Barrier Systems for Waste Disposal

2nd edition

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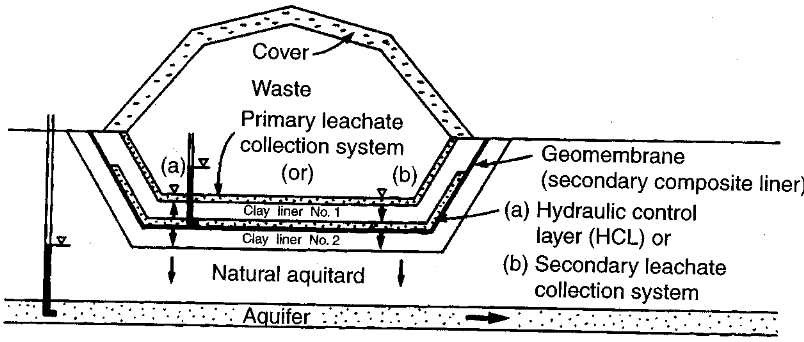

Figure 1.7 A compacted clayey primary liner used in conjunction with an engineered hydraulic control layer and hydraulic trap to minimize contaminant impact together with a composite secondary liner geomembrane (and clayey liner) used to minimize volume of fluid needed to maintain the hydraulic trap. By pumping the hydraulic control layer, this can also be used as a secondary leachate collection system. Note that second compacted clay liners could potentially be replaced by a GCL and foundation layer as discussed in Chapter 16.

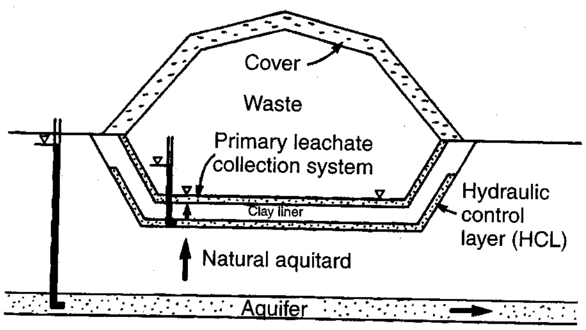

Figure 1.8 A compacted clayey liner used in conjunction with a primary leachate collection system and a hydraulic control layer to create a "natural" hydraulic trap.

1.2.3 Cut-off walls and permeable surrounds

Cut-off walls are most commonly used to limit contaminant migration from existing sites which have not been adequately designed; however, they can also be used in controlling migration from new sites where it may be desirable to isolate the (potentially contaminated) groundwater in a relatively thin and shallow aquifer beneath the landfill. For example, in the case shown schematically in Figure 1.9a, the thickness of the natural clay barrier may not be enough to prevent potential contamination of water flowing along the underlying minor aquifer.

By constructing cut-off walls around the site and hence reducing the flow in the aquifer locally, it is possible to change an advection-controlled system beneath the landfill into a diffusion-controlled system thereby substantially reducing the impact on off-site groundwater quality. It is, of course, still necessary to consider diffusive migration through the cut-off wall and into the aquifer. This can be achieved using techniques similar to those which will be discussed for natural or compacted clayey barriers in Chapter 10.

The containment of contaminated land by the construction of a vertical cut-off wall around part or the entire contaminated zone is growing in popularity (Pankow and Cherry, 1996). For example, these walls may be used to control the migration and spreading of the dense non-aqueous phase liquids (DNAPLs) and allow time to implement other remediation technologies. The walls may consist of steel, polyethylene or soil (soil-bentonite or soil-bentonite and cement). However, containment of DNAPL spills creates a situation unlike that in other remediation applications because the dissolved concentrations associated with such pools can potentially be as high as the solubility limit of the DNAPL spilled. This gives rise to a very large concentration gradient that has the potential to have a

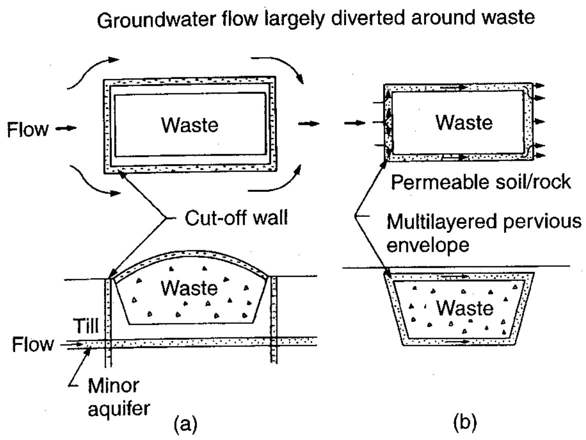

Figure 1.9 (a) A cut-off wall is used to divert groundwater flow from beneath a natural clayey barrier. (b) Pervious material is placed outside the waste to divert groundwater flow around (rather than through) the waste (after Rowe, 1988; reproduced with permission of the *Canadian Geotechnical Journal*).

substantial impact on the surrounding aquifer due to diffusion and it is important to select the appropriate barrier for the contaminated site.

