SLOPE STABILITY AND STABILIZATION METHODS

Second Edition

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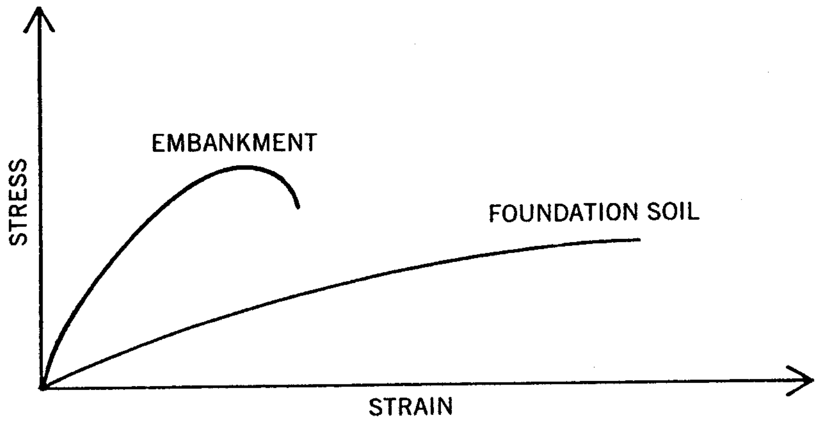

Figure 1.7 Example of stress–strain incompatibility.

Peak strengths of the embankment and the foundation soils cannot be mobilized simultaneously because of stress–strain incompatibility (Figure 1.7). Hence, a stability analysis performed using peak strengths of soils would overestimate the factor of safety. Many engineers perform stability analyses using soil strengths that are smaller than the peak values to allow for possible progressive failure.

Shale Embankments Embankments constructed of shale materials often have slope stability and settlement problems. According to DiMillio and Strohm (1981), the underlying causes of shale fill slope failures and excessive settlement frequently appear to be:

(1) Deterioration or softening of certain shales over time after construction
(2) Inadequate compaction of the shale fill
(3) Saturation of the shale fill

These types of failures have been found to be typical in many areas from the Appalachian region to the Pacific coast. In general, severe problems with shales in embankments are found in states east of the Mississippi River rather than west of the river (DiMillio and Strohm, 1981). Embankments can use fill originating from shale formations successfully if the borrow source is not particularly prone to long-term decomposition and if adequate compaction and drainage are required. In addition, shale embankments should be keyed into any sloping surfaces by using benches and installing drainage measures to intercept subsurface water that may enter the foundation area. Guidelines for design and construction of shale embankments have been established by Strohm et al. (1978).

1.4.2 Cut Slopes

Shallow and deep cuts are important features in any civil engineering project. The aim in a slope design is to determine a height and inclination that is economical

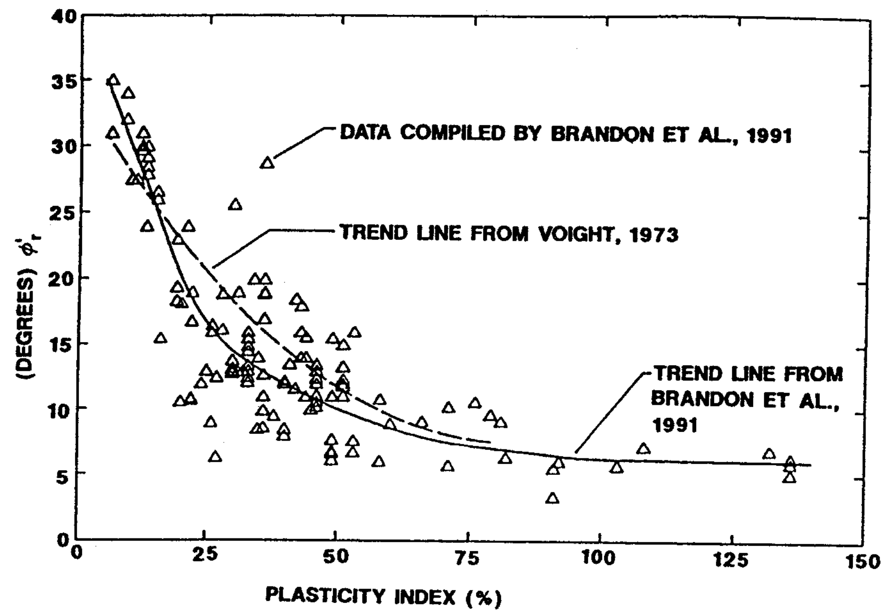

Figure 1.32 Plasticity index residual friction angle relationship. (Voight, 1973, and Brandon et al., 1991.)

Often to simplify the analysis, c' is assumed equal to zero and only the residual friction angle is used.

(2) A stability analysis is performed using slope geometry, groundwater levels, and external loading conditions at the time of failure. The analysis yields a factor of safety, FOS, that corresponds to the trial strengths from Step 1.

(3) The trial strengths from Step 1 are adjusted using the safety factor computed in Step 2, according to the following formulas:

$$\phi'_r(\text{adjusted}) = \tan -1\,\frac{\tan\{\phi_r(\text{trial})\}}{\text{FOS}} \qquad \text{(Eq. 1-15)}$$

$$c'_r(\text{adjusted}) = \frac{c_r(\text{trial})}{\text{FOS}} \qquad \text{(Eq. 1-16)}$$

If extensive local experience with a particular material is available, that experience, as well as Equations 1-15 and 1-16, could be used to adjust the strength of some materials more than others. Note that, in general, ϕ'_r is fixed at a certain value and c'_r is varied based upon local experience to expedite the time involved in the back-analysis.

(4) The results of Step 3 can be verified by reanalyzing the slide using the newly calculated strengths. The final back-calculated strengths that produce a safety factor equal to unity are appropriate for the existing sliding surface, where the shear strength has been reduced to residual.

TABLE 5.6 Comparisons of Existing Empirical Correlations for Drained Residual Friction Angle

Solid Index Properties	Reference
Liquid limit and clay fraction	Stark and Eid (1994)
Clay fraction	Collotta et al. (1989) Lupini et al. (1981) Skempton (1964, 1985)
Plasticity index	Kanji (1974) Lambe (1985) Mitchell (1993) Voight (1973)
Liquid limit	Bishop et al. (1971) Mesri and Cepeda-Diaz (1986) Mitchell (1993)

- The proportion of platy particles to spherical particles in the soil and the coefficient of interparticle friction of the platy particles are factors that control the residual shear strength of the cohesive soils.
- Correlations between residual strength and soil index properties and/or gradation cannot be general and should be used judiciously.

Stark and Eid (1994) suggested that the drained residual strength failure envelope is nonlinear for cohesive soils with a clay fraction greater than 50 percent, and a liquid limit between 60 and 220 percent. They also proposed that the drained residual friction angle is a function of liquid limit, clay fraction, and effective normal stress, as shown in Figure 5.47. To model the effective stress-dependent behavior of the

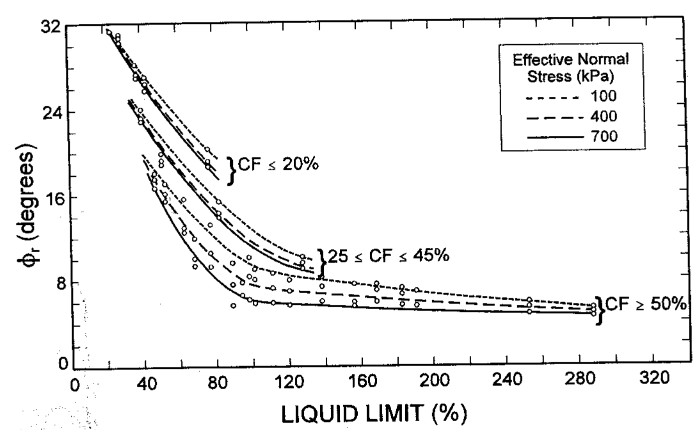

Figure 5.47 Empirical relationship between liquid limit and residual friction angle. (From Stark and Eid, 1994, reproduced by permission of ASCE.)

residual strength, Stark and Eid recommended that the nonlinear failure envelope or a secant residual friction angle corresponding to the average effective normal stress on the slip surface be used in a stability analysis.

5.9.3 Unsaturated Tests

Constant water and controlled suction tests are adequate for testing unsaturated soils. It is preferable to test unsaturated soils at their in situ moisture contents and in situ stress conditions. Dead load tests may be valuable for obtaining threshold suction (i.e., minimum suctions required for stability) and for studying the mechanism of failures that are initiated by infiltration.

Fredlund's stress state variables may be used to assign unsaturated strengths, which may be used directly for slope analysis using the *total cohesion* approach. To use this approach it is important to obtain the "correct" saturated ϕ' value at low confining pressures and the correct ϕ^b value from appropriate unsaturated tests. It must be noted that it is difficult to obtain the "corrected" saturated ϕ' value. Laboratory evidence (Howat and Shen, 1981) has shown that the conventional practice of back-saturation can destroy the fabric of an originally unsaturated soil specimen to such an extent that both dense and loose specimens can become medium dense. Typical values of ϕ^b for seven different soils were given earlier in Table 5.1, which also includes values of conventional c' and ϕ' Mohr–Coulomb parameters.

If it can be shown that suction is maintained in a slope after heavy rainfall, then it may be included in conventional stability analyses as an increase in apparent strength. If the variation of suction as a function of moisture content is high, as in the case of the volcanic and granitic residual soils in Hong Kong, then a small change in moisture content will result in a large change in suction. For such cases, it may be prudent to disregard the effects of suction for the slope stability analyses.

5.9.4 Selection of Design Shear Strengths

The in situ strength of the soil that will be used to evaluate the factor of safety (FOS) for a slope must selected carefully with full consideration given to the many complex facets that may have affected the laboratory determination of shear strength. Table 5.7 attempts to present a quantitative appraisal of the many features that may lead to an overestimation (unconservative) or an underestimation (conservative) of the in situ shear strength near a potential failure zone in a slope.

Johnson (1974) states:

> Evaluations of this type have little meaning unless they are done for a specific site and conditions, but even then required data are usually not available to permit reliable conclusions.

Thus it is desirable that the engineer consider each of the factors listed in Table 5.7 and assign a quantitative factor of confidence or uncertainty to each factor, as it may have influenced the reported laboratory test data.

TABLE 5.7 Factors Influencing Design Shear Strengths

Factor	Influence (percent)	Remarks
Sample disturbance of foundation materials (for relatively good, undisturbed samples)	−(5 to 20)	Remolding may increase strength of slickensided specimens. Disturbance is greatest for deep borings and soft soils
Effect of fissures in clays, especially highly overconsolidated clays and clay shales—effects not reflected in small samples	+(25 to 1,000)	Generally a factor for highly overconsolidated soils only
Rough caps and bases in laboratory tests	+5	
Triaxial compression tests instead of compression, simple shear, and extension tests	+(20 to 30)	Especially important for foundation soils
Triaxial instead of plane strain tests	−(5 to 8)	
Back-pressure saturation	Depends on field conditions	May cause grossly excessive strengths in CU tests at low confining stresses; conservative at high confining stresses
Conventional plotting of CU test data as total stress envelopes	−(15 to 20)	Effect may be eliminated by plotting data according to Taylor's method
Isotropic, instead of anisotropic, consolidation in CU triaxial compression tests		Values shown assume test envelopes for isotropic consolidation interpreted as τ_f versus of σ'_{fc}; i.e., as used by designers in stability analysis
(a) $A_f > \frac{1}{4}$	−(0 to 30)	
(b) $A_f < \frac{1}{4}$	+(0 to 20)	
Anisotropic material behavior—use of vertical instead of suitably inclined test samples	+(10 to 40)	

(continued)

TABLE 5.7 (*Continued*)

Factor	Influence (percent)	Remarks
Conventional rates of shear in the laboratory	+(5 to 200)	Effect depends on rate of testing, soil type, rate of consolidation in field, etc.
Progressive failure	+(0 to 20)	Depends on soil; mainly a factor for foundation soils. May be more serious than shown for some soils

Source: Johnson (1974), reproduced by permission of ASCE.

After evaluating the quality of the laboratory data, it is strongly recommended that the engineer compare the appraised shear strength values with available correlations and local experience. For granular soils, there are several correlations between field measurements such as SPT and CPT and drained strength. These have been discussed in the previous chapter on field exploration.

For fine-grained soils such as clays, the engineer may use the correlation between the plasticity index (PI) and the *peak* drained angle of internal friction shown in Figure 5.48. Alternatively, if presheared soils are being evaluated, Figure 5.47 and 5.49 may be used to correlate the PI with the residual angle of internal friction. Also, it should be noted that the cohesion component, c_r, will be negligible at large displacements where residual strength controls.

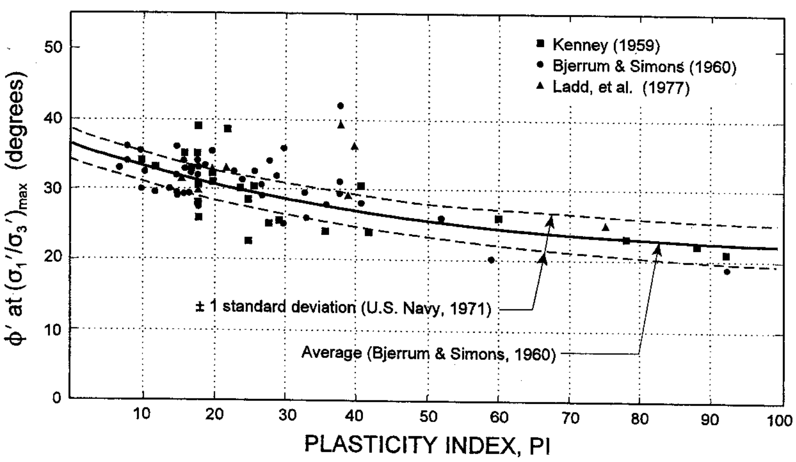

Figure 5.48 Correlation between plasticity index (PI) and angle of internal friction, ϕ'.

