# Designing with Geosynthetics

Fifth Edition

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If fines (silts and/or clays) are allowed for the reinforced zone backfill soil, any possible water in front, behind, and beneath the reinforced zone must be carefully collected, transmitted, and discharged. Proper drainage control is absolutely critical in this regard. Furthermore, the top of the zone should be waterproofed—for example, by a geomembrane or a geosynthetic clay liner—to prevent water from entering the backfill zone from the surface. Surface water drainage as well as drainage from the retained earth zone is obviously of concern with respect to potential buildup of pore water pressures behind or within the reinforced soil zone. (See Koerner and Soong [46] for wall drainage system designs in this regard.)

In closing this section on geogrid reinforced walls, the current tendency to create live (or evergreen) walls with open facing should be mentioned. As we saw earlier in Figure 3.14, the sequence is a steel wire mesh (alternatively a gabion), backed by a bidirectional geogrid and then by a geosynthetic erosion control material. The reinforcing geogrids (always unidirectional types) are either attached to the steel wire mesh facing, or they are frictionally connected by sufficient overlap length. Such walls avoid masonry block durability concerns and offer a considerably less expensive wall system. Of course, the durability of the steel wire and bidirectional geogrid backup must be considered and this is a viable research topic when considering 100-year permanent wall lifetimes.

### 3.2.6 Foundation and Basal Reinforcement

Geogrids have been used to increase bearing capacity of poor foundation soils in different ways: as a continuous layer, as multiple closely spaced continuous layers with granular soil between layers, and as mattresses consisting of three-dimensional interconnected cells. The technical database for the single-layer continuous sheets has been reported by Jarrett [47] and by Milligan and Love [48]; in both cases large-scale laboratory tests are used. Figure 3.19 presents some of Milligan and Love's work graphed in the conventional nondimensionalized $q/c_u$ versus $\rho/B$ manner and also as $q/\sqrt{c_u}$ versus $\rho/B$ where $q$ is the bearing capacity and $\rho$ is the settlement. The latter graph is not conventional but does sort out the data nicely. Clearly shown in both instances is the marked improvement in load-carrying capacity using geogrids at high deformation and only a nominal beneficial effect at low deformation. Beyond these observations, a precise design formulation is not currently available.

Instead of focusing on a global increase in bearing capacity, it is quite likely that single or multiple layers of geogrid (or geotextile) will aid in minimizing or eliminating differential settlement. Here localized settlements due to abruptly settling or subsiding weak zones can be spanned by the layer of reinforcement. This is known as *foundation improvement* (rather than bearing capacity via base reinforcement). Notable in this regard is a technique called *piggybacking*—the construction of new landfills above existing landfills. The approach is to use arching theory in the calculation of the vertical stress arising from localized subsidence (i.e., differential settlement) and to provide suitably strong reinforcement.

It should be recognized that arching in natural soils overlying a locally yielding foundation is well established. In the 1930s, both Karl Terzaghi in Austria (calculating

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$
$$= \frac{13.8 + \sqrt{(-13.8)^2 - 4(6.1)(2.22)}}{2(6.1)}$$
$$FS = 2.10$$

While the value appears to be acceptable, it is nevertheless disconcerting that the liner system per se is being used as the veneer reinforcement mechanism. Had higher reduction factors been used, the resulting FS value would be proportionately decreased. That said, when the solid waste is placed against the leachate collection soil, a resisting berm is created, bringing stability to the situation at that time.

### 5.6.11 Access Ramps

For below-grade landfills it is necessary to grade the subgrade to accommodate the necessary access ramp(s), line the entire facility, and then construct a road above the liner cross section. A typical geometry is shown in Figure 5.46a. A particularly troublesome aspect of this design is that the road must be built above the completed liner system. A variety of problems have occurred in the past:

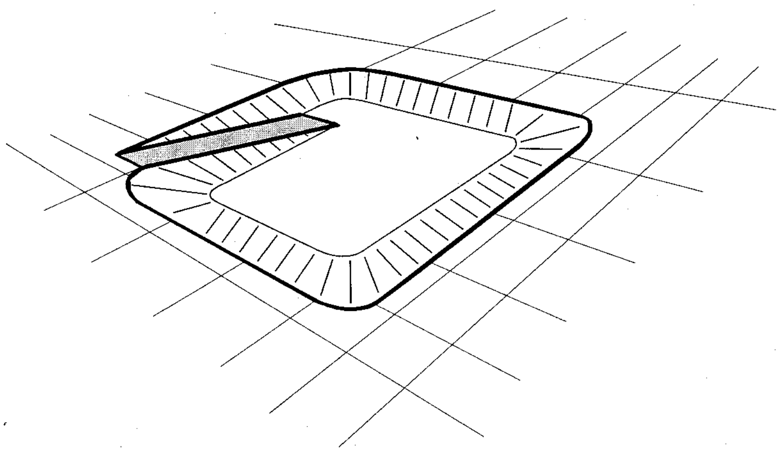
- Inadequate drainage where the ramp meets the upper slope, with subsequent erosion and scour of the roadway itself.
- Inadequate roadway material above the liner system, with ramp soil sliding off the upper geomembrane due to truck traffic.
- Inadequate roadway thickness above the liner system, with the upper geomembrane failing in tension along the slope due to truck traffic.
- Inadequate roadway thickness above the liner system, with an underlying hydrated GCL creating slippage of the overlying geomembrane and entire roadway.

Clearly, a conservative design is required; Figure 5.46b presents some recommendations. While a 600 to 900 mm thickness might seen excessive, the dynamic stresses caused by braking trucks are high, and furthermore, the ramp soil can be removed in whole or in part as the waste elevation rises during filling operations.

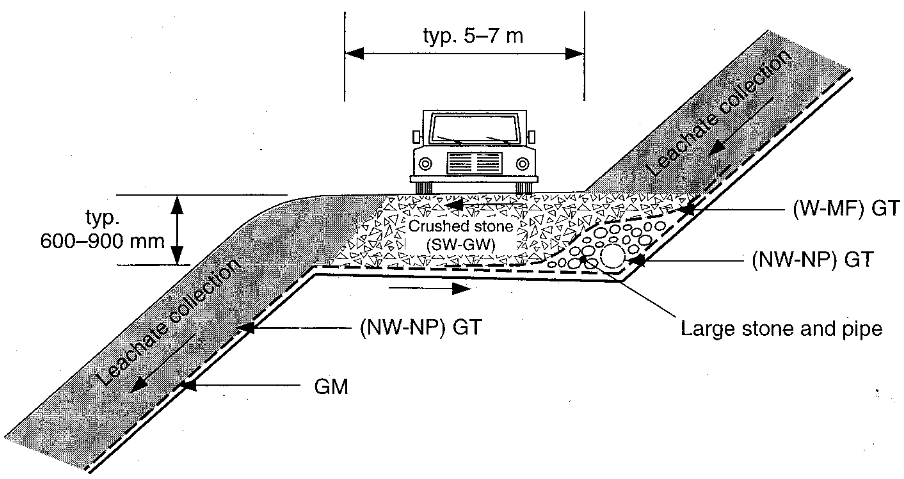
### 5.6.12 Stability of Solid-Waste Masses

Upon first consideration, the stability of solid waste failing within itself should present no particular concern since its shear strength characteristics should be quite high. Singh and Murphy [83] present shear strength parameters of solid waste transitioning from high in friction (24 to 36°) to being high in cohesion (80 to 120 kPa). Obviously, the aging of the waste is an issue, but at all times the shear strength is quite high. A widely used MSW shear strength evelope assembled by Kavazanjian [84] indicates a bilinear response of 33° friction transitioning at less than 30 kPa normal stress to a cohesion of 24 kPa.

