.

Although geophysical methods (e.g., electrical resistivity, electromagnetic, acoustic) offer the promise of cost-effective, noninvasive, postconstruction evaluation of flaws in vertical barriers, only a few case studies exist. In one study, three-dimensional surface electrical imaging was able to resolve the geometry of 0.4-m-high vertical walls emplaced at a depth of 0.6 m, but image quality was too poor to resolve walls emplaced at depths greater than 1 m (Chambers et

BOX 3.2 Case History on Monitoring of a Double-Liner System

This case history illustrates how measurements from a leak detection system can be used to assess performance of the liner system. An example set of flow data from collection systems in a double-lined landfill in Lake Charles, Louisiana, is shown in Figure 3.2. The landfill cell extends below grade with side slopes graded at 2.5 horizontal to 1 vertical and a bottom sloped at 2.5 percent toward a sump area (Gilbert, 1993). The primary liner consists of a single high-density polyethylene geomembrane on the side slopes and a composite liner with a high-density polyethylene geomembrane overlying a compacted clay liner on the bottom. The cell covers an area of 1.3 ha, with the side slopes covering 36 percent and the bottom covering 64 percent of the area, respectively. The average annual rainfall is 1,530 mm, or 55,000 l/day over the total area of the landfill cell. The data in Figure 3.2 were obtained by measuring the volume of fluid pumped from the primary and the secondary collection system sumps on a monthly basis.

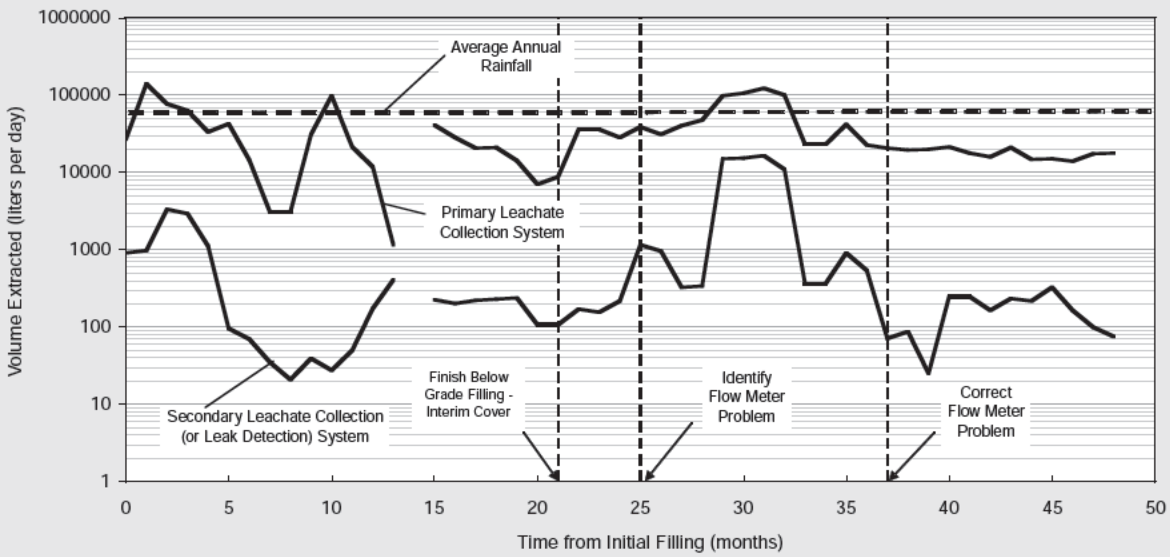


FIGURE 3.2 Field data from leachate collection and leak detection systems in a double-lined landfill. Waste placement began at time zero. SOURCE: Created from data provided by Chemical Waste Management, Inc.

The flow rate into the secondary collection system was initially high as the compacted clay liner in the primary liner system was consolidated from waste emplacement. Once the below-grade portion of the landfill cell was completed (21 months after the cell was opened), consolidation slowed and the flow rates into the primary and secondary collection systems became bette correlated with each other. This correlation allows estimation of the leakage through the primary liner, because as the flow rate into the primary collection system increases, the average hydraulic head in that system also increases. As a consequence, the leakage rate across the primary barrier layers is expected to increase. The data show that the flow rate through the primary liner is about 100 times smaller than the rate of flow into the overlying primary leachate collection system. Furthermore, the average leakage rate is less than 100 lphd (liter/ha/day) for the landfill cell, which is 1.3 ha in area.

While illustrating that small leakage rates through a liner can be achieved, this case history also demonstrates that the measured flow rate into the leak detection system should be considered carefully in assessing how the liner is actually performing. First, the leak detection system pumping data for the first 21 months during waste filling included consolidation flow as well as leakage. Second, reporting the data as a "leakage rate" per unit area is misleading in this case because the primary liner consists of a single geomembrane on the side slopes and a composite liner on the floor. The majority of the leakage is likely coming from the single geomembrane on the side slopes, which makes up less than one-half of the total area of the liner. Furthermore, the leachate head driving leakage can be significantly smaller on the side slopes versus the floor, and the leachate head may change with time even for a constant leachate generation rate due to clogging (see Box 4.4). Finally, the accuracy of the flowmeter system used to measure the volume of flow from both the primary and secondary sumps was flagged as questionable by the operator of the landfill during a 12-month period starting in month 25; the flowmeters were replaced in month 37 (Figure 3.2). Therefore, flow rates into leak detection systems provide at best an indirect measure of the performance of the entire primary liner system, including the drainage and barrier layers.

al., 2002). Electrical resistance tomography has been used to image the full-scale test emplacement of both thin-wall grout barriers and thick-wall polymer barriers (i.e., an approximately 1-m-thick wall constructed by injecting colloidal silica into fine- and coarse-grained sands; see Box 3.5; Daily and Ramirez, 2000). The thick-wall polymer barrier was readily imaged, whereas the thin-wall grout barrier could be resolved only by measuring the difference in resistivity before and after emplacement. This study also demonstrated that electrical resistance tomography can be an effective technique for assessing the performance of barriers.

Ground-penetrating radar surveys have been conducted to verify the integrity of thin-wall barriers (Pellerin et al., 1998) and the emplacement, location, and continuity of subsurface barriers (Rumer and Mitchell, 1995). Other geophysical techniques, such as seismic and acoustic methods, have been used to detect subsurface features such as a mud slurry trench, cement monoliths, and grout materials and to monitor the emplacement of a viscous liquid barrier and thin diaphragm wall (Pearlman, 1999).

However, with the exception of electrical leak detection in geomembranes, the use of geophysical methods to assess barrier integrity has not generally found its way into routine engineering practice. Consequently, few data exist on the long-term integrity of vertical barrier elements. Among the factors limiting the successful use of geophysics are small contrasts in the physical properties being measured, different geometries of the targets from those for which geophysical methods have been developed and used successfully in the past, lack of information about geophysical-barrier property relationships, instrument degradation over long time periods, and high costs. In addition, engineers and regulators may be unwilling to try what many believe to be unproven technologies. Improvements in several areas may help facilitate wider

BOX 3.3 Description of a Low-Level Radioactive Waste Landfill Monitoring System

The Fernald, Ohio, onsite disposal facility, completed in September 2006, represents a state-of-the-practice disposal facility for mixed wastes, including construction and demolition debris and low-level radioactive waste. The disposal facility covers 32 ha and contains 1.9 million m^3 of waste. The facility includes eight discrete cells underlain by a 1.8-m-thick double composite liner system and capped with a 2.9-m-thick composite final cover system (Figure 3.3). The final cover system includes a 0.9-m bio-intrusion layer and is designed to limit the release of radon-222 to the environment.

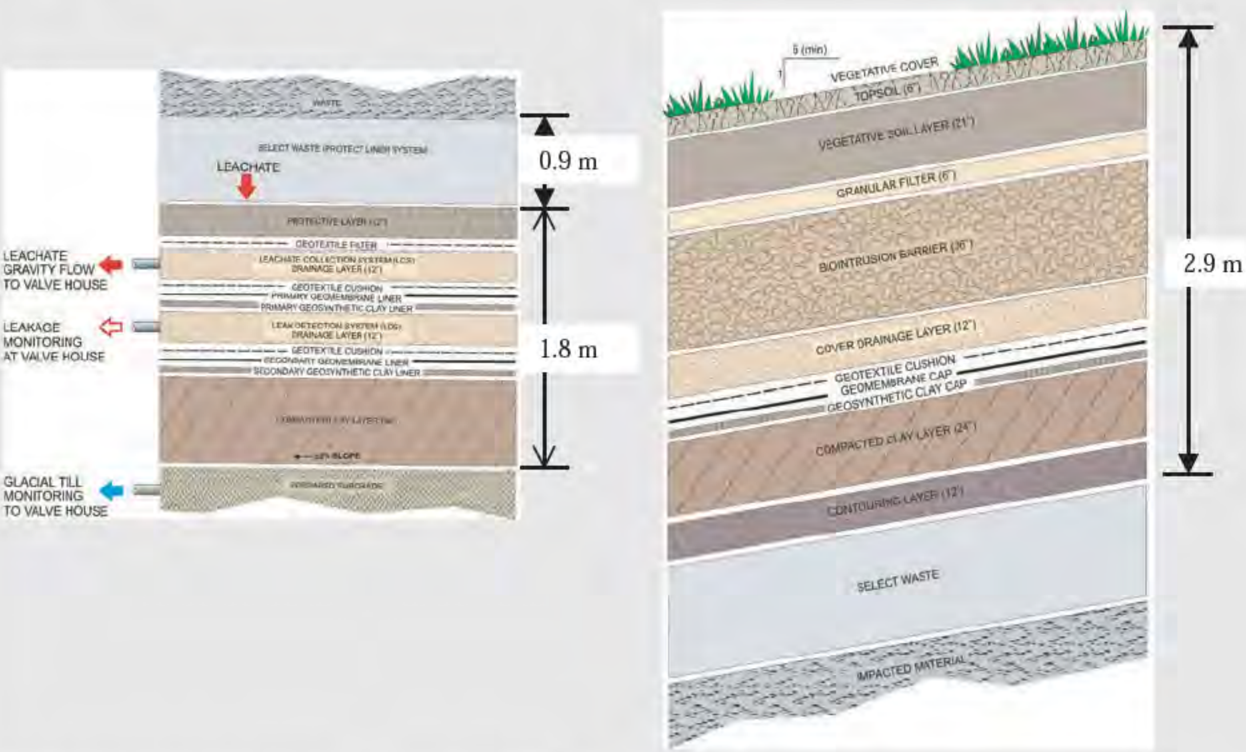


FIGURE 3.3 Fernald facility liner and cover cross sections. SOURCE: Geosyntec Consultants.

DOE design criteria called for the containment system to maintain its integrity for longer than 200 years and up to 1,000 years to the extent achievable. The cover system is designed with a low profile for "geomorphologic conformity" with the surrounding terrain (Figure 3.4).

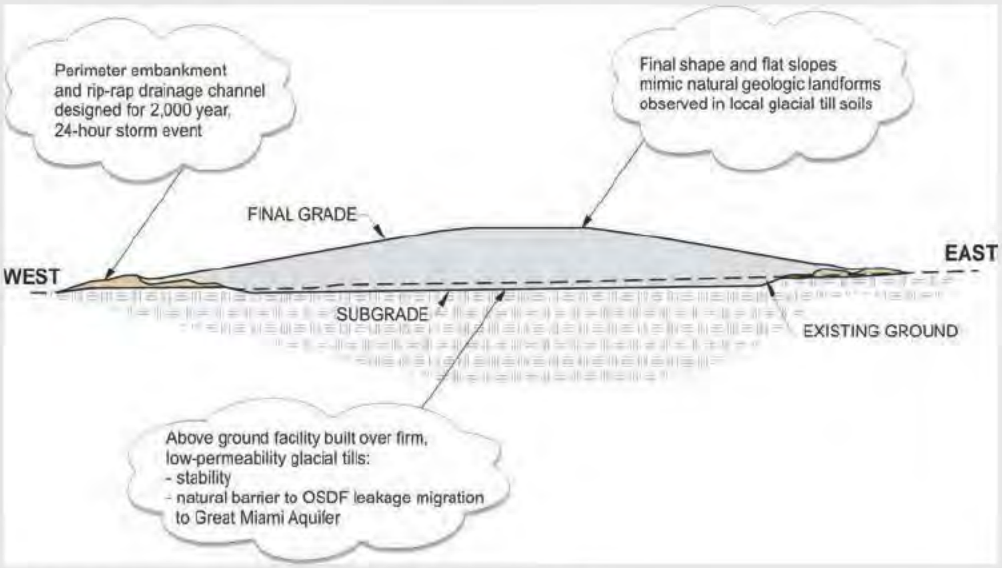


FIGURE 3.4 Fernald onsite disposal facility (OSDF). SOURCE: Geosyntec Consultants.

Short-term monitoring of the facility called for comprehensive CQA during containment system construction, including sealed double-ring infiltrometer testing for the compacted clay layers and electrical leak detection surveys following geomembrane installation. Medium- and long-term monitoring includes settlement monitoring of the cap, monitoring of the cap drainage layer, and independent monitoring of the leachate collection and leak detection systems for each of the eight cells. The monitoring system (Figure 3.5) also includes eight horizontal monitoring wells in the glacial till underlying the facility (one horizontal well beneath each unit). The leachate collection systems all include auxiliary removal pipe systems and all drainage systems include pipe cleanouts. During the construction period, liquids collected from the leachate collection and leak detection systems are transferred to an industrial wastewater treatment plant and scanned for radioactivity prior to discharge. In the postclosure period, the drainage layers in the liner and cover systems all drain by gravity to a constructed wetland (no pumping is required). The monitoring plan also calls for periodic inspection of the cap to detect and repair any erosional gullies and to remove any deep-rooted vegetation that could potentially penetrate the barrier layers in the cover.

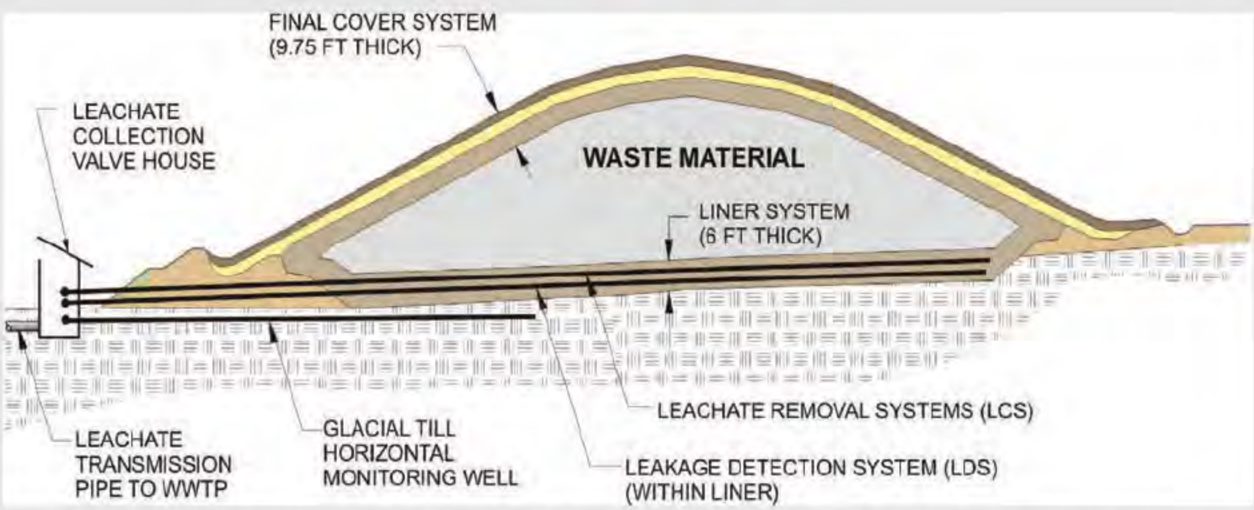


FIGURE 3.5 Fernald facility bottom liner system monitoring. NOTE: WWTP = wastewater treatment plant. SOURCE: Geosyntec Consultants.

BOX 3.4 Case History on the Puente Hills Landfill Canyons 1 and 3 Vertical Barriers

This case study illustrates the effective use of vertical barrier walls and an extraction system to control offsite migration of organic contamination. The Puente Hills Sanitary Landfill, owned and operated by the County Sanitation Districts of Los Angeles County (the Districts), is the largest operating municipal solid waste landfill in the United States, receiving approximately 12,000 metric tonnes of municipal solid waste per day, 6 days per week. The oldest disposal area of the Puente Hills Landfill facility (the Main Canyon area) is not underlain by an engineered liner system and contains over 150 m of municipal solid waste. While the bedrock underlying the Main Canyon has relatively low permeability, it is overlain by several alluvium-filled tributary canyons. Because groundwater monitoring indicated that these tributary canyons provided a conduit for transport of leachate-impacted groundwater offsite, the Districts constructed a system of vertical barriers and extraction wells to control leachate migration through the alluvium in the canyons. The vertical barriers were constructed of bentonite and Portland cement supplemented with fly ash to create a very low permeability barrier material.

Figure 3.6 shows the Canyon 3 barrier and monitoring system. Four extraction wells were placed in Canyon 3 on the landfill side of the cement-bentonite barrier wall and pumped to maintain an inward gradient (i.e., to draw water in the alluvium outside the landfill back toward the landfill and barrier). Monitoring wells on the opposite side of the barrier wall from the landfill monitor groundwater quality in both the alluvium and the low-permeability bedrock.

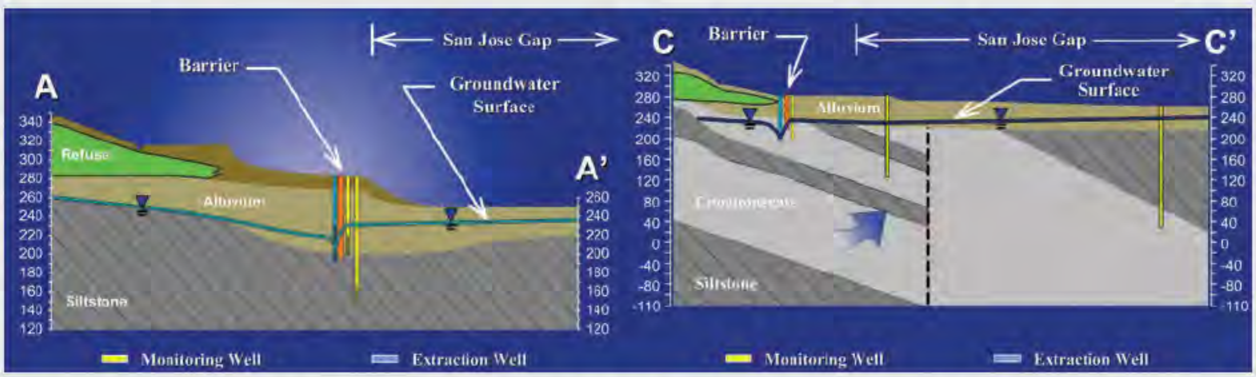


FIGURE 3.6 Cross sections through the vertical barrier systems at Canyons 3 and 1 at the Puente Hills landfill facility. SOURCE: County Sanitation Districts of Los Angeles County.

A similar strategy, but with 17 extraction wells, was employed in Canyon 1. A cross section through Canyon 1 in an area where the dipping bedrock includes more pervious strata interspersed among the low-permeability layers is shown in Figure 3.6. In this part of Canyon 1, the monitoring wells are placed in the pervious bedrock units outside the barrier wall.

Figure 3.7 shows the concentration of volatile organic compounds with time in the monitoring wells closest to, but outside, the vertical barriers in Canyons 3 and 1. The steady decrease in volatile organic compounds with time in both of these wells demonstrates the effectiveness of the vertical barrier and extraction well system in controlling the offsite migration of groundwater impacted by landfill leachate through the tributary canyons.

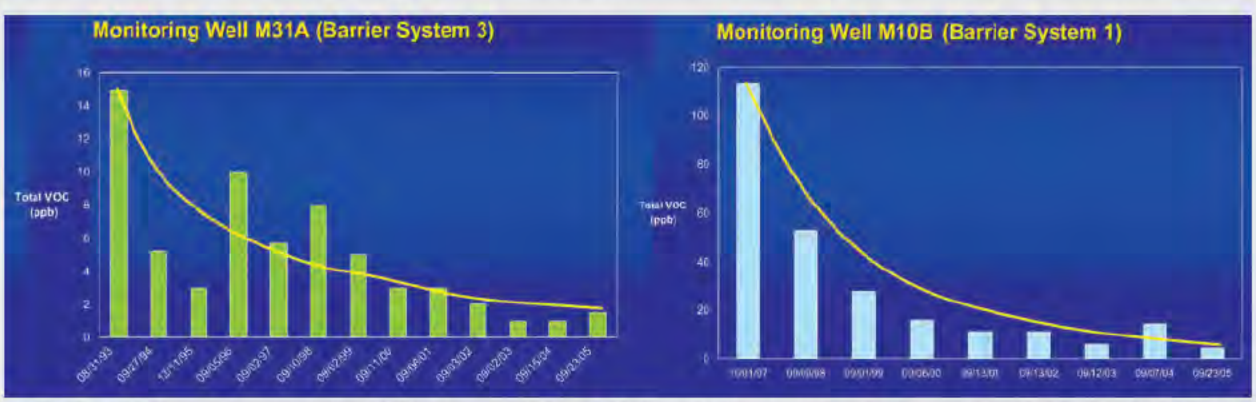


FIGURE 3.7 Volatile organic concentrations (VOCs) in groundwater outside the vertical barrier systems for Canyons 3 and 1 at the Puente Hills landfill facility. SOURCE: County Sanitation Districts of Los Angeles County.

BOX 3.5 Case History on Geophysical Imaging of Engineered Hydraulic Barriers

This case history illustrates the potential of using geophysical imaging techniques during emplacement of two prototype engineered barriers: a thin-wall grout barrier and a thick-wall polymer barrier. Electrical resistance tomography (ERT) was used to image the full-scale test emplacement of a thin-wall grout barrier installed by high-pressure jetting at Dover Air Force Base and a thick-wall polymer barrier installed by low-permeation injection at Brookhaven National Laboratory (Daily and Ramirez, 2000). ERT uses currents injected into the ground to image the electrical structure of the subsurface.

A plan view of the thin-wall grout barrier wall installation is shown in Figure 3.8a. The test site is underlain by medium to fine sands with gravel, silt, and clay lenses over 6 to 9 m of marine clays with thin laminations of silt. Electrical measurements were obtained before and after the installations. By comparing the two images it is possible to remove the heterogeneity of the natural background. The conductivity contrast between the emplaced materials, which are more conductive, and the native soils, which are more resistive, made the barriers excellent targets for electrical imaging. The electrical resistivity of the sands varied between 300 and 600 Ohm-m and the resistivity of the clay was <50 Ohm-m.

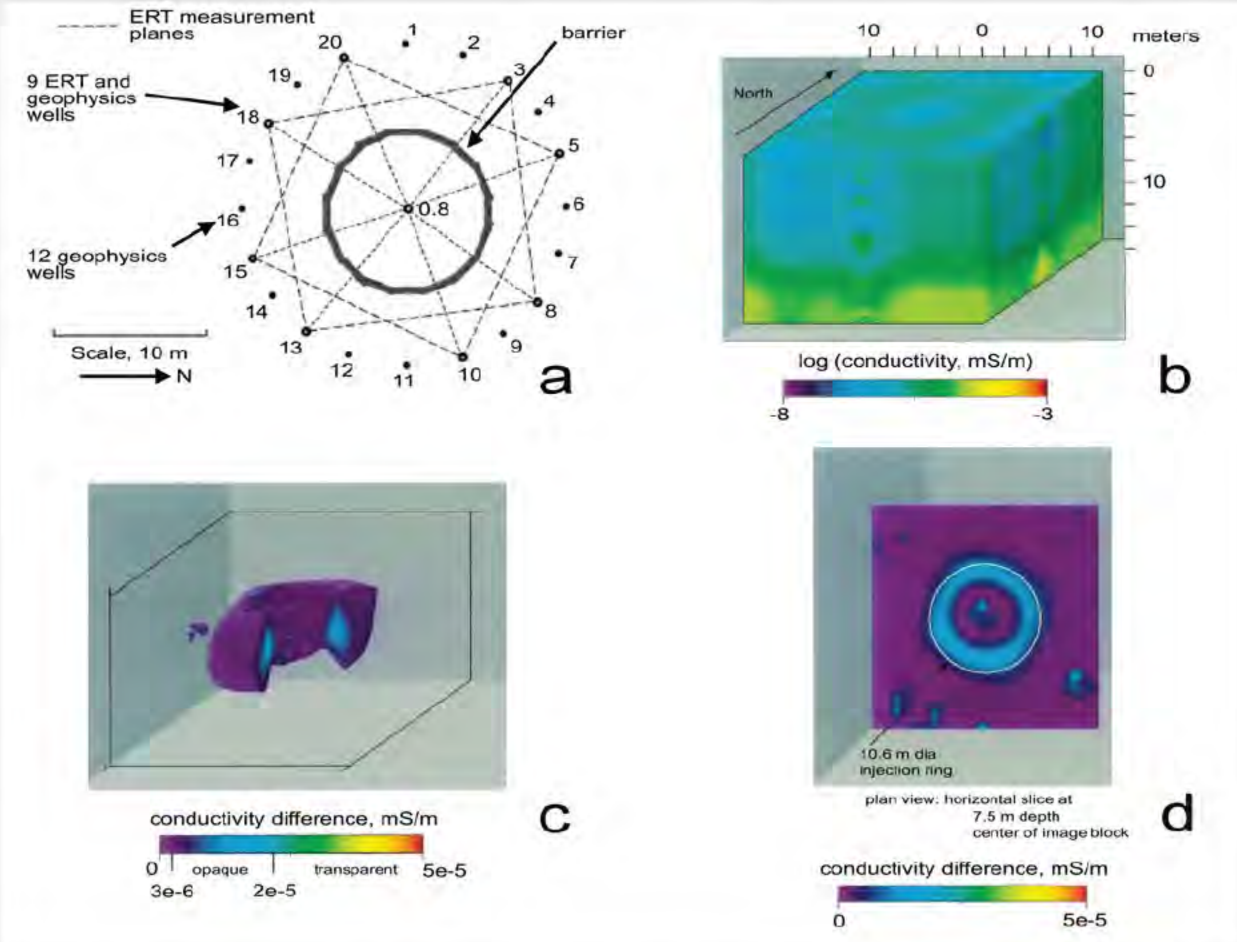


FIGURE 3.8 (a) Plan view of the thin-wall barrier showing locations of the ERT wells. The dashed lines indicate the hole pairs used to acquire the ERT data. (b) Three-dimensional image block of electrical conductivity before barrier emplacement. (c) Difference image block between the baseline and post emplacement image block with all boreholes and surface anomalies removed. (d) Plan view of the horizontal section of barrier at 7.5 m depth. SOURCE: Daily and Ramirez (2000).

The study found that the thin-wall grout barrier could only be imaged by obtaining the difference in resistivity between the background image obtained before installation and the image obtained after installation (Figure 3.8b-d). However, the viscous liquid barrier was electrically conductive compared to the background, so it could be imaged without having to subtract the background electrical resistivity structure (Figure 3.9).

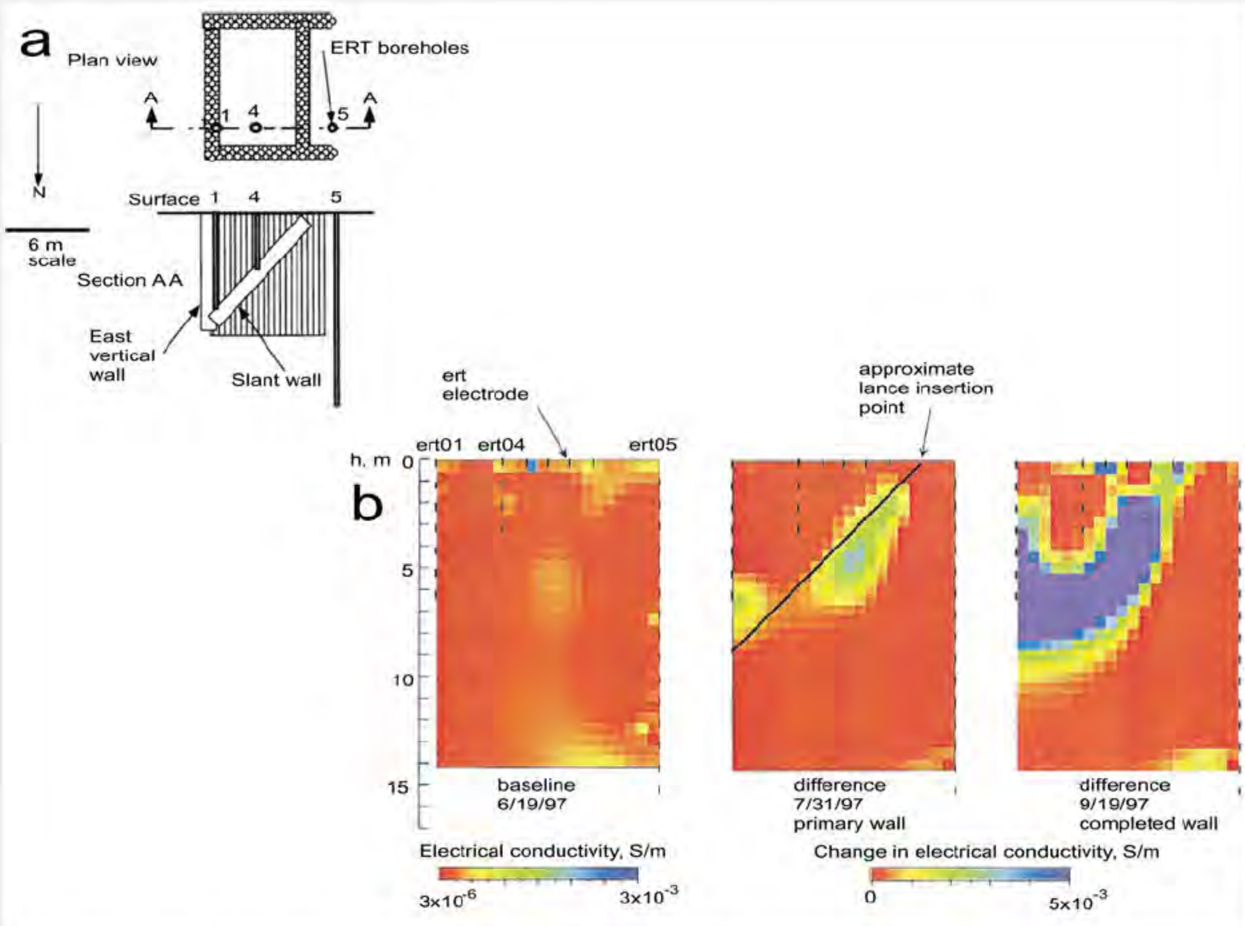


FIGURE 3.9 (a) Plan and schematic views of the viscous liquid barrier. (b) ERT two-dimensional section of the viscous liquid barrier. Left image is the baseline image taken on June 19; middle image is the difference taken on July 31 after the primary row was injected; right image is the difference taken on September 19 after all three rows were injected. SOURCE: Daily and Ramirez (2000).

ERT was also used to investigate the performance of another thin-wall barrier (Daily and Ramirez, 2000). Saltwater was used as an electrical tracer to determine the presence and location of any leakage flow paths. The authors were able to image electrical conductivity changes associated with defects in the wall or the joining of two walls.

The authors drew the following conclusions from these case studies. First, ERT can successfully image the location, size, and shape of subsurface barrier structures. Large defects can also be imaged with ERT, although resolution might be insufficient for imaging smaller defects. Second, ERT can be used to assess the performance of a barrier using gas tracers or water flooding. Third, ERT results can be substantially improved by obtaining differences between background resistivity structure before and after barrier emplacement. Fourth, the performance of ERT depends on the survey design, especially placement of the electrode wells. The best results come from placing electrodes in boreholes, rather than only on the surface.

and more successful uses of geophysics for monitoring of waste barrier systems, such as (1) better integration of geophysics into the design of monitoring plans; (2) improved methods for data collection, processing, and interpretation; (3) development of new computer codes; and (4) new instrumentation (Majer, 2006). For example, geophysical imaging of vertical barriers may be more effective if a background dataset is obtained prior to barrier emplacement (Slater and Binley, 2006). A background dataset would also be useful in liner monitoring systems. Any changes in subsurface physical properties caused by leaks could be detected through comparison of monitoring data and background data.

3.3.5 Geosynthetic Clay Liner Barrier Monitoring

Because geosynthetic clay liners (GCLs) are manufactured and thus undergo manufacturing quality assurance, their end-of-construction reliability tends to be significantly higher than that of compacted clay layers and is probably similar to that of geomembranes. GCLs are often used beneath relatively shallow depths (e.g., less than 1 m) of soil in cover systems. Because of serious performance concerns about GCLs buried under shallow depths of soil covers, GCLs have been exhumed and tested after several years of service to evaluate their integrity in the early medium term (e.g., Mansour, 2001; Henken-Mellies et al., 2002). However, this type of examination has been conducted only for research purposes and not as a routine part of barrier system monitoring.

Recently, exposure of the GCLs in several composite liner systems employing the GCL as the low-permeability soil layer beneath the geomembrane has shown that GCL seams can separate as a result of environmental effects (Thiel and Richardson, 2005). The GCL seams in these cases were generally exposed because of other performance concerns (e.g., during repair of mechanical defects to the overlying geomembrane). The accidental discovery of GCL seam separation indicates the value of direct monitoring of barrier components. Most barrier system components are hidden from view after construction, and thus component defects will not be identified until performance problems appear elsewhere in the system.

Geophysical techniques may be capable of detecting flaws in caps after construction (Slater and Binley, 2006). Moisture fluxes from cracks in the cap can be detected with electromagnetic induction methods and ground-penetrating radar (e.g., Ward and Gee, 2001). Caps could thus be monitored through long-term continuous measurements of near-surface moisture conditions (Meju, 2006). The development and deployment of permanent autonomous monitoring systems would facilitate such monitoring and also reduce labor costs.

3.4 MONITORING FREQUENCY AND REPORTING

The frequency of monitoring measurements and reporting is generally established in the monitoring plan for a waste containment facility. Fixed probes (e.g., groundwater monitoring wells) are typically monitored quarterly or semi-annually, but they can be monitored monthly or more frequently if warranted. Surface gas emission measurements and air quality measurements are often made at monthly intervals. Sometimes daily or even hourly measurements are warranted, as in time domain reflectometry measurements of soil moisture in evapotranspirative caps, where modeling tries to capture daily fluctuations in near surface water content caused by fluctuations in temperature, solar radiation, barometric pressure, and precipitation. However, measurements of this frequency are rarely required to assess barrier performance or to monitor environmental protection because subsurface contaminant migration patterns rarely change rapidly over scales in excess of tens of meters. Where electronic data acquisition systems are employed, data may be recorded at closely spaced intervals but may not be looked at in detail unless a problem arises. An example of electronic data collection and dissemination is the system installed in Fernald, Ohio. A final cover monitoring program developed for the Fernald Closure Project relies on a data logger to collect pore water pressure, drainage layer temperature, soil water content, soil water potential, and soil temperature above the geomembrane (Benson et al., 2003). The data are collected hourly and uploaded to a publicly accessible Web-based data management system.

Monitoring data may be reported only at quarterly or annual intervals, even if the data are captured more frequently. Quarterly reports are often simply data reports, and data interpretation is provided in annual or even multiyear summary reports. Some monitoring programs, however, have trigger levels, which, if exceeded, require immediate reporting and interpretation.

3.5 CONCLUSIONS

Performance of a containment barrier system is defined by how well the system and its components work over time. A good barrier system is one that meets or exceeds its design specifications. While well-designed engineered barrier systems generally function adequately immediately after construction, long-term performance depends on the long-term integrity of the system components, as well as proper operation and maintenance of the system before and after closure. It is important to recognize that the consequences of an engineered barrier system failure may have environmental and financial costs that far exceed the incremental cost of a facility monitoring program designed to detect potential problems before they occur. Consequently, it is critical to monitor the performance of engineered barrier systems with a variety of techniques and in a variety of media (surface water, groundwater, air, and soil). For landfills, one of the most effective ways to both monitor performance and minimize the impact of a failure of a primary barrier system is to have a double-lined system, as is now required in a number of states. Confidence in the long-term proper performance of waste containment systems can be gained only if the proper monitoring protocols are implemented.

Indirect monitoring of engineered barrier performance by monitoring for contaminant migration downstream of the waste containment system is commonplace, as it is mandated by regulations. Direct monitoring of barrier system component integrity is generally limited to an end-of-construction assessment of the component. Modern construction quality assurance procedures generally provide a high level of reliability for barrier component integrity in the short term. However, there has been little long-term direct monitoring of the integrity of individual barrier system components.

The primary liner in a double-liner system is perhaps the only type of engineered barrier system in which postconstruction integrity is routinely monitored directly. Liquids collected in the leak detection layer between the primary and secondary liners provide a direct assessment of the performance of the primary liner system. The absence of direct postconstruction monitoring of barrier integrity for other types of systems may be attributed to a variety of factors, including

- difficulty in directly monitoring barrier integrity, particularly for barrier systems overlain by tens to hundreds of meters of waste or soil;
- a philosophy that it is the overall performance of a waste containment system, not the integrity of individual elements of the system, that is important; and
- the reluctance of designers, owners, operators, and regulators to monitor something they may not be able to remedy or that would be exceedingly costly to address.

Of these factors, the technological limitation is perhaps the easiest to overcome, particularly for caps and many near-surface vertical barriers. A variety of techniques can be used to monitor the postconstruction integrity of caps. Since a cap generally has relatively shallow soil cover, exhumation and recovery of samples of cap material and tests for degradation of their properties are feasible in most cases. While this has been done for short-term or early medium-term monitoring of geosynthetic clay liners, no long-term evaluations of buried cover system elements of this type have been conducted to the committee's knowledge. In situ moisture content monitoring of soil layers in caps above, in, and below the barrier system can also provide an indication of cap performance. Furthermore, electrical surveys and leak detection surveys could be employed with geosynthetic (geomembrane and perhaps geosynthetic clay liner) caps if a wire grid is placed below the barrier layer during construction, and other geophysical monitoring techniques (e.g., electromagnetic surveys) could be used to assess changes in the physical properties of the cap over time. Temperatures can be monitored in caps and bottom liner systems to determine the service environments for soil and geosynthetic barrier components.

Geophysical techniques also offer promise for cost-effective long-term monitoring of vertical barriers. Electrical resistivity surveys and electromagnetic surveys offer the potential to detect gross defects that concentrate flow in vertical barriers. Tomographic imaging and seismic velocity surveys have the potential to detect changes in physical properties over time that may suggest barrier degradation. Inferred changes in barrier properties could be evaluated by in situ testing of the barrier or by physical sampling and laboratory testing when warranted.

Airborne and satellite-based remote monitoring techniques may offer the potential for cost-effective indirect monitoring of cap and vertical barrier effectiveness. Multispectral imaging can indicate water content and temperature changes in near-surface soils, as well as distress and other changes in vegetation, each of which may indicate barrier performance problems. Interferometric synthetic aperture radar, light detection and ranging, and other airborne/satellite techniques can resolve centimeter-scale deformations caused by local or global instability or barrier performance problems. Autonomous monitoring systems could detect moisture fluxes from cracks in caps. However, these techniques have not yet been demonstrated as useful tools for evaluating containment barrier effectiveness.

4

Performance of Barrier System Components

The performance of a barrier system component depends on two factors: (1) the performance of the component itself and (2) the way individual components interact as a system to contain contaminant transport. This chapter focuses on observations and predictions of the performance of individual barrier system components. Component interaction and overall barrier system performance are discussed in Chapter 5.