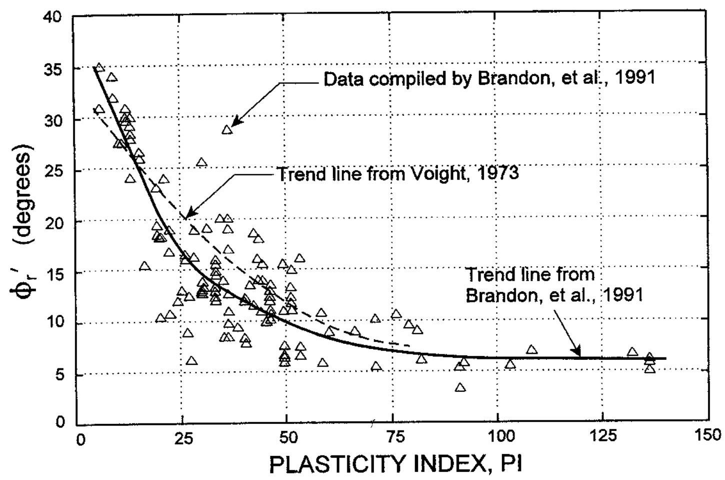

Figure 5.49 Correlation between plasticity index (PI) and the residual angle of friction, ϕ_r'. (From Filz et al., 1992.)

The undrained strength of a fine-grained soil will depend on the type of loading (i.e., compression or extension) and the overconsolidated ratio (OCR). For an OCR $\geq$ 4.0, the undrained strength in compression will be greater than the drained strength in the short-term, but this higher strength should not be used if the slope is expected to maintain long-term stability. Also, overconsolidated soils, will exhibit a cohesion value, c, below their preconsolidation stress. For normally consolidated soils, the undrained strength in compression will generally be about 50 percent of the drained strength, or $\phi_{cu} = \frac{1}{2}\phi'$.

As our database of undrained strengths grows, it has been found that for normally consolidated clays, the c_u/σ'_{vm} (or c_u/σ'_{vo}) is nearly a constant value equal to 0.23 ± 0.04 (Jamiolkowski et al., 1985; Mesri 1975). Typical values of the constants S and m, used by the SHANSEP approach, are given in Table 5.8.

If the engineer is reviewing laboratory test data for unsaturated soils, Table 5.1 may be used to assess the quality of the reported magnitude of the ϕ^b parameter. Admittedly, this table only includes data from seven different soils from eleven studies, but this database is likely to grow as more and more tests are performed for slopes that can be expected to remain unsaturated during their life.

5.10 OTHER PROPERTIES

In this section, several supplementary laboratory tests are reviewed on the basis of their indirect influence within the overall framework of slope stability and practical geotechnical engineering.

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CHAPTER 10

STABILITY OF LANDFILL SLOPES

10.1 UNIQUE NATURE OF LANDFILLS

Landfills pose unique slope stability issues chiefly because construction involves different combinations of cuts and fills in a variety of materials including soil, bedrock, water, landfill waste, and geosynthetics. There are three critical periods in the life of the landfill when slope stability is considered:

- Siting and preparation to receive waste
- Waste placement
- Capping and final closure

Initially, when the landfill is being sited and prepared to receive waste, the stability of the host site materials must be analyzed for cut and fill behavior depending on whether the landfill is being placed in a pit, in trenches, against a hillside, in a canyon, or above ground. As the waste is placed in the landfill, temporary fill slopes are constantly being created as different sections of the landfill are filled and then buttressed by additional filling. Finally, when the landfill is full, a cap is placed above the waste and the landfill is closed. Each of these phases presents unique combinations of materials in contact with each other, unique combinations of material and strength properties, and thus, unique slope stability issues.

10.2 TYPICAL LANDFILL CONFIGURATIONS

Five typical landfill configurations are shown in Figure 10.1. The most common landfill configuration in areas of deeper water table is probably the covered pit design

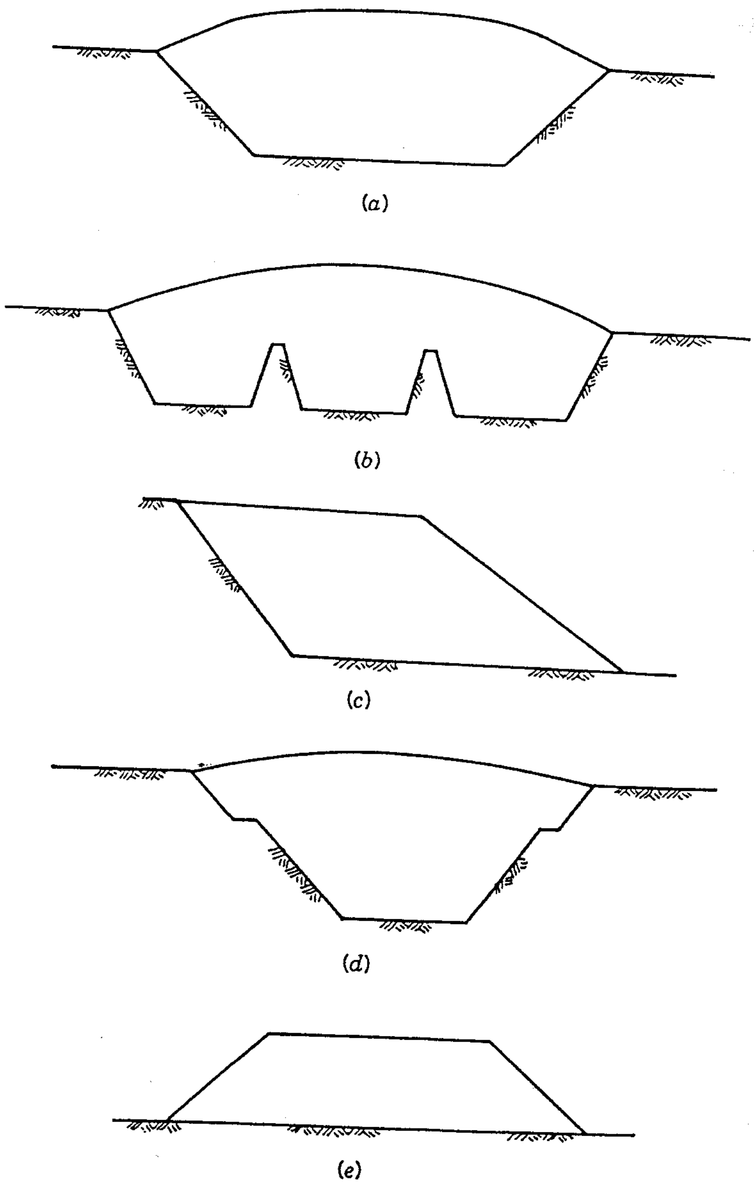

Figure 10.1 Typical landfill configurations. (Sharma and Lewis, 1994.)

shown in Figure 10.1*a*. The advantage of this configuration is that soils excavated from the pit may be used for daily cover. Depending on their nature, these excavated soils may also be suitable for clay liners or granular drainage material. The trenched landfill shown in Figure 10.1*b* has advantages similar to those of the covered pit. It is, however, typical of older landfilling practices, prior to the emphasis in landfill design

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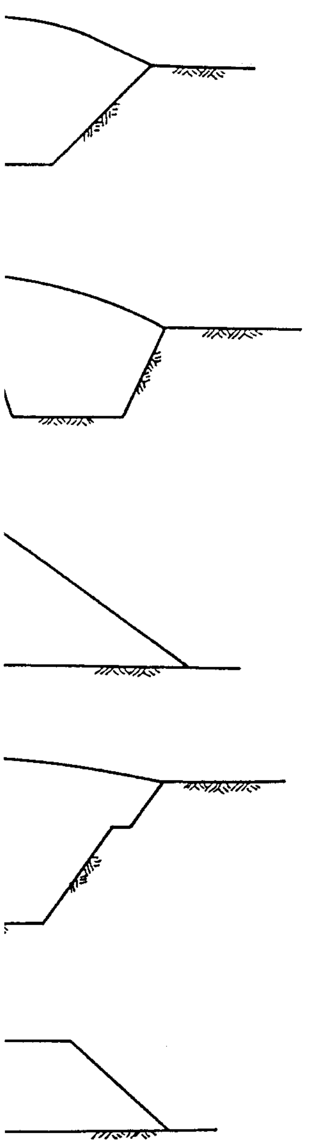

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on maximizing airspace. As is evident in Figure 10.1*b*, significant airspace is lost to the ridges of native soil remaining between cells.

Figure 10.1*c* depicts an upslope landfill. These types of landfills are generally constructed in regions of rolling or hilly terrains. The waste is generally placed on a liner constructed on an excavated natural slope. The canyon or valley landfills (Figure 10.1*d*) are so named because they are typically constructed in natural canyons or valleys. They can be similar to covered pits if the canyon is bowl-shaped; however the boundary side slopes are generally much higher and often steeper. For both upslope and canyon landfills, soil requirements are often satisfied by excavating the natural side slopes, selecting an on-site borrow area, or importing soil. Finally, as shown on Figure 10.1*e*, above-ground landfills are constructed entirely above the natural ground surface. These landfills typically exist in soft subgrade regions, where groundwater is near the ground surface, or where it would be impractical to excavate a pit. Operational soil requirements for these landfills are satisfied by on-site borrow areas or off-site soil import.

A landfill site must meet several locational and geotechnical design criteria and be acceptable to the public. It must be in reasonable proximity to waste generators. The type of site, whether hilly, flat, or whatever, will dictate the configurations that are possible at each site. Data used in the selection of a site often include:

- Topographic maps
- Soil survey maps
- Land use plans
- Transportation maps
- Water use plans
- Flood plain maps
- Geologic maps
- Aerial photographs

Other siting considerations include waste type, waste volume, landfill volume, availability of landfill equipment, recycling and incineration options, existing landfill sites, and funding (Bagchi, 1994). No landfills should be constructed near lakes, ponds, rivers, flood plains, highways, public parks, critical habitat areas, wetlands, airports, or water wells. In fact, a landfill will be an integral part of the landscape and environment (Figure 10.2) and must be evaluated in that context. In analyzing the feasibility of a landfill, the entire life cycle costs of the landfill must be evaluated, including the items shown in Table 10.1.

The landfill waste materials can be placed in a variety of ways, as shown in Figure 10.3. All wastes received by the landfill are spread and compacted in cells and layers within a confined area. At the end of each working day, or more frequently if needed, the area is covered completely with a thin, continuous layer of daily cover material that is then compacted. Daily cover is used to control moisture, control litter, reduce odors, limit rodent and bird contact, provide vehicle access, help prevent fires,

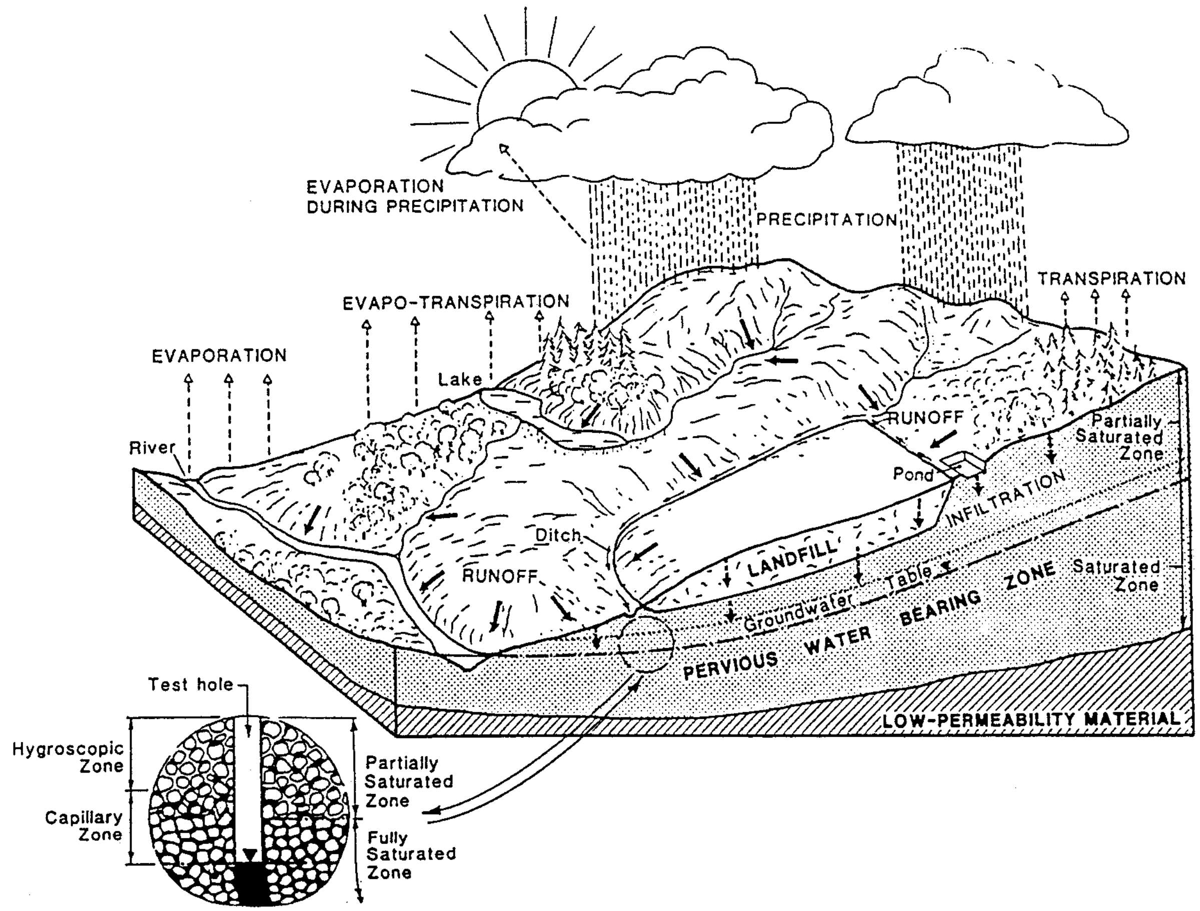

Figure 10.2 Waste storage within the hydrologic cycle. (Sharma and Lewis, 1994.)

