CHAPTER 25

Geosynthetics

25.1 GENERAL

Geosynthetics have been to geotechnical engineering what computers have been to humankind in general: a revolution. The use of these planar synthetic materials in soils to reinforce, to drain, and to separate has grown remarkably over the past 50 years to the point where it is a huge industry today. According to ASTM D4439, a *geosynthetic* is a planar product manufactured from polymeric material (plastics) to be used with soil, rock, earth, or other geotechnical engineering-related materials as an integral part of a human-made project, structure, or system. There are many types of geosynthetics, including geotextiles, geomembranes, geogrids, geosynthetic clay liners, geofoam, geonets, geocells, geobags, and geocomposites. Geotextiles and geomembranes are the two largest groups of geosynthetics. In 2013, the cost of geosynthetics was between \$1 and \$7 m^2. The book by Koerner (2012) is an excellent reference on geosynthetics.

25.2 TYPES OF GEOSYNTHETICS

Geotextiles (Figure 25.1) are textiles made of synthetic fibers. The fibers are either woven together or tied together (nonwoven). Weaving consists of standard interlacing with textile machinery. In nonwoven fabrics, the fibers are tied together by heating, gluing, or needle-punching. In needle-punching, short needles with barbs are punched through the fabric to provide a mechanical interlocking. Geotextiles are flexible and porous to liquid flow. They are used mainly for separation, reinforcement, filtration, and drainage.

Geomembranes (Figure 25.1) are relatively thin, impervious sheets of plastic material. They are made by first preparing the polymer resin and its additives. The actual forming of the membrane takes place by extrusion through two parallel plates or rollers. The resulting sheet is between 1 and 3 mm thick and can be smooth or roughened. Geomembranes are used mostly as nearly impervious barriers to contain liquids or vapors.

Geogrids (Figure 25.1) are plastic grids that have a very open configuration; they have large holes between ribs. They are formed by bonding rods together, by weaving and then coating, or by stretching. Their main use is reinforcement.

Geosynthetic clay liners or GCLs (Figure 25.1) are made of a thin layer of bentonite clay sandwiched between two layers of geotextiles or geomembranes. GCLs are manufactured by feeding the bentonite on top of a conveyor-belt-style geosynthetic and covering it after the feed point by a top geosynthetic. The two geosynthetic layers are kept together by needle punching, stitching, or gluing. The bentonite will expand dramatically when wetted. GCLs are about 4 to 6 mm thick when the bentonite is hydrated at water contents of 10 to 35%. They are used mostly as nearly impervious barriers to contain liquids or vapors.

Geofoams (Figure 25.1) are extremely light blocks made of polymer bubbles. They are fabricated by thermal expansion and stabilization of polystyrene bubbles. The density of the blocks is about 2% of the density of soils, but 3 to 4 times more expensive per unit volume. They are stacked together to form lightweight fills, and are used as compressible layers behind retaining walls, as vibration dampers for seismic protection, and as thermal insulation in foundations.

Geonets (Figure 25.1), like geogrids, are open netting geosynthetics made of plastic. They are different from geogrids that they are thicker; they are sometimes called *spacers* as they provide space for fluid to flow within the structure. Also, the openings are more like diamonds than the squares of geogrid openings. They are used primarily for drainage purposes.

Geocells are a form of geogrid in the sense that they have a very open configuration, but their purpose is to reinforce by confining the soil within the cells. The cells may be 1 m × 1 m in plan view and 1 to 2 m high. The soil is placed within the cells, which provide lateral confinement and thereby significantly increase the bearing capacity of the soil layer.

Geobags are literally bags made of geosynthetic material; they are usually filled with sand and used for erosion protection in lieu of rip rap. Their size is in the range of rip rap, and can be as large as 5 m^3.

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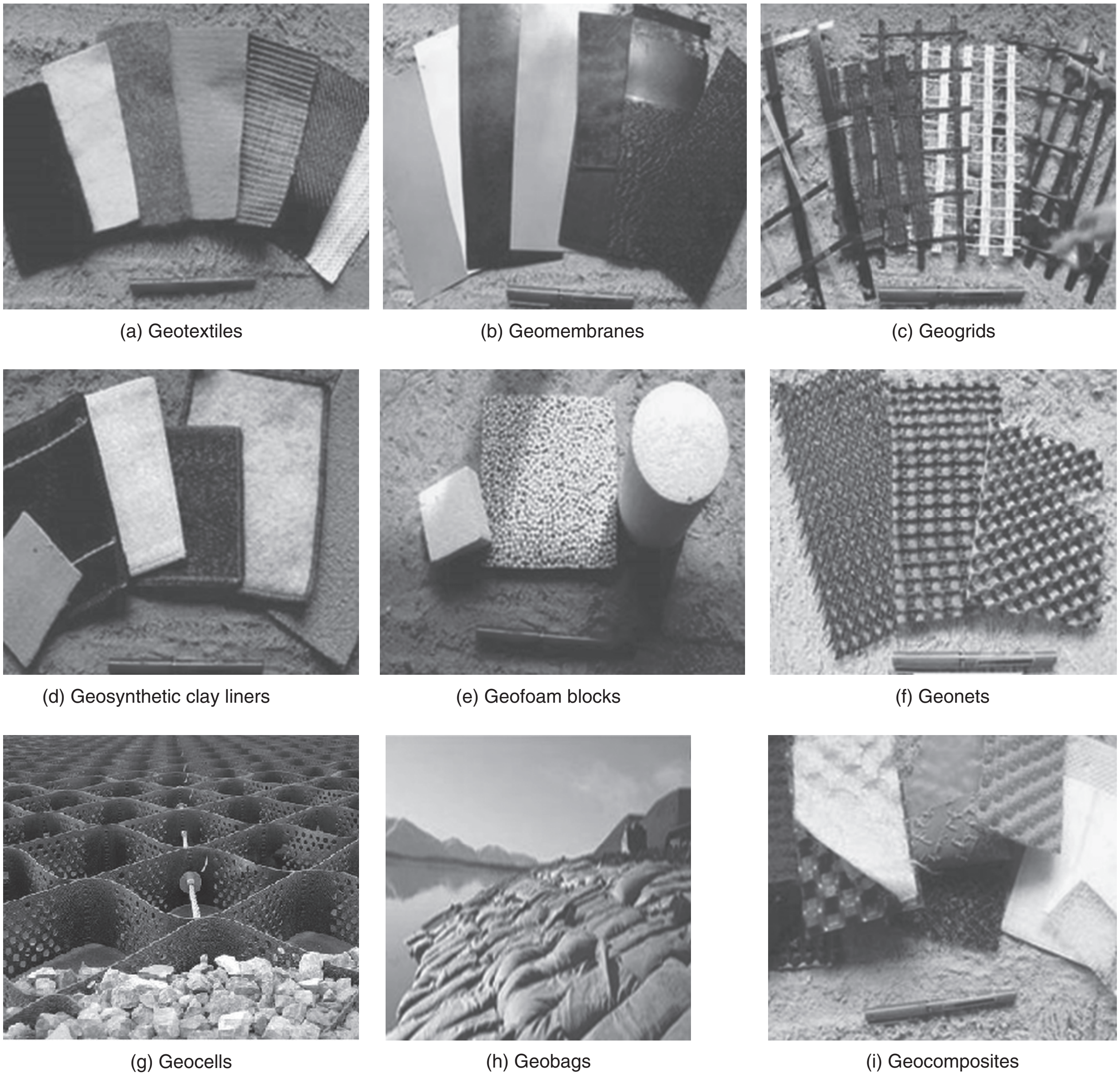