Problems that may arise with the performance of barrier system components in the short, medium, and long terms (see definitions in Box 1.2) are summarized in Table 4.1.

4.1 EARTHEN BARRIERS

Processed natural soils and mineral admixtures such as bentonite are extensively used as components of liners, covers, and vertical barrier walls in waste containment systems. At some sites the natural soil layer underlying a waste landfill or contaminated ground may be relied on to isolate and/or absorb contaminants in an engineered barrier system. If geological conditions and testing indicate that such layers are suitable in their initial state, they may still be subject to the same types of long-term performance concerns as earthen layers within liners and covers, including secondary structure development, incompatibility with the waste, and defects arising from induced deformations. Key performance concerns for earthen barriers, such as compacted clay liners, include the following:

- Short term: the ability to achieve the specified hydraulic conductivity upon construction in the field and slope stability.
- Medium and long term: increases in hydraulic conductivity with time caused by desiccation, shrink/swell, freeze/thaw, root penetration, thermal stresses, differential settlement, or chemical incompatibility; erosion of protective soil layers, development of secondary structures from cracking; and waste and slope stability.

Observations and predictions associated with these performance considerations are discussed below.

4.1.1 Hydraulic Conductivity

Field performance data on as-constructed compacted clay liners are summarized by Benson and Boutwell (1992), Daniel (1984, 1990), Cartwright and Krapac (1990), Elsbury et al. (1990), Gordon et al. (1990), Johnson et al. (1990), Reades et al. (1989), Trautwein and Williams (1990), King et al. (1993), Benson et al. (1999), Rowe et al. (2000), and Rowe (2005), among others. Reliable field permeability measurements generally require use of relatively large-scale tests, such as sealed double-ring infiltrometer tests or back calculation from lysimeter performance, to account for potential development of secondary structures within the compacted clay. An example of the measurement of the hydraulic conductivity of a compacted clay liner in the field using a sealed double-ring infiltrometer is given in Box 4.1. Available field performance data indicate that a target saturated hydraulic conductivity value (e.g., the prescriptive value of 1×10^{-9} m/s for Subtitle D low-permeability liner systems) can generally be achieved at the end of construction if the soil is compacted wet of the optimum moisture content and proper construction procedures are used.

Hydraulic conductivity data on compacted clay low-permeability soil layers for 85 cases from active landfill cells (8) and test pads (77) were reported by Benson et al. (1999). All of the compacted clay layers were intended to achieve a saturated hydraulic conductivity of less than 1×10^{-9} m/s. Although the test results indicated that the low-permeability soil layers for all 8 of the active landfill cells achieved a saturated hydraulic conductivity of less than 1×10^{-9} m/s (as measured by lysimeters), 22 of the 77 test pads tested did not achieve that goal, despite the fact that in all but 3 cases laboratory testing on thin-walled tube samples recovered from the test pad yielded a saturated hydraulic conductivity of less than 1×10^{-9} m/s.

TABLE 4.1 Performance Concerns for Barrier System Components

	Performance Time Frame	
Barrier System Component	Short-Term Concerns	Medium- and Long-Term Concerns
Earthen barriers	Defective material; inadequate compaction (density and/or moisture content); desiccation; slope stability	Cracking due to shrink/swell, freeze/thaw, root penetration, differential settlement, desiccation; chemical incompatibility; waste and slope stability
Geomembranes	Defective material; physical damage due to construction activities; defective seams	Puncture; global and local stability; degradation
Geosynthetic clay liners	Defective material; seam separation	Cracking due to shrink/swell, freeze/thaw, root penetration, differential settlement, etc.; chemical incompatibility; local and global stability; reinforcement degradation (needle-punched reinforced GCLs); inadequate hydration (encapsulated GCLs)
Granular and geosynthetic drainage layers	Inadequate capacity	Clogging due to soil infiltration, biological action, and mineral precipitation; geosynthetic drainage layers are also susceptible to soil and geosynthetics penetration and creep of the geonet core
Evapotranspirative barriers	Defective material; inadequate thickness; inability to establish vegetation	Inadequate storage capacity for infiltration; inability to sustain vegetation; cracking and development of other secondary permeability features; erosion, penetration by vegetation or animals
Vertical barriers	Defective material; "windows" due to caving and trapped low-quality material; leakage at joints between panels; lack of continuity in grouted barriers and extraction well systems	Cracking; desiccation of earthen barriers above the water table; chemical incompatibility of earthen and concrete barriers; corrosion of metallic barriers and of reinforcement in concrete barriers; anti-oxidant depletion and stress cracking of polymer barriers; clogging of vertical extraction trenches and wells
Asphaltic cement barriers	Defective material	Cracking due to shrinkage or deformation, degradation of the asphalt binder or supplemental material (e.g., crumb rubber)

NOTE: GCL = geosynthetic clay liner.

Relatively good agreement was achieved between hydraulic conductivities measured in the field using lysimeters (underdrains) and sealed double-ring infiltrometers, with geometric means of 9×10^{-11} m/s (8 cases) and 5×10^{-10} m/s (77 cases), respectively. However, the field-measured hydraulic conductivities did not correlate well with laboratory values of hydraulic conductivity measured on samples recovered from the test pad using thin-walled tubes when the soils were compacted dry of the optimum moisture content. This result reaffirms the need to compact clay liners wet of the optimum moisture content. Provided that this criteria is met and that there are no obvious visible secondary features (e.g., desiccation cracks), experience has shown that the hydraulic conductivity obtained in the laboratory on samples recovered with thin-walled tubes correlates well with values obtained in the field using lysimeters (e.g., Rowe et al., 2004).

Direct measurements of field hydraulic conductivity using sealed double-ring infiltrometers, as described in Box 4.1, are generally made only on test pads to establish compaction procedures and index properties (e.g., compaction moisture content and density) for quality control of the actual liner. Although the majority of field hydraulic conductivity measurements on compacted clay liners are made during or just after the completion of liner construction, some data are available on the hydraulic conductivity of a liner in the medium term. Box 4.2 describes the performance of a compacted clay liner test section subject to a constant hydraulic head of 0.3 m over a 14-year period. The performance of a liner in a waste containment environment may be influenced by the effects of increased temperatures, accompanying waste decomposition, and overburden pressures, as illustrated by the case history in Box 4.3.

4.1.2 Secondary Features

The difference between laboratory- and field-measured saturated hydraulic conductivity values for compacted earthen barriers can be attributed to macrostructure that often occurs when the soil is compacted at or dry of the optimum moisture content (Benson et al., 1999). Secondary features

BOX 4.1 Case History on Measuring Field Performance of Compacted Clay Liner

This case history illustrates the use of hydraulic conductivity measurements in the field to demonstrate the short-term performance of a compacted clay liner. The field data come from a test pad constructed to evaluate a compacted clay liner for a proposed hazardous waste landfill near Phoenix, Arizona (Gilbert, unpublished). The compacted clay liner was a soil-bentonite admixture, and the test method was a sealed double-ring infiltrometer (Daniel, 1989; ASTM D 3385-88). A 1.5-m square "ring" was placed on top of the liner and filled with water to a depth of 0.3 m. The infiltration rate (the volumetric rate of flow per unit area seeping into the top of the clay liner under a constant head) was measured by tracking the mass of water lost from the ring as a function of time. The infiltration rate in Figure 4.1 was calculated assuming that all of the water lost was infiltrated down into the clay liner. Tensiometers in the clay liner at depths of 0.15, 0.3, and 0.45 m were used to track the change in matric suction, and therefore pore water pressure, with depth and time as the water infiltrated the liner. When the measured matric suction is zero in a tensiometer, the hydraulic gradient can be estimated by assuming the pore water pressure is equal to zero at the elevation of the tip of the tensiometer.

The infiltration rate, q, is related to the hydraulic conductivity as follows: $q = ki$, where k is the hydraulic conductivity and i is the hydraulic gradient. Hence, the hydraulic conductivity in Figure 4.1 is calculated by dividing the infiltration rate by the estimated gradient. The calculated or "measured" hydraulic conductivity changes with time for several reasons: (1) the soil swells during the test, so some of the flow lost from the inner ring goes into storage and does not infiltrate through the clay liner; (2) the depth of soil used to estimate the hydraulic conductivity increases with time and the soil displays variability with depth (such as layering); and (3) the hydraulic gradient changes rapidly with time and is more difficult to estimate at the beginning of the test. The reported value is typically taken from the last reading, a hydraulic conductivity between 2×10^{-10} and 3×10^{-10} m/s in this case. Note that this hydraulic conductivity does not necessarily correspond to saturated conditions because of the possible existence of entrapped air, but the measured value probably corresponds reasonably well to the field value since the test conditions are similar to those expected in the field except for the lack of overburden stress on the clay liner. In the field application the applied stress from waste in the actual landfill will tend to increase the degree of saturation and reduce the saturated hydraulic conductivity due to consolidation.

The saturated hydraulic conductivity was measured in the laboratory on small-scale (64-mm-diameter), thin-walled tube samples taken from the clay liner at the end of construction. The laboratory measurements, which correspond to saturated conditions, ranged from 1×10^{-10} m/s to 7×10^{-10} m/s with a geometric mean of 3×10^{-10} m/s. Therefore, in this case, the hydraulic conductivity measured at the field scale (a 1.5-m square area), between 2×10^{-10} and 3×10^{-10} m/s, was both comparable to the laboratory measurements on small-scale samples under saturated conditions and less than the regulatory-specified maximum of 1×10^{-9} m/s.

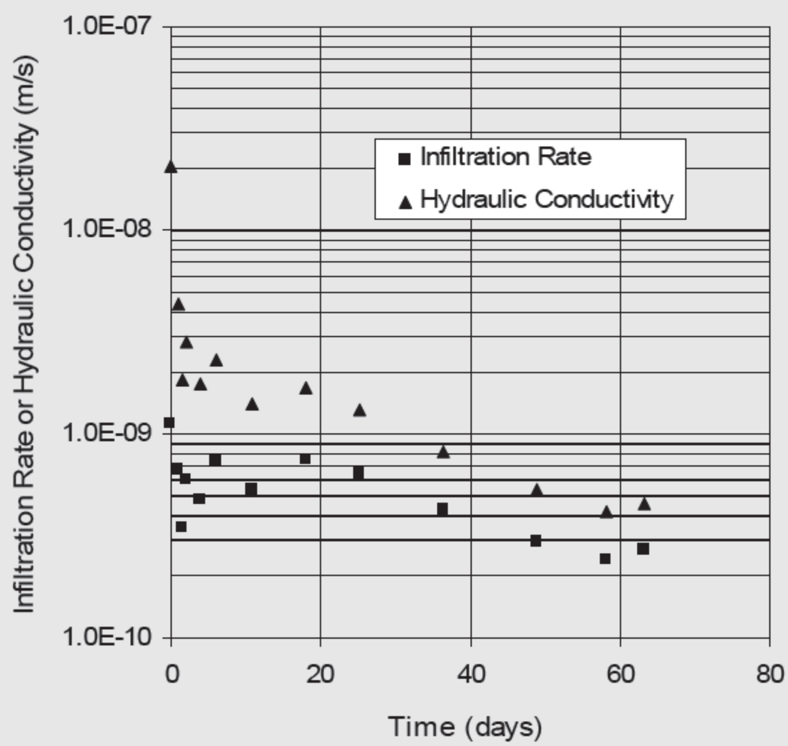


FIGURE 4.1 Field data from sealed double-ring infiltrometer test on a compacted clay liner.

BOX 4.2 Case History on Field Performance of a Compacted Clay Liner

This case history documents the monitoring of an instrumented earthen liner for changes in hydraulic conductivity over 14 years. In 1987 a heavily instrumented earthen liner was constructed by the Illinois State Geological Survey in Champaign, Illinois. The liner had dimensions of 7.3 m $\times$ 14.6 m $\times$ 0.9 m and was impounded with 0.3 m of water for 14 years. Two years after construction a hydraulic conductivity of 3–4 $\times$ 10^{-10} m/s was measured using large- and small-ring infiltrometers (Cartwright and Krapac, 1990). The liner was originally compacted to a mean gravimetric water content of 11.6 $\pm$ 1.2 percent, a dry density of 1.83 $\pm$ 0.071 kg/m^3, and a saturation of 64.0 $\pm$ 9.6 percent (Frank et al., 2005). After 14 years of ponding, the water content remained relatively constant (10.6 $\pm$ 1.0 percent during excavation), even though water infiltration into the liner was documented. The essentially constant water content was attributed to insufficient hydraulic pressure to saturate the unsaturated portions of the liner. In contrast, the mean dry density of the liner increased approximately 10 percent by the conclusion of the test, and the saturation increased to 80.0 $\pm$ 6.3 percent. Tracer studies and modeling performed at the conclusion of the test indicated that the advective transport of tracers through the liner was negligible, with model results yielding $k \leq 1 \times 10^{-9}$ m/s (Willingham et al., 2004).

BOX 4.3 Case History on Long-Term Field Hydraulic Conductivity Changes Resulting from Consolidation and Heating

This case history illustrates how the hydraulic conductivity of a clay layer in a bottom liner system in a municipal solid waste landfill can decrease over time because of consolidation but also increase over time (to a lesser extent) because of increased liner temperature. The Keele Valley Landfill, located in Maple, Canada, covers 99 hectares. The barrier system typically consists of a 1.5-m-thick compacted clay liner overlain by a 0.3-m-thick sand protection layer and a leachate collection system (described in Box 4.4). The landfill capacity is approximately 33 million m^3, and the maximum depth of waste at closure was about 65 m. Between the first acceptance of waste in 1984 and closure in December 2002, the landfill received 28 million tonnes of municipal solid waste. The landfill has been extensively monitored and studied (Rowe, 2005).

The hydraulic conductivity of the compacted clay liner was monitored by means of several lysimeters below the liner. Consolidation was an important factor affecting changes in the hydraulic conductivity k with time. For example, at 1 lysimeter the waste thickness increased from 1 m in November 1984 to 33 m in April 1987, then remained relatively constant. The annual average k of the liner dropped from 4×10^{-10} m/s in 1984 to 3.1×10^{-11} m/s in 1991. Two consolidation-related factors could have affected flow to the lysimeter during this period. First, expulsion of water from voids during consolidation is not explicitly considered in calculating k, so the inferred k value will be higher than the actual value until consolidation is complete. This effect was greatest from 1984 to 1987 when the waste thickness increased. Second, the decrease in void ratio of the soil due to consolidation will result in a decrease in k with time. Both effects occurred relatively quickly in the liner since consolidation was fast (95 percent consolidation in less than a year for a given load increment), but the decrease in void ratio was the dominant factor (Rowe, 2005).

Hydraulic conductivity of the compacted clay liner was also influenced by temperature. The temperature above the compacted clay liner was monitored at four locations over a 21-year period (Barone et al., 2000; Rowe, 2005). At one location the average annual temperature above the liner increased from 14°C in 1991 to 35°C in 1995, after which time the temperature remained constant (see Box 5.3 for the relationship between temperature and the performance of the overlying leachate collection system). Over the same period, the average annual hydraulic conductivity at this location increased from 3×10^{-11} m/s to 5×10^{-11} m/s. This increase matches what would be expected from theoretical consideration of the effect of temperature on the viscosity and density of the leachate. Since both the temperature and the hydraulic conductivity have stabilized, no significant effect of clay-leachate interaction over the subsequent 8 years is likely (Rowe et al., 2004).

such as cracks and voids can degrade the performance of both compacted earthen barriers and natural clay layers, where secondary features such as fissures are known to have a significant impact on performance (e.g., Keller et al., 1986; Rowe et al., 2000). Stress relief from differential settlement in soil covers can create secondary zones of high permeability, even in the absence of detectable cracking. The development of secondary features in a clay barrier as a result of desiccation, freeze/thaw damage, or tensile strains can cause significant increases in hydraulic conductivity. This effect is a particular concern for cover systems and for clay liners that are left uncovered during and after construction. Field data on air desiccation and freeze/thaw damage have been reported by Montgomery and Parsons (1990), Benson and Othman (1993), Corser and Cranston (1991), Benson and Khire (1995, 1997), Melchior (1997), Albrecht and Benson (2002), and Albright et al. (2006).

Desiccation of compacted clay liners in cover systems has been reported to result from both drying and freezing (Benson and Khire, 1997; Benson, 1999; Albrecht and Benson, 2002; Albright et al., 2006). Test pits excavated in compacted clay liners 6 months and 3 years after construction showed extensive cracking in a profile with a single compacted clay liner.

Desiccation of compacted clay liners in bottom barrier systems is also possible if the liners are left uncovered (or covered only by a geomembrane) for weeks or months after construction (Corser and Cranston, 1991). Heat generated in the waste may also cause desiccation over the long term. The potential for desiccation depends on the temperature gradient, the water retention curve of the soil below the clay liner, and the location of the water table (Rowe, 2005). For example, a test plot of a barrier system with a geomembrane overlying a compacted clay liner was subjected to a constant temperature of 40°C above the liner system and an overburden stress of approximately 40 kPa for 2.5 years (August et al., 1997). The compacted clay liner was underlain by a fine-grained soil in one section and a coarse-grained soil in another section. The water content of the section over the fine-grained soil remained relatively constant with maximum changes of less than 1 percent, whereas the water content of the section over the coarse-grained soil decreased up to 7 percent under thermal gradients as low as 5°C/m. Additional studies of desiccation of clay liners in composite liner systems caused by the heat from decomposing waste are reported by Rowe (2005).

4.1.3 Chemical Compatibility

The long-term performance of earthen barriers with respect to advective flow depends directly on the hydraulic conductivity of the soil barrier materials after saturation by the liquid being contained, which for liner systems is typically leachate rather than water. It is important to know, therefore, whether or not the barrier soil and permeating liquid are compatible so that permeation with the liquid causes no significant increase in hydraulic conductivity relative to that based on permeation with water (Fernandez and Quigley, 1985, 1988; Bowders et al., 1986; Bowders and Daniel, 1987; Shackelford, 1994; Shackelford et al., 2000; Rowe et al., 2004; Lee et al., 2005). Chemical incompatibility may create adverse effects both by increasing the saturated hydraulic conductivity of the soil and by creating secondary features that increase the mass saturated hydraulic conductivity (e.g., by inducing cracking due to shrinkage or embrittlement of the clay). An adverse interaction due to chemical incompatibility is a particularly important consideration when clay soils are used for waste containment, since an adverse interaction may result in an increase in saturated hydraulic conductivity and potentially unacceptable release rates of contaminants into the surrounding environment. The importance of compatibility testing is highlighted in the industry standard ASTM D 7100.

Most, if not all, of the available data on chemical compatibility of compacted clay liners or other earthen barriers are based on the results of laboratory tests, performed either as part of the design process for the waste containment facility or as research studies. Field data on chemical compatibility are not readily available for several reasons, including (1) less control inherent in field tests, especially when dealing with potentially harmful liquids; (2) lack of evidence of chemical incompatibility in waste containment facilities that are performing satisfactorily; and (3) years or even decades may be required before there is any evidence of chemical incompatibility processes that lead to volume change and cracking (Shackelford, 2005).

Numerous studies performed primarily to evaluate factors that potentially influence chemical compatibility of earthen soils used as barriers have provided some insight into expected behavior (e.g., Anderson and Jones, 1983; Fernandez and Quigley, 1985, 1988; Griffin and Roy, 1985; Mitchell and Madsen, 1987; Goldman et al., 1988; Shackelford, 1994; Stern and Shackelford, 1998). In general, the results indicate that qualitative predictions of chemical incompatibility can be made based primarily on three considerations: (1) the type and properties of the liquid, (2) the type and properties of the soil, and (3) the physical conditions imposed on the barrier soil.

The higher the concentration and/or the charge of ions in inorganic solutions, the greater the possibility for chemical incompatibility. In addition, decreasing pH tends to increase the potential for chemical incompatibility in two ways. First, as pH decreases, the concentration of ionic species in solution tends to increase. Second, solutions with pH less than about 2 can dissolve clay soils, resulting in the development of relatively large pores that cause increases in hydraulic conductivity (Shackelford, 1994). Very high pH (>12) solutions may also dissolve silica, although no examples of such solutions causing compatibility problems in compacted earthen liners have been published. Finally, high concentrations of hydrophilic organic compounds and pure-phase organic liquids (i.e., light and dense nonaqueous-phase liquids) can cause incompatibility due to clay shrinkage when the dielectric constant of the liquid is significantly lower than it is for water (e.g., Mitchell and Madsen, 1987; Shackelford, 1994). However, the specific threshold values at which chemical incompatibility becomes a potential issue for each of these parameters will also depend on the type and properties of the soil and the physical conditions imposed on the barrier soil.

The potential for incompatibility generally increases as the activity, defined as the plasticity index divided by the percent of clay-size particles (<2 μm), of the soil increases (Shackelford, 1994). The activity reflects both the mineralogy of the soil, with higher values for a plasticity index correlating with higher contents of high-activity clay minerals, such as sodium smectite, as well as the surface area effects, with greater surface areas reflected by higher contents of the smaller, clay-sized particles. Fortunately, available evidence suggests that many, if not most, natural clay soils used as compacted clay liners have relatively low plasticity index values. For example, of the 85 compacted clay liners evaluated by Benson et al. (1999), 49 were constructed using clays of low plasticity (ASTM D 2487), with the remainder exhibiting moderate rather than high plasticity index values (maximum of 71 compared with >300 for bentonite). Consequently, waste containment barriers that contain significant bentonite contents, such as compacted sand-bentonite and soil-bentonite liners, soil-bentonite backfills for vertical cutoff walls, and geosynthetic clay liners (GCLs) may be particularly susceptible to chemical attack by invading liquids, resulting in chemical incompatibility and increased hydraulic conductivity.

Three factors relating to the physical conditions are particularly important: (1) the stresses imposed on the barrier soil (Fernandez and Quigley, 1991; Shackelford et al., 2000; Lee et al., 2005), (2) the initial level or degree of hydration of the barrier soil (e.g., Shan and Daniel, 1991; Daniel et al., 1993; Shackelford, 1994; Didier and Comeaga, 1997; Gleason et al., 1997; Kajita, 1997; Petrov and Rowe, 1997; Petrov et al., 1997a; Quaranta et al., 1997; Ruhl and Daniel, 1997; Stern and Shackelford, 1998; Lin and Benson, 2000; Shackelford et al., 2000; Vasko et al., 2001; Ashmawy et al., 2002; Shan and Lai, 2002), and (3) the hydraulic gradient of the liquid across the barrier (Shackelford, 1994). In general, the higher the effective stress in the soil, the less susceptible the soil is to adverse consequences of chemical attack. Thus, consolidation of barrier soils (e.g., from the weight of the overlying waste) not only tends to decrease the hydraulic conductivity of the barrier soil but also reduce the potential for chemical incompatibility.

A higher initial degree of water saturation of the barrier soil prior to permeation with a chemical solution, commonly referred to as prehydration, also has been reported to provide increased resistance to increases in hydraulic conductivity. However, prehydration by continuous permeation results in a different degree of resistance to chemical attack than prehydration by capillary wetting. In addition, most of the studies reporting a significant prehydration effect have involved barrier soils either comprised of or containing high-activity sodium bentonite. Recent studies also indicate that the prehydration effect is concentration dependent and tends to diminish as the concentration of chemical species in solution decreases (Lee and Shackelford, 2005a). Lastly, much of the data supporting the claim of increased chemical resistance with increased prehydration has been criticized on the basis that the data were derived from hydraulic conductivity tests that were terminated before chemical equilibrium between the effluent and influent had been achieved (Petrov et al., 1997b; Shackelford et al., 2000).

Finally, two-phase flow, the migration of mixtures containing two separate liquid phases in soil, requires special consideration. The interaction between the two liquid phases is particularly important when nonpolar hydrophobic organic liquids migrate through a barrier soil at an initially high degree of water saturation, such as a compacted clay liner. Due to surface and interfacial tension effects, the non-polar liquid (a non-wetting fluid) can displace the water (a wetting fluid) in the pore space of the soil only after a minimum pressure (known as the entry pressure) for the non-wetting fluid is reached. Although some studies have shown that permeation of compacted clay soils by hydrophobic organic liquids has resulted in significant increases in hydraulic conductivity (e.g., Fernandez and Quigley, 1985), the hydraulic gradients required to force these liquids through the soils typically have been quite large (>100). Thus, the relevance of these results to situations where much smaller hydraulic gradients are applicable is questionable (Shackelford, 1994).

4.1.4 Chemical Transport

Chemicals migrate through earthen barriers primarily by two transport processes: advection and diffusion. A third process that causes spreading of the contaminant front due to variations in flow, referred to as mechanical dispersion, is typically insignificant in the case of thin, low-permeability earthen barriers and is consequently ignored. In addition to advection and diffusion, chemicals are subject to the myriad of physico-chemical processes (e.g., adsorption/desorption, precipitation/dissolution, acid/base, redox) and biological processes during migration that can ultimately affect the contaminant mass that flows through the barrier.

Temperature affects advective/diffusive transport as well as other physico-chemical and biological processes that contribute to contaminant mass transport. Hydraulic conductivity is expected to increase with temperature (see Section 4.1.2), which would increase advective transport. The free solution diffusion coefficient for species common in municipal solid waste (MSW) leachate (Rowe et al., 2004) and the soil diffusion coefficient for a clay soil beneath a landfill (Crooks and Quigley, 1984) increase with temperature, which would increase diffusive transport.

The advective mass flux of contaminants is essentially the product of the liquid flow or leakage rate and the contaminant concentration at a specified location and time. According to Darcy's law, typical steady state leakage rates for an intact compacted low-permeability earthen barrier (e.g., a compacted soil layer with no secondary permeability features such as cracks or fissures) are shown in Table 4.2. These values do not take into account that both the gradient and the hydraulic conductivity may vary with time for reasons such as those discussed above.

The results of several field studies indicate that diffusion can be a significant, if not dominant, transport process for

TABLE 4.2 Representative Leakage Rates for Clay Liners

Hydraulic Conductivity (m/s)	Steady State Leakage Rate[a] (lphd)
1×10^{-9}	100 to 10,000
1×10^{-10}	10 to 1,000
1×10^{-11}	1 to 100

[a]For typical range of hydraulic gradients from 1/10 to 10.

chemicals through intact clayey tills and low-permeability compacted clay liners (Goodall and Quigley, 1977; Crooks and Quigley, 1984; Johnson et al., 1989; Toupiol et al., 2002; Willingham et al., 2004). For example, the results of a tritium and bromide tracer test conducted over 13 years on a 0.3-m-thick compacted clay liner demonstrate that diffusion was the dominant transport mechanism and that there was no apparent preferential flow (Box 4.2; Toupiol et al., 2002; Willingham et al., 2004). As a result of these and other studies, minimizing advective mass flux transport has long been recognized as a necessary, but not sufficient, condition for minimizing chemical transport through low-permeability earthen barriers (Shackelford, 1988).

However, in contrast to these studies, Munro et al. (1997) found that advection and mechanical dispersion were more important than diffusion for a case involving migration of landfill leachate from an unlined landfill located directly on a fractured clayey till. This result underscores the importance of two considerations in terms of minimizing the extent of chemical release from waste disposal sites, namely the importance of a low-permeability engineered barrier system and the importance of disposal site location. Furthermore, in those cases where diffusion dominates advection, the total chemical transport is likely to be relatively small compared to those cases where advection dominates.

Recently, volatile organic compounds at various concentrations have appeared in a number of samples from 91 large (~ 110 m^2) collection lysimeters (underdrains) located beneath both composite and compacted clay liners at 38 Wisconsin landfills that have been in operation for about 10 to 20 years (Shackelford, 2005; Klett et al., 2006). For example, concentrations greater than allowable maximum contaminant levels for toluene, tetrahydrofuran, dichloromethane, benzene, and ethylbenzene have been found in 90 of the 1,200 samples (~ 8 percent) from cells with liners containing geomembranes (Benson and Edil, 2004). These observations were not anticipated because the liners in Wisconsin have traditionally been thicker than those required by Resource Conservation and Recovery Act (RCRA) Subtitle D for MSW disposal facilities (*Federal Register*, 1991). That is, prior to 1996, Wisconsin required landfill cells to be lined with a minimum of 1.5 m of compacted clay, compared to 0.91 m required by RCRA Subtitle D. Subsequent to 1996, both Wisconsin and RCRA have required a composite liner consisting of a geomembrane liner overlying compacted clay, but Wisconsin requires 1.2 m clay compared to 0.62 m required by RCRA. The effects of natural attenuation processes such as dilution, adsorption, and degradation likely will render concentrations of these volatile organic compounds below detectable levels by the time the contaminants reach a regulatory compliance point, such as a perimeter monitoring well (Benson and Edil, 2004). The performance record is still relatively short (~ 10 to 20 years), so continued monitoring of these and other landfills will be required over the longer term before any final conclusions can be drawn regarding the performance of modern waste containment systems.

4.1.5 Mechanical Performance Issues

The ability of an earthen barrier to serve its intended function can be affected by cracking, erosion, and slope stability. Although attention often focuses on the resistance of barriers to advective flux of liquids and diffusive flux of chemicals, earthen barrier layers may also serve as intrusion barriers, provide protection for other layers in the barrier system, and resist advective flux of gases.

Gas control is a persistent problem at municipal landfills with earthen cover systems. Frequent maintenance (i.e., reworking the cover in areas where monitoring indicates gas emissions exceed regulatory standards) is commonplace at landfills that produce gas. This need for frequent maintenance can be attributed in part to cracking induced by subsidence of the waste and in part due to desiccation (e.g., from freeze/thaw). Observations of the susceptibility of earthen covers to gas emission suggests that they may also be poor infiltration barriers because cracks that let gas out are likely to indicate secondary structure development that degrades the hydraulic conductivity of a cover barrier layer.

Erosion of the uppermost earthen layer in a barrier system can also be a persistent problem at sites with covers that are steeper than the surrounding stable landforms. Establishment of healthy vegetative cover and provision of a mechanically resistant erosion control layer can mitigate soil cover erosion. However, until a stable ecosystem that mimics stable regional landforms develops on the cover, continued maintenance is likely to be required for vegetative cover layers. The longevity of mechanical erosion control layers depends upon the steepness of the cover, the severity of erosion-inducing forces (e.g., wind, water, freezing, thawing), and the character of the erosion-resistant material.

The integrity of earthen barriers can also be affected by displacements associated with slope instability. Earthen barrier layers in liner systems can be disrupted by foundation instability that develops as the waste is placed and the load on the foundation increases. Earthen barrier layers in cover systems are susceptible to veneer instability, wherein a planar failure surface develops along one of the interfaces in the system. Soil-geomembrane interfaces may be particularly susceptible to veneer failure due to the reduced strength of the interface compared to the soil. Failures at drainage layer interfaces are also common due to clogging or inadequate capacity of the drainage layer and the associated buildup of pore pressures within the drainage layer.

4.1.6 Summary

Documented observations of the hydraulic performance of compacted clay barriers for more than about 15 years do not exist. However, the hydraulic conductivity of clay barriers is generally acknowledged to be susceptible to significant increase as a result of processes such as desiccation cracking, differential settlement, lateral spreading (induced tension), freezing and thawing, and root penetration. These processes are most significant in cover systems because bottom barrier performance is generally enhanced by increases in the confining pressure associated with waste disposal. Earthen barriers are also susceptible to a variety of mechanical performance problems, including erosion and cracking of earthen cover systems, disruption of base liner barrier layers due to foundation instability, and veneer failure of cover barrier layers due to low interface strength, clogging, or inadequate capacity of drainage layers.

Both laboratory research and field observations suggest that the following conclusions can be drawn about the performance of earthen barriers:

1. Clay layers, as components of barrier systems, are generally effective in the short and medium term unless poor construction and/or operational practices diminish layer integrity.
2. Unprotected clay layers in covers may develop secondary permeability that can lead to decreased effectiveness.
3. Temperature effects and postconstruction moisture changes in earthen barriers used in bottom liner systems have received insufficient attention.
4. Laboratory data indicate that there may be circumstances (e.g., when dealing with active clays) where there is cause for concern about chemical compatibility effects within clay liners, but corroborating field data are lacking.

4.2 GEOMEMBRANES

Short-term performance concerns for geomembrane barriers include defective material, physical damage caused by construction activities, and defective seams. Medium- and long-term performance concerns include geomembrane puncture due to increased overburden pressure and global stability and local slope stability failures, as well as loss of integrity caused by degradation. Global and local stability are addressed in Chapter 5. Geomembrane degradation is generally considered to be a long-term performance issue. However, as discussed below, under some conditions geomembrane degradation may develop in the medium term.

Modern manufacturing and construction quality assurance (CQA) processes have improved the as-manufactured quality of geomembranes. However, geomembranes can be easily damaged during installation, which emphasizes the importance of continuous observation of liner construction until a sufficient buffer layer of soil or waste shields the geomembrane from construction equipment and other potential sources of damage (see Box 5.2).

4.2.1 Chemical Transport

Chemical transport through geomembranes occurs via two mechanisms (Rowe, 1998; Katsumi et al., 2001): (1) advective transport of chemicals via leakage through holes or defects in the geomembranes and (2) molecular diffusion through the intact geomembrane. Equations for calculating the rate of leakage through geomembrane defects were proposed initially by Giroud and Bonaparte (1989) and refined subsequently by other investigators. Measurements of flow rates through geomembranes are sparse, although monitoring of leak detection systems provides information on likely leakage levels (Bonaparte et al., 2002) and the equations that should be used to predict leakage (Rowe, 2005). Chemical transport flow rates have usually been estimated from laboratory studies aimed at assessment of the prediction capabilities of proposed transport equations (e.g., Benson et al., 1995; Rowe et al., 2004), rather than evaluation of the performance of geomembranes used in liner systems (although Rowe, 2005, reports monitoring in a clay liner below a geomembrane over a 12-year period). In lieu of field measurements of leakage through geomembranes, the general practice has been to either estimate (in design) or measure (during CQA) the number of holes or defects that occur per unit area in a geomembrane and then estimate the resulting leakage rate using predictive equations. The amount of leakage through a geomembrane will depend on the particular situation but in general will be proportional to the number of holes or defects in wrinkles in the geomembrane. With good construction, the chemical transport of organic compounds (especially volatile organic compounds) will be controlled by diffusion through the barrier, rather than by the number of holes (Rowe, 2005).

Diffusion of volatile organic compounds occurs through geomembranes, in both the aqueous phase (e.g., Rowe et al., 1995; Rowe, 1998; Sangam and Rowe, 2001; Edil, 2003) and the gas (vapor) phases (e.g., Koerner, 2005). Shackelford (2005) compared the concentrations of dichloromethane in samples collected from lysimeters beneath both composite-lined cells (geomembrane overlying compacted clay) and clay-lined cells in Wisconsin landfills that had been in operation for periods ranging from about 10 to 20 years. The comparison indicated that the concentrations of dichloromethane collected in lysimeters beneath composite-lined cells were not substantially lower than those in samples collected beneath cells lined only with compacted clay. This result suggests that the geomembranes used in the composite liners have provided little, if any, resistance to diffusion of this volatile organic compound. Rowe (2007) showed that volatile organic compounds can diffuse (1) through high-density polyethylene (HDPE) geomembranes in as little as a few days to a few weeks, (2) through a 0.6-m-thick composite liner in 1 to 2 years (at 1 percent of the source concentration) or 4 years (at 10 percent of the source concentration), and (3) through a

1-m-thick composite liner in 4 to 6 years (at 1 percent of the source concentration) or 10 years (at 10 percent of the source concentration). Both theoretical modeling (Rowe, 2007) and laboratory tests (Sangam and Rowe, 2001) show that the geomembrane offers little resistance to diffusion of these contaminants. These results are consistent with the findings of Shackelford (2005).

In summary, leakage through holes or defects as well as diffusion of volatile organic compounds can result in significant chemical mass transport through single geomembrane liners. As a result, the use of single geomembranes alone as liners for liquid containment is not advisable.

4.2.2 Geomembrane Defect Frequency

Geomembranes can develop leaks as a result of defective materials, physical damage caused by construction activities, and defective seams. Information on the short-term integrity of geomembrane barriers, immediately after installation, can be obtained using electrical leak detection surveys. These surveys are generally conducted after the leachate collection gravel is placed on top of the primary geomembrane and can be a powerful addition to CQA programs. A variety of factors can affect the results of an electrical leak detection survey, but if properly performed they are capable of detecting all but the smallest defects in a geomembrane liner (Hruby and Barrie, 2003). Average hole frequencies have been estimated for a large number of emplaced geomembranes, ranging from 0.7 to 11 holes per hectare for landfills (Rollin and Jacquelin, 1999; Nosko and Touze-Foltz, 2000; Hruby and Barrie, 2003; Forget et al., 2005) and up to 15 holes per hectare for leachate impoundments (Rollin and Jacquelin, 1999). Compiling 205 results from four published leak detection surveys, Rowe et al. (2004) found that (1) no holes were detected for 30 percent of the cases; and (2) less than five holes per hectare were detected for half of the surveys. In general, hole frequencies were higher in liners of MSW landfills than in liners of hazardous waste landfills, and hole frequencies were inversely proportional to the size of the facility. Furthermore, there was no correlation between the number of leaks and the geomembrane thickness, and leak densities were significantly lower (0.5 per hectare) for systems installed under strict CQA programs compared to those installed with no CQA (16 per hectare; Forget et al., 2005).

Although geomembrane leak frequency is important, so are the size and location of the defect. Geomembrane defects are often assumed to be relatively small holes caused by poor seaming, inadvertent puncture of the geomembrane, or thinning of the geomembrane during manufacture. Colucci and Lavagnolo (1995) reported that 50 percent of holes had an area of less than 1 cm^2. While a hole in a geomembrane in intimate contact with the underlying clay may be relatively small, much greater leakage will be observed if that same hole is in a wrinkle (Rowe et al., 2004). Although defects are generally assumed to be on the geomembrane only, and not in the underlying low-permeability soil layer of a composite barrier, careless construction, waste placement, or compaction operations can cause relatively large through-going holes that can have a disproportionate effect on barrier effectiveness. Through-going holes may be particularly significant in composite liners that employ GCLs as the low-permeability soil layer because GCLs are thin (on the order of 6 mm) compared with typical compacted clay liners (on the order of 600 mm).

The primary medium-term concerns with respect to geomembrane integrity are geomembrane puncture by the overlying or underlying material (e.g., leachate collection gravel or rocks in the subgrade) and global or local stability failure. Once more than 1 or 2 m of material is placed on top of a geomembrane, there is no practical way to determine if the geomembrane has been punctured or torn. In landfill practice, cushion geotextiles are often used to protect geomembranes against puncture from the overlying leachate collection and removal system's gravel. A cushion should not be used for subgrade puncture protection because it would negate the "intimate contact" between the geomembrane and underlying low-permeability layer required for composite liner effectiveness.

4.2.3 Impact, Tear, Burst, and Puncture Protection

Geomembranes are protected from puncture during construction and waste placement with a layer of soil cover, select waste, geosynthetics, or a combination of these materials. Several methods are used to design adequate puncture protection. The two primary approaches used in practice are (1) the "European" approach, which employs site-specific laboratory testing (ASTM D5514), and (2) the U.S. approach, which employs design charts and equations (Wilson-Fahmy et al., 1996; Narejo et al., 1996; Koerner et al., 1996). The European method is generally more restrictive than the U.S. method and results in a thicker cushion or a smaller allowable grain size of the leachate collection and removal system's gravel. Although few field data exist to verify either approach, limited field tests of a geomembrane puncture seem to indicate that the U.S. approach provides adequate short-term protection against punctures (Richardson, 1996; Richardson and Johnson, 1998). However, the U.S. design method does not account for induced strains that could eventually lead to stress cracking (e.g., Tognon et al., 2000).