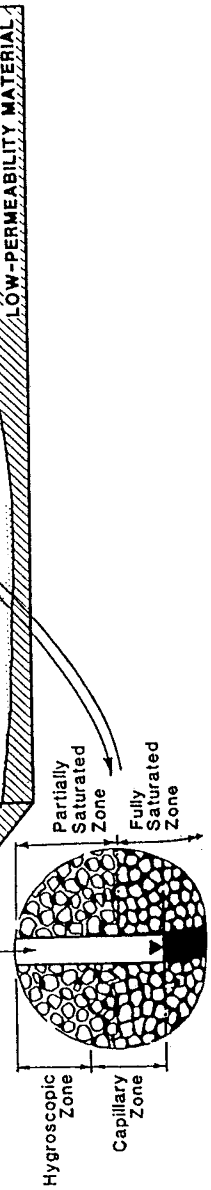

Figure 10.2 Waste storage within the hydrologic cycle. (Sharma and Lewis, 1994.)

TABLE 10.1 Components of Landfill Cost

Predevelopment Costs
Site characterization
Environmental assessment
Engineering design
Hydrogeologic investigation
Professional service fees—design/approvals
Legal consultation
Construction Costs
Land clearing
Excavation
Liner and leachate collection system installation
Leachate management—pumping station and/or treatment systems
Surface water control and final cover construction
Gas management system
Groundwater monitoring systems
Site structures
Operations
Equipment and personnel
Leachate and landfill gas management
Environmental monitoring costs
Community relations
Impact management—dust, odors, and birds
Closure costs
Cap/final cover
Seeding
Runoff control
Long-Term Care Costs
End use plan costs, including trees and shrubs
Site inspections
Land service care
Leachate and gas management
Environmental monitoring
Insurance

Source: McBean et al., 1995.

and improve the appearance of the landfill. The compacted wastes and daily cover material constitute a cell. A series of adjoining cells, all of the same height, make up a lift. The completed landfill consists of one or more lifts, as depicted in Figure 10.4. The two basic methods of operation for landfilling, or sequencing, of the daily cells are the trench method and the area method (Figures 10.5 and 10.6).

After the landfill is completely filled, a cover must be constructed to minimize water infiltration and isolate the waste from the environment, as shown in Figure 10.7. The purposes of the final soil cover over a landfill are to encourage surface water runoff, discourage erosion, retain moisture for vegetation, manage gas migration, provide shaping and contouring, and provide a base for the establishment of a suitable ground cover. Six typical layers of a final landfill cover are shown in Figure 10.8.

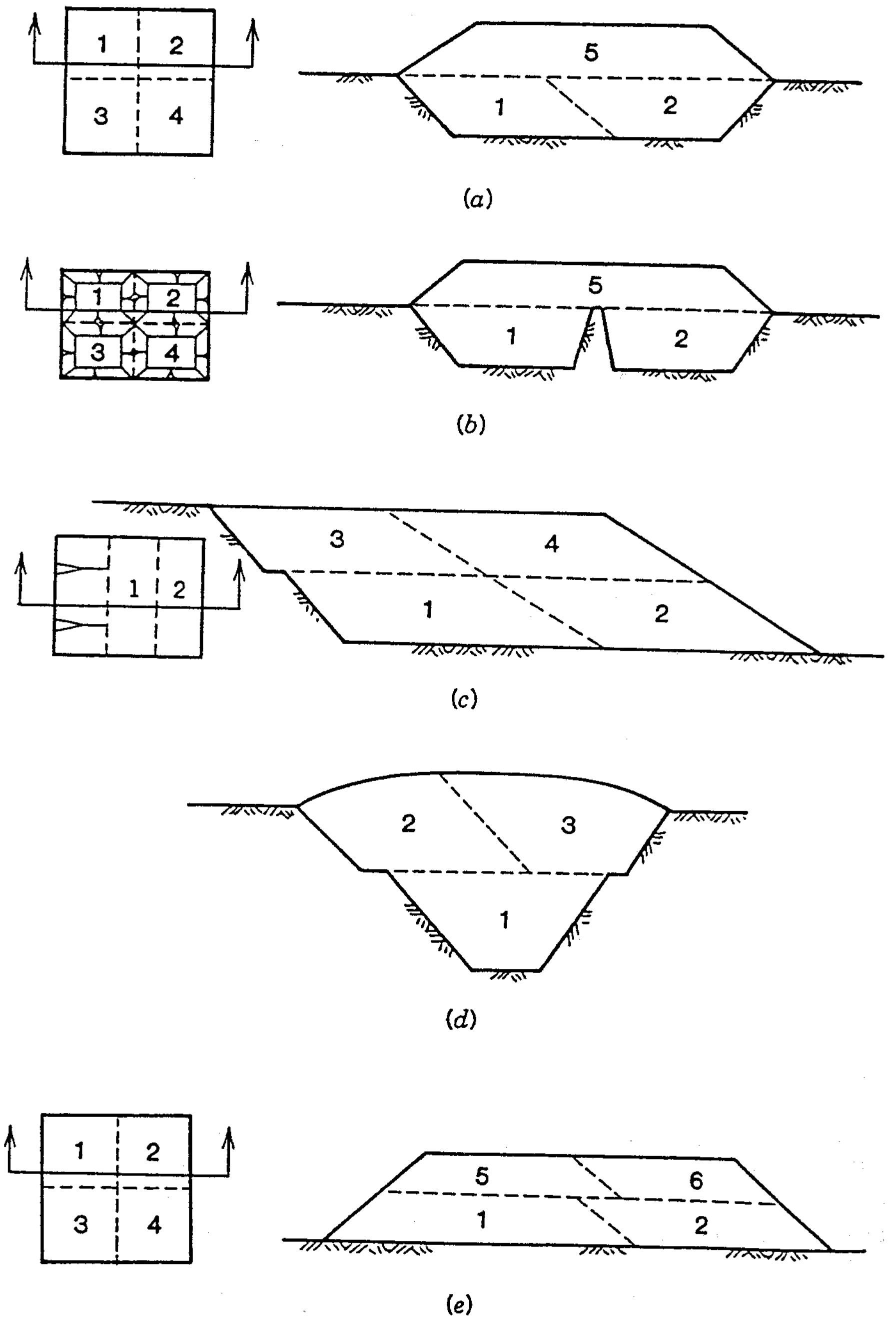

Figure 10.3 Landfill waste placement phases. (Sharma and Lewis, 1994.)

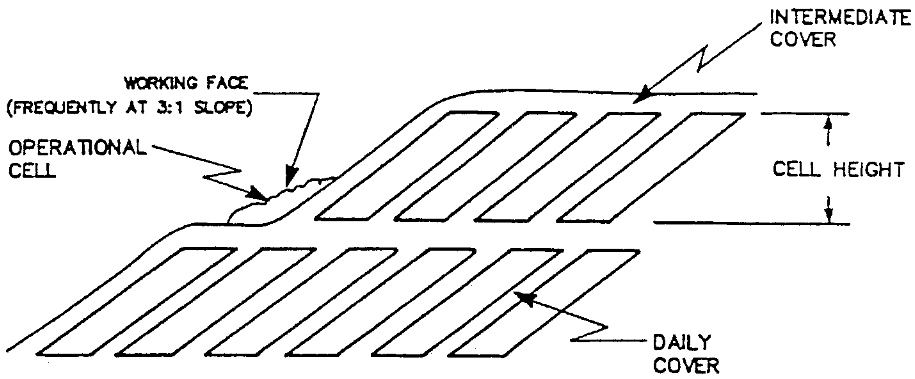

Figure 10.4 Configuration of landfill waste lifts and cells. (McBean et al., 1995.)

10.3 LANDFILL WASTE ENGINEERING PROPERTIES

The engineering properties of landfill waste are quite variable and unpredictable. Landfill waste may contain a multitude of materials including food, rubber, plastic, leather, textiles, wood, furniture, paper, metal, glass, machinery, transportation

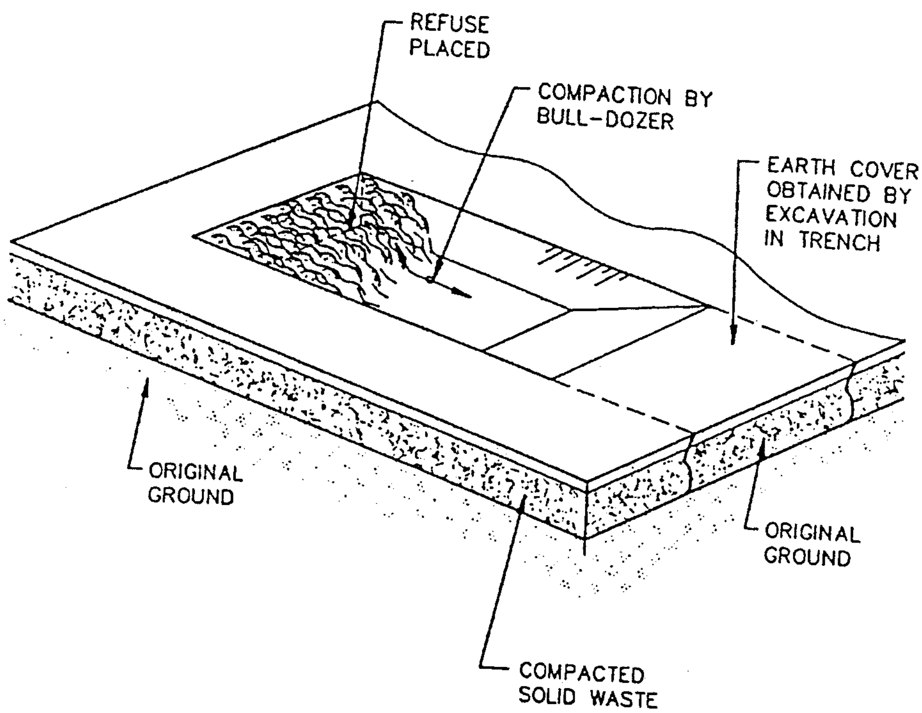

Figure 10.5 Trench method of landfilling. (McBean et al., 1995.)

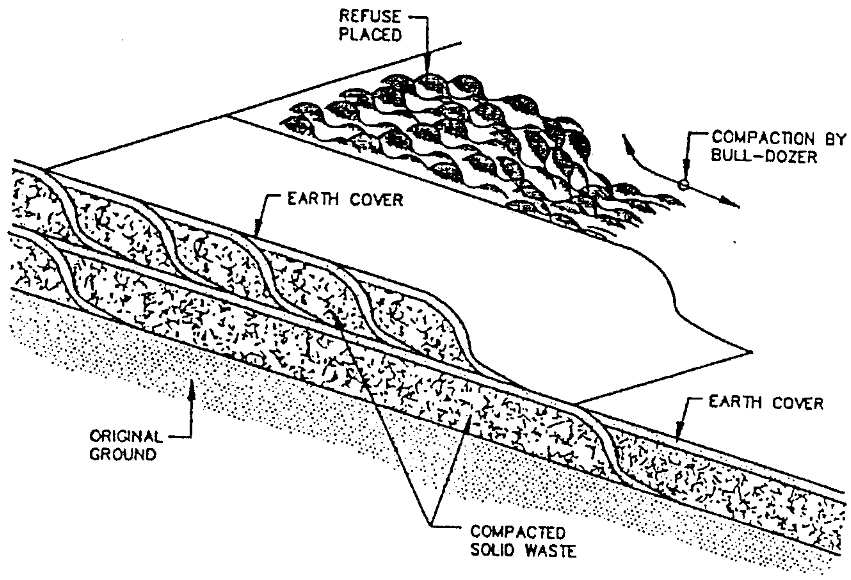

Figure 10.6 Area method of landfilling. (McBean et al., 1995.)