An interesting alternative to construct a low-permeability cut-off wall is the "pervious surround" concept developed by Matich and Tao (1984), which involves minimizing advective transport through a waste pit by surrounding it with a multilayered pervious envelope with less permeable material adjacent to the waste and more permeable material outside of this as shown schematically in Figure 1.9b. In this way, water flow is directed around the outside of the pit rather than through the pit, and contaminant migration would be predominantly by diffusion from the waste through the less permeable material, together with advective–dispersive transport within the more permeable outer zone. Thus, from the standpoint of modelling, determination of contaminant loading of the groundwater for this case is also very similar to that for waste sites separated from an underlying aquifer or drainage system by a clayey barrier as shown in Figure 1.2.

1.2.4 Bedrock

A topic of particular interest in some regions is the migration of contaminants from existing or proposed landfills excavated into, or sitting on top of, fractured rock. Typically, the intact rock has a very low hydraulic conductivity and contaminant migration will primarily involve advective–dispersive transport along the fractures in the rock (see Figure 1.10). In these cases, the primary mechanism limiting the movement of contaminant is the process of matrix diffusion whereby contaminant is removed from the fracture as it diffuses into the matrix of the rock. For example, monitoring of an existing landfill at Burlington, Ontario (Gartner Lee and Associates Ltd, 1986), suggests that after 15 years migration, contaminant movement in fractured shale downgradient of the Burlington landfill is probably not more than 25 m and that substantial attenuation has occurred. The migration of contaminants through fractured porous rock is discussed in Chapter 11.

These days landfills proposed for old worked out quarries in fractured rock will typically have a liner system, where the major challenge is constructing a suitable liner along the side walls of the quarry. However, landfills also have been

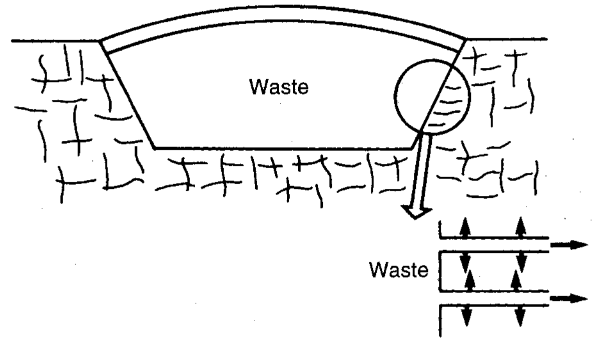

Figure 1.10 Landfill located in fractured shale. Contaminant transport along the fractures is attenuated by diffusion into the matrix of the shale adjacent to the fractures (after Rowe, 1988; reproduced with permission of the *Canadian Geotechnical Journal*).

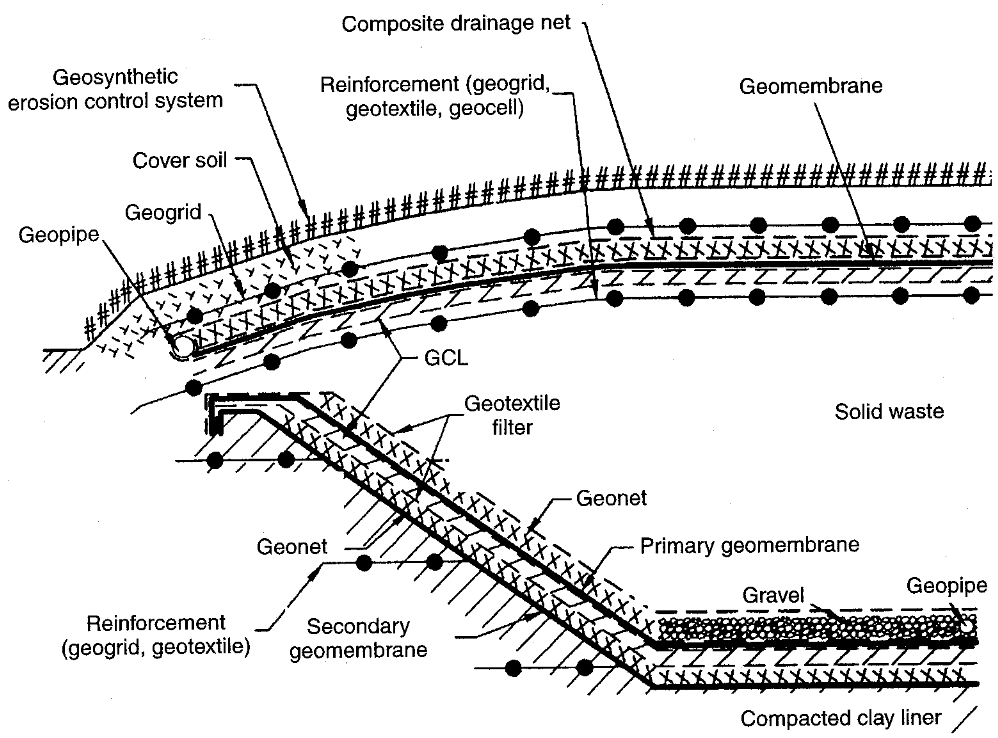

Figure 1.31 Multiple uses of geosynthetics in landfill design (modified from Zornberg and Christopher, 1999).

Leachate collection systems can experience clogging that can result in a substantial leachate mound on the base of a landfill. French drains and sand drainage blankets are particularly prone to clogging. The effect of clogging can be minimized and the service life of collection systems can be extended by appropriate design. Techniques for estimating the service life of granular drainage blankets in leachate collection systems are discussed in Chapter 2.