4.2.4 Geomembrane Durability

A key consideration in assessing the long-term performance of barrier systems is the durability and service life of the geomembrane component. The service life is the period of time that an engineered component of a barrier system performs in accordance with the design assumptions. Assessing the likely service life of geomembranes is difficult because of their relatively short history of use in waste containment applications. A few field studies have provided some guidance. An HDPE geomembrane used in a contaminated soil containment facility for 7 years exhibited lower tensile resistance and elongation at rupture than the original material (Rollin et al., 1994). HDPE geomembrane samples from a 7-year-old leachate lagoon showed no substantial change in the internal structure of the geomembrane or its engineering/hydraulic containment properties (Hsuan et al., 1991). In contrast, an exposed HDPE geomembrane exhumed from a leachate lagoon liner after 14 years of operation was significantly degraded, severely cracked, and highly susceptible to stress cracking (Rowe et al., 1998, 2003). Geomembranes from the bottom of the same lagoon (covered by leachate rather than exposed to the elements) appeared to have been sufficiently protected to prevent significant degradation over the 14-year period. Tests on four different geomembranes from metal sludge impoundments after 20 to 31 years of service life indicated no significant change in the mechanical properties of interest (Tarnowski et al., 2005). These field performance evaluations are relatively positive; however, they do not assure the long-term effectiveness of geomembranes in landfill applications.

The potential for medium- and long-term degradation of HDPE geomembranes has been examined in the laboratory by several investigators (Hsuan and Koerner, 1998; Sangam and Rowe, 2002; Müller and Jakob, 2003; Tarnowski et al., 2005; Rowe, 2005). The primary geomembrane degradation mechanism is antioxidant depletion. Once the antioxidant used in modern HDPE geomembrane formulations is depleted, other geomembrane properties begin to degrade and the geomembrane becomes very susceptible to stress cracking and other damage. Given the relatively slow aging of geomembranes at room temperature, predictions of service life must be based on data from accelerated aging in laboratory tests. Even though no free oxygen is present at the base of a landfill, laboratory tests have shown that geomembranes can break down and reach the end of their service lives under highly reduced conditions (Rowe and Rimal, unpublished).

Typical laboratory tests to examine aging involve immersion of the geomembrane in a fluid (air, water, or leachate). The depletion of antioxidants is most rapid when the geomembrane is immersed in leachate and then in water and is slowest in air (Sangam and Rowe, 2002). Tests have been performed in which the geomembrane was placed on dry sand and covered with wet sand (Hsuan and Koerner, 1998) and also with geomembranes placed in simulated landfill liners with leachate-filled collection material above (Rowe, 2005). The latter tests show that the antioxidant depletion rate in a simulated liner is much slower than when immersed in leachate but substantially faster than when immersed in air.

Based on data on antioxidant depletion for a geomembrane in a simulated liner combined with published data for polyethylene pipe with a wall thickness comparable to a geomembrane (2.2 mm), Rowe (2005) inferred estimates of the likely range of the service life of HDPE geomembranes in MSW landfills for a range of temperatures (Table 4.3). The estimated service life of geomembranes at a temperature of 20°C is 565 to 900 years, depending on the assumptions. While this is a large range, it suggests that geomembrane liners are likely to have a service life of more than 500 years at a temperature of 20°C (and longer at lower temperature). This is generally consistent with the findings of Tarnowski et al. (2005), who concluded that the service life of properly produced and installed HDPE geomembranes at least 2 mm thick could be more than 500 years. At a typical MSW landfill primary liner temperature of 35°C, the service life is on the order of 130 to 190 years (median 160 years). At temperatures of 50 to 60°C, the service lives are very short (15 to 50 years). These high (50 to 60°C) temperatures have been observed when there is a leachate mound in the waste (Rowe et al., 2004) but are not expected in modern solid waste landfills with operational leachate collection systems (Rowe, 2005). It should be noted that, while these temperatures are measured just above the liner, high temperatures can also be observed with depth. Even with a 1-m-thick primary composite liner, the temperature at 1 m may be only about 5°C lower than at the geomembrane (Rowe and Hoor, 2007). The depletion of antioxidants from geomembranes exposed to acid rock drainage has been reported by Gulec et al. (2004), and the data will assist in assessing the service life of HDPE geomembranes under these conditions.

TABLE 4.3 Estimated HDPE Geomembrane Service Life at Different Temperatures

Temperature (C)	Service Life (years)[a]
20	565–900
30	205–315
35	130–190
40	80–120
50	35–50
60	15–20

[a]All times have been rounded to nearest 5 years.
SOURCE: Rowe (2005).

The service life of cover geomembranes depends on the temperature and exposure conditions. Koerner and Koerner (2005) suggest that the service temperature of a soil-covered geomembrane used in a cover of a dry landfill will be approximately equal to the ambient temperature of the site. Thus, for cover geomembranes in a temperate climate, with a mean ambient temperature of less than 30°C, the service life should be on the order of several hundred years (Table 4.3). This logic does not apply to geomembranes exposed to solar radiation or to geomembranes made of materials other than HDPE (e.g., linear low-density polyethylene, flexible polypropylene). Tarnowski et al. (2005) report on the service life of exposed HDPE geomembranes estimated from testing on four different exposed geomembranes after 20 to 31 years of service life and on 2.5 mm HDPE geomembrane specimens stored in air for 13 years at 80°C. Based primarily on tests conducted on specimens recovered from the exposed geomembranes from the four projects in Galing, Germany, Tarnowski et al. (2005) concluded that a service life of more than 50 years could be expected for the 2.5 mm HDPE geomembranes studied. Inadequate data are available for materials other than HDPE to assess their long-term performance as geomembrane liners or covers.

The values in Table 4.3 provide order-of-magnitude estimates of the geomembrane service life and also highlight the importance of liner temperature. However, as discussed by Rowe (2005), these estimates should be used with caution. Only the results for antioxidant depletion are based on actual tests on geomembranes typically used in landfill applications in a simulated liner configuration. Degradation of the polymer is based on tests for polyethylene pipe with water inside and air outside and is subject to a number of limitations (Viebke et al., 1994). Long-term testing to address these limitations is currently under way and the estimates of service life are likely to evolve as more data becomes available.

The calculated service lives (Table 4.3) all assume a constant temperature. However, the temperature of the liner is likely to vary with time, and this will influence both the antioxidant depletion time and the service life. The effect of time varying temperature on service life has been examined theoretically by Rowe (2005).

4.3 GEOSYNTHETIC CLAY LINERS

The primary performance concerns with GCLs are defective material and seam separation in the short term and desiccation, freezing, chemical incompatibility, and strength degradation in the medium and long terms.

Numerous laboratory measurements have established that the saturated hydraulic conductivity of GCLs permeated with water at nominal overburden pressure (on the order of 20 kPa) generally ranges from about 1×10^{-11} m/s to about 5×10^{-11} m/s (e.g., Daniel et al., 1997). With increasing overburden pressure, the saturated hydraulic conductivity of a GCL can be significantly lower.

Rowe (2005) concluded that the service life of a GCL used in a composite liner system should be on the order of thousands of years, provided that (1) the seams of the GCL do not separate; (2) the GCL does not desiccate; (3) the design hydraulic conductivity is based on considerations of chemical effects such as bentonite-leachate compatibility, groundwater and subgrade soil chemistry, and applied stress; (4) there is no significant loss or movement (thinning) of the bentonite in the GCL during placement or in service; and (5) the geosynthetic component of the GCL is not critical to the long-term performance of the bentonite component of the GCL (otherwise the service life of the GCL is controlled by that of the geosynthetic component). Predictions of GCL performance must include consideration of local and global slope stability, strength degradation, and chemical transport.

4.3.1 Seam Separation

Seam separation is a relatively recently observed phenomenon. Separation of lapped seams beneath exposed geomembranes at a number of sites has been reported not only in areas of relatively steep (1.5H:1V to 2H:1V) side slopes but also on relatively flat slopes (Thiel and Richardson, 2005; Koerner and Koerner, 2005). In all cases, the GCL was installed with a state-of-the-practice CQA program using overlaps between 150 and 300 mm at the seams, and the separation occurred prior to placement of waste against the side slope. The observed separation between panels was typically between 0 and 300 mm and occurred from 2 months to 5 years after placement, although separation exceeding 1 m also was observed. For the cases reported, the GCL had a high initial (as manufactured) moisture content and no scrim reinforcement. Although research assessing the significance of various factors in causing shrinkage of GCL panels is ongoing, quick covering of the liner system clearly is desirable as a means for minimizing potential shrinkage. Figure 4.2 shows the GCL seam separation observed at a California landfill.

Postulated mechanisms for seam separation include necking of the GCL panel under gravity loads and shrinkage of GCLs manufactured at moisture contents greater than the in situ equilibrium moisture content (Thiel and Richardson, 2005). Exposure of the overlying geomembrane to large temperature extremes can result in moisture loss and shrinkage of GCLs on side slopes. At night the colder air temperature draws water vapor up to the bottom of the geomembrane, where it condenses, drains down the slope along the interface between the geomembrane and GCL, and is no longer available to rehydrate the GCL the following day. Free water has been found accumulating between the

FIGURE 4.2 GCL seam separation at a California landfill. SOURCE: County of Riverside (2004).

GCL and geomembrane at the toe of steep side slopes under these conditions (personal communication, J.P. Giroud, independent consultant, and Jeff Dobrowolski, Geosyntec Consultants, October 2004), which appears to support this shrinkage mechanism. Recent laboratory tests suggest that cyclical wetting and drying can explain a large percentage of the observed shrinkage (Thiel et al., 2006). However, no single mechanism can explain the magnitude and prevalence of panel separations in all cases.

4.3.2 Chemical Compatibility

Chemical compatibility of GCLs, including both cation exchange and pore fluid chemistry effects, has been investigated extensively in both the laboratory and the field. Chemical degradation of the hydraulic conductivity of GCLs is possible when relatively high concentrations of divalent cations such as calcium and magnesium in the natural pore water or soil cause cation exchange with the sodium in the bentonite. For example, in one field investigation of a cover, laboratory tests on exhumed samples and field percolation rates showed that the saturated hydraulic conductivity of the GCL increased by about 100 times due to cation exchange (Benson, 1999, 2001). In a study of the effects of cation concentration and valence, the saturated hydraulic conductivity was correlated to the volumetric swell of GCL bentonite (Jo et al., 2001). Although this investigation was limited to a single cation species, subsequent investigations showed similar correlations with multispecies inorganic chemical solutions as well as with waste leachate (Kolstad et al., 2004; Katsumi and Fukagawa, 2005).

Bentonite quality (e.g., percent sodium smectite, plasticity, cation exchange capacity) and microstructure also influence the chemical compatibility of GCLs. Lee and Shackelford (2005b) and Lee et al. (2005) investigated the effect of bentonite quality on the hydraulic behavior of GCLs permeated with inorganic salt solutions and demonstrated that the higher-quality bentonite GCLs (i.e., higher cation exchange capacity, higher plasticity, higher swell capacity in the presence of water, lower hydraulic conductivity) were more susceptible to chemical attack and increased hydraulic conductivity than the lower-quality bentonite GCLs. Sodium bentonite can be quite susceptible to an increase in hydraulic conductivity when permeated by solutions with a high salt content (Petrov and Rowe, 1997; Lee and Shackelford, 2005b; Lee et al., 2005), especially when the effective stress is low. Under these circumstances it may be more appropriate to use a modified bentonite. However, as noted below, there are many practical situations when the sodium bentonite commonly used in GCLs performs well.

A number of GCL tests have focused on the effects of prehydration (e.g., Katsumi et al., 2004; Lee and Shackelford, 2005a; Thiel and Criley, 2005), confining pressure, cation exchange, and other factors on chemical compatibility. The hydraulic conductivity of a prehydrated and calendared (density has been increased to achieve a uniform thickness and void ratio) GCL appears resistant to degradation from permeation with $CaCl_2$ solutions (Kolstad et al., 2004; Katsumi et al., 2006a). The effects of overburden pressure on the hydraulic conductivity of GCLs permeated by sodium and calcium solutions were found to be significant and depended on the type of chemicals (Katsumi and Fukagawa, 2005). The effect of prehydration is also affected by the applied head pushing the prehydration water into the GCL. However, prehydration by capillary action alone is relatively ineffective in improving chemical compatibility of GCLs in the presence of salts (Vasko et al., 2001). Thiel and Criley (2005) showed that the hydraulic conductivities of GCLs prehydrated and permeated with a variety of waste leachates and water under high effective confining stresses (>500 kPa) were independent of fluid chemistry.

The importance of performing hydraulic conductivity tests long enough to achieve chemical equilibrium between the effluent and influent has been demonstrated for both prehydrated GCLs (Shackelford et al., 2000; Lee and Shackelford, 2005a, 2005b) and nonprehydrated GCLs (Jo et al., 2004, 2005; Katsumi et al., 2006b). Unconservative (low) values of hydraulic conductivity may be measured if the test is terminated before complete chemical equilibrium has been established. For example, in a test for salt compatibility in which the influent contains no sodium cations, true chemical equilibrium is not established until the concentration of sodium in the effluent is zero or below the method detection limit for practical purposes. At this point, cation exchange with the chemical solution has essentially been completed. However, because the method detection limit for sodium is so low (e.g., 0.2 mg/l) and the process of cation exchange in sodium bentonites can be slow (Jo et al., 2005), establishment of true chemical equilibrium can require long test durations (>1 year), which generally increase with decreasing concentration of multivalent cations in the permeant liquid (Jo et al., 2005; Lee and Shackelford, 2005a, 2005b). Thus, measuring the chemical composition of the influent and effluent precisely may be impractical, and meeting method detection limit criteria may not be possible for most applications.

As a result, Jo et al. (2005) recommend that electrical conductivity be used as a measure of chemical composition for the influent and effluent as proposed by Shackelford et al. (1999) and that tests be terminated once the effluent-to-influent electrical conductivity ratio is within 1.0 ± 0.05. This would acknowledge that the long-term hydraulic conductivity may be as much as three times higher than the reported hydraulic conductivity. Alternatively, Jo et al. note that if tests are terminated when the electrical conductivity ratio is within 1.0 ± 0.1 (as per ASTM D 6766), the hydraulic conductivity could be as much as 10 times higher than the reported hydraulic conductivity for conditions similar to those examined by Jo et al. (2005).

The hydraulic conductivity k of nonprehydrated GCLs permeated with inorganic chemical solutions was correlated with two chemical parameters: (1) the ionic strength of the solution I, which is the sum of the product of the molar concentration and the square of the charge for all ionic species in solution, and (2) the ratio of the molar concentrations of monovalent cations M_M to divalent cations M_D in the solution, or RMD (= $M_M/M_D^{0.5}$; Kolstad, 2000; Kolstad et al., 2004). The RMD is similar to the sodium adsorption ratio commonly used in the soil science literature. The resulting correlation between the k of GCLs and I and RMD of the permeant liquids is shown in Figure 4.3.

Two observations are apparent from the data plotted in Figure 4.3. First, the hydraulic conductivity increases with increasing ionic strength for a given value of RMD and also increases for a given ionic strength as the RMD decreases toward zero. These trends are consistent with expected behavior in the sense that an increase in concentrations and/or charge is expected to result in an increase in hydraulic conductivity (Shackelford, 1994). Second, the hydraulic conductivity of GCLs to chemical solutions with ionic strengths greater than about 0.2 to 0.4 M can be higher than the typical regulatory standard value of 1×10^{-9} m/s, and much higher in some cases (e.g., for mine waste, $k = 1.2 \times 10^{-5}$ at $I = 0.81$ M). Although the relationship shown in Figure 4.3 is relatively new, several independent assessments using different GCLs and different chemical solutions have shown good agreement between measured k values and the predicted k values based on Figure 4.3 (e.g., Lee and Shackelford, 2005a; Shackelford, 2005; Brown and Shackelford, 2007).

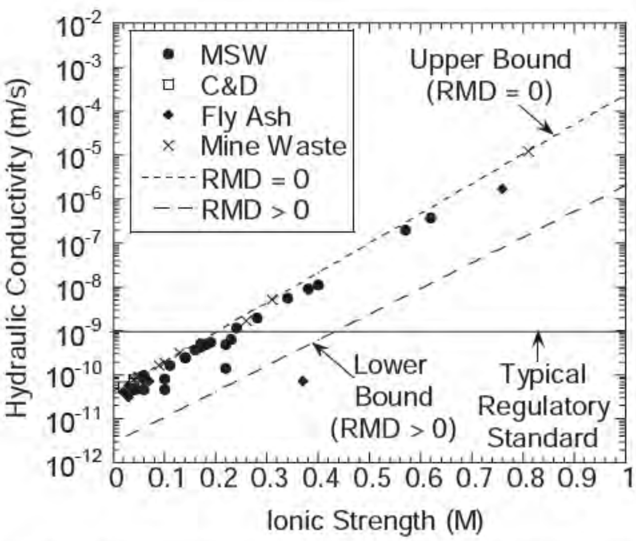


FIGURE 4.3 Correlation among hydraulic conductivity, ionic strength, and the ratio of monovalent-to-divalent cations (RMD) for nonprehydrated GCLs. NOTE: MSW = municipal solid waste; C&D = construction and demolition. SOURCE: Kolstad (2000).

4.3.3 Freeze/Thaw and Shrink/Swell

Freeze/thaw and shrink/swell are medium-term GCL performance concerns associated primarily with GCLs used in covers. The higher confining and overburden pressures applied to bottom-liner systems tend to mitigate these effects. Several studies have investigated resistance of GCLs in the laboratory after as many as 20 freeze/thaw cycles. Little if any change in hydraulic conductivity was measured, suggesting that freeze/thaw is not an important degradation process for GCL performance (Foose et al., 1996; Kraus et al., 1997; Della Porta et al., 2005; Rowe et al., 2006).

In contrast to freeze/thaw cycles, shrink/swell cycles can cause significant degradation of GCLs if (1) the wetting fluid contains multivalent cations, such as calcium or magnesium; (2) the dehydration during drying is severe enough for desiccation cracks to form in the bentonite layer; and (3) the overburden pressure is too low to prevent the formation of desiccation cracks. Under these circumstances, desiccation, combined with the chemical incompatibility effects of cation exchange discussed in the previous section, can cause increases of as much as three to five orders of magnitude in hydraulic conductivity. Desiccation is particularly important when combined with cation exchange because the multivalent cations reduce the ability of the bentonite to swell and heal the desiccation cracks (Lin and Benson, 2000; Sporer and Gartung, 2002a, 2002b). This mechanism of degradation is a particular concern for GCLs in cover systems where the potential for desiccation is greatest because of the combination of large fluctuations in temperature and low overburden pressures. For instance, Melchior (2002) showed the adverse effects of desiccation on five sodium-bentonite GCLs overlain by 0.45 m of cover soil. Both cation exchange and desiccation cracking were observed in exhumed samples (Figure 4.4) and the saturated hydraulic conductivities after exhumation were approximately five orders of magnitude

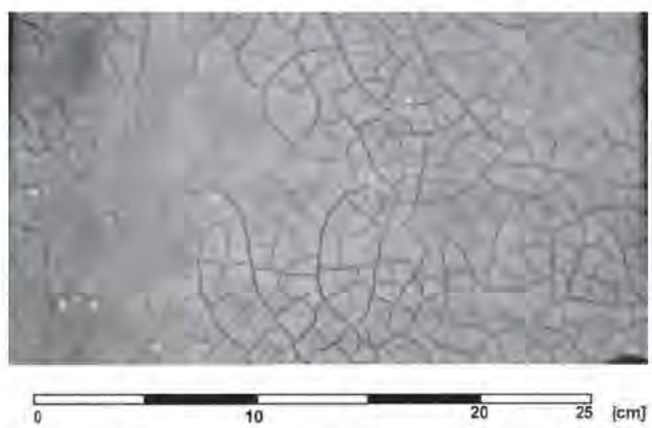


FIGURE 4.4 X-ray image showing desiccation cracks in the bottom layer of a GCL after 5 years in a landfill cover at Hamburg-Georgswerder. SOURCE: Melchior (2002).

higher than when the GCLs were installed. Plant-root penetration through the GCL also was observed.

Meer and Benson (2007) exhumed GCL samples from four landfill covers and found similar increases in hydraulic conductivity caused by cation exchange and desiccation cracking. In one case, although the GCL was overlain by a geomembrane, it was still affected by cation exchange, apparently from pore water in the underlying subgrade. Other field investigations confirm that desiccation occurs in GCLs after cation exchange and results in severe degradation of GCLs (Aboveground Tank Update, 1992; James et al., 1997; Benson, 2002; Mackey and Olsta, 2004).

Degradation caused by wet/dry cycles can be prevented by avoiding contact between the GCL and pore fluids containing relatively high concentrations of multivalent cations. A number of laboratory studies have shown that wet/dry cycles do not significantly degrade the performance of sodium-bentonite GCLs, provided that the sodium is not exchanged for divalent cations (e.g., Shan and Daniel, 1991; Boardman and Daniel, 1996; Lin and Benson, 2000). In addition, Mansour (2001) showed that the saturated hydraulic conductivity of a needle-punch-reinforced sodium GCL buried under a moderate thickness (0.66 m) of vegetated cover soil was unchanged after 5 years of exposure to the semiarid climate of Bakersfield, California. The cover soil was rich in sodium, and there was little potential for cation exchange in the GCL. The conclusion drawn from these studies is that desiccation cracks in a sodium-bentonite GCL may heal upon rehydration as long as the pore fluid chemistry in the GCL does not change with wetting and drying cycles and there is sufficient overburden pressure.

In addition, if the extent of drying is not severe, wet/dry cycles may not significantly degrade the performance of sodium-bentonite GCLs. For example, a GCL covered by a thick (1-m) layer of silty sand in a landfill cover subjected to a humid climate exhibited no significant desiccation cracking or other notable defects over a 3-year period despite dry summertime conditions (Blümel et al., 2002). Mineralogical testing on exhumed GCL samples showed that the transition from sodium- to calcium-bentonite due to cation exchange was essentially completed during the 3-year observation period but that ion exchange did not cause any significant increase in permeation (as measured by a lysimeter beneath a GCL) since the dehydration during the drying cycles was not severe. Della Porta et al. (2005) reported that drying temperatures of 105°C degraded GCL performance more than drying temperatures of 60°C.

Other examples of good and bad field performance of GCLs in covers has been discussed by Henken-Mellies et al. (2002), Sporer (2002), Heerten and Maubeuge (1997), Sivakumar Babu et al. (2002), and the potential for cracking and desiccation of GCLs in bottom composite liners due to heat generated by the waste has been discussed by Southen and Rowe (2002, 2004, 2005), and Rowe (2005).

4.3.4 Chemical Transport

Chemical transport parameters, including sorption effects, for GCLs exposed to volatile organic compounds have been reported by Lake and Rowe (2004, 2005a) and Rowe et al. (2004, 2006). Diffusion and sorption factors were such that volatile organic compound migration could be reduced but not controlled to the same level as with a compacted clay liner, unless the GCL was underlain with a suitable thickness of a soil attenuation layer. This effect results primarily because of the relative thinness of GCLs (typically 6 mm) compared to compacted clay liners (typically 0.6 to 0.9 m). Under diffusion-dominated conditions, such as can occur in low-permeability earthen barriers, the steady state diffusive mass flux is estimated by the product of the diffusion coefficient for the chemical species within the porous material (soil) and the concentration gradient, or the difference in concentration across the barrier divided by the barrier thickness. Thus, for the same concentration difference, the concentration gradient for a GCL will be much higher than that for a compacted clay liner. If, in addition, the effective diffusion coefficients for a given chemical species are similar between the GCL and a compacted clay liner (not unexpected since the GCL is usually composed of a natural clay mineral), the diffusive mass flux across the GCL may be expected to be greater than that across the compacted clay liner. In addition, the sorptive capacity of the GCL may be lower than that of the compacted clay liner because the GCL has a lower total mass of sorbent (clay) than the thicker compacted clay liner.

The results of studies aimed at measuring the diffusion coefficients of simple salt ions (e.g., Na^+, K^+, Cl^-) diffusing through GCLs under transient and steady state conditions have been reported by Lake and Rowe (2000), Malusis et al. (2001), and Malusis and Shackelford (2002a). Of particular note is the observation that GCLs can act as semipermeable membranes that restrict the migration of aqueous chemical species (Malusis et al., 2001; Malusis and Shackelford, 2002b). The membrane efficiency coefficient ranges from 0 percent in soils that exhibit no membrane behavior to 100 percent in ideal membranes that restrict the migration of all solutes. In general, the membrane efficiency of natural clay soils that exhibit semipermeable membrane behavior tends to be more prevalent in clay soils with a significant portion of high-swelling clay, such as the sodium bentonite in GCLs (Shackelford et al., 2003). For example, membrane efficiencies ranging from 8 to 69 percent have been reported for a GCL subjected to electrolyte solution with KCL concentrations ranging from 3.9 to 47 mM (Malusis and Shackelford, 2002b). Other factors that affect the existence and extent of semipermeable membrane behavior in clay soils include the stress conditions on the soil and the concentrations and valences of the cations in the pore water (Shackelford et al., 2003). Membrane behavior tends to be most significant for solutions with relatively low concentrations of monovalent cations. However, such dilute solutions can still significantly exceed regulatory limits (e.g., maximum contaminant levels) for individual contaminant species (e.g., cadmium, zinc, lead; Malusis et al., 2003; Shackelford et al., 2003).

When GCLs exhibit semipermeable membrane behavior, the diffusion of aqueous chemical species (both inorganic and organic) through the GCL will decrease as the membrane efficiency of the GCL increases, so at 100 percent membrane efficiency diffusion of the chemical species is effectively prevented. Thus, GCLs that act as semipermeable membranes perform better as barriers than those that do not. The potential existence of membrane behavior in GCLs adds an order of complexity to the analysis of chemical transport through GCLs that may not be warranted in practical applications, particularly since ignoring the existence of membrane behavior will result in conservative (higher) estimates of contaminant mass flux (Malusis and Shackelford, 2004). However, failure to recognize and account for the existence of membrane behavior also can affect the accuracy of predictions of contaminant mass flux through GCLs, as well as the accuracy of the measurement of transport parameters, which may be important in predicting long-term impacts.

In summary, laboratory and field observations of GCL performance under a variety of conditions suggest that although GCLs alone are not an effective barrier to diffusive transport, they can be effective replacements for low-permeability soil layers as operational barriers to advective transport, provided the integrity of the GCL is maintained during installation and operation. Further, although desiccation may be of concern to GCL integrity, particularly when coupled with cation exchange, it can be mitigated effectively by providing sufficient overburden pressure (e.g., in liner systems). Finally, the adverse effects of chemical incompatibility can be mitigated to some extent or delayed by prewetting the bentonite in the GCL or by permeating the GCL under sufficient overburden pressure.

4.4 DRAINAGE LAYERS

Modern engineered resistive barrier systems generally contain both a low-permeability barrier layer and an overlying high-permeability granular and/or geosynthetic drainage layer. The rate of leakage through a barrier layer increases as the hydraulic head on the barrier layer increases. For example, the rate of leakage through a 1-m-thick compacted clay liner with a hydraulic conductivity of 1×10^{-9} m/s would increase by about 300 percent if the head on the liner increases from 0.3 to 3 m (i.e., assuming a 1-m-thick clay liner with zero head at the base).

The effectiveness of the barrier system depends not only on the ability of the resistive low-permeable barrier layer to restrict flow but also on the ability of the drainage layer to limit the hydraulic head on the barrier by providing a path for effluent flows of liquid away from the barrier. Inadequate drainage layer capacity is a primary short-term performance concern as leachate flows are generally highest immediately after construction. Medium- and long-term performance concerns include the potential for reduction of capacity from clogging of the drainage layer or overlying filter due to soil infiltration, biological growth, and mineral precipitation. Geosynthetic drainage layers, which are relatively thin, are also susceptible to a reduction in flow capacity caused by penetration of adjacent geosynthetic or soil materials or by compression of the layer itself.

4.4.1 Field Performance

Measurements of field performance of drainage layers in leachate collection or cover systems are much less common than for barrier layers. Most of the available information is indirect, such as indications of leachate mounding or a significant reduction in pumping rates over time in a leachate collection system. The most significant concern with field performance of drainage layers over time is the buildup in head on the underlying barrier layer.

Clogging of leachate collection systems has been observed in landfills with a wide range of collection systems, ranging from French drains (gravel drains at spacing of 50 to 200 m leading to a collection sump) to continuous sand and gravel layers and in systems both with and without geotextile filters (see Table 4.4 and Rowe, 2005). Clogging typically involves a combination of biological clogging, mineral precipitation, and an accumulation into the clog material of fines, as discussed in detail by Rowe (2005). Several field examples of clogging have been reported. For example, field exhumations found that the upper portion of the sand protection layer over a compacted clay liner became clogged within 4 years and thus did not contribute to the hydraulic performance of the collection system (Reades et al., 1989; Barone et al., 1993). Koerner and Koerner (1995) reported clogging of a sand layer where, after 10 years, the hydraulic conductivity dropped three orders of magnitude from 4×10^{-4} m/s to 2×10^{-7} m/s and leachate was flowing through the waste rather than through the sand. Koerner et al. (1994) and Koerner and Koerner (1995) reported excessive clogging in two cases where the geotextile was wrapped around either the perforated pipe or the gravel in a drainage trench and some clogging in a third case. A detailed description of a case history where clogging occurred in a leachate collection system is provided in Box 4.4.

Field evidence of leachate collection system performance commonly exhibits deposits of both biofilm and inorganic mineral precipitate on the surface of granular materials. These deposits reduce the drainable porosity and hydraulic conductivity of the drainage material. Clogs of minerals precipitate have been reported to contain over 50 percent calcite, 16 to 21 percent silica, 8 percent iron, and 5 percent manganese (e.g., Brune et al., 1991; Fleming et al., 1999). Calcite ($CaCO_3$) was also the dominant mineral in clog scale obtained from a leachate collection pipe in a Florida landfill that received incinerator ash and municipal solid

TABLE 4.4 Observations from Exhumation of Leachate Collection Systems

Waste Type, Age; Leachate	Collection System Design	Key Observations
1. MSW after about 4 years Performance: adequate at time of exhumation	Waste over 50-mm relatively uniform gravel blanket underdrain	50–100% loss of void space near pipe; *k* of gravel decreased from ~10^{-1} m/s to ~10^{-4} m/s
2. MSW after about 4 years Performance: adequate at time of exhumation	Waste over woven geotextile over 50-mm gravel blanket underdrain	Substantially less clogging than observed in Case 1
3. MSW after about 10 years Performance: No flow in LCS; high leachate mound	Toe drain only comprised of a trench with 600 mm of crushed (6 to 30 mm) gravel around geotextile-wrapped pipe	Substantial clogging of gravel; *k* reduced from 2.5×10^{-1} m/s to 1.2×10^{-4} m/s. *k* of geotextile dropped from 4.2×10^{-4} m/s to 3.1×10^{-8} m/s
4. ISS (included slurried fines) after about 0.5 years Performance: No flow in LCS; high leachate mound	Waste over protection sand over geotextile over pea gravel blanket underdrain; perforated pipe wrapped in geotextile	Geotextile wrapping around perforated pipe excessively clogged (*k* dropped from 4.9×10^{-3} m/s to 4.4×10^{-8} m/s)
5. MSW after about 6 years Performance: no flow of methane into extraction wells	Injection wells—100-mm perforated PVC pipes wrapped in geotextile	Geotextile wrapped around extraction wells excessively clogged; *k* dropped from 2.3×10^{-3} m/s to 7.5×10^{-7} m/s
6. MSW and incinerator ash after 2.5 years Performance: no flow in LCS	Class 1 double-lined landfill	Layer of hard mineral (calcite) substance that filled ~25–100% of the collection pipes
7. MSW and LIW after 12 years Performance: high leachate mound	Waste over a 300-mm-thick gravel layer (20–40 mm) blanket underdrain	Clog deposits consisted of calcite and ferrous sulfur; residual drainable porosity values ranged from 25 to 50%

NOTES: LCS = leachate collection system; MSW = municipal solid waste; LIW = light industrial waste; ISS = industrial solids and sludge.
SOURCE: Summarized from Rowe (2005). Cases 1 and 2 are from Fleming et al. (1999); Cases 3, 4, and 5 are from Koerner and Koerner (1995); Case 6 is from Maliva et al. (2000); Case 7 is from Bouchez et al. (2003).

waste (Maliva et al., 2000) and in clog scale obtained from a leachate collection pipe in the United Kingdom (Manning, 2000).

Unpublished laboratory studies by Rowe suggest that geonet drainage layers are susceptible to clogging in a manner similar to granular drainage layers. However, much less research has been conducted on these systems. The potential for degradation of flow capacity in geonets—which could arise from clogging, sustained overburden stress, and/or elevated temperatures—is an area that requires more research and collection of field data.

Field evidence for clogging in cover drainage layers is mostly related to slope failures. Clogging by fine particles and/or roots has reduced the drainage capacity of a cover drainage layer and led to slope failure due to increased pore water pressures in several cases (e.g., Koerner and Soong, 1998; Bonaparte et al., 2002; Richardson and Pavlik, 2004). Drainage layers can also become temporarily clogged with ice during a cold spell (Bonaparte et al., 2002) or inadvertently clogged with soil during construction of an adjacent landfill cell (Gilbert, unpublished data).

4.4.2 Geotextile Filters

The use of geotextiles as filters in leachate collection systems has been controversial. Geotextiles will experience some amount of particulate, biological, and chemical clogging, and, as a consequence, a decrease in hydraulic conductivity of up to four to five orders of magnitude could occur. However, based on published data (Koerner et al., 1994; Rowe, 1998), it is unlikely that the hydraulic conductivity of a typical nonwoven needle-punched geotextile would be below 4×10^{-8} m/s for normal conditions and more likely would be on the order of 1×10^{-7} m/s or higher. Nonetheless, clogging can be quite problematic if the geotextile is wrapped around leachate collection pipes (Rowe et al., 2004). On the other hand, this level of clogging may not have a significant impact if it is used as part of a blanket drain (McIsaac and Rowe, 2006), especially if measures are taken to control perching of leachate above the geotextile (Rowe et al., 2000).

There is some evidence to suggest that monofilament woven geotextiles selected in accordance with Giroud's (1996) recommendations are likely to experience less clogging and reduction in hydraulic conductivity with time than the needle-punched nonwoven or slit film geotextiles normally used in practice. Giroud's recommendations are based on the premise that minimizing clogging of the filter is the objective. This may indeed be the case for some design situations (e.g., wrapping the geotextile around a pipe). However, while excessive clogging is undesirable, the processes that cause clogging provide some beneficial leachate treatment that (1) decreases the potential for clogging at more critical zones (e.g., near collection pipes) and (2) reduces the level of leachate treatment required after removal of leachate from the landfill (Rowe, 2005). Based on the available evidence, it appears that suitably selected geotextiles used as a filter above a coarse gravel leachate collection layer can have a

BOX 4.4 Case History on Clogging in a Leachate Collection System

This case history illustrates that leachate collection systems can clog, giving rise to both a significant leachate mound and a rapid increase in temperature on the liner. The Keele Valley Landfill, described in Box 4.3, is divided into four quadrants (Rowe, 2005). Stage 1 (northeast portion) was constructed between 1983 and 1985. Stage 2 (northwest portion) was constructed between 1986 and 1988. Construction of Stages 3 (southeast portion) and 4 (southwest portion) was started in 1988 and 1990, respectively, and completed in 1994. The leachate collection system in Stages 1 and 2 consists of a series of French drains (50-mm washed, crushed gravel mounds 0.5 m high and spaced 65 m apart) draining into four main 150-mm-diameter, perforated collection pipes (8-mm-diameter perforations) encapsulated in 40-mm washed, crushed gravel mounds 0.5 m high and 200 m apart. In Stages 3 and 4 the leachate collection system consists of a granular drainage blanket (0.3-m thick, 50-mm washed, crushed gravel) draining into the four main leachate collection pipes. The four primary pipes are 900 to 1,200 m long and run predominantly northwest to southeast across the full length of the landfill and drain to a common holding tank.

Field exhumations (Reades et al., 1989; Barone et al., 1993) found that the upper portion of the sand "protection" layer over the liner became clogged within the first 4 years and did not contribute to the hydraulic performance of the collection system. In addition to visual evidence of clogging in the upper portion of the sand layer, the absence of flow in the sand layer was clearly shown in the diffusion profile, which demonstrates that the sand layer was acting as part of the liner in terms of a "diffusion barrier." A particularly notable observation is that both the inorganic contaminant chloride and several organic contaminants (especially toluene) exhibit diffusion profiles through both the sand and the clay. Such profiles would not be found if there was any significant flow in the sand layer.

Exhumation of portions of the continuous gravel drainage blanket in later stages of the landfill (Fleming et al., 1999) indicated a drop of three orders of magnitude in the hydraulic conductivity of relatively uniform 50-mm gravel near the leachate collection pipe after 4 years (although the hydraulic conductivity was still sufficient to transmit leachate without the development of a significant leachate mound). Clogging was observed to be substantially less in areas where a geotextile filter was used between the waste and the gravel than in areas where the waste was in direct contact with the gravel. The main header line leading to the main manhole was found to be partially occluded by solid, biologically induced precipitant, which included some particulate material that had become part of the clog and had formed within the pipe. In the spring of 2001 the main header was so occluded with clog material that a pipe observation camera could not enter the pipe.

The liner temperature above the compacted clay liner has been monitored at four locations over a 21-year period (Barone et al., 2000; Rowe, 2005). In Stage 1 the temperature increased slowly and remained low (average 12°C) for the first 6 years and then increased rapidly as the leachate mound grew from 1 to 6.5 m. Subsequently, the temperature stabilized at about 37°C, although the leachate mound continued to increase to about 8.4 m (the leachate mound has subsequently reduced slightly, likely due to installation of the final cover). In Stage 2 the temperature remained relatively constant and low (average 10°C) for the first 5 years and then increased rapidly to 24°C over the next 6 years as the leachate mound grew from about 1 to 5 m. The temperature has continued to increase to 35°C (with no sign of stabilization to date), even though the leachate mound has dropped to about 3 m (due primarily to installation of the final cover). In Stages 3 and 4 the blanket drain leachate collection system continues to function well, although the temperature rose from 15°C in 1998 to 31 to 32°C in 2004. At monitors where the leachate head is less than 0.3 m, 8 to 12 years was required for the temperature on the liner to reach 20°C but only another 3 to 4 years to reach 30°C. The current temperature appears to have stabilized at about 35–40°C.

beneficial effect of extending the service life of the leachate collection system (Fleming et al., 1999; Rowe, 2005; McIsaac and Rowe, 2006).

4.4.3 Mechanisms of Biological and Chemical Clogging

Field evidence suggests that biological and chemical clogging is a significant concern for leachate collection systems. Biological clogging is initiated by microorganisms that are suspended in leachate attaching to and colonizing the porous media surfaces as a biofilm (Rowe et al., 2004). The development of the biofilm is a function of the growth rate of the microorganisms, substrate concentration, attachment of microorganisms from the leachate onto the surface of the porous medium, and the detachment of microorganisms from the biofilm into the passing leachate. For MSW landfills, the precipitation of Ca^{2+} as $CaCO_3$ is directly correlated with the biodegradation of organic acids.

Available information (Rowe, 2005) indicates that development of a biofilm occurs relatively quickly and then, with time, changes from a soft slime to a slime with hard particles (sand-size solid material in a soft matrix), to a solid porous concretion of a coral-like "biorock" structure (VanGulck and Rowe, 2004). During the early phases of clog development, the biofilm is relatively easy to clean from leachate collection pipes (e.g., by pressure jetting). However, once the biofilm becomes sufficiently established to cause significant precipitation of $CaCO_3$, the inorganic film becomes firmly attached to the adjacent media (e.g., perforation) and the inside of pipes (Fleming et al., 1999) and is very difficult to remove. This behavior implies that leachate collection pipes in landfills need to be regularly cleaned to remove the soft biofilm in its early stages of development. The rate of clog development has been found to accelerate with time, and hence the rate of inspection/cleaning required in pipes may need to increase with time as biofilms develop and as the concentration of volatile fatty acids in the leachate (or chemical oxygen demand) increases.