Figure 25.1 Examples of geosynthetics. (Photographs compliments of the Geosynthetic Institute.)

Geocomposites are combinations of the previous geosynthetics that are intended to maximize the usefulness of a geosynthetic layer. They are used as filter layers, for example.

Geosynthetics are useful in a number of geotechnical applications, as shown in Table 25.1

25.3 PROPERTIES OF GEOSYNTHETICS

The parameters used to characterize geosynthetics are much more numerous than and often different from those used for soils. The reason is that the material and the applications are quite different and more versatile than those associated with soils alone. Also, the field of geosynthetics is quite a bit younger than the field of geotechnical engineering. Although very significant progress has been made, some of the properties' definitions, the tests used to determine their value, and the design guidelines are still evolving.

25.3.1 Properties of Geotextiles

Physical Properties

The unit weight of typical plastics varies from 9 to 13 kN/m^3. The unit weight of dry, clean geotextiles is between 3 and 7 kN/m^3—but that is not the way it is typically given. Instead, it is quoted as mass per unit area (ASTM D5261) with values between 150 and 750 g/m^2 (Koerner 2012). The

Table 25.1 Applications for Some Geosynthetics

Geotextiles	Geogrids	Geomembranes	Geosynthetic Clay Liners	Geofoam
• Separation • Roadway reinforcement • Soil reinforcement • Filtration • Drainage	• Reinforcement • Roads • Slopes • Walls • Foundations	• Liners • Ponds • Canals • Landfills, dry • Landfills, wet • Landfill covers	• Liners	• Lightweight fill • Compressible inclusions • Thermal insulations • Drainage

thickness of commonly used geotextiles is between 0.5 to 4 mm (ASTM D5199).

Mechanical Properties

Stiffness is usually defined as the ratio between the force applied and the resulting displacement, as in stiffness of a spring. The stiffness of a geotextile is defined in a very different way; it is obtained from a laboratory test (ASTM D1388) in which a 25 mm wide strip of geotextiles is gradually pushed over the edge of the crest of a slope under controlled conditions (Figure 25.2). The slope is 41.5° with the horizontal and when the strip touches the slope, the length L of the overhanging strip is recorded. The stiffness of the geotextile is defined as:

$$G = M\left(\frac{L}{2}\right)^3 \tag{25.1}$$

where G is the flexural stiffness (g.m), M is the mass per unit area (g/m^2), and L is the overhang length (m). The G values for geotextiles are in the range of 0.01 to 1 g.m.

The average modulus of deformation of geotextile under tension stresses varies widely. It can be 60 MPa for some nonwoven, needle-punched geotextiles all the way to 400 MPa for some woven monofilament geotextiles (Koerner 2012). Because the evolution of the thickness during the test is not certain, this modulus is not commonly quoted for these products. Instead, it is more commonly presented as the ratio of the force per unit length of fabric over the normal strain

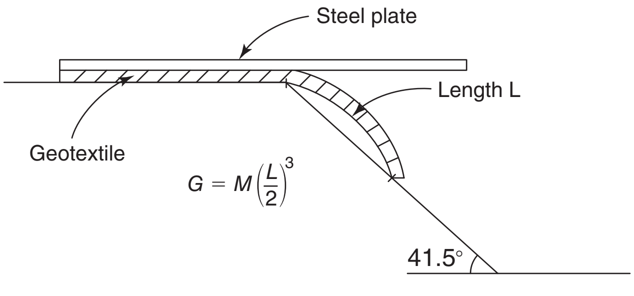


Figure 25.2 Flexure stiffness test for geotextile.

generated. Numbers in the range of 30 kN/m to 150 kN/m are common.