Leachate mounding appears to give rise to an increase in temperature on the underlying liner system. This has the potential to increase advective–diffusive contaminant transport and decrease the service life of some engineered component of the barrier systems (Chapters 12 and 13).

Leakage through geomembranes may be more than conventionally expected due to holes and wrinkles in the geomembranes. Data relating to the number of holes and hole size are presented in Chapter 13, and equations that may be used to estimate leakage are discussed in Chapter 5.

Diffusion through compacted clay liners, geosynthetic clay liners and geomembranes is discussed and typical parameters are given in Chapters 8, 12 and 13, respectively.

The service lives of compacted clay liners, geosynthetic clay liners and geomembranes are important considerations and are discussed in Chapters 3, 12 and 13.

While the focus of the design of barrier systems is on geoenvironmental issues, it is important not to overlook the geotechnical issues (see Chapter 15). Although there have been numerous landfills successfully constructed, there have also been a number of geotechnical failures that have included:

1. slides of the leachate collection layer;
2. sliding of waste and liner along a failure plane associated with liner construction;

3. slides associated with fluid pressures in landfills (e.g., due to leachate recirculation);
4. general shear failures associated with expansion of existing landfills;
5. general shear failures due to inadequate geotechnical stability assessment;
6. basal fracturing due to excessive water and gas pressures arising from an underlying aquifer.

Issues that need consideration include the geosynthetic (GS)–clay interface properties, the water content of clay near GS–CCL interface, the potential for a decrease in design interface strength during construction (e.g., due to rain during placement of GM), the selection of appropriate strength parameters (e.g., peak strength may only be mobilized over portion on failure surface), excess pore pressures developed in waste (e.g., due to recirculation of leachate or co-disposal of liquids), the effects of excavation at toe of exiting waste pile on stability, the risks associated with placing of waste above approved contours without checking stability, the selection of an appropriate waste density for stability calculations (e.g., neglecting the increase in density that occurs as water content increases has contributed to failures) and the effects of excavation on basal stability.

1.9 Impact assessment

The design of a barrier system is often intimately related to the environmental impact assessment of the proposed landfill. Environmental impact assessments are generally driven by regulatory requirements and as a consequence typical "acceptable" barrier systems can be expected to vary regionally as a result of variations in both hydrogeologic conditions and regulatory requirements. The fundamental question underlying most impact assessments is whether the proposed landfill will have no more than a negligible effect on groundwater quality at the site boundary. However, this perfectly reasonable question raises two subsidiary questions – what is a "negligible effect" and over what period of time must the effect be negligible? The answer to the latter question has very important implications since it is intimately related to the design life of the engineering features of the facility. The design of a barrier system for a landfill that is only required to have "negligible" impact for a 30-year post-closure period is likely to be different from the design for a 100-year period, which, in turn, may be quite different from that required to have negligible effect on groundwater quality in perpetuity.

Typically, environmental regulations fall into one of the following categories:

1. essentially no regulation;
2. prescriptive regulations which specify minimum requirements such as "two liners of which at least one is a synthetic liner";
3. regulations requiring "no impact" or "negligible impact" for a prescribed period of time (e.g., 30 years or 100 years post-closure); and
4. regulations requiring negligible impact in perpetuity.

The legal implications of different regulatory systems have been discussed by Estrin and Rowe (1995, 1997).

1.9.1 Non-existent regulations

The situation where there is no regulation provides considerable latitude to the landfill proponent and designer in terms of the barrier system adopted. It also provides little assurance that the environment will be protected unless the design is subjected to a rigorous pre-construction review.

1.9.2 Prescriptive regulations

Prescriptive regulations are simple. They are typically based on the perception of the regulators as to what constitutes a safe design and will implicitly have negligible effects on groundwater quality. Unfortunately, prescriptive regulations higher risk projects (e.g., where the waste slopes may exceed 4H:1V), published values may be used for an initial estimate, but large-scale direct shear tests on the expected waste may be required. Consideration of instability from sliding along weak planes (especially along interfaces involving geosynthetics – see Section 15.2.4) must be given when assessing the stability of the waste pile.

(d) Hydraulic conductivity

An assessment of waste hydraulic conductivity is needed to predict the rate and pattern of moisture movement within MSW, especially when practising leachate recirculation, and for assessing leachate mounding (Section 2.4.2). Hydraulic conductivity, like the other waste properties discussed in this section, is highly dependent on the composition of waste, degree of compaction (Manassero *et al.*, 1996), overburden pressure (Powrie and Beaven, 1999) and the age of the waste (Powrie and Beaven, 1999). Generally the higher the compaction and overburden stress and the greater the age of the waste, the lower the hydraulic conductivity.