Due to the level of "treatment" that occurs in a leachate collection system before the leachate reaches the collection point, end-of-pipe analyses of volatile fatty acids and Ca^{2+} concentrations in the leachate only indicate the condition at the collection point and provide little evidence regarding the nature of the leachate that enters the system (Rowe, 2005).

Use of carbonate gravel in the drainage layer has been suggested as unsuitable because it could dissolve and contribute to subsequent calcite crystallization on alkaline biofilms covering drainage gravel (e.g., Brune et al., 1991). However, a considerable body of evidence now suggests that calcium carbonate is generally supersaturated in landfill leachate very early in the life of the landfill such that it is unlikely dissolution would occur under these conditions (e.g., Rittmann et al., 1996; Owen and Manning, 1997; Jefferis and Bath, 1999; Manning and Robinson, 1999; Bennett et al., 2000). The evidence points to Ca^{2+} leaching out of the waste and being transported to the point of deposition by the leachate, while the CO_3^{2-} predominantly forms from the mineralization of organic carbon by microorganisms (Brune et al., 1991; Owen and Manning, 1997; Bennett et al., 2000).

An understanding of basic clogging mechanisms from laboratory and field studies led to the development of a simple method for estimating the rate of clogging of different collection system designs that uses calcium as a surrogate for other clog materials (e.g., magnesium, iron) that may deposit within the drainage layer (Rowe and Fleming, 1998; Rowe et al., 2004). Although this methodology is dependent on porosity, no distinction is made for different particle sizes and specific surface areas; thus, it should be used with caution and only for relatively uniformly graded granular material (Rowe, 2005).

New understanding of clogging has allowed the development of a numerical model, BioClog, to simulate clogging in both column experiments (Cooke et al., 2005) and two-dimensional flow systems. Reactive chemical transport is modeled with consideration of biological growth, mineral precipitation, and attachment and detachment of suspended solids. The porous medium is represented as a fixed-film reactor, and changes in porosity resulting from the development of biofilm and inorganic films (e.g., calcium carbonate) can be directly calculated based on a geometric model. The corresponding changes in hydraulic conductivity are then deduced based on empirical relationships derived from laboratory tests. For two-dimensional systems a flow model is coupled with the transport model.

The BioClog model is sufficiently mature to be used to predict clogging in landfill drainage systems. However, because of the highly coupled and complex nature of the model, it is unlikely to be used in routine engineering practice. Nonetheless, models such as this provide insight that can both improve interpretations of laboratory test results and field monitoring measurements and aid in evaluating the likely difference in the performance of different types of leachate collection systems.

4.4.4 Summary

In summary, drainage layers are important barrier components for reducing leachate head on liners and covers, thereby reducing advective transport and enhancing slope stability. However, field measurements of performance of drainage layers in leachate collection or cover systems are much less common than for barrier layers. The available evidence indicates that drainage layers can be susceptible to clogging from a variety of mechanisms, including chemical and biological precipitation and infiltration of fine particles. More research is needed on the field performance of drainage layers.

4.5 EVAPOTRANSPIRATIVE BARRIERS

Performance concerns for evapotranspirative barriers include the following:

- Short term: inadequate material, inadequate thickness, and inability to establish vegetation, which lead to excessive infiltration.
- Medium and long term: inadequate storage capacity for infiltration, inability to sustain vegetation, and the development of cracks and other secondary permeability features, which lead to excessive infiltration; erosion and invasion of inappropriate vegetative species, which lead to intrusion and/or exposure of the waste.

Evapotranspirative barriers have a similar capability as other vegetative soil layers to control gas emissions and resist intrusion and erosion. Pan lysimeters are the primary method used to evaluate the infiltration performance of evapotranspirative covers. Lysimeters provide an accurate representation of this performance, subject to the boundary conditions imposed by the lysimeter installation, and thus can provide valuable information on their effectiveness as a capacitive barrier to infiltration. Two case histories of lysimeter measurements illustrating the effectiveness of evapotranspirative barriers in controlling infiltration are given in Box 4.5. Lysimeter test pads are being used to monitor the performance of these barriers as elements of a waste containment system, although lysimeter measurements have shortcomings when used in this way (see Section 3.2.3).

Few conclusive data are available on the performance of evapotranspirative covers in temperate climates. However, available field evidence suggests that these covers can be an effective alternative to compacted clay or composite covers in arid and semiarid climates where evapotranspiration is an important component of the water balance and in wetter climates where it is not essential to minimize infiltration. Since most test sites being monitored were established in the late 1990s, it is difficult to effectively predict the performance of evapotranspirative barriers over extended periods of time. However, natural analogs can be used to estimate the long-term performance of cover systems (Caldwell and Reith, 1993). For instance, natural landscapes that have remained stable for hundreds or thousands of years exist in arid and semiarid regions, such as a desert varnish crust in the southwestern United States, and may provide analogs for erosion control. Geological and hydrological data from these established landscapes suggest that long-term infiltration to the subsurface is negligible. Studies of natural analog sites could provide valuable insights on the long-term efficacy of evapotranspirative systems in arid and semiarid climates as well as the long-term performance of other types of cover systems in other climate regimes.

4.6 VERTICAL BARRIERS

Vertical waste containment barriers (cutoff walls) have been constructed using soil-bentonite, cement-bentonite, soil-cement-bentonite, Portland cement concrete (PCC), sheet piling, and geomembrane panels. In addition, gravel-filled interceptor and extraction trenches can be effective vertical barriers by cutting off the continuity of lateral advective and diffusive flow and removal of contaminants from the system. These types of barriers are commonly incorporated as components of site remediation and cleanup systems.

Slurry wall construction methods have been used most extensively for vertical barrier construction. Deep soil mixing and jet grouting methods have also been used, although deep soil mixing is limited by the ability to achieve sufficiently low saturated hydraulic conductivity and the use of jet grouting is limited by concerns about continuity of the barrier. As few data specific to vertical barrier walls are available for evaluation of their effectiveness as waste containment barriers, unambiguous conclusions concerning their performance cannot be reached. Some relevant considerations, findings, and data that provide preliminary indications of how well they may meet their objectives are summarized below.

Short-term performance concerns for soil-bentonite and cement-bentonite vertical cutoff walls include defective material and "windows" caused by caving or trapped low-quality material at joints between panels. Extensive experience using these types of walls for groundwater control in excavations, however, indicates that if properly designed and constructed, they provide excellent barriers against groundwater flow under heads ordinarily considerably higher than those likely to be encountered in waste containment applications. Medium- and long-term performance concerns include property changes due to chemical incompatibility and to desiccation above the water table for soil-bentonite walls, cracking for soil-bentonite and cement-bentonite walls, and chemically-induced deterioration (e.g., sulfate attack) for cement-bentonite walls. Failure of vertical cutoff walls can be attributed to two primary mechanisms: (1) defects in the constructed wall, including entrapped sediment, improperly mixed backfill, inadequate excavation of the key, and formation spalling (Evans, 1991); and (2) changes in the material properties after the wall has been constructed. Inadequate excavation for keying the vertical wall into the underlying impervious layer is among the most common problems when using cement-bentonite walls. Property changes within cutoff walls can be caused by the same mechanisms that affect compacted clay liners: desiccation, freeze/thaw, chemical incompatibility, and excessive deformations.

Short-term concerns for PCC cutoff walls are similar to those for cement-bentonite walls, especially if they are constructed using tremie placement methods in slurry trenches. Formed Portland cement concrete walls are susceptible to a variety of short-term cracking mechanisms. Longer-term concerns for PCC walls include cracking caused by several possible mechanisms and corrosion of reinforcement.

Driven sheet pile walls can be an attractive alternative for construction of vertical waste containment barriers because of their relative ease of installation and the imperviousness of the sheet piles themselves. Sheet pile walls include both steel sheet piles and polymer sheet piles. Leakage through the interlocks and the possibility that some piles may be driven out of their interlocks during installation are major limitations on the short-term performance of sheet pile walls. Other factors that potentially affect the performance of sheet pile walls include the ability to effectively key sheet piles into low-permeability soils and rocks, and the potential limitation on the depth of penetration. Corrosion may limit the long-term effectiveness of steel sheet piles.

Geomembrane panels may also be used as vertical barriers. As is the case for steel sheet pile walls, one of the most critical factors that affect short-term performance of geomembrane panels is the integrity of the interlocks that connect the geomembrane panels in the barrier. Medium- and long-term durability concerns for vertical geomembrane panels are similar to those for geomembrane liners.

4.6.1 Soil-Bentonite Walls

Measurement of the hydraulic conductivity of soil-bentonite cutoff walls can be performed in the laboratory or in situ. Laboratory tests can be performed on samples recovered using thin-walled tubes from the wall after construction. The tests can be conducted in consolidometer permeameters or flexible-wall permeameters, although this type of laboratory testing may be difficult because the soft nature of the materials makes sample recovery and extrusion of the sample difficult (Britton et al., 2004). Laboratory tests on remolded specimens created using bulk samples of wall backfill recovered during wall construction are likely to be useful only if the testing procedures provide a sample state and confinement conditions that are representative of the material in situ. In situ hydraulic conductivity tests can be conducted within the wall following construction. In situ tests include slug tests, piezocone soundings with pore pressure dissipation measurements, and large-scale pumping and injection (Britton et al., 2004).

A comparison of laboratory and field methods to evaluate the hydraulic conductivity of soil-bentonite backfill in pilot-scale cutoff walls showed that laboratory tests performed on remolded and thin-walled tube samples, respectively, yielded

BOX 4.5 Case Histories on Performance Assessments for Evapotranspirative Barriers

These case histories illustrate the field performance of evapotranspirative barriers. Evapotranspirative barriers were constructed and monitored as part of the Environmental Protection Agency's Alternative Cover Assessment Program (Albright et al., 2004). All test sections were installed within a 10- to 20-m instrumented pan-type lysimeter for direct measurement of surface runoff, soil-water storage, and percolation. The base and sidewalls of each lysimeter consisted of polyethylene geomembrane. A geocomposite drainage layer was placed in the base of the lysimeter to route water to a collection and measurement system. Diversion berms were constructed around the perimeter of each test section to prevent run-on and to collect runoff for measurement. The water content of cover soils was measured with time domain reflectometry probes placed at varying depths in the cover profile. Thermal dissipation sensors were used to measure soil-water matric potential. A description of installation methods for the lysimeters is given in Benson et al. (1999). Surface runoff (*SRO*), percolation (P_r), and precipitation (*P*) were all measured directly, whereas soil-water storage (*SWS*) was calculated by integrating volumetric water content measurements over the volume of the lysimeter. Evapotranspiration (*ET*) was calculated according to the water balance equation $ET = P - SRO - P_r - L_o - \Delta SWS$, where L_o is lateral drainage. The test sections described in this case history did not contain internal drainage layers such that lateral drainage was assumed as nil ($L_o = 0$).

Boardman, Oregon. The alternative cover is located at the Finley Buttes Regional Landfill in Boardman, Oregon. Boardman receives an average annual precipitation of 225 mm with a precipitation/potential evapotranspiration (*P/PET*) ratio of 0.23, which corresponds to a semiarid climate (Albright et al., 2004). The weather in Boardman is warm and dry in the summer and cool and wet in the winter, with over two-thirds of the annual precipitation occurring between November and March. The alternative cover is a monolithic cover consisting of a storage layer over an interim cover (Figure 4.5). Construction of the test section was completed in November 2000, and data collection began in December 2000. The test section was seeded in December 2000 with a mixture of indigenous grasses, including crested wheatgrass, alfalfa, and clover (Bolen et al., 2001). The surrounding native vegetation is described as shrub-steppe (i.e., sagebrush and grasses).

Polson, Montana. The alternative cover at Polson, Montana, is located at the Lake County Landfill. Polson has an average annual precipitation of 380 mm and a *P/PET* ratio of 0.58, which corresponds to a subhumid climate (Albright et al., 2004). The alternative cover is a capillary barrier cover consisting of topsoil overlying a sandy silt storage layer overlying a capillary break comprised of coarse-grained silty sand, which in turn overlies a sandy gravel layer that serves as an interim cover and gas vent (Figure 4.5; Roesler et al., 2002). Construction of the test sections was completed in October 1999, and data collection began in November 1999. The test sections were seeded in March 2000 with a combination of native and introduced vegetation, including bluegrass, wheatgrass, alfalfa, and prickly rose shrubs (Bolen et al., 2001).

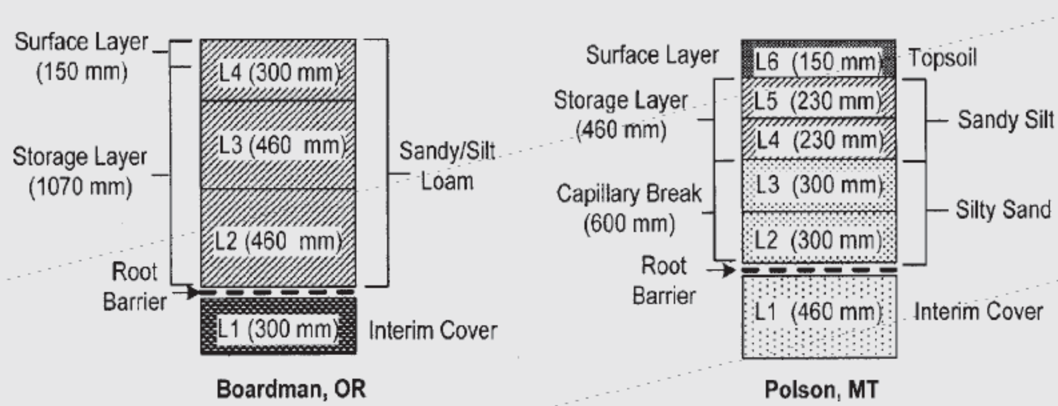


FIGURE 4.5 Schematic cross sections of alternative earthen final covers discussed in this case history. SOURCE: Ogorzalek (2005).

Water Balance Measurements. The measured water balances at both sites are shown in Figure 4.6. Although several factors differ, including the periods of record (3 years for Boardman and 4 years for Polson), the type of alternative cover, and the amount of precipitation, some consistent trends can be observed. First, the surface runoff at both sites is relatively insignificant. Second, the vast majority of the precipitation at each site is taken up by evapotranspiration, emphasizing the importance of vegetation in evapotranspirative barriers. Finally, both alternative covers apparently performed exceptionally well over the period of record, with no measured percolation occurring at Boardman and only 0.8 mm of cumulative percolation occurring at Polson.

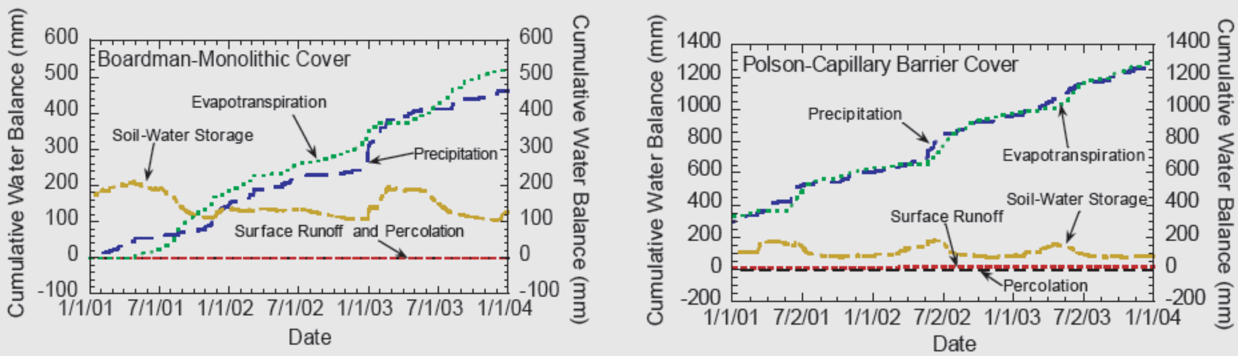


FIGURE 4.6 Measured water balances for alternative covers at Polson, MT, and Boardman, OR. SOURCE: Modified from Shackelford (2005); data from Bohnhoff (2005) and Ogorzalek (2005).

the lowest values for hydraulic conductivity, followed by the in situ methods (piezocone and piezometer; Britton et al., 2004). Large-scale pumping/injection tests performed using wells located in the aquifer contained by the cutoff wall gave higher values than either the laboratory or field measurements, suggesting that isolated defects within the wall exert a significant influence on barrier effectiveness. Testing of soil-bentonite walls using slug tests (instantaneous change in water level, followed by monitoring until the water level returns to static conditions) with a push-in piezometer tip has been recommended as an efficient method for evaluating soil-bentonite wall hydraulic conductivity that limits disturbance while still testing a relatively large volume of material (Britton et al., 2005a; Choi and Daniel, 2006a, 2006b). Nevertheless, such tests do not ensure detection of isolated defects that could dominate the overall effectiveness of the barrier, as suggested by the large-scale pumping tests.

The possible effects of chemical incompatibility, described above for earthen barriers and geosynthetic clay liners, may also apply to soil-bentonite cutoff walls. However, factors such as the type of soil used to create the soil-bentonite backfill and stress conditions within the wall may also be important in the case of soil-bentonite walls because of the lack of significant waste overburden.

4.6.2 Cement-Bentonite Walls

Cement-bentonite is sometimes used as an alternative to soil-bentonite for construction of subsurface vertical barriers. Cement-bentonite is also sometimes used as a flowable low-permeability backfill around pipes carrying leachate or other contaminated liquids and in trenches. Cement-bentonite barriers are much more popular in Europe than in the United States, where soil-bentonite barriers predominate because of cost considerations and concerns about the hydraulic conductivity of cement-bentonite mixtures. The technical basis for resistance to the use of cement-bentonite as a barrier material in the United States is the difficulty of achieving a sufficiently low initial hydraulic conductivity. Cement-bentonite mixes usually have an initial hydraulic conductivity on the order of 10^{-8} and 10^{-9} m/s, whereas a value no greater than 10^{-9} m/s is typically employed as the standard for vertical barriers. However, extensive research and experience in the United Kingdom and Europe show that a hydraulic conductivity of $<10^{-9}$ m/s can be achieved by supplementing the cement-bentonite mixture with ground granulated blast furnace slag (Jefferis et al., 1999). Although the initial hydraulic conductivity of a cement-bentonite-slag mixture may be $>10^{-9}$ m/s upon initial emplacement, laboratory testing of both archived samples and cores recovered from walls shows that the hydraulic conductivity subsequently decreases and routinely achieves a value of $<10^{-9}$ m/s within a year (Jefferis, 1995), as illustrated in Figure 4.7. In the United States, fly ash is often added to cement-bentonite mixes to lower the initial hydraulic conductivity below 10^{-9} m/s. Blast furnace slag has also been used in some U.S. installations (Opdyke and Evans, 2005) to decrease the permeability of cement-bentonite mixes.

Limited information on chemical compatibility suggests that both the short-term resistance and the medium-term

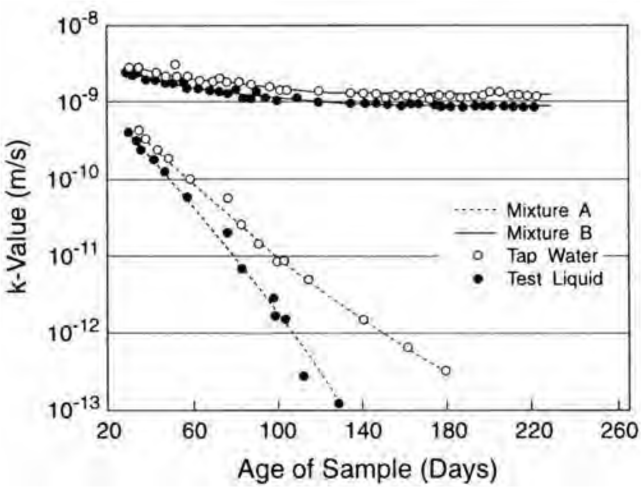


FIGURE 4.7 Hydraulic conductivity of soil-bentonite (mixture A) and cement-bentonite (mixture B) mixtures from Jefferis (1995). SOURCE: Rumer and Mitchell (1995).

resistance of cement-bentonite mixtures to degradation by organic solvents are good. Compatibility tests conducted by Manassero et al. (1995) on cement-bentonite mixtures with pure ethanol indicate that the ethanol did not have a significant effect on the hydraulic conductivity of the test specimens because the intrinsic permeability of the specimens with ethanol and water were similar. Similarly, relatively low effects were observed on the hydraulic conductivity of cement-bentonite mixtures tested in the laboratory with leachate from a phosphogypsum (calcium sulfate) landfill (pH = 2.0) when compared to permeation with water (Fratalocchi et al., 2005). Diffusion coefficients similar to those for compacted clay soils were reported for cement-bentonite mixes (Manassero et al., 1995).

Construction quality assurance procedures for cement-bentonite walls are similar to those for soil-bentonite vertical barriers. Besides monitoring the placement of the cement-bentonite in the slurry-filled trench used for wall construction, cylinders of the cement-bentonite mix are periodically recovered, cured for 7 to 28 days, and then tested for hydraulic conductivity. Recommendations for the design of cement-bentonite walls and CQA procedures are provided in Manassero et al. (1995). Cement-bentonite vertical barriers have been employed for environmental remediation in the United Kingdom since the late 1960s, and as of 1994 no failures of these types of barrier walls had been reported (Rumer and Mitchell, 1995).

4.6.3 Portland Cement Concrete Barriers

The intact (uncracked) hydraulic conductivity of PCC depends primarily on the water/cement ratio, the degree of hydration, and supplemental cementitious materials or mineral admixtures (e.g., fly ash, silica fume, blast furnace slag) used in the concrete mix. The bulk hydraulic conductivity depends on the intact hydraulic conductivity and microstructural features, including cracks in the concrete, concrete diffusivity, the internal pore system of the concrete, and the character of the cement paste-aggregate transition zone. High-strength concrete generally has a relatively low intact hydraulic conductivity, suggesting that it can provide a high degree of effectiveness as a barrier to advective flow. However, its brittleness and susceptibility to plastic shrinkage and restrained cracking create secondary features that reduce the bulk hydraulic conductivity. Hydraulic conductivity on the order of 1×10^{-10} cm/s is readily achievable in uncracked PCC with a low to moderate water-cement ratio (Hearn et al., 1994). The inclusion of supplemental material, such as blast furnace slag, silica fume, and fly ash, in addition to or as a partial replacement for Portland cement, further decreases the intact hydraulic conductivity of concrete (Thomas et al., 1999; IAEA, 2001). However, relatively few through-going cracks can increase the bulk hydraulic conductivity by several orders of magnitude compared to the intact value (Snyder, 2000).

There are two main approaches to mitigating the impact of cracking on the performance of concrete barriers: (1) accept that cracking will occur and engineer a containment system that meets performance requirements even after the concrete cracks or (2) develop measures to minimize cracking. Cracking mechanisms in PCC include plastic shrinkage and restrained shrinkage cracks that develop during curing, settlement cracking that occurs during placement, flexural cracking caused by deformation after placement, thermal cracking, drying shrinkage cracking, cracking caused by corrosion and subsequent expansion of steel reinforcement, and autogeneous shrinkage cracks caused by chemical shrinkage stresses that develop after curing (ACI, 2006). The use of coarser cement paste with longer curing times may mitigate cracking associated with curing. Cracks that develop as a result of placement conditions (e.g., plastic shrinkage, settlement) can be minimized by using good construction practices, but some cracking of this sort may be inevitable. Similarly, some, but not all, thermal cracking and flexural cracking can be mitigated through reinforcement. Finally, cracking may be mitigated by incorporating supplemental water into the concrete mixture. Approaches that attempt to employ this mechanism include the use of lightweight aggregates saturated with water and the inclusion of absorbent polymers saturated with water into the concrete mixture (Bentz and Jensen, 2004). However, these approaches are still experimental, and little information is available on how they affect long-term performance.

Cracking caused by ion transport is a problem for long-term performance of PCC barriers. Both sulfate and chloride ion diffusion can induce degradation and cracking in concrete. While the use of ASTM Type V cement can slow the onset and progression of sulfate degradation, the ability of sulfate-resistant concrete to prevent cracking from sulfate ion diffusion for hundreds to thousands of years is unknown (Ferraris et al., 1997). Chloride diffusion is perhaps an even more serious problem for steel-reinforced concrete. Chloride ions that move through concrete by diffusion and advection can induce corrosion of metallic reinforcement and generate expansion products. The permeability of concrete to chloride may be inhibited by the use of supplemental cementitious materials and other admixtures. Techniques for measuring the chloride permeability of concrete include ponding tests and rapid chloride permeability measurement (Mobasher and Mitchell, 1988).

The corrosion of reinforcement and the associated expansion products can lead to both expansion-induced stresses and loss of tensile capacity of the reinforcement and thus make the concrete more susceptible to cracking. Nonmetallic reinforcing elements (e.g., fiber, fiber-reinforced plastic, or polymer grid reinforcement) may be used to mitigate problems with corrosion of steel reinforcement. However, experience with this type of reinforcement for long-term durability is limited. Other mitigation measures include the use of ASTM Type V low-alumina cement to minimize sulfate-induced degradation and/or blended cements (e.g., cements with supplemental cementitious materials and mineral admixtures to reduce concrete permeability). Extensive laboratory testing indicates that supplemental cementitious admixtures combined with relatively low water-cement ratios reduce both concrete permeability and chloride diffusivity (IAEA, 2001). However, in some cases diffusion may be beneficial. For example, fine cracks may close through autogeneous healing caused by transport of sulfate and other minerals with low gradients.

The susceptibility of PCC barriers to cracking is demonstrated by the difficulty in controlling vapor transmission through building slabs (EPA, 2002a). Indoor air pollution caused by transmission of volatile organic compounds through concrete foundation slabs is a persistent problem at sites underlain by contaminated groundwater plumes. When buildings are constructed on top of or within MSW landfills that generate methane, building codes often require that a geomembrane be placed beneath the slab to control vapor intrusion. Although incorporation of supplemental water or beneficial material (e.g., blast furnace slag) into the concrete mixture, use of nonmetallic reinforcement, and more rigorous CQA procedures may limit cracking in concrete slabs used as barriers for waste containment, some cracking appears to be inevitable with current technology and must be accommodated in the design of the barrier.

Construction quality assurance of PCC barriers generally consists of strength testing of cylinders created at the time the concrete mix is poured. Although these tests can detect deficiencies in the concrete mix, they are not really suitable for detecting cracks that develop in the concrete during or after curing. Furthermore, statistical methods used to evaluate the test results are generally targeted at evaluating the overall strength of the concrete mass, not its hydraulic conductivity or susceptibility to cracking. However, there appears to be a good correlation between the strength and cracking resistance of PCC in the short term. Several nondestructive testing methods have been proposed as a means of PCC quality assurance. Seismic velocity testing is now used routinely to evaluate the integrity of cast-in-drilled-hole deep foundations. However, this method is aimed at identifying relatively large zones of defective concrete rather than the micro-cracking of concrete that is likely to impair the long-term integrity of a PCC barrier. Air permeability measurements (Kollek, 1989; Torrent, 1992; Lydon, 1993; Ismail et al., 2006) may provide a more direct measure of microstructural cracking in concrete.

4.6.4 Sheet Pile Barrier Walls

The primary short-term problem with the performance of sheet pile barriers is inability to achieve an adequate seal at the interlock between adjacent sheet piles. Several different sealing systems have been developed, including the use of hydrophilic material to line the interlocks and the provision of tubes and channels to facilitate grouting of the interlocks after driving is complete. The so-called Waterloo Barrier, a patented sheet pile system specially designed for environmental containment system applications that employs a sealable joint, is one of the more common systems.[1]

The results of two studies to evaluate flow through sheet pile walls used as vertical barriers were reported by McMahon (1995). In one study, bentonite slurry and an organic polymer were used to seal the interlocks between the panels of Waterloo Barrier sheet piles. With the bentonite slurry as an interlock sealant, a bulk hydraulic conductivity of 6×10^{-11} m/s was reported for the wall system, whereas the bulk hydraulic conductivity for the Waterloo sheet pile wall using the organic polymer sealant was reported to be less than 1×10^{-11} m/s. In the other study, a sheet pile barrier using Arbed sheet piles (European sheet piles with a different interlock system than sheet piles manufactured in the United States) was driven through a silty sand deposit into an underlying clay deposit and a field pumping test was performed using a pumped well and four observation wells. In this case, no interlock sealant was used and the measured bulk hydraulic conductivity of the wall system was 7×10^{-9} m/s.

Concerns about medium- and long-term performance of sheet pile vertical barriers include deterioration of the joint sealing material due to chemical incompatibility, material degradation, sustained hydraulic gradients, the impact of ground displacement and vibrations on the integrity of joint seals, corrosion of steel sheet piles, and stress cracking of polymer sheet piles. Few data on medium- and long-term field performance exist for sheet pile barriers. However, sheet pile containment systems are frequently used only for short-term containment, after which they can be removed

[1]See <http://www.waterloo-barrier.com>.

and reused, which is beneficial because of their greater cost compared to soil-bentonite vertical barrier containment systems.

4.6.5 Geomembranes as Vertical Barriers

Geomembranes have been used as vertical barriers either alone or in conjunction with other relatively low permeability materials, such as in composite cutoff walls (see Chapter 2) since about 1985. Koerner and Guglielmetti (1995) describe several case studies involving the use of geomembranes as vertical barriers, either alone or in conjunction with other materials (e.g., cement-bentonite slurry) or other technologies (e.g., jet grouting) for a variety of purposes, including as seepage cutoffs in earth dams and for containment of hazardous, petroleum, municipal, and contaminated drilling wastes. However, the case studies focused on the use and installation of the geomembranes, not their field performance. Similar to vertical sheet pile barriers, the primary short-term performance concern associated with geomembrane barriers is the integrity of the joints between panels. Several joint sealing systems have been developed, but little performance data on these systems is available. The installation of a vertical geomembrane barrier within a soil-bentonite wall is one approach that has been used to alleviate concerns over joint integrity. Long-term performance concerns with vertical geomembrane barriers are similar to long-term performance concerns associated with geomembrane liners.

4.6.6 In Situ Grouting for Vertical Barriers

In situ grouting is an established technology that has been used extensively in construction applications for two primary purposes: (1) to shut off seepage or create a barrier against groundwater flow and (2) to increase strength and decrease compressibility of soil formations for the purposes of increasing bearing capacity and stability and decreasing settlement and ground movement (Koerner, 1984; Karol, 2003). However, the use of in situ grouting for waste containment applications is relatively new. The primary short-term concern with the use of grouted barriers for environmental protection is the lack of continuity in the barrier. This concern, as well as the lack of a reliable method for evaluating the continuity of a grouted barrier, are perhaps the most significant reasons that grouting is not used more extensively for containment applications. The primary long-term concern with the performance of grouted barriers is the potential for degradation of the grout due to chemical incompatibility. While extensive laboratory tests have been performed for a wide range of chemical grouts, only a few chemical grouts have been tested in field studies, and those field studies have emphasized evaluation of placement methods, not barrier performance (Whang, 1995). As a result, data on the long-term field performance of vertical waste containment barriers created by in situ grouting are virtually nonexistent.

4.6.7 Gravel-Filled Trenches as Vertical Containment Barriers

Vertical trenches, typically up to a meter wide and filled with gravel, can cut off the lateral advective and diffusive flow of contaminants, while also facilitating removal and treatment or disposal of the contaminants. This system provides both an effective waste containment barrier and source reduction. A fluid collection and removal system at the bottom of the trench is an essential component of such a barrier. In hard ground it may be possible to excavate an open trench and then dump in gravel. In soft ground, lateral support is needed prior to replacement of the native material by the gravel backfill. This can be done by using the slurry trench method and a biodegradable drilling mud to hold the trench open and then backfilling with gravel. A case history illustrating the effective use of a gravel-filled trench at a Superfund site is described in the next chapter. The primary performance concerns with gravel-filled trenches are long-term clogging of the collection system and mechanical breakdown of the extraction system. Clogging can occur due to accumulation of sediment or, in some cases, biological growth and/or mineral precipitation. Because gravel-filled extraction trenches are active systems, they require constant maintenance and monitoring, which increase costs but facilitate identification and remediation of clogging and mechanical breakdown.

4.6.8 Contaminant Transport Through Vertical Barriers

The possible scenarios for chemical transport through vertical cutoff walls are depicted in Figure 4.8. Pure diffusion (a) is an ideal case; diffusion with advection (case b) occurs when unmitigated buildup of contaminated water retained by the wall is allowed to occur; and diffusion against advection (case c) occurs when the contaminated water inside the containment area is drawn down (e.g., by pumping or drainage) to induce inward flow of water.

Recently, the effect of variability in aquifer hydraulic conductivity on chemical transport through soil-bentonite vertical barriers has been evaluated (Filz et al., 2003; Britton et al., 2005b). In the Britton et al. study, the impact of log-normal variation in hydraulic conductivity values on both steady state and transient contaminant flux through a cutoff wall with idealized initial and boundary conditions was investigated. The results showed that contaminant flux through cutoff walls increases as the variability in hydraulic conductivity increases, all other variables being constant. The effect of variability was found to be most pronounced for the case of diffusion against advection (Figure 4.8c) because the increase in inward advective flux in areas where the seepage velocity is higher than average is more than offset by the increase in outward diffusive flux in areas where the seepage velocity is lower than average.

As with GCLs, the existence of membrane behavior in soil-bentonite cutoff walls has been postulated on the basis of

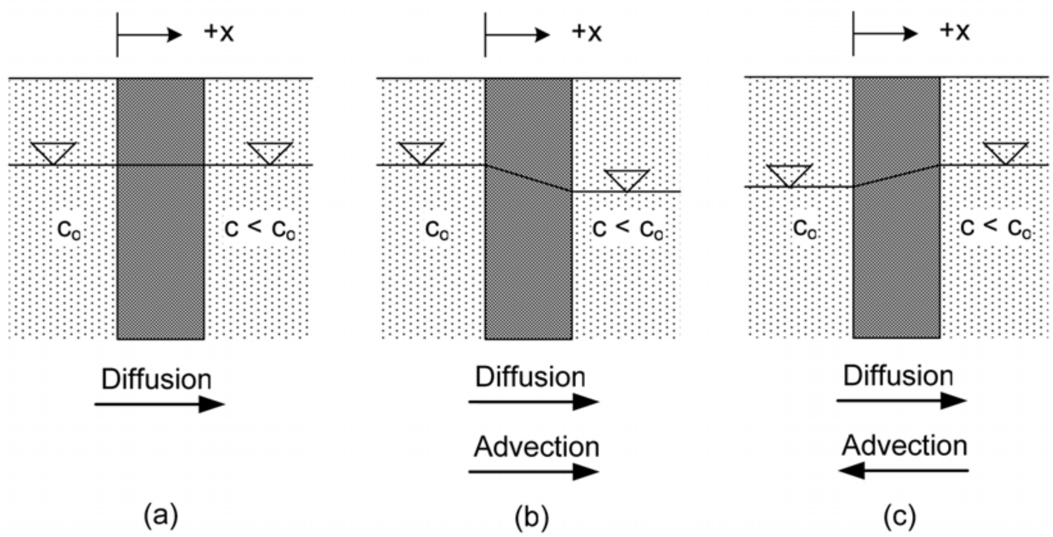


FIGURE 4.8 Chemical transport scenarios for vertical barriers: (a) pure diffusion, (b) diffusion with advection, and (c) diffusion against advection. SOURCE: Sleep et al. (2006). Copyright 2006 by Taylor and Francis Group LLC, Books. Reproduced with permission of Taylor and Francis Group LLC Books via Copyright Clearance Center.

laboratory tests performed on model soil-bentonite backfills consistent with those expected in field-constructed walls (Yeo et al., 2005). More recently, laboratory tests performed on field-recovered backfill materials have confirmed the existence of membrane behavior, but the measured membrane efficiencies were lower than those previously reported for the model soil-bentonite backfills (Henning et al., 2006). The difference was attributed primarily to the lesser amount of fine-grained soils in the field-constructed backfills.

Advection and diffusion are the primary contaminant transport mechanisms through concrete barriers, and sorption also plays an important role. Advective and diffusive transport of ions (e.g., chloride, sulfate) can also have a major impact on the properties of concrete. The resistance of a concrete barrier to advective flow depends on both the intact hydraulic conductivity of the barrier and the presence of through-going cracks that can serve as conduits for the advective flow of liquids and gas. Cracks also affect the rate of diffusive transport through the barrier, even if they are not through-going, as they allow fluids containing the diffusing ions to penetrate the barrier through advective flow, shortening the diffusion pathway and increasing the rate at which the adsorptive capacity of the barrier is expended. Diffusion of ions, particularly chloride and sulfate ions, can also lead to material degradation and enhanced cracking, although at low concentrations sulfate diffusion can help seal cracks. Sorption affects both the time required for radionuclides to break through and the rate at which various degradation mechanisms proceed. These include, notably, corrosion of steel reinforcement, which is a significant contributor to concrete cracking. Hence, the coupling between advective and diffusive flow and interaction between the concrete, concrete reinforcement, and transported ions is an important consideration in concrete barrier performance.

4.6.9 Conclusions

Although few monitoring data on field-installed vertical barrier walls used for environmental protection exist, available evidence suggests that the primary short-term factor affecting vertical cutoff wall performance is poor construction. Chemical incompatibility and corrosion and/or material degradation may be important medium- and long-term concerns for almost all types of vertical barriers.

Only a few high-permeability defects can have a significant adverse impact on the performance of a barrier (Benson and Dwyer, 2006), and soil-bentonite, soil-cement, and tremie concrete walls are particularly susceptible to construction defects. Poor joint seals in sheet pile and geomembrane panel barriers and lack of continuity in grouted barriers can have a similar effect on vertical barrier effectiveness.

Despite the existence of several laboratory and field methods to measure the hydraulic conductivity of soil-bentonite and cement-bentonite walls, none are free of complicating factors that can compromise the reliability of the results (Benson and Dwyer, 2006). These factors include the extensive lengths of cutoff walls and corresponding large volumes of the subsurface contained in the walls, the potential for leakage from the underlying floor (e.g., aquitards into which the wall is keyed) or from the overlying cap, and the inability to distinguish the volume of water passing through the wall during a pump test from that due to drainage of aquifer materials. In addition, most, if not all, of the studies that have been performed to evaluate the potential for chemical incompatibility of vertical barriers were conducted over time frames that are too short to provide a realistic assessment of the long-term effects that contaminated groundwater can cause in cutoff wall materials (Benson and Dwyer, 2006). Finally, there is virtually no available field evidence or data on the performance of vertical barriers with respect to chemical transport. Because of problems such as these, determination of the effectiveness of vertical barriers in assuring adequate long-term performance of the overall containment system is difficult and requires more study.

4.7 ASPHALT CONCRETE BARRIERS

The primary performance concerns for asphalt cement concrete (ACC) barriers are cracking and degradation. Asphalt concrete barriers are susceptible to similar cracking concerns as PCC walls, although their greater ductility may reduce the cracking potential caused by ground deformations. On the other hand, asphalt is more susceptible to chemical attack by solvent-bearing wastes.

Asphalt cement concrete can have a relatively low intact hydraulic conductivity. By limiting the air voids ratio and increasing the asphalt cement or binder content, the intact hydraulic conductivity of ACC can be as low as 1×10^{-8} cm/s (Asphalt Institute, 1976). ACC can be used for hydraulic containment structures, such as dams (Creegan and Monismith, 1996). A major limitation of ACC as a barrier material, however, is its susceptibility to chemical attack from solvents (Bowders et al., 2002). Like PCC, the bulk hydraulic conductivity of ACC is also increased by cracking. ACC is often said to be able to last thousands of years based on the recovery of asphalt artifacts up to 5,000 years old (Kays, 1977; Freeman et al., 1994). However, physical longevity does not guarantee that the material can maintain a sufficiently low hydraulic conductivity.