Paradoxically, there have been some massive failures of solid waste. Koerner and Soong [85] report on ten such failures of which half were unlined or soil-lined sites, and



(a) Geometry of typical ramp-grades from gentle to 25% (14.0°)




(b) Suggested cross section

**Figure 5.46** Typical geometry and cross section of a below-grade landfill access ramp.

half were at sites that contained geomembranes. Table 5.19a presents some details which were evaluated on the basis of both two-dimensional and three-dimensional analyses. On average the 3-D analyses were 16% higher than the comparable 2-D analyses. The 2-D representations of the individual failures are shown in Figure 5.47. Figure 5.48 shows the enormity of the problem at one of these sites. All of the failures were most dramatic and many involved litigation and fines, to say nothing of the deaths at one site and the environmental damage that ensued at all of the sites. The failure surfaces were either rotational or translational, the latter always occurring at the geomembrane-lined sites. Commercially available slope stability computer codes are

**TABLE 5.19** SUMMARY OF LARGE LANDFILL FAILURES AND RELATED TRIGGERING MECHANISMS INVOLVED

**(a) Site Listings and Related Information**

| Identification | Year | Location | Type | Quantity of Waste Involved (m$^3$) |
|---|---|---|---|---|
| Unlined or soil-lined sites | | | | |
| U-1 | 1984 | North America | Single rotational | 110,000 |
| U-2 | 1989 | North America | Multiple rotational | 500,000 |
| U-3 | 1993 | Europe | Translational | 470,000[1] |
| U-4 | 1996 | North America | Translational | 1,100,000 |
| U-5 | 1997 | North America | Single rotational | 100,000 |
| Geomembrane-lined sites | | | | |
| L-1 | 1988 | North America | Translational | 490,000 |
| L-2 | 1994 | Europe | Translational | 60,000 |
| L-3 | 1997 | North America | Translational | 100,000 |
| L-4 | 1997 | Africa | Translational | 300,000 |
| L-5 | 1997 | South America | Translational | 1,200,000 |

[1]Included 27 deaths!

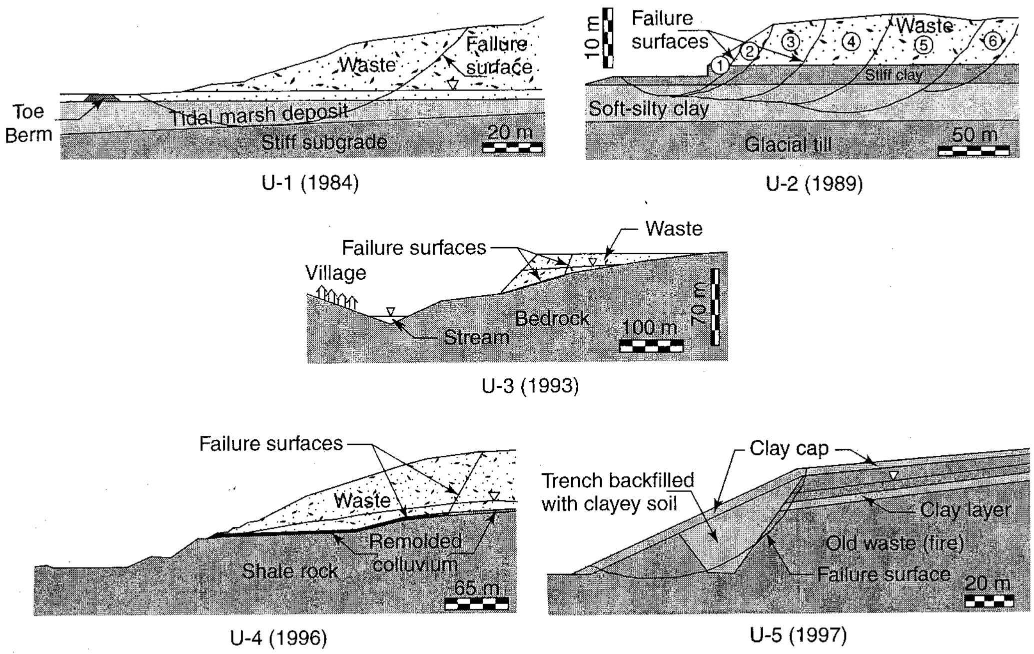
**(b) Contributing Cause (Trigger) of Failures**

| Case History | Reason for Low Initial FS Value | Triggering Mechanism |
|---|---|---|
| U-3 | Leachate buildup within waste mass | Excessive buildup of leachate level due to ponding |
| U-4 | | Excessive buildup of leachate level due to ice formation |
| L-4 | | Excessive buildup of leachate level due to liquid waste injection |
| L-5 | | Excessive buildup of leachate level due to leachate injection |
| L-1 | Wet clay beneath GM (i.e., GM/CCL or GM/GCL) | Excessive wetness of the GM/CCL interface |
| L-2 | | Excessive wetness of the GM/CCL interface |
| L-3 | | Excessive wetness of the bentonite in an unreinforced GCL |
| U-1 | Wet foundation or soft backfill soil | Rapid rise in leachate level within the waste mass |
| U-2 | | Foundation soil excavation exposing soft clay |
| U-5 | | Excessive buildup of perched leachate level on clay liner |