The average tensile strength of geotextiles (ASTM D4632) is in the range of 50 to 100 MPa; again, however, that is not the way it is typically cited. Instead, the average tensile strength is quoted as the force S_t per unit length of fabric that creates rupture; average numbers are in the range of 25 to 60 kN/m. One of the problems is that the thickness varies during elongation of the geotextiles. Another problem is that the strain to failure is much larger than in soils, with values around 25% for some woven fabrics and up to 70% for some nonwoven fabrics. As a result, the tensile strength is usually quoted together with a value of the strain at failure. The tensile strength of the seams (ASTM D4884) is typically 50 to 75% of the tensile strength of the intact fabric. The compressibility of geotextiles is generally not a concern except when they are used to convey water or other liquids in the in-plane direction. In this case it is important to make sure, by testing, that the small conveyance tubes within the geotextiles will not collapse under the in situ compression.

The puncture strength is important and may be quoted as an impact puncture strength or a static puncture strength. The impact puncture strength is tested by dynamically puncturing the fabric with a pendulum test (energy to puncture) (ASTM D256) or a drop cone (penetration distance). The impact puncture strength of a geotextile is quoted as the energy that leads to puncture. Common values of geotextile impact puncture strength vary from 25 Joules to 300 Joules. It is named after the English physicist James Prescott Joule who contributed in the middle to late nineteenth century. The static puncture strength (ASTM D6241) is determined by slowly pushing a 50 mm diameter beveled plunger into the fabric and recording the puncture failure load P. The following empirical relationship between the puncture load P (kN) and the tensile strength S_t (kN/m) has been proposed (Cazzuffi and Venezia 1986):

$$P = S_t \pi d \tag{25.2}$$

where d is the diameter of the punching plunger.

The interface shear strength between a geotextile and a soil can be very important in design and should be measured using

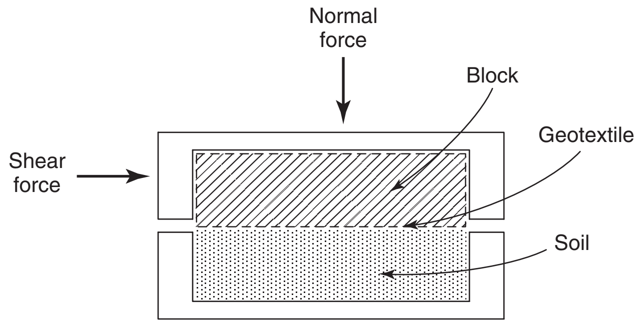


Figure 25.3 Direct shear test for soil-geotextiles interface shear strength.

site-specific materials. The accepted test (ASTM D5321) is a variant of the soil direct shear test in which the top part of the soil is replaced by a geotextile-covered block (Figure 25.3). The interface shear strength is typically in the range of 75% to 100% of the soil shear strength.

Creep or deformation under constant stress is also important for geotextiles, and creep tests are necessary for long-term applications under load (ASTM D5262). The general model used for soils can be extended to geotextiles:

$$\frac{\varepsilon_1}{\varepsilon_2} = \left(\frac{t_1}{t_2}\right)^n \tag{25.3}$$

where ε_1 and ε_2 are the strains reached in a time t_1 and t_2 respectively and n is the viscous exponent.

Table 25.2 gives estimates of the n values for several polymers based on the data from den Hoedt (1986). Note that the time-temperature equivalency, which has been used for a long time in the asphalt field, is also used for geotextiles for speeding up time in creep tests.

Hydraulic Properties

The percent open area of a geotextile can be measured by shining a light through the geotextile onto a poster board and measuring the illuminated area on the poster. Monofilament woven geotextiles have percent open areas in the range of 6 to 12%. The apparent opening size (AOS) of a geotextile is obtained through a test (ASTM D4751) in which glass beads of uniform diameter are placed on top of the geotextile and wet-sieved through the geotextile. The diameter for which 95% of the beads by weight are retained on the geotextile is the AOS, designated as O_{95}. Typical values range from 0.01 mm to 0.5 mm.

A distinction is made between the cross-plane hydraulic conductivity k_{cp} and the in-plane hydraulic conductivity k_{ip}. *Cross-plane* refers to the case where the liquid flows in a direction normal to the plane of the geotextile, this is called *filtration*. *In-plane* refers to the case where the liquid flows parallel to the plane or within the geotextile; this is called *drainage*. Typical values of the hydraulic conductivity of geotextiles (ASTM D4491) range from 10^{-3} m/s to 10^{-5} m/s for the stress-free product. This is in the range of gravel to coarse sand. Because the thickness of the geotextile can vary due to the in situ stress condition, the permittivity ψ (s^{-1}) is used for the cross-plane flow and the transmissivity Θ (m^2/s) is used for in-plane flow instead of the hydraulic conductivity. They are defined as follows:

$$\text{Permittivity} \quad \Psi = \frac{k_{cp}}{t} = \frac{q}{iAt} = \frac{q}{\frac{\Delta h}{t}At} = \frac{q}{\Delta h\ A} \tag{25.4}$$

$$\text{Transmissivity} \quad \Theta = k_{ip}t = \frac{q}{iA}t = \frac{q}{iWt}t = \frac{q}{iW} \tag{25.5}$$

where q is the flow, A is the flow area perpendicular to the flow, i is the hydraulic gradient, Δh is the loss of total head over the flow distance, t is the thickness, and W is the width of the geotextile involved in the flow.