ACC mixes are designed with a variety of mix gradations and air void structures. The air void structure (porosity) is typically about 4 percent (conventional dense graded mix) but can be as high as 20 percent (open graded mix). However, low-permeability mixes generally strive for an air void structure of less than 4 percent. Although the hydraulic conductivity of asphalt mixes is measured easily in the laboratory, field measurement of permeability is problematic. ACC barrier quality assurance and quality control therefore rely on indirect assessment based on measurement of factors that control the in-place hydraulic conductivity of the ACC: the asphalt cement content and the in-place unit weight (Bowders at al., 2003). The quality assurance and quality control of asphalt barriers rely on measuring these properties in much the same way that water content and unit weight are monitored during placement of earthen barriers. However, large-scale field permeability tests like the sealed-ring infiltrometer, which are used to detect the effects of cracking and nonhomogeneities on the hydraulic conductivity of earthen barriers (see Box 4.1), are not conducted for asphalt barriers.

ACC is subject to similar cracking mechanisms as PCC, although the greater ductility of ACC may mitigate their severity. Thermal, flexural, and chemical shrinkage cracks can cause significant problems with the hydraulic integrity of ACC barriers. The use of resin-impregnated nonwoven geotextile reinforcing in the asphalt barrier layer has been suggested as a measure to mitigate cracking and to provide secondary advective flow resistance (through the low-permeability nature of the geotextile) should cracking develop (Marienfeld and Baker, 1998; Kavazanjian and Dobrowolski, 2003). Fiber reinforcement has also been used to mitigate cracking, but little is known about the long-term performance of this material.

The use of asphalt rubber (or rubberized asphalt) has proven successful in minimizing or eliminating cracking in ACC pavements. The asphalt cement used in this process is a field-blended asphalt rubber material, which is composed of roughly 80 percent performance-graded asphalt cement and 20 percent ground tire rubber. The asphalt cement content in these mixes is typically higher (7.5 to 9.5 percent) than that in conventional asphalt mixtures (~ 5 percent) (Kaloush et al., 2004; Zborowski and Kaloush, 2006). However, the long-term durability of rubberized asphalt is largely unknown, and the application of this technology to the construction of engineered barrier layers has not been investigated.

Low-permeability asphalt layers are generally more ductile but weaker than conventional asphalt pavement mixes. Asphalt concrete barriers are often used in final covers when a firm working surface is required for postclosure use, such as for parking vehicles or storing materials. The higher ductility of low-permeability asphalt mixes can be detrimental for this postclosure use. However, there is no evidence that asphalt remains ductile for thousands of years. In fact, the resins and oils in the asphalt binder are known to be susceptible to reduction/transformation from oxidation and weathering. The potential for volatilization of hydrocarbons in the asphalt binder is often cited as a reason for not considering ACC liners for municipal solid waste and hazardous waste containment facilities, as it may be difficult to determine if hydrocarbons detected in landfill monitoring systems are from the binder or from other sources. Research on aging of asphalt binders and mixtures for pavements has been addressed through test protocols developed under the American Association of State Highway and Transportation Organizations Strategic Highway Research Program (Bell and Sonsnovske, 1994; Bell et al., 1994).

4.8 SUMMARY AND CONCLUSIONS

Most containment barrier performance evaluation studies to date have focused on the components used in cover and liner systems. Available quantitative information documenting the field performance of earthen barriers, particularly compacted clay liners, is based largely on the results of in situ testing of prototype barriers and test pads. Compaction control is the most important consideration with respect to the ability of the barrier to achieve a suitably low hydraulic conductivity (e.g., 1×10^{-9} m/s). Few medium- or long-term data on the performance of earthen barriers exist primarily because (1) the time frames associated with the construction of modern waste containment systems are still relatively short (<30 years), and (2) no formal program was established for assessment of barrier performance at the time of construction. Available medium- and long-term data indicate that compacted clay layers generally perform effectively as components in barrier systems unless poor construction and/or operational practices diminish layer integrity. Over longer terms, unprotected clay layers in covers may develop secondary permeability that can lead to decreased effectiveness. Desiccation is especially a concern for compacted clay liners used in cover systems because of the close proximity of the liner to the atmosphere. Available evidence suggests that compacted clay liners should be used in cover systems only as the underlying component of a composite liner with an overlying geomembrane and sufficient overburden to maintain intimate contact between the geomembrane and the compacted clay liner.

Diffusion can be a significant contributor to the migration of chemical contaminants through well-constructed earthen barriers. Consolidation of the barrier due to the weight of overlying waste can decrease its hydraulic conductivity with time, thereby enhancing the overall performance of the barrier. However, high temperatures near the barrier (e.g., due to biodegradation of solid waste) as well as chemical incompatibility (i.e., adverse interactions between migrating chemicals and the earthen materials used for the barrier) can potentially increase the hydraulic conductivity in both the medium and long terms. The effects of chemical incompatibility may not be apparent for decades, although such effects likely will be significant only for barriers comprised of highly active soils, such as bentonite. Additional long-term monitoring will be required before any definitive conclusions can be drawn about the long-term effectiveness of compacted clay and composite barriers to halt volatile organic compound migration.

Short-term performance concerns with geomembrane barriers include defective material, physical damage caused by construction activities or defective seams, and the potential for leakage through such defects. The number and sizes of leaks vary, depending on such factors as the size and type of waste containment system (e.g., MSW vs. hazardous waste), and therefore so does the performance of the geomembrane barrier. Field data indicate that membranes installed with strict CQA exhibit significantly fewer leaks and perform better than geomembranes installed in the absence of CQA.

Medium- and long-term performance concerns for geomembranes include puncture due to increased overburden pressure, geomembrane degradation, and limited resistance to diffusion. Once more than 1 or 2 m of material has been placed on top of a geomembrane, there is no practical way to determine if the geomembrane has been punctured. The estimated service lives of geomembranes vary widely and are strongly dependent on the temperatures to which the geomembrane is exposed, ranging from about 1,000 years at 10°C to as little as 15 years at 60°C. To refine predictions of geomembrane performance in the medium and long term, continuous monitoring is required over these same time frames. Although intact geomembranes provide adequate resistance to advective liquid transport, available evidence indicates that they offer little, if any, resistance to diffusive transport of several volatile organic compounds. This can be a short-term problem if a geomembrane is used as the sole barrier, or a medium-term or long-term problem if the geomembrane is used as a component of a barrier system comprised of more than one barrier material or type. However, this problem can be mitigated by having a suitable diffusion attenuation layer (e.g., a relatively thick layer of compacted soil) incorporated into the barrier system.

Short-term concerns with geosynthetic clay liners include the possibility of defective materials and separation of overlapped GCL panels. Chemical incompatibility (increases in hydraulic conductivity due to interactions with contaminant liquids) may develop in the short term if there is exposure to relatively strong liquids (e.g., high ionic strength chemicals). Medium- and long-term concerns for GCLs include effects of desiccation and chemical incompatibility, as well as overall local and global slope stability concerns. Although chemical incompatibility could even become an issue in the presence of relatively weak strength liquids due to the low flow rates associated with chemical transport through GCLs, adverse consequences may be mitigated by prehydration or applied overburden stresses. Chemical transport through individual GCLs also is a concern in cases where defects (holes) are sufficiently large that self-healing (e.g., swelling of bentonite) cannot fill them and because of the short distances of transport required for chemical transport through the GCL. Although field observations indicate that GCLs alone may not be an effective barrier to diffusive transport, they may be effective replacements for low-permeability soil layers as barriers to advective transport, and when combined with a suitable diffusion attenuation layer they can be effective as part of a diffusion barrier.

Short-term performance concerns for granular drainage layers include inadequate discharge capacity and clogging. Over the medium and long terms, granular drainage layers can be susceptible to clogging due to soil infiltration, biological growth, and chemical precipitation. Recent studies have led to a better understanding of clogging mechanisms and the development of models for predicting long-term performance of granular drainage layers.

Short-term performance concerns for geosynthetic drainage layers include installation damage and inadequate capacity. Medium- and long-term performance concerns include clogging due to soil infiltration, soil and geosynthetics penetration, creep of the geonet core, biological activity, and mineral precipitation. While unpublished laboratory studies show that geonet drainage layers are quite susceptible to clogging in a manner similar to that of granular drainage layers, little research has been done on these systems and few field data exist on their performance.

Evapotranspirative barriers are used in capacitive cover systems (monolithic covers and capillary break covers). This type of cover has only recently been considered as an alternative for more traditional types of covers, such as those required under RCRA Subtitles C and D for hazardous and MSW disposal facilities. Thus, most available data cover only a few years. The results of field-scale evaluations suggest that evapotranspirative covers can be an effective alternative to compacted clay or composite covers for infiltration control in arid and semiarid climates where evapotranspiration is an important component of the water balance. Evapotranspirative covers may also be effective alternatives in wetter climates where infiltration control is not a primary concern. However, significantly more data over much longer time frames are required to make a reliable prediction of the long-term performance of evapotranspirative barriers.

Short-term performance concerns for vertical barriers include gaps in the wall as a result of poor mixing, defective material, and high-permeability zones caused by caving or trapped low-quality material and leakage at joints between panels. Medium- and long-term performance concerns include chemical incompatibility, desiccation above the water table, cracking caused by ground deformation, and deterioration of the barrier material. The addition of supplemental cementitious materials (e.g., fly ash, blast furnace slag) can significantly reduce the saturated hydraulic conductivity of cement-bentonite barriers, at least over the short term. Although few monitoring data on field-installed cutoff walls exist, available evidence suggests that the primary short-term issue affecting vertical barrier wall performance is poor construction. An important medium- and long-term issue for soil-bentonite barriers includes chemical incompatibility. Overall, it is not yet possible to determine whether vertical barrier walls are effective containment barriers for the long term.

Short- and medium-term performance of cementitious barriers appears, in general, to be good. Long-term performance concerns for cementitious barriers are associated primarily with degradation and cracking of the cementitious barrier material. Cracking caused by restrained shrinkage during curing, settlement during placement, flexure, thermal changes, drying shrinkage, and chemical shrinkage can all create microstructural features that increase the bulk permeability of a PCC barrier. Sulfate-induced degradation of concrete and expansion due to chloride-induced corrosion of reinforcement within the concrete are also important degradation mechanisms for PCC barriers. A variety of mitigation techniques have been developed to address these problems, including the use of ASTM Type V cement; the addition of supplemental cementitious materials, superabsorbent polymers, and saturated lightweight aggregate to the concrete mix; and the use of nonmetallic reinforcement. However, little is known about the long-term behavior of concrete barriers that incorporate these mitigation measures. Although 5,000-year-old asphalt artifacts have been recovered, this does not demonstrate the ability of asphalt concrete to serve as effective hydraulic intrusion barriers for thousands of years. ACC is subject to many of the same cracking mechanisms as PCC. Nonmetallic reinforcement (e.g., synthetic fibers, geogrids, geotextiles) has been used to mitigate cracking in ACC, but little is known about its long-term performance. Asphalt rubber mixtures have been shown to have reduced cracking potential compared to conventional ACC for pavements, but no work has been done on the use of rubberized asphalt in engineered barriers.

The performance of many of these systems could be improved by the use of better component materials. Areas that are fruitful for future research on the properties and behavior of different materials include

- HDPE and other geomembrane formulations that have greater resistance to degradation by thermal oxidation.
- Drainage materials that are more resistant to clogging.
- GCLs that are more effective acting as semipermeable membranes.
- Geomembranes that have lower coefficients of thermal expansion and contraction.
- Admixtures that reduce shrinkage and cracking of cement-bentonite, soil-cement-bentonite, and concrete barriers.

5

Containment System Performance

Observational data and predictive models for the performance of engineered barrier systems—including liner systems, cover systems, leachate collection systems, and vertical barriers—and the overall performance of containment systems are evaluated in this chapter. Unfortunately, few direct observational data on performance are available for most of these systems and none of the data extend beyond three decades. Consequently, predictions of long-term performance generally rely on extrapolations from relatively short-term data (see Box 1.2 for definitions of performance periods) and on assumptions based on the long-term performance of barrier system elements (described in the previous chapter).

An example of the types of performance information that are available on waste landfills and impoundments is given in Box 5.1. The information focuses on liners and covers; comparable information on vertical barriers for waste containment has not been summarized.

5.1 OBSERVATIONS OF PERFORMANCE

5.1.1 Liner Systems

Overall Liner System Performance

The best-available information on the overall performance of liner systems comes from monitoring data for the environment surrounding the liner system. The New York Department of Environmental Conservation (NYDEC) reviewed groundwater monitoring data at all modern municipal solid waste (MSW) and hazardous waste landfills in the state (letter to the committee from Stephen Hammond, Director, NYDEC, August 30, 2006). For New York, "modern" means since 1988, when the state issued new regulations for MSW landfills that require double-liner systems. The number of facilities reviewed includes 27 MSW landfills and 4 hazardous waste landfills. Of these 31 landfills, 28 have double-composite liners, while 3 have double liners with a single geomembrane in the primary liner. In total these landfills comprise 1,100 acres of barrier systems and 450 years of operation. Considering that landfill cells are developed gradually over a period of years, the landfills assessed correspond to 7,000 to 10,000 acre-years of operation.

Based on groundwater monitoring data from onsite monitoring wells, NYDEC did not find a single instance of an adverse impact to groundwater that could be attributed to leakage through a containment system at any one of these facilities. NYDEC did find several instances where groundwater was impacted by older unlined portions that were also present at some of the landfill sites and by onsite activities not related to the barrier system, such as a leaking seal in a leachate conveyance line outside the landfill cell.

In addition, NYDEC reviewed water quality monitoring data from pressure relief systems, which existed at 20 of the 27 MSW landfills included in this study. These systems directly underlie the base liner, so they potentially provide direct information on leakage through the containment system. At all but 4 of the 20 landfills with pressure relief systems, the pressure relief systems covered the entire footprint of the barrier system. NYDEC also did not find a single instance where these data indicated the presence of contaminants that had been released from the overlying barrier system into the pressure relief system.

Fluid Leakage Through Liner Systems

Leakage of fluid through the primary liner system in a double-liner system can be estimated using measured pumping or flow rates from the secondary leachate collection (leak detection) system. The flow rate reflects how well both the leachate collection system and the primary barrier layers are working. The measurement is actually the pumping rate required to maintain a constant head in the sump of the secondary collection system; therefore, clogging in the secondary collection system could also affect the measured flow rate.

BOX 5.1 Types of Data Available on Waste Containment Landfills and Impoundment System Performance

Few data exist on containment system performance. One of the most comprehensive studies on short- and medium-term performance of containment systems was conducted by Bonaparte et al. (2002), based on information collected from publications and from discussions with facility owners, operators, designers, and regulators. The study reported performance problems from multiple sources, including operators, at approximately 2,000 U.S. landfills and impoundments designed and constructed under Resource Conservation and Recovery Act (RCRA) Subtitles C and D. A total of 85 "problems" were identified at 79 landfill and 5 surface impoundment facilities (listed in Table 5.1 and shown graphically in Figure 5.1). Care must be taken in drawing conclusions from these data, since the search for problem facilities was not exhaustive.

TABLE 5.1 Types of Problems Encountered at Waste Containment Facilities

		Principal Human Factor Contributing to the Problem		
Waste Containment System Component or Attribute	Prevalence of Problem (%)	Design	Construction	Operation
Landfill liner construction	17	1	11	2
Landfill liner degradation	8	3	3	1
Landfill LCRS or LDS construction	7	0	6	0
Landfill LCRS or LDS degradation	6	2	3	0
Landfill LCRS or LDS malfunction	5	2	0	2
Landfill LCRS or LDS operation	5	1	0	3
Landfill liner system stability	14	9	0	3
Landfill liner system displacement	5	4	0	0
Cover system construction	2	0	2	0
Cover system degradation	2	2	0	0
Cover system stability	21	16	2	0
Cover system displacement	2	1	1	0
Impoundment liner construction	4	0	3	0
Impoundment liner degradation	1	1	0	0
Impoundment LDS	0	0	0	0
Impoundment liner system stability	1	0	1	0
Impoundment liner system displacement	0	0	0	0

NOTES: LCRS = leachate collection and removal system; LDS = leak detection system.
SOURCE: Bonaparte et al. (2002).

FIGURE 5.1 (*Left*) General distribution of problems by waste containment system component or attribute. (*Right*) Distribution of problems by principal human factor contributing to the problem. SOURCE: Bonaparte et al. (2002).

In the landfill performance assessment discussed in Box 5.1, flow rates through primary liners into the leak detection system were reported for active cells with double liner systems with top geomembrane liners (single or composite; Bonaparte et al., 2002). Facilities that used conventional construction quality assurance (CQA) programs had substantially lower leakage rates through geomembrane liners than facilities that did not.

Table 5.2 summarizes the mean, maximum, and minimum average monthly flows for single geomembrane, geomembrane-compacted clay liner (GM-CCL), and geomembrane-geosynthetic clay liner (GM-GCL) composite systems overlain by sand liquid collection layers for the initial, active, and postclosure periods. Of the three systems, leakage was by far the greatest through the single geomembrane. Leakage through the GM-GCL systems was generally equal to or less than that through the GM-CCL systems, and the rates were dramatically lower (typically by an order of magnitude) during the active life and postclosure period for the GM-GCL systems compared to the GM-CCL systems. Similar data were reported for GM-GCL systems compared to GM-CCL systems when a geonet liquid collection layer was used.

The findings discussed above and the relatively low leakage rates (even with a single geomembrane, provided a good CQA program is in place) indicate that modern geomembrane and composite liners are working reasonably well. A composite liner has a significantly lower leakage rate than a single geomembrane. The data suggest that a composite liner with a GCL leaks less than one with compacted clay, but more care must be taken during construction of the liner system and placement of the waste when a GCL is used.

Figure 5.2 shows the flow rates over time of the primary leachate collection and removal system and the secondary leak detection system in a Pennsylvania MSW landfill. Leachate generation, as represented by leachate collection and removal system flow rate measurements, is highest during the initial period of landfilling. Flow rates decrease as the waste thickness increases and daily and intermediate covers are applied (i.e., active period of operation) and become almost negligible once the final cover is placed and the landfill enters the postclosure period. The monitoring data also show that over time the flows in the leak detection system are small to negligible, confirming that the composite liner system performed efficiently over the 7 years monitored.

Diffusion Through Bottom Liner Systems

Diffusive flux may constitute a significant portion of total flux through a liner system, particularly for well-constructed composite liners and low hydraulic conductivity (less than about 1×10^{-10} m/s) compacted clay liners. Diffusion is driven by chemical gradients, that is, by the difference in the concentrations of a chemical or compound above and below the barrier layer. Geomembranes are an excellent barrier to diffusion of ionic contaminants (e.g., salts, metals, volatile fatty acids), but they will readily allow diffusion of volatile organic compounds (Rowe, 2005). Because the concentration of volatile organic compounds in the leachate in modern MSW landfills is typically low (generally less than 1,000 parts per million and often less than tens of parts per million), diffusive flux through a composite liner and underlying attenuation layer will not generally pose a significant environmental hazard while the geomembrane remains intact. However, in hazardous waste facilities and at MSW landfills where a sensitive receptor is in close proximity to the liner, diffusive flux can be a concern.

TABLE 5.2 Liquid Collection Rates for Double-Liner Leak Detection Systems

Stage	Initial Rate (l/ha/day)	Active Rate (l/ha/day)	Postclosure Rate (l/ha/day)
Geomembrane			
Mean average monthly flow	307	187	127
Minimum average monthly flow	4	0	1
Maximum average monthly flow	2,144	1,603	328
Sand/Geomembrane/Compacted Clay			
Mean average monthly flow	114	142	64.4
Minimum average monthly flow	1.2	22.7	0
Maximum average monthly flow	1,192	672	274
Sand/Geomembrane/GCL			
Mean average monthly flow	133	22.5	0.3
Minimum average monthly flow	0	0	0
Maximum average monthly flow	984	284	0.9

NOTES: The initial period of operation corresponds to the first few months after the start of waste disposal in a cell. Until that time there is insufficient waste to significantly impede the flow of rainfall into the leachate collection system. The active period is when the cell is being filled with waste and daily and intermediate layers of cover soil are being applied. The postclosure period is after the final cover system has been placed.
SOURCE: Averages calculated and Max/Min directly from Bonaparte et al. (2002).

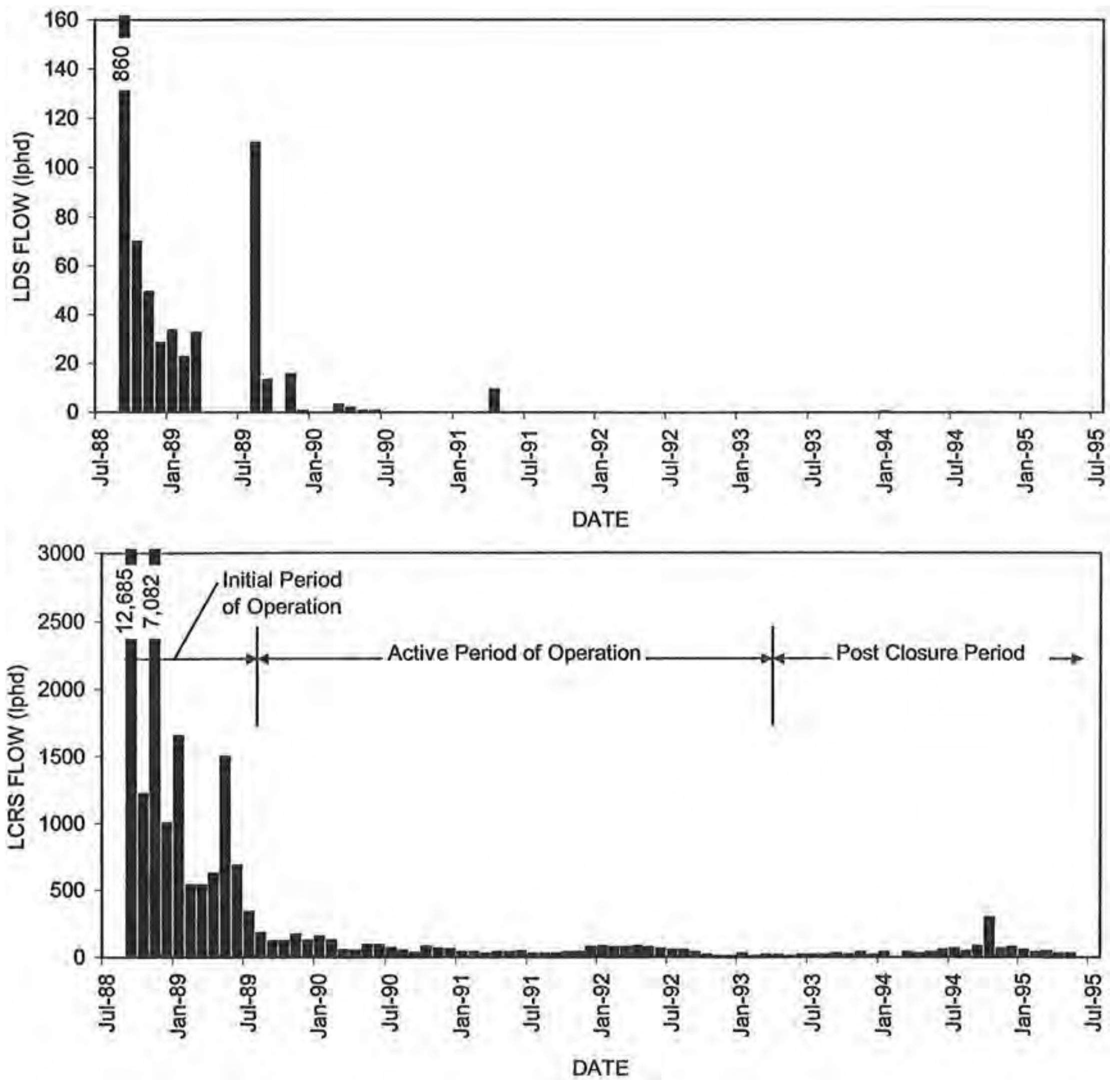


FIGURE 5.2 Leachate collection and removal system (*top*) and leak detection system (*bottom*) flow rates over time at an MSW landfill in Pennsylvania. SOURCE: Bonaparte et al. (2002).

Medium-term data on the chemical composition of leachate have been reported from lysimeters installed beneath single- and composite liner systems (Bonaparte et al., 2002; King et al., 1993). Exhumation and undisturbed sampling have also been used to obtain chemical concentrations in compacted clay liners (Reades et al., 1989; King et al., 1993; Rowe, 2005). These studies showed that chloride diffused over a distance of approximately 0.75 m over 4.3 years through a 0.3-m-thick clogged sand layer underlain by a 1.2-m-thick compacted clay liner. A diffusion coefficient of 6×10^{-10} m^2/s provided a good fit for the field concentration profile of chloride and is consistent with the diffusion coefficient measured in the laboratory. Sodium was also retarded at a low level (Na^+ migrated about 35 cm into the compacted clay liner), and potassium was retarded at a high level (K^+ migrated about 5 cm into the liner) through the same liner system. Desorption of calcium was observed in the concentration profile.

The results of an ongoing study on leachate chemistry for single- and composite liner systems in Wisconsin for periods exceeding 20 years indicate that a wide variety of volatile organic compounds in various concentrations have appeared at different frequencies in the liquid effluent collected from lysimeters beneath the liners (see Section 4.1.4 and Klett et al., 2006). The exact mechanism for the transport of these volatile organic compounds is not yet known with any certainty and could be leakage or diffusion through the liner, gas migration around the liner, or both. In general, the arrival of the volatile organic compounds occurred approximately 10 years after waste placement. This timing is generally consistent with the observation that volatile organic compounds (especially toluene) diffused to a depth of about 0.6 m in

4.3 years at the Keele Valley landfill (Barone et al., 1993; Rowe, 2005) and the rate of diffusion of ions to depths of over 1 m in low-permeability clay (Quigley and Rowe, 1986; Lake and Rowe, 2005b). Overall, these studies indicate that the potential for diffusive flux, including the long-term flux of volatile organic compounds, should be considered when designing a facility.

Gas Migration Control for Bottom and Side Slope Liner Systems

Modern MSW landfill liner systems are intended to control both gas and liquid (leachate) migration. However, gas migration pathways around even a gas-tight liner system can adversely impact the gas containment effectiveness of the liner system. The California Integrated Waste Management Board's (CIWMB's) 2003 landfill facility compliance study examined 16 landfills with base liners employing geomembrane barriers over the entire waste footprint (Geosyntec Consultants, 2003). Six of the landfills were reported to be in "corrective action," indicating that monitoring systems detected a release of contaminants from the waste mass. In five of these six cases, analysis of the groundwater chemistry indicated that the release was related to landfill gas, while in the sixth case the cause of the release was undetermined but was suspected to be related to construction.

The results of the CIWMB study suggest that there may be a systematic flaw with respect to gas containment in modern geosynthetic liner systems. Kavazanjian and Corcoran (2002) discussed the occurrence of landfill gas impacts on groundwater at MSW landfills with geomembrane liner systems. They attribute these gas impacts to geomembrane liner termination details that allow landfill gas that has accumulated in the leachate collection and removal system to travel around the buried edge of the liner termination and penetrate into the ground. This pathway is illustrated in Figure 5.3. This problem can be mitigated in a number of ways, including placing the leachate collection system under vacuum, placing gas collection trenches in the waste adjacent to the side slope (taking care not to draw too much oxygen into the system), and/or modifying the termination detail for the side slope leachate collection layer to include a geomembrane flap or other type of cap around the end of the leachate collection and removal system.

Mechanical Damage to or Deterioration of Bottom Liner Systems

The performance of bottom liner systems can be affected significantly by damage or deterioration during operation or after closure. Box 5.2 illustrates how damage to a composite liner during filling increased the leakage rate dramatically. This example also demonstrates the value of redundancy in a liner system; while the primary liner was compromised significantly, the secondary liner remained intact and prevented an adverse impact to the environment.

The 2002 Environmental Protection Agency (EPA) database of observed problems in liner systems at modern U.S. municipal solid waste and hazardous waste landfills lists eight instances of liner system leakage caused by damage. In five cases the liner was a single geomembrane that was damaged during construction or operation, and in three cases it was a geomembrane or composite liner that leaked at a leachate collection pipe penetration. In all but one case the leakage was detected using measured flow rates from the leak detection system. An important point is that only a small

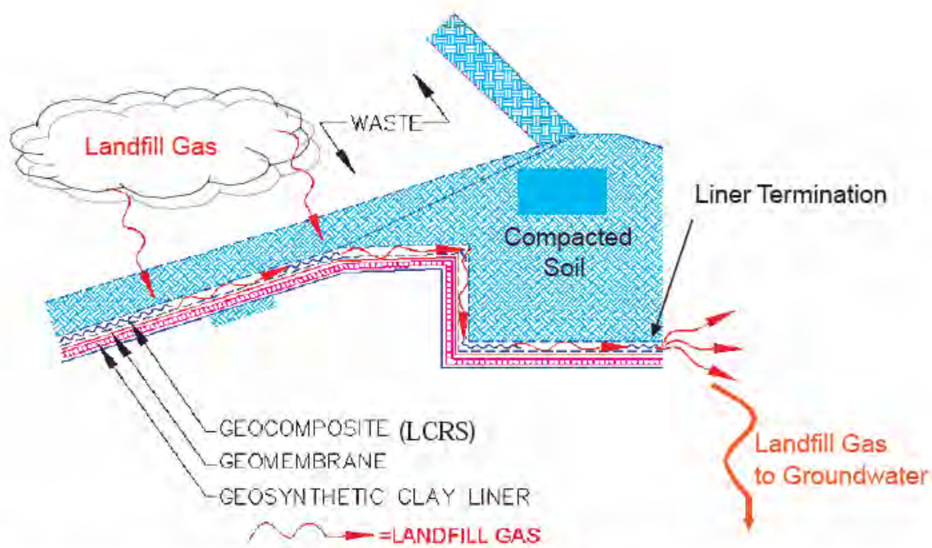


FIGURE 5.3 Landfill gas migration pathway around geosynthetic liner system. SOURCE: Kavazanjian and Corcoran (2002).

BOX 5.2 Case History on Mechanical Damage to a Geosynthetic Composite Liner

This case history illustrates how the leakage rate through a composite geomembrane-geosynthetic clay liner can increase significantly due to relatively large through-going holes. The barrier was a double-liner system for a hazardous waste landfill located in the Midwest (Daniel and Gilbert, 1996). The primary liner was a composite liner with a geomembrane overlying a geosynthetic clay liner. The rate of pumping from the leak detection system averaged about 10 lphd for the first 3 months of operation, which is typical for a composite liner. However, the pumping rate increased to more than 30,000 lphd after the third month of operation. The suspected cause of the increased leakage rate was damage to the primary liner during waste placement. An extensive field investigation, which ultimately involved removing the waste and uncovering the liner system, identified 28 holes that were apparently caused by a backhoe moving waste in the landfill. The holes extended through the geosynthetic clay liner (Figure 5.4), meaning that the composite system performed similarly to a single geomembrane liner with holes. The holes penetrated through the secondary geomembrane and a small distance (less than 3 cm) into the secondary compacted clay liner. However, the secondary clay liner was still intact and able to contain the waste at the location of the holes. The holes were repaired and pumping rates into the leak detection system dropped back to about 10 lphd. This case history demonstrates the value of redundancy in a barrier system; the damage to the primary liner was immediately detected and leakage through it was contained by the secondary detection system and barrier.

FIGURE 5.4 Damage to the primary liner. The hole extends through all of the geosynthetic layers into the top of the clay liner. SOURCE: Golder Associates, Inc.

fraction (less than 10 percent) of the approximately 2,000 landfills in the database contained double-liner systems with data from a leak detection system. In one case, damage to a geomembrane was detected during construction using an electrical leak location survey. Therefore, mechanical damage to the liner system occurred in approximately 5 to 10 percent of the facilities where it could be detected.

In the Bonaparte et al. (2002) study, other types of liner damage were observed that did not necessarily result in leakage because they were detected immediately. The vast majority of these problems involved slope failures; in fact, nearly one-half of the problems identified in the facilities surveyed were the result of slope failures or excessive displacement in the liner system (Figure 5.1). In addition, two instances of liner damage were caused by landfill fires, two were caused by installing gas wells, and one was caused by desiccation cracking when the compacted clay liner was left exposed for 3 years prior to waste placement.

Degradation of leachate collection and leak detection systems was also observed in the Bonaparte et al. survey. The observed problems included clogging of geotextile filters (two cases) by fines, clogging of the sand drainage layer and pipes (one case), uncontrolled leachate seeps due to perched leachate in waste (one case), and clogging or problems with flow rate measuring systems such as the one described in Box 4.4 (four cases).

Temperature variations above and within a liner system can be substantial, as illustrated by the case history in Box 5.3. Elevated temperatures can accelerate degradation of geosynthetic liner materials and increase the hydraulic conductivity of clay liners (Rowe, 2005). In addition, thermal gradients can affect the transport of liquids and gases through liner systems.

Temperatures of 30 to 40°C have been observed on base liners after 5 to 10 years, and higher temperatures can develop even earlier when there is moisture augmentation (Koerner and Koerner, 2005). Thus, while primary composite liners have performed well to date, the long-term performance is in some doubt. This, as well as the problems noted above regarding damage to primary liners, suggests the desirability

BOX 5.3 Case History on the Temperature Environment for a Geosynthetic Composite Liner

This case history illustrates how temperatures within a liner system vary with time and are influenced by waste placement conditions. Data were collected from a Michigan landfill with a bottom liner that included (from top to bottom) a 45-cm-thick protective sand layer, a geotextile-geonet composite leachate collection layer, a geomembrane, and a geosynthetic clay liner (GCL). Temperatures were measured in three cells using thermocouple sensor arrays placed above the leachate collection system within the protective sand layer and immediately below the GCL on the subgrade for variable periods from approximately 1 year prior to waste placement and up to 5 years thereafter (Yesiller and Hanson, 2003; Hanson et al., 2005a). Under exposed conditions, temperatures in the liner system followed seasonal temperature variations. The thickness of the protective sand layer was insufficient to prevent excessive temperature variations and freezing of the liner system. Seasonal temperature fluctuations were dampened after the first lift of waste was placed over the liner systems, then temperatures increased with time. Temperatures in the liner system in the middle of the landfill had reached 20°C to over 30°C within 2 to 4 years after waste placement. The effects of seasonal variations were still observed at locations within approximately 20 m of cell perimeters (Figure 5.5). The average absolute thermal gradient was 26°C/m for the exposed liner and 16°C/m for the waste-covered liner.

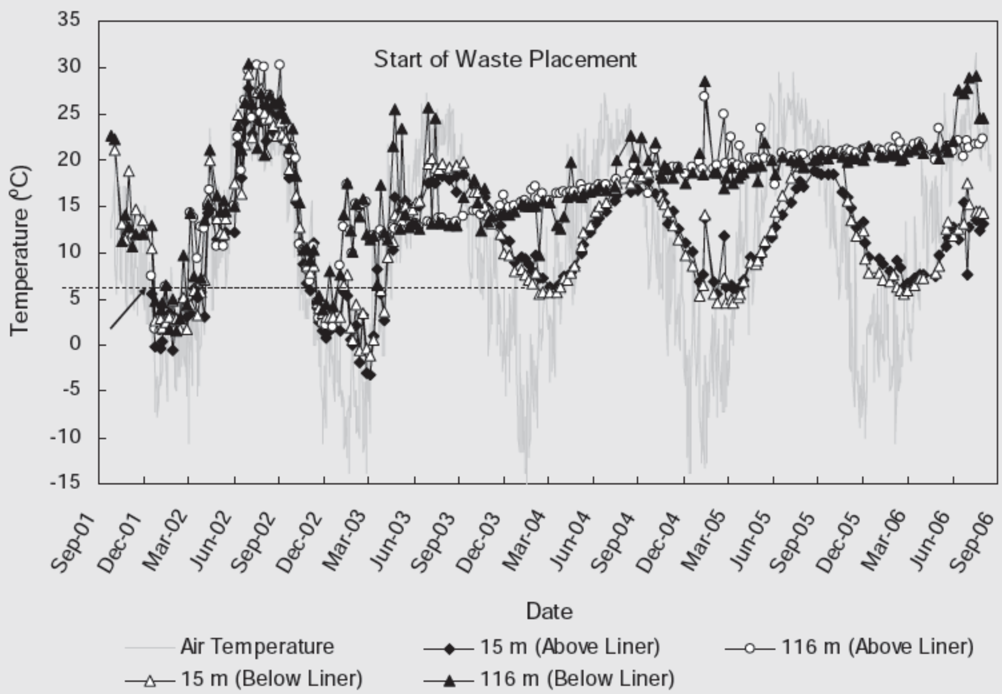


FIGURE 5.5 Example of liner temperatures in a cell before and after waste placement for locations near the edge of the cell (15 m) and in the center of the cell (116 m). SOURCE: Adapted from Hanson et al. (2005a). Reprinted with permission of the American Society of Civil Engineers.

of double-lined systems for ensuring satisfactory long-term performance of the overall barrier system. However, even with double liners high temperatures on the secondary liner are possible, and consideration should be given to designing the secondary system to accommodate high temperatures (Rowe and Hoor, 2007).

5.1.2 Cover Systems

Overall Cover System Performance

The overall performance of cover systems may be assessed in terms of their effectiveness in providing the basic environmental protective functions of a cover. These include infiltration control, isolation of waste from the environment, protection against inadvertent intrusion, protection against wind and water erosion, and control of gases. The use of cover systems, or capping, as the selected alternative ("final remedy") for remediation of RCRA and Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) sites has been increasing, and the committee did not find any reports of capping remedies that have to be reconsidered because of cover system failure. Although these observations suggest that the great majority of cover systems have performed well in at least the short and medium terms, it is possible that the absence of reported cover system failures may simply reflect the lack of systematic studies. A case history illustrating how a cover system can be used effectively to remediate a pre-RCRA hazardous waste landfill is presented in Box 5.4.

The evidence suggests that properly designed and constructed cover systems at MSW and hazardous waste landfills and RCRA and CERCLA sites can effectively isolate waste and contaminated soil and groundwater from the environment. In most cases, failure of a cover system to perform this function would be readily obvious to even casual observers. Thousands of cover systems have been constructed around the world in the past 30 years, and many of these systems are inspected regularly. Although cover systems are occasionally eroded by water or wind action, they generally appear to be performing satisfactorily.

The CIWMB study (Geosyntec Consultants, 2003) included a limited evaluation of overall cover system effectiveness in landfills. The environmental compliance records of 224 landfills were surveyed for the time period from January 1998 through December 2001, including 211 landfills that had received waste since October 9, 1993, when the EPA Subtitle D landfill regulations took effect, 10 landfills that had closed, and 3 landfills that had ceased to accept waste prior to October 9, 1993. In Phase I of the study, 48 of the landfills were described as fully covered, including the 10 that had closed prior to October 9, 1993. With the exception of landfill gas migration problems, the environmental performance of fully covered sites was superior to that of sites that were not fully covered. This conclusion is consistent with numerous observations that construction of an effective gas-tight cover without adequate enhancement of the gas collection system exacerbates lateral gas migration by forcing gas that had been diffusing vertically through the top of the landfill toward the sides of the landfill and downward to groundwater.