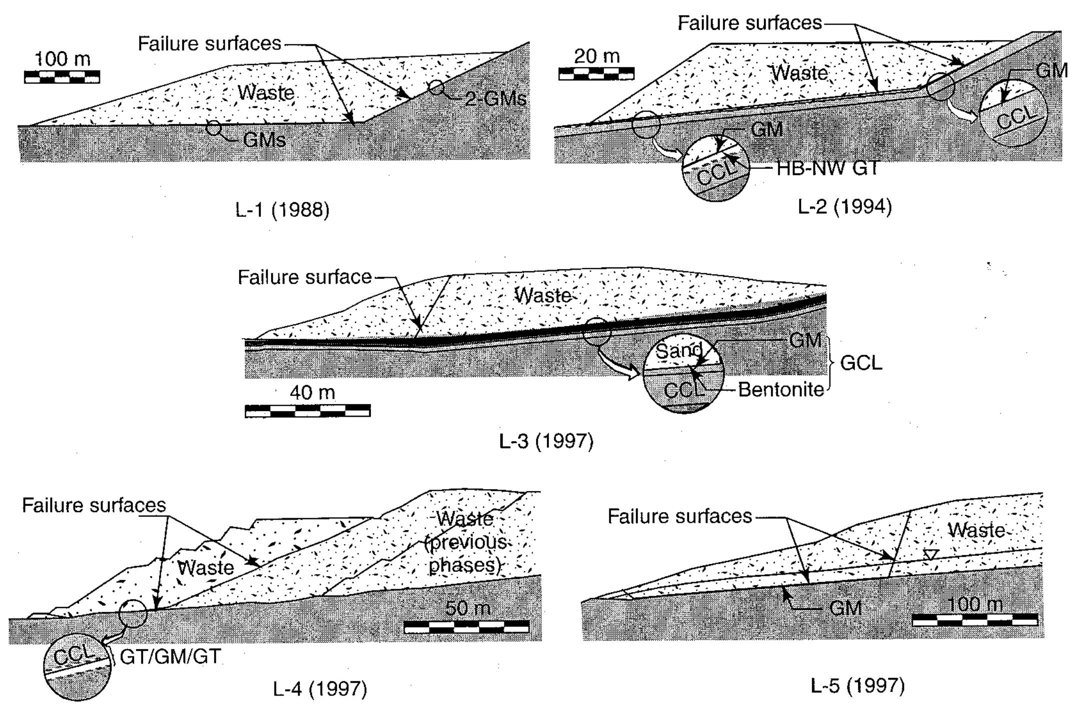
*Source*: After Koerner and Soong [85].

readily configured to handle these failures provided that accurate values of shear strength of the material and surfaces involved are known. The importance of direct shear testing (as described in Section 5.1.3) cannot be overstated.

While the stability factors of safety of all of the sites were relatively low prior to failure, each had a unique aspect that Koerner and Soong [85] call a *triggering mechanism*. It was found that all ten failures had triggering mechanisms that involved liquids. Table 5.19b groups the failures according to triggering mechanisms where the excessive liquids are either (1) in the waste mass itself above the liner system, (2) within

