Chapter 6
Geotechnical Analysis And Design

6.1 Introduction

This chapter provides information on select topics related to cover system geotechnical analysis and design. The specific topics discussed in this chapter are:

- static slope stability (Section 6.2);
- seismic slope stability and deformation (Section 6.3);
- settlement (Section 6.4);
- steep slopes (Section 6.5); and
- soft waste materials (Section 6.6).

6.2 Static Slope Stability

6.2.1 Overview

Slope stability is a critical issue in the design of cover systems. Slopes on landfills, waste piles, and other waste containment structures are sometimes quite steep. Sideslope inclinations can range from flatter than 5H:1V (11.3°) to steeper than 2H:1V (26.6°). For example, cover systems have been constructed over waste slopes steeper than 1.5H:1V (33.7°) as part of the remediation of old dumps. This section of the guidance document addresses issues associated with the static slope stability of cover system components. Both internal and interface downslope sliding of one or more components are considered. Failure surfaces that extend into the waste are not addressed herein, but should be considered in slope stability analyses. Special stability issues associated with cover system sideslopes steeper than about 2.5H:1V and cover systems installed over soft waste materials are discussed in Sections 6.5 and 6.6, respectively.

The frequency of occurrence of cover system stability problems has been high. More than a dozen case studies of past problems of this nature are described by Gross et al. (2002) and briefly discussed in Section 7.4 of this guidance document. One example of a cover system stability problem is shown in Figure 6-1. The photographs in this figure show a topsoil surface/protection layer that has slid downslope over a reinforced GCL barrier in a cover system that did <u>not</u> contain an internal drainage layer. Figure 6-2 shows another example, this one involving a topsoil surface/protection layer and underlying sand drainage layer that has slid over a textured HDPE GM barrier. In this case, the sand drainage layer (specified hydraulic conductivity of 1×10^{-5} m/s) had inadequate flow capacity and the drainage layer outlets were constricted. With GMs, GCLs, CCLs, GTs, and GCs commonly used in a variety of cover system configurations, the stability of potential low shear strength materials and interfaces must be considered for most

Figure 6-1. Example of Cover System Slope Stability Problem. The Topsoil Surface/Protection Layer Slid Downslope Over the Reinforced GCL Barrier.

Figure 6-2. Example of Cover System Slope Stability Problem. The Topsoil Surface/ Protection Layer and Underlying Sand Drainage Layer Slid Downslope Over the Textured HDPE GM Barrier.

designs. Significantly, past failures have involved sliding along each of the geosynthetic interfaces listed in Table 6-1.

Table 6.1 Interfaces upon which cover system components have undergone sliding.

• Topsoil surface/protection layer sliding on: GT GM GCL CCL • Sand drainage layer sliding on: GT GM GCL CCL	• GN drainage layer sliding on GM • GC drainage layer sliding on GM • GT sliding on GM • GM sliding on: GT GCL CCL • GCL sliding on: CCL prepared subgrade

6.2.2 Limit Equilibrium Analyses

6.2.2.1 Overview

The simplest limit equilibrium (LE) formulation to analyze the slope stability of cover systems assumes infinite slope conditions and neglects the stabilizing influences of passive soil resisting forces at the toe of the slope, any true cohesion/adhesion in cover system materials and interfaces, and tension in the geosynthetic layers. More sophisticated LE formulations account for these factors. Both the infinite slope and more sophisticated LE formulations are discussed below. In all of the closed-form, two-dimensional LE solutions, force equilibrium is satisfied in the directions normal and parallel to the slope, but moment equilibrium is ignored.

6.2.2.2 Infinite Slope

For cover system geometries where the cover soil thickness is constant, infinite slope equations provide a simple and conservative basis for design. Equations can be formulated in terms of: (i) total unit weights of the cover system materials and boundary water pressures; or (ii) buoyant unit weights and body seepage (or drag) forces. In keeping with the approach of Giroud et al. (1995a), equations are formulated herein using buoyant unit weights and seepage forces.

Body seepage forces occur in cover systems when water infiltrating the cover system develops a significant flow component in the downslope as opposed to vertical downward direction. This occurs, for example, when infiltration is blocked by a hydraulic barrier. If the rate of infiltration is sufficient, hydraulic head will build up above the barrier layer and induce downslope flow. Downslope flow of water has a destabilizing effect on the cover system. The seepage force per unit volume on soil particles in the direction of laminar flow is expressed as:

$$f_w = \gamma_w i \qquad \text{(Eq. 6.1)}$$

where: f_w = seepage force per unit volume (N/m^3); γ_w = unit weight of water (N/m^3); and i = hydraulic gradient (dimensionless). The concept of a seepage force, F_w (N) (acting parallel to the slope), and buoyant unit weight, W_b (N) (acting vertically), in an infinite soil slope underlain by a hydraulic barrier is illustrated in Figure 6-3. For a 1-m thick cover system at a 3H:1V (18.4°) slope and with water flowing in the entire soil thickness, the water induces a downslope body seepage force of 3 kPa.

If there is no water flow in an infinite slope, the slope stability factor of safety is given by (Giroud et al., 1995a):

$$FS = \frac{\tan\phi_i}{\tan\beta} + \frac{a_i}{\gamma_t t \sin\beta} \qquad \text{(Eq. 6.2)}$$

where: FS = factor of safety (dimensionless); ϕ_i = angle of internal or interface friction for the critical potential slip surface (degrees); a_i = adhesion (for an interface) or cohesion (for internal strength) for the critical potential slip surface (Pa); β = slope angle (degrees); γ_t = total unit weight of material above the critical potential slip surface (N/m^3); and t = thickness of material above the critical potential slip surface (m). Use of this equation assumes that there is a unique critical potential slip surface in the cover system. For the case of no adhesion or cohesion (a_i = 0), Eq. 6.2 reduces to the classical solution:

$$FS = \tan\phi_i / \tan\beta \qquad \text{(Eq. 6.3)}$$

For a hydraulic barrier system, two conditions need to be considered: (i) stability above the hydraulic barrier; and (ii) stability below the hydraulic barrier. These two conditions must be considered because effective stresses above and below a non-porous hydraulic barrier, such as a GM, are different. The infinite slope factor of safety for "full flow" (t_w = thickness of water flow parallel to the slope = t in Figure 6-3) parallel to the slope along an internal or interface slip surface <u>above</u> the hydraulic barrier is (Giroud et al., 1995a):

$$FS_A = \frac{\gamma_b}{\gamma_{sat}} \frac{\tan\phi_a}{\tan\beta} + \frac{a_a}{\gamma_{sat}\, t \sin\beta} \qquad \text{(Eq. 6.4)}$$

where: FS_A = factor of safety for critical potential slip surface above the hydraulic barrier (dimensionless); ϕ_a = angle of internal or interface friction for the critical potential slip surface above the hydraulic barrier (degrees); a_a = cohesion (for internal strength) or adhesion (for an interface) for the critical potential slip surface above the hydraulic barrier (Pa); γ_b = average buoyant unit weight of material above the critical potential slip surface (N/m^3); and γ_{sat} = average saturated unit weight of material above the critical potential slip surface (N/m^3); and all other terms are as defined previously. The buoyant unit weight, γ_b, is equal to total unit weight, γ_t, minus the unit weight of water, γ_w.

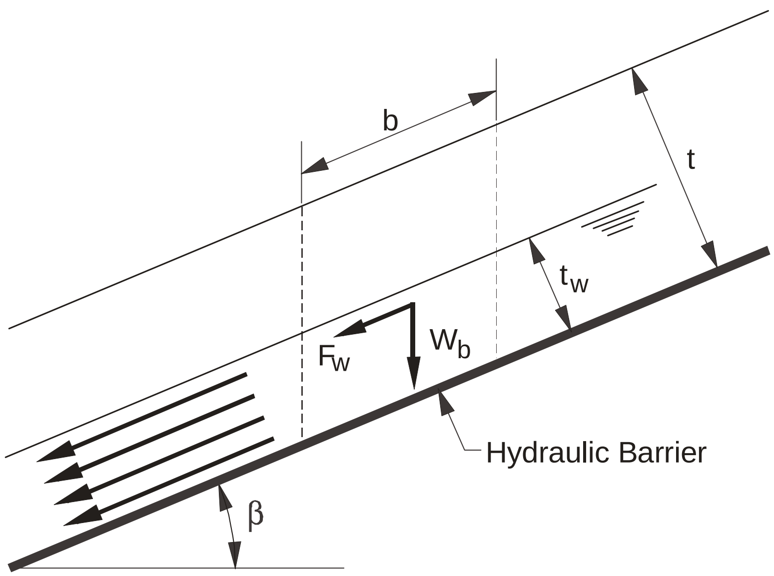


Figure 6-3. Seepage Force and Buoyant Unit Weight for a Soil Layer Overlying a Hydraulic Barrier on an Infinite Slope (modified from Giroud et al, 1995a).

This factor of safety can be compared with the factor of safety expressed by Eq. 6.2 for the case of no water flow. The comparison shows that for typical soils:

- $FS_{A\ full\ flow} / FS_{no\ flow} \approx 0.5$ if $a_i = 0$; and
- $FS_{A\ full\ flow} / FS_{no\ flow} \approx 0.9$ if $\phi_i = 0$.

Based on these results, for slip surfaces located above the hydraulic barrier, the factor of safety can decrease by a factor of two due to water flow parallel to the slope if shearing resistance is generated primarily through friction.

The factor of safety ratios presented above are based on the assumption that the shear strength properties, ϕ_a and a_a, are not influenced by the presence of water. If the presence of water reduces the magnitudes of these parameters, the effects noted in the above comparison would be even more substantial.

The infinite slope factor of safety for "full flow" parallel to the slope along an internal or interface slip surface below a non-porous hydraulic barrier is given by (Giroud et al., 1995a):

$$FS_B = \frac{\tan\phi_b}{\tan\beta} + \frac{a_b}{\gamma_{sat}\, t \sin\beta} \qquad \text{(Eq. 6.5)}$$

where: FS_B = factor of safety for critical potential slip surface below the hydraulic barrier (dimensionless); ϕ_b and a_b are the internal or interface shear strength parameters for the critical potential slip surface below the hydraulic barrier; and all other terms are as defined previously. It should be noted that the shear strength parameters ϕ_b and a_b, used in Eq. 6.5, will typically be different than the parameters ϕ_a and a_a, used in Eq. 6.4, as the interfaces in the two equations are different. This factor of safety is to be compared with the factor of safety expressed by Eq. 6.2 for the case of no water flow. The comparison shows that for typical soils:

- $FS_{B\ \text{full flow}} / FS_{\text{no flow}} = 1$ if $a_i = 0$; and
- $FS_{A\ \text{full flow}} / FS_{\text{no flow}} \approx 0.9$ if $\phi_i = 0$.

Based on these results, the factor of safety along critical potential slip surfaces below the hydraulic barrier is only affected to a relatively minor degree by water flow above the hydraulic barrier.

The final infinite slope case to be considered is for "partial-depth" flow ($t_w < t$ in Figure 6-3) parallel to the slope. The appropriate equations are (Giroud et al., 1995a):

$$FS_A = \left(\frac{\gamma_t (t - t_w) + \gamma_b t_w}{\gamma_t (t - t_w) + \gamma_{sat} t_w}\right)\frac{\tan\phi_a}{\tan\beta} + \frac{a_a / \sin\beta}{\gamma_t (t - t_w) + \gamma_{sat} t_w} \qquad \text{(Eq. 6.6)}$$

and

$$FS_B = \frac{\tan\phi_b}{\tan\beta} + \frac{a_b / \sin\beta}{\gamma_t (t - t_w) + \gamma_{sat} t_w} \qquad \text{(Eq. 6.7)}$$

where: t_w = thickness of water flow parallel to the slope (m), as defined in Figure 6-3, and all other terms are as defined previously.

Based on the foregoing equations, the effect of water flow on the stability of a cover system is much greater if the slip surface is above the hydraulic barrier than if it is below the hydraulic barrier. The reasons for this can be summarized as follows:

- The main effect of water flowing downslope within a cover system slope is the significant decrease in the effective normal stress above the hydraulic barrier.
- Other effects of water flowing downslope within a cover system are a slight increase in the effective normal stress below the hydraulic barrier layer and a slight increase in the shear stress above and below the hydraulic barrier.
- As a result of the changes in effective normal stress, the frictional component of shear strength decreases significantly above the hydraulic barrier but decreases only slightly below the hydraulic barrier.
- As a result of the changes in shear strength and the slight increase in shear stress, the factor of safety is significantly affected above the hydraulic barrier and only mildly affected below the hydraulic barrier.

It can also be inferred from the above assessment that waste-generated gases beneath a cover system effect the stability of the interface between a non-porous hydraulic barrier and an underlying material by decreasing the frictional component of shear strength along the interface while the shear stress along the interface remains unchanged. This is one reason why gases may need to be collected and controlled via a gas collection layer, gas wells, or other means. One example of a cover system stability problem caused by gas pressures is described in Section 7.7. Briefly, gas generated in a MSW landfill uplifted the GM barrier of a cover system and resulted in the GM and overlying materials moving downslope over a GT. Though the landfill had vertical gas extraction wells, the upper portions of the wells were not perforated. As a consequence, gas accumulated beneath the cover system, generating uplift pressures on the underside of the GM.

6.2.2.3 Slope of Finite Length

Equations for the LE evaluation of sloping geosynthetic-soil layered systems (such as a cover system) for a slope of finite length have been presented by Giroud and Beech (1989), EPA (1991), Koerner and Hwu (1991), McKelvey and Deutsch (1991), Bourdeau et al. (1993), Druschel and Underwood (1993), Giroud et al. (1995a,b), Soong and Koerner (1997), Koerner and Daniel (1997), and Koerner and Soong (1998), among others. The most detailed treatments of the subject have been presented by Koerner and coworkers and Giroud et al. (1995a,b). Giroud et al. (1995b) have shown that compared with the method they present, the method utilized by Koerner and coworkers is more rigorous, but somewhat more complicated to use because it requires solution of quadratic equations. The formulation by Giroud et al. (1995a,b) involves an approximation that allows expression of the factor of safety as a closed-form algebraic equation where each term in the equation has a distinct physical meaning and is sufficiently accurate for practical purposes. The simpler formulation is presented below, but either method is acceptable when properly applied.

The two-part wedge considered by Giroud et al. (1995a,b) is illustrated in Figure 6-4. For this condition, the slope stability factor of safety for a slope with constant soil thickness above the critical potential slip surface and for the case of no water flow ($t_w = 0$ in Figure 6-4) is given by:

$$FS = \frac{\tan\phi_i}{\tan\beta} + \frac{a_i/\sin\beta}{\gamma_t\, t} + \frac{t}{h}\left(\frac{\sin\phi_s}{\sin(2\beta)\cos(\beta+\phi_s)}\right) + \frac{c_s}{\gamma_t h}\left(\frac{\cos\phi_s}{\sin\beta\cos(\beta+\phi_s)}\right) + \frac{T/h}{\gamma_t t} \qquad \text{(Eq. 6.8)}$$

where: ϕ_s = angle of internal friction for the soil material (i.e., protection layer and/or granular drainage layer) above the critical potential slip surface (degrees); c_s = cohesion of soil material above the critical potential slip surface (Pa); h = height of slope (m), as defined in Figure 6-4; T = geosynthetic tension above the potential slip surface (N/m); and all other terms are as defined previously.

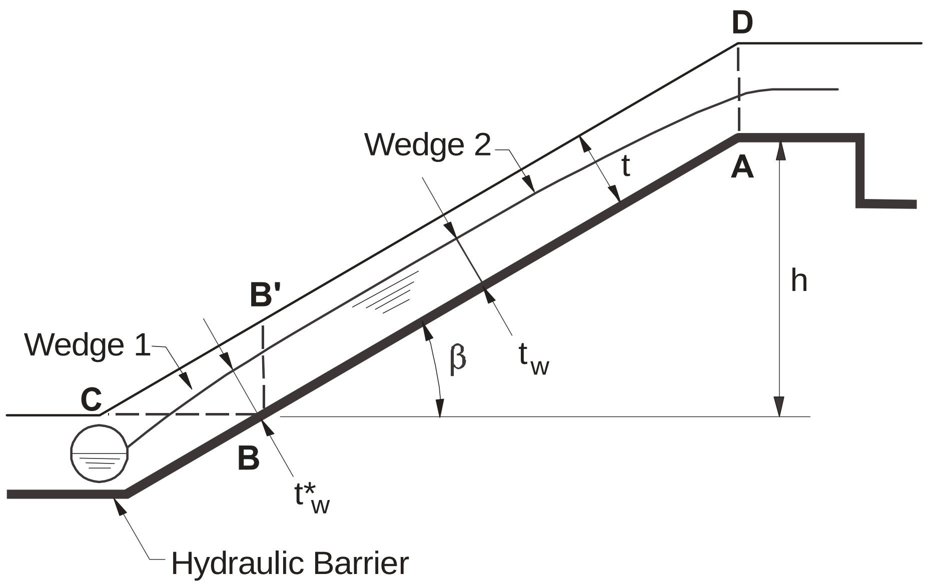


Figure 6-4. Definition of Two-Part Wedge and Flow Thickness for the Case of a Slope of Finite Height (modified from Giroud et al., 1995a).

Eq. 6.8 consists of five terms, each of which has physical significance. The significance of each term is as follows:

- The first term quantifies the contribution of the frictional component of the critical interface or internal shear strength to stability (i.e., the frictional component along line segment AB in Figure 6-4).
- The second term quantifies the contribution of the adhesion component of the critical interface or internal shear strength to stability (i.e., the adhesion component along line segment AB in Figure 6-4).
- The third and fourth terms quantify the contribution of the toe buttressing effect, which results from the shear strength of the soil located at the toe of the slope above the slip surface (i.e., the soil shear strength along line segment BC in Figure 6-4). Both terms depend on the soil internal friction angle, whereas only the fourth term depends on the soil cohesion.
- The fifth term quantifies the contribution to the factor of safety of any tension in the geosynthetics located above the slip surface (which may include one or more

geosynthetics specifically used as reinforcement).

The case of partial-depth and full-depth flow of water for a slope of finite height was addressed by Giroud et al. (1995a). For the case of a slope of uniform thickness above the critical potential slip surface, the factor of safety above the hydraulic barrier may be calculated using the following equation:

$$
\begin{aligned}
FS_A = {} & \left(\frac{\gamma_t(t-t_w)+\gamma_b t_w}{\gamma_t(t-t_w)+\gamma_{sat} t_w}\right)\frac{\tan\phi_a}{\tan\beta} + \frac{a_a/\sin\beta}{\gamma_t(t-t_w)+\gamma_{sat} t_w} \\
& + \left(\frac{\gamma_t(t-t_w^*)+\gamma_b t_w^*}{\gamma_t(t-t_w)+\gamma_{sat} t_w}\right)\left(\frac{t}{h}\right)\left(\frac{\sin\phi_s}{\sin(2\beta)\cos(\beta+\phi_s)}\right) \\
& + \left(\frac{c_s t/h}{\gamma_t(t-t_w)+\gamma_{sat} t_w}\right)\left(\frac{\cos\phi_s}{\sin\beta\cos(\beta+\phi_s)}\right) + \frac{T/h}{\gamma_t(t-t_w)+\gamma_{sat} t_w}
\end{aligned}
\quad \text{(Eq. 6.9)}
$$

where: t^*_w = thickness of water in Wedge 1 (m), as defined in Figure 6-4; and all other terms are as defined previously. For potential slip surfaces below a non-porous hydraulic barrier:

$$
\begin{aligned}
FS_B = {} & \frac{\tan\phi_b}{\tan\beta} + \frac{a_b/\sin\beta}{\gamma_t(t-t_w)+\gamma_{sat} t_w} + \left(\frac{\gamma_t(t-t_w^*)+\gamma_b t_w^*}{\gamma_t(t-t_w)+\gamma_{sat} t_w}\right)\left(\frac{t}{h}\right)\left(\frac{\sin\phi_s}{\sin(2\beta)\cos(\beta+\phi_s)}\right) \\
& + \left(\frac{c_s t/h}{\gamma_t(t-t_w)+\gamma_{sat} t_w}\right)\left(\frac{\cos\phi_s}{\sin\beta\cos(\beta+\phi_s)}\right) + \frac{T/h}{\gamma_t(t-t_w)+\gamma_{sat} t_w}
\end{aligned}
\quad \text{(Eq. 6.10)}
$$

When there is full flow of water in Wedge 1 ($t^*_w = t$) as well as in Wedge 2 ($t_w = t$), Eq. 6.9 gives the following equation for the factor of safety for a critical potential slip surface above the hydraulic barrier:

$$
\begin{aligned}
FS_A = {} & \frac{\gamma_b}{\gamma_{sat}}\left(\frac{\tan\phi_a}{\tan\beta}\right) + \frac{a_a}{\gamma_{sat} t\sin\beta} + \frac{\gamma_b}{\gamma_{sat}}\left(\frac{t}{h}\right)\left(\frac{\sin\phi_s}{\sin(2\beta)\cos(\beta+\phi_s)}\right) \\
& + \frac{c_s}{\gamma_{sat} h}\left(\frac{\cos\phi_s}{\sin\beta\cos(\beta+\phi_s)}\right) + \frac{T/h}{\gamma_{sat} t}
\end{aligned}
\quad \text{(Eq. 6.11)}
$$

and Eq. 6.10 reduces to:

$$
\begin{aligned}
FS_B = {} & \frac{\tan\phi_b}{\tan\beta} + \frac{a_b}{\gamma_{sat} t\sin\beta} + \frac{\gamma_b}{\gamma_{sat}}\left(\frac{t}{h}\right)\left(\frac{\sin\phi_s}{\sin(2\beta)\cos(\beta+\phi_s)}\right) \\
& + \frac{c_s}{\gamma_{sat} h}\left(\frac{\cos\phi_s}{\sin\beta\cos(\beta+\phi_s)}\right) + \frac{T/h}{\gamma_{sat} t}
\end{aligned}
\quad \text{(Eq. 6.12)}
$$

where all terms are as defined previously.

Another case sometimes encountered is that of a tapered cover soil thickness, as illustrated in Figure 6-5. For this geometry, the factor of safety for the case of no water flow is given by the equation:

$$FS = \frac{\tan\phi_i}{\tan\beta} + \frac{t_b}{t_{avg}}\left[\frac{a_i}{\gamma_t t_b \sin\beta} + \frac{t_b}{h}\left(\frac{\sin\phi_s}{\sin(2\beta)\cos(\beta+\phi_s)}\right) + \frac{c_s}{\gamma_t h}\left(\frac{\cos\phi_s}{\sin\beta\cos(\beta+\phi_s)}\right) + \frac{T/h}{\gamma_t t_b}\right] \quad \text{(Eq. 6.13)}$$

where:

$$t_{avg} = (t_a + t_b)/2 \quad \text{(Eq. 6.14)}$$

t_{avg} = average thickness of soil layer between points A and B, which are defined in Figure 6-5 (m); t_a = thickness of soil layer at point A (m), as defined in Figure 6-5; t_b = thickness of soil layer at point B (m), as defined in Figure 6-5; and all other terms are as defined previously.