The hydraulic conductivity, the permittivity, and the transmissivity should be tested under the compressive stress likely to be experienced in the field (ASTM D5493). While it is always important to do this testing, the values under load do not appear to be very different from the values under no load. As mentioned earlier, the cross-plane hydraulic conductivity k of geotextiles for water flow typically ranges from 10^{-3} to 10^{-5} m/s. If the fluid is not water, then the hydraulic conductivity and the permittivity should be corrected as follows:

$$\frac{k_f}{k_w} = \frac{\Psi_f}{\Psi_w} = \frac{\Theta_f}{\Theta_w} = \frac{\rho_f}{\rho_w}\frac{\mu_w}{\mu_f} \tag{25.6}$$

Table 25.2 Viscous Exponent n for Several Polymers

Geotextile Polymer	Viscous Exponent n Value at 20% of Ultimate Strength	Viscous Exponent n Value at 60% of Ultimate Strength
Polyester (PET)	0 to 0.01	0 to 0.01
Polyamide (PA) (nylon)	0.02	—
Polypropylene (PE)	0.07	0.19 to 0.2
Polyethylene (PP)	0.08 to 0.14	0.12 to 0.19

(From data by den Hoedt 1986.)

where k_f and k_w are the hydraulic conductivity for the fluid and for water respectively, ψ_f and ψ_w are the permittivity for the fluid and for water respectively, Θ_f and Θ_w are the transmissivity for the fluid and for water respectively, ρ_f and ρ_w are the density of the fluid and of water respectively, and μ_f and μ_w are the viscosity of the fluid and of water respectively.

Other properties of geotextiles include resistance to abrasion from repeated action of gravel impacting the geotextile, soil retention and clogging (as in filters and silt fences), sunlight degradation from long-term exposure to ultraviolet rays, and degradation due to temperature, oxidation, chemical action, and biological action. Designers use reduction factors to take into account these factors, all of which affect the long-term strength and function of the geotextile.

25.3.2 Properties of Geomembranes

Physical Properties

Remember that geomembranes are solid and have no intended holes in them. The unit weight of dry, clean geomembranes is between 8.5 and 15 kN/m^3 depending on the polymer used to make them (ASTM D792). The high-density polyethylene (HDPE) membranes have a unit weight of about 9.2 kN/m^3. However, the density of geomembranes is usually given in terms of mass per unit area (ASTM D1910), in g/m^2. The thickness of commonly used smooth geomembranes is between 0.5 to 3 mm (ASTM D5199). The height of asperities for textured geomembranes can be 0.25 to 0.75 mm; these asperities do increase the interface shear strength.

Mechanical Properties

The stress-strain curve of geomembrane specimens tested in tension exhibits the same two types of shapes as soils: some have a peak strength followed by a residual strength, like overconsolidated soils; and some have a gradual increase in strength with no strain softening, like normally consolidated soils. One major difference is that the range of strains is drastically larger for geomembranes. Strains to failure for soils are in the 2 to 10% range, whereas strains to failure for geomembranes are in the 20 to 100% range. *Failure* refers to no more increase in resistance, but rupture may take as much as 1000% strain. The initial tangent modulus of deformation in tension can vary from 30 MPa for polyvinyl chloride (PVC) to 250 MPa for HDPE geomembranes. The peak tensile strength shows values in the range of 10 to 50 MPa depending on the type of polymer. The high peak strengths tend to lead to lower residual strengths, which can be 50 to 70% of the peak value. The tensile strength of the seams can be less than that of the parent material and should be tested. Seams are manufactured by overlapping two sheets and fusing them together, or by pinching two sheets and fusing them together. The tensile test for the overlapped seam is a shear test and the tensile test for the pinched seam is a peel test (e.g., ASTM D6392 and D882). Peeling tends to offer less resistance than shearing.

The interface shear strength between a geomembrane and the soil is important in many designs, especially for slope stability. It is measured with the same test as for geotextiles (Figure 25.3). A major difference exists between smooth membranes and textured membranes. Koerner (2012) quotes friction angles of 17^o and 18^o for fine sand and smooth HDPE, and 22^o to 30^o for fine sand and textured HDPE. Sometimes geomembranes are placed against geotextiles. Again Koerner (2012) quotes friction angles of 6^o to 11^o for geotextiles and smooth HDPE, and 19^o to 32^o for geotextiles and textured HDPE. The worst combination seems to be a woven monofilament on top of a smooth HDPE (6^o); the best combination appears to be a nonwoven, needle-punched geotextile on top of a textured HDPE (32^o).

The puncture strength of a geomembrane is important and can be quoted as impact puncture strength or static puncture strength. The impact puncture strength relates to the ability of the geomembrane to resist shocks from falling objects. It is tested by dropping a heavy object on the membrane (ASTM D3029) or through a pendulum test (energy to puncture) (ASTM D1822). The static puncture strength is related to the ability of the geomembrane to resists puncturing when the membrane is in contact with large aggregates under high pressures. Two alternatives exist to test static puncture strength: a small-scale test and a large-scale test. In the small-scale test (ASTM D4833), an 8 mm diameter beveled-edge piston is pushed through a geomembrane stretched over a 45 mm diameter empty mold. The puncture failure load P is expected to be in the range of 50 to 500 N for thin, nonreinforced geomembranes and 200 to 2000 N for reinforced geomembranes (Koerner 2012). The large-scale test consists of pressing the geomembrane against a bed of cones simulating aggregates (ASTM D5514).

Hydraulic Properties

Geomembranes are often used to prevent a fluid from passing from one side of the membrane to the other: this is called *separation*. Such geomembranes are essentially impervious. Nevertheless, nothing is truly and completely impervious. When the minute amount of fluid passing through the geomembrane must be known, the hydraulic properties of the geomembrane become important. Conventional hydraulic conductivity of geomembranes is in the range of 10^{-13} to 10^{-15} m/s, but, in the language of geosynthetics, terms such as *water-vapor transmission* and *permeance* (which actually designate other parameters) are often used. If solvents are to be retained instead of water, the hydraulic conductivity can increase drastically, by a factor of 100 or even 1000, depending on the nature of the chemical.