In Phase II of the CIWMB study, 53 landfills, including the 13 that had closed prior to 1993, were examined in more detail (Geosyntec Consultants, 2004). The study noted that closed landfills have a significantly lower occurrence of surface water actions and concluded that "construction of an approved final cover system can reduce the potential for surface water impacts" (p. 18). The degree of effectiveness of cover systems could not be judged from the report and the committee was unaware of corroborating studies regard-

BOX 5.4 Case History on Final Cover Performance for Site Remediation

This case history illustrates how a final cover system that includes geosynthetic components, when coupled with appropriate leachate management measures, can be used to effectively manage postclosure maintenance costs and to mitigate impacts to shallow groundwater. The approximately 51-ha North Parcel of the Acme Landfill operated from the 1950s through the late 1980s and accepted municipal solid waste, designated waste, and limited amounts of RCRA hazardous waste (RMC Geoscience, 1998). The unlined landfill is located near the margin of San Francisco Bay on relatively soft and compressible bay mud deposits, and saline groundwater occurs at or near the ground surface.

The approximate quantity of leachate generated in the landfill was evaluated using a water balance of the general form $\Delta S = \Delta V_{L(in)} - \Delta V_{L(out)}$, where ΔS is the change in leachate volume (or storage) within the landfill for a given period of time, $\Delta V_{L(in)}$ is the sum of water inflows to the refuse (volume of leachate generation) for a given period of time, and $\Delta V_{L(out)}$ is the sum of water outflows from the refuse (volume of leachate removed) for a given period of time.

Leachate generation sources were assumed to include subsurface flow into the refuse fill from consolidation of the compressible bay mud deposits and infiltration of precipitation through the intermediate cover of the landfill. Outflow from the landfill was assumed to include active leachate extraction, leachate seepage or evapotranspiration at the perimeter of the landfill, and leachate outflow in the subsurface. Subsurface inflow was evaluated using both a numerical model and consolidation calculations. Infiltration of precipitation was evaluated using a modified version of the Hydrologic Evaluation of Landfill Performance model. Surface water inflow to the parcel was assumed to be negligible. Evapotranspiration and subsurface outflow were evaluated using a numerical model. The change in storage over time was calculated using data from a number of leachate piezometers located throughout the fill.

Closure design evaluations and monitoring indicated that leachate within the parcel contributed to shallow groundwater contamination. The data and water balance evaluation further showed that leachate within the parcel was generated by an equal combination of the water squeezed out of the consolidating bay mud and infiltration of precipitation through the interim cover, and hydrographs showed that leachate elevations were increasing throughout the fill at a relatively constant rate. Numerical modeling suggested that future impacts to groundwater could be mitigated by reducing the amount of leachate within the parcel. As a result, interim closure activities in 1992 included installation of several interior leachate extraction wells and construction of an onsite leachate treatment plant.

Final closure design for the North Parcel also focused on leachate management and control and included (1) installation of a perimeter leachate extraction system, (2) installation of a combination geosynthetic clay liner and geomembrane final cover system, and (3) evaluation of upgrades to the treatment plant. A target 95 l/min extraction rate for the treatment plant was ultimately selected based largely on the estimates of leachate generation and the amount of time required to reduce an approximately 3.7 m mean sea level average leachate level within the fill to an average elevation of 0 m mean sea level. The assumption was that the final cover would, for all practical purposes, eliminate leachate generation from infiltration.

Final cover closure construction began in 1997 and was substantially completed in 1998. Leachate monitoring data collected during construction showed relatively direct and measurable response to heavy precipitation, particularly when portions of the waste were exposed during construction. Leachate elevation monitoring data collected since the cover was completed show measurable decreases in leachate volume, as shown in Figure 5.6. The reduction in total leachate volume is consistent with the elimination of infiltration as a source of leachate generation. Leachate extraction and piezometer monitoring data indicate that the current leachate generation rate in the parcel is on the order of 45 l/min and results principally from groundwater inflow from consolidation of the underlying bay mud. The results have, within a relatively short period of time, allowed the landfill operator to adjust leachate extraction from the parcel and to better manage the overall costs associated with postclosure monitoring and maintenance. Equally significant, groundwater monitoring since the final cover was completed shows that groundwater quality has improved at the perimeter of the parcel as leachate levels in the parcel have decreased.

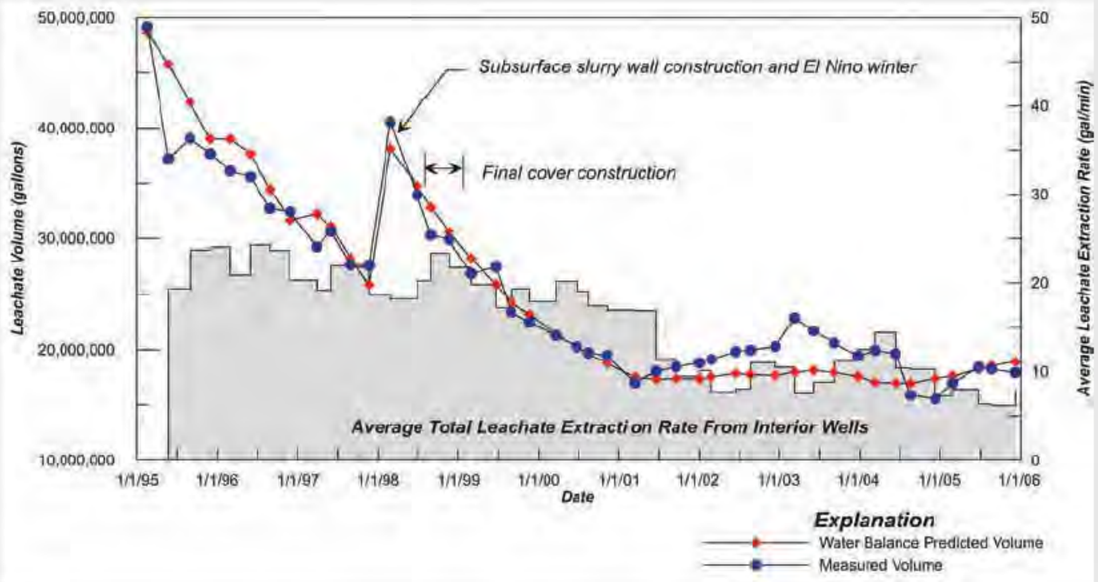


FIGURE 5.6 Comparison of predicted water balance and measured leachate volumes in the North Parcel of Acme Landfill. SOURCE: Courtesy of RMC Geoscience.

ing the effectiveness of capping as a remedy at RCRA or CERCLA sites.

The observed and inferred good performance of cover systems does not suggest that periodic maintenance to repair erosion or other forms of cover distress (e.g., cover veneer stability failures) is not required. In fact, most engineered cover systems rely on regular maintenance to maintain their integrity. Some systems (e.g., for low-level radioactive and mixed waste) are designed to survive for 1,000 years or more without regular maintenance. In these cases, cover systems are often designed to emulate natural landscapes that have resisted erosion, and perhaps limited infiltration and intrusion, for thousands of years. "Natural analog" cover systems typically have low profiles to provide stability and minimize erosion, and they are populated by a natural succession of native plant species until the native landscape is emulated (Caldwell and Reith, 1993). The Fernald mixed waste landfill in Ohio has such a cover system. This system has an essentially flat barrier layer, with a top deck grade not greater than 6 percent and side slopes with a maximum inclination of 6H:1V.

In summary, while the overall performance of covers that have been constructed to date is generally satisfactory, erosion and instability continue to be persistent problems for relatively steep (i.e., steeper than 4H:1V) covers. Most cover systems rely on continuing maintenance for good operation. Maintenance-free covers have not yet been demonstrated to work.

Percolation Through Cover Systems

A primary function of most (but not all) cover systems is minimization (or sometimes simply control) of percolation through the cover and into the underlying waste or impacted soil. Measured percolation rates through cover systems can provide a variety of insights on the performance of the cover system, including the effectiveness of the surface at promoting runoff, the effectiveness of soil layers above or within the barrier at storing and removing moisture, the effectiveness of drainage layers at minimizing the hydraulic head on the underlying barrier layers, and the effectiveness of evapotranspirative barrier layers at minimizing leakage.

Percolation rates for cover systems containing single compacted soil layers have been measured using pan lysimeters in test plots in four climatic regions for durations up to 7 years (Benson, 1999, 2001). The compacted clay layers were between 600 and 900 mm thick and they were overlain by varying thicknesses of vegetative layers. Percolation through the cover systems generally increased at all of the test sites during the respective study periods. Percolation rates for the compacted clay layers were initially between 10 and 50 mm/year in humid climates (approximately 1 to 4 percent of the total precipitation) and approximately 1 to 4 mm/year in semiarid climates (approximately 1 to 2 percent of precipitation). Percolation increased if cracks developed in the clay layers: rates of 100 to 150 mm/year were measured in humid climates (10 to 20 percent of precipitation) and approximately 30 mm/year in semiarid climates (approximately 10 percent of precipitation). Percolation as high as 500 mm/year was observed through a cover that included a cracked single compacted clay liner at a landfill located in a warm humid area. These data are consistent with other work showing that desiccation, freeze/thaw, root penetration, animal intrusion, and settlement-induced cracking were major factors affecting the performance of covers with compacted clay layers (Bonaparte et al., 2002). The Bonaparte et al. study recommended against using compacted clay layers alone in the final cover systems of landfills, especially landfills with wastes that settle significantly.

Compacted clay layers are also used in conjunction with geomembranes to form composite barriers in cover systems. Percolation rates obtained from lysimeters beneath cover systems containing composite barriers in test plots in a humid region were summarized by Benson (1999). The data show that percolation through the covers generally increased with time over the 7-year study period. The measured percolation rate at the end of the study period was 2 to 3 mm/year, which was significantly lower than the percolation rate of approximately 150 mm/year measured in test plots with a single compacted clay liner cover at the same site. No cracking of the clay liner beneath the geomembrane was observed at the site, consistent with findings from the Bonaparte et al. (2002) study. Additional studies indicated that percolation through composite barriers was generally less than 10 mm/year, with values generally in the range of 1 to 3 mm/year in humid climates and 0.1 to 1 mm/year in semiarid climates (Benson, 2001).

Another indirect measure of cover system performance is the reduction in leachate pumping rates following capping of a landfill containing a leachate collection and removal system. Data in Bonaparte (1995) and Bonaparte et al. (2002) indicate that a well-designed and well-constructed final cover system can significantly reduce leachate generation rates, and by inference percolation rates, by over an order of magnitude in MSW landfills. A drop in flow rate from 60 percent of the annual rate of precipitation when the landfill cells were receiving waste to 13 percent and 1 percent of annual precipitation after 1 and 10 years of closure, respectively, was reported in a hazardous waste containment facility (Haikola et al., 1995). The landfills in the facility were covered with a barrier system that included a composite liner. Statistical analysis of the data indicated that flow into the leachate collection system was generally independent of precipitation subsequent to placement of a final cap over the wastes.

Gas Emissions

Gas emissions from landfills are highly variable in both space and time. The variations reflect differences in waste conditions (type and decomposition rates), type and thickness of cover materials, atmospheric conditions, and measurement techniques. Point measurements of methane emissions in MSW landfills yield values that vary over seven orders of magnitude (from 0.0004 to 4,000 g/m^{2}d; Bogner et al., 1997).

Bonaparte et al. (2002) reported systematic problems with gas emissions for landfills with compacted clay liner covers attributed to cracking in the clay due to desiccation and settlement. Comparisons of measured cover emissions and of methane recovery rates (from landfill gas collection systems) for a single compacted clay liner (1 m thick), single GCL, and a single geomembrane liner were reported by Spokas et al. (2006). All of the barrier materials were covered with 300 mm of topsoil. The GCL was underlain by a sand layer, and the geomembrane was underlain by a gravel layer. The study was conducted at three landfills in France in test cells that were filled with similar wastes but capped with the different cover configurations. The emissions were significantly higher and the recovery rate was significantly lower for the GCL cover than for the other covers.

In laboratory studies, Vangpaisal and Bouazza (2004) showed that GCL gas permeability was sensitive to variations in water content in the GCLs. Decreases of five to seven orders of magnitude in the gas permeability of the GCLs at water contents above approximately 70 percent were measured. Decreases were generally lower for GCLs tested subsequent to hydration under unconfined conditions than if hydrated under confined conditions (σ = 20 kPa), but gas permeability still decreased by several orders of magnitude. Gas permeability was also affected by GCL structure (needle-punched, stitch-bonded, etc.) and the form of bentonite (powder or granular), with higher permeabilities obtained for stitch bonded and granular GCLs. These findings suggest that, when GCLs are used in covers, they should have a sufficient thickness (900 mm or greater) of soil above them to provide sufficient overburden pressure to minimize the effects of freeze/thaw or desiccation.

Data on the effectiveness of evapotranspirative cover systems in controlling gas migration are sparse, but these types of covers are expected to be more transmissive to gas transport than systems containing compacted clay layers, hydrated GCL, and geomembrane barrier materials. However, both gas transport potential and gas generation rates must be considered in assessing the effectiveness of alternative covers for gas migration control. Furthermore, methanotropic bacteria can oxidize methane in vegetated soil covers (Bogner at al., 1997; Liptay et al., 1998; Borjesson et al., 2001; Park et al., 2005). Comparison of emissions from a Florida landfill with a lightly vegetated cover over relatively young wastes, a heavily vegetated cover over relatively old wastes, and a daily cover with no vegetation on fresh wastes indicated that emissions were highest from the lightly vegetated cover and lowest from the highly vegetated cover (Abichou et al., 2006). Emissions were generally higher in flat areas than in sloped areas. In addition, emissions were relatively uniformly distributed across the lightly vegetated cover but were generally localized to defects (cracks) in the thickly vegetated cover. Methane oxidation was also higher in the thickly vegetated cover than in the lightly vegetated cover (Abichou et al., 2006). The low emission rates in the nonvegetated areas may be related to periodic scarifying and recompacting of the soil cover in these areas, a common operational practice at landfills.

In the long term, gas-generating waste in a landfill will degrade to the point where it no longer generates significant quantities of gas (from an environmental protection standpoint). The length of time required for this stabilization to occur depends on waste composition, climate, and landfill operational practices. For typical MSW landfills operated in compliance with EPA Subtitle D standards, the amount of time required to deplete 80 percent of the landfill's gas generation capacity is expected to be approximately 30 years at a temperate site and 80 years at an arid site (Bonaparte et al., 2002). However, because the decomposition process decays exponentially with time for a homogeneous mass and because different sections of large landfills have varying decomposition rates, the remaining 20 percent of the degradation capacity may take significantly longer. Furthermore, construction of a cover system that essentially prevents percolation of moisture into the waste can slow and virtually halt degradation. However, any subsequent breach of the cover system that allows renewed infiltration can restart or accelerate the degradation process. The long duration over which gas generation remains an environmental protection consideration is part of the motivation behind research to intentionally add oxygen and liquids to MSW landfills to accelerate decomposition processes.[1]

[1]For example, see <http://www.epa.gov/garbage/landfill/bioreactors.htm>.

Deterioration of Cover Systems

A smaller but unknown subset of the landfills surveyed by Bonaparte et al. (2002) had a final cover system that had been constructed and in operation for more than several years. Of the 79 landfills for which problems were reported, 24 of the problems were related to performance of the cover system. Of these problems the vast majority (18) were caused by slope failures, mainly related to rainfall events. In addition, two instances of erosion were identified: one of a topsoil layer and one of an erosion control mat, both of which were on long, 3H:1V slopes. Two problems were caused by differential settlement; in one case, settlement caused high-density polyethylene geomembrane boots (sleeves that maintain a seal between a geomembrane and a through-passing pipe) to tear where gas wells penetrated through the cover. In the other case, settlement caused GCL panels to separate at the overlapped seams. Finally, two problems were caused by construction. Deterioration also led to increases of one to two orders of magnitude in the saturated hydraulic conductivity of the clay barrier layer within 5 years of placement (Benson and Khire, 1997; Albrecht and Benson, 2002; Benson et al., 2007; EPA, 2007).

Thermal Conditions

Temperature variations have significant effects on cover systems because the soil components are susceptible to cracking from desiccation and freeze/thaw cycles. High temperatures can affect the durability of geosynthetic components in covers, and thermal gradients can affect the transport of liquids and gases through cover systems. The direction of heat flow through cover systems varies with the seasons; upward heat flow occurs in winter and downward heat flow occurs in summer. Exceptions may occur for covers in locations with extreme hot or cold climates. Measured thermal trends in a cover system at an MSW landfill in Michigan are illustrated in Box 5.5.

5.1.3 Leakage Through Vertical Barriers

The primary function of most vertical waste containment barriers is to control the lateral subsurface migration of liquids, either hazardous liquids themselves or contaminated groundwater. The performance of vertical walls used to contain contaminated soils and groundwater was assessed at 24 sites across the United States, most of which had been operational for less than 10 years (EPA, 1998). The contaminants at these sites were hazardous in nature and included heavy metals, volatile organic compounds, and polycyclic aromatic hydrocarbons. Seventeen of the sites included active containment systems, which maintained an inward gradient across the barrier walls. The other seven sites (usually the oldest systems) had passive containment systems with essentially no gradient across the walls. The wall types

BOX 5.5 Case History on the Temperature Environment for a Composite Cover System

This case history illustrates how temperatures in a cover system vary with time. Data were collected from a Michigan landfill with a cover system that included (from top to bottom) an approximately 900-mm-thick vegetative and protective soil layer, a geotextile-geonet composite, a geomembrane, and an approximately 900-mm-thick compacted clay layer. Temperatures have been measured through the cover profile and into the underlying soil foundation layer and top of the waste layer using thermocouple sensors (the depth of each sensor is presented in the legend in Figure 5.7) since 2002 (Yesiller and Hanson, 2003; Hanson et al., 2005b). Temperatures in the cover system followed seasonal variations and demonstrated phase lag and amplitude decrement with depth. The temperature of barrier components (geomembrane and compacted clay) varied seasonally by over 15°C. The average absolute thermal gradient was over 20°C/m at the location of the compacted clay component of the cover system.

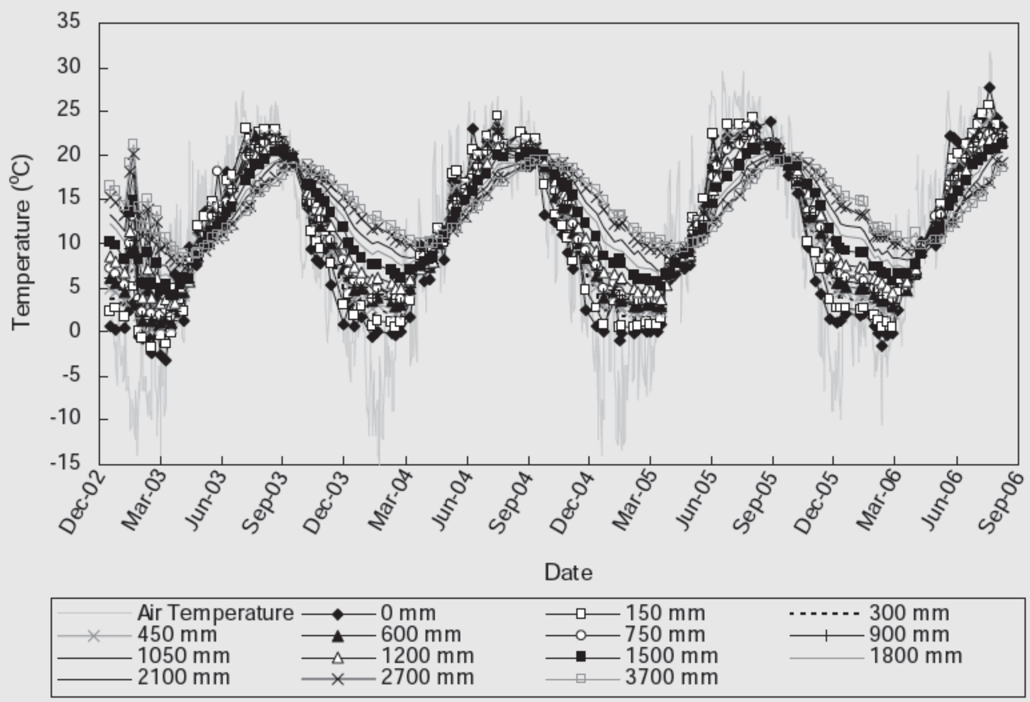


FIGURE 5.7 Example of cover temperatures in a landfill cell (0 mm indicates the location of ground surface). SOURCE: Adapted from Yesiller and Hanson (2003).

included soil-bentonite (21 sites), cement-bentonite, clay, and vibrating beam walls. Performance was evaluated mainly using groundwater quality and hydraulic head criteria. The study found that 83 percent of the 20 sites met design objectives (e.g., wall hydraulic conductivity and compatibility, wall thickness, key-in depth) and performed satisfactorily. The most significant factor in poor performance was leakage near areas where the walls are keyed into underlying low-permeability barrier layers. The leakage was attributed to poor construction of the keys. Data from four sites with seepage cutoff walls indicated that all of the walls constructed using soil-bentonite, concrete, and plastic concrete performed their intended functions well (EPA, 1998). A case history illustrating field performance of a vertical barrier is presented in Box 5.6.

Few problems with vertical waste containment barrier walls have been reported. However, this may simply reflect the paucity of data, either good or bad, on the actual performance of these walls. Construction defects in soil-bentonite and soil-cement-bentonite slurry walls, nonuniformities that develop during their placement, and subsequent settlement of wall materials could all lead to zones of higher hydraulic conductivity and increased contaminant transport. In the case of concrete panel walls, geosynthetic panel walls, and sheet pile walls, improperly filled joints between adjacent panels can be sources of leakage. Inadequate seals or keys into natural barriers beneath the wall can also be a problem for all types of vertical barrier walls.

Extraction trenches and vertical walls also serve as vertical barriers in some containment systems. However, the

BOX 5.6 Case History on Vertical Wall Performance

This case illustrates how, under the right conditions, a gravel-filled trench can serve as an effective vertical barrier against offsite migration of hazardous and toxic wastes. About 75 million liters of liquid hazardous and toxic industrial wastes were disposed of at the Hardage Site near Oklahoma City, from 1972 to 1980. Following Superfund designation of this site and a court-ordered excavate, incinerate, and reentomb remedy for its remediation, the Hardage Steering Committee, a consortium of the potentially responsible parties for the site, developed an alternative remedy that it believed would better protect the environment and be more cost effective than the EPA remedy. A trial before a federal district judge in late 1989 resulted in a decision in favor of the steering committee's remedy.

The Hardage disposal area is underlain by fine-grained sandstones and siltstones, which are sufficiently strong to allow deep vertical trenches without lateral support. Remediation actions included construction of an 820-m-long, 20-m-deep (on average), 0.9-m-wide gravel-filled trench keyed into an underlying low-permeability layer to intercept contaminated fluids and a low-permeability composite cap over the disposal area. The gravel-filled interceptor trench, a unique feature of this waste containment system, is shown in plan view in Figure 5.8 and in cross section in Figure 5.9. Originally a plastic concrete (cement bentonite) cutoff wall was planned. However, EPA concerns about the possibility of fractures in the Stratum III rock, the buildup of fluid pressure against the cutoff wall, and leakage through the wall led to adoption of the V-shaped gravel-filled interceptor trench.

Numerical modeling of groundwater flow was used to establish the trench location. The trench bottom is sloped to a series of liquid recovery sumps, with pumps for removal of the captured flow. The performance of the trench segments is monitored by water level measurements in the recovery sumps and piezometers located along the trench between the sumps. The site remedy has been operational since September 1995. The First Five Year Review (in 2002) indicated that all immediate threats at the site had been addressed and that the remedy components are expected to continue to protect human health and the environment (EPA, 2002b). The use of a cutoff trench appears to be effective as a cutoff wall for containment of liquid wastes at this site. It offers the additional advantages that liquids reaching the trench can be removed and treated, quantities are known, and an inward gradient is maintained.

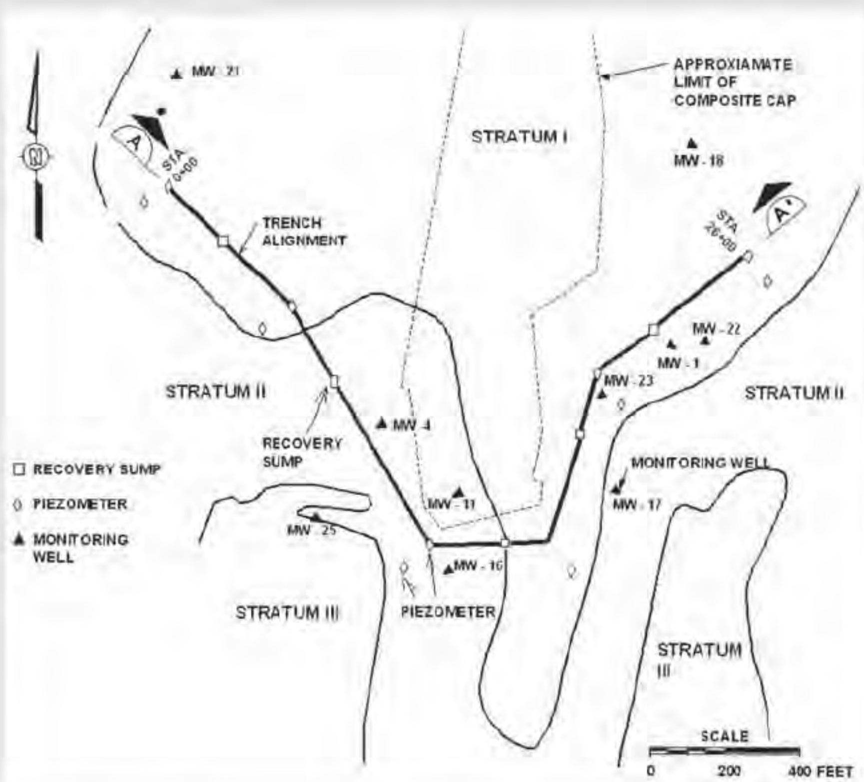


FIGURE 5.8 Location and layout of the gravel-filled interceptor trench at the Hardage Superfund site. SOURCE: Mitchell and Mitchell (2004).

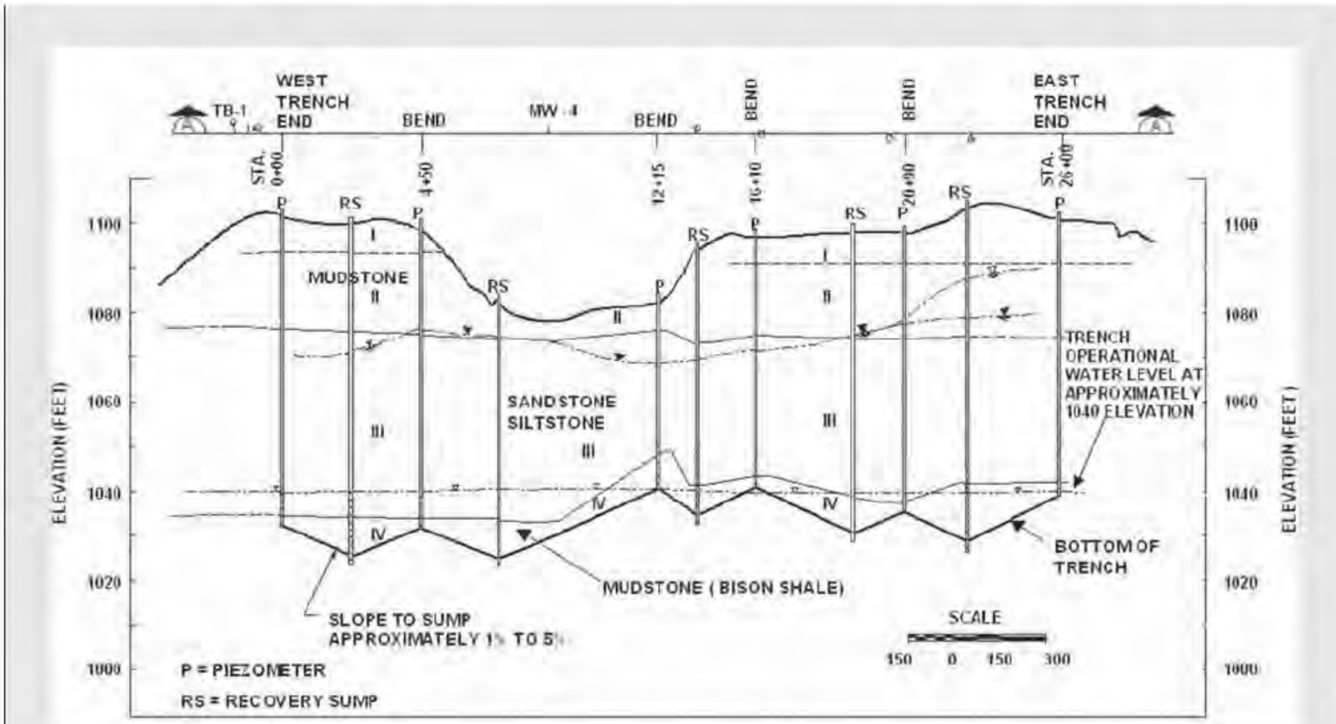


FIGURE 5.9 Profile along the V-shaped interceptor trench showing rock strata, fluid levels, recovery sump, and piezometers locations. SOURCE: Mitchell and Mitchell (2004).

committee was unable to find published data on the performance of these systems. At a visit to the Love Canal site in New York, the committee was told that the vertical extraction trench used to control contaminant migration at the site was performing well but that no published data were available. Since extraction trenches and wells are active systems, their long-term performance relies on continued operation of the associated pumping system as well as the ability of the trench or well to resist clogging.

5.2 PREDICTING THE PERFORMANCE OF BARRIER SYSTEMS

Prediction of the properties of the components of engineered barrier systems was discussed in Chapter 4. This section describes predictive models of how the individual components of the system interact to contain contaminant transport. Both elements are necessary in order to adequately predict containment system behavior.

5.2.1 Predicting the Performance of Liner Systems

Assessing Leakage Through Smooth Composite Liners

Leakage through composite liners can be calculated using empirical equations, analytical equations, or numerical analysis (Rowe, 2005). The analyses assume that holes are present in geomembrane components of the barrier systems. Empirical equations are established by curve-fitting families of solutions from analytical equations (Giroud and Bonaparte, 1989; Giroud, 1997); example results are provided in Table 5.3. Analytical equations involve either the assumption of perfect contact (Rowe and Booker, 2000) or lateral migration in a transmissive zone below the geomembrane combined with one-dimensional flow into the underlying soil (Jayawickrama et al., 1988; Rowe, 1998; Touze-Foltz et al., 1999, 2001a). Numerical methods allow modeling of actual three-dimensional conditions near a hole (Foose et al., 2001) or the complete variability of the interface topography if it is known (Cartaud et al., 2005a).

The assumption of perfect contact between the geomembrane and the subsoil can provide a lower bound to the leakage through holes in a geomembrane over a compacted clay liner or GCL. Assuming there is a uniform transmissive zone between the geomembrane and clay liner, Rowe's (1998) direct contact analytical equation or Giroud's (1997) empirical charts can be used to calculate leakage. The assumption of one-dimensional flow in the liner is not correct, however. Nonetheless, for a geomembrane over a GCL, Rowe's direct contact analytical solution provided good agreement with the results of numerical analysis for typical reported transmissivity values (Foose et al., 2001). Similarly, Rowe (2005) found

TABLE 5.3 Representative Leakage Rates for Single Geomembrane and Composite Liners

	Steady-State Leakage Rate[a] (lphd)	
Hole Frequency (per hectare)	Single Geomembrane Liners[b]	Composite Liners[c]
0	1	1
1	100 to 100,000	1 to 100
10	1,000 to 1,000,000	1 to 1,000
100	10,000 to 10,000,000	10 to 10,000

[a]For range of hole diameters from 1 to 10 mm and hydraulic heads from 0.1 to 10 m.

[b]Assumes steady state flow through an orifice with a coefficient of discharge of approximately 1.

[c]For an underlying clay layer that has a thickness of 0.915 m, a hydraulic conductivity of 1×10^{-9} m/s, and a freely draining boundary at its base. Assumes that holes in the geomembrane do not extend through the underlying clay liner, a gap with a uniform thickness between the geomembrane and the clay of 0.01 to 0.05 mm (i.e., no wrinkles), and a no-flow boundary equidistant between holes.

excellent agreement (error of less than 4 percent) between the direct contact analytical solution and an axisymmetric numerical analysis for interface transmissivities within the practical range of interest for both a GM-GCL composite liner and a GM-CCL composite liner.

Both the numerical and analytical solutions assume a uniform transmissivity of the interface. Analytical solutions have also been developed for regular variations in the transmissivity of the interface (Touze-Foltz et al., 2001a), and numerical methods are available to model more complex situations (Cartaud et al., 2005b). Although these may be useful in interpreting laboratory tests where the actual interface topography is well defined, in practical situations the interface topography will vary significantly (e.g., as is evident from the work of Cartaud et al., 2005a, 2005b) and is unknown at the location of the (assumed) holes used in design calculations. Thus, the more simplified approaches used in conjunction with a range of likely transmissivities will provide the information needed for design purposes provided there are no significant wrinkles in the geomembrane. If there are wrinkles, as is often the case, predictions made using simplified approaches will not be consistent with the observed leakage and the likely number of holes/ha for either GM-CCL or GM-GCL systems (Rowe, 2005).

Assessing Leakage Through Composite Liners with Wrinkles

Wrinkles in a geomembrane exacerbate the effect of holes on leakage rates if the wrinkles coincide with the holes (Rowe, 2005). The potential for contaminant migration increases through a hole in the geomembrane at or near the wrinkle. Future holes are also more likely to develop due to stress cracking at points of high tensile stress in the wrinkle. Wrinkles in a geomembrane arise both during construction and, in particular, from thermal expansion when the geomembrane is heated by the sun after placement. These wrinkles do not disappear when the geomembrane is loaded (Stone, 1984; Soong and Koerner, 1998; Brachman and Gudina, 2002). Pelte et al. (1994) reported wrinkles that were 0.2 to 0.3 m wide and 0.05 to 0.1 m high at a spacing of 4 to 5 m. Touze-Foltz et al. (2001b) reported wrinkles that were 0.1 to 0.8 m wide and 0.05 to 0.13 m high at a spacing of 0.3 to 1.6 m. Rowe et al. (2004) report a case of 1,700 wrinkles/ha.

Validating Models for Leakage Through Liner Systems

Techniques for calculating leakage through composite liners can be validated using data from landfills with leak detection systems. Predictions from equations that assume holes are in direct contact between the geomembranes and the underlying clay liners (Rowe, 2005) were compared with leakage rates reported by Bonaparte et al. (2002). The comparison showed that 20 to 30 holes/ha (with a hole radius of 10 mm and depth of fluid above the geomembrane of 0.3 m) would be required to match the observed leakage rates if contact with the compacted clay liner (GM-CCL) is poor (as defined by Giroud and Bonaparte, 1989) and that 90 to 100 holes/ha would be required if contact with the compacted clay liner is good. Many more than 100 holes/ha would be required to explain the maximum flow observed. For composite liners with a GCL (GM-GCL), about 40 to 100 holes/ha would be required to explain the typical range of flow values (Rowe, 2005). The maximum observed leakage was more than an order of magnitude greater than the predictions of leakage from 100 holes/ha. Although this leakage could be explained by a large number of holes, the number of holes predicted by these calculations is much higher than can be reasonably expected. Thus, the use of calculations that assume direct contact (or even poor contact) between geomembranes and clay liners and that do not explicitly consider wrinkles is not appropriate for estimating leakage through composite liners unless the landfill is constructed with negligible wrinkles. Wrinkles are common in North American landfills. The extent of wrinkles has recently been illustrated by Chappel et al. (2007).

When holes in wrinkles are factored into the calculation, the typically observed leakage could be readily explained by 12 to 22 holes in 10-m-long wrinkles/ha or as few as 1 to 3 holes in 100-m-long wrinkles/ha (Rowe, 2005). (The length of a wrinkle is the total linear distance that fluid can migrate along a wrinkle and its interconnections.) A similar comparison for a geomembrane over a GCL indicated that the observed leakage could be explained by 2 to 3.5 holes in 10-m-long wrinkles/ha, while the maximum leakage can be explained by about 5 holes in 100-m-long wrinkles/ha. Thus, the typical observed leakage for composite liners with both compacted clay liners and GCLs can be readily explained by holes in wrinkles for the typical number of holes/ha and reasonable combinations of other parameters (Rowe, 2005).

Leakage is a function of the size and number of holes (especially in wrinkles, as discussed above) and the head on the liner. The head on the liner depends on the rate at which leachate is pumped from the primary leachate collection system and the hydraulic conductivity of the drainage layer. A numerical model was developed by Gilbert (1993) to predict the rate of leakage through a liner system as a function of the pumping rate from the primary collection system versus time (as a proxy for the infiltration rate into the drainage system from the overlying waste) and the geometry and configuration of the leachate collection system and liner. The side slopes, base slopes, pipes and sumps, and the properties of the materials in the liner system were taken into account. This model was calibrated with measured pumping rates for double-liner systems at 16 hazardous waste landfill cells. The leakage rate through the liner system was approximately proportional to the square root of the flow rate into the leachate collection system. The average proportionality constant was about $l/d \big/ \sqrt{l/d}$ (e.g., a flow rate of 5,000 l/day into the collection system produced an average leakage rate of about 50 l/day into the leak detection system).

The above model results show that to obtain accurate predictions of leakage through composite liners it is necessary to take into account holes in wrinkles and elevated head.

Diffusive Contaminant Transport Through Liner Systems

Contaminant transport via diffusion has been well documented. Examples include the migration of chloride to a depth of about 0.75 m in 4.25 years (Reades et al., 1989), the migration of volatile organic compounds up to 0.6 meters in about 4.25 years at the Keele Valley Landfill (Barone et al., 1993), and the migration of heavy metals less than 0.1 m in 15 years at the Confederation Road Landfill (Yanful et al., 1988). The extent of diffusion through thick clay deposits over a period of 10,000 to 12,000 years has been successfully predicted by a number of investigators (e.g., Quigley et al., 1983; Desaulniers et al., 1981; Rowe and Sawicki, 1992) using the diffusion-advection equation (where diffusion dominates). Diffusion of contaminants through clay liners beneath waste has also been successfully predicted (Rowe, 2005).

A composite liner containing a 1.5-mm high-density polyethylene geomembrane over a 3-m-thick compacted clay liner was investigated after 14 years of use as a leachate lagoon liner (Rowe, 2005; Lake and Rowe, 2005b). The geomembrane had no overlying protection layer. Inspection at decommissioning revealed numerous holes in the geomembrane, and cores through the compacted clay showed that contaminants had diffused about 1.7 m in 14 years. Observed and predicted diffusion rates in the clay would match if it is assumed that the geomembrane failed in the first 4 years of operation. A more positive example of composite liner performance was reported by Rowe (2005) for two test sections at the Keele Valley Landfill. For the section with only a compacted clay liner, a clear diffusion profile is evident with ion migration to a depth of more than 1.1 m (data limited by the depth of the lowest monitor at 1.1 m) over 12 years. The composite-lined section showed no evidence of a concentration profile for ionic species over the same time period, and the measured conductivity was representative of background values. Both findings are consistent with predictions based on laboratory-determined parameters. This suggests that (1) there is negligible advective flow (leakage) through the geomembrane near the conductivity sensors, and (2) there has been negligible diffusion of ionic species through the geomembrane in 12 years.

Overall, the available data suggest that current techniques for predicting diffusive contaminant transport give reasonable results when compared with observed field behavior. Consideration needs to be given to diffusion in well-designed and well-constructed barrier systems where advective transport (leakage) is small.

