Chapter 2
Individual Components of Cover Systems

2.1 Introduction

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

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

2.2 Surface Layer

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

2.2.1 General Issues

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

2.2.2 Elements of Design

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

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

2.2.2.1 Slope Inclination

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

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

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

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

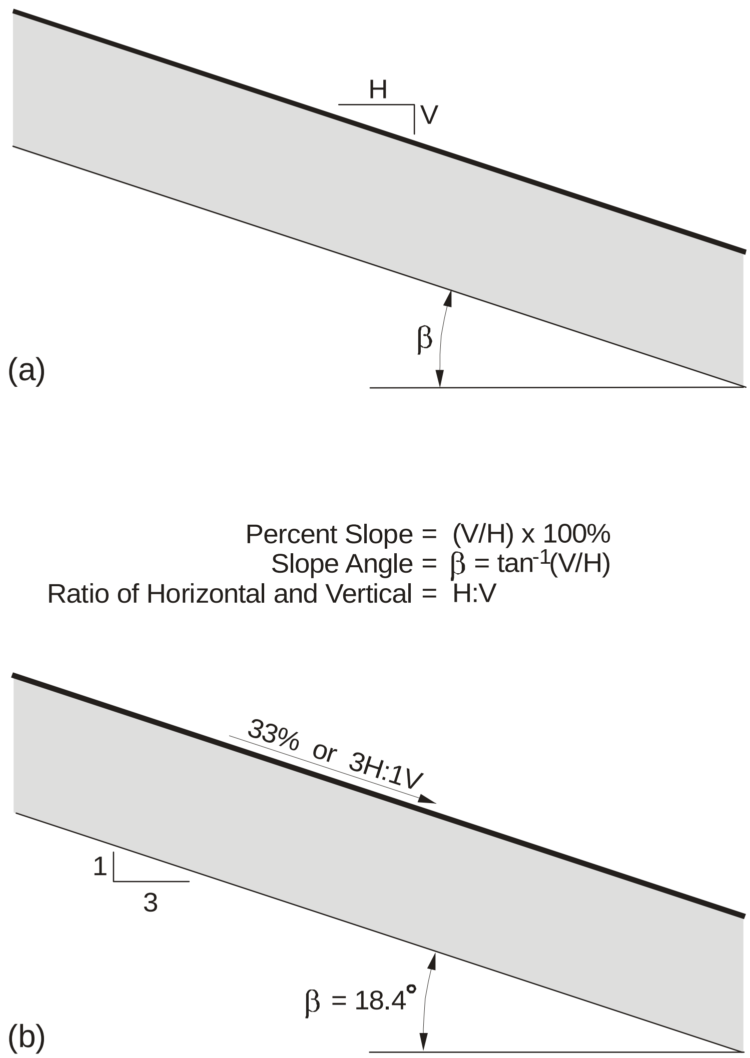

Figure 2-1. Slope Inclination: (a) Definitions; and (b) Example.

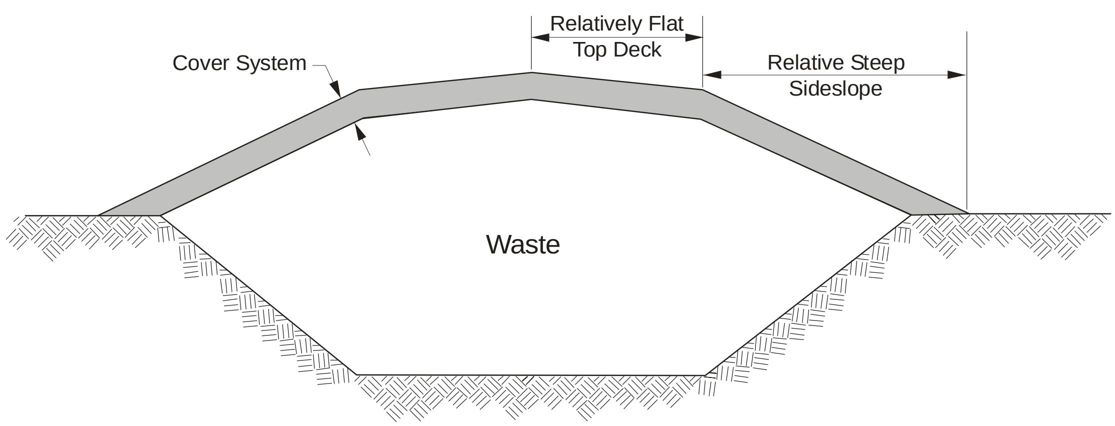

Figure 2-2. Relatively Steep and Flat Sections on a Typical Landfill Cover System.

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

2.2.2.2 Materials

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

2.2.2.2.1 Topsoil

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

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

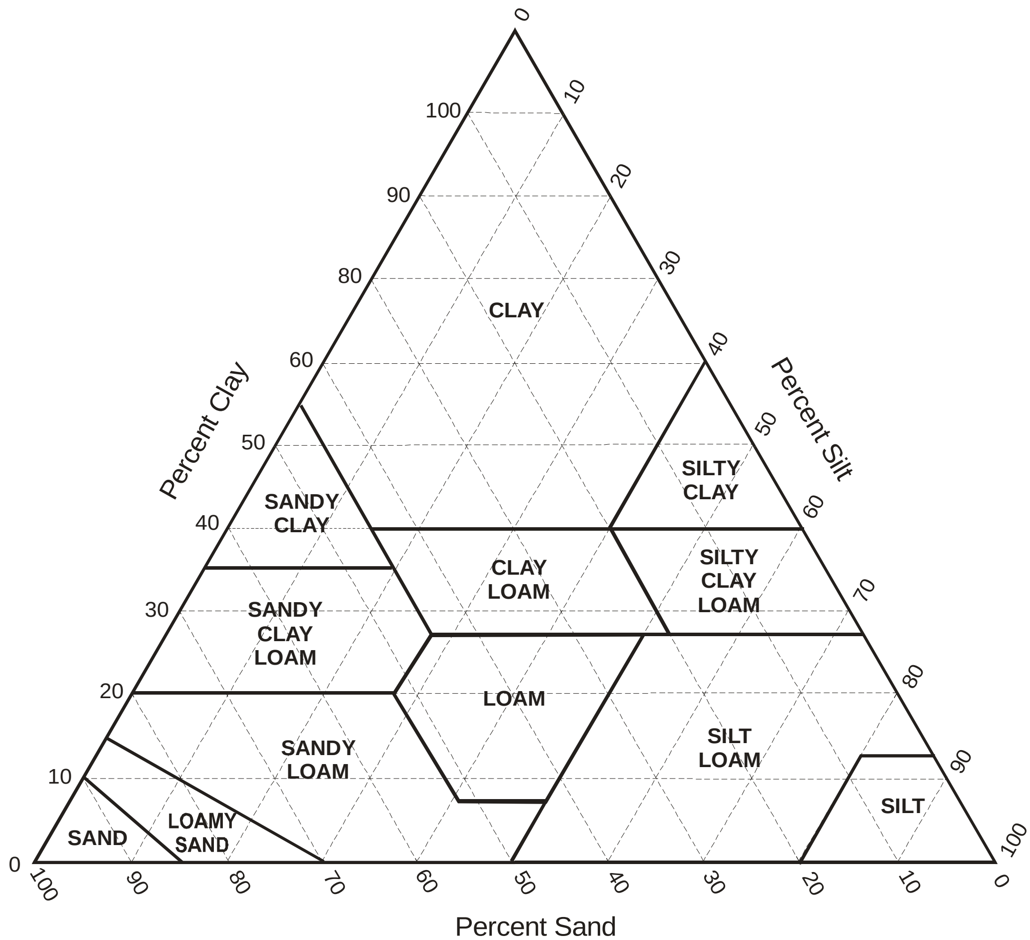

Figure 2-3. USDA Soil Classification System. USDA Particle Sizes: Sand, 0.05 – 2 mm; Silt, 0.002 - 0.05 mm; and Clay, < 0.002 mm.