As in the case of geotextiles, other properties of geomembranes include sunlight degradation with long-term exposure to ultraviolet rays, radioactive degradation (limit of 10^6 to 10^7 rads), and degradation due to hot and cold temperatures, chemical action, and biological action. Designers use reduction factors to take into account these factors, all of which affect the long-term strength and function of the geomembrane.

25.3.3 Properties of Geogrids

Physical Properties

Geogrids are open-grid geosynthetics (Figure 25.1). The distance between ribs is in the range of 10 to 100 mm. They can be unidirectional (applied stress is in one direction) or bidirectional (direction of applied stress can be random). The percent open area (POA) is measured by shining a light through the geogrids onto a poster board where the illuminated area is measured. Most geogrids have POAs in the range of 40 to 95%. The mass per unit area varies quite a bit, from 200 to 1000 g/m^2.

Mechanical Properties

The flexural stiffness G was defined in Eq. 25.1. Stiff geogrids have G values above 10 g.m, whereas flexible geogrids have G values of less than 10 g.m. From the point of view of tensile strength, several strengths can be identified: the rib strength, the junction strength, and the wide width strength. The *rib strength* refers to the strength of the individual longitudinal elements. The *junction strength* refers to the strength of the connection between the longitudinal and transversal elements. The *wide width strength* F_{ug} is the strength of the geogrid at the field scale where all element strengths are integrated (ASTM D6637). The wide width tensile strength F_{ug} of many geogrids is in the range of 20 to 140 kN/m (force per unit length of fabric) reached at a wide range of strains from 5% to 30%. The tensile modulus of a geogrid is defined as the tensile load applied per unit length of geogrid (kN/m) divided by the corresponding strain of the geogrids. Numbers in the range 125 to 255 kN/m have been measured at small strains (1% to 5%) for stiff geogrids (Austin et al. 1993).

The interface shear strength between soil and geogrid can be measured in a direct shear test, as shown in Figure 25.3. In this test the geogrid is glued to a solid block that fits in the upper part of the direct shear box, which is 0.3 m by 0.3 m minimum (ASTM D5321). Results of such tests indicate that the ratio of the soil-geogrid shear strength over the soil-soil shear strength (called *efficiency*) is close to 1 in bidirectional loading and somewhat less (e.g., 75%) in unidirectional loading (Koerner 2012). Furthermore, Sarsby (1985) showed that if the geogrid aperture (distance between two ribs) is at least equal to 3.5 times the mean particle size d_{50} of the soil, it is likely that the efficiency of the geogrid will be 1, meaning that the shear strength of the soil-geogrid interface will be equal to the soil-soil shear strength.

The pull-out strength of geogrids embedded in soils is very important, as geogrids are most often used as reinforcement. There are two pull-out strengths: you can break the geogrid (F_{ug} given by the manufacturer), or you can break the soil (F_{us}). *Breaking the soil* means failure in shear at the interface between the soil and the geogrid. That ultimate pull-out load can be written as:

$$F_{us} = 2L_e K \sigma'_v \tan\varphi' \tag{25.7}$$

where F_{us} is the ultimate pull-out load per unit width of geogrid, L_e is the embedment length of the geogrid, K is a pull-out coefficient specific to the soil and geogrid involved, σ'_v is the vertical effective stress at the depth of the geogrids, and φ' is the soil effective stress friction angle.

The value of K should be obtained by testing. Pull-out tests can be carried out in the field on full-scale structures, or in the laboratory on large containers simulating the field conditions (Figure 25.4).

Note that two phenomena contribute to the ultimate load F_{us} and therefore the coefficient K: friction between the soil and the geogrid on one hand and penetration of the geogrid transversal elements into the soil. The first one can be calculated from friction laws while the second one calls for bearing capacity estimates. Consider a 2.5 mm thick geogrid made of 7 m long, 5 mm wide longitudinal ribs with a spacing of 100 mm and 3 mm wide transverse elements with a spacing of 200 mm. It is located 2 m below the ground surface of a soil weighing 20 kN/m^3. The interface shear strength between the geogrid and the soil is 18 kPa and the bearing capacity of the transverse element is 900 kPa, then the pull out load per unit width of geogrid is given by:

$$\begin{aligned} F_{us} &= \text{friction longitudinal} + \text{friction transverse} \\ &\quad + \text{bearing capacity transverse} \end{aligned} \tag{25.8}$$

$$\begin{aligned} F_{us} &= 2 \times 0.005 \times 7 \times 18 \times \frac{1}{0.1} + 2 \times 0.003 \\ &\quad \times \left(1 - \frac{1}{0.1} \times 0.005\right) \times 18 \times \frac{7}{0.2} + 0.0025 \\ &\quad \times \left(1 - \frac{1}{0.1} \times 0.005\right) \times 900 \times \frac{7}{0.2} \end{aligned} \tag{25.9}$$

$$F_{us} = 12.6 + 3.59 + 74.81 = 91\ \text{kN/m} \tag{25.10}$$

As can be seen from this example, the bearing capacity on the ribs is the major contribution, but the friction on the longitudinal ribs is not negligible. Recall that it takes a lot less displacement to mobilize the friction than the bearing

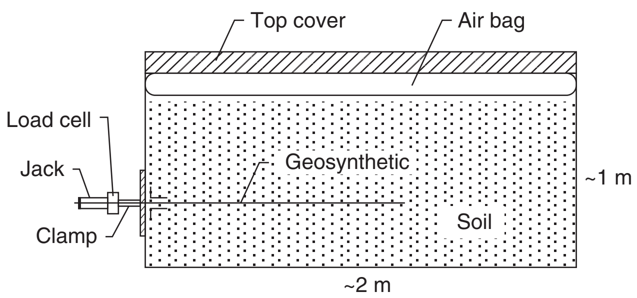


Figure 25.4 Laboratory container for pull-out tests on geogrid.