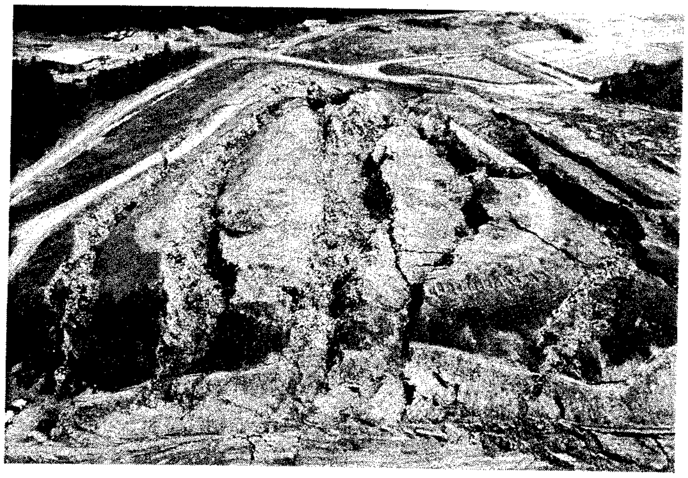



(a) Cross sections of unlined or low permeability soil-lined landfill failures

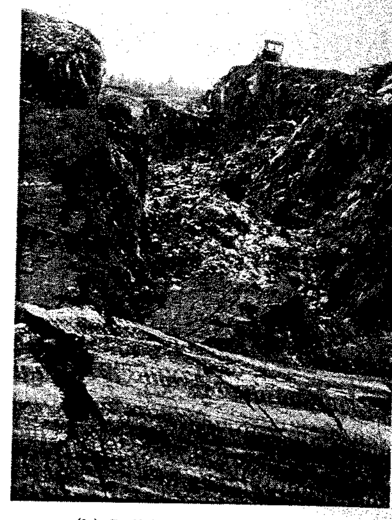



(b) Cross sections of geomembrane-lined landfill failures

**Figure 5.47** Two-dimensional cross sections of ten landfill failures. (After Koerner and Soong [85])



(a) Six individual failures which occurred sequentially within minutes of one another



(b) Solid waste within one of the failures

**Figure 5.48** Failure of a municipal solid-waste landfill within the waste mass itself.

components of the liner system in the form of excessively wet CCLs or GCLs, or (3) in the foundation soil beneath the waste and/or liner system. This recognition of the negative influence of liquids on waste mass stability cannot be overemphasized. Of all of the problems mentioned in this book, this class of failures is the most serious and must be avoided at all costs.

### 5.6.13 Verital Expansion (Piggyback) Landfills

In closing this section on geosynthetic systems related to solid waste, the concept of vertical expansions—*piggybacking* a new landfill on an existing one—should be mentioned. When many existing landfills are filled, there is nowhere else to go but up. Thus a new landfilling operation above an existing one sometimes becomes necessary. As noted in Qian et al. [86], certain precautions regarding this type of vertical expansion must be followed:

- Total settlement of the existing landfill must be anticipated and estimated accordingly. Thus, the slopes of the leachate collection system must reflect this requirement and will probably be quite high, as much as 10 to 15%.
- Estimation of differential settlements within the existing landfill may require a high-strength geogrid or geotextile network to be placed over all or a portion of the site (recall Section 3.2.6 and Example 3.11).
- Waste placement in the new landfill must be carefully sequenced to balance stress on the existing landfill [86]. The stability of the waste situation just discussed

is exacerbated greatly by the addition of a large surcharge stress, which is what the piggybacked landfill represents to the underlying waste.

- Methane gas (if generated) migrating from the existing landfill must be carried laterally under the new landfill liner to side-slope venting and/or collection locations. Active gas collection systems may be required.
- Leachate collection from the existing landfill should be considered. If required, directionally drilled withdrawal wells at the perimeter of the facility may be a consideration.
- Access to the site via haul roads must be carefully considered so that there will be no damage to, or instability of, the underlying liner system.

### 5.6.14 Heap Leach Pads

Heap leach pads consist of a geomembrane with an overlying drainage system, and then a precious metal (gold, silver, or copper) bearing ore heaped above. A cyanide or sulfuric acid solution is sprayed on top of the ore, leaches through it reacting with the metals, and carries the solution to the drainage system where it is collected. Beneath the drainage system is a geomembrane barrier, hence the topic is included at this location. Separation of the ore from the leachate occurs in an on-site processing plant. The leaching solution is renewed and the process is repeated until it is no longer economical. Figure 5.49a illustrates the general configuration.

The heap itself is often enormous in its proportions (see Figure 5.49b). Ores of 22 $kN/m^3$ unit weight at heights up to 150 m produce enormous stresses on the drainage system and geomembrane. The concept is used widely in the western United States and Canada and in many South American countries (see Smith and Welkner [88]).