Data summarized by Manassero *et al.* (1996) showed a wide range of hydraulic conductivity with values between 2×10^{-4} and 1×10^{-8} m/s being reported. Most of the published values fell between 10^{-4} and 10^{-6} m/s; however, much of this data were from tests performed at relatively low effective stresses in the uppermost portion of the waste. Large-scale test results from the Pittsea compression cell (Powrie and Beaven, 1999) gave hydraulic conductivity values that decreased from 1.5×10^{-4} m/s at an applied stress of 40 kPa to 3.7×10^{-8} m/s at 600 kPa. They reported the following best-fit line estimate of hydraulic conductivity, k_w (in m/s) in terms of the vertical effective stress, σ' (in kPa):

$$k_w(\text{m/s}) = 17[\sigma'(\text{kPa})]^{-3.26} \qquad (15.1)$$

Rowe and Nadarajah (1996c) reported the following correlation, based on field data at different depths, relating hydraulic conductivity, k_w (in m/s) to depth, z (in m):

$$k_w(\text{m/s}) = 1.8 \times 10^{-4} \exp[-0.269\, z(\text{m})] \qquad (15.2)$$

While both equations 15.1 and 15.2 represent best-fit curves to data, it must be recognized that there is considerable scatter of data around these curves as a consequence of the intrinsic variability of waste. Thus these, like any other empirical relationships, should be used with considerable caution.

The ratio between horizontal, k_{wh}, and vertical, k_{wv} hydraulic conductivity has been investigated by Hudson *et al.* (1999) and Landva *et al.* (1998). Hudson *et al.* (1999) reported that the anisotropy ratio, k_{wh}/k_{wv}, increased from about 2 at an applied vertical stress of 40 kPa to about 5 at an applied stress of 600 kPa. Landva *et al.* found the ratio to be relatively constant at $k_{wh} \approx 8k_{wv}$, regardless of the level of applied vertical stress in the range of 150–500 kPa. This anisotropy may be particularly important in designing leachate recirculation systems and has the potential to give rise to leachate seeps with the injection of leachate.

15.2.2 Settlement

Settlement of soils occurs because of increases in effective stresses; for waste containment facilities, the increase in effective stress normally arises from the weight of the waste. The magnitude of ground settlements can be estimated using standard procedures described in conventional Soil Mechanics texts (e.g., Lambe and Whitman, 1979). Settlements do not normally pose a major problem, provided that they are uniform across the site. Concern arises when the settlements are larger in one region relative to another. This can occur due to variations in the applied pressures (e.g., from variable thickness in waste), thickness of compressible layers and stiffness of underlying soil materials. Such differential settlements of the soil beneath a landfill and/or differential settlement of CCLs may reduce the effectiveness of leachate collection systems. Leachate will pond in areas where settlement occurs, increasing the hydraulic head and consequent flow through the underlying soil. Thus, the magnitude of differential ground settlements should be estimated when selecting the slope of the leachate collection system (Section 2.4), and whenever possible this slope should be selected to minimize leachate ponding after ground settlements have occurred.

Additionally, tensile stresses induced in geomembrane liners from differential settlement of underlying materials may initiate holes or ruptures. Giroud and Bonaparte (2001) present equations that can be used to calculate the strains in geomembranes arising from differential settlements. Differential settlements may also cause axial tensile stresses in leachate collection pipes, especially near connections with manholes. Existing solutions derived to obtain the stresses in laterally loaded piles (e.g., Poulos and Davis, 1980) may be used to estimate the axial stresses in the pipes in these cases. It is conceivable that under some extreme circumstances, differential ground settlements may lead to tension cracks in compacted or natural clay barriers.

Disposal of waste on top of an existing landfill (often referred to as vertical expansion) is becoming increasingly common to satisfy demand for landfill space and given the public aversion to the approval of new landfills. Vertical expansions may be expected to increase the occurrence of problems related to differential settlements. The use of geogrid reinforcement over areas of potential differential settlements may reduce such impacts. Since the settlements of the waste are expected to be highly variable (leading to large differential movements), the impact of these settlements on engineered components in the landfill needs to be carefully considered, and redundancy should be included in design if the consequences of excessive differential settlements could lead to unacceptable performance.

15.2.3 Bearing capacity

Consideration should be given to the overall bearing failure of the soil beneath the landfill. This may be especially important for deep landfills underlain by very soft soils where the strength of the soil may be insufficient to resist the pressures from the landfill above. Bearing failure may lead to large vertical and lateral displacements that can cause damage to the barrier system. The bearing resistance depends on the geometry of the landfill, the shear strength of the underlying soil and the nature of the underlying ground materials with depth (e.g., the presence of stiff or weak stratum) to a depth beneath the landfill approximately equal to the width of the landfill. Assessment of the potential of bearing failure can be made using conventional bearing capacity theory (that can be found in standard geotechnical engineering texts) treating the landfill as a flexible foundation resting on the underlying ground materials.

15.2.4 General stability

A landfill can represent a major loading on the underlying soil. Just as with the design of any structure on soil, care should be taken to ensure the overall stability of the landfill under both static and seismic conditions where appropriate. This involves an evaluation of the geotechnical conditions (e.g., type and strength of soil and waste materials, pore pressures in the soil and waste) and assessment of anticipated loadings followed by an appropriate stability analysis (e.g., see Leroueil *et al.*, 2001) to identify the most critical conditions in the development of the landfill. A factor of safety is used to keep the magnitude of the disturbing forces less than that of restoring forces along a potential rupture surface. With respect to global stability of a landfill, the factor of safety attempts to quantify uncertainty in material properties (both loads and resistances) and assumptions of the stability analysis, and the implications of failure with one single value. Thus, the required factor of safety for stability is not a unique value and depends on the situation being examined.