capacity, so the friction will be mobilized first and the bearing capacity last. Now we can calculate the global friction factor K in Eq. 25.7 knowing that the friction angle is 30° and the vertical effective stress at the depth of the geogrid is 40 kPa:

$$K = \frac{F_{us}}{2L_e\sigma_v' \tan\varphi'} = \frac{91}{2\times 7\times 40\times \tan 30} = 0.281 \tag{25.11}$$

The geogrid covers an area A_g per meter of geogrid width, which is a very small fraction of the total area A_t:

$$A_g = 0.005\times 7\times \frac{1}{0.1} + 0.003\times\left(1-\frac{1}{0.1}\times 0.005\right)\times\frac{7}{0.2} = 0.35+0.10 = 0.45\ \mathrm{m}^2 \tag{25.12}$$

The total area A_t is:

$$A_t = 7\times 1 = 7\ \mathrm{m}^2 \tag{25.13}$$

Therefore, the area covered by the geogrid is 6.4% (0.45/7) of the total area, yet it develops 28.1% (Eq. 25.11) of the total friction. Of course, in addition to the pull-out resistance of the geogrid itself, it is important to ensure that the connection can handle such a force.

Creep tension properties are also very important for geogrids, as they are usually subjected to constant tension during their design life. At low stress levels (low fraction of the tensile strength F_{ug}), the geogrid will exhibit strain that increases linearly with the log of time. The slope of that line is the constant strain rate (e.g., 0.5% strain per log cycle of time). This creep strain rate depends on the type of polymer, the temperature, and the stress level. At intermediate stress levels, the geogrid may exhibit a delayed failure, in which the creep strain rate is constant for a while but increases dramatically after a certain time, leading to failure (Figure 25.5). Delayed creep failure typically occurs within the range of 25% to 50% of the ultimate tension F_{ug}. Geogrids should be tested for creep response (ASTM D5292 and ASTM D6992). The time temperature superposition (TTS) principle can be used to shorten the testing time. In TTS, advantage is taken of the fact that a long time at low temperature is equivalent to a short time at high temperature.

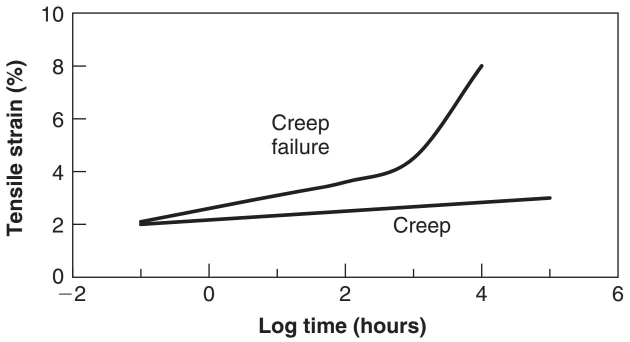


Figure 25.5 Creep behavior of geogrids.

As in the case of geotextiles and geomembranes, other properties of geogrids include temperature effects, chemical effects, biological effects, radioactive effects, and sunlight effects. Designers use reduction factors to take into account these factors, all of which affect the long-term strength and function of the geogrid.

25.3.4 Properties of Geosynthetics Clay Liners

Physical Properties

Geosynthetic clay liners (GCLs) are a recent innovation; they are made of a thin layer of bentonite clay sandwiched between two layers of geotextiles or geomembranes (Figure 25.6). They come in large flexible rolls and are used as containment barriers in the case of landfill liners and covers, for example. They are either nonreinforced or reinforced. The reinforcement solves the following problem. When hydrated, the bentonite clay is extremely slick and represents a weak shear plane that would initiate failure when placed in a slope. To remedy this situation, GCLs can be reinforced by fibers (needle-punched) or stitches (stitch bonds) that tie the two sides of the GCL together. Nonreinforced GCLs are used as barriers on flat ground, whereas the more common reinforced GCLs are used on sloping ground.

The clay type can be sodium bentonite or calcium bentonite. Sodium bentonite has the lowest hydraulic conductivity, but its availability worldwide is limited. The thickness of a GCL varies significantly because of the difference in hydrated and dry thicknesses. Furthermore, it is difficult to isolate the thickness of the clay layer from its boundaries. The hydrated thickness is more important, as it affects the hydraulic properties. The total hydrated thickness of GCLs typically varies between 10 and 30 mm. The mass per unit area of GCL is in the range of 5 to 6 kg/m^2, with 4 to 5 kg/m^2 of dry bentonite between the geosynthetic layers. Once hydrated, the GCL can easily become twice as heavy. The GCL is sold "dry," which means that it has a low initial water content of around 10%. When the bentonite hydrates, it can reach water contents well over 100%.

Hydraulic Properties

The hydraulic properties of GCLs are very important, as GCLs are mostly used as barriers. The chemistry of the

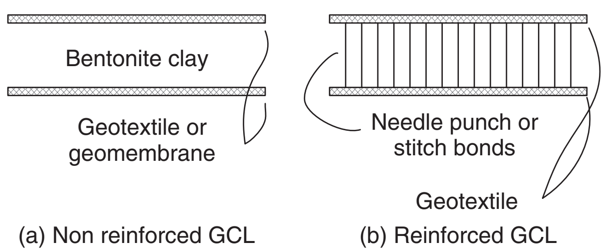


Figure 25.6 Geosynthetic clay liners cross section.

liquid hydrating the GCL can make a significant difference. Koerner (2012) reports on the difference between distilled water, tap water, mild landfill leachate, harsh landfill leachate, and diesel fuel. The results show that the swell movement is largest with distilled water and zero with diesel fuel; the other liquids lead to intermediate swell movement. The swell test consists of placing a "dry" sample of bentonite in a consolidometer, submerging it in water, and allowing it to swell under light vertical pressure. The bentonite will swell, reaching the maximum swell movement in a time that can vary from 2 weeks to 2 months.