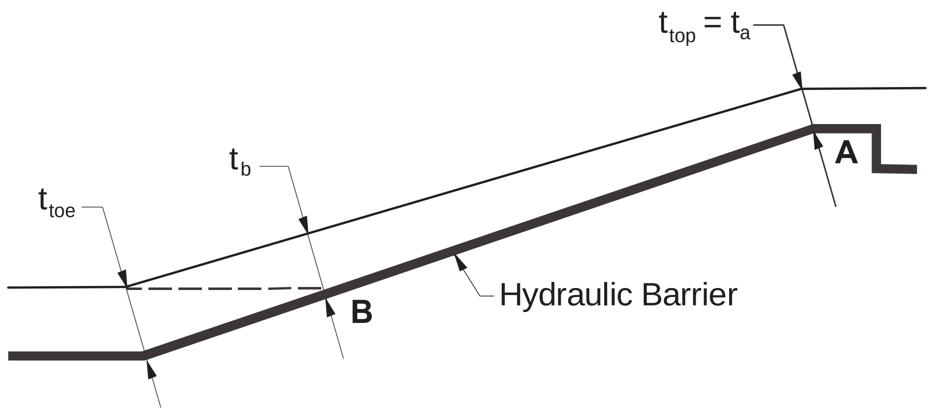


Figure 6-5. Definition of Slope with a Tapered Soil Layer (from Giroud et al., 1995b).

Eq. 6.13 can also be used to calculate the factor of safety for a partly tapered slope of height h, illustrated in Figure 6-6, by calculating an average soil thickness for the entire slope using the equation:

$$t_{avg} = \frac{t_a}{2}\left(1 + \frac{h_u}{h}\right) + \frac{t_b}{2}\left(1 - \frac{h_u}{h}\right) \quad \text{(Eq. 6.15)}$$

where: h_u = height of slope above the slope grade break (m), as illustrated in Figure 6-6, and all other terms are as defined previously.

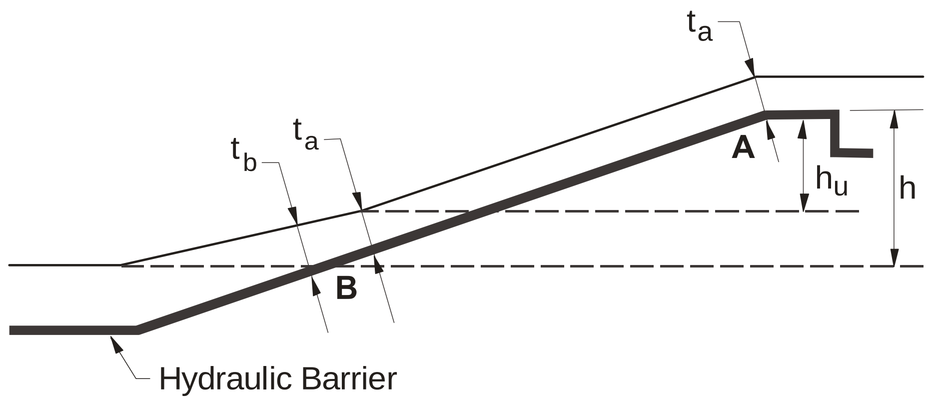


Figure 6-6. Definition of Slope with a Partly Tapered Soil Layer (from Giroud et al., 1995b).

The equations presented above provide closed-form solutions to a variety of cover system slope stability situations. Some situations are too complex, however, to address using closed-form solutions and are more easily evaluated using commercially available two-dimensional slope stability computer software (e.g., PCSTABL5M (Achilleous, 1988), UTEXAS4 (Wright, 1999), XSTABL (Sharma, 1994), or SLOPE/W (available from Geo-Slope Int. Ltd., Alberta, Canada)). Available software has the advantage over closed-form solutions in that it can be applied to non-uniform slope, soil cover, and hydraulic head conditions, and can incorporate a pseudo-static seismic coefficient for use in seismic stability evaluations.

It is noted that the above two-dimensional LE methods are based on a plane-strain condition and do not consider the shear resistance along the two sides of the slide mass that parallel the direction of movement. A two-dimensional analysis, however, is considered appropriate for cover system design because it yields a conservative estimate of the slope stability factor of safety for design geometries encountered in cover systems. The degree of conservatism decreases as the cover system geometry approaches a two-dimensional configuration (i.e., the ratio of cover system slope width to slope length increases). For the majority of cover system geometries, the incremental increase in stability calculated by considering three-dimensional effects will be negligibly small.

The LE method is useful for evaluating cover system stability under most conditions but is subject to several limitations. With the LE method, material and interface shearing resistances are assumed to be independent of displacement. For geosynthetic materials and interfaces, however, mobilized shearing resistance is not constant but increases with increasing displacement to a peak value. For many materials and interfaces, the shear resistance decreases with increasing displacement after reaching the peak, and ultimately reaches a "residual" value (Figure 6-7). This behavior is sometimes referred to as "strain-softening." In using the LE method, judgment must be applied to the selection of shear strength values for strain-softening materials (i.e., peak, residual, or some other value). The LE method is similarly limited with respect to tension forces in cover system geosynthetic components and, therefore, cannot be used to estimate the magnitude and distribution of stresses and deformations in these components. These limitations of the LE method can be overcome by using another slope stability evaluation method, stress-deformation analyses discussed below in Section 6.2.3.

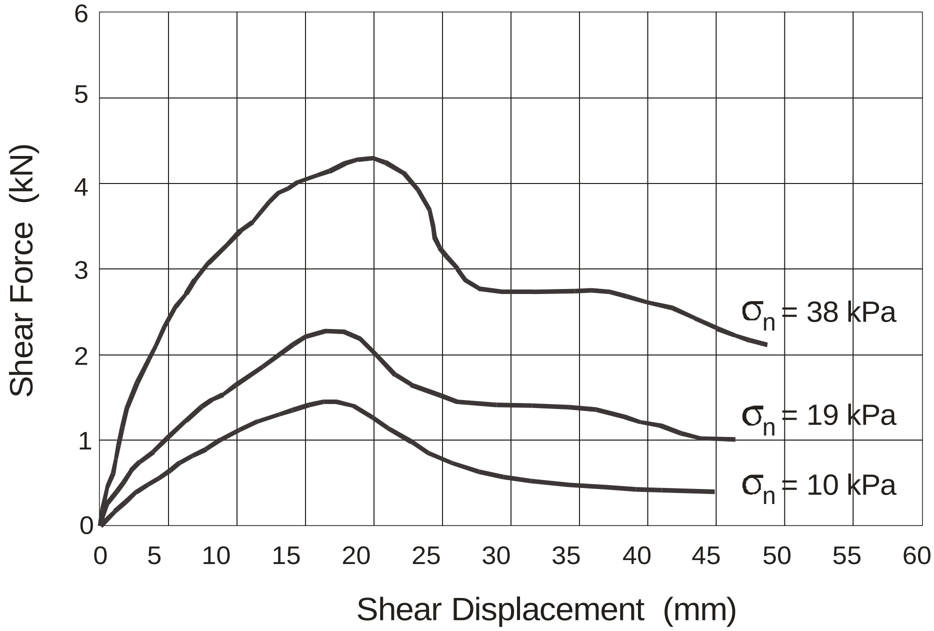


Figure 6-7. Results of Direct Shear Test on a GCL Illustrating Peak and Large-Displacement Shearing Resistances at Different Normal Stresses (σ_n).

As a final comment on the LE equations presented in this section, the equations incorporate terms to account for material internal cohesion or interface adhesion and for geosynthetic tension. Caution should be taken when selecting cohesion or adhesion and geosynthetic tension values for design. As suggested by Koerner and Daniel (1997), cohesion and adhesion values should be used only when there is clear physical justification. From the analysis results presented previously, characterization of internal or interface shearing resistance by a cohesion or adhesion term instead of a friction angle will greatly affect the results of slope stability analyses when hydraulic heads are present in the cover system. In general, geosynthetic tension should not be in the equations unless the design includes a geosynthetic reinforcement layer. Other types of geosynthetics, such as GMs, GNs, GCs, etc., are not designed to permanently transmit tensile loads, are potentially subject to significant tensile creep, and typically have a low tensile modulus (which means that the geosynthetic must elongate to generate tension). Even if geosynthetic reinforcement is used, it should only be relied upon for the tensile force that it can generate at a specified acceptable level of deformation. This acceptable level of deformation must be selected considering the overall performance of all system components.

6.2.3 Stress-Deformation Analyses

Stress-deformation analysis methods may be used for cover system design when the limitations of LE methods are considered significant. The primary advantage of stress-deformation methods is their ability to account for the stress-strain response of materials and interfaces and, therefore, to predict the distribution of stresses and strains within the cover system components, particularly geosynthetic components. Stress-deformation methods can also account for the effects of construction sequencing. The primary disadvantage of stress-deformation methods is the relatively large effort required to obtain material stress-deformation relationships and perform the calculations compared to the effort required with LE methods.

Several studies have been published on the application of stress-deformation methods to cover systems. For example, Long et al. (1993, 1994) described a finite difference model (GEOSTRES) that considers stress equilibrium and strain compatibility. GEOSTRES uses inelastic, non-linear springs to model the shear resistance-displacement behavior at each interface and to model the axial load-displacement behavior within each component. Wilson-Fahmy and Koerner (1992, 1993) adopted a two-dimensional finite element model to account for stress equilibrium and deformation compatibility in stability analyses of soil-geosynthetic systems on slopes.

6.2.4 Shear Strength Parameters

It is recommended that laboratory testing using project-specific materials, coupled with testing procedures and conditions representative of the anticipated field application, be performed to establish design shear strength parameters on a project by project basis. Sabatini et al. (2001) have shown that for a given factor of safety, designs based on project-specific laboratory testing programs are more reliable and less prone to slope instability than designs that utilize shear strength parameters obtained from more general sources, such as databases or the published technical literature.

The various methods used for laboratory shear strength testing of soils are well known and are fully described in a number of geotechnical textbooks and laboratory guides (Lambe, 1951; Holtz and Kovacs, 1981; Bardet, 1997). The most commonly used methods for laboratory shear strength testing of soils are the triaxial compression test and direct shear test.

Currently there are several types of laboratory devices available for the evaluation of shear strength of geosynthetic materials and interfaces. These laboratory devices include:

- large-scale (300 mm x 300 mm) direct shear box specified by ASTM D 5321;
- conventional (50 to 100 mm square or circular) direct shear box with testing generally following ASTM D 3080;
- torsional shear device (ASTM standard under development);
- tilt table; and
- large-displacement shear box.

A summary of the advantages and disadvantages of the first four devices is presented in Table 6-2. Shallenberger and Filz (1996) described the capabilities and limitations of the large-displacement shear box. More recently, Marr (2001) discussed the attributes of test equipment and methods used to evaluate the shear strength of geosynthetic materials and interfaces.

Most project-specific laboratory testing being performed presently uses the ASTM standard 300 mm x 300 mm direct shear box. The large scale of this box is advantageous due to the structure of many geosynthetics, which requires a large test specimen to achieve a representative size of material for testing.

Table 6-2. Summary of advantages and disadvantages associated with test devices for measuring interface shear strength (modified from Gilbert et al., 1995).

Test Device	Advantages	Disadvantages
Large-scale direct shear box	Industry standard Large scale Large displacement Minimal boundary effects	Machine friction Load eccentricity Limited continuous displacement Limited normal stress Expensive
Conventional direct shear box	Large experience base with soil Large normal stress Inexpensive	Machine friction Load eccentricity Small scale Limited displacement Boundary effects
Torsional shear device	Unlimited continuous displacement	Machine friction Anisotropic shearing Small scale Expensive
Tilt table	Minimal machine effects Minimal boundary effects Inexpensive	Small experience base Limited continuous displacement Limited normal stress No post-peak behavior Large effort to prepare sample

Project-specific shear strength testing programs are designed to simulate the anticipated field conditions by selecting appropriate testing procedures and conditions. These include the soil compaction conditions (i.e., water content and density), soil consolidation stress and time, wetting conditions for the materials and interfaces, range of applied normal stresses, direction of shear for geosynthetic interfaces, and shear displacement rate and magnitude. The potential effects of many of these testing conditions on measured interface shear strength parameters are reported in the literature (e.g., Martin et al., 1984; Saxena and Wong, 1984; Bonaparte et al., 1985; Williams and Houlihan, 1987; Seed et al., 1988; Giroud et al, 1990; Seed and Boulanger, 1991; Swan et al., 1991; Pasqualini et al., 1993; Stark and Poeppel, 1994; Bemben and Schulze, 1995; Gilbert et al., 1995; Nataraj et al., 1995; Bonaparte et al., 1996; Gilbert et al., 1996; Shallenberger and Filz, 1996; Stark and Eid, 1996; Stark et al., 1996; Dove et al., 1997; Eid and Stark, 1997; Sharma et al., 1997; Daniel et al. 1998; De and Zimme, 1998; Fox et al., 1998; Sabatini et al. 1998; Snow et al., 1998; Li and Gilbert, 1999; Breitenbach and Swan, 1999). Particular attention should be given to the following:

- Testing should be performed with materials and boundary conditions representative of the anticipated field conditions.
- Soils used in the tests should be compacted to representative field conditions. The compaction moisture content for CCLs used in a direct shear testing program should be near the upper limit of acceptable moisture content and near the lower limit of dry unit weight allowed by the construction specification.
- For GM/CCL interface shear tests, a variety of opinions exist with regard to the application of additional moisture to the interface just prior to assembly of the test

specimen. Options include not adding moisture, lightly or moderately "spritzing" water onto the CCL, or submerging the assembled sample. The rationale for any of these techniques is to simulate suspected installation (e.g., rainfall or moisture conditioning) or post-installation (e.g., condensation collecting at the interface or consolidation-induced water movement to the interface) increases in CCL moisture content at the interface. Counterbalancing these potential mechanisms for moisture content increase at the interface is the effect of thermal gradients typically induced in CCLs beneath GMs prior to covering the GMs with soil. The thermal gradients tends to induce water vapor migration away from the hotter interface and into the underlying cooler soil (Bowders et al., 1997a). Another factor to consider is the post-compaction thixotropic effect identified by Shallenberger and Filz (1996), wherein residual interface shear strengths were found to increase with "curing time" after sample preparation. Given all of these factors, the design engineer must give careful consideration as to the application of additional moisture to the interface just prior to assembly of the test specimen.

- Hydration (soaking) times for GCL samples should be adequate to achieve minimum strength. Daniel and Scranton (1996) showed that hydration times of 24 hours were sufficient for small, 64-mm diameter samples. Koerner and Daniel (1997) noted, however, that complete hydration of relatively large (300 mm x 300 mm) direct shear tests samples takes longer than traditionally required for hydration of soils in relatively small direct shear boxes. Gilbert et al. (1996) reported hydration times, as determined by cessation of GCL swelling under constant normal stress, for reinforced GCLs of up to 25 days. However, Gilbert et al. (1996) used deionized water as the permeating liquid (which increases swell potential), and Daniel et al. (1993) showed that full hydration is not necessary to achieve minimum shear strength. Given this information, an acceptable approach to GCL hydration is to monitor vertical deformation of the GCL and continue to hydrate until these deformations have ceased under the applied normal stress (see discussion of normal stress below). When this procedure cannot be performed, a minimum hydration time of 72 hours is recommended for GCLs to be tested in a 300 mm x 300 mm direct shear box. It should be remembered that without adequate hydration time, the measured GCL strength may be larger than the fully hydrated strength.

- Testing conditions must adequately reflect the field consolidation conditions of the GCL or CCL components. GCLs hydrated as indicated above will be fully consolidated under the normal stress applied during hydration. Consolidation requirements for CCLs may be established using ASTM D 3080. Specimen consolidation times of 48 hours or more may be required for some CCL materials. For both GCLs and CCLs, the normal stress applied during hydration should be equal to the normal stress applied by the cover system in the field, if the full thickness of overlying cover materials is to be placed quickly. Alternatively, a more conservative approach would be to apply a normal stress during hydration equal to only a portion of the overburden stress (e.g., one-third or one-half) that will exist once the cover system is fully constructed. In this latter approach, after hydrating the GCLs, they should be consolidated at the normal stress associated with the full weight of the overlying cover system layers. Under the low normal stresses associated with most cover systems, GCLs will typically swell during hydration.

- ASTM D 5321 and ASTM D 6243 recommend that tests be performed at a minimum of three normal stresses, with each test conducted on a new test specimen. The three selected normal stresses should bracket the normal stress applied by the cover system to the material or interface being tested. This is important because many of the materials used in cover systems exhibit a non-linear relationship between internal or interface shear resistance and normal stress. For cover systems, the applicable range of normal stresses will typically be in the range of about 5 to 40 kPa. Uniformity of normal stress over the entire test specimen must be maintained during hydration, consolidation and shearing so as to avoid stress concentrations.

- Shear displacement rates should be selected considering the type of slope stability analysis to be performed and the types of potentially critical materials or interfaces to be tested. For geosynthetic/geosynthetic interfaces (excluding GCLs), the maximum rate allowed by ASTM D 5321 of 0.08 mm/s will generally be acceptable. For long-term stability conditions where the potentially-critical material or interface includes a CCL or GCL component, the shear displacement rate should be as slow as reasonably achievable; the default shear displacement rate of 0.017 mm/s given in ASTM D 5317 is too fast to achieve drained shearing conditions for CCLs and GCLs. Procedures for estimating shear rates to obtain fully-drained conditions for CCLs are given in ASTM D 3080. Procedures and data for estimating shear rates to obtain fully-drained conditions for GCLs are given in ASTM D 6243. It is noted, however, that it may not be necessary to achieve fully-drained test conditions to obtain test results suitable for long-term analyses. Available data suggest that for design purposes, a shear displacement rate of not more than 0.0005 mm/s will produce test results appropriate for use in slope stability analyses involving GCL materials and interfaces. In contrast, for the evaluation of seismic stability, shear displacement rates should be as fast as reasonably achievable. For both conditions, testing should be performed using samples fully-consolidated under the applied normal stresses.
- Tests should be carried out to a shear deformation adequate to evaluate both the peak and large-displacement shear resistance of the material or interface being tested. Many geosynthetic/geosynthetic and soil/geosynthetic interfaces exhibit very significant post-peak reductions in shear strength (Figure 6-7). ASTM D 5321 states that one should "*run the test until the applied shear force remains constant with increasing displacement.*" To achieve a large-displacement shear condition (defined as a relatively-flat, post-peak shear stress versus shear displacement line) in a direct shear test, shear displacements of 50 mm or more may be necessary. It should also be noted that this large-displacement shear strength is close to, but typically not as low as, the absolute minimum (i.e., residual) shear strength of the material or interface. Residual shear resistances may not occur until shear displacements reach 200 mm, or more. Torsional ring shear testing (Stark and Poeppel, 1994) can be used to evaluate residual shear strengths for soils and geosynthetics for which representative samples can be produced for the small size and torsional shearing mode of this type of test. Alternatively, large-displacement shear box testing (Shallenberger and Filz,1996) can be used to evaluate residual shear strengths for larger-size test specimens in a linear displacement mode. For most practical design applications, true residual strength can be estimated to an acceptable degree of accuracy as 90 to 95% of the large-displacement strength obtained from a 300 mm x 300 mm direct shear test.
- Multi-component cover systems may have more than one potentially-critical slip surface.

The shear test program for a project may need to consider several materials or interfaces.

- Some materials exhibit significant manufacturing variability. For example, the degree of texturing on GMs and the amount of internal-reinforcing in needlepunched GCLs has been observed to vary significantly from lot to lot. This variability should be considered both in design and in the selection of project QC/QA protocols.
- Test results can be interpreted in terms of a secant friction angle that varies as a function of normal stress, or by a tangent friction angle and apparent cohesion (or adhesion for interface strength) applicable to the range of considered normal stresses. Both of these approaches are illustrated in Figure 6-8. For cover system applications, internal and interface shear strength parameters should be defined in terms of a secant friction angle for cases where hydraulic heads could develop in the cover soil. Since the apparent cohesion or adhesion may not be a true material or interface property, the use of this parameter with high heads (relative to the total normal stress) could lead to an over-prediction of the true slope stability factor of safety. Also, as previously mentioned, Koerner and Daniel (1997) suggested that cohesion and adhesion values should be used only when there is clear physical justification.

All of the foregoing factors should be considered in designing a laboratory shear testing program to evaluate internal and interface shear strengths and in using the results of the program in slope stability analyses.

Several other factors may affect long-term shear strength properties of the cover system materials and interfaces. For example, in cold regions, freeze-thaw may reduce the shear strength of cover system CCLs and CCL/geosynthetic interfaces. Research has shown that many CCLs undergo significant change in soil fabric and reduction in shear strength as a result of freeze-thaw cycling (e.g., Nagasawa and Umeda, 1985; Othman et al., 1994). A case study illustrating how this problem contributed to slope failure for a landfill cover system in Ohio is presented in Section 7.4.3. In addition, both heating and cooling result in soil moisture migration, which can cause changes in material and interface, shear strengths (e.g., Daniel et al., 1993). Furthermore, long-term creep may also be significant, particularly in geosynthetic components. No consistent standard of practice presently exists for directly addressing the potential effects of all of these factors on cover system stability. These factors may be indirectly accounted for through the use of higher minimum acceptable factors of safety, when appropriate, or through placement of a greater thickness of cover soil above the critical layers for thermal insulation and isolation from environmental factors.

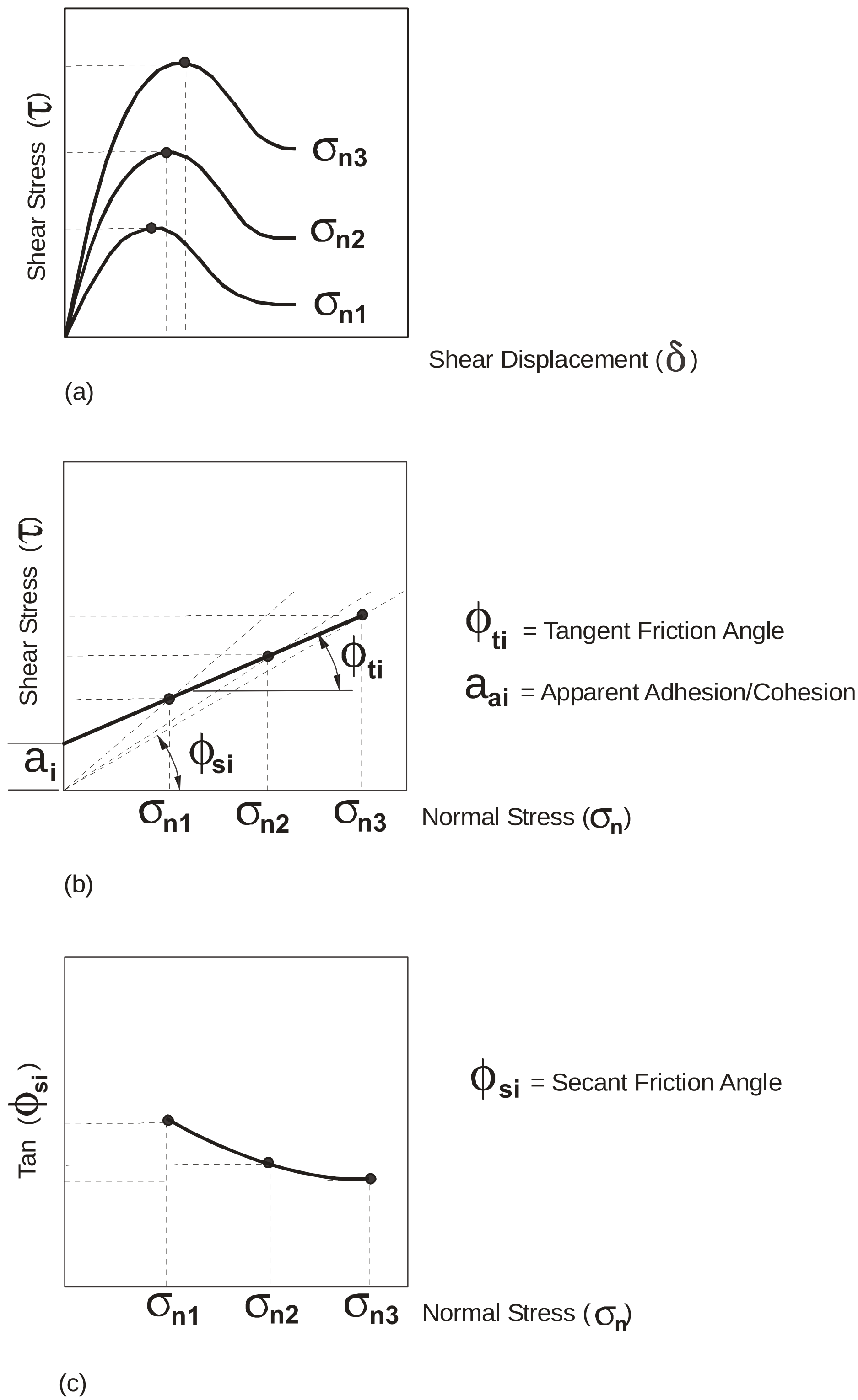


Figure 6-8. Interpretation of Cover System Interface or Internal Shear Test: (a) Test Results for Peak Strengths; (b) Tangent Friction Angle, ϕ_{ti}, and Apparent Adhesion or Cohesion, a_{ai}; and (c) Secant Friction Angle, ϕ_{si}. Similar Interpretations are Applied to Large-Displacement and Residual Conditions.