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

2.2.2.2.2 Amended Topsoil

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

2.2.2.2.3 Soil-Gravel Mixture

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

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

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

2.2.2.2.4 Gravel Veneer

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

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

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

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

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

2.2.2.2.5 Riprap

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

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

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

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

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

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

2.2.2.2.6 Asphaltic Concrete

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

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

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

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

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

2.2.2.2.7 Other Materials

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

2.2.2.3 Thickness

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

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

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

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

2.2.3 Vegetation

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

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

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

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

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

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

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

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

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

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

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

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

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

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

2.2.4 Surface-Water Control

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

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

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

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

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

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

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

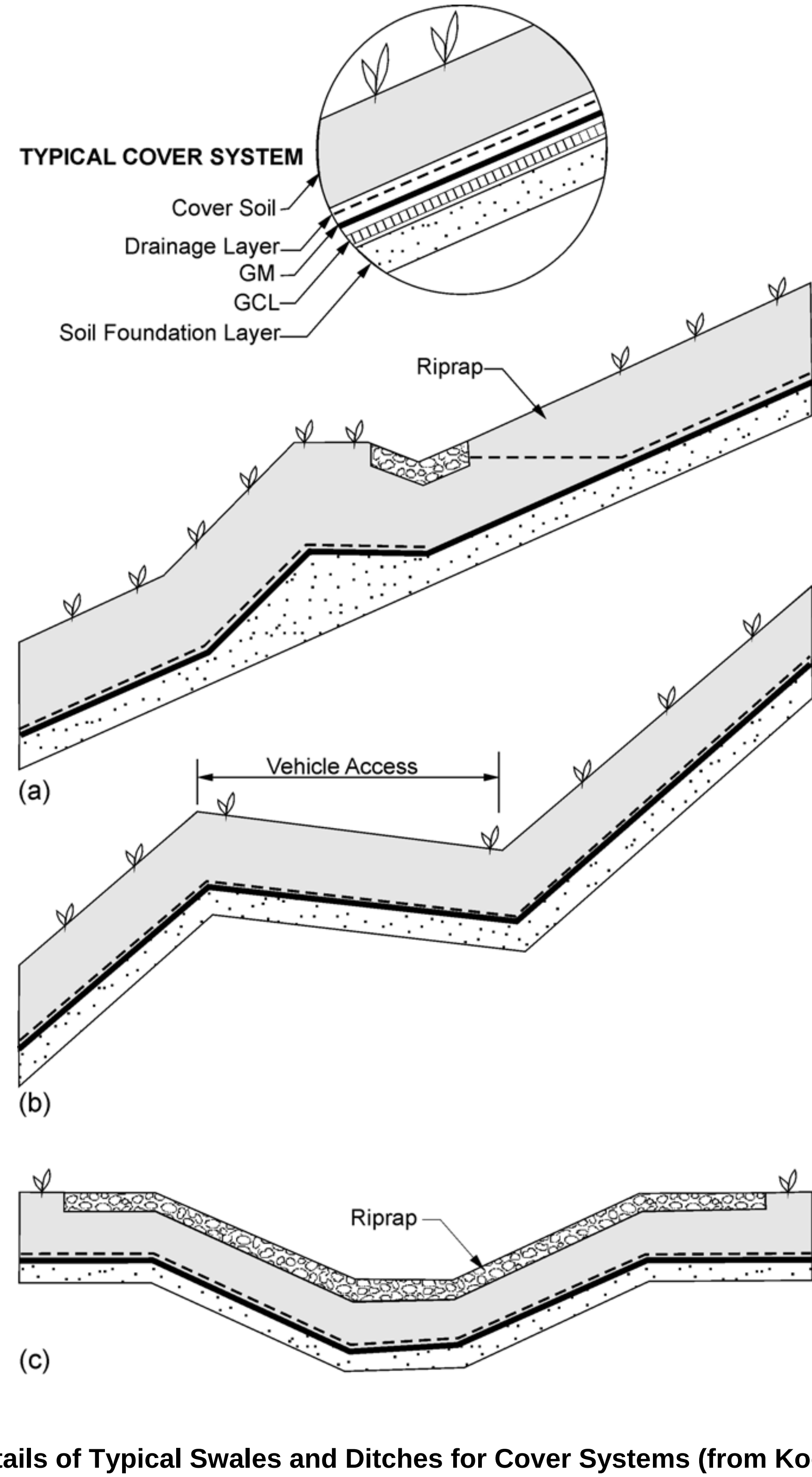

Figure 2-5. Details of Typical Swales and Ditches for Cover Systems (from Koerner and Daniel, 1997): (a) Swale Constructed with Add-on Berm; (b) Swale Constructed by Benching Sideslopes; and (c) Ditch Sometimes Constructed Adjacent to Access Road.

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

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

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

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

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

Input values to the rational equation are as follows:

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

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

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

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

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

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

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

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

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

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

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

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

2.2.5 Erosion Protection

2.2.5.1 Overview

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

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

2.2.5.2 Nature of Erosion

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

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

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

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

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

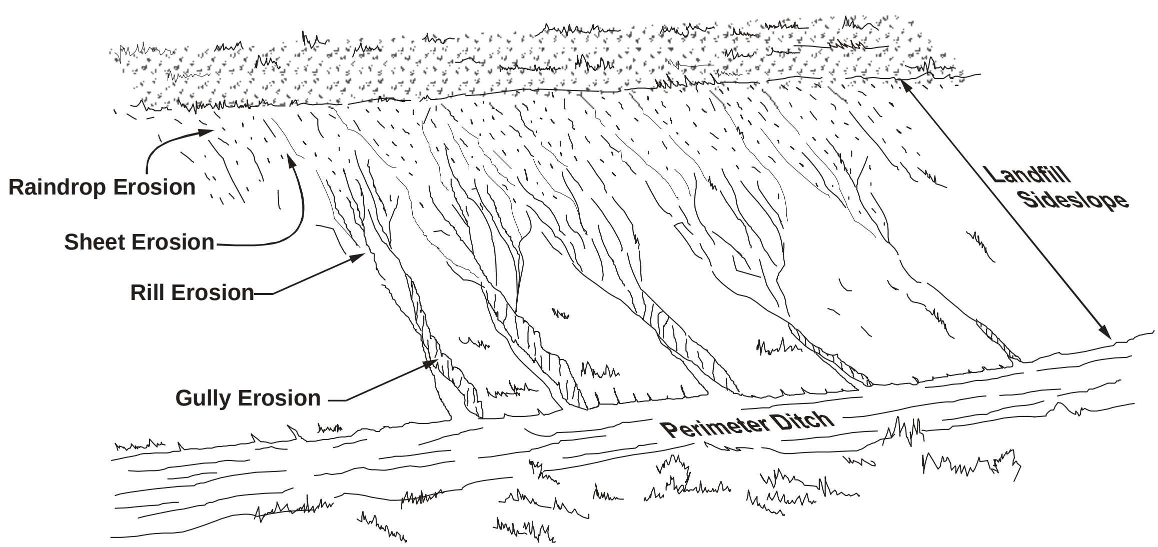

Figure 2-7. Types of Water Erosion That May Occur on a Cover System.