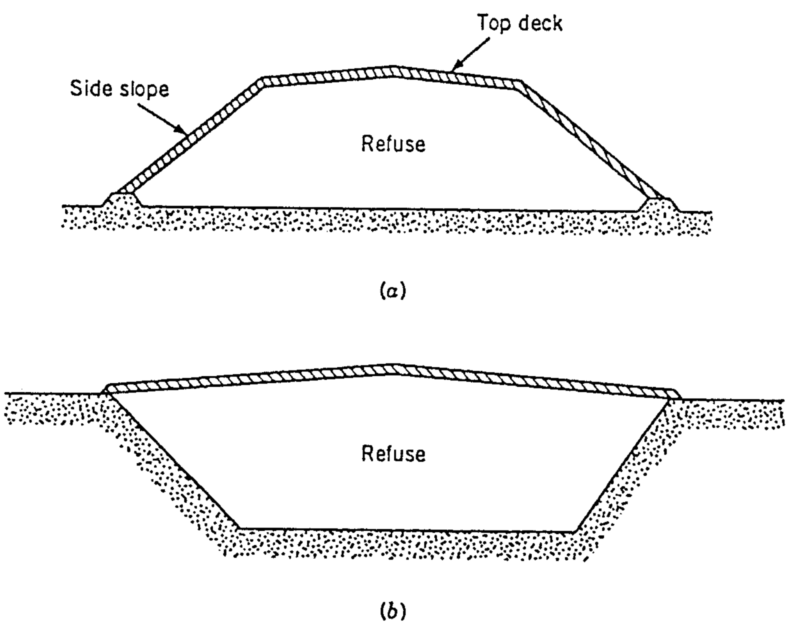

Figure 10.7 Landfill final cover configurations. (Sharma and Lewis, 1994.)

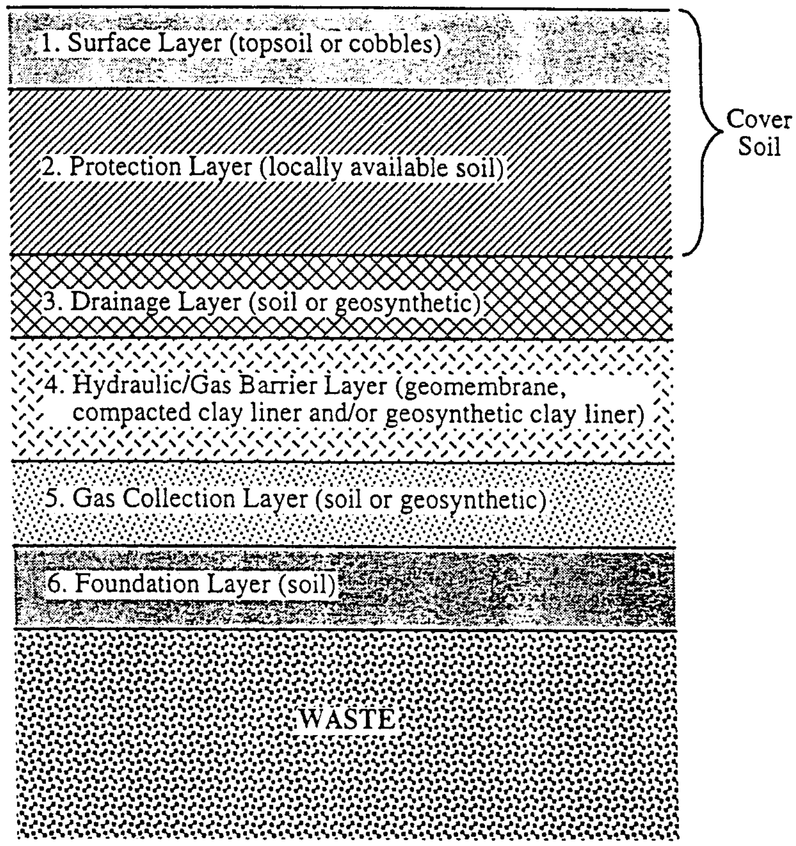

Figure 10.8 Typical layers of landfill final covers. (Koerner and Daniel, 1997.)

equipment, electrical materials, petroleum, coal, chemicals, and other manufacturing products. While conventional geotechnical exploration tools can be used to study and evaluate them, no standard test methods are available. Yet, to carry out slope stability analyses involving these peculiar materials, the engineer must make some assumptions and develop the suite of required material properties and strength values.

There are a multitude of properties related to landfill waste that could be discussed including grain size distribution, porosity, moisture content, hydraulic conductivity, Atterberg limits, unit weight, strength, dynamic properties, and compressibility. Additionally, chemical and leach properties of landfill waste must be known to determine the appropriate disposal locations and methods. The properties most germane to slope stability analyses are unit weight and strength. Some average values for unit weight are given in Table 10.2. Strength properties are often expressed in a graph, as shown in Figure 10.9. The strength of wastes from industrial facilities such as power plants and other manufacturing facilities has received study and some examples are

TABLE 10.2 Average Unit Weights for Landfill Waste

Source	Refuse Placement Conditions	Unit Weight kg/m³	Unit Weight lb/ft³
U.S. Department of the Navy (1983)	Sanitary landfill		
	Not shredded		
	Poor compaction	320	20
	Good compaction	641	40
	Best compaction	961	60
	Shredded	881	55
Sowers (1968)	Sanitary refuse: depending on compaction effort	481–961	30–60
NSWMA (1985)	Municipal refuse:		
	In a landfill	705–769	44–49
	After degradation and settlement	1,009–1,121	63–70
Landva and Clark (1986)[a]	Refuse landfill (refuse to soil cover ratio varied from about 2–1 to 10–1)	913–1,346	57–84
EMCON Associates (1989)[b]	For 6–1 refuse to daily cover soil	737	46

Source: Sharma et al. (1990).

[a]These values were obtained from test-pit measurements of refuse at 11 municipal landfills in Canada. Values measured for the Halifax landfill and the August 1983 measurements at the Edmonton and Calgary landfills have not been included, as suggested by the authors.

[b]Based on tonnage records and areal survey maps recorded during the period from April 1988 through April 1989.

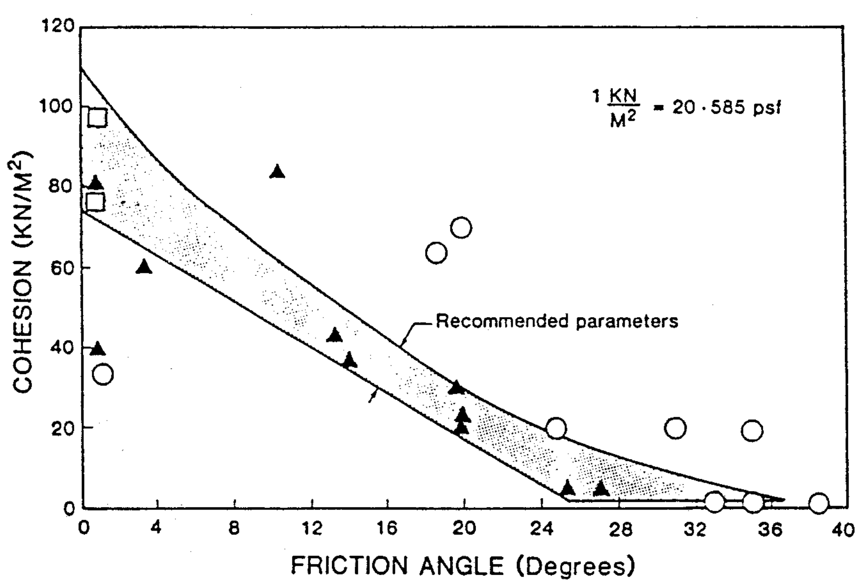

Figure 10.9 Typical shear strength values for landfill waste. (Singh and Murphy, 1990.)

TABLE 10.3 Strength Properties of Mineral Landfill Waste

Description	Cohesion, c (kPa)	Friction Angle, ϕ (degrees)	Undrained Compressive Strength, q_m (kPa)	Water Content, w (%)
Coal refuse				
Undrained condition	10–40	10–28		
Effective stress	0–40	25–43		
Fly ash, Arizona 7-day				
Unit wt. 12.6 kN/m^3			223	
Unit wt. 13.4 kN/m^3			331	
Unit wt. 13.8 kN/m^3			587	
Fly ash (silica 46%, aluminum 34%, calcium 7%)				
Slurry samples	0	37		
Compacted, undrained effective stress	0	41		
Compacted, drained	0	37		
West Virginia fly ash				
Shelby tube samples (consolidated, undrained)		0	34	
Shelby tube samples (consolidated, drained)	0	37.5		
West Virginia bottom ash		38–43		
Flue-gas desulfurization (FGD) sludge				
Consolidated, drained test	0	41.5		
Compacted	0–40	10–40		
Red mud (bauxite residue)				
Unleached			63	52
Leached			0	49
Mud: sand (5:1)			38	41

Source: Oweis and Khera (1990).

presented in Table 10.3. For slope stability analyses, strength values for landfill waste of $c = 400$ pounds per square foot and $\phi = 20°$ is a good starting point (Sharma and Lewis, 1994). Also, of great importance is interface friction angles between waste and other construction materials such as clay caps, geosynthetics, and so on.

10.4 GEOSYNTHETICS IN LANDFILLS AND ENGINEERING PROPERTIES

Fifty to one hundred years ago, landfills did not require linings. Waste was dumped directly onto the native ground and then covered and vegetated as well as possible.

Environmental regulation now requires very specific multiple-layer redundant lining and cover systems (Figure 10.10). Concurrent with stiffer environmental regulation of landfills, the use of geosynthetic materials has forged its way into the geotechnical engineering community including usage at landfills. Geosynthetic materials are materials, mostly plastic, which are commonly used in place of, or to enhance the function of, natural soil materials. Landfill lining and cover systems generally require a variety of geosynthetic materials including geomembranes, geotextiles, geonets, geogrids, and geosynthetic clay liners (GCLs).

10.4.1 Geomembranes

Geomembranes are flexible, polymeric sheets that have extremely low permeability and are typically used as liquid or vapor barriers (Figure 10.11). Table 10.4 lists the major types of geomembranes in current use. Advantages and disadvantages of some commonly used geomembranes are provided in Table 10.5. In landfills, base liners are placed below waste to minimize liquids expelled from and/or filtered through the waste (known as leachate) from contaminating the underlying ground and most important, the groundwater. Cover liners are placed above the final waste configuration to keep water, usually from rain or snow, from entering the waste and producing leachate.

10.4.2 Geotextiles

Geotextiles are synthetic fabrics used in geotechnical engineering for various applications. The majority of geotextiles are composed of polypropylene or polyester fibers; a small percentage are composed of polyamide or polyethylene (Sharma and Lewis, 1994). Geotextiles are manufactured from monofilament, staple, or slit film fibers that are twisted or spun together into yarn. The fibers or yarns are formed into geotextiles using either woven or nonwoven methods. Woven geotextiles are formed using traditional weaving methods and a variety of weave types (Figure 10.12). To create nonwoven geotextiles, the manufactured fibers are placed and oriented on a moving conveyor belt. Needle punching, melt bonding, or resin bonding bonds the fibers. At landfills, nonwoven geotextiles are most commonly used for filtration, separation, cushioning, and drainage. Woven geotextiles are usually used for reinforcement. Both types can be used as an alternative for daily cover.

10.4.3 Geonets

Geonets are used for drainage and consist of two sets of parallel solid or foamed extruded ribs that intersect at a constant angle to form an open net configuration (Figure 10.13). Channels are formed between the ribs to convey either liquid or gases. To prevent intrusion by soils or other adjacent materials in the field, a geocomposite drainage net may be used that consists of a geotextile bonded to the geonet.

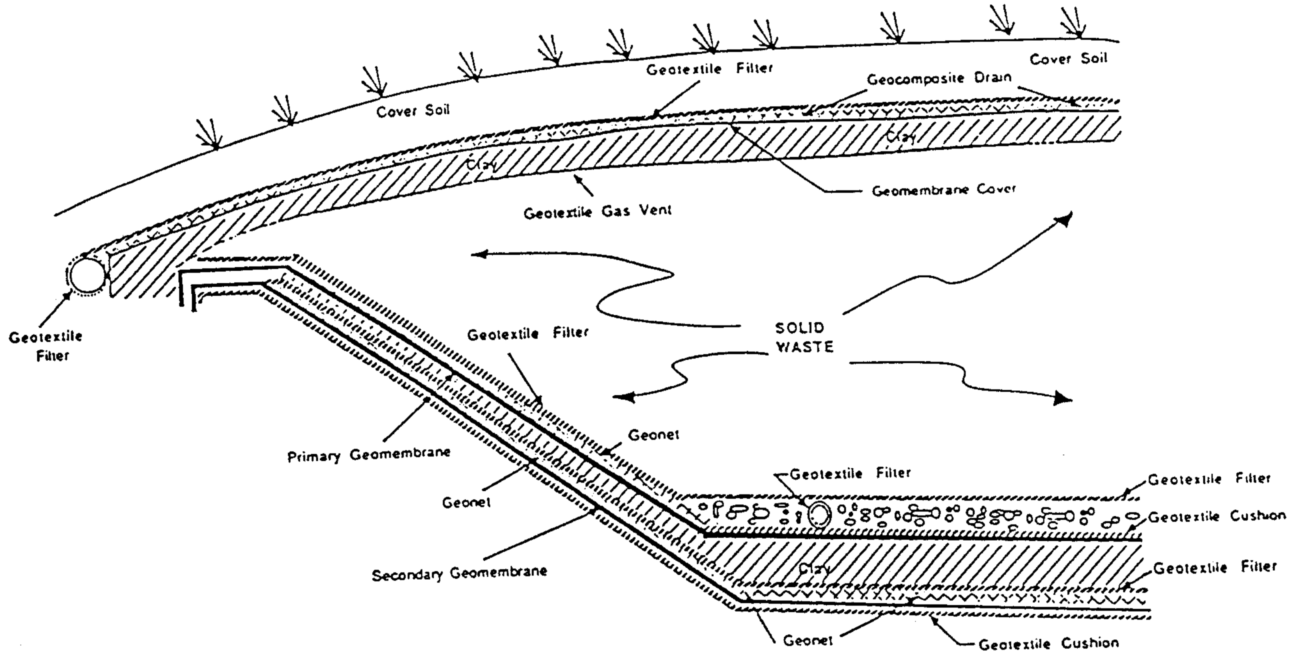

Figure 10.10 Typical landfill liner and cover system. (Koerner and Koerner, 1990.)