The hydraulic conductivity k of a GCL can be measured in the laboratory using a flexible-wall triaxial permeameter (ASTM D5887). The in situ conditions should be reproduced as closely as possible, including the applied pressure and type of liquid. The value of *k* is obtained as follows:

$$q = kiA = k\left(\frac{\Delta h}{t}\right)A \tag{25.14}$$

where q is the flow rate, *k* is the hydraulic conductivity, *I* is the hydraulic gradient, *A* is the cross-sectional area through which the liquid flows, Δh is the loss of total head across the GCL, and t is the thickness to be permeated.

This thickness is very difficult to measure accurately and leads to inaccuracies in quoting the *k* value. Daniel et al. (1997) reported that the same GCL tested by many different laboratories yielded values between 2×10^{-11} and 2×10^{-12} m/s. This range can be expected for the *k* values of sodium bentonite GCLs and for water as permeate. Because of the difficulties associated with thickness measurements, the results of a GCL permeability test are usually given in terms of flow per unit area (q/A in $m^3/s.m^2$). At the junction between contiguous sheets of GCL, there is an overlap. A minimum overlap of 150 mm is recommended to maintain a hydraulic conductivity equal to that of the GCL itself. Sometime a layer of bentonite without GCL is added at the junction. From the long-term endurance point of view, freeze-thaw cycles and shrink-swell cycles do not seem to affect the hydraulic conductivity of GCLs significantly.

Mechanical Properties

The bentonite contributes very little to the wide width tension strength of a GCL. As a result, the tension strength of a GCL is estimated by using the values of the geotextile or geomembrane within which the bentonite is sandwiched.

The shear strength of the GCL depends on the interface considered: upper geosynthetic and soil or waste, bentonite clay layer with or without reinforcement, lower geosynthetic and soil or waste. The upper and lower interface shear strengths between the materials above or below the GCL are addressed by considering the type of geosynthetic involved. For geotextiles, see section 25.2.1; for geomembranes, see section 25.2.2. The shear strength of the bentonite clay layer is measured by a direct shear test (ASTM D6243). Koerner (2012) reports on tests where the shear strength parameters c and φ decrease dramatically upon hydration of the bentonite clay layer with water when tested in a relatively rapid direct shear test. This decrease in shear strength parameters is not as severe when the hydrating liquid is leachate, and no decrease was found when the hydrating liquid was diesel fuel. The shear strength of reinforced GCL is much higher than that of unreinforced GCL and larger displacements are required to reach failure.

One concern is the long-term shear strength of reinforced GCLs. This is related to the long-term strength of needle-punched fibers or stitch bonds. The long-term (100-year) internal shear strength of reinforced GCL is up to 50% of the short-term shear strength (Koerner 2012). The *peel strength* of reinforced GCLs refers to the maximum force per unit length that the upper and lower geosynthetic layers can resist when pulled away from each other at a 90° angle to the main direction of the GCL seam; it is measured in kN/m (ASTM D6496). Resistance to puncturing is also important and should be measured. The tests include ASTM D4883 and ASTM D6241. Squeezing of the bentonite layer away from a location by local pressure is avoided by placing a layer of sand, for example, above the GCL.

25.3.5 Properties of Geofoams

Physical Properties

Geofoams are blocks made of light yet hard polystyrene materials. They are used as light fills, as thermal insulations, and as compressible inclusions. The width and the height of the blocks vary from 0.3 to 1.2 m, and the length from 1.2 to 5 m. A distinction is made between expanded polystyrene (EPS) and extruded polystyrene (XPS). EPS is made from solid beads of polystyrene expanded by blowing gas through them. XPS consists of melted polystyrene crystals mixed with additives and a blowing agent and shaped by extrusion through a die; the white Styrofoam coffee cups are made of extruded polystyrene. EPS geofoam blocks are typically larger than XPS geofoam blocks. The unit weight of geofoams ranges from 0.1 to 0.5 kN/m^3, which is much smaller than the average unit weight of soil (~20 kN/m^3). Geofoams do not absorb much water, but are combustible and should not be exposed to temperatures in excess of 95°C.

Mechanical Properties

Because geofoams are often used as lightweight fill, the unconfined compressive strength is of interest. Figure 25.7 shows results from Negussey (1997), as presented by Koerner (2012).

This unconfined compression stress-strain data indicates that the geofoam exhibits a linear behavior with a modulus E until a yield strength σ_y, and then strain hardens at a modest rate. The yield strength σ_y is reached at around 2% compressive strain. The unit weight of the geofoam γ_{GF} has

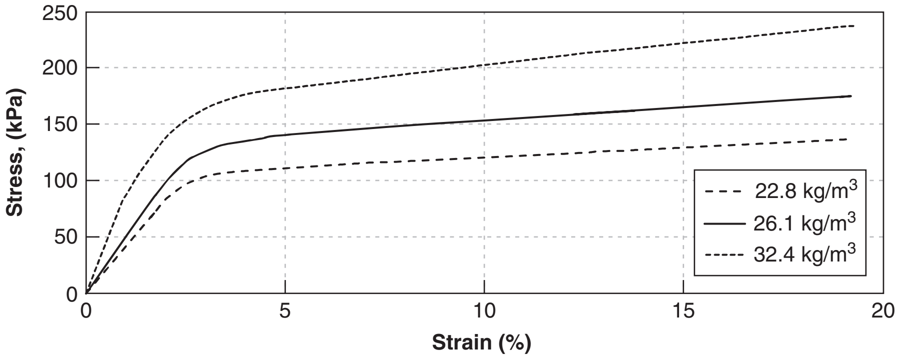


Figure 25.7 Stress-strain curves for geofoam. (After Negussey 1997.)