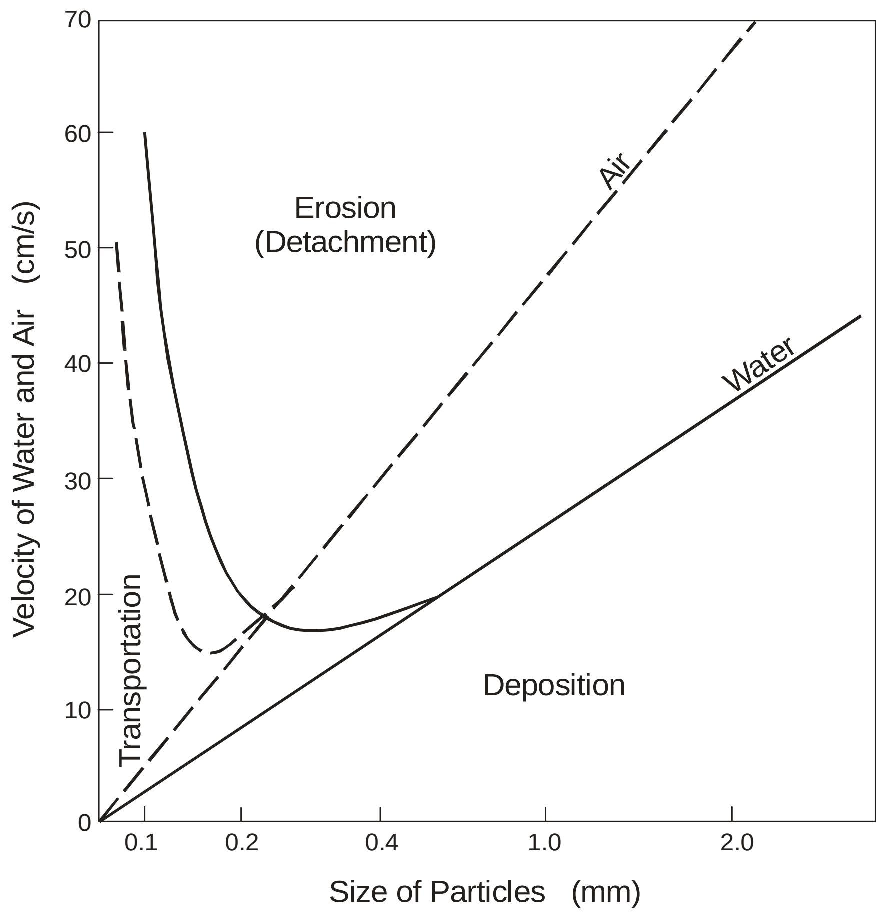

Figure 2-8. Relationship Between Erosion Mechanism (Air or Water), Particle Size and Fluid Velocity (Garrels, 1951 as referenced by Mitchell, 1993).

2.2.5.3 Short-Term and Long-Term Erosion

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

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

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

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

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

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

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

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

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

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

2.2.5.4 Sheet and Rill Erosion

2.2.5.4.1 Universal Soil Loss Equation

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

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

The RUSLE is expressed as:

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

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

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

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

2.2.5.4.2 Water Erosion Prediction Project (WEPP) Model

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

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

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

2.2.5.5 Gully Erosion

2.2.5.5.1 Overview

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

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

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

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

2.2.5.5.2 Tractive Force Method for Vegetated Surface Layers

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

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

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

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

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

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

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

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

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

2.2.5.5.3 Horton/NRC Method for Vegetated Surface Layers

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

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

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

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

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

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

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

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

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

where: β = slope angle (degrees).

2.2.5.5.4 Permissible Velocity Method for Vegetated Surface Layers

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

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

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

where all other terms are as defined previously.

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

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

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

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

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

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

2.2.5.5.5 Stephenson Method for Gravel or Riprap Surface Layers

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

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

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

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

2.2.5.6 Wind Erosion

2.2.5.6.1 Revised Wind Erosion Equation.

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

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

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

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

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

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

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

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

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

2.2.5.6.2 Wind Erosion Prediction System

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

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

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

2.2.5.7 Erosion Control Materials

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

2.2.5.7.1 Temporary Erosion Control Materials

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

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

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

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

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

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

2.2.5.7.2 Permanent Erosion Control Materials

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

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

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

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

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

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

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

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

2.2.6 Construction

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

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

2.2.7 Maintenance

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

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

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

2.2.8 Monitoring

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

2.3 Protection Layer

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

2.3.1 General Issues

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

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

2.3.2 Elements of Design

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

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

2.3.2.1 Materials

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

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

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

2.3.2.2 Thickness

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

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

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

2.3.2.2.1 Desiccation Protection

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

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

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

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

2.3.2.2.2 Frost Penetration Protection

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

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

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

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

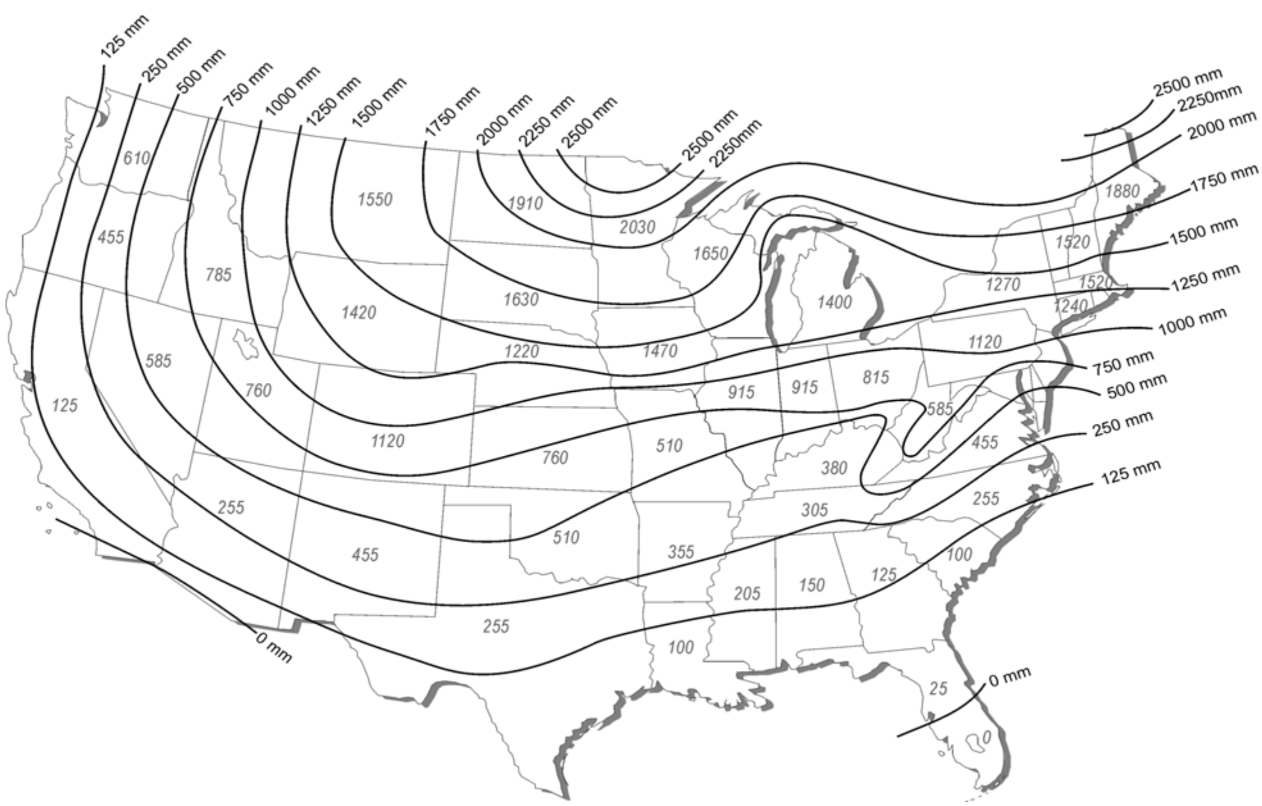

Figure 2-9. Contours of Maximum Frost Penetration Depth (mm) and State Averages (mm) (modified from Koerner and Daniel, 1997).