5.2.2 Vertical Barrier Performance Modeling

The performance of vertical barriers can be modeled using two-dimensional solutions for advective and diffusive flow. Advection and diffusion can be modeled separately or together using differential equations for combined advective diffusive flow (Krol and Rowe, 2004). There are a limited number of closed-form solutions to these equations, and most solutions used in practice today are numerical. They are generally based on the assumption of saturated flow. The required input parameters include the geometry of the problem, the material properties (e.g., saturated hydraulic conductivity, diffusivity of the materials with respect to the migrating compounds of interest, sorption/desorption coefficients, reactive flow parameters), and the boundary conditions (e.g., hydraulic heads or fluid flux, chemical concentration or chemical compound flux). Furthermore, changes in material properties and boundary conditions with time are required to predict long-term performance. These changes with time are perhaps the most difficult parameters to evaluate given the sensitivity of the material properties to environmental impacts, including chemical concentrations in the pore fluid, deformations of the barrier system, and temperature. Moreover, diffusive and advective flow modeling of cementitious vertical barriers is further complicated by the difficulty of separating the material/physical coefficients from the chemical coefficients. Studies show that, while it is feasible to predict contaminant transport through vertical barrier walls around contaminated sites (e.g., Krol and Rowe, 2004), few field data are available to evaluate the actual performance of the walls that have been constructed to date.

5.2.3 Predicting the Performance of Covers

Performance evaluations of waste containment covers are generally based on predictions of the amount of water that percolates through the cover. Percolation through the cover is the primary performance index because this water migrates into and possibly through the underlying waste, generating gas, solid waste leachate, and/or acid drainage that can lead to ground and surface water contamination or other adverse environmental impacts. Percolation is commonly predicted from a water balance analysis that includes processes such as surface runoff, evaporation, transpiration, internal lateral drainage within the cover or intralayer flow, and soil water storage.

Cover performance assessments also can be based on the gas flux. A gas flux criterion may be important, for example, when minimizing the ingress of oxygen into sulfidic mine tailings would prevent or minimize the generation of acid drainage, when preventing the egress of methane gas generated from biodegradation of solid wastes, or when minimizing the egress of radon gas from radioactive waste materials, such as uranium mine tailings (Shackelford, 1997).

The performance of cover systems also depends on the system integrity. Cover system integrity is maintained by providing sufficient resistance to wind and surface water erosion, providing adequate stability for the cover on side slopes, minimizing the amount of differential settlement of the cover to prevent excessive cracking and leakage, and minimizing the effects of environmental distress of the cover, such as desiccation cracking due to wet/dry cycles and/or freeze/thaw cycles.

Models for Predicting Cover Performance

Some of the models typically used to predict the percolation performance (i.e., water balance) of cover systems are listed in Table 5.4. The most widely used model is probably the Hydrologic Evaluation of Landfill Performance (HELP) model (Schroeder et al., 1994). Unlike most other water balance models, HELP assumes unit gradient flow and an unsaturated hydraulic conductivity that varies with water content in accordance with Campbell's equation for all layers except "barrier layers." For clay barrier layers, a saturated hydraulic conductivity is used and the hydraulic gradient is computed based on the depth of liquid pooled on the surface and an assumed pore water pressure of zero at the base of the clay layer. The gradient is set equal to zero when no water is pooled on the surface of the clay barrier. Composite barriers are simulated using a Giroud-type equation. This relatively simple assumption for barrier layers in HELP, together with relatively simple algorithms for routing the water balance, minimizes data input requirements and shortens computational times but sacrifices accuracy and versatility (flexibility) with respect to evaluating the

TABLE 5.4 Models for Predicting Percolation Performance of Waste Containment Covers

Model	Process	Solved Parameters	Comments
FEHM	1D, 2D, 3D, transient FEM/ FVM	Multiphase, multicomponent heat, mass, gas, air, including double porosity flow; can solve contaminant flow as advection/ dispersion or particle tracking	Limited pre- and postprocessor with 3D grid generator available from independent sources; USA only
HELP	1D, quasi 2D, analytical	Water balance	Climate and soil database included
HYDRUS-2D	2D, transient and steady state FDM	Pressure, with vapor flow, temperature, and chemical transport	Pre- and postprocessor included; CAD mesh generation add on
LEACHM	1D, quasi 2D, transient and steady state FDM	Pressure, temperature flow, and chemical transport	Originally an agricultural model; quick run-times; free online
RAECOM	1D steady state radon-gas diffusion	Radon-gas concentration and flux through a multilayer system	Can automatically optimize layer thickness
SoilCover	1D, transient FEM	Pressure, temperature, vapor pressure, oxygen flux	Pre- and postprocessor included; code unavailable; freeware
TOUGH 2	1D, 2D, 3D transient and steady state IFDM	Pressure, temperature, vapor, gas in porous or fractured media	Limited pre- and postprocessor available from independent suppliers; code available; users can customize
UNSAT-H	1D, transient FDM	Pressure with vapor; temperature (optional)	Pre- and postprocessor available but excluded; code available
VADOSE/W	2D, transient and steady state FEM	Pressure, temperature, vapor pressure, oxygen or radon diffusion. Can be linked with slope stability software and contaminant transfer software	Enhanced pre- and postprocessor, climate and soils database, and user support included; commercially developed for cover/cap design

NOTES: FDM = Finite Difference Method; FEM = Finite Element Method; FVM = Finite Volume Method; IFDM = Integral Finite Difference Method.
SOURCE: Modified from Sleep et al. (2006). Reprinted with permission. Copyright 2006 by Taylor and Francis Group LLC, Books.

different factors influencing cover performance (Khire et al., 1997).

Comparisons of models with field-measured water balance data have shown that the water-routing algorithms incorporated in HELP do not accurately simulate the complex hydrodynamics of landfill covers (e.g., see Table 5.5). Accurate predictions of water balance in covers typically require algorithms that use much more sophisticated models for unsaturated flow that are based on Richards's equation and that use nonlinear functions to describe the distribution in unsaturated hydraulic conductivity (e.g., the van Genuchten-Mualem function) and soil water characteristic curves (e.g., Brooks-Corey or van Genuchten functions). All of the water balance models listed in Table 5.4 except HELP employ these more sophisticated unsaturated flow equations and functions and evapotranspiration models. Of course, the greater data input requirements and the longer computational times (days or weeks in some cases) make the use of these more sophisticated models less desirable for many practical applications.

Overall, the available evidence suggests that the HELP model provides relatively poor predictions of the performance of water balance systems, such as earthen final covers. Accurate predictions of the performance of water balance systems require more sophisticated models with greater input data and possibly longer run times.

Issues and Limitations

The ability to predict the performance of waste containment covers is limited by two problems in particular: (1) the existence of time-varying properties and processes (e.g., climate, vegetation, soil) and (2) the role of heterogeneities on flow through cover systems (Sleep et al., 2006). Because covers are exposed to the environment and are under relatively low confining stresses, they are susceptible to the impacts of surface and climatic processes. All models of cap performance require climatic data, including precipitation, temperature, and solar radiation to determine infiltration and evapotranspiration. Although historical data are available for many locations, methods for estimating extreme values of these variables are not well developed.

Physical deterioration must be considered when modeling cover percolation (e.g., see Benson et al., 2007). Changes in surface vegetation affect surface runoff, erosion, and evapo-

TABLE 5.5 Selected Studies Comparing Predicted and Field-Measured Performance of Water Balance Systems

Reference	Location (Climate)	System Monitored	Model(s)	Conclusions
Fayer et al. (1992)	Hanford, WA (semiarid)	8 field lysimeters	UNSAT-H	Water-balance models should be calibrated with field data to improve predictions; hydraulic conductivity, snow cover, hysteresis in the SWCC, and PET have a significant effect on predictions
Fayer and Gee (1997)	Hanford, WA (semiarid)	Nonvegetated weighing lysimeter	UNSAT-H, HELP	Models based on Richards's equation should be used; hysteresis in the SWCC should be considered when predicting percolation through landfill covers; heat flow was a minor factor in the predictions
Khire et al. (1997)	Georgia (humid) and eastern Washington (semiarid)	Landfill covers	UNSAT-H, HELP	UNSAT-H underpredicted and HELP overpredicted percolation, except where damage caused preferential flow and increased measured percolation; predictions with UNSAT-H were in better agreement with the measured water balance, although both models captured the seasonal trends in SRO, ET, and SWS
Scanlon et al. (2002)	Texas (arid) and Idaho (semiarid)	Unvegetated covers	HELP, HYDRUS-1D, SHAW, SoilCover, SWIM, UNSAT-H, VS2DTI	Models employing Richards's equation are superior to the HELP model; boundary conditions (seepage face versus unit gradient), duration of the precipitation event, and the SWCCs also significantly affected predictions
Scanlon et al. (2005)	Texas (arid) and New Mexico (semiarid)	ET soil covers	UNSAT-H	Accurate predictions of transpiration are critical to accurate predictions of water balance of ET covers; vegetative response to changes in SWS should be simulated internally in water balance analyses rather than prescribed in the input data

NOTES: ET = evapotranspiration; PET = potential ET; SRO = surface runoff; SWCC = soil water characteristic curve; SWS = soil water storage.

transpiration. Covers are (1) penetrated by roots or burrowing animals, resulting in the generation of highly conductive pathways for water infiltration; (2) cracked because of settlement; and (3) desiccated through environmental stresses, such as freeze/thaw cycles and wet/dry cycles (e.g., Albrecht and Benson, 2001). At present there is no reliable way to predict the occurrence or effects of such time-dependent changes in material properties and processes.

Finally, a capability to predict the occurrence and impact of local heterogeneities in soil on the flow through cover systems does not yet exist. Most predictions are based on models that assume the properties of each soil layer in a cover system are homogeneous. However, the existence of local heterogeneities resulting from compaction, settlement-induced cracking, and desiccation can result in significant differences between predicted and actual performance.

5.2.4 Predicting Gas Transport Through Containment Systems

Transport of gases through individual barrier materials as well as through barrier systems in bottom and cover liners and vertical barriers has received far less attention than the transport of liquids through these materials and systems. In general, it is assumed that the trends in hydraulic conductivity apply broadly to gas conductivity (i.e., low hydraulic conductivity indicates low gas conductivity) and that conditions and mechanisms that change hydraulic conductivity cause a comparable change in gas conductivity. Advective transport of gas is expected to control gas movement through porous materials (e.g., leachate collection layers), and diffusive transport is expected to be the dominant mechanism for geomembranes. For low-permeability soil barrier materials (e.g., compacted clay liners, GCLs), the dominant gas transport mechanism is not clear and may depend on the degree of saturation of the material. Although laboratory tests have been reported on gas transport in porous barrier materials (e.g., Izadi and Stephenson, 1992; Vangpaisal and Bouazza, 2004), these data are not normally collected. Water and solvent vapor transmission rates for geomembranes are reported by Matrecon, Inc. (1988), and diffusive flux of methane for high-density polyethylene is reported in Spokas et al. (2006). Regulations require emissions data (e.g., 500 ppmv for methane), not gas conductivity or diffusion rates for individual barrier materials or barrier systems (EPA, 2005).

As discussed above, an effective cover (with respect to liquid percolation) does not necessarily ensure that gas will not escape; the details of the leachate collection/removal system are important to make sure the gas does not bypass the containment system.

5.2.5 Predicting the "Active" Lifetime of Waste

The active lifetime of waste in a landfill (i.e., the time span over which the waste can actively generate gas or leachate with potentially harmful constituents) is an important factor in evaluating the long-term performance of engineered barriers, not only because it defines the desired service life of the containment system but also because gas and leachate can interact with containment system components and thereby affect their longevity. Analysis of leachate and gas data from 50 municipal solid waste landfills in Germany with ages up to 30 years indicated that concentrations of individual leachate constituents rather than gas generation controlled the duration of the postclosure care period (Kruempelbeck and Ehrig, 1999). Extrapolations of temporal variations of measured leachate quality in the 50 German landfills as well as other studies suggested that postclosure care periods were highly variable (<10 to 1,700 years).

The active life of other waste types may be significantly different than the active life of municipal solid waste. The active life of hazardous and low-level radioactive waste is typically on the order of centuries to a thousand years (GAO, 2005; NRC, 2006). For hazardous waste remediation sites, however, the active life may vary from tens of years (e.g., when capping is an interim solution accompanied by source control measures) to hundreds or thousands of years (e.g., for dense nonaqueous-phase liquids in fractured rock strata) and must be determined on a case-specific basis.

In summary, landfills, especially some of the larger ones, are likely to require attention for centuries, not decades.

5.2.6 Local and Global Slope Stability

The integrity and performance of landfill liner, cover, and vertical barrier systems can be affected by both global and local slope stability. Of the 85 problems with landfill containment systems identified in Bonaparte et al. (2002), 14 percent involved liner instability and 21 percent involved cover system instability. Stability issues associated with vertical barrier construction are discussed in Filz et al. (2004). Stability failure of a barrier system can be defined in terms of two different performance states:

- complete loss of stability or function (e.g., waste slope or slurry trench side wall failure), also known as a global stability failure; and
- impairment of a structure's function (e.g., deformation of a landfill liner leading to loss of function, local soil slumping during slurry trench excavation for a vertical barrier), also known as a local stability failure.

Global stability considerations include static and seismic stability of the foundation, waste mass, and cover systems for landfills (Mitchell and Mitchell, 1992) and side wall stability for vertical barriers (Fox, 2004). Figure 5.10 illustrates different global stability modes for landfills. A global stability failure in a landfill is likely to breach any barrier layer that it crosses. A global stability failure during construction of a vertical barrier system will cause the excavated trench to

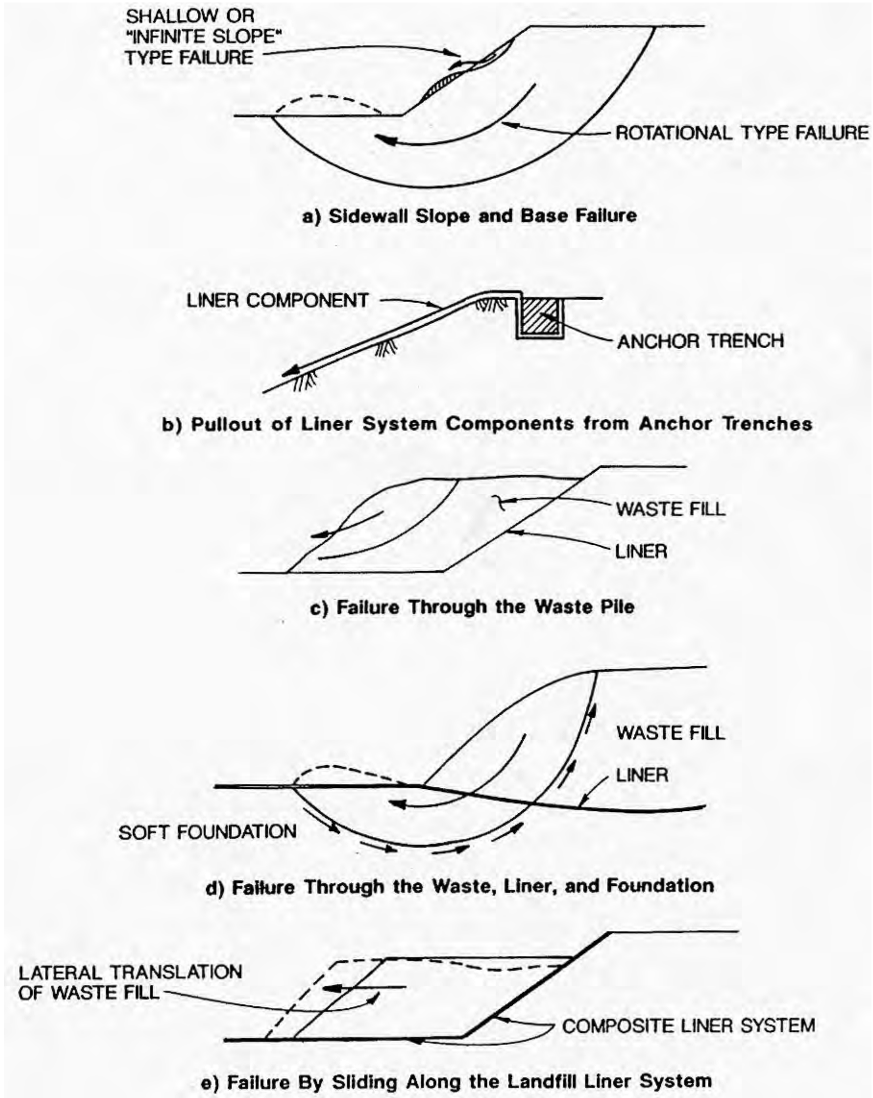


FIGURE 5.10 Modes of global stability failure. Upward-facing arrows in (d) show shearing resistance along the slip plane. SOURCE: Mitchell and Mitchell (1992). Reprinted with permission from the American Society of Civil Engineers.

close, and a failure after construction would likely breach the barrier layer. Even if the barrier is not breached, a global failure may significantly diminish its capacity to withstand external loads (e.g., soil slipping off of a geosynthetic barrier layer, exposing it to ultraviolet radiation and other external loads). Generally, limit equilibrium stability analyses are conducted to determine a factor of safety against various modes of global slope stability failure. Barrier layers may be particularly susceptible to global stability failure as they create a planar surface along which the shear strength may be less than that of the material on either side.

Perhaps the best-known global landfill failure is that of the Kettleman Hills hazardous waste landfill in 1988, described in Box 5.7. Kettleman Hills was the first double-lined landfill constructed in the western United States, and procedures for landfill stability analyses were not yet well developed. Back analyses of the Kettleman Hills case history (Seed et al., 1990) indicated that a critical factor contributing to the failure was the low shear strength of the interface between the geomembrane base liner and the underlying compacted clay soil. Ironically, the composite geomembrane low-permeability soil barrier system developed to minimize advective transport of contaminants from the landfill provided a plane of weakness along which the failure surface developed. Subsequent to the Kettleman Hills failure, evaluation of global stability along barrier system interfaces using the results of laboratory interface shear testing became standard practice for the design of geosynthetic barrier systems. However, global stability failures along geosynthetic interfaces continued to occur in both cover and liner systems because of inadequate attention to slope stability in design, construction, and operation.

An important set of field performance data for interface stability was obtained from 14 full-scale test plots with GCLs in their covers in Cincinnati, Ohio, in 1994 (Daniel et al., 1998). Test plots were constructed using a variety of GCL cover configurations at inclinations of 3H:1V and 2H:1V. Based on limit equilibrium analyses using the laboratory interface shear test data, all of the 3H:1V test plots were expected to be stable, but some of the 2H:1V test plots were expected to be unstable (i.e., a factor of safety of less than 1.0). Field performance of the test plots was in substantial agreement with the stability analyses based on the laboratory interface shear data. Cover configurations with a factor of safety of less than 1.0 failed, while all but one that was predicted to be stable remained stable. The one exception

BOX 5.7 Case History of Stability Failure at Kettleman Hills Landfill

This case history illustrates the importance of considering stability in designing engineered barriers that perform effectively. Landfill B-19, Phase 1A, was a hazardous waste landfill located near Kettleman City, California, operated by Chemical Waste Management, Inc. Construction of the landfill was completed in February 1987, and approximately 450,000 m^3 of waste had been placed into the cell at the time of failure in March 1988. The aerial photograph in Figure 5.11 shows the landfill just prior to the failure. The waste was placed to a height of about 28 m on a relatively flat base surrounded by excavated side slopes. The barrier consisted of a double-liner system on the side slopes and base; the primary liner on the base was a composite liner, and the secondary liner on the base was underlain by a tertiary liner system (i.e., a vadose zone monitoring system). On March 15, 1988, the entire mass of waste slid horizontally about 10 m away from the side slopes.

The slope failure caused extensive damage to the liner system. Figure 5.12 shows one of the side slopes after the failed waste mass was removed from the landfill. The waste mass slid along interfaces in the barrier system and ripped through the primary collection system, the primary liner, and the secondary collection (or leak detection) system. It cost tens of millions of dollars to repair the damage. However, there was no release of waste from the barrier system due to substantial redundancy; both the secondary and tertiary liners on the base remained intact and contained the waste even after the failure.

Failure occurred primarily because the possibility of a stability failure during the waste placement period was not taken into account: The shear strengths for interfaces between materials in the barrier system were not measured, and the stability calculations were simplistic. Analyses of the failure showed that several of the interfaces had very low shear strengths and exhibited strain-softening behavior, meaning that once equilibrium was lost there was the potential for large movement before the waste mass regained stability (Mitchell et al., 1990; Seed et al., 1990; Byrne et al., 1992; Stark and Poeppel, 1994; Gilbert et al., 1998).

FIGURE 5.11 Kettleman Hills Landfill B-19, Phase 1A, showing waste placed just prior to the stability failure. SOURCE: Golder Associates.

FIGURE 5.12 Kettleman Hills Landfill B-19, Phase 1A, showing tears in the liner system on the side slope caused by the failure. SOURCE: Golder Associates.

was a test plot in which stability was predicted from the assumption that the bentonite in the GCL would not hydrate because it was encapsulated by geomembranes. Postfailure investigation indicated that the bentonite in the GCL hydrated sufficiently to lower the factor of safety to less than 1, possibly because of construction defects in the test section. A subsequent test section constructed at the same location to remove this problem remained stable throughout the test period.

Based on studies following the Kettleman Hills failure, the Cincinnati GCL test plots, and similar investigations, procedures for evaluating global interface stability are now well established in engineering practice. With proper study and analysis, it should be possible to avoid short-term global interface stability failure for most waste containment systems. All of the stability problems identified in Bonaparte et al. (2002) were attributable to design or construction errors. Significant uncertainties, however, remain about the long-term performance of containment system elements (e.g., the long-term durability of reinforced GCLs, the long-term performance of leachate collection and removal layers and cover system drainage layers), as well as about certain aspects of waste shear strength (e.g., the shear strength of saturated waste in a bioreactor landfill, the shear strength of waste held in containers in a hazardous waste landfill).

Local instability considerations may be important when large deformations adjacent to or across landfill barrier systems are anticipated (e.g., in side slope liner systems of MSW landfills). Local instability during vertical barrier construction is generally associated with local collapse of small underground sections of the excavated trench prior to backfilling, leading to "windows" of native material within the vertical barrier. Figure 5.13 illustrates how waste settlement can lead to loss of the integrity of geosynthetic liner system elements on shallower slopes and cracking and bulging of the low-permeability soil liner on steeper slopes, even when the global stability is adequate. Relatively complex numerical stress deformation analyses are generally required to assess local stability. Jones and Dixon (2005) used numerical modeling to investigate the local stability of a side slope liner system in response to waste settlement. However, while sophisticated analyses may be required to predict these effects, simple engineering measures can often be used to mitigate local instability risk (e.g., use of a "slip sheet" above a side slope liner to mitigate downdrag on the liner due to waste settlement). Although local stability assessments are not generally employed in engineering practice and monitoring systems are generally not designed to identify local stability failures, analyses suggest that local instability may affect the integrity of subsurface barrier systems in steep-sided land-

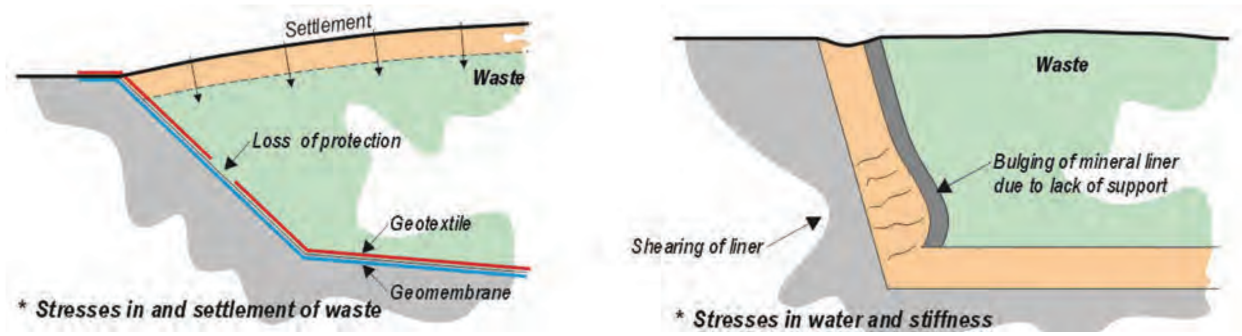


FIGURE 5.13 Mechanisms of local side slope integrity failure for a shallow slope (left) and a steep slope (right). SOURCE: Jones and Dixon (2005). Reprinted with permission from Elsevier.

fills and other facilities where large deformations adjacent to or across the barrier system are anticipated.

Impact of GCL Strength on Stability

The internal shear strength of GCLs is of particular concern for local and global stability of landfills because of the potential for low strengths upon hydration of the bentonite. Reinforcement was introduced into GCLs as a means of mitigating this problem. Most modern reinforced GCLs used in waste containment employ needle-punched reinforcement. Reinforced GCLs have high peak strengths but relatively low postpeak shear strengths at large displacements (75 mm or larger). The postpeak in-plane strength of hydrated GCLs is likely to be the lowest in-plane strength in a barrier system that employs a GCL (Bouazza et al., 2002). Even when the GCL is reinforced, the postpeak shear strength of a hydrated GCL in a landfill liner system may be represented by a friction angle on the order of 8 to 10 degrees. However, strengths this low are likely to be associated with higher overburden pressures (i.e., pressures representative of base liner conditions), where the interface plane of weakness often makes a transition from the geomembrane-GCL interface to an internal GCL failure plane (Sharma et al., 1997).

As discussed by Gilbert (2001), interface failures generally occur along the interface with the lowest peak strength in the barrier system. Therefore, if the peak strength of the GCL is higher that the peak strength of another interface in the liner system, the postpeak strength of that other interface will govern stability. This suggests that the internal shear strength of a reinforced GCL will only affect stability in barrier systems where the peak strength of the GCL is the lowest peak strength in the barrier system (e.g., in liner systems at relatively high overburden pressures). This is consistent with the results of the Cincinnati GCL test sections (Daniel et al., 1998), wherein hydrated reinforced GCLs with 0.9 m of soil overburden slopes remained stable on 3H:1V and failed on 2H:1V slopes along geotextile-geomembrane interfaces rather than internally.

A new generation of reinforced GCLs with thermal-locked fibers has shown significantly higher postpeak shear strengths in laboratory tests than previous GCLs, with failure occurring at the geotextile-geomembrane interface over a wide range of overburden pressure, even when the GCL is hydrated under low overburden pressures prior to testing (Kavazanjian et al., 2006b). The peak strengths developed in these GCLs are high enough to suggest that in many cases a reinforced GCL may not impact stability even after hydration.

It is common practice to assume that a GCL deployed in the field will eventually hydrate and therefore to base the interface stability assessment of a barrier system that employs a GCL on the hydrated strength of the GCL. It has been suggested that GCLs can be encapsulated with two geomembranes to inhibit hydration and thus enhance the in-plane shear strength of a barrier system in which stability is governed by the hydrated GCL shear strength (Giroud et al., 2004). Results of the Cincinnati GCL test section indicate that, with proper attention to design details, encapsulation can inhibit hydration and enhance the shear strength of the GCL, at least in the short term. Although calculations suggest that encapsulation can inhibit hydration for hundreds to thousands of years (Giroud et al., 2004), encapsulated GCLs in liner and cover systems in a number of landfills (Kavazanjian et al., 2006b) have not been used long enough to confirm the calculations.

Strength Degradation

The long-term stability of GCLs may be affected by degradation of the reinforcement. The physical and chemical degradation processes for polypropylene and polyethylene fibers that are used in needle-punched and stitch-bonded reinforced GCLs were studied by Hsuan and Koerner (2002). They suggested possible performance and index test methods for monitoring the polymeric degradation and concluded that, when GCLs are subjected to long-term shear stresses, fiber durability is important, particularly for sloping surfaces and canyon-type landfill liners. Factors involved in fiber durability are stress level, environmental conditions (e.g., oxygen level), required lifetime, and polymer formulation. The key to the polymer formulation is the manufacturing process for the fibers and the type and amount of antioxidants.

Since the strength of GCLs is a function of the strength of the reinforcing fibers, an understanding of the long-term behavior of the GCL reinforcement is important. Thomas (2002) tested the long-term oxidative stability of a polypropylene textile made from fibers used to reinforce a commercial needle-punched GCL and estimated that the reinforcing materials would retain 50 percent of their strength for 30 years when exposed to air at 20°C. However, when accounting for the effects of oxygen limitation, a service lifetime approaching 100 years for the reinforcing fibers has been estimated for buried applications (Salman et al., 1998). Lower service lifetimes can be expected under elevated temperature conditions, such as those that occur at MSW landfills.

Degradation of polymer and slow disentanglement of fibers can differentially compromise the strength of a reinforced GCL (Thies et al., 2002). Some needle-punched GCLs, in which the needle-punched fibers were thermally bonded to the carrier geotextile in an attempt to enhance the reinforcing fiber anchorage, actually failed sooner in elevated temperature aging tests. The enhanced anchorage resulted in failure accompanied by breaking of the reinforcing fibers from their anchoring, rather than simple disentanglement. In addition, GCLs made with polypropylene took 20 to 60 times as long to fail as those made from polyethylene. This result corroborated data from short-term peel tests that measure the strength required to peel the geotextile off of the bentonite core (Müller et al., 2004), demonstrating the need for long-term shear tests to properly assess the lifetime of GCLs.

In summary, field observations indicate that global stability can be a significant threat to the short-term integrity of liner and cover systems and that cover stability can be a concern in the medium term. Clogging or undercapacity of drainage systems on the side slopes of covers appears to be the most significant factor affecting the medium-term global stability of cover systems. No data exist on the long-term stability of modern liner and cover systems because few of these systems have been in place for more than 30 years. However, the long-term stability of liner and cover systems that rely on the strength of a geosynthetic element may become an issue as the polymers degrade with age. The long-term stability of side slope cover systems in seismically active regions may also be a significant concern, as unconditional stability of side slopes steeper than 5H:1V may not be attainable in areas of even moderate seismicity because of the high initial static shear stress acting on the slope and the potential for amplification of seismic motions at the landfill cover. The initial static shear stress on a 5H:1V slope can lead to yield accelerations as low as 0.1 g for typical geosynthetic interface friction angles in the absence of cohesion. Furthermore, amplification can result in peak accelerations in landfill cover as high as 0.3 g for earthquakes with maximum horizontal accelerations (free-field peak ground accelerations) as low as 0.1 g. Whenever the peak acceleration at the landfill cover exceeds the yield acceleration, the cover is not unconditionally stable and a seismic deformation analysis is required. Local stability of geosynthetic liner systems, particularly in steep-sided landfills, is an issue that is not often considered in landfill design. However, the high compressibility of municipal solid waste may impose significant loads on side slope liner systems, which can create local stability problems that impact the integrity of the containment system in the short and medium terms.

5.2.7 Modeling Concrete Barrier Performance

A variety of models have been developed to predict different aspects of concrete barrier performance. Models of concrete barrier performance must consider complex interactions among the physical properties of the barrier (e.g., permeability, porosity, crack structure) and advection and diffusion of contaminants (e.g., radioactive species, volatile organic compounds), chemical species that affect the barrier properties (e.g., sulfates, chloride), and transport media (solid, liquid, and gas phases). A one-dimensional model based on micromechanics theory and the diffusion-reaction equation can predict the expansion of mortar bars (Krajcinovic et al., 1992). Models have also been developed to address nonlinear diffusion-reaction conditions that lead to cracking, changes in diffusivity, and degradation of concrete subjected to external sulfate sources (Tixier and Mobasher, 2003a, 2003b). In the STADIUM model, chemical and physical phenomena are described by a general equation expressing the variation in concentration of ionic species through a permeable material (Marchand et al., 1999a, 1999b). Dissolution of portlandite and decalcification of calcium silicate hydrate (the "glue" that binds aggregate together in Portland cement concrete) are among the effects predicted by the model. Atkinson and Hearne (1989) developed a model in which the rate of spalling of a concrete barrier is expressed as a function of the elastic and fracture properties of concrete, its intrinsic sulfate diffusion coefficient, the external sulfate concentration, and the concentration of ettringite (an expansive mineral).

4SIGHT is used to model the degradation of buried concrete vaults due to advective and diffusive transport of sulfate and chloride ions (Snyder and Clifton, 1995; Snyder et al., 1995). The model predicts both the bulk hydraulic conductivity of the concrete and the structural integrity of the concrete and reinforcement. The model includes precipitation and dissolution of salts caused by changes in pore fluid pH as well as their impact on the porosity of the concrete. Input to the model includes the initial crack density and crack geometry in the concrete; the spacing of joints in the concrete and the permeability of any joint-filling compound; the properties of the concrete; and external ion concentrations, including sodium, chloride, potassium, calcium, chloride, and sulfate. The key properties controlling the diffusion of chloride and sulfate ions through the concrete include the porosity of the concrete and the formation factor (the ratio of the electrical conductivity of the pore fluid to the bulk electrical conductivity of the concrete). The pore fluid conductivity is a function of the various ionic species present and their relative concentrations, which can be established from chemical equilibrium considerations.

The 4SIGHT model can be modified to make probabilistic predictions of concrete service life using Monte Carlo simulations (Snyder, 2001). The laboratory data used by Snyder to validate the probabilistic model had a maximum duration of less than 100 days, and field data that were sufficiently quantitative spanned only one or two decades. One advantage of the probabilistic approach is that it allows uncertainty about the properties of the concrete to be incorporated into the model. Therefore, while models exist for the complex interactions that govern concrete barrier performance, considering the relatively short duration for which field data are available, the ability to predict the degradation of PCC barriers for periods longer than several decades remains largely unproven.

5.3 PREDICTING THE OVERALL PERFORMANCE OF CONTAINMENT SYSTEMS

Given the absence of observations and performance data over the long term, models are used to predict the long-term performance of waste containment systems. However, few field data exist to calibrate or validate these models. Moreover, it is difficult to model these systems when so many of the parameters have a wide range of possible values. Because waste containment systems often need to be effective for timescales that stretch across decades or even centuries, long-term performance predictions must be an essential part of containment design. Unfortunately, long-term performance models often rely on extrapolations of data using time-temperature superposition and other assumptions that introduce significant uncertainty into the models. Because of these uncertainties, it will be necessary to continue to monitor the performance of critical systems and to compare observations to long-term predictions for the foreseeable future.

Prediction of the overall performance of containment systems requires a combination of predictive elements. Cover performance analyses are required to predict the generation of leachate and landfill gas (although both of these can also be predicted empirically) and the migration of gas through the cover and laterally (if contained by the cover). Liner performance analyses are required to predict the advective transport of leachate and gas and diffusive transport of chemical compounds through the liner. Advective-diffusive-dispersive transport models are required to predict contaminant transport across vertical barriers. Models for the active life of the waste and for changes in the properties of the containment system elements with time also are required. Once all of these models are integrated, it should be possible to predict the rate of transport of constituents of concern across the boundaries of the containment system. However, even after the rate of transport is established, additional analyses will be required to predict the advective and diffusive transport of these constituents to the “point of compliance” (i.e., the point at which containment system performance is evaluated) and beyond to a point where the constituents of concern may have an impact on a sensitive receptor.

5.3.1 One-Dimensional Contaminant Transport Models

A limited number of closed-form solutions are available for one-dimensional advective-diffusive-dispersive flow. The closed-form one-dimensional solutions are generally available for situations with well-posed boundary conditions, that is, for constituent concentrations and fluxes or advective potentials at the boundaries of the domain that are known and either constant or that conform to a well-behaved mathematical function. Common boundary conditions for one-dimensional contaminant transport modeling are discussed in Rabideau (1995) and Khandelwal et al. (1997). Rabideau also presents graphical solutions for some well-posed cases where diffusion and advection dominate. However, even if a solution to the advective-dispersive-reactive equation (ADRE) is available, estimation of the governing parameters for that equation is no simple task. In particular, evaluation of the effective diffusion coefficient, D^*, which includes the effect of both diffusion and dispersion, and the retardation factor, Rd, which governs sorption, may require batch equilibrium or column testing in the laboratory. Some guidance on diffusion coefficients and sorption is available in the literature (e.g., Rowe et al., 2004). The accuracy of ADRE modeling is further complicated by unsaturated flow, coupled processes, nonlinear and/or rate-dependent sorption, cation and anion exchange, matrix diffusion, temperature effects, and the other chemical and biological processes mentioned previously. In many but not all cases these factors lead to attenuation of the contaminants, and ignoring them is conservative with respect to the rate of contaminant migration. However, some of the processes can lead to the generation of harmful daughter products, and desorption and dissolution can also have adverse effects.

Migration from a landfill liner or cover system or through a barrier wall can often be modeled as one-dimensional transport. In such cases the closed-form solutions provide a basis for assessing barrier performance. Furthermore, one-dimensional solutions can often be used effectively in quasi-two- and three-dimensional models, as described below.

5.3.2 Multidimensional Contaminant Transport Models

Numerical solutions to the advective-dispersive-reactive equation are generally required for multidimensional flow.

Multidimensional ADRE modeling adds additional complexity to an already complex problem. Lateral dispersion creates additional uncertainty of the effective diffusion and cannot be modeled well in laboratory column experiments. In general, numerically complex finite element or finite difference models are required to solve the ADRE in more than one dimension. However, several simplified models for multidimensional contaminant transport are available. POLLUTE (Rowe and Booker, 1986, 2005; Rowe et al., 2004) is a widely used computer program that models finite layer contaminant migration for landfill design employing a "one and one-half dimensional solution to the advection-dispersion equation." POLLUTE also considers radioactive and biological decay, phase changes (allowing modeling of diffusion through geomembranes and in unsaturated leak detection systems), and transport through fractures. Applications of POLLUTE to assess contaminant transport from a landfill are described by Lo (1992), Rowe (1998), Simms et al. (2001), and Lake and Rowe (2005b). It has been approved in many jurisdictions for comparing alternative barrier systems.

The Multimedia Exposure Assessment Model (MULTIMED) is an EPA-developed hybrid model for simulating the movement of contaminants leaching from a waste disposal facility (Salhorta et al., 1995; Sharp-Hansen et al., 1995). The program consists of modules for contaminant transport through the subsurface, on the surface, and in water and air. A semianalytical one-dimensional transport model is used to transmit contaminants from the landfill vertically through the vadose zone (Figure 5.14) using either (1) an analytical model that considers the effects of longitudinal dispersion, linear adsorption, and first-order decay or (2) a numerical model that includes longitudinal dispersion, nonlinear adsorption, first-order decay, time-variable infiltration, and arbitrary initial chemical concentrations in the vadose zone. The vadose zone transport model is coupled with a semianalytical saturated zone transport model that considers one-dimensional uniform flow, three-dimensional dispersion, linear adsorption, first-order decay, and dilution. MULTIMED can consider parameter uncertainty, both steady state and transient flow, and up to 11 different chemical species simultaneously.

MULTIMED can be coupled with the HELP model to evaluate contaminant transport from geomembrane-lined landfills, although it does not correctly model the diffusion of organic contaminants through geomembranes. The EPA has approved the use of MULTIMED for demonstrating that alternatives to the prescriptive liner system meet RCRA Subtitle D MSW landfill performance requirements. However, EPA recommendations for MSW landfill applications (Sharp-Hansen et al., 1995) impose several conservative restrictions on the MULTIMED analyses, including (1) no decay of the contaminant source, (2) the contaminant concentration is calculated at the top of the aquifer, (3) only steady state transport, (4) the concentration of the contaminants entering the aquifer system is constant with time, (5) the contaminant pulse is continuous and constant for the duration of the simulation, (6) the point of compliance is located directly down gradient of the facility and intercepts the center of the contaminant plume, and (7) a Gaussian source geometry is assumed for the contaminant plume. Two case histories of the application of MULTIMED to demonstrate compliance of an alternative liner system with Subtitle D MSW landfill performance standards are described in Dobrowolski and Kavazanjian (2003).