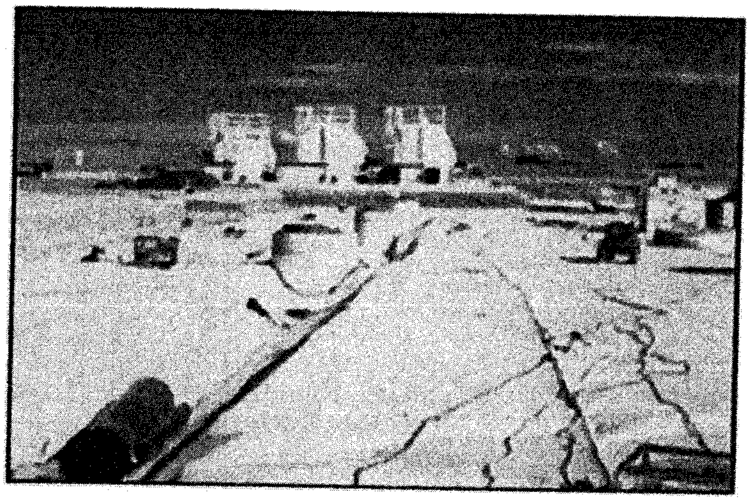

Figure 10.11 Geomembrances.

10.4.4 Geogrids

Geogrids are high-strength, soil reinforcement products composed of polypropylene, polyethylene, polyester, or PVC-coated polyester. All geogrids have an open mesh configuration with apertures ranging from $\frac{1}{2}$ to 3 inches (Figures 10.14 and 10.15). They are formed by several different methods. The polyester and polyester PVC-coated geogrids are typically woven or knitted. The polypropylene geogrids are either

TABLE 10.4 Major Types of Geomembranes in Current Use

Thermoplastic Polymers	Thermoset Polymers	Combinations
Polyvinyl chloride (PVC)	Butyl or isoprene-isobutylene (IIR)	PVC–nitrile rubber
Polyethylene (VLDPE, LLDPE, MDPE, HDPE, referring to very low, linear low, medium, and high density)	Epichlorohydin rubber	PE–EPDM
Chlorinated polyethylene (CPE)	Ethylene propylene diene monomer (EPDM)	PVC–ethyl vinyl acetate
Elasticized polyolefin (3110)	Polychloroprene (neoprene)	Cross-linked CPE
Ethylene interpolymer alloy (EIA or XR-5)	Ethylene propylene terpolymer (EPT)	Chlorosulfonated polyethylene (CSPE or Hypalon)
Polyamide	Ethylene vinyl acetate (EVA)	

Source: Koerner (1990).

TABLE 10.5 Advantages and Disadvantages of Commonly Used Synthetic Membranes

Synthetic Membrane	Advantages/Disadvantages
Butyl rubber	Good resistance to ultraviolet (UV) ray, ozone, and weathering elements
	Good performance at high and low temperatures
	Low swelling in water
	Low strength characteristics
	Low resistance to hydrocarbons
	Difficult to seam
Chlorinated polyethylene (CPE)	Good resistance to UV, ozone, and weather elements
	Good performance at low temperatures
	Good strength characteristics
	Easy to seam
	Poor resistance to chemicals, acids, and oils
	Poor seam quality
Chlorosulfonated polyethylene	Good resistance to UV, ozone, and weather elements
	Good performance at low temperatures
	Good resistance to chemicals, acids, and oils
	Good resistance to bacteria
	Low strength characteristics
	Problem during seaming
Ethylene-propylene rubber (EPDM)	Good resistance to UV, ozone, and weather elements
	High strength characteristics
	Good performance at low temperatures
	Low water absorbance
	Poor resistance to oils, hydrocarbons, and solvents
	Poor seam quality
Low-density and high-density polyethylene (LDPE and HDPE)	Good resistance to most chemicals
	Good strength and seam characteristics
	Good performance at low temperatures
	Poor puncture resistance
Polyvinyl chloride (PVC)	Good workability
	High strength characteristics
	Easy to seam
	Poor resistance to UV, ozone, sulfide, and weather elements
	Poor performance at high and low temperatures

Source: Bagchi (1994).

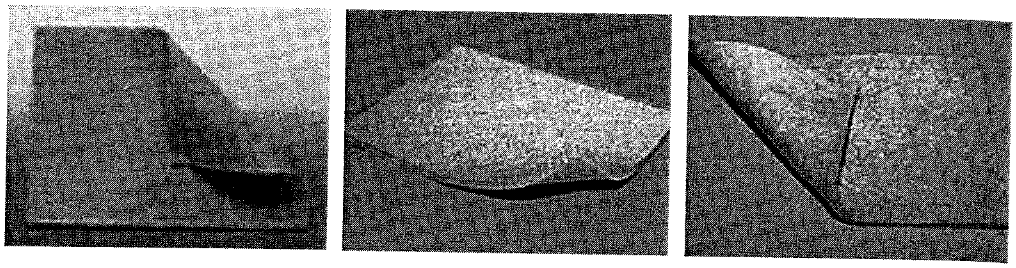

Figure 10.12 Geotextiles.

extruded or punched sheet drawn, and polyethylene geogrids are exclusively punched sheet drawn. At landfills, geogrids may be used to support a lining system over a weak subgrade or to support final landfill cover soils on steep refuse slopes. They are also used sometimes in "piggyback" landfills between the old landfill and the new one. Typical ranges of tensile strengths for geogrids made of different materials are listed in Table 10.6.

10.4.5 Geosynthetic Clay Liners (GCLs)

Geosynthetic clay liners (GCLs) are very low permeability barriers consisting of a layer of unhydrated, loose granular or powdered bentonite that is chemically or mechanically adhered to a geotextile or geomembrane. They are generally used as an alternative to compacted clay liners. Differences between GCLs and compacted clay liners are presented in Table 10.7.

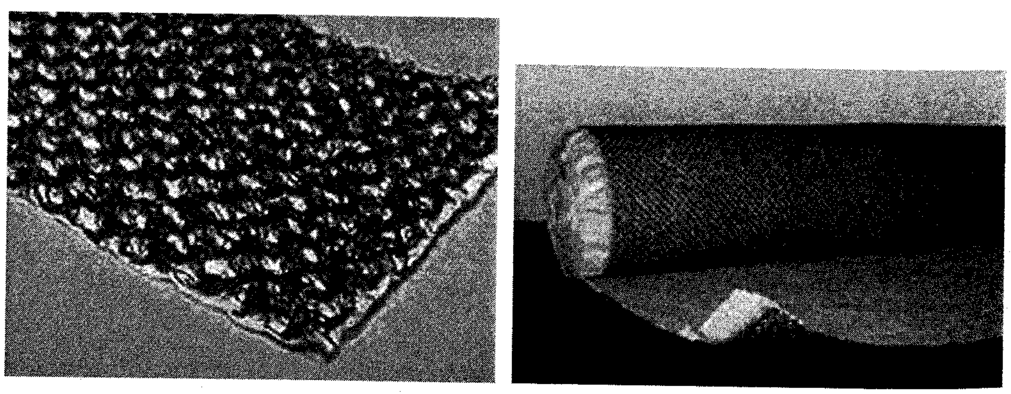

Figure 10.13 Geonets.

Figure 10.14 Geogrid.

10.4.6 Engineering Properties of Geosynthetics

There are a multitude of engineering properties that are of interest when using geosynthetics in landfills. Many of these deal with the ability of geosynthetics to perform their intended function as a barrier or pathway for liquid and gas migration. Many of the material properties of concern are listed in Table 10.8. However, from a slope stability standpoint, the frictional behavior of these materials and between these materials and other materials is of paramount importance. This property is known as interface friction. The possible interfaces that may be of interest in a landfill slope stability analysis are:

- Soil/geotextile
- Soil/geomembrane

Figure 10.15 Geogrid. (Martin, 1998.)

TABLE 10.6 Typical Range of Wide Width Geogrid Strengths in Maximum Strength Direction

Geogrid	Tensile Strength, 5% Strain (lb/in.)	Ultimate Tensile Strength (lb/in.)
Polyester	750–2,300	2,600–8,500
Polyester, PVC coated	600–7,600	1,500–25,400
Polypropylene	50–110	70–190
Polyethylene	140–460	300–810

Source: Sharma and Lewis (1994).

- Geosynthetic/geosynthetic
- Geosynthetic/clay liner

Interface friction angles can be measured in the laboratory using ASTM D 5321. Using this method, each of the combinations of materials can be tested. There is also an abundant amount of information available in the literature, as shown in Tables 10.9, 10.10, 10.11, and 10.12.

10.4.7 Anchor Trenches

In addition to selecting the appropriate types and combinations of geosynthetics, anchoring details must be specified to hold the geosynthetics in place and prevent

TABLE 10.7 Differences between GCLs and Compacted Clay Liners

Characteristic	Geosynthetic Clay Liner	Compacted Clay Liner
Materials	Bentonite clay, adhesives, geotextiles, and geomembranes	Native soils or blend of soil and bentonite
Construction	Manufactured and then installed in the field	Constructed in the field
Thickness	Approximately 10 mm	Approximately 0.5 to 1.0 m
Hydraulic conductivity of clay	10^{-10} to 10^{-8} cm/s (typical)	10^{-8} to 10^{-7} cm/s (typical)
Speed and case of construction	Rapid, simple installation	Slow, complicated construction
Water content at time of construction	Essentially dry; cannot desiccate during construction and produces no consolidation water	Nearly saturated; can desiccate and can produce consolidation water
Cost	\$5 to \$11 per square meter	Highly variable (estimated range: \$8 to \$32 per square meter)
Experience level	Limited due to newness	Has been used for many decades

Source: USEPA (1993).

in Maximum

Ultimate Tensile Strength (lb/in.)
2,600–8,500
1,500–25,400
70–190
300–810

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eter)
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TABLE 10.8 Importance of Geosynthetic Material Properties

Geosynthetic	Main Material Properties of Concern	Common Test Methods
Geomembranes	Thickness	ASTM D 374, D 751, D 1593
	Tensile behavior	ASTM D 412, D 638, D 882, D 4885
	Tear resistance	ASTM D 1004
	Punch resistance	FTMS 101C
	Chemical resistance	USEPA 9090
	Seam strength	ASTM D 4437
Geotextiles	Thickness	ASTM D 177, D 5199
	Tensile behavior	ASTM D 4632, D 4595
	Bursting behavior	ASTM D 3786
	Tear behavior	ASTM D 4533
	Apparent opening size	ASTM D 4751
	Permittivity	ASTM D 4491
	Transmissivity	ASTM D 4716
	Seam strength	ASTM D 1683, D 4884
Geonets	Thickness	ASTM D 1777, D 5199
	Crush strength	ASTM D 1621
	Transmissivity	ASTM D 4716
Geogrids	Tensile strength (Also see geomembranes and geotextiles)	ASTM D 4595
Geosynthetic clay liners (GCLs)	(Address each composite material separately including standard tests on bentonite materials)	

tearing. As shown in Figure 10.16, anchor trenches can generally be classified as flat, rectangular, or V-shaped. Selection of the appropriate anchor trench configuration for any particular site depends on the required holding capacity, access considerations, dimensional constraints, and available construction equipment. The holding capacity of anchor trenches is developed by the applied normal load of the soil placed above the geosynthetics that creates frictional resistance between the geosynthetics and the

TABLE 10.9 Typical Range of Reported Soil Geotextile Friction Angles

Geotextile	Sand Friction Angle (degree) (Efficiency)	Clay Friction Angle (degree) (Efficiency)
Woven	23–42 (0.68–1.0)	16–26 (0.61–0.93)
Nonwoven, needle-punched	25–44 (0.67–1.0)	15–28 (0.62–0.99)
Nonwoven, resin- or heat-bonded	22–40 (0.56–0.91)	17–33 (0.60–0.85)

Source: Sharma and Lewis (1994).

TABLE 10.10 Typical Range of Reported and Recommended Soil Geomembrane Friction Angles

Geomembrane	Reported Sand Friction Angles (degree) (Efficiency)	Recommended Sand Friction Angles, δ (degree)	Reported Clay Friction Angles (degree) (Efficiency)	Recommended Clay Friction Angles, δ (degree)
PVC	21–33 (0.62–0.93)	20–30	6–39 (0.53–1.0)	6–15
HDPE	17–28 (0.45–0.81)	17–25	5–29 (0.47–0.88)	5–10
Textured HDPE	30–45 (0.86–1.0)	30–40	7–35 (0.70–1.0)	9–15
VLDPE[a]	21–28 (0.62–0.67)	—	—	—

Source: Sharma and Lewis (1994).