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

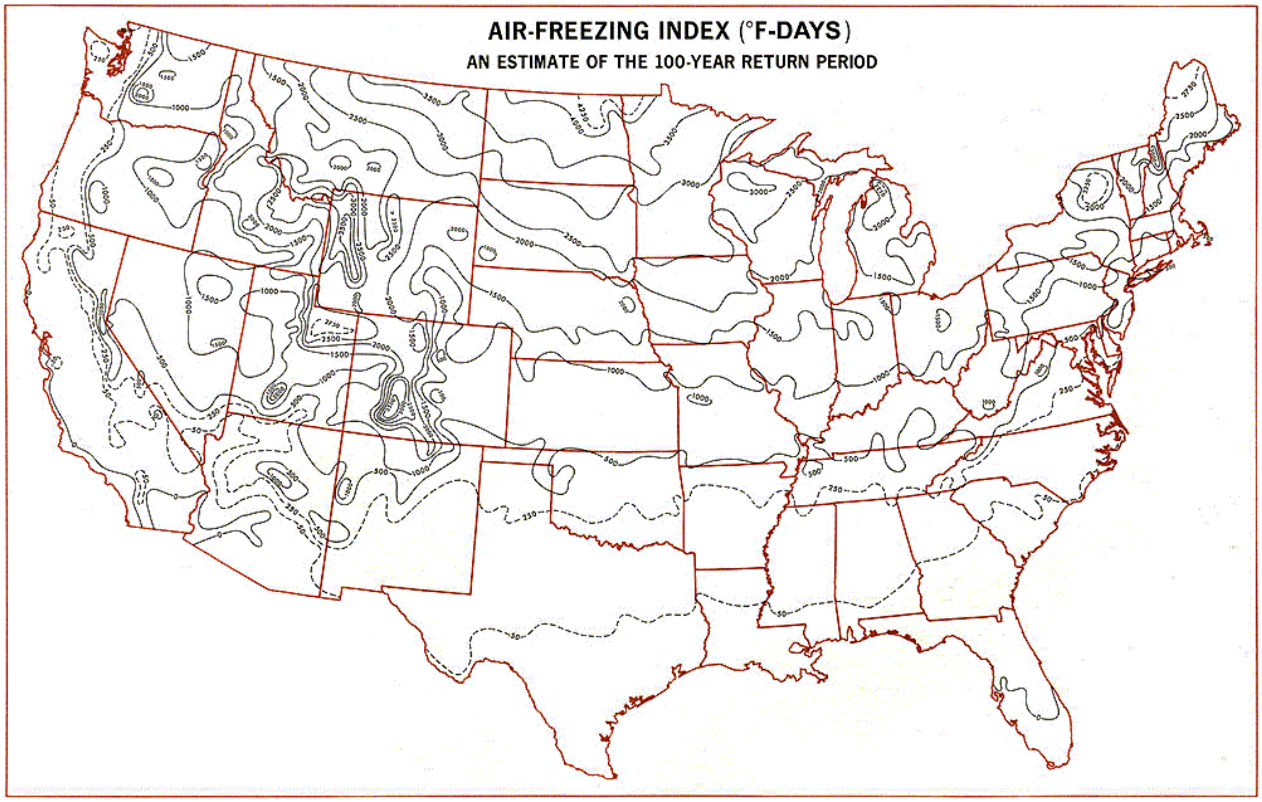

Figure 2-10. Contours of Air Freezing Indicies (°F-days) with a 100-yr Return Period (downloaded from http://lwf.ncdc.noaa.gov/oa/fpsf/fpsf.html).

2.3.2.2.3 Accidental Human Intrusion Protection

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

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

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

2.3.2.2.4 Root Penetration Protection

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

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

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

Climate influences the depth of root penetration, and even the materials into which roots penetrate have an influence on root depth. Roots generally seek out lightly-compacted soils that contain moisture. Roots will not, as a general rule, penetrate into dry or heavily compacted soils. In soil profiles containing a finer-grained soil overlying a coarser-grained soil, roots will remain in the relatively moist, finer-gained soil and will not penetrate into the coarser-grained soil as long as the coarser soil remains dry. If the coarser-grained soil becomes wet, then the roots will seek moisture in this soil.

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

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

2.3.2.2.5 Burrowing Animal Protection

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

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

- Burrowing animals cast soil on the surface, thereby increasing erosion of the cover system.

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

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

2.3.2.2.6 Vegetation Support

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

2.3.2.2.7 Water Storage

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

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

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

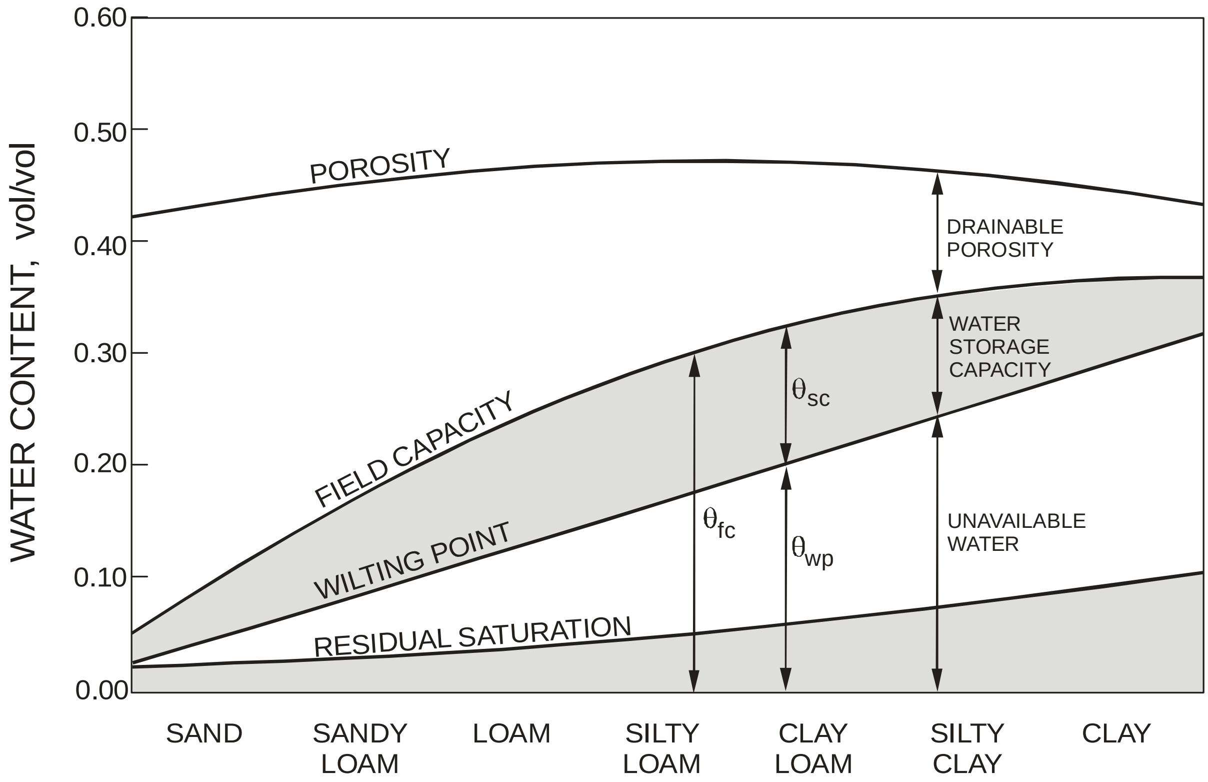

Figure 2-11. Relation Among Moisture Retention Parameter and Soil Texture Class (modified from Schroeder et al., 1994).