Stability analyses should consider potential instability: (1) during excavation of the landfill (e.g., side slopes, Figure 15.1a, or trenches); (2) during development of the landfill (e.g., waste slopes, Figure 15.1b) and (3) of the completed landfill (e.g., waste slopes, Figure 15.1c and around the landfill Figure 15.1d,e). Particular care is required not to underestimate the pressures that can be applied by partially saturated waste, and a conservative estimate of the unit weight of waste should be considered when evaluating landfill stability (see Section 15.2.1). This is particularly critical for landfills being constructed in soft soils, sensitive soil, near escarpments or on slopes. Proper assessment of pore pressures is essential, and the potential for increased pore pressures in the waste (Figure 15.1c) due to clogging of the leachate collection system or leachate recirculation should not be overlooked. Consideration must be given to non-circular failure surfaces either because of planes of weakness in underlying ground materials, in the waste, or along interfaces between geosynthetic components (Section 15.2.5). When existing landfills are being expanded particular care is required to integrate the new and old portions of the landfill, and, in particular, to avoid stability problems when excavating at or near the toe of the existing landfill (Figure 15.1f).

Landfill failures have occurred due to failure to adequately assess overall stability. Selected case histories involving instability of landfills are presented in Table 15.1. In one such example reported by Reynolds (1991), failure occurred during the expansion of an existing 22-m high MSW landfill. Although there were a number of factors contributing to the failure, excavation of 1–3 m of surficial stiff clay and a trench at the toe of the slope of the old landfill were the principal causes of the instability. In this case, the density of the waste was also underestimated

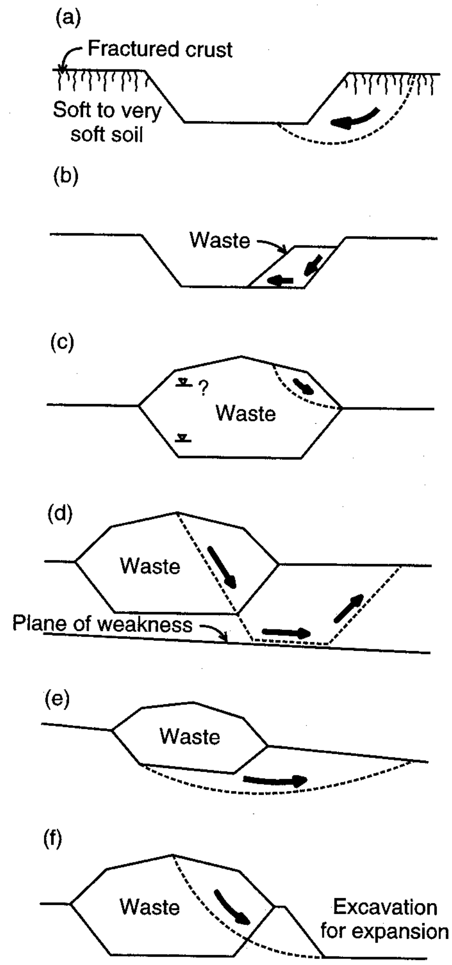

Figure 15.1 Illustration of a number of potential mechanisms to consider when examining landfill stability: (a) after excavation of cell, (b) during placement of waste, (c) waste slopes, (d) failure along planes of weakness, (e) general slope failure initiated by presence of landfill and (f) failure initiated by excavation for expansion.

(by more than a factor of two) as greater compactive effort and more daily and intermediate cover (to control odour, birds and windblown waste) were used during placement of the waste than was assumed in the initial design calculations. Additionally, a stockpile of material that was placed near the crest of the landfill and heavy rains that occurred prior to the slide also

Table 15.1 Summary of selected case histories involving instability of landfills (based on reports in papers cited)

Case	*References*	*Barrier system*	*Description of instability*	*Likely cause*	*Lessons learnt*
Maine Slide	Reynolds (1991)	Existing landfill: unlined Expansion: GM + 0.6-m CCL	Large mass movements of waste (up to 50 m) and flow of remoulded soft clay (up to 120 m) from general shear failure during expansion of existing landfill	Several factors: excavation of stiff clay crust at toe of slope, density of waste was higher than expected, pile of excavated soil placed near landfill crest, and heavy rainfall	Must carefully consider how construction activities during expansion may influence stability
Kettleman Hills Slide	Mitchell *et al.* (1990a,b), Seed *et al.* (1990), Byrne *et al.* (1992), Stark and Poeppel (1994)	Triple composite liner (from top down): 1.5-mm GM + 0.45-m CCL + 0.3-m SLCS+ 1.5-mm GM + 1-m CCL +GT + drainage rock+ 2-mm GM + subgrade	Mass movement of waste with lateral displacement up to 11 m and vertical slump up to 4.3 m from slip along multiple geosynthetic interfaces	Low interface strength between secondary GM and CCL from undrained clay response	Need to carefully select interface strength considering field conditions, strain level, water content
French Slide	Ouvry *et al.* (1995)	Base: 2-mm smooth GM + GT + 3-m CCL Slopes: 2-mm smooth GM + 3-m CCL	Mass movement of waste up to 6.7 m at the top of the waste during waste placement	Slip between GM/CCL interface on slopes and GM/GT along base	Consider possible decrease in interface strength from heavy rainfall
Cincinnati Slide	Stark *et al.* (2000)	1.5-mm GM + 1.5-m CCL	Translation towards excavation near toe of existing landfill	Strength of ground beneath landfill exceeded	Proper geotechnical assessment of global stability essential during expansion