[a] Since VLDPE is a relatively new product, limited results were reported in the literature. It is anticipated that the range of efficiencies for VLDPE to sand interfaces is broader than shown. Blank (—) means insufficient data at this time.

underlying soil. There is minimal friction resistance developed between the upper soil and the geosynthetic since the soil above the geosynthetic is likely to move with the geosynthetic. The soil depth, type of soil or other material underlying the geosynthetics, and geosynthetic anchorage length are therefore the key factors in developing the required anchor trench holding capacity. Sharma and Lewis (1994) provide additional information and design formulas for anchor trench design.

10.5 LANDFILL CONSTRUCTION

The construction of a landfill is a carefully planned, methodical, highly regulated, and dynamic process. Substantial planning, study, and design must take place before a landfill can be sited, opened, and/or modified. Additionally, constant monitoring, testing, and reporting are required throughout the life of the landfill. More relevant

TABLE 10.11 Typical Range of Reported Geosynthetic to Geosynthetic Friction Angles in Degrees

	PVC	HDPE Smooth	HDPE Textured	Geonet
Woven geotextile	10–28	7–11	9–17	9–18
Nonwoven, needle-punched geotextile	16–26	8–12	15–33	10–27
Nonwoven, resin/heat-bonded geotextile	18–21	9–11	15–16	17–21
Geonet	11–24	5–19	7–25	—

Source: Sharma and Lewis (1994).

TABLE 10.12 Direct Shear Test Results under Variable Hydrating Conditions

Hydrating Fluid	GCL Type[a]	Measured Property	Dry[b]	Constrained Swell[c]	Free Swell[d]
Distilled water	Claymax	Ø (deg)	37	16	0
		c (kPa)	6.9	3	4
	Gundseal	Ø (deg)	26	19	0
		c (kPa)	50	5	3
	Bentomat	Ø (deg)	42	37	23
		c (kPa)	14	6	5
	Bentofix	Ø (deg)	36	31	10
		c (kPa)	68	7	9.0
Tap water	Claymax	Ø (deg)	37	18	0
		c (kPa)	6.9	3	3
	Gundseal	Ø (deg)	26	18	0
		c (kPa)	50	5	3
	Bentomat	Ø (deg)	42	43	26
		c (kPa)	14	6	10
	Bentofix	Ø (deg)	36	34	15
		c (kPa)	68	6.9	7
Mild leachate	Claymax	Ø (deg)	37	24	4
		c (kPa)	6.9	6	3
	Gundseal	Ø (deg)	26	18	13
		c (kPa)	50	5	4
	Bentomat	Ø (deg)	42	39	25
		c (kPa)	14	8.3	14
	Bentofix	Ø (deg)	36	43	20
		c (kPa)	68	5	12
Harsh leachate	Claymax	Ø (deg)	37	19	0
		c (kPa)	6.9	6	3
	Gundseal	Ø (deg)	26	13	0
		c (kPa)	50	7.6	3
	Bentomat	Ø (deg)	42	45	32
		c (kPa)	14	5	12
	Bentofix	Ø (deg)	36	39	30
		c (kPa)	68	4	8.3
Diesel fuel	Claymax	Ø (deg)	37	44	38
		c (kPa)	6.9	4	6
	Gundseal	Ø (deg)	26	24	29
		c (kPa)	50	4	6
	Bentomat	Ø (deg)	42	42	40
		c (kPa)	14	6	5
	Bentofix	Ø (deg)	36	51	46
		c (kPa)	68	4	5

Source: USEPA (1993).

[a]Claymax and Gundseal are unreinforced GCLs; Bentomat and Bentofix are reinforced. Claymax also manufactures a reinforced GCL, but it was not used in this testing program.

[b]Dry refers to product as-received, placed under desired normal stress, than sheared at midplane.

[c]Constrained swell refers to product hydrated under desired normal stress, (i.e., constrained swell) then sheared at midplane.

[d]Free swell refers to product hydrated under zero normal stress, then placed under desired normal stress, and then sheared at midplane.

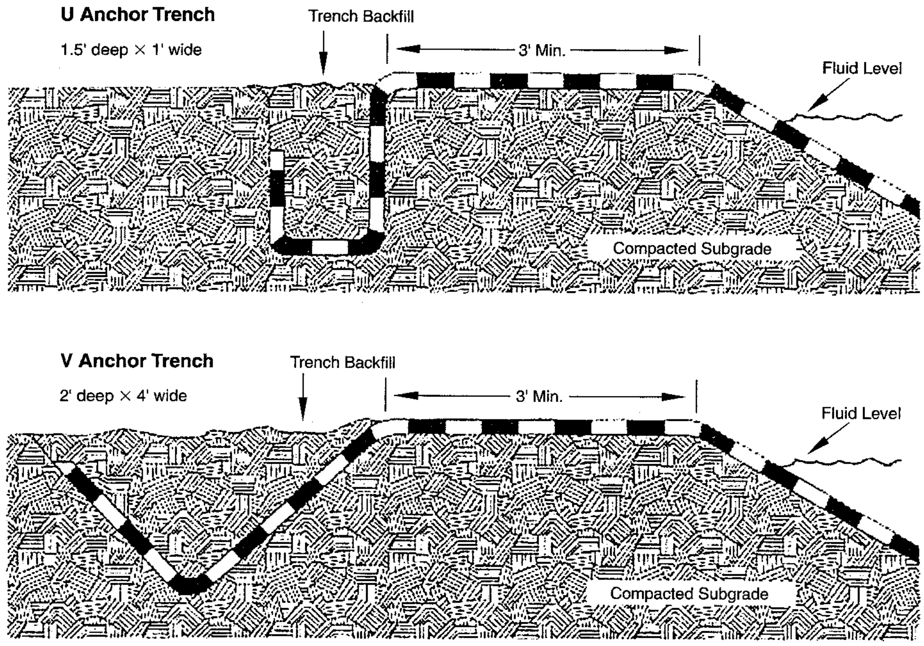

Figure 10.16 Typical anchor trench configurations. (Martin, 1998.)

to slope stability issues is the fact that numerous activities are going on at one time in multiple locations with a variety of materials and equipment. Also, precipitation, wind, and other environmental factors further contribute to the variability of site conditions.

The basic sequence of construction at a landfill includes:

- Subbase preparation
- Liner construction
- Waste placement
- Landfill closure

The subbase for a landfill refers to the ground surface on which the liner is constructed. Compaction and grading of the subbase are necessary so that the actual liner can be constructed easily. The subbase may be developed on existing ground surface or may be constructed on fill or cut. Great care must be taken so that the subbase provides a sound foundation for the landfill. Conventional earthwork equipment and methods are used to prepare the subbase for a landfill.

Liner construction often consists of multiple layers of soil and geosynthetics (Figure 10.17). Placement of the soil follows conventional earthwork practices. Construction of geosynthetic liners requires specialized methods particularly related to connection of adjacent sheets/layers and prevention of damage (i.e., puncturing, tearing, blocking of drainage, etc.) during construction. As each new layer is placed,

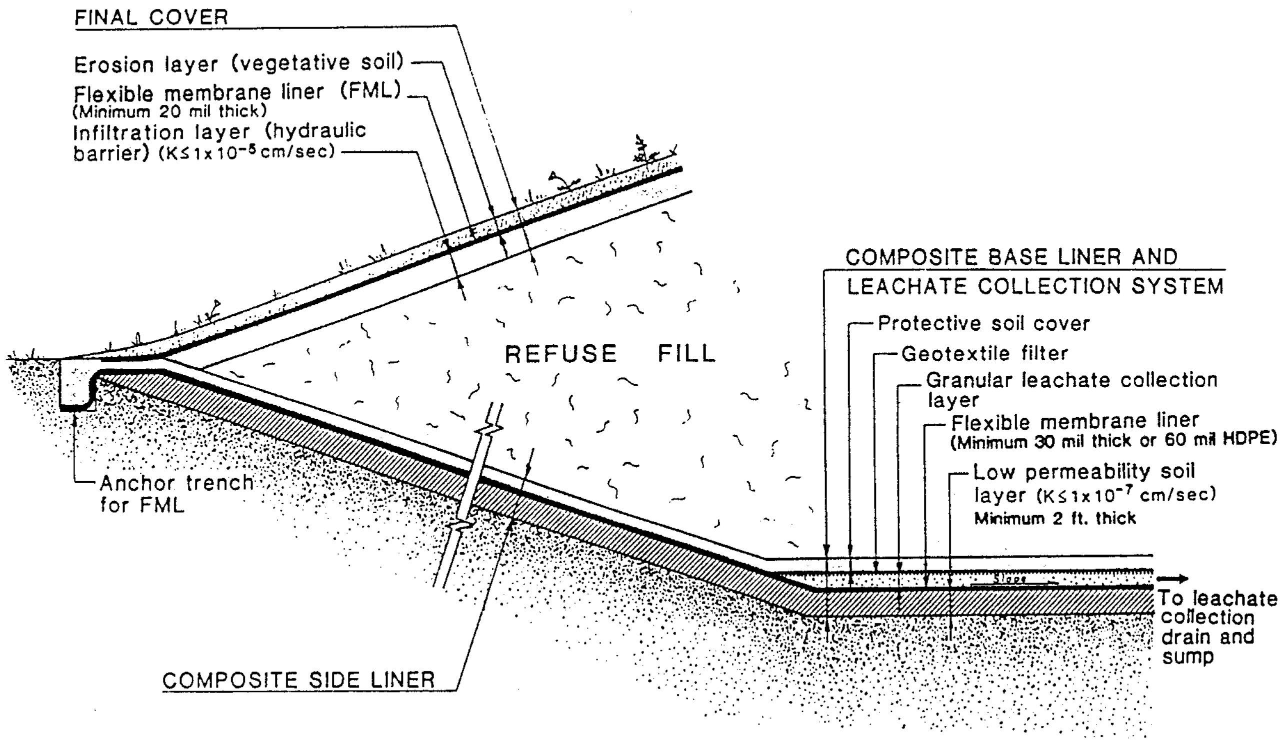

Figure 10.17 Subtitle D liner and final cover for a municipal solid waste landfill. (Sharma and Lewis, 1994.)

the shear strength between layers could change significantly. On a slope, the layer that possesses the lowest shear strength governs stability. It is conceivable therefore that as the liner layers are built up and then piled with waste, a slope could go from being stable to being unstable due to the introduction of a weaker layer.

Waste placement is a unique process during the life of the landfill because a variety of waste materials are being placed in a variety of configurations. Temporary slopes are commonly made and modified on a regular basis. Some of these slopes may be in contact with the liner system; some may not. Also, slopes made during this phase will be traversed several times by a variety of equipment and vehicles going a multitude of different directions. Since a slope stability analysis cannot be carried out every time a new load of waste arrives at the landfill, there must be governing restrictions on the steepness and location of temporary and permanent slopes. These restrictions must take into account precipitation and groundwater conditions.

After the landfill has received its maximum amount of waste, it must be closed such that it performs well and does not impact the environment in the long term. It receives a final cover or cap that is similar to the underlying liner system. In other words, there are a series of layers that get placed over the waste to control infiltration of water into the landfill, thus minimizing leachate, controlling the release of gases from the landfill, and providing for a physical separation between the waste and environment for protection of public health. A typical cross section of a landfill cap is shown in Figure 10.18.

Of the documented case histories of failures, most occur during liner construction and waste placement. Often, when failures have occurred, they were due to underestimation of interface friction angles between landfill, soil, and geosynthetic materials or the dynamic effects of equipment sequencing or loading. For instance, Martin (1998) reports on a failure in Washington that was due to bulldozer braking acceleration and deceleration loads. From a slope stability standpoint, all materials that are placed at some angle from horizontal must be analyzed for stability relative to the adjacent materials as well as all construction effects including sequencing, layering, equipment, weather, and so on.

10.6 SLOPE STABILITY CONSIDERATIONS

Slope stability considerations for a landfill must include all phases of development including subbase preparation, liner construction, waste placement, and landfill closure. Slope stability issues during subbase preparation are the same for any conventional cuts or fills as discussed previously in Chapter 6. Landfill slope stability is unique because after subbase preparation, multiple layers of soil, geosynthetics, and waste get piled on top and underneath each other, creating a unique combination of materials that behave in a composite fashion with uniquely different shear strength properties (Figure 10.19).

All of the methods of slope stability analysis discussed in this book apply to landfills. However, it is important to note the potential modes of failure in a landfill and the unique materials and material properties found at a landfill.