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

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

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

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

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

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

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

2.3.2.2.8 Radon Attenuation

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

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

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

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

2.3.3 Construction

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

2.3.4 Maintenance

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

2.3.5 Monitoring

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

2.4 Drainage Layer

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

2.4.1 General Issues

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

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

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

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

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

2.4.2 Elements of Design

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

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

2.4.2.1 Materials

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

2.4.2.1.1 Granular Materials

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

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

Specifications for granular materials often require:

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

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

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

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

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

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

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

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

2.4.2.1.2 Geosynthetics

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

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

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

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

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

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

Specifications for geosynthetic drainage layers often require:

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

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

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

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

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

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

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

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

2.4.2.2 Thickness of Granular Layers

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

2.4.2.3 Required Flow Capacity

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

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

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

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

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

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

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

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

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

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

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

2.4.2.4 Drainage Layer Outlets

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

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

2.4.3 Construction

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

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

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

2.4.4 Maintenance

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

2.4.5 Monitoring

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

2.5 Hydraulic Barrier

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

2.5.1 General Issues

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

2.5.2 Elements of Design

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

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

2.5.2.1 Materials

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

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

2.5.2.1.1 GMs

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

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

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

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

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

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

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

Specifications for GM hydraulic barriers often require:

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

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

2.5.2.1.2 GCLs

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

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

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

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

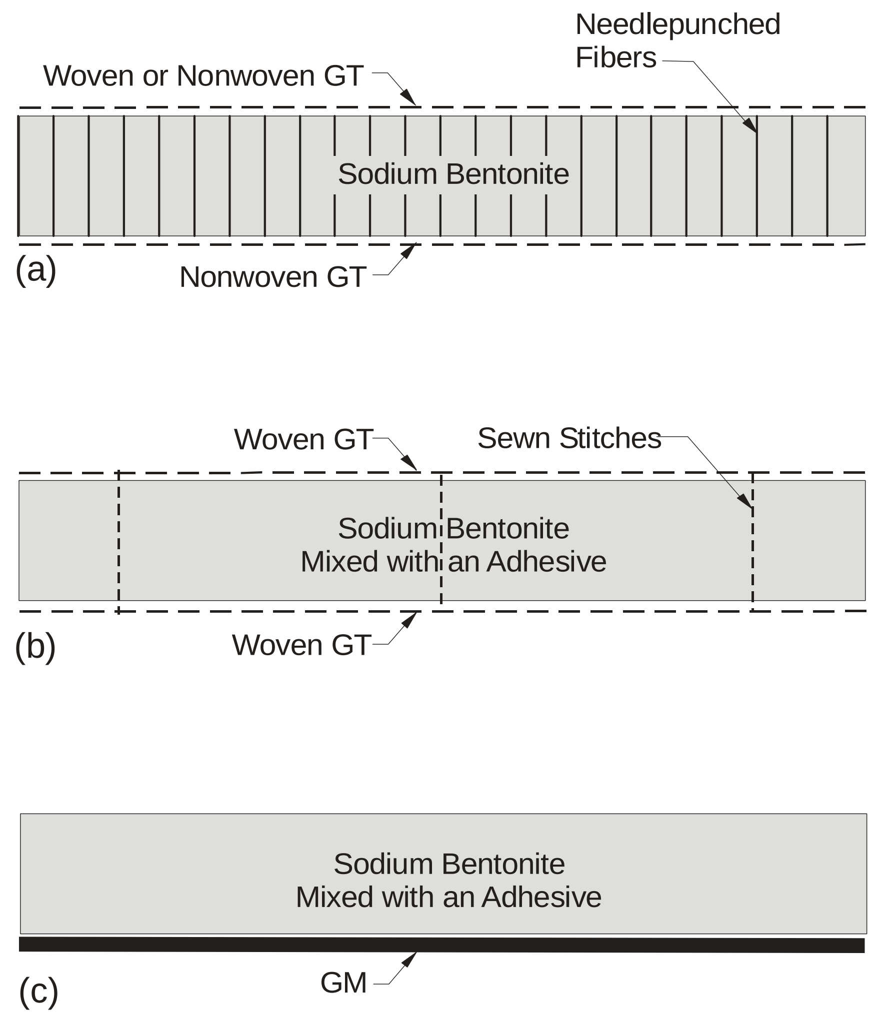

Figure 2-12. Types of GCLs Commonly Used as Cover System Barriers: (a) Reinforced, GT-Encased, Needlepunched GCL; (b) Reinforced, GT-Encased, Stitch-Bonded GCL; and (c) Unreinforced, GM-Supported GCL.

Specifications for GCL hydraulic barriers often require:

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

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

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

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

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

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

2.5.2.1.3 CCLs

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

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

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

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

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

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

2.5.2.2 Thickness

2.5.2.2.1 GMs

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

2.5.2.2.2 GCLs

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

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

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

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

2.5.2.2.3 CCLs

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

2.5.2.3 Percolation

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

It is recommended that the percolation objective for the cover system be defined, at least qualitatively, prior to design. For example, if the cover system is for a MSW landfill, the design maximum percolation might be in the range of 0.1 to 1.0 mm/year (see Section 1.2.3). Methods for estimating percolation rates through cover systems are presented in Chapter 4.

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

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

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

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

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

2.5.2.4 Gas Containment

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

2.5.2.5 Differential Settlement

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

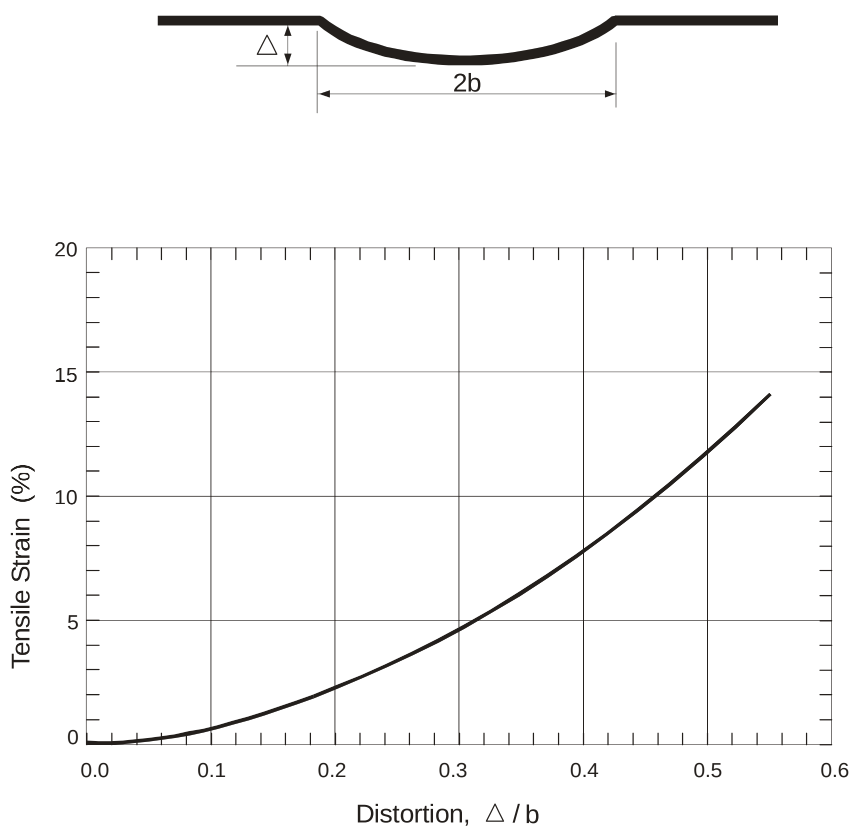

Figure 2-13. Theoretical Relationship Between Tensile Strain and Angular Distortion (modified from Gilbert and Murphy, 1987).