Table 15.1 *Continued*

Case	*References*	*Barrier system*	*Description of instability*	*Likely cause*	*Lessons learnt*
Dona Juana Landill	Hendron *et al.* (1999)	1-mm PVC GM + CCL or native soil	Large down slope movement of waste. Failure surface not along liner	Large pore pressures in waste from leachate recirculation system	Recirculating leachate can lead to large pore pressures that must be included in stability assessment
Bulbul Drive Landfill	Brink *et al.* (1999)	CCL liner below phase 1A, composite 1.5-mm fPP GM + CCL below phase 1B in valley landfill with a longitudinal slope of approximately 10% and side slopes of approximately 36%	Rapid translational slide when waste height reached 45 m in the phase 1B. About 160,000 m^3 of waste flowed into the valley below	Liquid waste deposited into trenches excavated into uppersurface of landfill near interface between phases 1A and 1B. A failure surface developed along the interface between the two phases of waste and then along the interface with the composite liner at the base	Interface between phases of waste disposal may be weaker than the waste mass. Injection of fluid raised pore pressures and reduced shear strength
Beirolas Slide Lisbon	Santayana and Pinto (1998)	1.5-mm GM + GCL+ CCL (on base) liner on the landfill base. Foundation soil consisted of 4–5 m of silty clay fill overlying a 20–35-m thick estuarine and alluvial soft clay deposit	Slide extended about 270 m along most of the area where 110,000 m^3 of contaminated soil had been placed. Nearly vertical failure scarp associated with 4 m of vertical and several metres of horizontal movement	Shear strength of the soft clay subsoil overestimated. Also actual failure mechanism not considered in the design calculations	Perform a proper site investigation. Examine all potential failure mechanisms. Avoid optimism in shear strength when faced with data that is inconsistent with that optimism

GM, geomen brone; GT, geotextile; CCL, compacted clay liner; SLCS, secondary leachate collection system; PVC, polyvinyl chloride; FPP, flexible polypropylene decreased stability. This case clearly highlights the need to consider many issues when assessing stability.

Careful consideration must also be given to the development of excess porewater pressures within a landfill, especially those practising leachate recirculation, since this has been identified as a principal factor in the failure at the Dona Juana Landfill (Hendron *et al.*, 1999). Finally, it is essential not to overlook traditional geotechnical issues related to the stability of the subsoil as illustrated by the Beirolas slide in Lisbon. Here the post-failure investigation (Santayana and Pinto, 1998) concluded that the failure occurred because the shear strength of the soft clay had been overestimated. Although it had been considered to be normally consolidated under a surcharge of 4 m from the existing fill, it was reported to be underconsolidated with very little dissipation of the excess pore pressures caused by placement of the fill in the 1970s and 1980s. Furthermore, the failure extended out into the river in an area where no fill had been placed (and hence no strength gain could have occurred due to consolidation) but, reportedly, the failure mechanism had not been considered in the design calculations.

15.2.5 Stability of engineered systems on side slopes

Figure 1.31 illustrates an engineered barrier system involving geomembranes, geonet drains, geotextiles and compacted clay all extending up a side slope. Tensile forces will be mobilized in the geosynthetic components lining the side slopes of the waste containment facility due to the waste overburden loads, waste settlement (down drag forces), and from the self-weight of the geosynthetic components themselves. The current method for the evaluation of these tensile forces and general stability is static equilibrium method (Richardson and Koerner, 1988). This evaluation requires knowledge of the interface strength characteristics between the various components of the lining system. Except possibly for only very low-risk projects, the interface properties should be measured for each specific project on a case-by-case basis (as opposed to relying on published test values) due to disparities of interface strengths between products from different manufactures, or even for otherwise identical materials from the same manufacturer (Bonaparte *et al.*, 2002). Bonaparte and Yanful (2001) summarize laboratory methods to obtain interface strength parameters. Most commonly direct shear friction testing (e.g., Bove, 1990) is used. It is essential that these tests be conducted with conditions representative of the actual field conditions (e.g., materials, stress conditions, water contents, stress and strain levels, and strain rates). From these experiments both peak and residual strengths can be obtained. The peak strength corresponds to the maximum strength obtained from the test while the residual strength is often much lower and occurs at large strains. The selection of the appropriate value for use in a stability calculation is the subject of much debate and depends on the strains (or displacements) expected in the field and the factors of safety to be applied. In the extremes, if only small displacements can be assured it is reasonable to use the peak strength, whereas the residual strength should be used if large displacements are likely to occur. However, in many practical situations it is not so straightforward to select the appropriate strength, and each case needs to be evaluated carefully by an experienced geotechnical engineer as illustrated by the cases summarized in Table 15.1 and highlighted in the following paragraphs.