6.2.5 Construction Considerations

The placement of soil over a slope with underlying low shear strength materials or interfaces will induce shear stresses that can reduce slope stability. These shear stresses result from the operation of construction equipment on the slope, the weight of the soil, and, if the soil is pushed down the slope, from the moving soil itself. Construction-induced stresses have been investigated by McKelvey and Deutsch (1991) and Koerner and Daniel (1997). These references present closed-form LE equations that can be used to evaluate the effect of construction equipment operation on cover system stability. The clear recommendation that comes out of these investigations is that cover soils should be placed over low shear strength materials and interfaces from the bottom of the slope upward and not from the top of the slope downward (Figure 6-9).

The following comments are provided with respect to placement of soil materials in cover systems:

- By placing cover soils from the bottom of the slope upward, a passive, stabilizing soil wedge is established at the toe of slope prior to placement of soil higher on the slope. The operation of construction equipment over this lower wedge tends to compact and strengthen the wedge.
- Relatively small, wide-track dozers (i.e. low-ground pressure dozers) are recommended for placing the soil cover material. This type of equipment limits both the dynamic force imparted to the slope during acceleration and braking and the tractive force applied through the dozer tracks.
- Downslope dynamic forces can be limited further by limiting the dozer speed on the slope and by instructing the dozer operator to avoid hard breaking, particularly when backing downslope.

By application of the construction procedures described above, construction-induced impacts to the stability of a cover system slope (designed to conventional slope stability factors of safety described next in Section 6.2.6) are minor. For other conditions (e.g., lower factors of safety than recommended in Section 6.2.6, placement of soil from the top of slope downward, use of large construction equipment) construction-stage stability should be checked using the procedures described by McKelvey and Deutsch (1991) or Koerner and Daniel (1997).

6.2.6 Factors of Safety

LE analysis methods provide a calculated slope stability factor of safety (FS). Minimum acceptable FS values for cover systems depend on project-specific conditions and uncertainties. For example, when cover systems include strain-softening materials or interfaces, differing minimum factors of safety are often applied to peak strength analyses and analyses based on large-displacement or residual strength. Other criteria may also influence selection of a minimum acceptable FS, including regulatory requirements, reliability of laboratory test methods, similarity between laboratory testing conditions and field conditions, completeness of laboratory test data, uncertainty with respect to other design input parameters (e.g., unit weights, hydraulic heads, geometry), and consequences of slope failure.

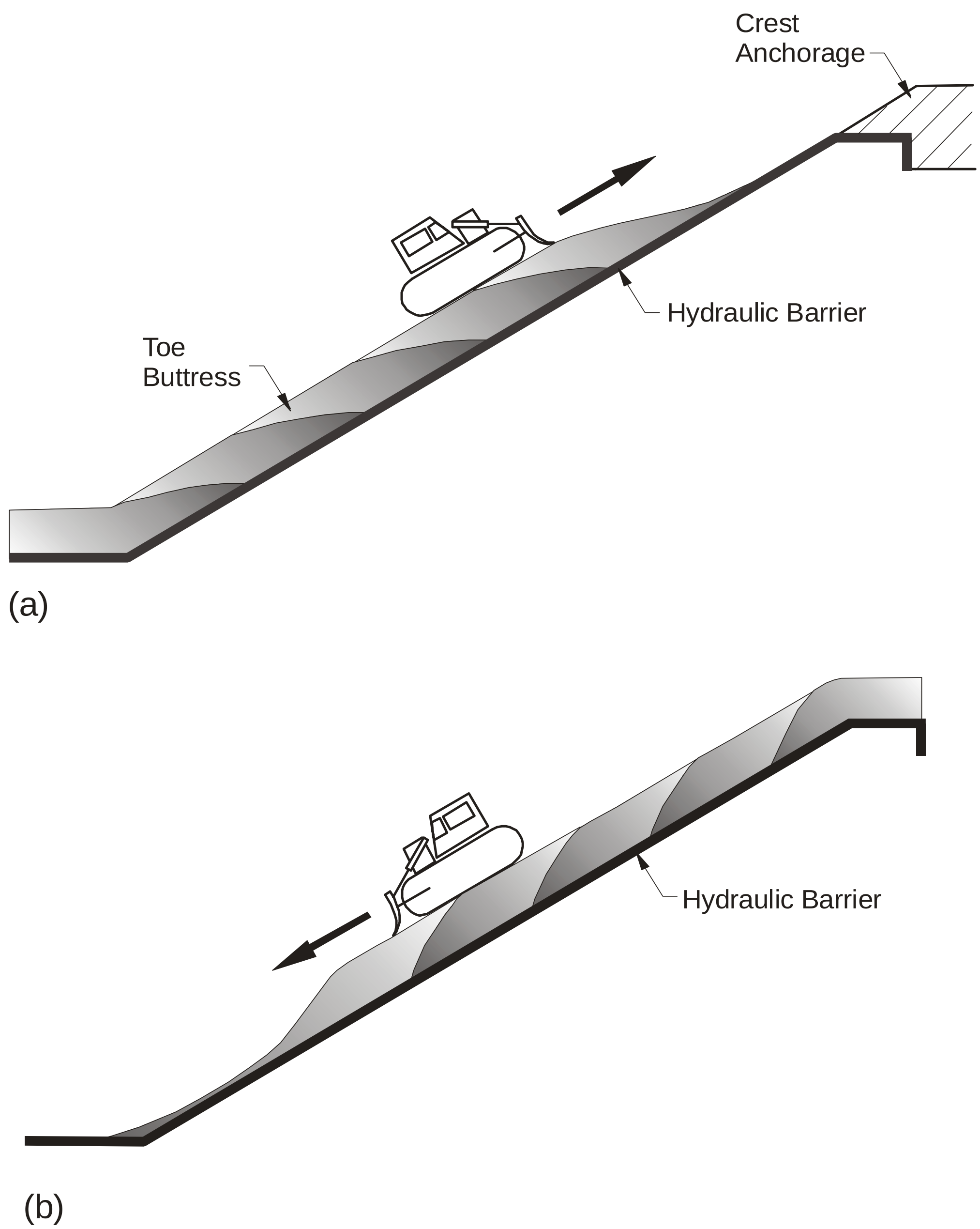


Figure 6-9. Cover Soils Should be Placed Over Low Shear Strength Materials and Interfaces from the Bottom of the Slope Upward (a) and not from the Top of the Slope Downward (b).

Previous agency guidance on selecting slope stability factors of safety was given in EPA (1988). The FS values given in the 1988 document were meant to apply to excavation and embankment (soil) slopes used in the construction of landfills and surface impoundments. As the reported FS values represent general guidance, however, they have sometimes been cited as criteria for the design of cover systems. The values from EPA (1988) are given in Table 6-3 below.

Table 6-3. Previously-recommended minimum FS values (modified from EPA, 1988).

Consequences of Slope Failure	Uncertainty of Strength Measurements	
	Small[1]	*Large*[2]
No imminent danger to human life or major environmental impact if slope fails	1.25	1.5
Imminent danger to human life or major environmental impact if slope fails	1.5	2.0 or greater

[1] The uncertainty of the strength measurements is smallest when the soil conditions are uniform and high quality strength test data provide a consistent, complete, and logical picture of the strength characteristics.
[2] The uncertainty of the strength measurements is greatest when the soil conditions are complex and when available strength data do not provide a consistent, complete, or logical picture of the strength characteristics.

Duncan (1992), in a state-of-the-art paper on slope stability of soils, provided the following discussion on the selection of an appropriate factor of safety:

> "*Criteria for acceptable values of safety factor should be established with two important considerations in mind. These are, (1) what is the degree of uncertainty involved in evaluating the conditions and shear strengths for analysis, and (2) what are the possible consequences of failure? When the uncertainty and the consequences of failure are both small, it is acceptable to use small factors of safety, on the order of 1.3 or even smaller in some circumstances. When the uncertainties or the consequences of failure increase, larger factors of safety are necessary. Large uncertainties coupled with large consequences of failure represent an unacceptable condition, no matter what the calculated value of the factor of safety. Typical minimum acceptable values of factor of safety are about 1.3 for end-of-construction and multi-stage loading, 1.5 for normal long term loading conditions, and 1.0 to 1.2 for rapid drawdown, in cases where rapid drawdown represents an improbable or infrequent loading condition.*"

While the guidance was developed by Duncan for soil slopes, the philosophy on FS selection is directly applicable to the design of cover systems for waste containment applications and is generally consistent with Table 6-3.

More recently, Koerner and Soong (1998) presented recommendations on FS selection that incorporate a similar philosophy and that are specific to cover systems. The first step in the approach suggested by Koerner and Soong (1998) is to qualitatively classify the project as critical or non-critical and temporary or permanent. This qualitative classification is adapted from Bonaparte and Berg (1987), who suggested its use for geosynthetic reinforcement in highway applications. With this classification system, a critical application is one in which the consequences of failure include a potential for personal injury, significant property damage, or significant environmental release of contaminants. In contrast, a non-critical classification would apply to a cover system of limited extent that could be readily repaired (e.g., a monolithic soil cover) and for which the consequences of failure are minor. Table 6-4 presents the qualitative classification system proposed by Koerner and Soong (1998). The classification system is used to assign a ranking (low, moderate, or high) to the cover system so that the appropriate FS value can be selected.

Table 6-4. Qualitative classification for cover system applications (Koerner and Soong (1998)).

Concern	Duration	
	Temporary	*Permanent*
Noncritical	Low	Moderate
Critical	Moderate	High

Based on the cover system rankings in Table 6-4, Koerner and Soong (1998) recommended the minimum static slope stability FS values for cover systems given in Table 6-5. Koerner and Song (1998) suggested lower FS values for non-hazardous (principally MSW) landfills as compared to hazardous waste landfills due to the differences in waste characteristics and the larger magnitude of post-closure settlements for MSW landfills compared to hazardous waste landfills (which results in an appreciable flattening of the MSW cover system slopes with time). For Table 6-5, Koerner and Soong (1998) considered remediation waste piles to primarily consist of low-hazard materials such as construction and demolition wastes and mine wastes. Abandoned dumps on the other hand were considered to include CERCLA remediation sites and other sites containing potentially-hazardous wastes or unknown wastes. Hence, abandoned dumps were considered to pose a higher hazard than either non-hazardous waste landfills or remediated waste piles.

Liu et al. (1997) have suggested that factors of safety for design of cover systems be selected by a multi-step process that involves:

- estimating the mean value, standard deviation, coefficient of variation, and correlation coefficient (between parameters) for each variable in the slope stability analysis;
- calculating failure probabilities and correlating these probabilities to the LE factor of safety for the potential ranges in parameter values; and
- defining an acceptable probability of failure based on the cost and consequences of failure.

Table 6-5. Minimum FS values for static slope stability of cover systems recommended by Koerner and Soong (1998).

	Type of Waste			
Ranking	*Remediated waste piles*	*Non-hazardous waste landfills*	*Abandoned dumps*	*Hazardous waste landfills*
Low	1.2	1.3	1.4	1.4
Moderate	1.3	1.4	1.5	1.5
High	1.4	1.5	1.6	1.6

The approach described by Liu et el. (1997) provides a rational, probability-based approach to designing safe cover system slopes. It is recognized, however, that not all engineers are comfortable with probabilistic approaches and that the standard of practice is to use deterministic methods to establish factors of safety. At a minimum, however, Liu et al. (1997) provides a useful framework for the design engineer to systematically consider the areas of uncertainty and consequences of failure in the project.

A number of technical publications have addressed the issue of FS selection as it relates to the use of peak versus large-displacement (or residual) internal or interface shear strength where the large displacement strength of the material or interface is smaller than the peak shear strength (Byrne, 1994; Stark and Poeppel, 1994; and Bonaparte et al., 1996).

The foregoing discussion should make it clear that there is no single value of FS applicable to all situations. Selection of a FS value for a particular project is a key design decision that should be the responsibility of the design engineer. Based on the foregoing discussion, the following general guidance is given. This guidance applies specifically to cover systems, where the geometry is well defined and the mass being analyzed consists entirely of manufactured or constructed materials placed under controlled conditions. These minimum FS recommendations may not be appropriate for other applications.

- A minimum acceptable factor of safety (FS_{min}) for static stability analyses of 1.5 will often be appropriate for permanent cover system applications where the design is based on peak internal and interface shear strengths conservatively established using project-specific interface direct shear tests, two-dimensional limit equilibrium slope stability analyses, and appropriate consideration of the potential for internal hydraulic head build-up during the representative design storm events. This FS_{min} is applied to normal operating conditions (e.g., no seismic loading or live loading).
- A smaller or larger FS_{min} may be considered based on an evaluation of: (i) consequences of cover system failure; and (ii) uncertainty associated with each design parameter.
- If the cover system contains geosynthetic materials that exhibit strain-softening internal or interface shear strengths, FS_{min} for large-displacement conditions should also be checked. A FS_{min} of 1.2 is suggested where large-displacement shear strengths have been conservatively established using project-specific interface direct shear tests conducted in accordance with ASTM D 5321. For purposes of this evaluation, 50 to 75 mm of displacement, coupled with the observation that the shear stress-displacement plot is essentially flat at the end of the test, is considered to satisfy the large-displacement condition. If true residual shear strengths are obtained using either a torsional-ring or large-displacement shear apparatus, FS_{min} values as low as 1.15 may be considered.
- Cover system designs should be checked for low-probability extreme loading conditions. These conditions need to be identified on a case-by-case basis, but may include extreme storm events, live loads, or earthquakes. Design for earthquakes is addressed subsequently. Values of FS_{min} for extreme loading conditions may, in general, be lower than those associated with the representative design conditions as described above. FS_{min} values for these conditions should be selected on a case-by-case basis.

If FS_{min} cannot be achieved for a given set of conditions, there are a variety of measures that can be taken to increase its value. Examples of these measures are listed in Table 6-6.

Table 6-6. Engineering measures to increase cover system slope stability factor of safety.

Use cover system materials that have higher internal or interface shear strengths, as available
Provide for a flatter cover system slope by initially placing waste to a flatter slope (for new facilities) or waste excavation (for existing facilities) (Figure 6-10)
Shorten the slope length through the use of benches or berms
Use perimeter retaining walls or buttresses to achieve a flatter cover system slope angle (Figure 6-11)
Improve cover system internal drainage if hydraulic head buildup is predicted to occur
Utilize geosynthetic reinforcement, but only within the limitations of this approach described in this chapter

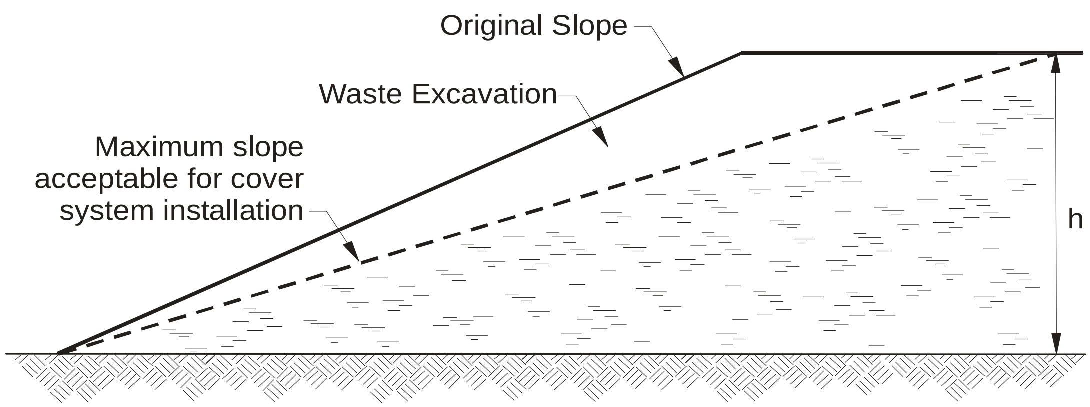


Figure 6-10. Waste Excavation Approach for Constructing Cover Systems Over Steep Waste Slopes.

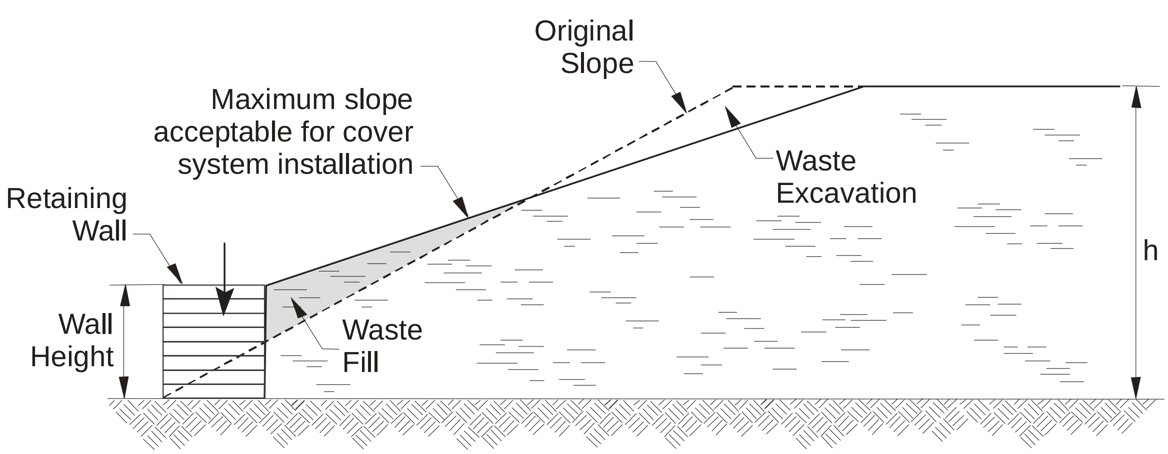


Figure 6-11. Buttress Approach for Constructing Cover Systems Over Steep Waste Slopes Without the Need for Significant Waste Excavation.

6.3 Seismic Slope Stability and Deformation

6.3.1 Overview

The cover system for a landfill or other waste containment unit or for a remediation site may be subject to damage as a result of strong ground accelerations that can accompany an earthquake. Impacts may involve either excessive seismic displacement of one or more of the cover system components or complete instability of the cover system. For most situations, peak seismic accelerations in a cover system will be larger than in the surrounding free field, due to amplification of the ground movements by the underlying waste.

State and federal regulations have various requirements with respect to the evaluation of the potential impact of seismically-induced ground motions on cover systems. EPA regulations for hazardous waste landfills (40 CFR §264 and §265) are silent with respect to seismic design and performance criteria. EPA seismic regulations for MSW landfills, contained in 40 CFR §258.14, require that "*all containment structures, including liners, leachate collection systems and surface water control systems, are designed to resist the maximum horizontal acceleration in lithified earth material for the site*" if the landfill is located in a "seismic impact zone". EPA defines a seismic impact zone as "*an area with a ten percent or greater probability that the maximum horizontal acceleration in lithified earth material, expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10g in 250 years*." EPA further elaborates that the "*maximum horizontal acceleration in lithified earth material means the maximum expected horizontal acceleration depicted on a seismic hazard map, with a 90 percent or greater probability that the acceleration will not be exceeded in 250 years, or the maximum expected horizontal acceleration based on a site-specific seismic risk assessment.*" While this regulation does not explicitly mention cover systems, EPA considers the cover system to be part of a landfill "containment structure" and therefore covered by the regulation. However, the agency recognizes that although difficult and potentially costly, cover systems can be repaired if damaged. In contrast, landfill bottom liner systems generally cannot be repaired once covered with waste. As a consequence, the agency believes that seismic performance criteria (e.g., acceptable FS or magnitude of permanent seismic deformation) applicable to cover systems may not always need to be as stringent as those applied to landfill bottom liner systems.

The EPA guidance document entitled "*RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities*" (Richardson et al., 1995) presents available information and analysis methods to evaluate the seismic performance of landfills. The information contained in this guidance document is consistent with and includes new information that has become available after publication of the EPA document listed above.
Evaluation of the seismic stability of a cover system involves four steps, each of which can be performed using either conservative, simplified approaches, or more complex, detailed analyses. These four steps, which are discussed in more detail below, are as follows:

1. conduct a seismic hazard evaluation to estimate peak horizontal bedrock accelerations for a site and representative causative earthquake events to associate with that acceleration (Section 6.3.2);
2. perform a seismic response analysis to evaluate peak horizontal accelerations at the ground surface or in the waste mass cover system due to the causative earthquake events

(Section 6.3.3);

3. select dynamic shear strength properties for cover system materials and interfaces to use in seismic slope stability and/or deformation analyses (Section 6.3.4); and
4. perform seismic slope stability and deformation analyses (Section 6.3.5).

6.3.2 Seismic Hazard Evaluation

The objective of a seismic hazard evaluation is to characterize the design earthquake with respect to the parameters required for engineering analysis (e.g., magnitude, style of faulting, site-to-source distance, peak ground acceleration, and spectral accelerations). The peak horizontal bedrock acceleration at a project site may be estimated using seismic hazard probability maps or site-specific seismic hazard assessments. The most commonly used maps in the U.S. are those developed by the U.S. Geological Survey (USGS) depicting peak and spectral horizontal bedrock accelerations with 10, 5, and 2% probabilities of exceedance in 50 years (corresponding, respectively, to a 90% probability of not being exceeded in 50, 100, and 250 years). These maps, which can be downloaded from the USGS website (http://geohazards.cr.usgs.gov/eq/index.html), are periodically updated to reflect recent developments in the field of seismology. Background information on the development of these maps is provided by Frankel et al. (1996). Figure 6-12 presents the U.S. national map for peak horizontal acceleration in bedrock with a 90% probability of not being exceeded in 250 years. A map for California and Nevada is presented in Figure 6-13. These maps are included in this guidance document because the probability-recurrence relationship for these maps corresponds to the EPA regulatory criterion for seismic design of MSW landfills.