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

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

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

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

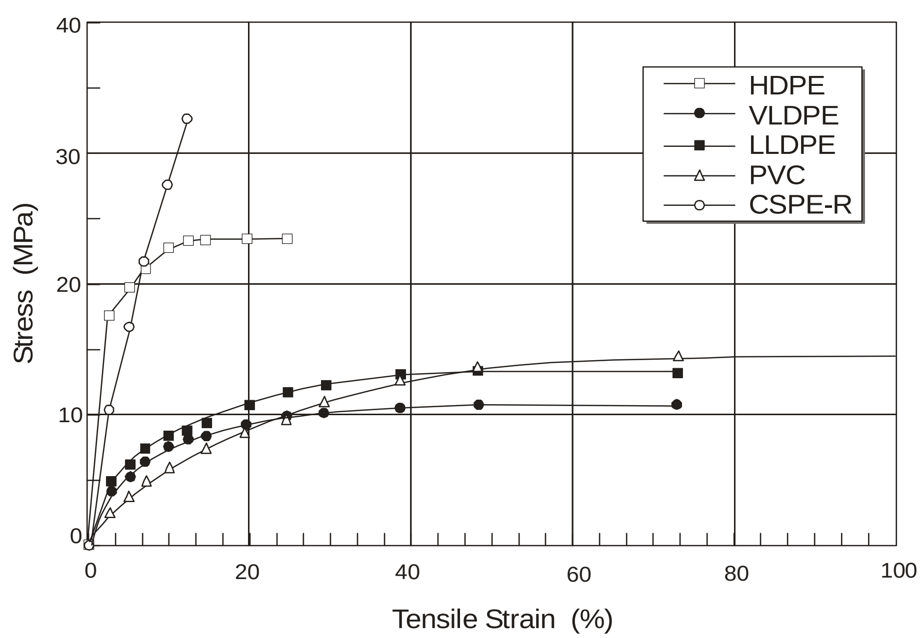

Figure 2-14. Stress-Strain Behavior of Common GM Materials Subjected to Axi-Symmetric, Out-of-Plane Tensile Strain (modified from Koerner et. al., 1990).

2.5.2.6 *Wet-Dry Cycles*

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

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

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

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

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

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

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

2.5.2.7 Freeze-Thaw Cycles

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

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

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

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

2.5.2.8 Shear Strength

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

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

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

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

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

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

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

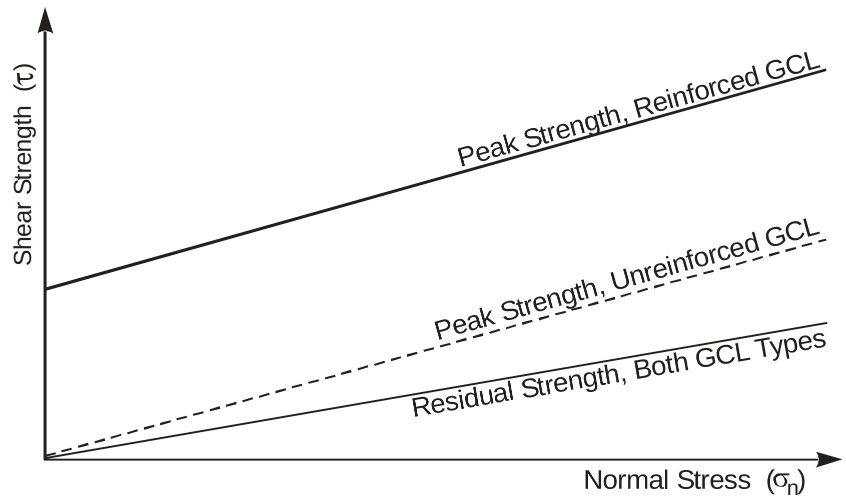

Figure 2-15. Comparison of Shear Strengths for Internally Reinforced GCLs and Unreinforced GCLs.

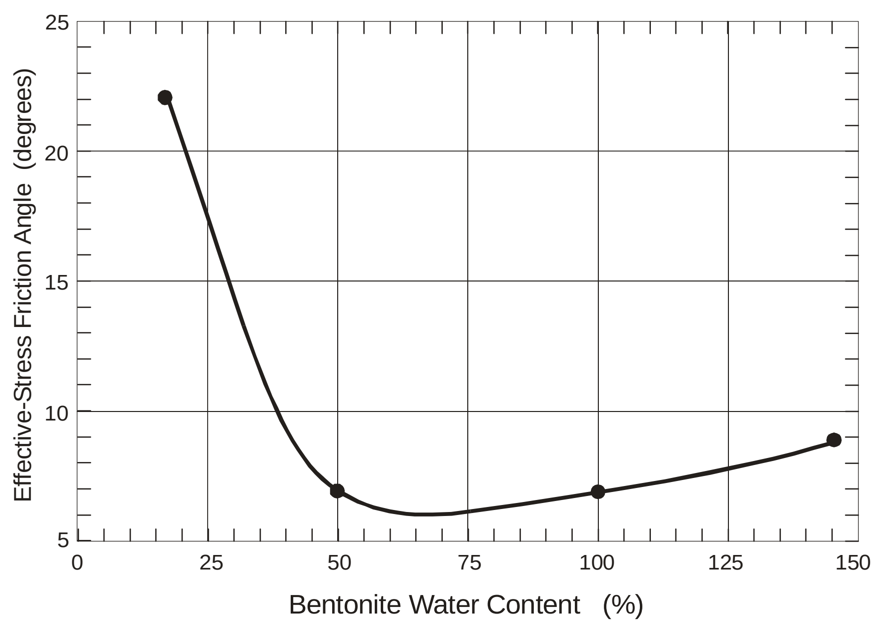

Figure 2-16. Effect of Bentonite Water Content on Shear Strength of an Unreinforced GCL (modified from Daniel et al, 1993).

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

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

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

2.5.2.9 Accidental or Intentional Puncture

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

2.5.2.10 Anticipated Lifetime

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

2.5.2.10.1 GMs

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

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

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

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

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

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

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

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

2.5.2.10.2 GCLs

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

2.5.2.10.3 CCLs

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

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

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

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

2.5.3 Composite Hydraulic Barriers

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

2.5.3.1 Prompt Placement of Overlying Materials

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

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

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

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

2.5.3.2 Intimate Contact

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

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

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

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

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

2.5.4 Construction

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

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

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

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

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

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

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

2.5.5 Maintenance

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

2.5.6 Monitoring

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

2.6 Gas Collection Layer

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

2.6.1 General Issues

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

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

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

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

2.6.2 Elements of Design

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

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

2.6.2.1 Materials

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

2.6.2.1.1 Granular Materials

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

Specifications for granular materials often require:

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

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

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

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

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

2.6.2.1.2 Geosynthetics

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

Specifications for geosynthetic gas collection layers often require:

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

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

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

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

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

2.6.2.2 Thickness of Granular Layers

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

2.6.2.3 Required Flow Capacity

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

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

2.6.3 Gas Collection Layer Outlets

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

2.6.4 Construction

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

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

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

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

2.6.5 Maintenance

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

2.6.6 Monitoring

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

2.7 Foundation Layer

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

2.7.1 General Issues

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

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

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

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

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

2.7.2 Elements of Design

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

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

2.7.2.1 Materials

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

2.7.2.2 Thickness

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

2.7.3 Construction

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

2.7.4 Maintenance

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

2.7.5 Monitoring

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

2.8 Examples of Cover Systems for Different Applications

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

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

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

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

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

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

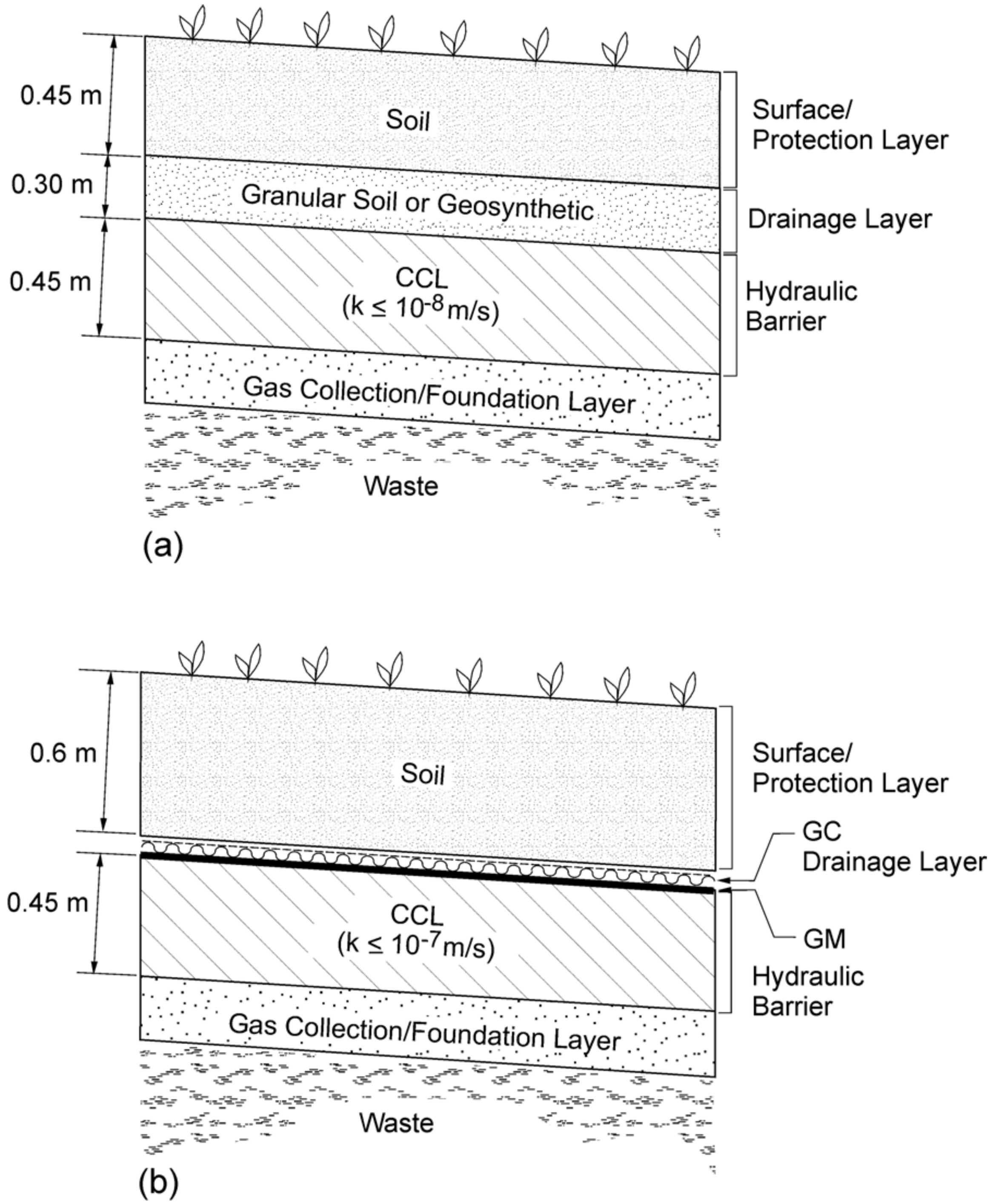

Figure 2-17. Examples of Hydraulic Barrier-Type of Cover Systems for MSW Landfills: (a) Cover System with CCL Hydraulic Barrier; (b) Cover System with GM/GCL Composite Hydraulic Barrier.

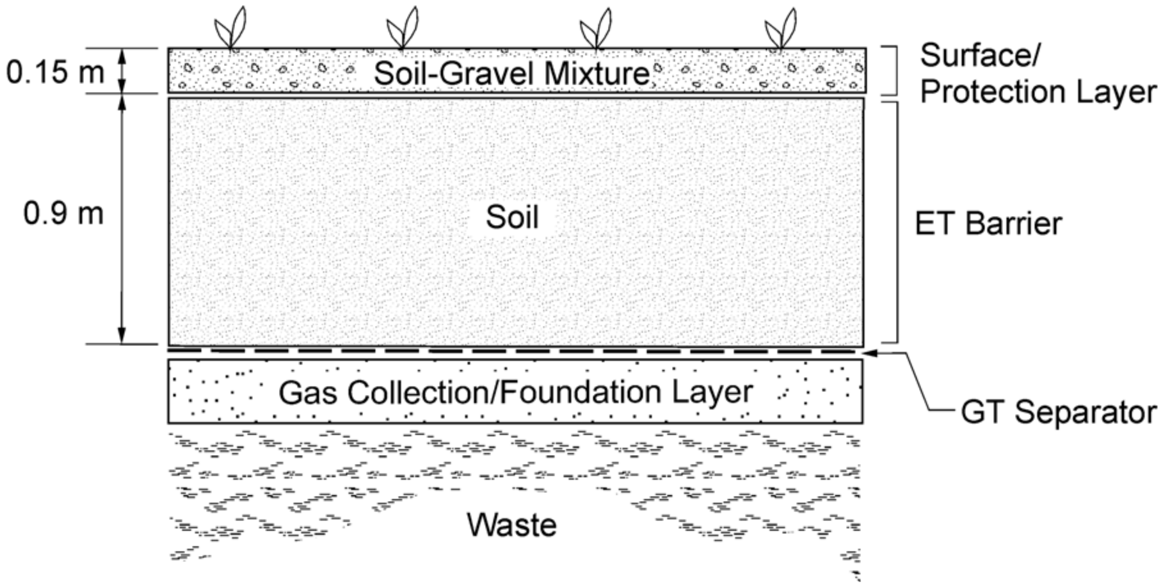

Figure 2-18. Example of ET Barrier-Type of Cover System for MSW Landfills.