A case involving failure to sliding along the interfaces within a composite, multilayered geotextile, geomembrane and clay liner system has been reported and studied by Mitchell *et al.* (1990a,b), Seed *et al.* (1990), Byrne *et al.* (1992) and Stark and Poeppel (1994). Down slope mass movements of about 11 m horizontally and up to 4.3 m vertically were observed when the waste was reaching its maximum height of 27 m above the base. Failure was attributed to sliding along multiple interfaces within the landfill liner system, primarily resulting from low interface shear strength between the geomembrane and clay in a secondary composite liner system. The strength of this interface was essentially undrained even one year following construction since the low hydraulic conductivity of the clay and the presence of the geomembrane limited the dissipation of excess pore pressures in the clay. Additionally, it was believed that the peak interface strength might have only been mobilized along a portion of the failure surface. Thus, an evaluation of stability based solely on peak strengths is inappropriate for the large mass movements that occurred in this particular case.

Another case that demonstrates the need to carefully consider stability of the barrier system has been described by Ouvry *et al.* (1995). At the base of this landfill, the barrier system consisted of a 2-mm thick smooth HDPE geomembrane placed on top of a 190 g/m^2 spun-bonded nonwoven geotextile overlying compacted clay, and on the side slopes the geomembrane was placed directly on top of clay. Down slope mass movements of waste (as large as 5–6.7 m) were caused by slip along the geomembrane/clay interface on the side slopes and along the geotextile/clay interface at the base. At the time of the slide, the waste had an average thickness of 12–15 m (maximum 20 m). The geomembrane was pulled out of the anchor trench over a length of 60 m. The moisture content of the clay from beneath the geomembrane was found to be 5–9% higher than it had been after compaction because of heavy rainfall during placing of the geomembrane on the clay in this case. Since the water content influences the shear strength developed at the interface between geomembrane and clay, the potential for a decrease in design interface strength due to events occurring during construction should be considered. Instability due to fluid pressures at the interface can also arise from landfilling operations that involve injection of fluid into the landfill. This has been illustrated by the failure at the Bulbul drive landfill (Brink *et al.*, 1999) where injection of fluid caused a reduction in the shear strength at the interface between two phases of waste placement and along the interface with the lining system.

When interface friction alone is not sufficient to prevent sliding, either the inclination of the slope must be reduced or tensile elements must be introduced to carry loads that would otherwise result in shear along a potential failure surface. The geosynthetics in tension must be anchored at the top of the slope and descriptions of anchoring methods and determination of the anchorage capacity are available (e.g., see Richardson and Koerner, 1988).

Figure 1.31 shows the inclusion of geogrids to steepen the side slopes, which will increase the available landfill airspace. Jewell (1991) has published design charts for geogrid reinforced slopes. Note that an increase in slope angle will be accompanied by an increase in the tensile forces mobilized in the geosynthetic lining the slope and an increased risk of sliding unless appropriate measures are taken.

The geosynthetics in tension must have sufficient strength to withstand the tensile forces to which they are subjected, and high safety factors should be used since strength losses due to installation damage, long-term creep and degradation mechanisms must be expected. Possible degradation mechanisms include UV exposure during construction on unprotected slopes, chemical reactions with leachate, swelling due to chemical adsorption, extraction, oxidation and biological attack (Koerner *et al.*, 1990).

15.2.6 Blowout or basal heave

In addition to considering general stability due to bearing capacity or slope failure, consideration should also be given to the potential for blowout (or basal heave) of the bottom of the excavation. This occurs when the uplift from water pressure in an underlying aquifer is similar to the self-weight of the overlying materials and is particularly critical for landfills being designed with hydraulic containment (see Section 1.2.1) where the water pressure in an underlying aquifer is to be used to minimize contaminant transport from the landfill. This same water pressure (if not controlled) can cause blowout of the base of the landfill. The factor of safety against blowout FS_{bh} is defined as:

$$FS_{\text{bh}} = \frac{\sum_{i=1}^{n} \gamma_i t_i}{h_{\text{p}} \gamma_{\text{w}}} \tag{15.3}$$

where γ_i is the unit weight and t_i the thickness of n layers of material overlying the aquifer, h_{p} the pressure head in the aquifer (see Section 5.2.1) and γ_{w} the unit weight of water. Blowout considerations may necessitate pumping of underlying aquifers to reduce water pressure, $h_{\text{p}}\gamma_{\text{w}}$ (and hence to ensure an adequate factor of safety against blowout) during excavation of a cell, placement of the barrier systems and the waste.

For example, consider the geometry shown in Figure 15.2. Assuming a unit weight of the clay till of $\gamma_{\text{t}} = 21\,\text{kN/m}^3$, of the engineered barrier system of $\gamma_{\text{e}} = 20\,\text{kN/m}^3$ and of the waste of $\gamma_{\text{ws}} = 6\,\text{kN/m}^3$, the maximum pressure head,

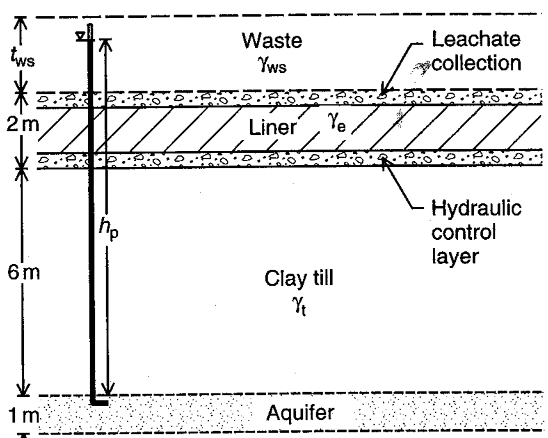

Figure 15.2 Schematic of natural soil, liner system and waste used for example blowout calculation.