Seismic hazard maps like those of USGS discussed above usually present the estimated free-field peak horizontal acceleration for a hypothetical bedrock outcrop on level ground at a particular location. If bedrock is not present at or near the ground surface, the peak acceleration may need to be modified to account for local site conditions. The presence of a waste mass will further modify the earthquake ground motions, as discussed subsequently. The primary difficulty associated with using seismic probability maps is that the maps by themselves do not provide information on the magnitude, site-to-source distance, or duration of the earthquake associated with the map acceleration values. For some types of seismic analyses, information on these variables is necessary. Because they are probabilistically derived, the acceleration values provided on such maps are composed of contributions of earthquakes of many different magnitudes from several to many different seismic sources. Each source may be associated with a different site-to-source distance and each magnitude-distance combination with a different duration. The USGS website has recently made available information on the distribution of earthquake magnitudes and site-to-source distances associated with the bedrock accelerations obtained directly from the USGS seismic hazard maps. Using this feature, the peak bedrock acceleration for a given site with a 2% probability of exceedance in 50 years (90% probability of not being exceeded in 250 years) is deaggregated by earthquake magnitude and site-to-source distance. Deaggregated spectral accelerations are also provided for spectral periods of 0.2, 0.3, and 1 second. USGS currently provides deaggregated data for 64 central and eastern U.S. cities and 56 western U.S. cities. As an example of the information available at the USGS website,

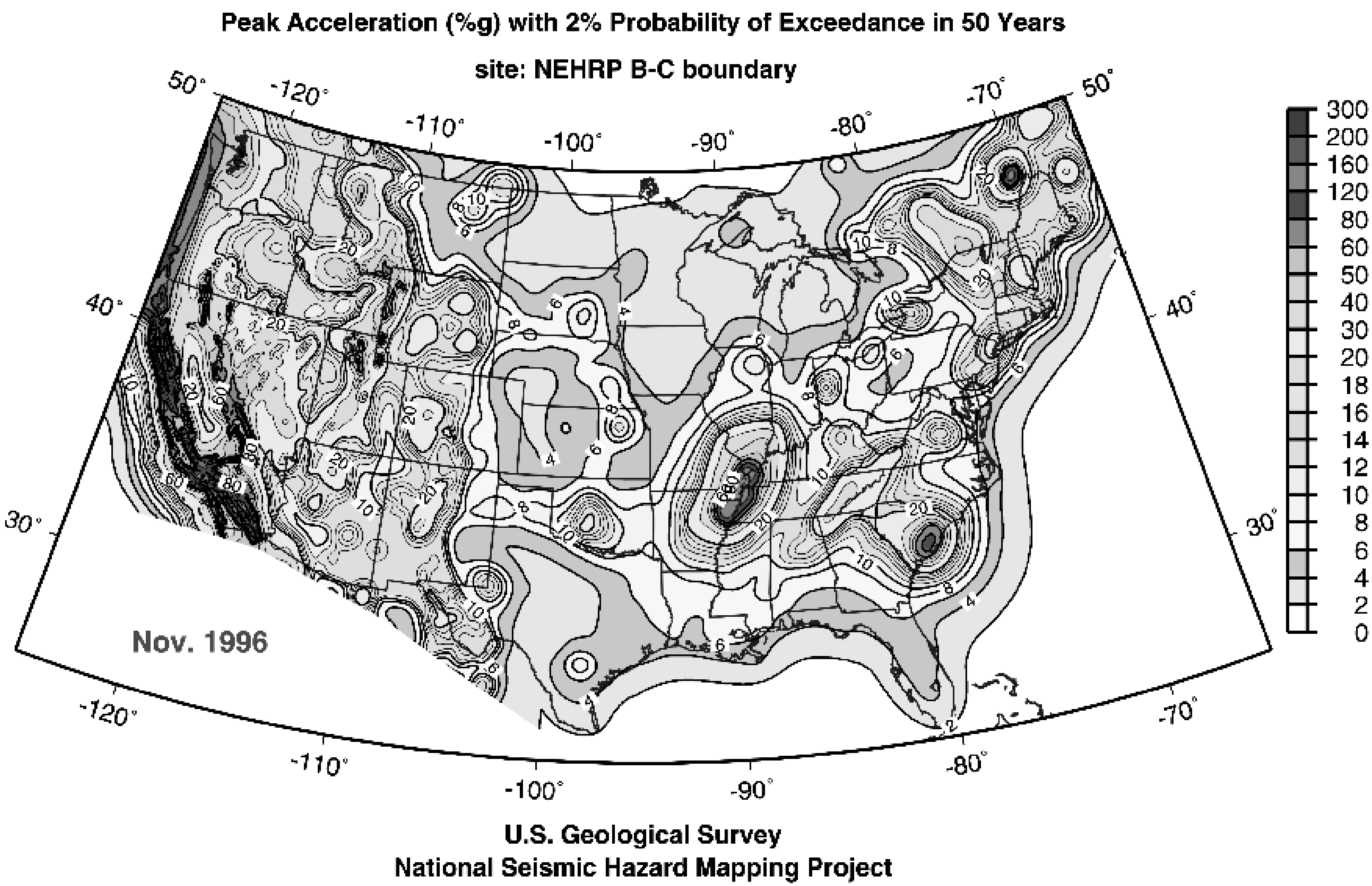


Figure 6-12. Seismic Hazard Probability Map for the U.S. for Peak Horizontal Acceleration in Bedrock with a 90% Probability of not Being Exceeded in 250 Years (downloaded from http://geohazards.cr.usgs.gov/eq/).

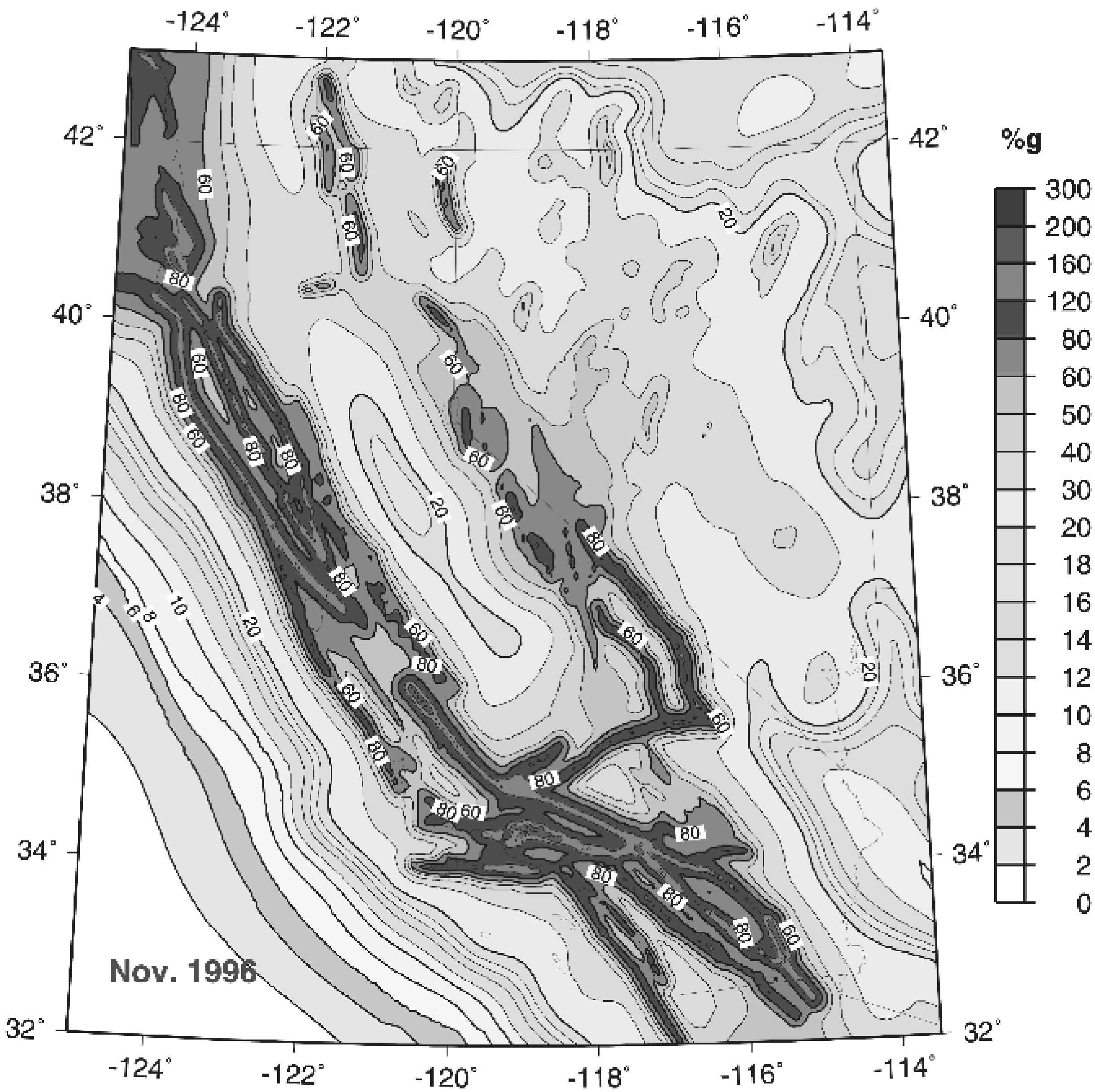


Figure 6-13. Seismic Hazard Probability Map for California and Nevada for Peak Horizontal Acceleration in Bedrock with a 90% Probability of not Being Exceeded in 250 Years (downloaded from http://geohazards.cr.usgs.gov/eq/).

deaggregated bedrock acceleration and site-to-source distance data for Evansville, Indiana are presented in Table 6-7.

In using deaggregated data, such as given in Table 6-7, the engineer should identify the earthquake magnitude and distance combination that encompasses about two-thirds of the seismic hazard. For example, if more than two-thirds of the seismic hazard for a given site is from a small magnitude, near-source earthquake, then seismic analyses should be performed using input variables (e.g., strong motion records) appropriate for this type of earthquake event. In some cases, more than one combination of earthquake magnitude and source distance may need to be considered. The values in Table 6-7 for Evansville illustrate such a case: a significant portion (≈ 40%) of the seismic hazard for Evansville is derived from earthquakes less than 25 km from the site with magnitudes between 5 and 6 (though the magnitude of some of the local events contributing to the seismic hazard may be as great as 7.5). However, over 20% of the seismic hazard is from a distant earthquake more than 150 km from the site with a magnitude of 8.0. Therefore, for some projects in Evansville, the impact of both local and distant events may warrant consideration.

Table 6-7. Deaggregated peak horizontal bedrock accelerations as a percentage of the aggregated peak probabilistic acceleration of 0.328 g for Evansville, Indiana, for a 2% probability of exceedance in 50 years (modified from USGS website).

Hypocentral Distance (km)	Earthquake Magnitude						
	5.0	*5.5*	*6.0*	*6.5*	*7.0*	*7.5*	*8.0*
25	13.888	13.372	9.591	5.787	2.447	1.584	0.000
50	1.507	3.176	4.624	5.054	2.818	2.440	0.000
75	0.093	0.349	0.927	1.737	1.449	1.757	0.000
100	0.011	0.060	0.247	0.670	0.591	0.938	0.000
125	0.002	0.017	0.095	0.348	0.250	0.488	0.000
150	0.000	0.004	0.030	0.143	0.114	0.269	0.000
175	0.000	0.001	0.008	0.050	0.051	0.146	15.718
200	0.000	0.000	0.002	0.019	0.023	0.083	6.904
225	0.000	0.000	0.001	0.007	0.010	0.042	0.000
250	0.000	0.000	0.000	0.003	0.004	0.020	0.000
275	0.000	0.000	0.000	0.001	0.002	0.012	0.000
300	0.000	0.000	0.000	0.000	0.001	0.006	0.000
325	0.000	0.000	0.000	0.000	0.000	0.003	0.000
350	0.000	0.000	0.000	0.000	0.000	0.002	0.000
375	0.000	0.000	0.000	0.000	0.000	0.002	0.000
400	0.000	0.000	0.000	0.000	0.000	0.001	0.000
425	0.000	0.000	0.000	0.000	0.000	0.001	0.000

As a means to reduce uncertainty and increase accuracy, site-specific seismic hazard analyses may be preferred to seismic hazard maps for assessing the seismic hazard to critical structures in regions of high seismic activity. A site-specific seismic hazard analysis involves:

- identification of the seismic sources capable of strong ground motions at the project site;
- evaluation of the seismic potential for each capable source; and
- evaluation of the intensity of the design ground motions at the project site.

Site-specific seismic hazard analyses may be performed using either a deterministic or probabilistic approach. Detailed discussion of this topic is beyond the scope of this chapter. The reader is referred to Reiter (1990), Krinitzsky et al. (1993), Richardson et al. (1995), and Kramer (1996). An example of a site-specific seismic hazard analysis applied to a landfill site (including cover system) in California is given in Kavazanjian et al. (1995a).

6.3.3 Seismic Response Analysis

6.3.3.1 Introduction

The seismic hazard assessment as discussed above provides an estimate of peak horizontal accelerations in bedrock for a given site along with information on the causative earthquake event(s). A response analysis is used to estimate the seismically-induced motions (e.g., acceleration, velocities, and/or displacements) at the ground surface or in the waste mass cover system. Response analyses are needed because soil layers and waste modify the bedrock motions, sometimes in a manner that can significantly increase damage potential.

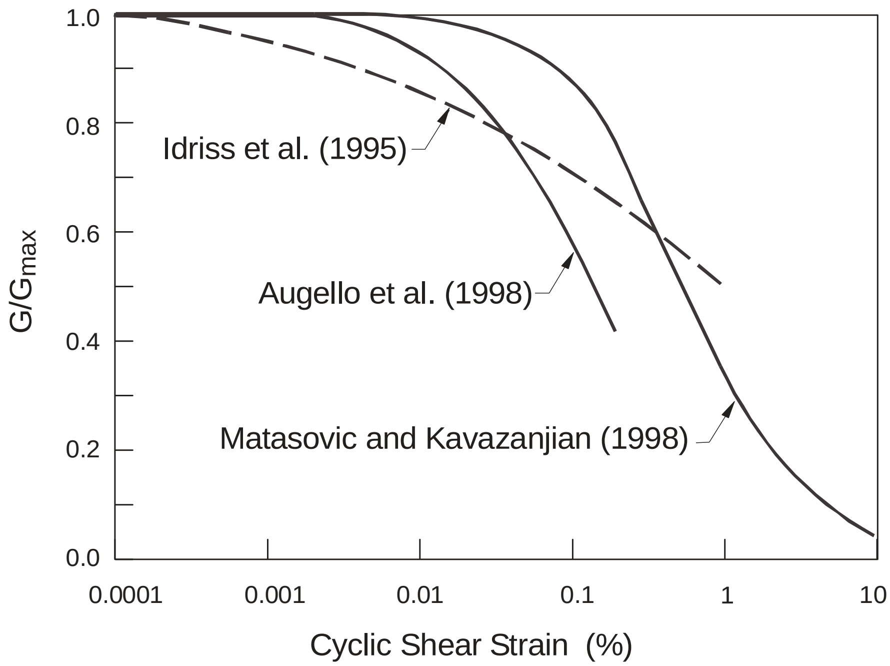
6.3.3.2 Material Properties Selection

The first step in the seismic response analyses is to characterize the soil and waste material properties needed to perform the analysis. For equivalent linear analyses of vertically-propagating shear waves (the most common type of seismic response analysis performed for geotechnical and waste management applications), these properties include total unit weight, dynamic shear modulus, and damping ratio for each material through which the waves propagate. Kramer (1996) provided an extensive review of the available technical literature on the selection of soil and rock properties for response analyses. Guidance on selecting MSW waste properties can be found in Sharma et al. (1990), Fassett et al. (1994), Richardson et al. (1995), Kavazanjian et al. (1995b), Kavazanjian and Matasovic (1995), and Matasovic and Kavazanjian (1998).

Shear modulus reduction factor and damping ratio curves for the Operating Industries Inc. (OII) site, a large inactive MSW and IW landfill in Monterey Park, California, were developed independently by Idriss et al. (1995), Augello et al. (1998), and Matasovic and Kavazanjian (1998). The curves proposed by these three sets of investigators are shown in Figure 6-14. These curves represent the most reliable information currently available for use in estimating strain-dependent dynamic shear modulus reduction factors, G/G_{max} (dimensionless), and damping ratios for MSW and other solid waste materials for use in equivalent linear response analyses. The strain-dependent damping ratio is obtained directly from Figure 6-14. The dynamic shear modulus, G (Pa), is obtained by multiplying the shear modulus reduction value from Figure 6-14 by the maximum small-strain dynamic shear modulus, G_{max} (Pa), which can be calculated using the equation:

$$G_{max} = \frac{\gamma_{t,waste} v_{s,waste}^2}{g} \qquad \text{(Eq. 6.16)}$$

where: $v_{s,\,waste}$ = shear wave velocity of waste (m/s); $\gamma_{t,\,waste}$ = total unit weight of waste (N/m^3); and g = acceleration of gravity (m/s^2). The small-strain shear modulus is ideally obtained from project-specific field testing. For landfills, this type of testing may be performed with the non-



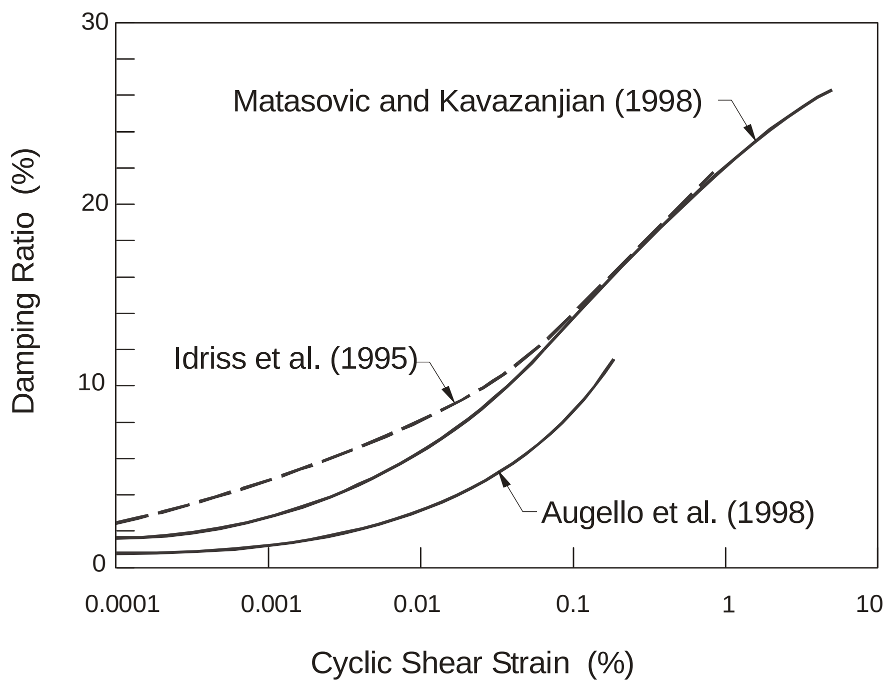


Figure 6-14. Estimated Shear Modulus Reduction Factor and Damping Ratio Curves for the OII Landfill, California (modified from Idriss et al.,1995; Augello et al, 1998; and Matasovic and Kavazanjian, 1998).

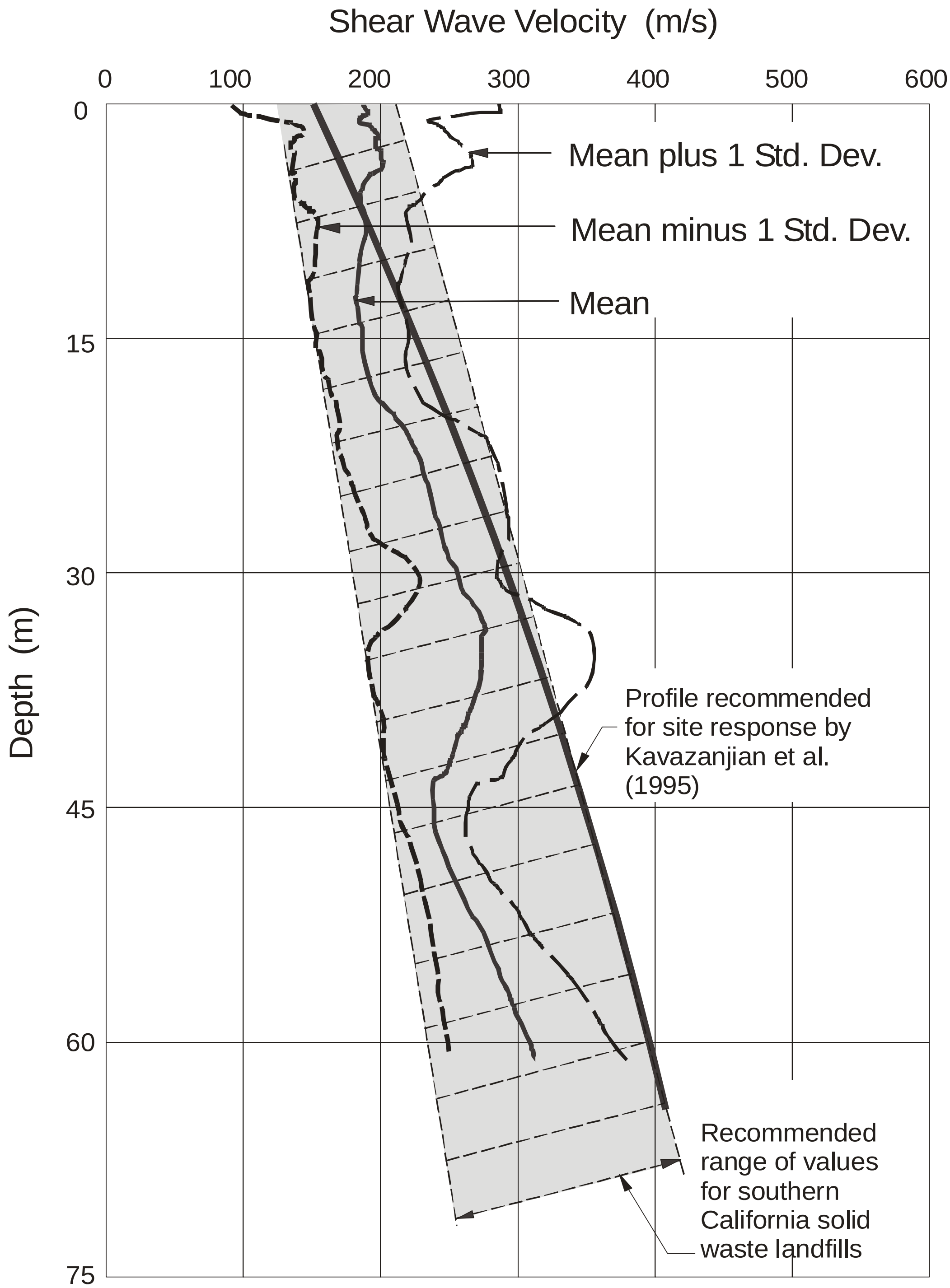


Figure 6-15. Shear Wave Velocities for Southern California Solid Waste Landfills (modified from Kavazanjian et al., 1996).

intrusive spectral analysis of surface waves (SASW) technique (Kavazanjian et al. 1994, 1996). Intrusive downhole or cross-hole geophysical testing techniques may also be used. In the absence of project-specific testing, the data for southern California landfills from Kavazanjian et al. (1996), presented in Figure 6-15, can be used. It is noted that results obtained from a limited amount of SASW testing of MSW landfills in the eastern U.S. (unpublished) suggests that shear wave velocities for waste in these facilities may be lower, on average, than shear wave velocities for waste in relatively dry southern California landfills. In the absence of better information, the lower portion of the recommended range of shear wave velocities shown in Figure 6-15 can be used for MSW landfills located in the eastern U.S and other temperate to wet climates.

6.3.3.3 Simplified Response Analysis

Simplified approaches to seismic response analyses involve the empirical correlation of peak horizontal waste mass or cover system acceleration, as applicable, to peak bedrock acceleration. Correlations of this type were first used in geotechnical engineering to relate peak ground accelerations at a site with a soil profile overlying bedrock to peak bedrock accelerations at the same site (e.g., Seed and Idriss (1982) and Idriss (1990)). More recently, Bray et al. (1995), Kavazanjian and Matasovic (1995), Singh and Sun (1995), Bray and Rathje (1998), and Matasovic et al. (1998) have extended this type of relationship to solid waste landfills.