Regarding the design of the geomembrane, its thickness and type is very subjective and all resin types have been used to varying degrees. The drainage system is coarse gravel along with an embedded pipe system allowing for rapid and efficient removal of the ore-bearing solution from beneath the heap. This situation requires consideration of a sand cushion layer or a very thick protection geotextile between the geomembrane and drainage/collection gravel. The design method presented in Section 5.6.7 should be considered, with the reminder that it is developed on the basis that different geomembrane thicknesses and types will behave differently. Thiel and Smith [89] have summarized the key geotechnical concerns with respect to heap leach pads and related issues (see Table 5.20).

### 5.6.15 Solar Ponds

There are a number of solid material liner systems that have not yet been mentioned. A small but growing segment of these systems is solar ponds [90]. Here the geomembrane is placed in an excavation and then it is filled with salt. Solar energy is collected and stored as heat. A salt gradient effect is created, whereby zones are set up constantly replenishing new heat as it is gradually withdrawn from the lower storage zone for useful purposes. The main consideration insofar as the geomembrane is concerned is

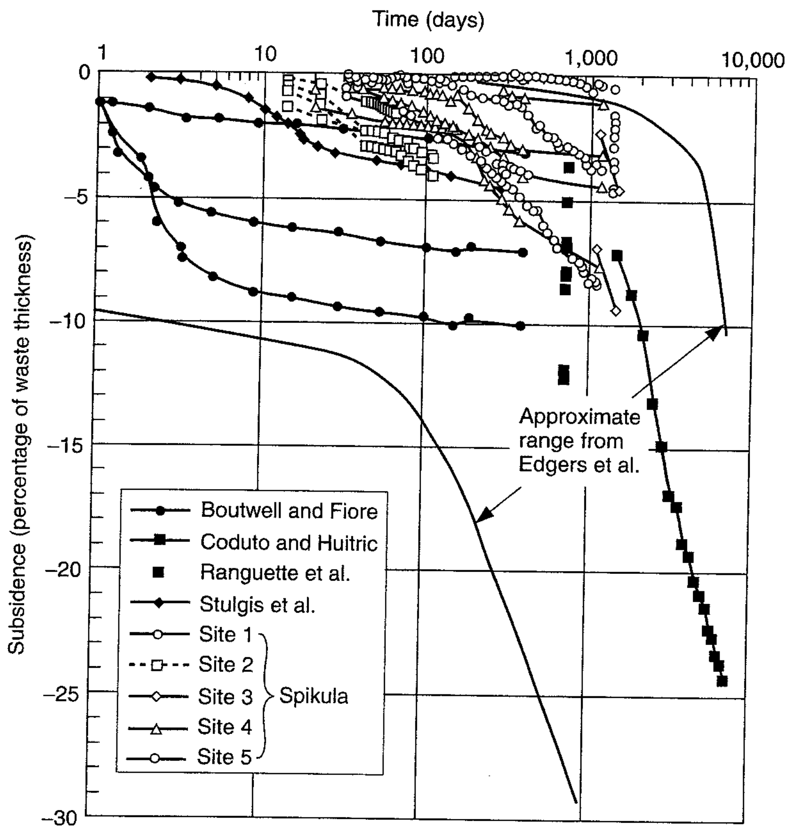



**Figure 5.51** Municipal solid-waste landfill subsidence. (After Spikula [92])

- The geomembrane barrier above the compacted clay should have a minimum thickness of 0.75 mm.
- There should be adequate bedding above and below the geomembrane.
- The drainage layer above the geomembrane should have a minimum hydraulic conductivity of 0.01 cm/s and a final slope of 2% or greater after settlement and subsidence (thus necessitating subsidence predictions).
- The topsoil and protection soil above the drainage layer must have a minimum thickness of 600 mm.

As seen in Figure 5.50, there are many geosynthetic alternatives to the above-mentioned natural soils, for example:

- The CCL should be replaced by a GCL. (CCLs simply do not belong above a subsiding waste mass resulting in total and differential settlement.)
- The drainage layer could be replaced by a geocomposite or geonet drain.