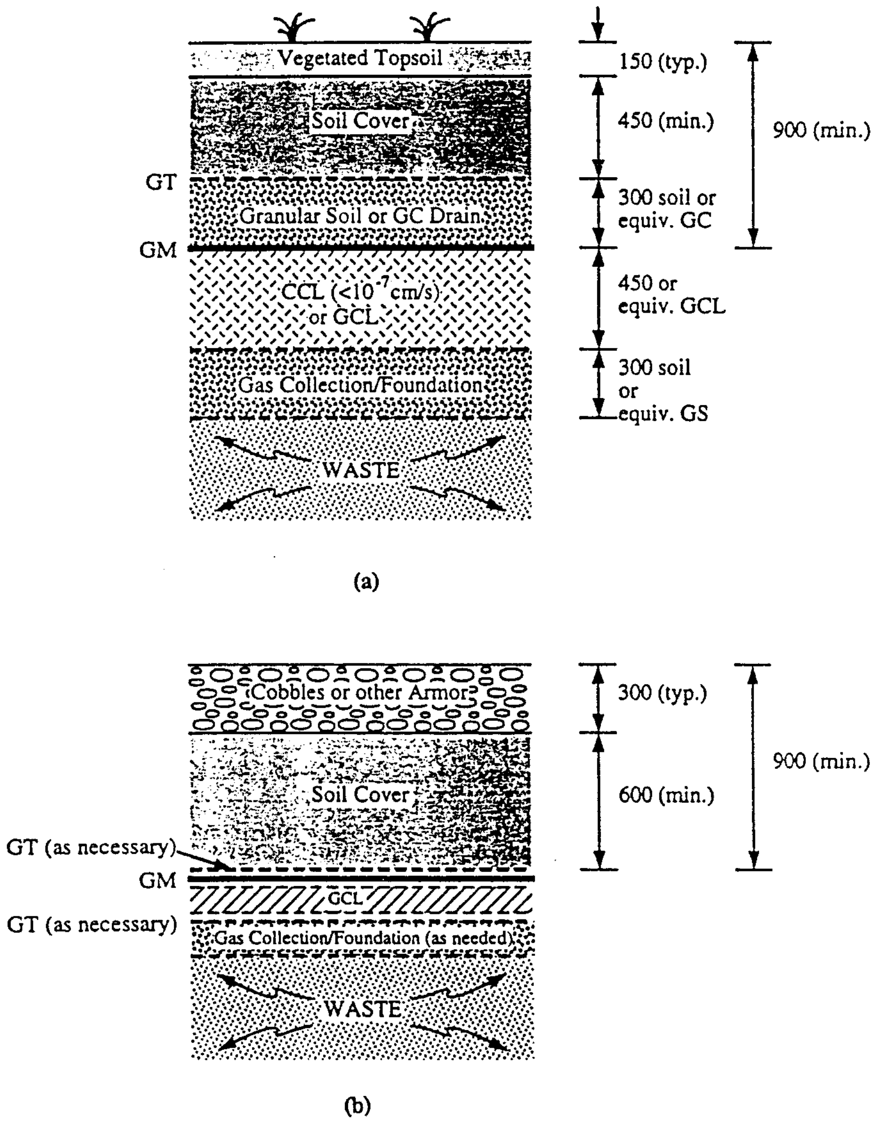

Figure 10.18 Examples of municipal solid waste final covers. (Koerner and Daniel, 1997.)

10.6.1 Excavation Slope Stability

Figure 10.20 illustrates some of the potential slip surfaces that may occur in landfill excavation slopes (Sharma and Lewis, 1994). For covered pit and trenched landfills, it is often simplest to adjust the excavation slope to meet stability and constructability requirements. Since the crest of the landfill excavation slope is typically limited by property boundaries, flattening an excavation slope from 2H–1V to 3H–1V can have a significant effect on landfill airspace and revenue. Most landfill owners therefore

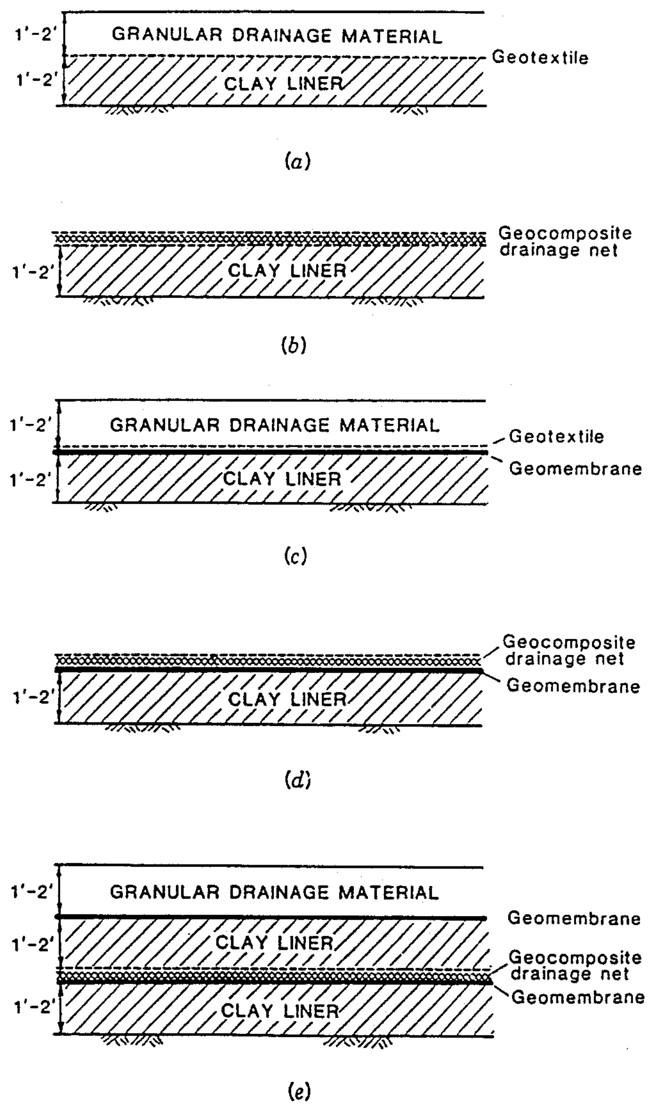

Figure 10.19 Types of landfill containment lining systems. (Sharma and Lewis, 1994.)

attempt to excavate slopes to the maximum grade achievable. Typical excavation slopes for covered pit and trenched landfills are approximately 3H–1V. This is because most soil slopes are stable at 3H–1V inclination and clay liners may be constructed on the side slopes at this grade.

Upslope and canyon landfills are often constructed at grades steeper than 3H–1V, due to the steep inclination of the natural slopes. Although these slopes may be trimmed back to meet a 3H–1V grade, a significant volume of excavation may be required due to the typically large slope heights. A groundwater table or weak soil

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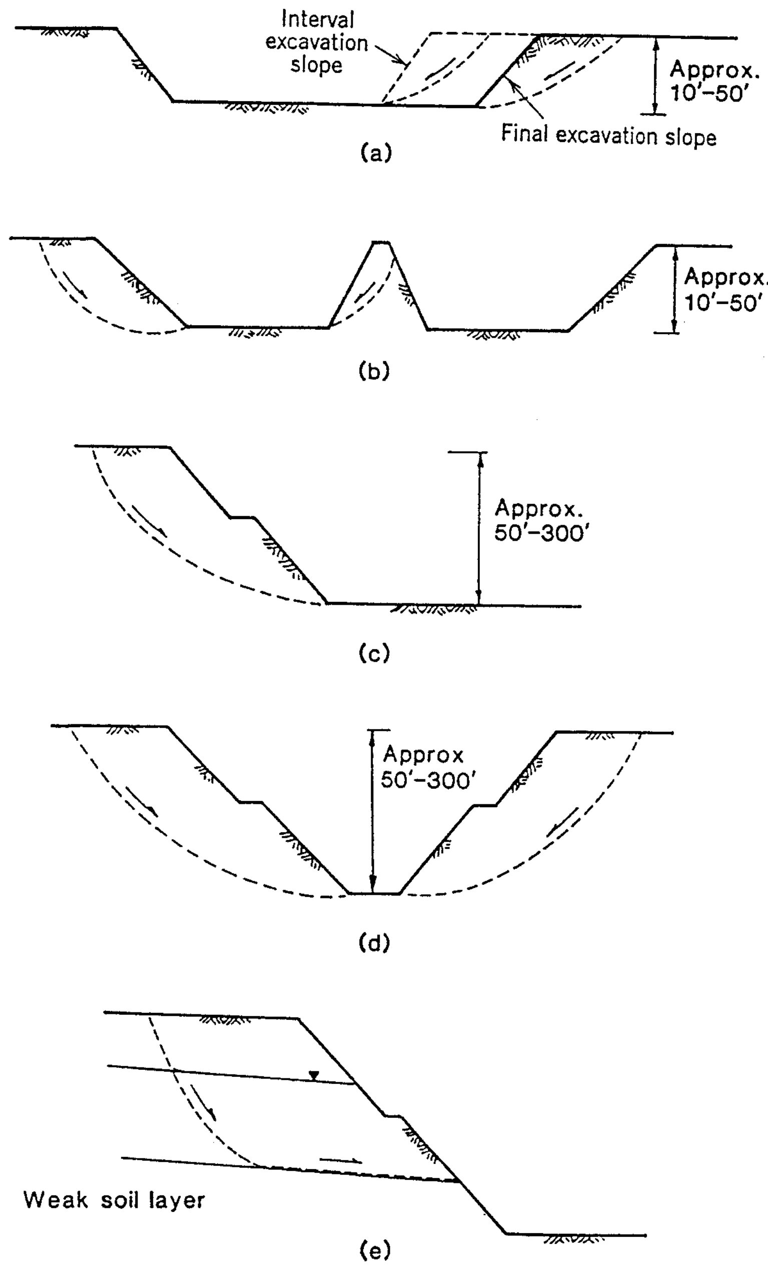

Figure 10.20 Potential slip surfaces in landfill excavation slopes. (Sharma and Lewis, 1994.)

layer also often influences the stability of upslope and canyon landfills. These items must be considered in the stability analyses.

For many excavation slopes, the worst-case conditions for analysis are either during construction or immediately following construction. Often, waste placed against the excavation slope acts as a buttress and prevents movement.

10.6.2 Waste Fill Stability

Potential slip surfaces through landfill waste generally occur in one of three ways, as illustrated in Figure 10.21:

- Through the waste alone
- Along the liner system
- A composite surface through waste and along the liner

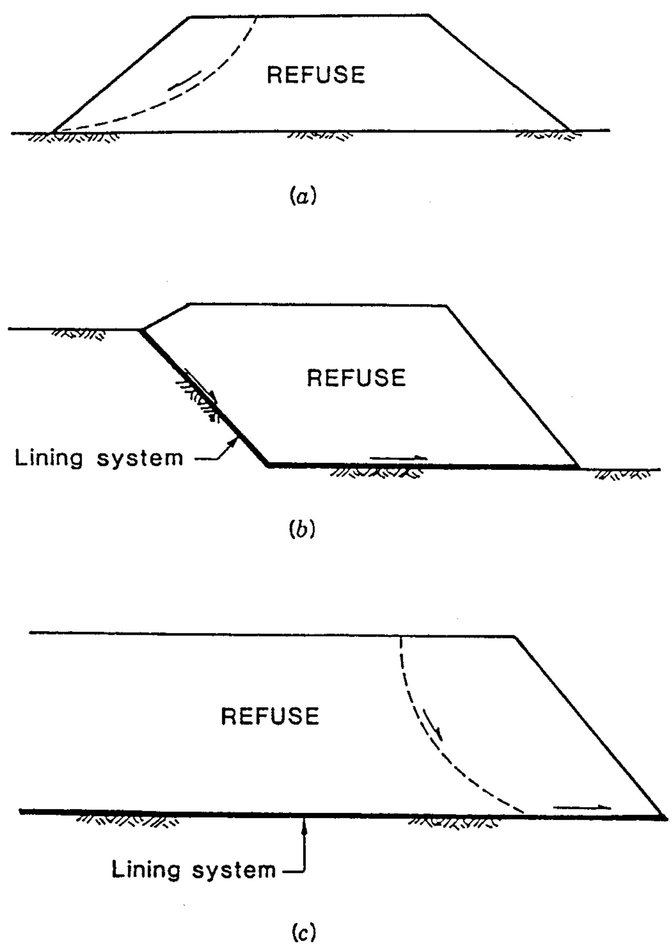

Figure 10.21 Potential slip surfaces in landfill waste. (Sharma and Lewis, 1994.)

Since waste is generally a heterogeneous material that may contain isolated masses of high-strength materials, such as discarded refrigerators or demolition debris, it is unlikely that slip surfaces through waste materials are circular. However, since the current state of technology affords no better solutions, waste fill stability analyses are typically performed using circular surfaces through waste materials. If a geosynthetic lining system is placed in a landfill, a translational block surface along the liner system or a composite slip surface through waste and along the liner system may be critical. There are several interface surfaces within a geosynthetic lining system where a weak plane may exist.

Figure 10.22 depicts typical slip surfaces that may occur through covered pit and trench-type landfills. For these types of landfills, the critical stability condition typically occurs in interim waste fills, prior to the pit or trench being filled to the ground surface (Figure 10.22*a*). Once the pit or trench is filled to the ground surface it is inherently stable (Figure 10.22*b*). If waste is placed above the ground surface, either a circular surface through the waste itself, or a block-type surface extending to the base lining system may occur (Figure 10.22*c*).

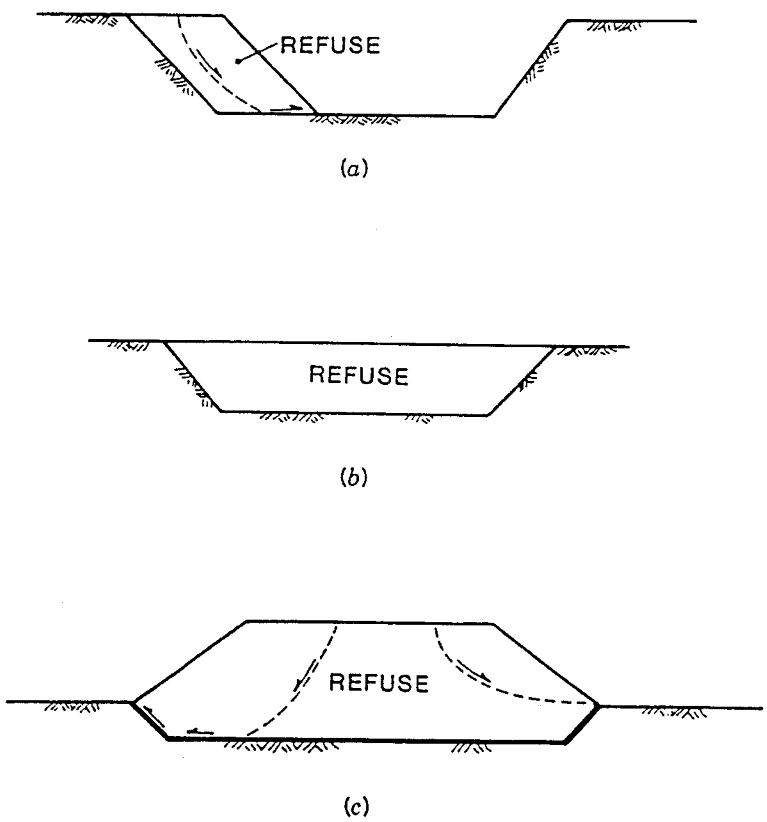

Figure 10.22 Potential slip surfaces in waste for pit and trench landfill configurations. (Sharma and Lewis, 1994.)