Matasovic et al. (1998) compared estimated horizontal bedrock accelerations to recorded peak horizontal accelerations at the OII site. Table 6-8, taken from Matasovic et al. (1998), presents peak acceleration values (average of two horizontal components) recorded at the top deck of the OII landfill versus the estimated peak horizontal bedrock accelerations for the site. Based on these results, Matasovic et al. (1998) concluded that peak horizontal bedrock accelerations from both near-field and far-field earthquakes up to at least 0.15 g can be significantly amplified by solid waste landfills. They suggested that, based on the OII data, the curve developed by Harder (1991) for the upper-bound amplification of seismic accelerations in earth dams, shown in Figure 6-16, provides a conservative upper bound for amplification of peak accelerations in solid waste landfills. They also suggested that the relationship of Idriss (1990) for soft soil sites, shown in Figure 6-16, provides a reasonable representation of average amplification potential of solid waste landfills.

Table 6-8. Earthquake parameters, corresponding peak horizontal bedrock acceleration estimates, and peak horizontal accelerations recorded on the top of the OII Landfill, California (modified from Matasovic et al., 1998).

Earthquake	Moment Magnitude	Style of Faulting	Site-to-Source Distance (km)	Estimated Peak Bedrock Acceleration (g)	Peak Acceleration at Top Deck (g)
Pasadena (3 Dec 88)	5.0	Strike-Slip	13	0.075	0.105
Malibu (19 Jan 89)	5.0	Thrust	50	0.018	0.009
Joshua Tree (23 Apr 92)	6.1	Strike-Slip	163	0.006	0.017
Landers (28 Jun 92)	7.3	Strike-Slip	140	0.032	0.085
Big Bear (28 Jun 92)	6.4	Strike-Slip	119	0.015	0.049
Mojave Desert (11 Jul 92)	5.5	Strike-Slip	131	0.004	0.012
Northridge (17 Jan 94)	6.7	Thrust	43	0.104	0.230

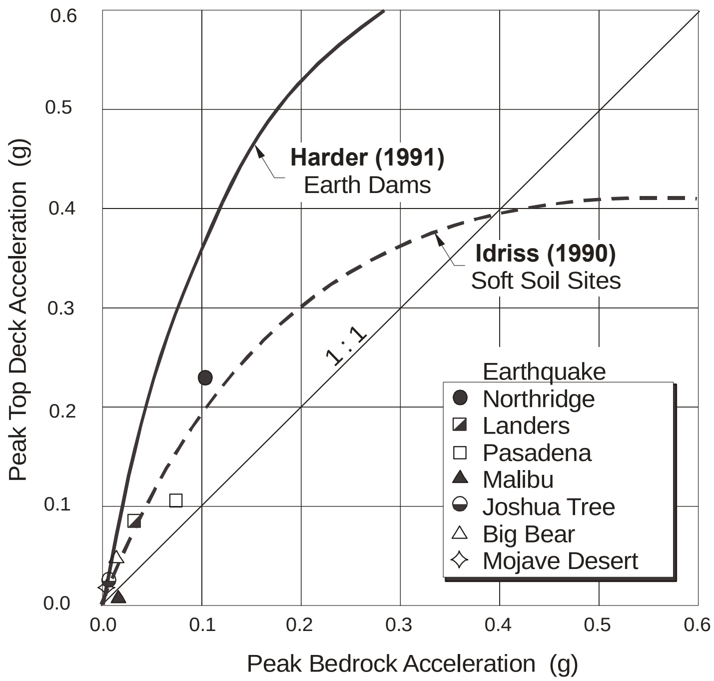


Figure 6-16. OII Data from Matasovic et al. (1998), Harder (1991) Curve for Upper-Bound Amplification of Seismic Acceleration in Earth Dams, and Idriss (1990) Curve for Soft Soil Sites.

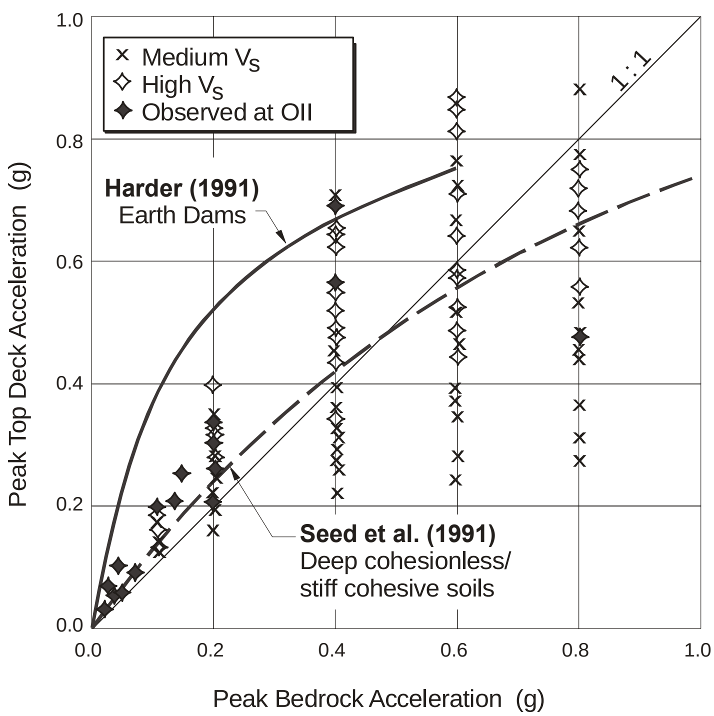


Figure 6-17. Results of Parametric Study Comparing Calculated Peak Horizontal Acceleration for OII Landfill Top Deck and Peak Bedrock Acceleration (modified from Bray and Rathje, 1998).

Independent of Matasovic et al. (1998), Bray and Rathje (1998) used the non-linear one-dimensional dynamic response analysis D-MOD (Matasovic, 1993; Matasovic and Vucetic, 1995) to perform parametric analyses of landfill response for a range of waste properties, waste heights, site conditions, and bedrock ground motions. The results of their parametric evaluation for cover systems are given in Figure 6-17. This figure presents a plot of peak horizontal acceleration at the landfill top deck versus peak horizontal bedrock acceleration. Bray and Rathje (1998) also compared their results to the Harder (1991) curve and the Seed et al. (1991) curve for stiff soil sites. Inspection of Figure 6-17 shows that the Harder (1991) curve provides a conservative upper bound to the calculated cover system accelerations. Bray and Rathje noted that the large amount of scatter in their parametric analysis results is due in large part to the sensitivity of the results to the input ground motion (i.e., variability among earthquakes). Variability in the assumed foundation conditions and waste profile also influenced the results. These findings are significant and engineers should consider this sensitivity when performing and interpreting the results of seismic response analyses.

Until more data become available, the Harder (1991) curve is conservatively recommended as a conservative upper-bound amplification of ground motions for simplified seismic site response of solid waste cover systems. Knowing the peak horizontal bedrock acceleration (from a USGS seismic hazard map, other map, or site-specific analysis), the Harder (1991) curve can be used to estimate an upper-bound peak horizontal acceleration at the top deck of the landfill. Should the Harder (1991) curve result in excessive cover system accelerations, detailed seismic response analyses can be conducted to assess whether a lower value of peak acceleration can be used on a project-specific basis. Site-specific seismic response analyses should also be used for any site where the average shear wave velocity in the upper 30 m of the foundation is less than 120 m/s (i.e., soft soil sites).

6.3.3.4 Analytical and Numerical Response Analyses

A one or two-dimensional seismic site response analyses may be performed for sites where significant cover system accelerations are anticipated or it is necessary to obtain a better estimate of seismically-induced motions in the cover system than can be obtained with the simplified approach. These analyses are also recommended for sites with soft soil foundations and for critical facilities or facilities with special features. Such projects include those in regions with the potential for very large earthquakes, where waste thicknesses are relatively large, or where cover system material or interface shear strengths are particularly low. The site response analysis is performed considering both the foundation soils and waste mass.

The computer program SHAKE, originally developed by Seed and coworkers (Schnabel et al., 1972) and updated by Idriss and Sun (1992), is perhaps the most commonly used computer program for one-dimensional seismic site response analysis. The SHAKE model idealizes the soil (and waste mass) profile as a system of homogeneous, visco-elastic sublayers of infinite horizontal extent. The response of this system is calculated considering vertically propagating shear waves. An equivalent linear analysis accounts for the strain-dependent non-linearity of soil and waste stiffness and damping using an iterative procedure to obtain modulus and damping values that are compatible with the equivalent uniform strain induced in each sublayer. At the outset, a set of properties (shear modulus, damping ratio, and total unit weight) is assigned to each sublayer of the soil or waste deposit. The analysis is conducted using these properties, and the shear strain induced in each sublayer is calculated. The shear modulus and damping ratio for each sublayer are then modified based on the applicable relationship relating these two properties and shear strain (see Figure 6-14).

Basic input to SHAKE includes the soil and waste profile, soil and waste properties, and selected earthquake acceleration time history. Soil and waste properties include the shear wave velocity (v_s) or maximum (small-strain) dynamic shear modulus (G_{max}) and total unit weight (γ_t) for each soil layer plus shear modulus reduction and damping ratio curves for each soil and waste material.

Computer programs are available for equivalent-linear and truly non-linear two and three-dimensional seismic site response analyses. A discussion of these more sophisticated models is provided by Kramer (1996). These models are only occasionally used in cover system design practice. Application of these models to the evaluation of cover system earthquake response often may result in lower-intensity seismically induced cover system motions than obtained using the one-dimensional SHAKE analysis. Use of non-linear analysis methods is recommended when the peak horizontal bedrock acceleration exceeds 0.4 g (Kavazanjian and Matasovic,1995; Bray and Rathje, 1998). Examples of the use of these more sophisticated models are presented by Idriss et al. (1995), Augello et al. (1998), and Matasovic and Kavazanjian (1998), who used the two-dimensional finite element program QUAD4M (Hudson et al., 1994) to evaluate the seismic response of the OII landfill, and Kavazanjian and Matasovic (1995), Bray and Rathje (1998),and Matasovic et al. (1998), who used the one-dimensional non-linear program D-MOD (Matasovic, 1993) to evaluate landfill seismic response. These more sophisticated models should only be applied by experienced geotechnical earthquake engineers, as their application is quite complex.

To perform seismic site response analyses and/or to perform permanent seismic deformation analyses, discussed subsequently, it is necessary to select earthquake acceleration-time histories as an input parameter to the analyses. Acceleration-time histories can be developed either by selecting a representative instrumental (accelerogram) record from the available catalog of records obtained during previous earthquakes or by synthesizing an artificial accelerogram. Acceleration-time histories should be selected for each seismic source having a potentially controlling influence on a site. Both near-field and far-field earthquake events should be considered. A higher magnitude far-field event with sufficient energy near the fundamental period of a solid waste mass may be more damaging to an overlying cover system than a near-field event characterized by a higher peak horizontal bedrock acceleration, higher frequency motions, and a shorter duration. For analysis, each acceleration-time history is scaled to the peak horizontal bedrock acceleration for the site. Selection of a time history from the available catalog of time histories is, in general, a preferable approach as opposed to synthesizing a time history. However, due to limitations in the catalog of available records, it is not always possible to find a representative time history from the catalog, particularly for sites in the eastern U.S. Richardson et al. (1995) and Kramer (1996) provide guidance on the selection of acceleration-time histories for use in seismic analyses.

6.3.4 Dynamic Shear Strength

The dynamic shear strengths of the components and interfaces of a cover system must be estimated to perform seismic slope stability and/or deformation analyses. These estimates are typically based on static or cyclic undrained shear strength tests. Shaking-table laboratory test results and observed earthquake performance of cover system components and interfaces are also used to develop information on cover system performance in earthquakes. Information on the cyclic shear strength of soils used in cover systems can be obtained from the geotechnical earthquake engineering literature (e.g., Kramer, 1996; Kavazanjian et al., 1997; and Lai et al., 1998). Shear strengths of CCLs, dry GCLs, and unsaturated granular soils typically used in cover systems are not significantly degraded by seismic loading, and cyclic shear strength is assumed to equal static shear strength. For hydrated unreinforced GCLs, this may not be the case, depending on the anticipated stress level and number of cycles of loading (Lai et al., 1998). The limited available data on the cyclic shear strength of interfaces involving geosynthetics (Kavazanjian et al., 1991; Yegian and Lahalf, 1992, Augello et al., 1995; Yegian et al., 1995; and Chaney et al., 1997) suggest that cyclic shear strengths of geosynthetics can be approximated using the results of static shear strength tests.

6.3.5 Seismic Stability and Deformation Analysis

6.3.5.1 Overview

The static LE slope stability analysis methods discussed previously in this document may be adapted for use in the seismic stability evaluation of cover systems. This adaptation can be achieved using a number of different approaches, of which the following three represent the current state of practice: (i) the pseudo-static factor of safety method; (ii) the modified pseudo-static factor of safety method; and (iii) the permanent seismic deformation method. These three approaches are discussed below.

6.3.5.2 Psuedo-Static Factor of Safety Method

Due to its simplicity, the psuedo-static factor of safety method remains the most common method of analysis used in practice for seismic design of cover systems. With this approach, the factor of safety for the cover system is calculated using a LE analysis that incorporates a specified seismic coefficient that is applied as a horizontal body force to the potential slide mass. The factor of safety obtained for the calculation is compared to a minimum acceptable factor of safety to determine the adequacy of the design. The seismic coefficient equals the fraction of the weight of the potential failure mass that is applied as a horizontal force to the centroid of the mass in a pseudo-static limit equilibrium stability analysis.

For the case of an infinite slope with no water flow, the pseudo-static factor of safety is given by:

$$FS = \frac{(\cos\beta - k_h \sin\beta)\tan\phi_i}{(\sin\beta + k_h \cos\beta)} + \frac{a_i}{\gamma_t t(\sin\beta + k_h \cos\beta)} \qquad \text{(Eq. 6.17)}$$

where: k_h = pseudo-static seismic coefficient (dimensionless); and all other terms are as defined previously.

For the case of a slope of finite length, the pseudo-static factor of safety can be calculated, for the case of no water flow, using the approximate solution for sliding of the two-part wedge shown in Figure 6-4 (Matasovic et al., 2002):

$$FS = \frac{A}{B}\tan\phi_i + \frac{a_i}{B\gamma_t t} + \frac{t}{2h}\left(\frac{\sin\beta\tan\phi_s}{1-(B/A)\tan\phi_s}\right)\left(\frac{1+k_h^{\ 2}}{AB}\right)^2 + \frac{c_s}{\gamma_t h}\left(\frac{\sin\beta}{1-(B/A)\tan\phi_s}\right)\left(\frac{1+k_h^{\ 2}}{AB}\right) + \frac{T/h\sin\beta}{B\gamma_t t} \qquad \text{(Eq. 6.18)}$$

where A is a dimensionless parameter, given by:

$$A = \cos\beta - k_h \sin\beta$$

and B is also a dimensionless parameter, given by:

$$B = \sin\beta + k_h \cos\beta$$

and all other terms were defined previously. Note that if $\phi_s = 0$, $c_s = 0$ and $T = 0$, Eq. 6-18 reduces to Eq. 6-17.

The case of a slope of finite length has also been addressed by Koerner and Daniel (1997), who provide a solution that requires the solving of a quadratic equation. For more complicated geometries and slope conditions, design calculations are more easily performed using one of the LE slope stability computer programs described previously. It is common in performing seismic stability analyses of cover systems to assume no water flow in the slope. The rationale for this assumption is that the probability of occurrence of both a design-level earthquake event and a design-level storm event at the same time is extremely low.

The main drawback of the pseudo-static factor of safety approach lies in the difficulty in relating the value of the seismic coefficient to the characteristics of the design earthquake. Use of the peak acceleration at the top of the waste mass as the seismic coefficient, coupled with a psuedo-static factor of safety of 1.0, results in a very conservative design basis. This result would imply no displacement of the cover system during the design earthquake, not even for the milli-seconds during which the peak accelerations are applied. A seismic coefficient smaller than that corresponding to the peak ground acceleration is sometimes used, but the magnitude of cover system displacement in this case is unknown.

6.3.5.3 Modified Pseudo-Static Factor of Safety Method

The problem of selecting an appropriate seismic coefficient for the pseudo-static approach can be addressed by implicitly considering the potential for seismically-induced deformations. Based on Hynes and Franklin (1984), Richardson et al. (1995) suggested that seismically-induced displacements in a slope will be less than 0.3 m if the yield acceleration, $k_y g$ (m/s^2), defined as the horizontal acceleration producing a pseudo-static factor of safety of 1.0, is no less than 50%

of the peak horizontal acceleration of the slope (i.e., cover system). This result represents an upper-bound value for the seismic deformations calculated by Hynes and Franklin (1984) using almost 400 earthquake strong motion records. The value of $k_y g$ required to produce 0.3 m of permanent seismic displacement drops to about 15% of the peak horizontal acceleration if the mean plus one standard deviation curve is considered rather than the upper-bound curve. Other values of $k_y g$ can be derived from Figure 6-18.

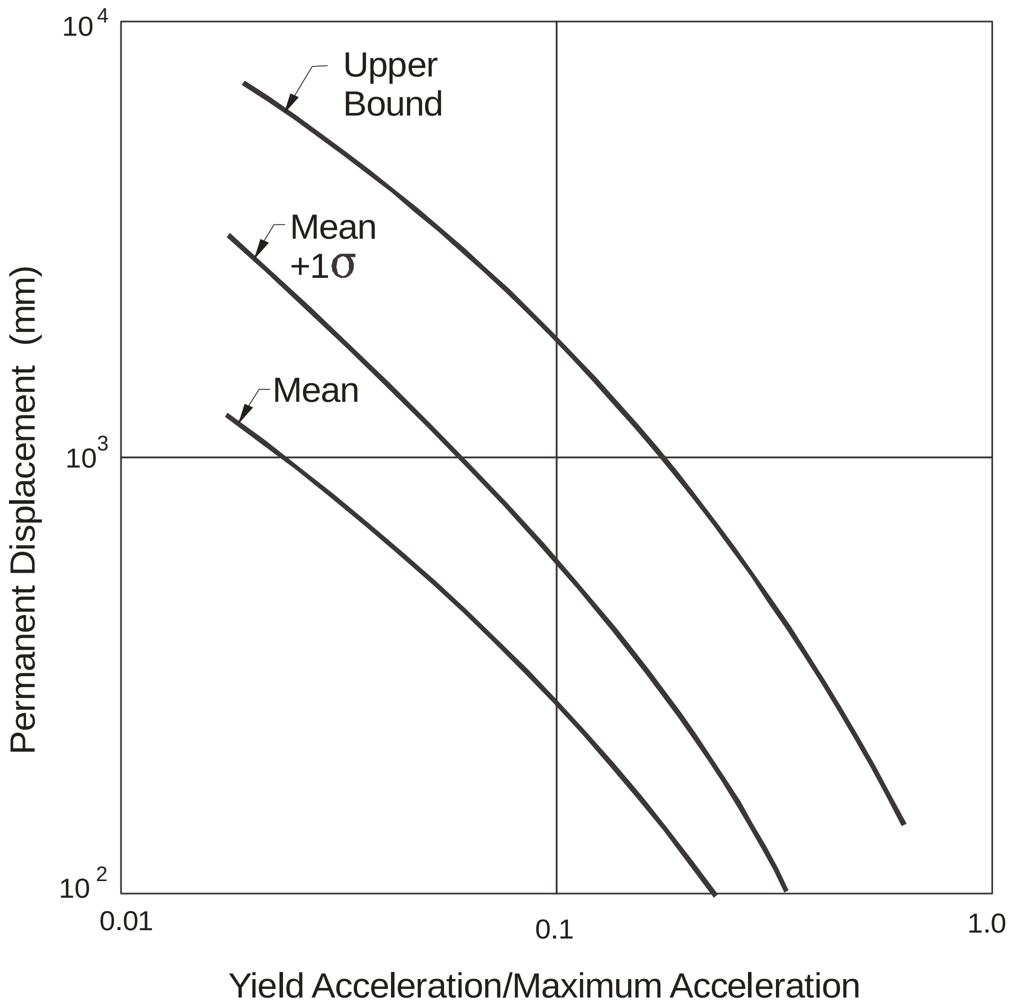


Figure 6-18. Hynes and Franklin (1984) Permanent Seismic Displacement Chart (modified from Richardson et al., 1995).

Kavazanjian (1998) presented a refined procedure for deriving a displacement-based seismic coefficient value specifically for the design of cover systems for solid waste landfills. Seismic coefficient values for specified levels of permanent seismic displacement are calculated by multiplying a ratio, obtained from Table 6-9, by the peak horizontal acceleration of the cover system obtained using the Harder (1991) curve shown in Figure 6-14. Kavazanjian has suggested that for earthquakes of magnitude less than or equal to 6.5 within 10 km of the project site, and for any earthquake of magnitude less than or equal to 5.5, the mean ratios in Table 6-9 be used. Kavazanjian further recommended that for earthquakes of magnitude greater than 6.5, and for earthquakes between magnitude 5.5 and 6.5 that are more than 10 km from the project site, the mean plus one standard deviation ratios in Table 6-9 be used. Kavazanjian (1998) recommended that seismic coefficients derived using Table 6-9 should be used with a factor of safety of 1.0. It is cautioned that the use of peak shear strength parameters with this approach is unconservative. Shear strength values should be selected considering the displacement value from Table 6-9 associated with the chosen seismic coefficient. It is noted that this simplified approach is not recommended for soft soil sites; soft soil sites should be evaluated using a site-specific seismic response analysis and permanent seismic displacement analysis with acceleration-time histories selected as previously described in this chapter.

Table 6-9. Ratio of yield acceleration, $k_y g$, to peak acceleration of cover system as a function of calculated permanent seismic displacement (based on Hynes and Franklin (1984) curves shown in Figure 6-15). Note: σ = standard deviation.

Calculated Displacement (mm)	Mean Ratio	Mean + 1σ Ratio
100	0.23	0.35
150	0.17	0.27
300	0.08	0.17
500	0.05	0.11
1,000	0.03	0.06

6.3.5.4 Permanent Seismic Deformation Method

With the permanent seismic deformation method, cumulative permanent seismic deformations are calculated on the basis of that portion of the acceleration-time history of the cover system that exceeds $k_y g$. For the infinite slope case, k_y is calculated using Eq. 6.17 and FS = 1.0. For the case of a finite length slope with uniform soil thickness above the critical potential slip surface, k_y is calculated using Eq. 6.18 and FS = 1.0. For more complex cases, k_y is calculated using a LE slope stability computer program.

The actual calculation of permanent seismic displacement is usually performed using Newmark's "sliding block on a plane" method of analysis (Newmark, 1965). In a Newmark analysis, acceleration pulses (in the earthquake acceleration-time history) exceeding $k_y g$ are double-integrated to calculate the accumulated "permanent" seismic displacement (Figure 6-19). Theoretically, this calculated permanent displacement is a rigid body displacement that accumulates everywhere along the critical potential slip surface. Typically, only the horizontal component of the earthquake acceleration-time history is considered in the analysis. The acceleration-time history of the cover system used in the analysis is obtained from a seismic response analysis. With this approach, the response analysis is "decoupled" from the computation of permanent displacement (i.e., seismic response is calculated assuming no slip displacement between the cover system and landfill, and cover system displacement is calculated using the results of the seismic response analysis (Bray and Rathje, 1998). The decoupled approach is generally conservative for cover system displacement analyses.