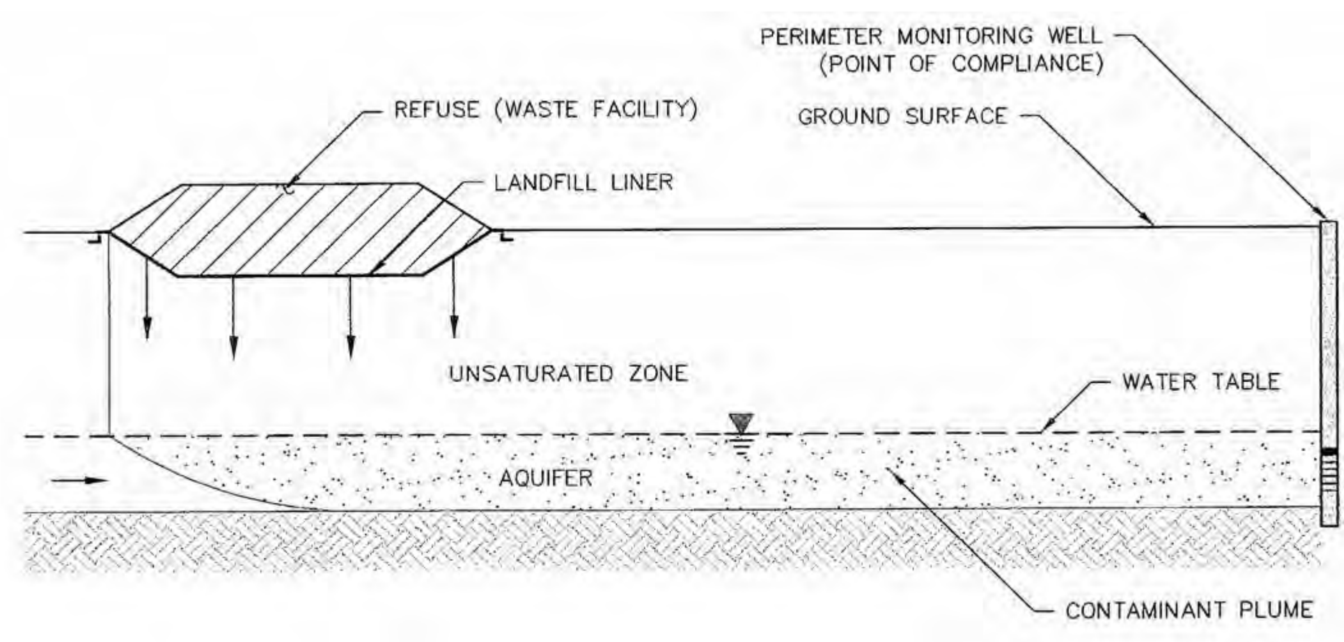


FIGURE 5.14 MULTIMED transport modeling. SOURCE: Dobrowolski and Kavazanjian (2003).

5.4 CONCLUSIONS

Observations of the performance of liner and cover systems suggest that when properly designed and constructed these systems do a good job of limiting the migration of harmful contaminants over the 10 to 20 years for which data are available. However, the data are limited and interpretations rely more on the absence of observed adverse impacts on the environment than on direct observations of barrier system performance. In the case of vertical waste containment barriers, little information is available to evaluate their performance or to predict their integrity and effectiveness over the long term. The adequacy of systems for monitoring environmental impacts of each of these barrier systems may be questionable, particularly in the long run. Synthesized, publicly available data on the performance of landfills are sparse. Notable exceptions include the reports of Bonaparte et al. (2002) and Rowe (2005) and the letter to the committee from the New York Department of Environmental Conservation, which contain key information from up to 20 years of landfill monitoring. Models are available to predict the long-term performance of containment systems, but they rely heavily on predictions of the long-term integrity of containment system elements. Thus, if it is accepted that containment systems are performing satisfactorily in the short and medium terms, maintaining the integrity of containment system elements over the long term (i.e., for the active life of the wastes they contain) appears to be the most significant requirement to assure satisfactory long-term performance of engineered barrier systems.

The key findings regarding the performance of engineered barrier systems can be summarized as follows:

- Liner systems: Liners appear to be working reasonably well over periods of up to 20 years. A composite liner limits leakage significantly better than a single geomembrane. A composite liner with a GCL has a lower leakage rate than one with compacted clay, but more care must be taken during construction. An additional attenuation layer may be required to control diffusive transport in liner systems involving a GCL or thin compacted clay liner.

The lifetime of a primary liner is related to the temperature on the liner, with higher temperatures causing greater likelihood of desiccation cracking and degradation of geosynthetics. The addition of a secondary liner provides a substantial increase in the ability of the barrier system to contain contaminants. The potential for diffusive flux, including the long-term flux of volatile organic compounds, should be considered when designing a facility.

- Cover systems: Although cover systems that employ a single clay or GCL barrier layer have been known to crack, desiccate, or otherwise degrade, liquid percolation rates suggest that cover systems that employ geomembrane barriers are generally performing well. The only somewhat persistent problems with covers employing geomembrane elements are side slope instability, erosion, and gullying, often caused by clogging or insufficient capacity of the cover drainage layer. Although evapotranspirative covers perform well in the short term in arid and semiarid climates, their long-term performance and their performance in temperate climates have not been demonstrated. Finally, most cover systems rely on continuing maintenance for good operation. Maintenance-free covers have not been demonstrated to be effective.
- Vertical barriers: Little information is available to evaluate the performance of vertical barriers or to predict their integrity and effectiveness over the long term. More monitoring is required to determine whether these systems are performing adequately.
- Barrier integrity: Local slope stability and global slope stability are significant short-term concerns but can be mitigated with proper attention during design and operations. More work has to be done on local stability, especially on steep slopes.

Principal findings concerning the prediction of barrier system performance are as follows:

- Predicting the performance of covers: The HELP model does a good job predicting the volume of leachate, but it is not reliable for other uses.
- Predicting the performance of liner systems: Accurate predictions of leaks through composite liners need to take into account holes in wrinkles and elevated leachate head.
- Predicting gas transport through containment systems: A well-designed and well-constructed cover will not necessarily ensure that gas will not escape; the details of the leachate collection/removal system are important to make sure that gas does not bypass the containment system.
- Predicting the performance of vertical barriers: While it is feasible to predict contaminant transport through vertical barrier walls around contaminated sites, the paucity of field data limits our ability to evaluate the accuracy of the predictions.
- Predicting the performance of concrete barriers: The ability of computer models to predict long-term performance is largely unknown. Material coefficients that control transport and reaction in concrete must also be better characterized in models.
- Predicting the "active" lifetime (contaminating life span) of waste: Landfills, especially some of the larger ones, are likely to require attention for centuries, not decades.
- Predicting the overall performance of containment systems: Existing data suggest that modern containment systems are performing well and that predictive models are capable of predicting their performance. However, this positive finding is tempered by two facts: (1) there are relatively few field data that can be used to verify models, and (2) modern landfills have not been in existence long enough to allow an empirical assessment of long-term performance.

6

Summary and Recommendations

"Modern" engineered waste containment systems have been in existence for only a few decades. Thus, the committee's assessment of these systems is necessarily limited to their short- and medium-term performance. The principal findings and overarching recommendations for actions and studies needed to both reduce the uncertainties surrounding the evaluation of barrier performance and better ensure that contained wastes do not provide a risk to health, safety, or the environment in the future are given below.

6.1 ENGINEERED BARRIER PERFORMANCE

As much as 20 years of field observations suggest that engineered waste containment barrier systems that have been designed, constructed, operated, and maintained in accordance with current statutory regulations and requirements have so far provided environmental protection at or above specified levels. Extrapolations of long-term performance can be made from existing data using both empirical and physical and chemical models, but they will have high uncertainties until field data are accumulated for longer periods, perhaps 100 years or more for some systems.

Our ability to predict the long-term performance of engineered waste containment systems depends strongly on accurate prediction of the service life of the individual components (i.e., compacted clay layers [CCLs], geomembranes, geosynthetic clay liners [GCLs], vertical walls of various types). Based on the available data, the committee draws the following conclusions regarding the performance of barrier systems:

- Modern composite liners generally appear to be working well, up to the assessed period of about 20 years, with double composite liners constructed according to rigorous construction quality assurance guidelines providing the best protection against advective contaminant migration.
- Cover systems can be effective at isolating waste and limiting infiltration. Most cover systems require periodic maintenance to maintain their integrity.
- Direct monitoring of vertical barriers (e.g., soil-bentonite and cement-bentonite walls) has been insufficient to draw conclusions about their field performance. Monitoring of contaminants down gradient of the barrier (indirect monitoring) suggests that most vertical barriers are functioning as intended. However, more extensive monitoring and detailed analyses are needed before definite conclusions can be reached.

Individual components of barrier systems can degrade as a result of chemical interactions, environmental effects (freeze/thaw and desiccation of CCLs and GCLs), elevated temperature (geomembranes), deformation-induced cracking (covers and vertical barriers), and clogging caused by biological action or soil intrusion (drainage layers). However, even if the performance of an individual component degrades with time, redundant design appears to enable the overall waste containment system to still serve as an effective barrier to contaminant transport. Incorporation of specific provisions for repair or for recovery and replacement would further strengthen designs and likely enhance the performance of new waste containment systems.

6.2 DATA COLLECTION AND DISTRIBUTION

Although engineered waste containment barrier systems have been working well in the short term, it is not known how long they will continue to work. A number of observations about performance (e.g., liners can get hot, leachate collection systems and drains can clog, long-term exposure of geosynthetics to high temperatures and chemicals can degrade their properties, incompatibility between CCL, GCL, and leachate may require many years to develop) suggest that considerable care in design is needed to avoid problems in the 20- to 100-year time frame. Consequently, it is important to continue monitoring performance well beyond the few decades for which performance data are now available.

Much information on barrier components and systems is collected in accordance with regulations. This data collection is necessarily targeted toward regulatory compliance, which tends to focus on chemical concentrations in gas and groundwater at defined points of compliance. The overall result of the focus on compliance is that key data on barrier performance are either not collected or are not collected long enough to enable reliable predictions of performance. A systematic approach to data collection and reporting that targets the most important data on barrier performance and makes the data readily accessible would greatly facilitate periodic assessments of the long-term performance of engineered barriers.

Recommendation 1: Monitoring programs for new facilities should include provisions for collecting data needed to assess the long-term performance of engineered barriers, and operators of existing facilities should collect these data to the extent practical using in-place monitoring systems.

Key types of data that should be collected are listed in Table 6.1.

Noninvasive geophysical monitoring techniques (e.g., electrical surveys, radar, seismic tomography) have the potential to reduce the number of monitoring and observation wells needed and thus reduce costs. Geophysical techniques may also enable continuous, rather than episodic, assessments of barrier integrity. Additional evaluation is needed to determine the extent to which these methods are capable of providing the information listed in Table 6.1.

Most field monitoring of waste containment systems is performed either immediately at the end of construction and before the placement of waste or indirectly afterward through measurements such as cover settlement or concentrations of chemical constituents in gas and groundwater. Although this practice satisfies regulatory requirements, the lack of direct monitoring data introduces uncertainties about how well the individual parts of the overall containment system are working. Such information could help operators avoid an unacceptable release of contaminants and is also essential to designing better systems and materials for future waste containment systems. New techniques are needed to directly monitor the integrity and performance of other barrier configurations and individual barrier system components.

Recommendation 2: Regulatory agencies should develop guidelines to increase direct monitoring of barrier systems and their components, and NSF should sponsor research for the development of new cost-effective monitoring techniques, especially for assessing the effectiveness of vertical barriers, for this purpose.

Assessing or predicting the performance of engineered barriers is made more difficult because the necessary data and observational information do not exist, are hard to find, are incomplete, or have not been analyzed. Although the law requires operators of waste containment facilities to make data publicly available, reports, databases, and tables are often not readily accessible. The effort required to collect relevant information from disparate sources can discourage the types of broad-scale analyses needed to evaluate performance. However, accumulation of new information on field performance, as well as advances in understanding of material behavior and in monitoring and modeling capabilities, would make an assessment of performance worthwhile about every 5 years.

Recommendation 3: Federal agencies responsible for engineered barrier systems should commission and fund assessments of performance on a regular basis. Given the rate at which performance data and knowledge of waste behavior, contaminant transport, and monitoring accumulate, the interval at which these assessments should take place is probably on the order of once every 5 to 10 years. The results of the assessment should be placed in the public domain in a form that is readily accessible.

Many data used to predict performance come from laboratory experiments, models, and field-constructed prototype barriers, such as test pads. Although useful for understanding material properties and behavior, these data are no substitute for performance data collected in the field from operating

TABLE 6.1 Recommended Data and Information Collection for Long-Term Assessment of Engineered Barrier Performance

Parameter[a]	Measurement Technique	Purpose	Frequency	Location
Existing but should be more accessible				
Leachate flow rate	Lysimeters, LCRS, and extraction trench flow rate measurements	Cover, LCRS, and extraction system effectiveness; demand on liner or barrier	Collect continuously, report monthly averages and peak flows annually	At collection and discharge points
Leachate composition	Chemical analyses	Demand on liner or barrier	Collect indicators semiannually	At collection points
Leak detection system flow rate	Fluid levels, piezometers	Effectiveness of primary liner	Collect monthly peaks and averages	LDS collection or discharge points

TABLE 6.1 continued

Parameter[a]	Measurement Technique	Purpose	Frequency	Location
Composition of LDS liquid	Chemical analyses	Source of leakage	Collect indicators semiannually	LDS sumps
Geomembrane defect frequency	Electrical leak detection	Short-term integrity of geomembrane	Once at the end of construction after emplacement of the LCRS	Geomembrane covers and liners
Leachate head in sumps	Observation wells, piezometers	Head on sump liner	Monthly average	At the sumps
Hydraulic head and concentration differences across vertical barriers	Piezometers, groundwater wells	Hydraulic gradient, concentration gradient, mass flux across barrier	Semiannually	Opposite sides of the barrier
Physical condition of cap (cracking, settlement, erosion, stability)	Visual observations, surveys, photographs, LIDAR surveys	Vegetative health, erosion, demand on barrier layers	Quarterly and after extreme events	Full site
Physical condition of vertical barrier (at the surface)	Visual observations	Cracks and settlement along the wall alignment	Quarterly	Along the entire alignment
Gas emissions through cap	Handheld probes, flux box	Effectiveness of cap at gas containment	Monthly	Specified distributed measurement points
Subsurface gas migration	Perimeter gas probe measurements	Effectiveness of containment system for gas migration control	Monthly	Multidepth probes at specified maximum spacings around the perimeter
Groundwater monitoring sample compositions	Chemical analyses	Containment system effectiveness—assurance that maximum contaminant levels are not exceeded	Semiannually	Groundwater monitoring points located based on hydrogeology of the site
Proposed				
Liner temperature	Temperature sensors	Thermal environment on liner (for degradation prediction)	Daily (average and/or maximum and minimum), reported monthly	Multiple points from the edge to the center of a cell at selected MSW and ash landfills
Head on liner	Leachate-level measurement in wells, piezometers	Demand on liner, effectiveness of LCRS	Monthly peak and average values	Representative points at selected MSW, ash, and hazardous waste landfills
Leakage (lysimeter) beneath sump	Leachate collection in lysimeter	Fluid flow and mass flux through area over lysimeter	Monthly totals	All single-lined landfills
Defects in vertical barriers	Geophysical techniques (e.g., electrical imaging of gas tracers or injected brine); fluid head and chemical concentrations on opposite sides of the barrier	Barrier integrity	End of construction and periodically thereafter	All vertical barriers
Change in hydraulic conductivity of vertical barriers	Coring and sampling; in situ testing	Barrier effectiveness	Once every 5 years	Soil-bentonite and cement-bentonite barriers
Geomembrane oxidation induction time	Testing of sacrificial coupons[b]	Geomembrane aging	Every 5 years	Coupons placed in sumps and in the cover
GCL hydraulic conductivity	Testing of sacrificial coupons	GCL degradation	Every 3 years	Panels buried in landfills with single GCL covers

NOTES: LCRS = leachate collection and removal system; LDS = leak detection system; LIDAR = LIght Detection and Ranging; MSW = municipal solid waste.
[a]Accompanying metadata are essential (e.g., rainfall, temperature, location of facility, site activity reports).
[b]Sacrificial coupons are loose pieces of geomembrane that can be retrieved periodically from the sump for testing.

containment systems. A comprehensive assessment of performance requires long-term monitoring and analysis of data from different types of waste containment systems, constructed from a variety of components, and located in different climate regimes. Some of this information could be gathered from existing facilities where sufficient funding is available to expand monitoring or from new facilities where collecting and reporting the types of information listed in Table 6.1 are built into operational plans. But even taking advantage of existing and planned facilities misses opportunities to test innovative concepts and new materials or to control instrument spacing and monitoring periods.

Recommendation 4: EPA, USNRC, NSF, and DOE should establish a set of observatories at operational containment facilities to assess the long-term performance of waste containment systems at field scale. The program would involve building one or more field facilities, monitoring the site, and analyzing and archiving the data. New sites could be created or adjustments could be made to existing observatories when promising new and innovative concepts and materials become available.

6.3 MODELS

Because published high-quality field data are sparse, facility operators commonly rely on analytical and numerical models to predict contaminant transport, containment effectiveness, degradation of materials, and changes in behavior over time, even though some models have well-known shortcomings. For example, some do not account for known processes (e.g., advection-dispersion processes, leakage caused by holes in geomembrane wrinkles), and others (e.g., the HELP model) are widely used in applications for which they were not designed.

Recommendation 5: Regulatory agencies (e.g., EPA, DOE, USNRC) and research sponsors (e.g., NSF) should support the validation, calibration, and improvement of models to predict the behavior of containment system components and the composite system over long periods of time. These models should be validated and calibrated using the results of field observations and measurements.

6.4 MONITORING PERIODS

Almost all statutory monitoring programs require an initial 30-year postclosure monitoring period. At the discretion of regulatory authorities, the owners and operators of some sites may have to continue monitoring and maintenance if the waste still poses a threat to human health or the environment. However, financial assurance is frequently required only for the initial postclosure monitoring period. The committee's analysis of data from engineered barrier systems that contain low-level radioactive waste, hazardous waste, and municipal solid waste suggests that extended monitoring periods (hundreds to thousands of years) will be required in many cases. The necessary duration of monitoring varies with the facility, type of waste, climate, and observed performance. Yet without an appropriate financial assurance mechanism, funding will often not be available to continue monitoring until the site no longer poses risk to human health and the environment. Legislation has been introduced into the Senate (S. 452) that would direct the EPA to develop financial assurance regulations to ensure that liable parties meet their Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) obligations. Whether or not it passes, a national policy that ensures the availability of funding would alleviate concerns that different state financial assurance requirements might create an incentive for shipping waste across state lines.

Recommendation 6: EPA should develop financial assurance mechanisms to ensure that funding is available for monitoring and care for as long as the waste poses a threat to human health and the environment.

6.5 PERFORMANCE CRITERIA

Common performance criteria used in practice include measures such as acceptable percolation rates through covers and contaminant mass fluxes through low-permeability barriers. Performance-based design is now the norm in many other countries. These performance criteria are generally based on the performance of the prescriptive designs specified in the regulations for many waste containment barrier systems or system components (e.g., prescriptive cover and liner designs described in Chapter 2, prescriptive low-permeability barrier layers described in Chapter 4). However, performance criteria that are based on a prescriptive barrier system design are often narrowly focused on the performance of these components and may miss key aspects of the overall performance of the waste containment system. As a result, flexibility in the design of waste containment systems to provide cost-effective environmental protection is limited.

Regulations often provide for sophisticated risk-based design of containment systems. For instance, Subtitle D regulations for MSW landfills provide a table of risk-based chemical concentrations at the point of compliance that can be used as a basis for regulatory approval of alternative barriers that do not meet the prescriptive requirements. In theory, a risk-based design could result in more effective and economical systems that balance technical performance, cost, and risk on a project-specific basis. The engineered barrier systems in a risk-based design could be either more or less protective than the prescriptive barrier, depending on the project-specific characteristics of the waste, the geological setting, and the exposure potential for humans and the environment. For example, a design that limits the concentrations in an underlying receptor aquifer provides a measure of the performance of the entire system, but if it fails to take into account the service lives of the system components, the long-term performance of the system could be compromised.

In practice, a risk-based design will rely heavily on validated and calibrated models to minimize uncertainties in predicted performance and is accompanied by field monitoring to confirm performance. Given the current lack of performance data and deficiencies in monitoring technology and validated and calibrated models (Sections 6.2 to 6.4), there is a significant potential for misuse or even abuse of risk-based designs in practice. Thus, until further developed and validated in practice, risk-based designs should be subjected to independent review.

Recommendation 7: EPA and USNRC should develop guidance for the practical implementation of performance-based criteria for assessment of containment system performance as an alternative to prescriptive designs.

In conclusion, effective long-term containment of wastes is difficult and requires high-level engineering, comprehensive design, use of suitable materials, carefully controlled construction, continual monitoring, and maintenance as required. Evidence to date reveals few failures of engineered waste containment barrier systems that have been designed, constructed, operated, and maintained in accordance with statutory regulations. In those few cases where failures have occurred, repair or limited reconstruction has been possible.

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Appendix A

Predicting Human Health and Ecological Impacts

Environmental risk assessments are used to predict the impact that a barrier system might have on human health and the environment. Risk assessments may yield a variety of possible products:

- Incremental lifetime cancer risk for humans. Exposure to contaminants from the site may cause an incremental increase in the frequency of individuals who develop cancer over their lifetimes. The Environmental Protection Agency requires that this incremental increase in frequency be less than 1×10^{-6} to 1×10^{-4} for Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) and Resource Conservation and Recovery Act (RCRA) sites.
- Hazard index or reference dose for humans. A reference dose is a mass of chemical or millirem of radiation per unit of time that represents a threshold at which human health would be affected. For a given component, the ratio of the actual dose measured at the site divided by the reference dose is called the hazard quotient. The sum of hazard quotients for all substances is called the hazard index. A value of less than 1 for the hazard index is considered acceptable.
- Toxicological unit or hazard quotient for ecological systems. The concentration of a chemical measured at the site is compared with the chemical concentration that would cause an effect (like toxicity) in a receptor population (an assessment end point such as fish in a stream). The ratio of these two concentrations is called the hazard quotient or the toxicological unit. The sum of toxicological units for a particular receptor is used as a metric, with values less than 1 considered acceptable.
- Qualitative or lines of evidence for ecological systems. Because of the diversity and complexity of ecological systems, the ultimate product of a risk assessment is generally not a single number. Rather, the product includes conclusions about whether effects are occurring in different classes of assessment end points (e.g., fish, microorganisms, plants, wildlife) and a discussion of the supporting evidence.

The basic methodology for an environmental risk assessment is presented in NRC (1983) and summarized below. The methodology consists of four steps:

1. *Hazard identification*. Identify chemicals of concern in disposed wastes and their potential to affect human or ecological health.
2. *Exposure assessment*. Establish the mass of chemicals of concern at specific locations. The product of the exposure assessment is a dose, such as the mass of chemical inhaled per unit time (referred to as an exposure profile for ecological risk assessments). For an engineered barrier, this step requires the following information:

   a. Release rates of chemicals from the barrier (e.g., release of aqueous-phase chemicals into the groundwater through the leakage of leachate);
   b. Fate and transport of released chemicals along pathways from the source to a receptor location (e.g., the transport of the aqueous-phase chemical to below a house foundation, partitioning of the chemical into the soil vapor phase, migration of the soil vapor into the house through cracks in the foundation); and
   c. Means by which a receptor comes into contact with the chemicals at a receptor location (e.g., duration of exposure and inhalation rate for a receptor in the house).

3. *Dose-response assessment*. Establish the relationship between effects and doses for chemicals of concern. For humans, these relationships are expressed as cancer slope factors for carcinogens (a proportionality constant relating the incremental frequency of cancer incidence for a receptor that is exposed to the dose over their lifetime) or reference doses. For ecological systems the dose response is called a stressor-response profile, and it is expressed in a variety of ways, including as a point threshold value or as a distribution showing the percentage of a population showing effects as a

function of the dose. A significant difference between human health and ecological risk assessments is in the methodology used to establish the dose-response relationship. For humans these relationships are generally extrapolated from laboratory studies for animals. Ecological risk assessments, however, employ epidemiological studies in which tests are performed on samples from the medium and the organisms at the site. This step is generally the most difficult and controversial part of environmental risk assessment.

4. *Risk characterization*. Integrate the exposure assessment and the dose-response assessment to evaluate whether human health and the environment will be affected by the chemicals of concern. Risk characterization can be both quantitative and qualitative.

Typical pathways that would appear in a risk assessment for a barrier system include:

- leachate leakage→groundwater transport→groundwater pumping→water ingestion
- leachate leakage→groundwater transport→groundwater pumping→inhalation (showers and faucets)
- leachate leakage→groundwater transport→partitioning to vapor phase→vapor-phase transport to confined space (structures or excavations)→inhalation
- leachate leakage→surface water transport→direct contact/ingestion (both human and ecological receptors)
- leachate leakage→surface water transport→partitioning to sediments→ingestion by ecological receptors
- leachate leakage→partitioning to soil particles→ingestion
- gas leakage→partitioning to groundwater→all of the above pathways with groundwater transport
- gas leakage→vapor-phase transport to confined space (structure or excavation)→inhalation or explosion
- inadvertent intrusion through barrier→direct contact with waste or even transport of waste (e.g., inadvertently using wastes as fill materials for construction in the surrounding area)

With the exception of inadvertent intrusion, all of these pathways start with a source term expressing the mass flux of release from the barrier system as a function of time. However, inadvertent intrusion can be a significant pathway, particularly when the period for the assessment is so long (e.g., thousands of years) that institutional controls may no longer be effective.

Uncertainty is incorporated into an environmental risk assessment both implicitly and explicitly. Uncertainty is accounted for implicitly in practice by generally selecting conservative values for input variables (e.g., neglecting a depletion of the source term with time or using a maximum or a 95th percentile value instead of an average for the concentration of a chemical of concern). In some cases, uncertainty is accounted for explicitly by performing a probabilistic analysis and expressing a range or even a probability distribution of possible results.

Appendix B

Methods for Monitoring Engineered Barrier Performance

Parameter	How Measured	Use	Comments
1. Phreatic surface (water table)	Observation (monitoring) wells	Establish hydraulic gradient in uppermost aquifer or perched groundwater surface	Monitoring zone depends on screened interval
2. Hydraulic head in groundwater	Vibrating wire, pneumatic, and standpipe (Casagrande) piezometers	Establish hydraulic gradients and groundwater flow velocities	Flow velocities are based on permeability values; requires knowledge of point of measurement to establish elevation head
3. Constituent chemical concentrations in groundwater	Chemical analysis of groundwater samples for organic and key inorganic constituents	Establish background concentrations and concentration gradients and detect releases	Representative background values may be difficult to establish in complex geologies
4. Subsurface distribution of chemical concentrations	Electrical and acoustic surveys	Identify breaches in barriers and preferred groundwater flow paths	Rarely used in practice
5. Surface projection of extent of chemical concentrations	Geophysical surveys (e.g., electrical resistivity, EM, GPR)	Identify and map groundwater plumes of certain contaminants	Rarely used in practice
6. Volumetric moisture content in soil (θ)	Time domain reflectometry	Determine wetting front and determine indirectly unsaturated hydraulic conductivity (k) and soil suction (ψ) via established k versus θ and ψ versus θ relationships	Provides a direct measurement of moisture content, which also can be determined indirectly through measurement of soil suction (see 7) and use of an established soil-water characteristic curve (ψ vs. θ)
7. Soil suction (ψ)	Gypsum blocks, psychrometers, suction lysimeters, tensiometers	Establish soil suction gradients and infer seepage under unsaturated flow conditions	Range of suctions measured varies depending on instrument
8. Percolation through barriers	Pan lysimeters (underdrains)	Establish leakage rates for bottom barriers before and after waste emplacement and for covers	Accuracy of measurement is a function of boundary conditions
9. Gas-phase constituent concentrations and flow rates through cover systems	Gas/air samples analyzed using handheld instruments and/or flux chambers	Determine quantity and quality of gas emissions and air quality	Complex geospatial modeling may be required to analyze downwind measurements obtained from tracer tests; point measurements from flux chambers may not capture emission patterns; results of questionable quality
10. Gas-phase constituent concentrations in gas collection systems	Subsurface probes (see above) placed at the mouth of boreholes	Establish constituents of concern, identify releases, and establish concentration gradients	Provides a direct indication of the performance of the gas collection system and an indirect indication of cover performance

Parameter	How Measured	Use	Comments
11. Leachate hydraulic head on the primary liner	Vibrating wire piezometers and liquid-level measurements in sumps using drop-down resistivity probes	Assess the performance of the leachate collection and removal system	Measurements beyond sumps are rare, although vibrating wire piezometers on the liner have performed well in some cases
12. Volumetric seepage in the LCRS and LDS	Pumped volume or flow meter, depending on the system	Evaluate the effectiveness of LCRS and the primary liner system	Can provide an indirect assessment of cover performance, LCRS efficiency, liner integrity, and development of clogging
13. LCRS continuity	Dye testing and pumping tests	Indicates any clogging in the LCRS	Rarely used in practice
14. Leachate constituent concentrations	Chemical analysis of leachate samples for organic and inorganic constituents	Identify constituents of concern and evaluate the potential for mass flux of contaminants and degradation of the barrier system (e.g., hydraulic conductivity)	May be misleading (with respect to constituents of concern) due to chemical transformation within the liner system and subgrade
15. Geomembrane continuity	Electrical leak detection using conductive geomembranes or wire grids placed below membranes	Establish the location and frequency of defects in geomembranes	Typically used only in CQA, as the measuring techniques are ineffective when soil or waste cover on the geomembrane exceeds a meter or more
16. Settlement (surface and at depth)	Survey markers, settlement forks, extensometers	Determine settlement of cover systems	Total and differential settlements are required to assess cover performance
17. Temperature of soil and geosynthetic barrier components	Thermocouples	Estimate the service life of geosynthetics, determine thermal gradients, and conduct heat and moisture transfer analysis	Historically, rarely used in practice, but some recently reported field studies indicate measurement is important
18. Vertical barrier continuity	Geophysical methods, field measurements of hydraulic conductivity of slurry walls and of heads and constituent concentrations inboard and outboard of the wall	Identify defects in vertical barriers	Geophysical methods have potential but are rarely used in practice; hydraulic conductivity measurements are employed primarily for CQA via tests on field-recovered samples
19. Vertical barrier leak detection	Wells, drainage layers installed along the midsection of vertical barriers	Determine the amount of leakage and thus the performance of vertical walls	Results of questionable reliability; rarely used in practice; requires installation of the collection and removal system in the barrier; integrity of half of the thickness of the barrier is assessed
20. Radioisotope concentrations	Total radiation dose	Identify releases and establish concentration gradients	Primarily of concern for low-level radioactive waste

NOTES: CQA = construction quality assurance; EM = electromagnetic; GPR = ground-penetrating radar; LCRS = leachate collection and removal system; LDS = leak detection system; TDR = time domain reflectometry.

Appendix C

Construction Quality Assurance Monitoring Techniques

TABLE C.1 CQA Techniques for End-of-Construction Barrier Element Integrity

Barrier Element	CQA Integrity Monitoring Techniques
Compacted low-permeability soil	In situ density and water content testing; in situ hydraulic conductivity testing using infiltrometers, pan lysimeters, and borehole permeameters; physical sampling and laboratory testing for index properties and saturated hydraulic conductivity
Evapotranspirative cover soil	In situ water content testing; in situ flux rate testing using pan lysimeters; physical sampling and laboratory testing for index properties, including saturated hydraulic conductivity, and for the soil water characteristic curve
Geomembranes	Nondestructive seam testing; physical sampling and laboratory testing of seams; electrical leak detection testing; interface shear testing
Geosynthetic clay liners	Physical sampling and testing for bentonite unit weight and saturated hydraulic conductivity; interface shear testing
Soil-bentonite walls	Sounding of the trench prior to backfill placement, physical sampling, and testing backfill soil from the borrow pit for saturated hydraulic conductivity; physical sampling and testing backfill soil from the wall for saturated hydraulic conductivity; in situ saturated hydraulic conductivity testing of the wall backfill
Cement-bentonite walls	Sounding of the trench prior to backfill placement, physical sampling and testing of the cement-bentonite backfill for saturated hydraulic conductivity; physical sampling and testing of cores from the wall for saturated hydraulic conductivity
Soil-mixed walls	Physical sampling and testing of cores from the wall for saturated hydraulic conductivity

Appendix D

Biographical Sketches of Committee Members

James K. Mitchell, *Chair,* is University Distinguished Professor, emeritus, in the Department of Civil and Environmental Engineering, Virginia Polytechnic Institute and State University. Prior to joining Virginia Tech in 1994, he spent 35 years on the civil engineering faculty of the University of California, Berkeley. He received his Ph.D. in civil engineering from the Massachusetts Institute of Technology. Dr. Mitchell's research interests are in geotechnical engineering, with emphasis on soil properties and behavior, ground improvement, environmental geotechnics, and in situ testing. Much of his recent work has focused on the application of knowledge in these areas to waste landfills, waste containment barriers, and mitigation of seismic risk to earth structures. He is a widely known and well-respected leader who has received many awards for notable research achievements and for international contributions to engineering practice and education. He has served on several National Research Council (NRC) boards and committees dealing with geotechnical engineering and waste containment systems, including the Geotechnical Board (chair), the Committee for Noninvasive Characterization of the Shallow Subsurface for Environmental and Engineering Applications, the Committee on Subsurface Contamination at Department of Energy Complex Sites: Research Needs and Opportunities (vice chair), the Committee on Geological and Geotechnical Engineering in the New Millennium: Opportunities for Research and Technological Innovation, and the Committee for Review of the Hanford Site's Environmental Remediation Science and Technology Plan. He is a member of both the National Academy of Sciences and the National Academy of Engineering.

Lisa Alvarez-Cohen is the Fred and Claire Sauer Professor of Environmental Engineering at the University of California, Berkeley. She received her Ph.D. in environmental engineering and science from Stanford University. Her current research interests include the biotransformation of contaminants in the subsurface and innovative methods for evaluating in situ bioremediation, including molecular biological and stable isotopic techniques. Dr. Alvarez-Cohen has served on several NRC committees related to subsurface contamination and remediation, including the Committee on Source Removal of Contaminants in the Subsurface and the Committee on In Situ Bioremediation. She is a fellow of the American Academy of Microbiology.

Estella A. Atekwana is a professor at the Boone Pickens School of Geology at Oklahoma State University, Stillwater. Previously she was a professor of geophysics at the University of Missouri, Rolla. She received her Ph.D. in geophysics from Dalhousie University. Her research focuses on tectonic studies and the application of near-surface geophysical monitoring techniques (gravity, magnetic, seismic, geoelectrical) to aquifer vulnerability, groundwater contamination, and remediation. She is also pioneering the field of biogeophysics using geophysical methods to examine microbe-mineral interactions and the effect of this interaction on the subsurface environment. Dr. Atekwana chaired the International Committee of the Environmental and Engineering Geophysical Society and recently completed a term as vice president for committees.

Susan E. Burns is an associate professor in the School of Civil and Environmental Engineering at the Georgia Institute of Technology. Prior to joining the faculty in 2004, she spent 7 years on the faculty at the University of Virginia. She received her Ph.D. in civil engineering from the Georgia Institute of Technology. Her research focuses on the transport of air bubbles through saturated porous media, physical and chemical behavior of organic-exchanged soil minerals, and remediation of organic compounds using in situ treatment technologies. Dr. Burns received the Edmund Friedman Young Engineer Award from the American Society of Civil Engineers in 2000. She is a board member of the U.S. Universities Council on Geotechnical Engineering Research and a former member of the NRC Committee on Geological and Geotechnical Engineering.

Robert B. Gilbert is a professor in the Civil, Architectural, and Environmental Engineering Department at the University of Texas at Austin. In addition to his faculty responsibilities, he teaches short courses for geo-professionals on risk-based decision making and waste containment systems. He received his Ph.D. in civil engineering from the University of Illinois, Urbana-Champaign. Dr. Gilbert's research interests include performance reliability and risk management for geotechnical and geoenvironmental systems, waste containment, and site remediation. He chairs the Transportation Research Board's Subcommittee on Reliability in Geotechnical and Pavement Engineering and is a member of the risk analysis and management committees of both the American Society of Civil Engineers Geo-Institute and the International Society of Soil Mechanics and Geotechnical Engineering.

Edward Kavazanjian, Jr., is associate professor of civil and environmental engineering at Arizona State University in Tempe. Prior to moving to the university in 2004, Dr. Kavazanjian spent 20 years in engineering practice. He received a Ph.D. in geotechnical engineering from the University of California, Berkeley. Dr. Kavazanjian is recognized for his work on analysis and design of waste containment systems and on geotechnical aspects of earthquake engineering. He has served as engineer in charge of major infrastructure development projects involving up to $8.5 million in engineering services and $150 million in construction and as principal and co-principal investigator on geotechnical engineering research projects sponsored by the Department of Transportation, the National Science Foundation, the U.S. Geological Survey, and the U.S. Army Corps of Engineers. He currently serves on the board of governors of the Geo-Institute of the American Society of Civil Engineers and as chair of the geoseismic concerns subcommittee of the Transportation Research Board's Committee on Seismic Design of Bridges.

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Charles D. Shackelford is a professor in the Department of Civil Engineering and director of the Rocky Mountain Regional Hazardous Substance Research Center at Colorado State University. He received his Ph.D. in civil engineering from the University of Texas. His research interests concern the flow and transport of hazardous liquids and contaminants through clay soils and geosynthetic containment barriers. Dr. Shackelford's work on diffusion in containment barrier design was acknowledged in 1995 with the Walter L. Huber Civil Engineering Prize from the American Society of Civil Engineers (ASCE). He has been involved with several committees for the Geo-Institute of ASCE, including the environmental geotechnics committee (past chair and current member) and the Technical Coordination Council (member). He also was an elected board member of the U.S. Universities Council on Geotechnical Education and Research.

Hari D. Sharma is a principal of Geosyntec Consultants, a private company that specializes in waste management, engineered barriers and synthetics, geotechnical engineering, and design, permitting, and construction quality assurance. He received his Ph.D. in geotechnical engineering from Purdue University. Dr. Sharma has over 30 years of experience directing field investigations, designing and managing landfills, conducting related remediation, and monitoring landfill construction in the United States and Canada. In addition to his practical work, he has published or presented papers on all aspects of landfills. His three books, including *Waste*

Containment Systems, Waste Stabilization and Landfills: Design and Evaluation, and his recently published book, *Geoenvironmental Engineering: Site Remediation, Waste Containment, and Emerging Waste Management Technologies*, are widely used in industry and academia. For many years he served on the Environmental Geotechnics Committee of the American Society of Civil Engineers.

Nazli Yesiller is an independent consultant in San Luis Obispo, California. She was previously an associate professor in the Department of Civil and Environmental Engineering at Wayne State University. She received her Ph.D. in civil and environmental engineering from the University of Wisconsin. Her research interests focus on nondestructive testing and image analysis of geosynthetics and soils, desiccation of barrier systems, and thermal performance of landfill systems. Dr. Yesiller is a member and officer of several committees of the American Society for Testing and Materials International, which are developing standards for materials ranging from geosynthetics to soils. She is also a member of the Geoenvironmental Engineering Committee of the American Society of Civil Engineers.

Acronyms and Abbreviations

ACC	asphalt cement concrete
ADRE	advective-dispersive-reactive equation
CCL	compacted clay liner
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act
CIWMB	California Integrated Waste Management Board
CQA	construction quality assurance
DOE	Department of Energy
EPA	Environmental Protection Agency
GCL	geosynthetic clay liner
GM	geomembrane
HDPE	high-density polyethylene
HELP	Hydrologic Evaluation of Landfill Performance
MSW	municipal solid waste
MULTIMED	Multimedia Exposure Assessment Model
NYDEC	New York Department of Environmental Conservation
PCC	Portland cement concrete
RCRA	Resource Conservation and Recovery Act
RMD	ratio of monovalent-to-divalent cations
UMTRA	Uranium Mill Tailings Remedial Action