h_{p}, in the aquifer for a factor of safety against blowout of $FS_{\text{bh}} = 1.4$ may be calculated using equation 15.3 for conditions during construction. The most critical condition with respect to blowout occurs after excavation of the till to the base of the landfill but before placing the engineered system, and the maximum allowable pressure head is equal to:

$$h_{\text{p}} = \frac{1}{\gamma_{\text{w}}}\left(\frac{6\gamma_{\text{t}}}{FS_{\text{bh}}}\right) = \frac{1}{9.8}\left(\frac{6 \times 21}{1.4}\right) = 9.2\,\text{m}$$

Placement of the 2-m thick engineered barrier system will increase the resistance to blowout; thus, the maximum allowable pressure head increases to:

$$h_{\text{p}} = \frac{1}{\gamma_{\text{w}}}\left(\frac{6\gamma_{\text{t}} + 2\gamma_{\text{e}}}{FS_{\text{bh}}}\right) = \frac{1}{9.8}\left(\frac{(6 \times 21) + (2 \times 20)}{1.4}\right) = 12.1\,\text{m}$$

Assuming that the natural pressure head in the aquifer h_{p} is 14 m, one could then estimate the thickness of waste t_{ws} that must be placed before pumping could be terminated and the aquifer is allowed to return to natural conditions from equation 15.3 as:

$$t_{\text{ws}} = \frac{FS_{\text{bh}} h_{\text{p}} \gamma_{\text{w}} - 6\gamma_{\text{t}} - 2\gamma_{\text{e}}}{\gamma_{\text{ws}}} = \frac{(1.4 \times 14 \times 9.8) - (6 \times 21) - (2 \times 20)}{6} = 4.35\,\text{m}$$

In this case, 4.35 m of waste would need to be placed in order to have a factor of safety against blowout of 1.4 with the natural hydraulic conditions in the underlying aquifer.

Basal stability requires special attention in areas where there is excavation into soil containing dissolved gas as illustrated by the cases reported by Rowe *et al.* (2002a). After excavation to a depth of about 25 m (in a 38–40-m thick clay deposit), venting of gas and water occurred at three separate locations. Additional details are given in Section 9.2.2.

15.2.7 Summary

Numerous landfills are safely constructed without stability problems. However, the issues discussed in this section illustrate the need for careful consideration to be given to the potential for instability during: (a) barrier construction, (b) placement of the waste, (c) the period of time after landfill closure and (d) expansion of existing landfills. The likelihood of failure occurring can be minimized by:

1. a proper geotechnical investigation of the subsoil properties;
2. carefully considering all potential failure mechanisms;
3. avoiding optimism regarding geotechnical properties;
4. taking account of the effect of a potential increase moisture content (e.g., due to leachate recirculation or injection of liquid waste) in increasing the unit weight of the waste and decreasing the shear strength of the waste and interfaces;
5. appropriate design and material selection (including appropriate laboratory tests and stability analyses);
6. good CQC/CQA to ensure that the barrier system is installed as designed;
7. taking account of the effect of excavation on stability (e.g., at the toe of existing waste);
8. development plans for expanded landfills that limit toe excavation and overfilling and define allowable conditions for construction of the expansion area and a means of monitoring adherence to the development plans;
9. operation plans that include consideration of stability as the waste is placed and means of monitoring adherence to the operation plans;
10. avoiding co-disposal of liquid waste or increasing the amount of liquid waste without fully assessing the potential impact on both stability and geoenvironmental protection;
11. contingency plans in the event of changed conditions occurring during construction (e.g., excessive rain, unexpected foundation conditions, etc.); and
12. disposal alternatives so that waste can be diverted if expansion schedules are not met.

15.3 Design of geotextiles

Geotextiles have found widespread use in modern waste containment facilities. They may be used as separators, filters, protection systems for geomembranes, reinforcement for soil and/or waste, or for drainage. Description of the different types and engineering properties of geotextiles can be found elsewhere (e.g., Koerner, 1998). The objective of this section is to discuss the design of geotextiles based on their intended function as separators and filters.

15.3.1 Geotextiles as separators

Geotextiles are used to separate dissimilar materials in the leachate collection system. For example, as shown in Figure 2.3, they may be used to separate waste from the leachate collection gravel, different gravels in the leachate collection system, and/or leachate gravel from an underlying clay liner. The design requirements of the separator geotextile between the waste and leachate collection system were discussed in Section 2.4.6. In all of these cases, the separator geotextile must have adequate strength to minimize damage during construction.

A separator geotextile is also required for barrier designs involving a secondary leachate collection system (SLCS) (or hydraulic control layer) beneath a primary CCL as shown in Figures 16.8, 16.10 and 16.11. This geotextile must also have adequate strength to survive construction, but in addition must have sufficient strength to span the voids in the underlying gravel when subject to overburden pressures