Several commercially-available, PC-based computer programs exist to perform Newmark analyses (Houston et al., 1987; Yan et al., 1996). These models assume a constant value of k_y. Recognizing that most geosynthetic materials and interfaces exhibit strain-softening shear behavior, Matasovic et al. (1997) proposed a modification to the standard Newmark procedure

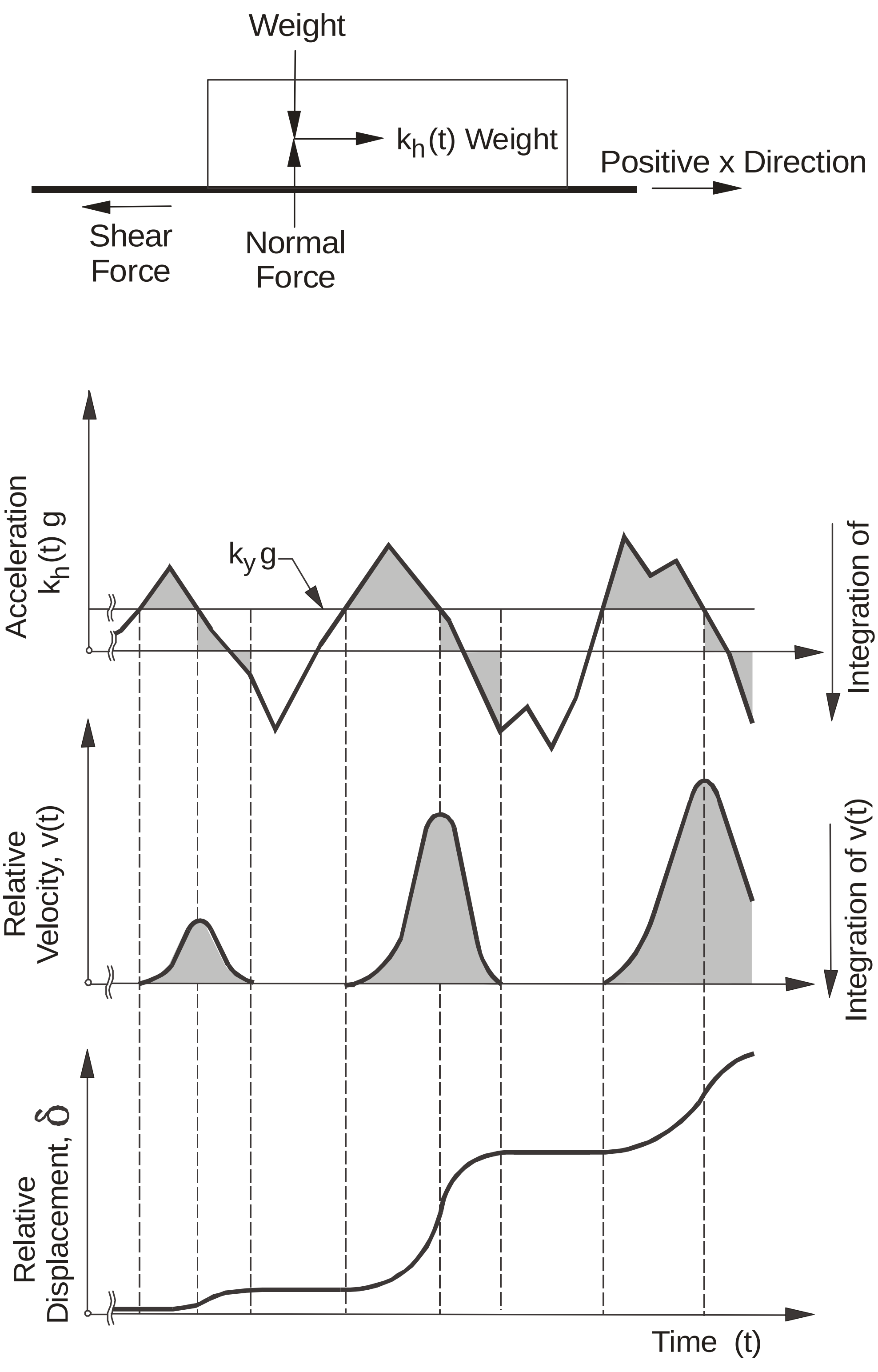
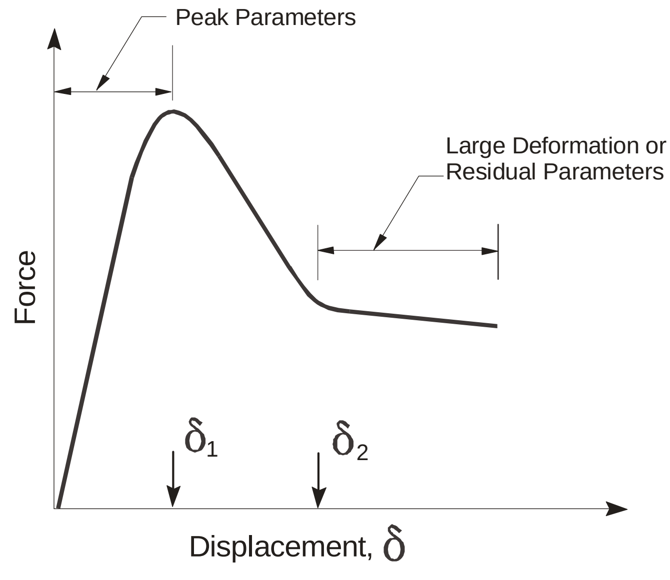
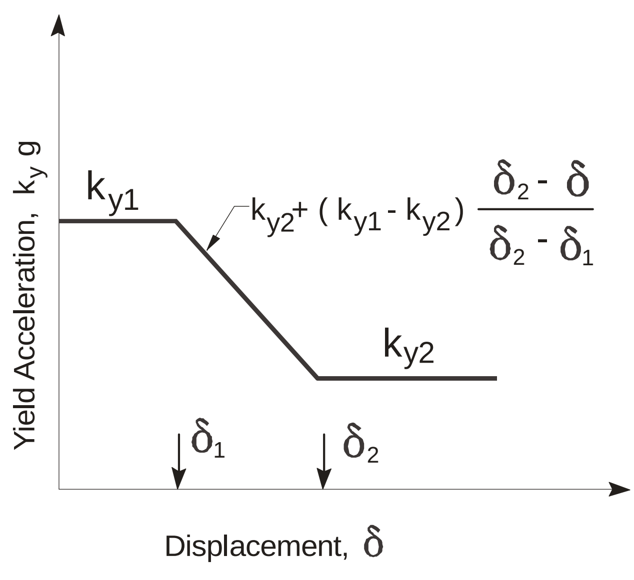


Figure 6-19. Basic Elements of Classical Newmark Sliding-Block Analysis with Constant Yield Acceleration.



(a)



(b)

Figure 6-20. Yield Acceleration Degradation Model (modified from Matasovic et al., 1997). $k_y = k_{y1}$ and is Based on Peak Strength Parameters at Displacements up to the Displacement at Peak Strength (δ_1). $k_y = k_{y2}$ and is Based on Residual Strength Parameters at Displacements Greater than the Displacement at Residual (or Large-Displacement) Strength (δ_2). At Displacements Between δ_1 and δ_2, k_y Varies Linearly Between k_{y1} and k_{y2}.

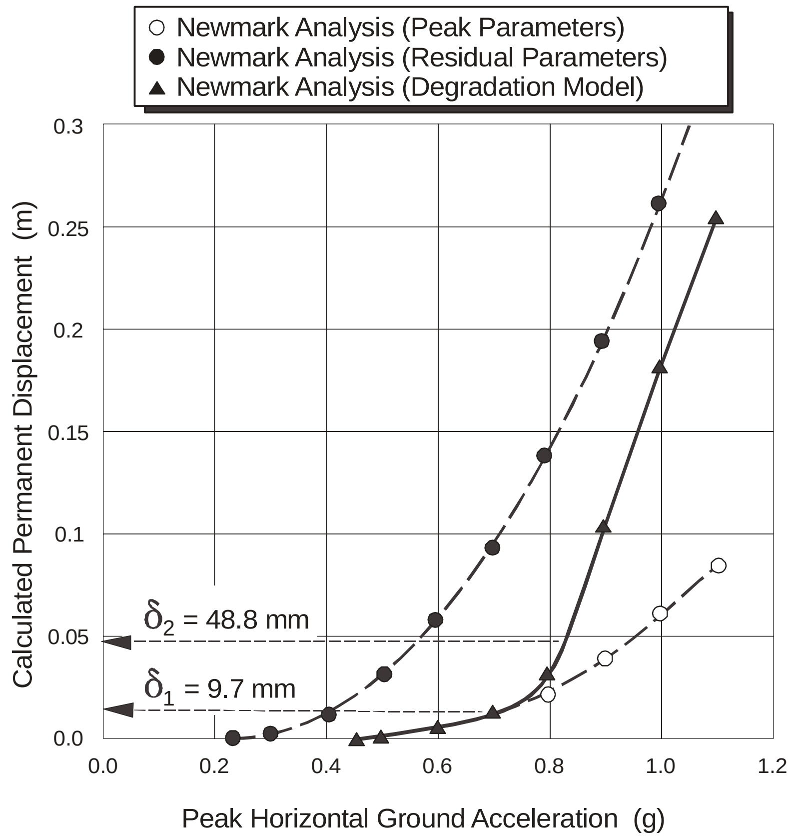


Figure 6-21. Results of Newmark Seismic Deformation Analysis for Constant and Degrading Yield Acceleration at a Normal Stress of 20.7 kPa (modified from Matasovic et al., 1997). δ_1 and δ_2 are as Defined in Figure 6-20.

specifically for cover systems incorporating geosynthetic interfaces. The modified version incorporates a linear k_y degradation model to account for strain-softening materials and interfaces (Figure 6-20). Matasovic et al. (1997) demonstrated the sensitivity of the calculated permanent seismic deformation for a typical GT/CCL interface and three differing assumptions regarding k_y: (i) constant, based on peak interface shear strength parameters; (ii) constant, based on residual (or large-displacement) interface shear strength parameters; and (iii) degrading, in accordance with Figure 6-20. Figure 6-21 presents typical calculation results from Matasovic et al. (1997) for the post-peak strain-softening exhibited by a GT/CCL interface. The sensitivity of the calculation results to the k_y assumption is evident.

6.3.5.5 Seismic Deformation Performance Criteria

In the current state-of-practice for design of cover systems, it is common to require permanent seismic deformations calculated using a conservative, Newmark-type approach to be less than 150 to 300 mm (Seed and Bonaparte, 1992; Anderson and Kavazanjian, 1995). Smaller values are sometimes considered if the potential slip surface underlies a non-ductile critical component, such as a HDPE GM. Larger deformations are sometimes considered if the potential slip surface is above all non-ductile critical components. Inherent in the selection of an allowable displacement value is an understanding that the calculation methodology is conservative, and actual earthquake-induced deformations would be less than calculated. In this regard, some engineers prefer to view the calculated seismic displacement as a performance index as opposed to a true prediction of actual deformations.

In applying seismic performance criteria to cover systems, several factors can be considered that do not typically apply to liner systems. First, the condition of a cover system can be readily observed after an earthquake through a post-earthquake inspection program. Second, the potential adverse impacts associated with excessive deformation of a cover system will involve tearing of geosynthetics, cracking of soils, disruption of gas management systems, and disruption of surface-water management systems. The risk of personal injury or environmental impact resulting from these types of problems will typically be small. The damage to cover systems caused by seismic displacement is typically repairable, although some at considerable cost and effort. For these reasons, it may be acceptable in some cases to consider calculated permanent displacements that are near the upper limit of the current state-of-practice for cover system applications.

Kavazanjian (1998) proposed two criteria for the seismic design of cover systems: (i) design without damage; and (ii) design accepting some limited damage to the cover system, but without "harmful discharge." For the "no damage" criterion, Kavazanjian suggested that a calculated permanent seismic displacement of up to 300 mm is acceptable for simplified analyses which use upper bound displacement curves from generic Newmark displacement charts (e.g., Hynes and Franklin, 1984), residual shear strengths, and/or simplified seismic response analyses. Kavazanjian further suggested that a calculated permanent seismic displacement of up to 150 mm represents an acceptable "no damage" criterion in cases where more sophisticated analyses are used to calculate the permanent seismic displacement using project-specific seismic response and formal Newmark displacement analyses.

For the case of "no harmful discharge", Kavazanjian (1998) suggested that a permanent deformation criterion of up to 1 m may be acceptable. With respect to this criterion, Kavazanjian (1998) states:

> "*When designing a cover system to withstand* [an earthquake] *without discharge, provisions are needed to mitigate potential hazards associated with discharge of leachate and/or gas from disrupted conveyance systems (e.g., use of automatic shut-off valves, secondary containment, and/or articulated seismic joints) and facilitate post-earthquake repair of damage.*"
>
> "*Multiple penetrations through geomembrane cover elements for gas and leachate collection or other purposes may limit allowable displacement to less than 1 m on an economic basis due to the cost of repair. However, if the anticipated displacement is above the geomembrane, there are not penetrations through the geomembrane on slopes, and benches provide sufficient capacity to retain cover soil that sloughs from above, the allowable seismic displacement of a geosynthetic landfill cover system may be unlimited, provided the owner is prepared to repair and/or replace the protective soil cover and drainage layer (if any) on top of the geomembrane after a severe earthquake.*"

The choice of a "no damage" or "no harmful discharge" design criterion will need to be made on a case-by-case basis by the design engineer, facility owner, and regulatory agency. Obvious factors that should be considered in choosing a criterion are: (i) potential impacts of large displacements; (ii) type of waste being covered; (iii) cost to repair cover system damage; and (iv) level of assurance that personnel and funds will be available for post-earthquake inspections and repairs after the earthquake occurs.

6.4 Settlement

6.4.1 Mechanisms of Settlement

Cover systems may be subject to settlements resulting from a variety of mechanisms. For purposes of evaluating cover system performance, settlements can be considered to have one of three sources (see Figure 6-22): (i) settlement of foundation soil; (ii) settlement due to overall waste mass compressibility; and (iii) settlement due to localized mechanisms in the waste.

Angular distortion or differential settlement may: (i) induce unacceptable tensile stress and strain in one or more cover system components, which can lead to component tearing or cracking; or (ii) cause cover system slopes to change or reverse grade which, in turn, can affect the performance of the cover system drainage layer and/or gas collection layer.

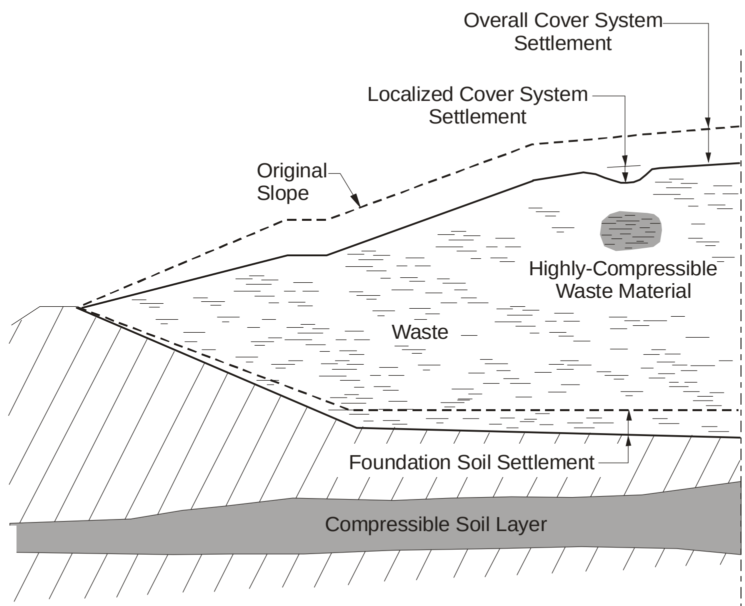


Figure 6-22. Sources of Cover System Settlement (modified from Othman et al., 1995).

6.4.2 Settlement of Foundation Soils

Impacts of foundation settlement on the performance of a cover system are usually not significant. Occasionally, situations arise where foundation settlements are of sufficient magnitude to affect the cover system design. For example, if the waste mass is underlain by a thick layer of soft clay, consolidation settlements can be large. Both primary settlement and long-term secondary settlement should be considered. Calculations are performed using equations from conventional geotechnical engineering practice (e.g., Holtz and Kovacs, 1981;

Lambe and Whitman, 1969) and a timeframe at least equal to the active life and post-closure care period of the facility.

6.4.3 Overall Waste Compression

Overall waste mass compressibility results in area-wide waste mass settlement. The potential for waste settlement is highly dependent on the type of waste. Relative to most other wastes, MSW is very compressible, due to both its initial compressibility when placed and the additional compressibility induced by the biodegradation of the organic component of the MSW. This latter component creates a significant time dependency to waste settlement. Other types of facilities that can undergo large settlement include impoundments containing high water content industrial sludges (typically inorganic). Materials such as mine waste, ash and slag, construction and demolition waste, and soil waste have relatively lower settlement potential. The following discussion of overall waste settlement focuses primarily on the settlement potential of MSW waste and other highly compressible waste material. The evaluation of ash, soil waste, or other low-compressibility inorganic waste is typically performed using equations for conventional geotechnical engineering practice (e.g., Holtz and Kovacs, 1981, Lambe and Whitman, 1969).

MSW waste compression results from complex factors including (Sowers, 1973; Edil et al., 1990; Sharma and Lewis, 1994):

- mechanical compression due to self-weight and surface loads;
- raveling (i.e., movement of fines into larger voids);
- physiochemical changes, including corrosion, oxidation, and combustion; and
- biochemical decomposition under aerobic and anaerobic processes.

The magnitude and rate of MSW settlement are controlled by many factors, among which are the waste fill height, organic content, age, moisture content, degree of compaction, and temperature. Figure 6-23 presents data from Edgers et al. (1992), König et al. (1996), and Spikula (1996), which shows that MSW landfills can settle from about 5 to 20% (and even up to 30%) of the initial landfill thickness (measured from the time the landfill first reached final grade).

A number of methods have been proposed for evaluating the short-term and long-term compression of waste. Three settlement models that have been adopted from geotechnical engineering and applied to waste are: (i) one-dimensional compression model; (ii) power creep model; and (iii) Gibson and Lo model (Gibson and Lo, 1961). A discussion of the latter two models is presented in Edil et al. (1990), and they are not discussed further herein. Presently, there is little experience in applying these last two models, and their applicability to the prediction of long-term settlements is not well demonstrated.

Conventional one-dimensional compression models have been widely used to estimate waste settlements (Sowers, 1973; Yen and Scanlon, 1975; Rao et al., 1977; Burlingame, 1985; Landva and Clark, 1990; Morris and Woods, 1990; Fassett et al., 1994; Stulgis et al., 1995). However, it is often assumed that primary self-weight settlement is complete prior to installation of the cover system. Thus, it is assumed that calculated primary settlements do not directly influence cover system performance.

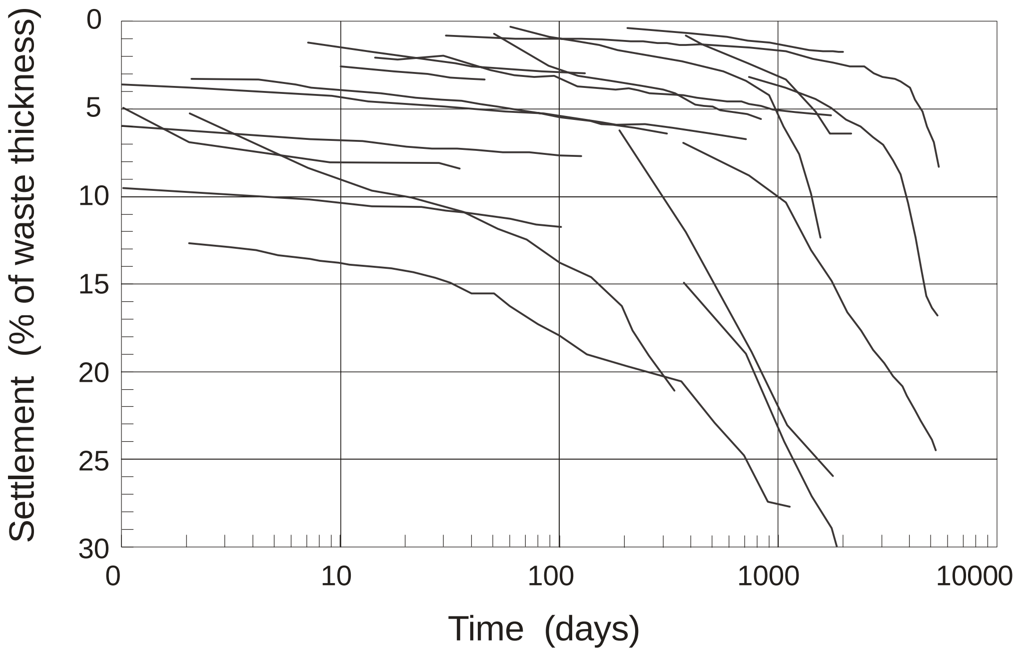


Figure 6-23. Total Settlement Data from Edgers et al. (1992), König et al. (1996), and Spikula (1996) for MSW Landfills, Measured from the Time the Landfill Reached Final Grade.

Cover system performance will be affected, however, by secondary waste settlement. The secondary waste settlement, ΔH_s (m), that occurs between times t_1 and t_2 is calculated with the one-dimensional model using an equation of the form:

$$\Delta H_s = C_{\alpha\varepsilon} H_1 \log \frac{t_2}{t_1} \qquad \text{(Eq. 6.19)}$$

where: $C_{\alpha\varepsilon}$ = modified secondary compression index (dimensionless); H_1 = height of waste at time t_1 (m); t_1 = starting time for the period of secondary compression (s); and $t_2 = t_1$ plus the time duration of secondary compression (s). Use of Eq. 6.19 implies that the magnitude of secondary settlement is independent of the applied stress. A modified form of Eq. 6.19 has been suggested by Bjarngard and Edgers (1990) and Stuglis et al. (1995) to account for a variable value of $C_{\alpha\varepsilon}$ between "intermediate" and "long-term" secondary compression times. Their equation is formulated herein as:

$$\Delta H_s = C_{\alpha\varepsilon 1} H_1 \log \frac{t_2}{t_1} + C_{\alpha\varepsilon 2} H_2 \log \frac{t_3}{t_2} \qquad \text{(Eq. 6.20)}$$

where: $C_{\alpha\varepsilon 1}$ = modified secondary compression index during the intermediate secondary compression period (dimensionless); $C_{\alpha\varepsilon 2}$ = modified secondary compression index during the long-term secondary compression period (dimensionless); H_1 = height of waste at time t_1 (m); H_2 = height of waste at time t_2 (m); t_1 = starting time for the period of secondary compression (s); $t_2 = t_1$ plus the time duration of intermediate secondary compression (s); and $t_3 = t_2$ plus the time duration of long-term secondary compression (s). Inspection of Figure 6-23 suggests that for some MSW materials, $C_{\alpha\varepsilon}$ is more or less constant during the period for which data exist, while for other facilities, a variable $C_{\alpha\varepsilon}$ could be used to better fit the data.

The reader should be aware that the choice of a value of $C_{\alpha\varepsilon}$ cannot be made without consideration of the normalization term t_1. For a given $C_{\alpha\varepsilon}$, the calculated value of ΔH_s will be significantly affected by the choice of t_1. Ideally, $C_{\alpha\varepsilon}$ and t_1 should be selected by empirically fitting Eq. 6.19 or Eq. 6.20 to field settlement data. In the absence of this type of correlation, it is suggested that t_1 be taken as the time period between when waste reaches final grade and when the cover system is installed over the waste.

Since $C_{\alpha\varepsilon}$ and t_1 are empirically derived, ΔH_s is assumed to be independent of applied effective stress, and the primary purpose of calculating ΔH_s herein is to assess potential impacts to the performance of the cover system, it is not necessary to subdivide the waste mass into a series of horizontal layers for purposes of calculating ΔH_s. With this approach, calculations are typically performed for increasing time intervals after closure to obtain a relationship between cover system settlement and elapsed time since closure.