Figure 10.23 illustrates potential waste fill stability concerns in upslope or canyon-type landfills. From a stability standpoint, upslope landfills are perhaps the least desired waste fill configuration. This is because both interim slopes (Figure 10.23*a*) and final slopes (Figure 10.23*b*) have a significant potential for instability, especially

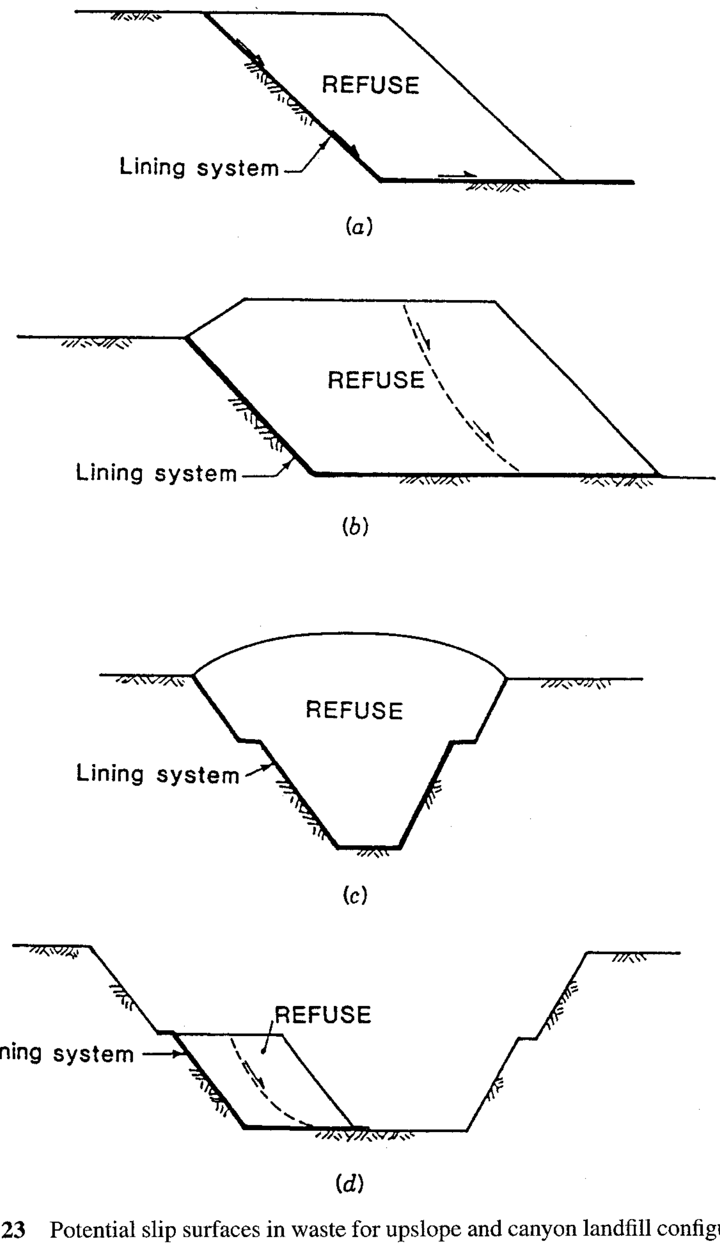

Figure 10.23 Potential slip surfaces in waste for upslope and canyon landfill configurations. (Sharma and Lewis, 1994.)

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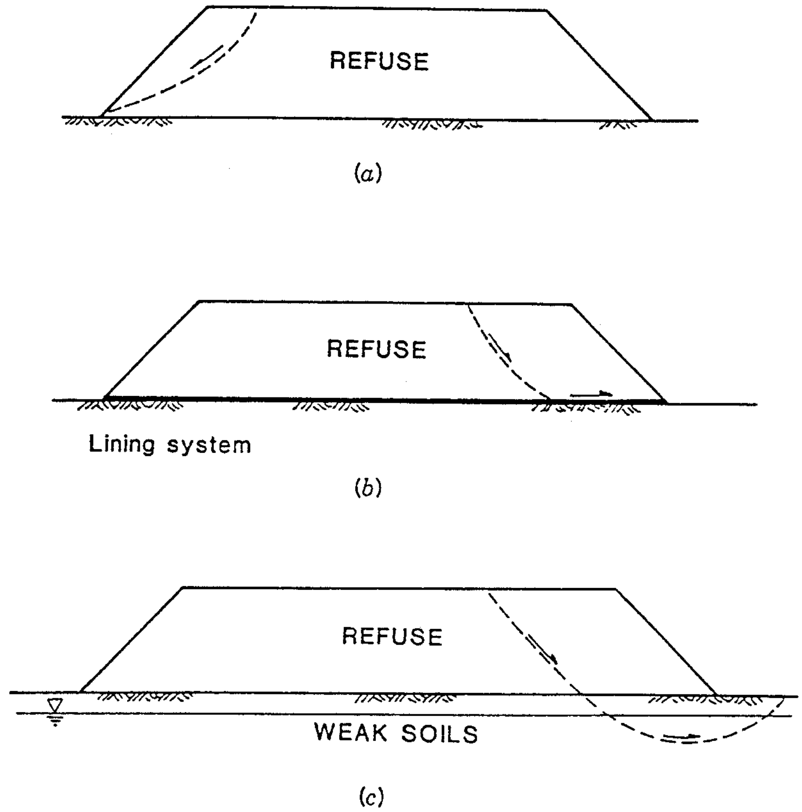

Figure 10.24 Potential slip surfaces in waste for aboveground landfill configurations. (Sharma and Lewis, 1994.)

when overlying a geosynthetic lining system. Canyon-type landfills are generally stable in their final configuration (Figure 10.23*c*), but may be unstable during interim fill conditions (Figure 10.23*d*).

Finally, aboveground landfills are generally stable when founded over firm ground, with some potential for a circular slip surface occurring if the side slopes are too steep (Figure 10.24*a*). If a geosynthetic lining system is employed, a composite-type slip surface may extend through the waste mass (Figure 10.24*b*). Aboveground landfills are also often constructed over soft subgrades or in zones of high groundwater. Bearing capacity subgrade failures (Figure 10.24*c*) should therefore be checked in these types of landfills.

In all of the landfill configurations illustrated, proper fill sequencing can alleviate many of the stability concerns. The key is to maximize the "resisting" portion of the potential slip surface prior to increasing the "active" portion. Figure 10.25 illustrates how this is achieved for covered pit and upslope landfills.

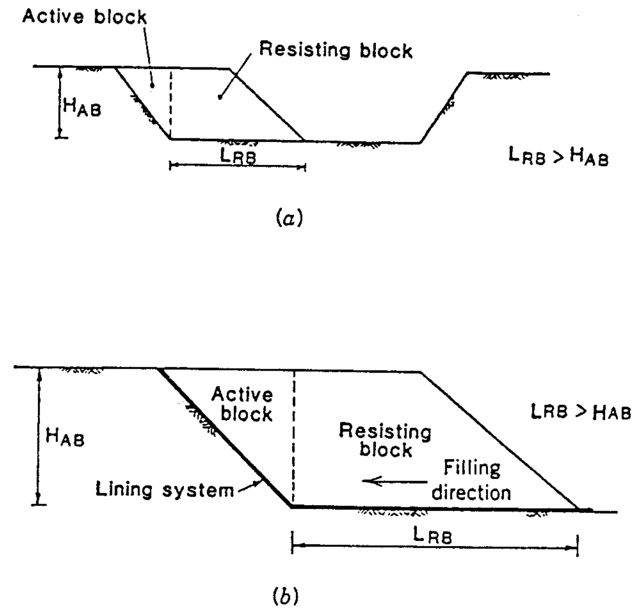

Figure 10.25 Active and passive stability blocks in covered pit and upslope landfills. (Sharma and Lewis, 1994.)

10.6.3 Cover System Stability

During preparation of landfill site development plans, consideration should be given to the stability of the landfill final cover system so that appropriate final landfill grades may be selected. Most landfills are constructed with final grades of approximately 3H–1V, which are generally stable for soil cover systems. There is some concern, however, regarding the ability of soil cover systems to deform with the underlying

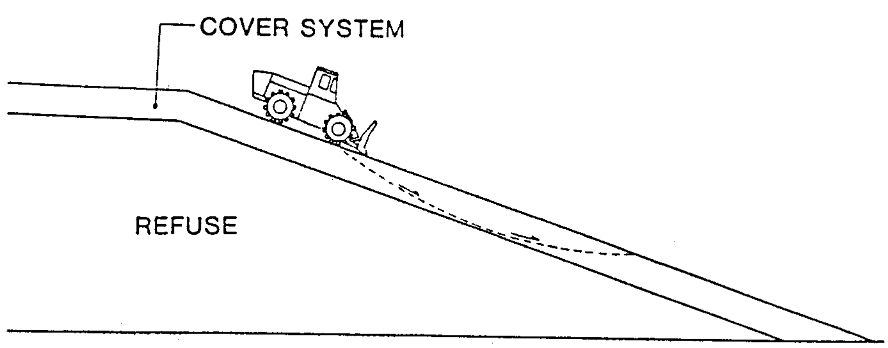

Figure 10.26 Shallow failure surface in final landfill cover due to construction equipment. (Sharma and Lewis, 1994.)

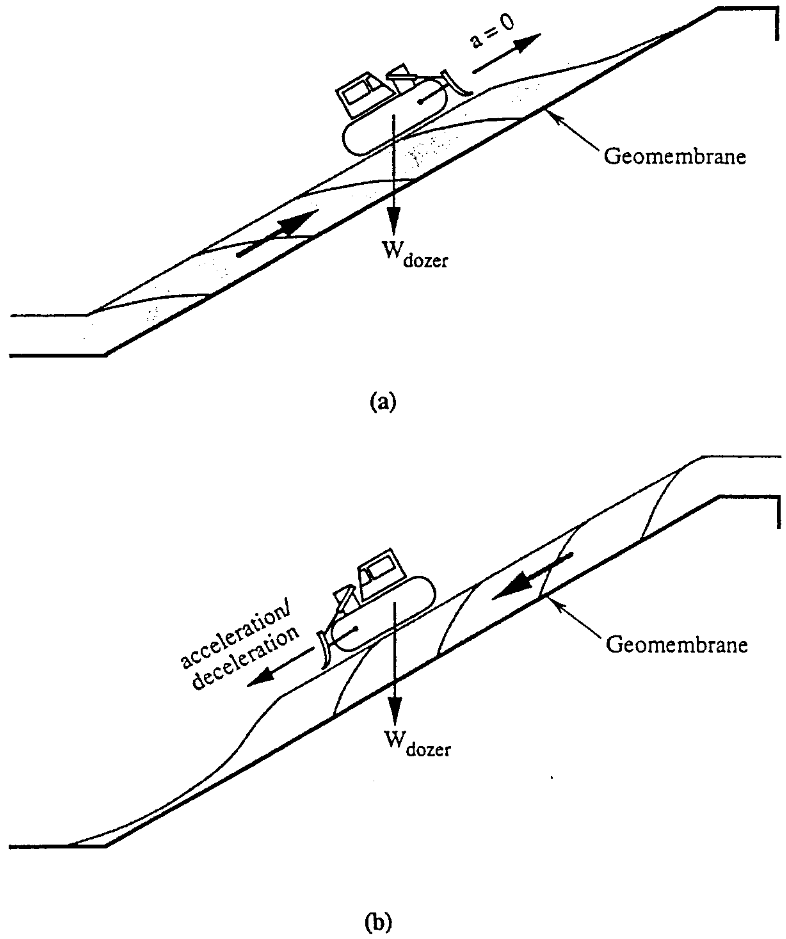

Figure 10.27 Effect of construction equipment loads on final landfill cover stability. (Koerner and Daniel, 1997.)

waste settlements. Geosynthetics are therefore often used in final cover systems since they are inherently more flexible. They may, however, create stability concerns.

Potential slip mechanisms in final covers are generally planar, occurring between material interfaces or through the material itself. Potential slip surfaces could occur along the interfaces or materials shown on Figure 10.26; however, there are some interfaces that are more likely than others to be critical weak planes. In some situations, the worst-case condition for a final cover system is during construction. This is due to the large equipment loads applied during construction. Failure due to equipment loads would probably result in a localized shallow circular slip surface especially when the final cover is being placed from the top of the landfill down to the base as illustrated in Figure 10.27.

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