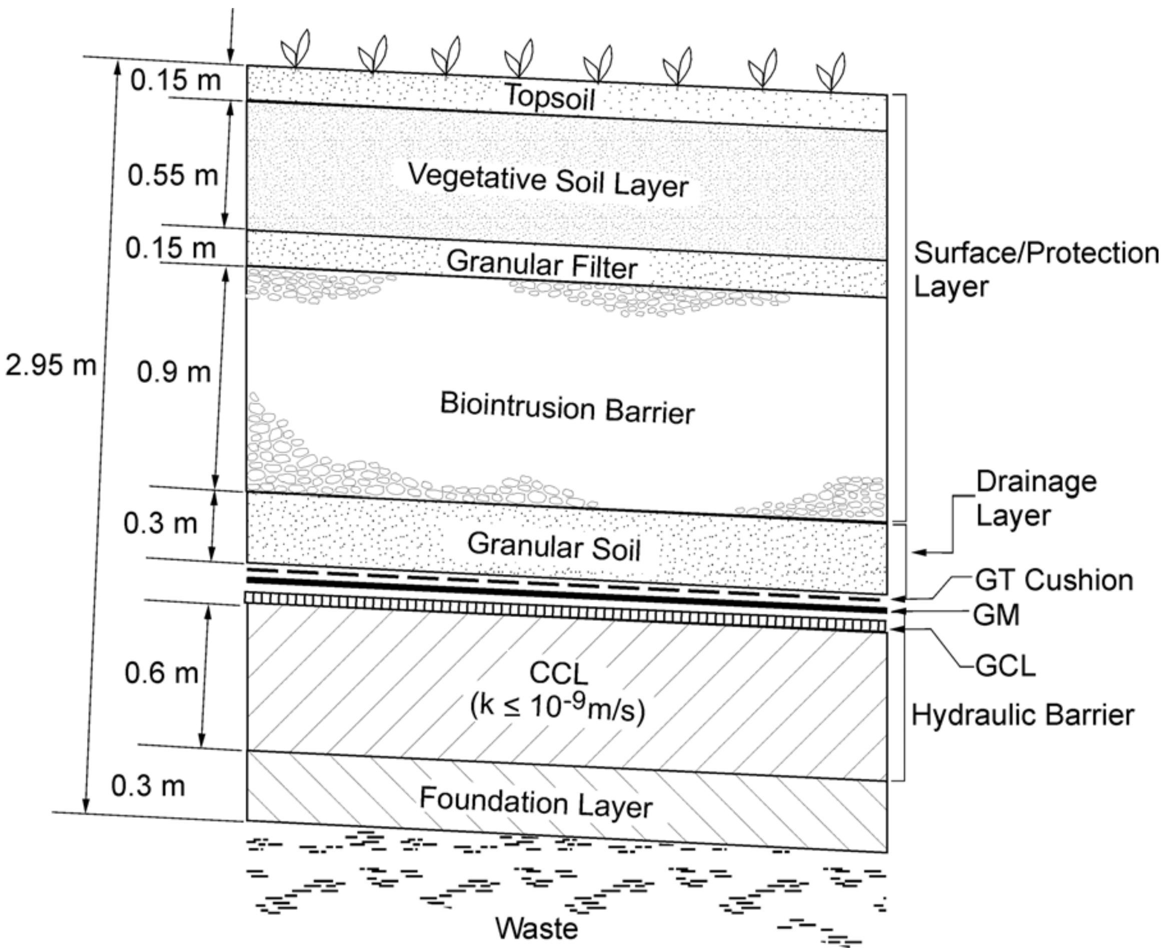

Figure 2-19. Example of Cover System for a Low-Level Radioactive Waste Landfill.

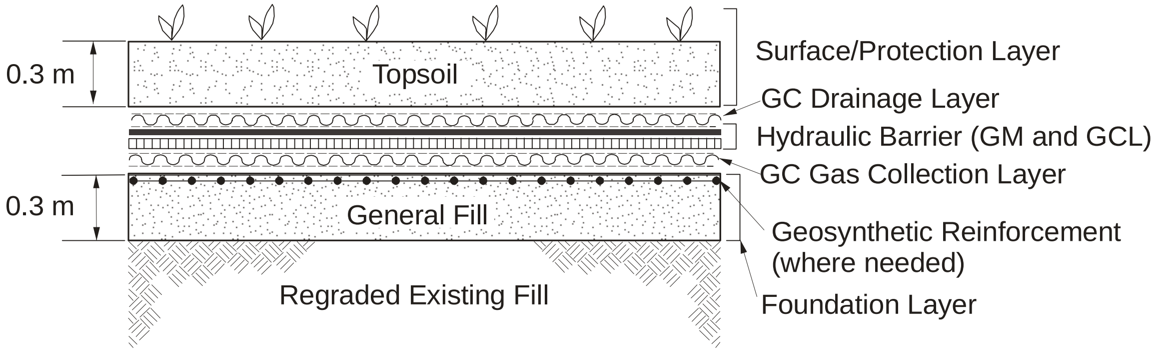

Figure 2-20. Example of Lightweight Cover System for a HW Remediation Site.

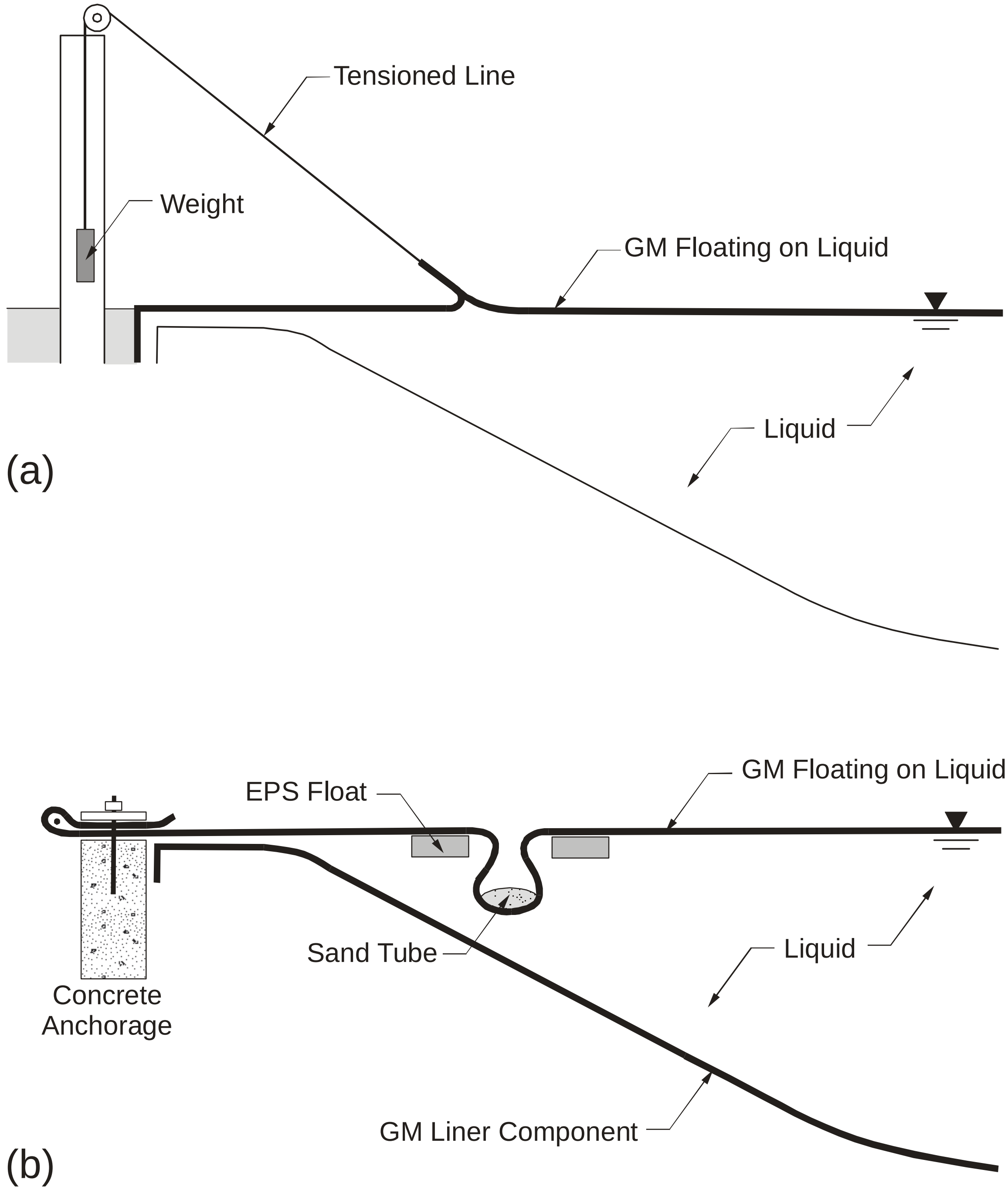

Figure 2-21. Examples of Floating "Cover System" for HW Impoundments: (a) GM with Tensioned Lines; and (b) GM with Floats and Sand Tubes.