Values of $C_{\alpha\varepsilon}$ for MSW reported in the technical literature have generally been in the range of 0.01 to 0.1 (Sowers, 1973; NAVFAC, 1983; Burlingame, 1985; Landva and Clark, 1990; Fassett et al., 1994; Stulgis et al., 1995). Given the empirical nature of $C_{\alpha\varepsilon}$ and t_1, it is interesting to compare calculated values of $(\Delta H_s/H_1)$ obtained using Eq. 6.19 to the range of observed time-dependent post-closure settlements for MSW (Figure 6-20). For the comparison, the waste mass is considered as a single unit with an average t_1 value of 100 days (approximately 3 months). Table 6-10 presents calculated values of $\Delta H_s/H_1$ (in percent) for values of $C_{\alpha\varepsilon}$ ranging from 0.01 to 0.1 and post-closure times of 100, 1,000 and 10,000 days (to which 100 days are added to obtain t_2 values).

Table 6-10. Results of parametric study of calculated post-closure secondary settlements (ΔH_s) as a percentage of initial landfill height (H_1).

$C_{\alpha\varepsilon}$	$(t_2 - t_1)$ (days after closure)		
	100	*1,000*	*10,000*
0.01	0.30	1.0	2.0
0.03	0.90	3.1	6.0
0.06	1.8	6.2	12.0
0.10	3.0	10.4	20.0

Based on the calculated values in Table 6-10, $C_{\alpha\varepsilon}$ values less than about 0.03, coupled with t_1 values of 100 days, are too small to model MSW time-dependent settlements. Careful review of the source references used to develop Figure 6-23 suggests that $C_{\alpha\varepsilon}$ values in the range of 0.04 to 0.08, coupled with t_1 values of about 100 days, provide a reasonable modeling of the settlement trends for modern MSW landfills that are typically filled fairly quickly and compacted using heavy trash compactors. Larger values of $C_{\alpha\varepsilon}$, in the range of 0.08 to 0.12, coupled with t_1 = 100 days, are needed to model the settlement trends in some of the older landfills in the source database. These landfills may have been filled with more variable waste placed under conditions less controlled than for modern landfills. Larger values of $C_{\alpha\varepsilon}$ would also be expected for landfills undergoing leachate recirculation or otherwise managed to increase biological activity and methane production in the waste mass.

If t_1 is assumed to be 30 days rather than 100 days, calculated $\Delta H_s/H_1$ values at 10,000 days would be about 25% larger than given in Table 6-10. Thus, if t_1 = 30 days is assumed, $C_{\alpha\varepsilon}$ values should be reduced about 25% from the recommended ranges given above. This calculation exercise clearly points out the sensitivity of calculated $\Delta H_s/H_1$ values to the magnitude of t_1.

6.4.4 Differential Settlement Due to Localized Mechanisms

Localized settlements, in the form of depressions, can develop within the first several years after cover system installation over MSW. These types of localized occurrences appear to be more common in older waste fills where a number of factors may contribute to the problem, including: (i) little initial waste compaction; (ii) variable waste characteristics; (iii) placement of sludges in the waste fill; and/or (iv) poor surface-water control leading to ponding of water on the waste. Localized differential settlement can lead to excessive stress or strain in cover system components (Gilbert and Murphy, 1987). Localized differential settlement of waste is generally attributed to one or more of several mechanisms, namely: (i) deterioration and collapse of objects (e.g., drums) in the waste; (ii) settlement associated with a highly-compressible zone of waste; and (iii) migration or raveling of waste particles into underlying voids.

Typically, analyses to evaluate impacts of localized differential settlement on the cover system are not performed as part of cover system design. However, in a few situations it may be necessary to evaluate potential effects of localized areas of high waste compressibility on cover system performance (e.g., cover systems for old dumps where the composition of waste is unknown or there is reason to believe that significant local waste heterogeneity may exist (due to any of the factors described above)). Several analysis methods are available for use in evaluating the potential effects of localized settlements on cover system performance. None of the methods have been field calibrated to any significant degree and selection of input parameters to the analyses is based primarily on engineering judgment. The analysis methods include:

- the application of mine subsidence models to the prediction of waste differential settlements (Murphy and Gilbert, 1985);
- an approach based on the uncoupled combined use of the tensioned membrane and soil arching theories for analyzing the stresses and strains in geosynthetics (such as geosynthetic layers within a cover system) that lose foundation support after construction due to development of a foundation void or depression (Giroud et al., 1990); Poorooshasb (1991) used a somewhat different analytical approach to address a similar problem;
- a boundary element formulation to model deformations around a collapsing void within an existing waste mass (Jang and Montero, 1993);
- two-dimensional finite element analyses to evaluate the response of a waste mass containing compressible zones (Carey et al., 1993); and
- the displacement method of Sagaseta (1987) to evaluate the response of a cover system over a waste mass containing localized compressible zones (Othman et al., 1995)

In the situation where differential settlement is likely to occur and the localized depressions cannot be eliminated, the choices are (i) continuously grading and maintaining the site; (ii) installing a thick buffer soil or waste prior to cover system construction; or (iii) installing geosynthetic reinforcement beneath the cover system. One or more of the analysis methods described above can be used to design soil buffer or geosynthetic (geogrid or high strength GT) support systems. The critical design parameters in any such analysis are the locations and dimensions of the anticipated localized depression or void. Since it is generally not possible to predict where such a feature will occur, any buffer soil or reinforcement layer, if used, will typically need to be installed over the entire waste mass.

6.4.5 Impacts of Settlement on Cover Systems

In design, settlement profiles accounting for the various settlement mechanisms are developed to evaluate potential impacts to the cover system. The evaluation usually considers: (i) post-settlement cover system grades; (ii) potential for depressions and ponding in the cover system; and (iii) stresses and strains in cover system components. Post-settlement grades should be adequate to shed runoff, prevent ponding, and prevent excessive stress or strain in cover system components, particularly the CCL, GCL, and GM hydraulic barriers.

Tensile strains causing cracking in compacted clays have been evaluated by Leonards and Narain (1963); Ajaz and Parry (1975a,b, 1976); Gaind and Char (1983); Chandhari and Char (1985); Jessberger and Stone (1991); and Lozano and Aughenbaugh (1995). Based on these studies, compacted clays tested under unconfined or low confinement conditions exhibit relatively brittle behavior and reach failure at axial extensional strains of 0.02 to 4%, with most compacted clays exhibiting failure at extensional strains of 0.5% or less. The studies also showed that the magnitude of tensile strain causing cracking increases with increasing percentage of fines and water content, and with increasing confining stress.

LaGatta et al. (1997) evaluated the impact of differential settlement on the hydraulic conductivity of GCLs overlain by a 0.6-m thick layer of pea gravel. The GCLs were tested either dry or hydrated and either intact or with a 230 mm overlap. The overlapped GCLs were tested across the overlap. The angular distortions (see Figure 2-13) of the upper surface of the GCLs were monitored and used to calculate tensile strain. The results of their evaluation indicate that intact and overlapped samples of needlepunched GCLs can withstand angular distortions of 0.35 to 0.6, equivalent to tensile strains of 5 to 16%, while maintaining a saturated hydraulic conductivity of 1×10^{-9} m/s or less. Stitch-bonded GCL samples were found to achieve the same hydraulic conductivity criterion up to an angular distortion of 0.35, equivalent to a tensile strain of 5%. For GT-encased, non-reinforced GCL samples, the maximum allowable angular distortion was much less, only 0.1, which is equivalent to a tensile strain of about 1%. This type of GCL, which is no longer available, had an open weave GT on its lower surface. The GT provided essentially no support to the GCL and allowed bentonite to migrate downslope within the depressed area and experience significant swelling. At the end of the test, the thickness of hydrated bentonite was approximately 5 mm on the sides of the depression and 50 mm on the floor of the depression. GCLs samples consisting of bentonite adhered to a GM maintained an equivalent hydraulic conductivity of 1×10^{-9} m/s or less when subjected to angular distortions of up to 0.8, producing a maximum tensile strain of almost 30%. Migration of bentonite was not observed because the GM component of the GCL blocked most of the flow. Within the test area, the GCL was only hydrated along the part of the overlap.

The tensile behavior of GMs varies depending on the polymer type, stress-strain characteristics, susceptibility to stress cracking, temperature, and other factors. The present state-of-practice for the design of strain-softening GM barriers (e.g., polyethylene GMs) is to limit the allowable GM tensile stress (or strain) to the short-term yield value divided by a factor of safety. The allowable tensile stress (or strain) for GMs exhibiting strain-hardening behavior (e.g., PVC GMs) is based on the short-term failure (break) value divided by a factor of safety. It should also be remembered that GMs are designed to be barriers, not tensile inclusions (as is geosynthetic reinforcement, for example). The long-term stress-strain, creep, and brittle fracture behavior of these materials under stress is not well understood. To the extent possible, applications should be designed to minimize tensile stresses and elongations in GMs.

The authors recommend that when it is necessary to specify allowable geosynthetic tensile stresses and strains, the yield stress and strain of the GM material be determined in a wide-width tension test (for plane deformation) or axisymmetric tension test (for spherical deformation) and that the factor of safety used to calculate the allowable values be at least five. The factor of safety should be applied to the yield values for strain-softening GMs and to the failure (break) values for strain-hardening materials. This recommendation should be conservative for virtually all types of commercially-available GMs used in cover system applications. If a higher value of allowable tensile stress or strain is desired, the design engineer must demonstrate that the product to be specified can sustain the allowable values without long-term creep, brittle rupture, or other type of long-term problem. This demonstration must also apply to GM seams.

6.5 Steep Slopes

6.5.1 Introduction

Occasionally, in the closure of old, existing landfills, it is necessary to address the issue of steep existing waste slopes. One option is to cut the slope back to a shallower grade by excavation and then relocate the excavated waste either on-site or off-site at another landfill (Figure 6-10). The advantage of this approach is that it increases the stability of the waste mass and results in a final slope inclination within the "conventional" range for cover systems. Disadvantages associated with waste excavation and relocation include construction-related instability, health and safety concerns associated with exposing the waste, leachate generation, nuisance (e.g., odor) issues, waste characterization necessary for on-site or off-site disposal of the excavated waste, and cost.

Several alternative approaches exist for constructing cover systems over steep waste slopes without need for waste excavation, or at least with very limited waste excavation. These alternatives include the use of: (i) a waste buttress coupled with a conventional slope cover system (Figure 6-11); or (ii) a steep slope cover system. Both of these alternatives are illustrated below, primarily in the form of case studies illustrating their use.

6.5.2 Waste Buttress

Two examples of the use of a waste buttress in the closure of old, existing landfills are presented below in order to illustrate the application of this technology.

Cargill and Olen (1997, 1998) describe the closure of an inactive hazardous waste landfill located on Long Island, New York. The landfill covers approximately 19 ha and has waste slopes extending up to 42 m above the surrounding ground surface. The steepness of the existing waste slopes, with inclinations up to 1H:2V and an average inclination steeper than 2H:1V, prevented the use of a conventionally-designed cover system. Regrading of the landfill to achieve slopes on which a conventional cover system could be constructed was not feasible due to limits on the final landfill height and lack of alternate landfilling locations for the excavated waste. For this project, the cover system design for the steeper slope sections incorporated a

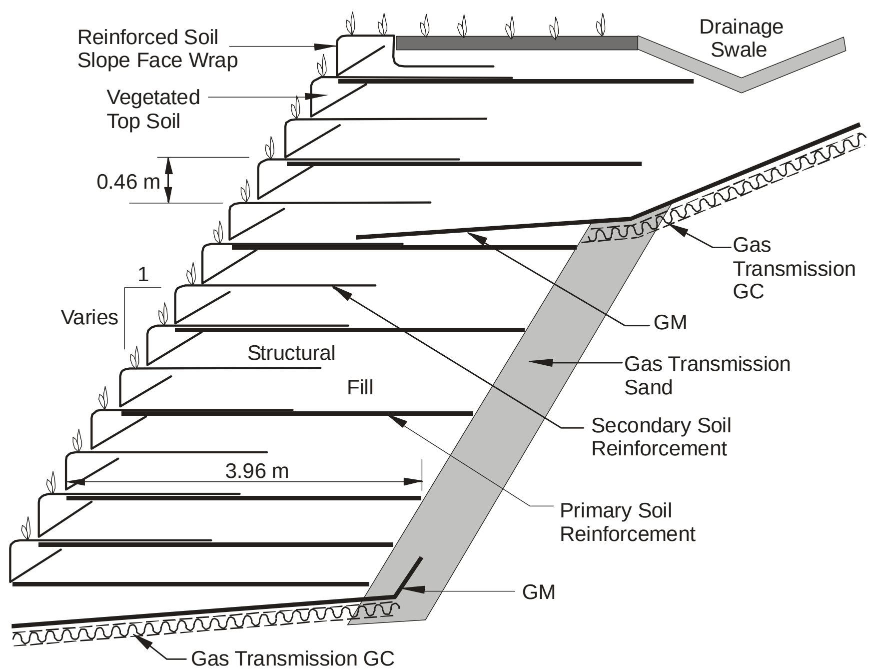


Figure 6-24. Detail of Reinforced Soil Slope Cover System Used on Steeper Slope Sections of a Hazardous Waste Landfill Cover System (modified from Cargill and Olen, 1997, 1998).

shingled GM within a geogrid-reinforced waste buttress (Figure 6-24). Approximately 4,300 linear m of slope buttress was constructed at heights up to 6.1 m. A cross section of this buttressed cover system is shown in Figure 6-25, and photographs of the system during construction and after completion are shown in Figures 6-26 and 6-27.

Slope stability is a major factor in the design of a cover system such as that described above. Three broad types of stability conditions must be considered for this type of closure. The first condition involves internal and interface stability of the components of the conventional portion of the cover system. The stability of these conventional components are evaluated using the procedures described in Sections 6.2 and 6.3 of this document.

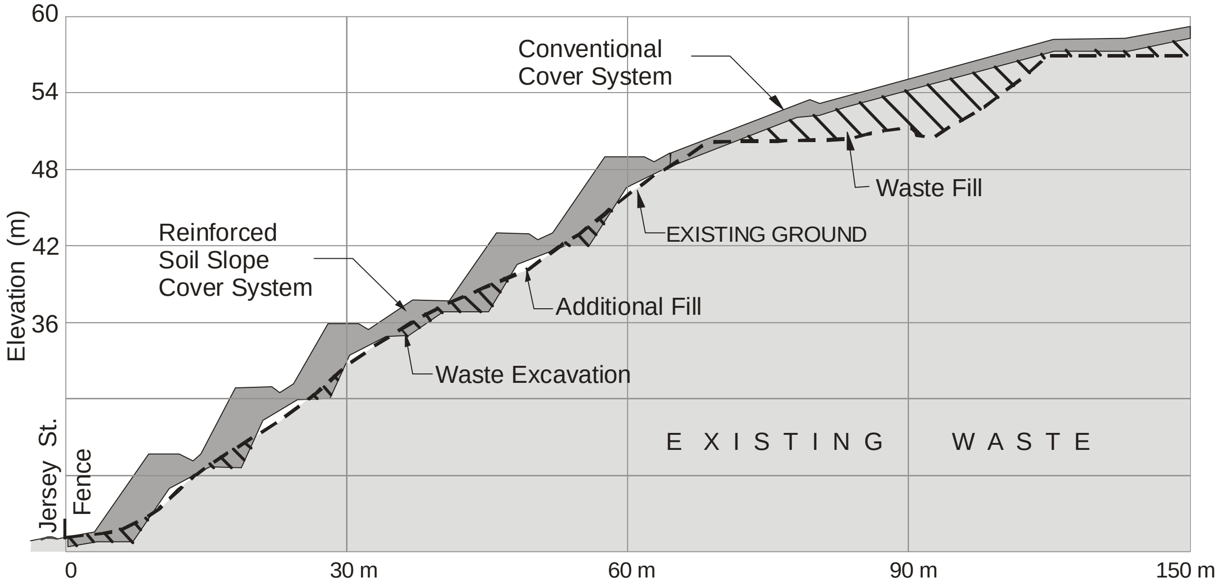


Figure 6-25. Reinforced Soil Slope Cover System on Steeper Slope Sections and Conventional Cover System on Shallower Slope Sections of a Hazardous Waste Landfill (modified from Cargill and Olen, 1997, 1998).

Figure 6-26. Construction of Reinforced Soil Slope Cover System on Steeper Slope Sections of a Hazardous Waste Landfill.

Figure 6-27. Constructed Reinforced Soil Slope Cover System for a Hazardous Waste Landfill.

The second condition involves the internal stability of the waste buttress. Many different types of earth retaining structures, including crib walls, mechanically stabilized earth (MSE) walls, and reinforced soil slopes, can be used in this application. If the structure is to be founded on firm ground, it can be fairly rigid, such as a precast concrete bin wall. However, if the structure is to be founded on waste, it must be flexible and able to undergo significant settlement and distortion while maintaining functionality. Geosynthetic-reinforced MSE walls and slopes with flexible facing elements meet these criteria. Cargill and Olen (1997, 1998) utilized geogrid-reinforced soil to form the buttress component of the cover system and a flexible facing (Figure 6-28). Procedures for design of earth retaining structures and evaluation of the internal stability of these structures can be found in a series of documents published by the U.S. Department of Transportation Federal Highway Administration (FHWA) (i.e., Holtz et al., 1995; Elias and Christopher, 1996; Sabatini et al.,1997) and in geosynthetic textbooks (Koerner, 1998).

The third stability condition that must be considered is the global stability of the buttress, waste mass, and landfill foundation. Global stability is typically evaluated using classical two-dimensional, LE slope stability analysis methods (e.g., Bishop, 1955; Spencer, 1967; Morgenstern and Price, 1965), as coded in the previously-mentioned commercially-available, PC-based computer programs (see Section 6.2.2.3). A critical aspect in the evaluation of global stability is the establishment of shear strength and unit weight parameters for soil and waste materials, and liquid heads (e.g., leachate heads in the waste and/or groundwater heads in the

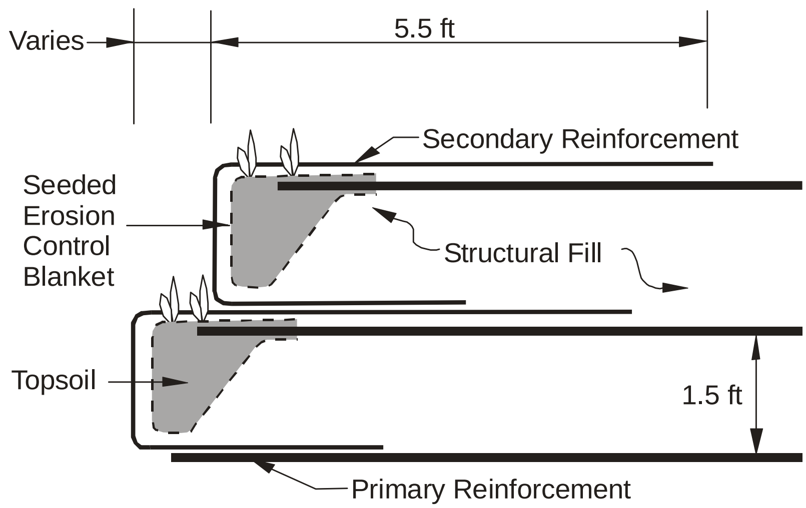


Figure 6-28. Flexible Facing Used with Reinforced Soil Slope Cover System for a Hazardous Waste Landfill (modified from Cargill and Olen, 1997; 1998).

foundation). Shear strengths for soil materials can be established using project-specific geotechnical site investigations and laboratory testing programs. For waste, shear strength and unit weight parameters can be established from information in the technical literature (e.g., Landva and Clark, 1990; Fassett et al., 1994; and Kavazanjian et al., 1995b) or through the use of project-specific field and laboratory test programs. For liner system materials and interfaces, shear strength parameters can be established using a laboratory testing program that includes consideration of the relevant testing procedures discussed for cover systems in Section 6.2.4. If the potential slip surface passes through a strain-softening soft soil foundation or liner system interface, the shear strength values selected for the waste, foundation, and/or liner system must be based on strain compatibility between the various materials along the potential slip surface. For example, the shear strain necessary to develop the peak shear strength of MSW may correspond to post-peak (e.g., residual) shear strength of a soft soil foundation material.

For another project, Graves et al. (1998) discussed the use of a pre-cast concrete crib wall 915 m long and up to 9 m high as part of an upgraded closure activity and flood protection for a 20-ha unlined sanitary landfill in Cuyahoga County, Ohio. In the years after landfill closure, an adjacent creek caused erosion and undermining of the landfill, creating localized near-vertical waste slopes (and overall waste slopes of about 2H:1V) and causing concerns about overall stability of the landfill. Flattening of the slope to allow installation of a conventional final cover system and achieve adequate slope stability factors of safety would have required relocation of approximately 700,000 m^3 of waste. Through use of a crib wall buttress at the toe of the landfill, the required waste excavation volume was reduced to 280,000 m^3. Design details and construction photographs for this project are illustrated in Figures 6-29 through 6-32.

The case study presented above utilized a pre-cast concrete crib wall as a gravity buttress to support waste slopes. Other types of wall systems could also be used for this application. Reference should be made to Sabatini et al. (1997) for an inventory of available wall types and typical wall unit costs. An emerging technology that holds promise for future use in landfill stabilization projects involves the use of geofiber reinforcement. Geofibers consist of relatively

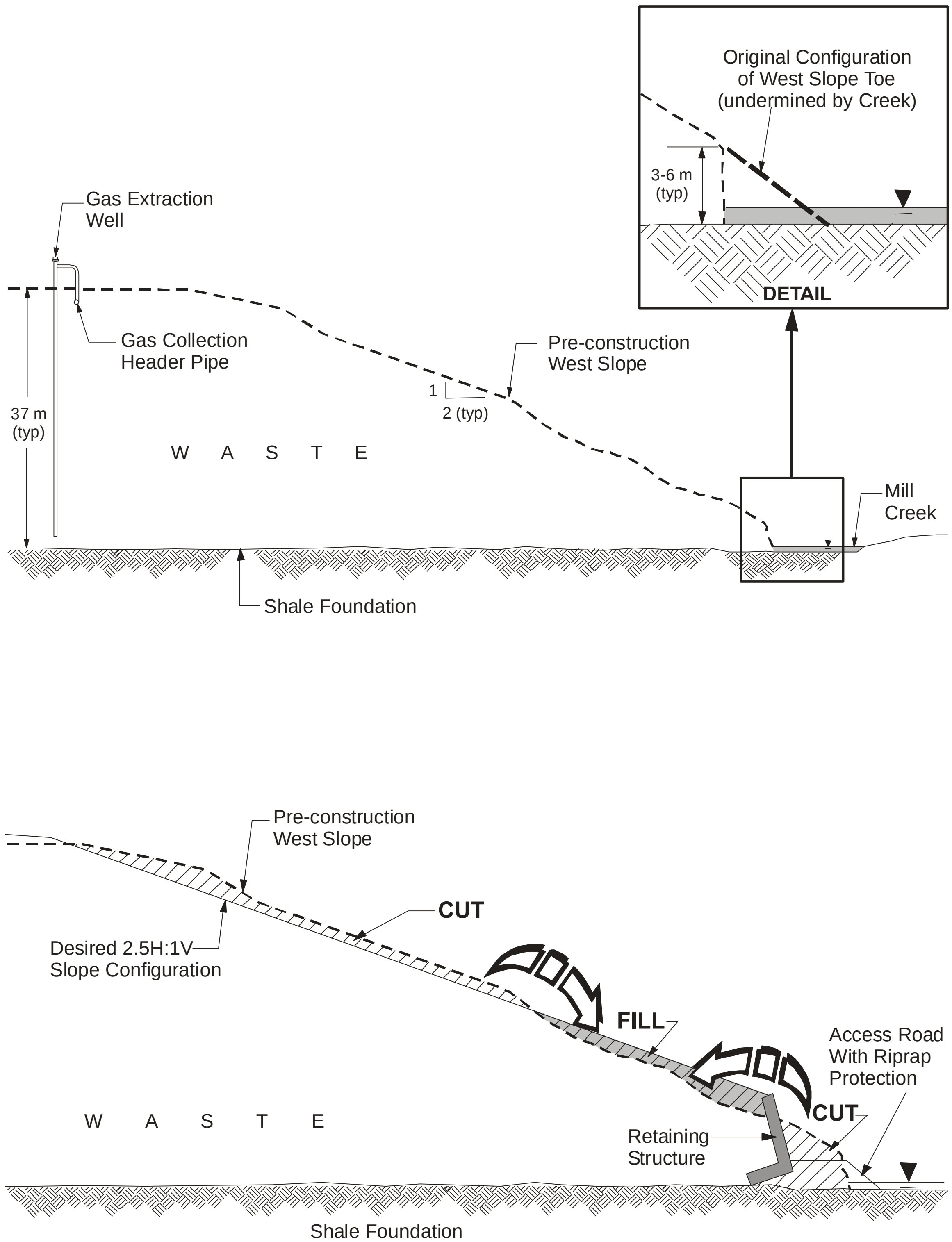


Figure 6-29. Waste Buttress Reduced Waste Excavation Volumes Required for an Upgraded Closure Activity at a Sanitary Landfill (modified from Graves et al., 1998).

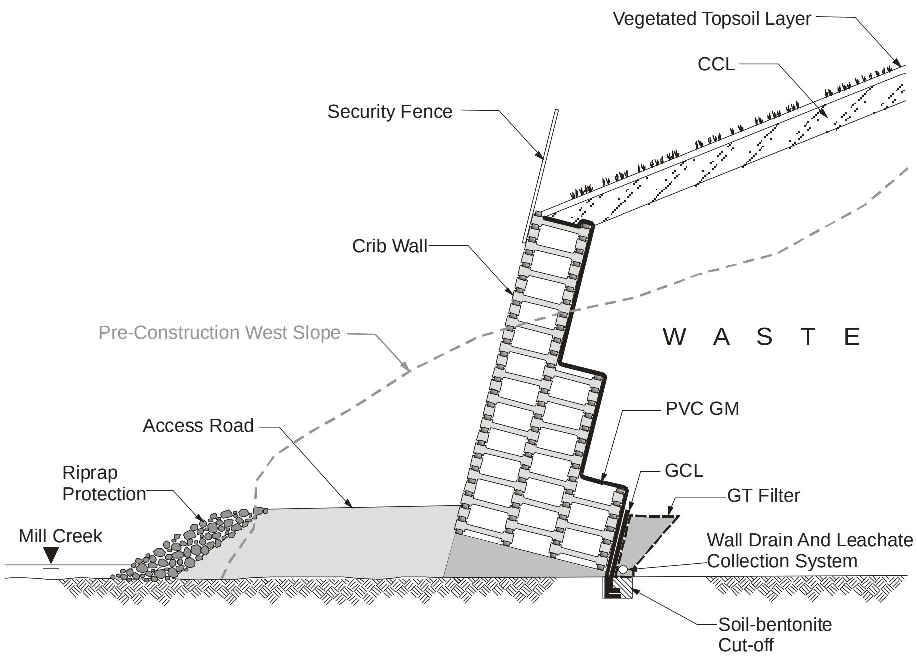


Figure 6-30. Pre-cast Concrete Crib Wall Used as Waste Buttress for a Sanitary Landfill (modified from Graves et al., 1998).

Figure 6-31. Construction of the Pre-cast Concrete Crib Wall Waste Buttress for a Sanitary Landfill.

Figure 6-32. Aerial View of Constructed Cover System with a Pre-cast Concrete Crib Wall Waste Buttress at a Sanitary Landfill.

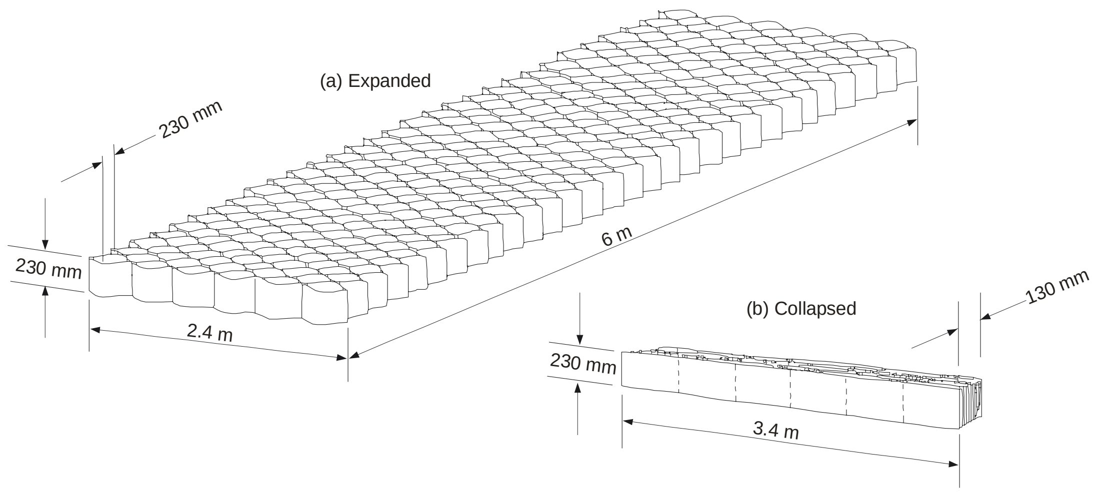
small (e.g., 25 to 50 mm in typical length) polymeric inclusions, distributed throughout the reinforced soil mass. There are a variety of techniques for mixing the fibers into a soil fill, including pneumatic application and mechanical mixing. An example of the use of geofibers for a slope stabilization project is given in Gregory and Chill (1998).

6.5.3 Steep Cover System Slopes

Cover system slopes somewhat steeper than those conventionally used can be achieved through the careful selection of cover system components. Materials that can be used to increase the inclination of cover system slopes include:

- textured GMs or GMs manufactured from polymers that generate higher interface shear strengths compared to smooth GMs manufactured from HDPE;
- geosynthetic reinforcement installed parallel to the landfill slope in the internal drainage layer or protection layer and anchored at the crest of the slope;
- geosynthetic drainage layers;
- geofiber reinforcement of cover system soil layers;
- geocell (e.g., Figure 6-33) and geosynthetic erosion control materials (described in Section 2.2.5.3); and

- topsoil surface/protection soil layer with adequate cohesion to resist erosion, yet with adequate characteristics to support vegetative cover.



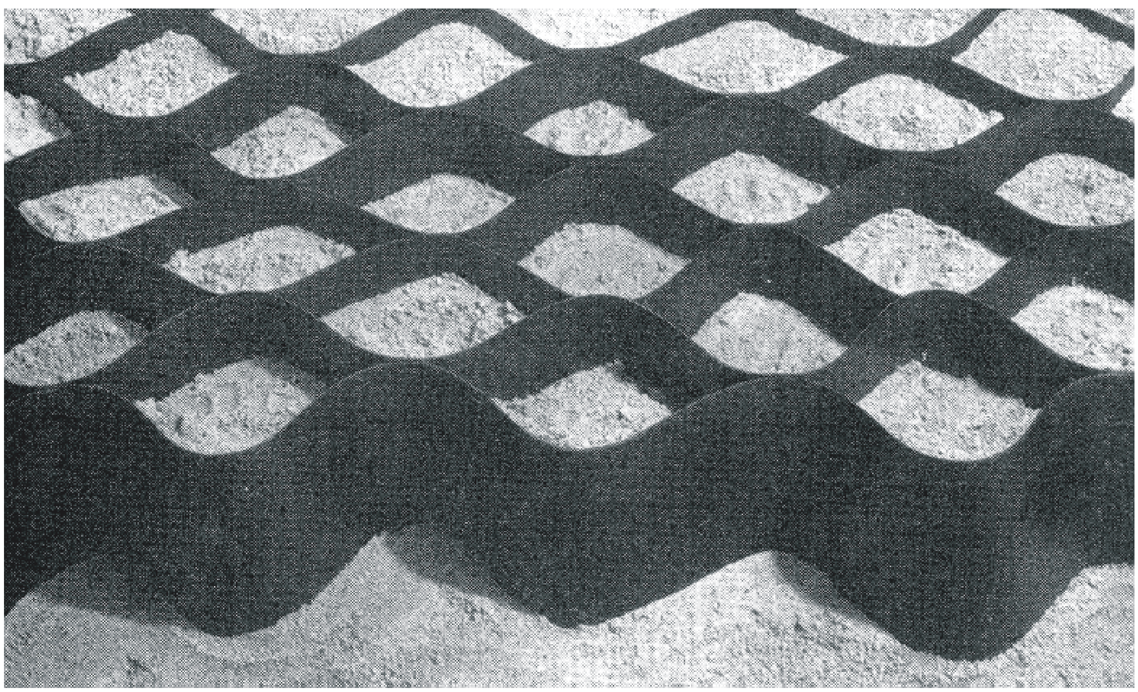


Figure 6-33. Geocells can be Used to Reinforce the Topsoil Surface/Protection Layer on Steep Cover System Slopes.

Using these materials, cover system slopes as steep as 2.5H:1V, and possibly slightly steeper can be constructed and maintained at a factor of safety at or near the target range discussed previously in this chapter. Even steeper slopes (e.g., 2H:1V, as demonstrated in the GCL test plot program described in Chapter 7 of this document) can be constructed, but factors of safety are likely to be lower than the target range given in herein. Moreover, long-term surface erosion problems should be anticipated when steep slopes are used. With steeper slopes, several other design aspects take on even greater importance than they might otherwise. For example, greater attention must be given to the selection of internal and interface shear strengths due to the greater potential for slope instability; thus, project-specific shear strength testing is essential. Also, seepage in a slope containing geosynthetic reinforcement can greatly reduce the effectiveness of the reinforcement. The stress-elongation characteristics of the various cover system components also become more important as slopes become steeper. Thus, the consideration of deformation compatibility of the cover system components is essential. It is possible, for example, that the elongation required to induce the design tensile force in a geosynthetic reinforcement layer is larger than the shear deformation needed to cause a GCL to exhibit large displacement rather than peak internal shear strengths characteristics. For this case, the design should be based on the large-displacement GCL shear strength and not the peak shear strength. Deformation compatibility can be evaluated as previously discussed in Section 6.2.3.

6.6 Soft Waste Materials

Another type of design issue sometimes encountered is the in-situ closure of impoundments or the capping of remediation source areas containing soft waste materials. These soft materials include high moisture content sludges, saturated process wastes, and saturated sediments or solid wastes. The common characteristic of these materials is that they have very low bearing capacity, which precludes using conventional techniques for cover system construction. These materials are also prone to large post-construction settlements that must be accounted for in design.

In general, if the undrained shear strength of the near surface waste is less than about 15 to 20 kPa, the waste may not be able to support a conventional cover system and the bearing capacity of the waste will be an important consideration in the design process. At undrained shear strengths below about 10 kPa, waste bearing capacity may become the controlling design criterion. Guidance on performing foundation bearing capacity analyses can be found in a number of textbooks, including Lambe and Whitman (1969) and Holtz and Kovacs (1981), and a number of more focused technical papers and reports, including Bonaparte and Christopher (1987), Humphrey and Rowe (1991), and Holtz et al. (1995). The three latter references provide information on the use of geosynthetics reinforcement to increase the bearing capacity of a soft foundation material (e.g., waste). The critical bearing capacity condition for a cover system over soft waste will typically occur during construction The critical condition is often associated with the timeframe during which the leading edge of construction is inducing relatively high shear stresses in the soft waste. Thus, construction equipment loads must be taken into account.

The engineering evaluation of a cover system over soft waste includes an assessment of overall bearing capacity, rotational stability, and lateral spreading. These potential failure mechanisms are illustrated in Figure 6-34. In addition to these stability evaluations, long-term settlement of

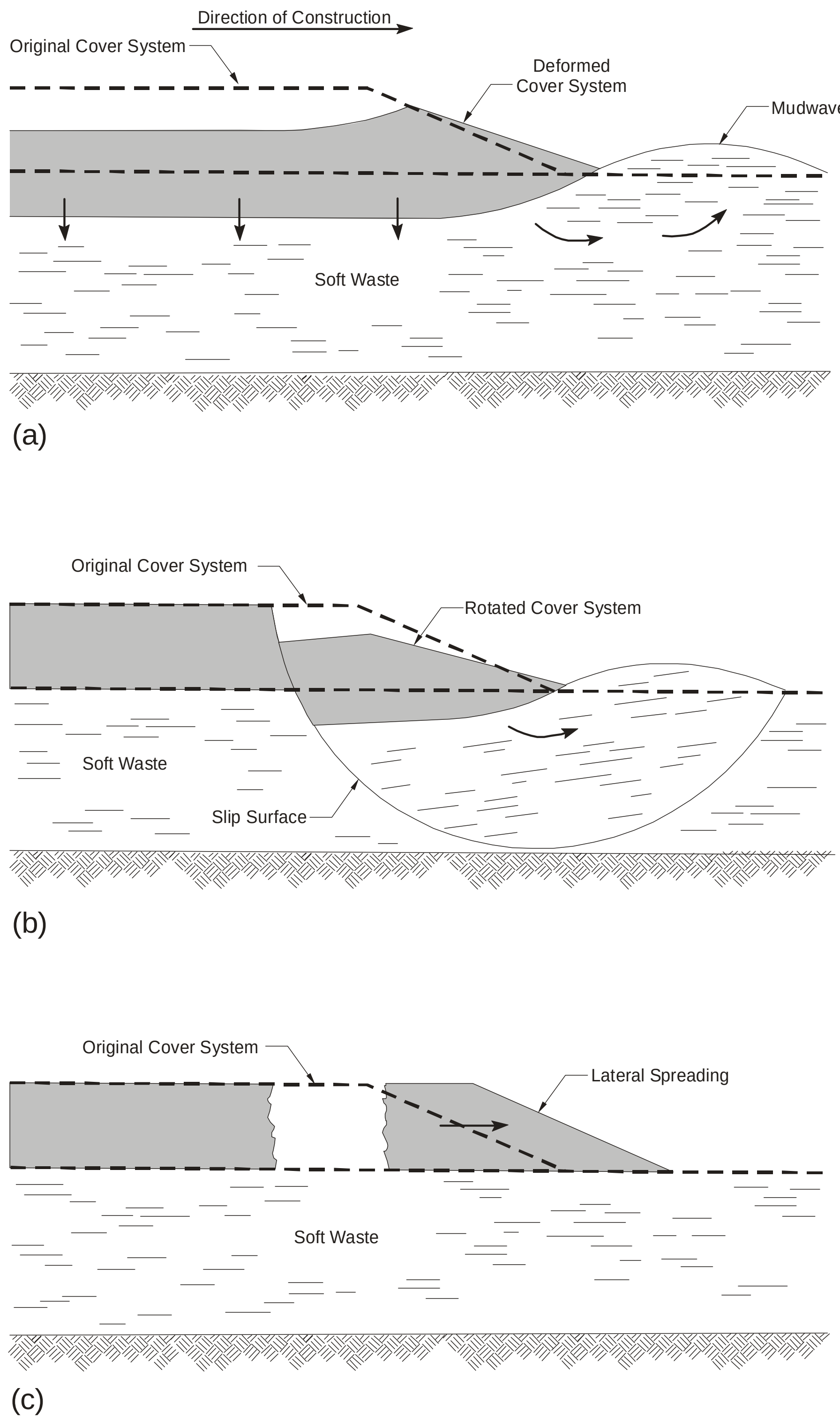


Figure 6-34. Three Potential Failure Mechanisms to be Considered for Cover Systems Constructed Over Soft Wastes: (a) Bearing Capacity Failure; (b) Rotational Failure; and (c) Lateral Spreading.

the cover system is estimated using classical geotechnical engineering calculation methods for soil or waste, as appropriate, as described previously in Section 6.4.

The design engineer has several available options when soft waste has inadequate shear strength to support the overlying cover system. These options include:

- strengthening of the waste by physical solidification;
- strengthening of the waste by preloading;
- strengthening of the waste by dewatering;
- strengthening of the waste by ET drying;
- supporting the cover system over the waste using reinforcement; and
- using lightweight cover system components.

Solidification is defined by EPA as a process in which materials are added to a waste to produce a solid to achieve one or more goals (Battelle, 1993). In the application being considered herein, the goals are to increase the waste's shear strength and decrease its compressibility. Agency guidance on waste solidification technology regulatory status, range of applicability and limitations, and use on a project-specific basis is given in the agency document (Battelle, 1993). Typical solidifying agents for this application include cement, cement kiln dust, lime, lime kiln dust, and fly ash. The final product may vary from a granular, soil-like material to a cohesive solid, depending on the properties of the solidifying agent and waste and the ratio of the solidifying agent to waste. One disadvantage of this approach is that the solidification process causes significant bulking (increase in volume) of the waste. In some cases, this increased volume can be used to advantage to build up the top elevations of what is initially a flat impoundment to achieve the sloping grades required for the cover system. As an example of a solidification project, Bodine and Trevino (1996) describe a case study where a cover system with a GM/CCL barrier was constructed over an oily sludge storage basin after the sludge was solidified in-situ using Portland cement. A 3.7-m diameter crane-mounted rotary auger was used to mix the cement with the sludge.

Strengthening of the waste by preloading involves spreading a layer of soil fill over the waste, then allowing the waste to consolidate under the weight of the fill. Additional layers of fill can be placed and the consolidation step repeated. Each consolidation step increases the undrained shear strength of the waste by about 20 to 25% of the applied vertical stress resulting from the weight of the fill. Disadvantages of this technique are that it is time consuming, due to the time required for waste consolidation, it involves multiple construction steps, and it requires significant amounts of fill. The time duration required for waste consolidation can be decreased through the use of wick drains. However, installation of wick drains into the soft waste may not be feasible due to access, settlement, and clogging issues. Vacuum consolidation can be considered as an alternative means to soil fill for applying a consolidation stress to the soft waste. However, as with wick drains, installation issues may render this technique unfeasible for some applications. Guidance on the design of preload systems can be found in Holtz and Kovacs (1981) and Ladd (1991).

Strengthening of the waste mass by dewatering involves the use of drainage trenches or other means to reduce the water level in the soft waste. A reduced water level has two benefits: (i) as the water level is pulled down, the effective stress in the waste, and hence the waste's strength, is increased; and (ii) evaporation from exposed waste above the water table tends to dry out the waste and increase its shear strength. In some cases, this surface drying can by itself lead to a stable crust upon which to build a cover system.

Strengthening of the waste by ET drying involves using high moisture uptake plant species to remove water from the waste by transpiration, thereby strengthening the near-surface waste. With ET drying, select plant species are planted or hydroseeded over the area to be strengthened. Because soft waste materials typically have a high moisture content, plants can readily access moisture in the waste matrix. As plants become established, two complementary benefits occur: (i) the waste surface dries and a strengthened crust develops; and (ii) the plant roots form a mat that reinforces and further strengthens the crust. Depending on the type of vegetation selected, the root mat may extend several inches (as in the case of grasses) or several feet (as in the case of certain tree species) into the waste. The success of the ET approach is dependent on the physical properties of the waste and the ability to keep the waste surface dewatered for the period of time required to establish healthy plant growth. The application of fertilizers or conditioning agents may be necessary to establish and sustain adequate plant growth. Simple greenhouse testing can be used to evaluate the potential effectiveness of ET drying. Pilot testing is recommended to quantify the amount of strengthening that can be achieved in a particular field application.

Geosynthetic reinforcement materials can be used to support cover systems over soft waste. This technique has found increasing use in recent years. With the approach, one or more layers of geogrid or GT reinforcement are placed over the soft waste, fill is placed on top of the reinforcement, and then the cover system is constructed on top of the fill. This technique has been used successfully with very soft waste materials (i.e., materials with undrained shear strengths below 10 kPa). Michalski et al. (1995) provide an example of the application of this technique to the closure of a 10-ha pickle liquor sludge lagoon in Pennsylvania. Closure of the lagoon required construction of a RCRA Subtitle C cover system, which was supported over the soft sludge by geogrid reinforcement. Guidance on the design of geosynthetic reinforcement systems can be found in the technical references cited previously in this subsection, and in Koerner (1998). While this technique has been shown to be effective, it is not without limitations. The technique does not reduce the inherent compressibility of the waste. Thus, when utilized with very soft materials, large total settlements of the cover system may occur. Also, when using this system over large areas, it may not be possible to build up the required final grades for the cover system. Even with a saw-tooth final grading plan, fill thicknesses can be significant to achieve cover system grades of at least several percent. The problem is exacerbated by the fact that the largest waste settlements will occur at the location where the fill thickness is greatest, which will tend to reduce cover system grades as the waste settles. The effects of settlements on cover system grades need to be considered in the design of geosynthetic reinforcement systems and may also be important for some, or all, of the other technologies described above.

The final option considered for construction of cover systems over soft waste materials is the use of lightweight cover system components. Examples of lightweight materials include, from the bottom of the cover system upward:

- geosynthetic reinforcement as a replacement for structural fill;
- lightweight structural fill as a replacement for structural fill; potential lightweight fill materials include slag, expanded clay and shale, vermiculite, tire chips, and geofoam; expanded clay and shale materials, manufactured by heating clay or shale in a kiln, are discussed by Bowders et al. (1997b); the geofoam class of geosynthetics is discussed in Section 3.7.1 of this guidance document;
- GC drainage layer or a thick needlepunched nonwoven GT as a replacement for a granular gas transmission layer;
- GCL as a replacement for a CCL; and
- GC drainage layer as a replacement for a granular drainage layer.

An example of a lightweight cover system designed and constructed as part of a CERCLA remedial action at a soft waste and soil site near Beaumont, Texas, is illustrated in Figure 6-35. Each of the options for constructing cover systems over soft waste materials have advantages and disadvantages that must be carefully evaluated for each project application. For most applications, several of the options will be used in combination to achieve the project design criteria.

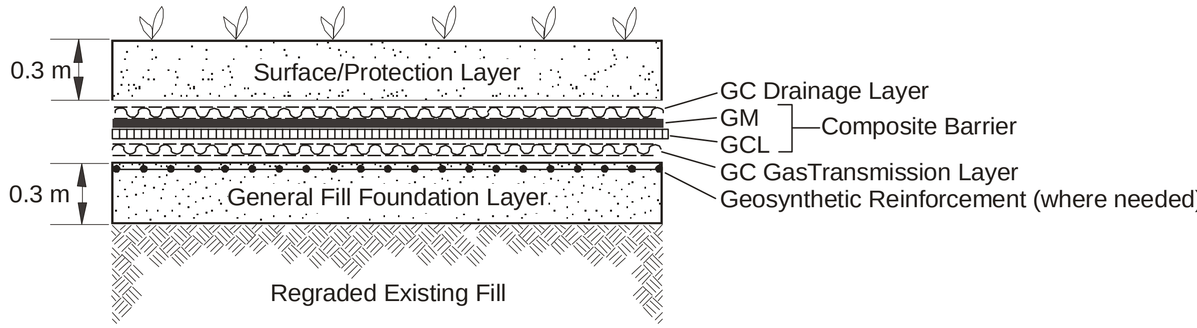


Figure 6-35. Example of a Lightweight Cover System Constructed Over Soft Waste and Soil at CERCLA site near Beaumont, Texas.