a direct impact on its mechanical properties and the following equations can be derived from Negussey's data:

$$\text{Modulus of EPS geofoam} \quad E\,(\text{MPa}) = 20\gamma_{GF}\,(\text{kN/m}^3) \tag{25.15}$$

$$\text{Modulus of XPS geofoam}\ E\,(\text{MPa}) = 60\gamma_{GF}\,(\text{kN/m}^3) \tag{25.16}$$

$$\text{Yield strength of EPS geofoam}\ \sigma_y\,(\text{kPa}) = 500\gamma_{GF}\,(\text{kN/m}^3) \tag{25.17}$$

$$\text{Yield strength of XPS geofoam}\ \sigma_y\,(\text{kPa}) = 800\gamma_{GF}\,(\text{kN/m}^3) \tag{25.18}$$

The internal shear strength of geofoams can be tested by following ASTM C253, and the shear strength between geofoam blocks can be tested by direct shear testing (ASTM D5321). The tensile strength of geofoams, σ_t, is much larger than that of soils. Using the data from Styropor (1993), the following equation can be proposed:

Tensile strength of EPS geofoam

$$\sigma_t\,(\text{kPa}) = 1250\gamma_{GF}\,(\text{kN/m}^3) \tag{25.19}$$

Creep properties are important because geofoams may be subjected to long-term loads (in lightweight embankments, for example). In creep testing of geofoams, the time temperature superposition can be used to shorten the time required to characterize the long-term behavior. Data from Negussey (1997) indicates that when the sustained compression stress is below 50% of the unconfined compression strength, the viscous exponent (Chapter 14, Eq. 14.9; section 15.8) of geofoam is within the range of values found in soils (n = 0.01 to 0.08).

Thermal Properties

Geofoams can be used as thermal insulation under buildings. Therefore, their thermal properties are important. The main property is the R value (see section 16.3). The R value is defined as:

$$R(^\circ\text{C.m}^2/\text{W}) = \frac{\Delta T(^\circ\text{C})}{q\,(\text{J/s.m}^2)} \tag{25.20}$$

where R is the R value or thermal resistance in °Celsius.m^2/Watt, ΔT is the difference in temperature on either side of the geofoam in °Celsius, and q is the heat flow in J/s.m^2. The higher the R value is, the more insulating the geofoam is. The R value per unit width of geofoam is R′ expressed in °Celsius.m/Watt. The R′ value of geofoams varies from 20 to 40 and increases with unit weight. It is typically higher for XPS than for EPS.

Other aspects to be addressed are the chemical resistance of geofoams, which are readily attacked by hydrocarbons such as gasoline; degradation due to long-term exposure to UV rays; and flammability. This is why it is best for geofoams to be covered by a soil backfill as soon as possible after installation.

25.3.6 Properties of Geonets

Geonets are open-grid geosynthetics very similar to geogrids; however, their purpose is to serve as spacers by providing flow conduits within their thickness. They are typically used in conjunction with a geotextile or geomembrane on top and bottom of the geonet. Whereas geogrids have a single layer of ribs typically perpendicular to each other, the ribs in geonets are stacked on top of each other (2 or 3 layers) and lined up in diagonals to facilitate flow. The mass per unit area varies from 0.8 to 1.6 kg/m^2 and the thickness from 4 to 8 mm. The mechanical properties of geonets are similar to those of geogrids, but the hydraulic properties are most important, as geonets are used primarily for drainage purposes.

The drainage capacity is quoted in flow per unit width of geonet. Values in the range of 10^{-3} to 10^{-4} m^3/s.m are common, but can decrease by 30% when the pressure increases to 1000 kPa. The drainage capacity of geonets may also be quoted in terms of transmissivity Θ (Eq. 25.5), which is related to the flow rate per unit width (*q/W*) by:

$$\Theta = \frac{1}{i} \times \frac{q}{W} \tag{25.21}$$

where i is the hydraulic gradient. The EPA has regulations indicating that a geonet must have a transmissivity of at least 3×10^{-5} m^2/s for landfills and 3×10^{-4} m^2/s for surface impoundments.

25.4 DESIGN FOR SEPARATION

Separation means that the two materials on each side of the geosynthetic cannot penetrate it. This is associated with failure mechanisms by impact, punching, or tear (Figure 25.8). *Impact* refers to the case where a stone falls on top of the geosynthetic. *Punching* refers to the case where the geosynthetic is pushed through an opening between large aggregates. *Tear* refers to the case where the geosynthetic is pulled apart by stones that are moving away from each other in the deformation process of the geotechnical structure.

Designing for impact first requires estimating the energy of the falling object. If the stone is represented by a sphere, the energy E_{stone} to be dissipated at impact is:

$$E_{stone} = Wh = \frac{\pi d^3}{6}\gamma h \tag{25.22}$$

where W is the weight of the stone, h is the height of drop, d is the stone diameter, and γ is the unit weight of the stone.

This energy is absorbed in part by the geosynthetic layer and in part by the soil immediately below the geosynthetic. If the soil is soft, the stone has a soft landing and the peak force in the geosynthetic is lower than if the soil is stiff but the deformation is large. If the soil is extremely weak, only the geosynthetic resists the impact. To take the soil support contribution into account, the value of E_{stone} is divided by a soil support factor F_s varying between 5 and 25 (Koerner 2012). This energy is then compared to the impact strength E_{geosyn} (Joules) of the geosynthetic (section 25.3.1). The impact strength of the geosynthetic is divided by a cumulative reduction factor F, which accounts for installation damage, creep, and chemical/biological degradation, for example. F_r varies from as low as 1.1 to as high as 9. The design ensures that:

$$\frac{E_{stone}}{F_{soil}} \leq \frac{E_{geosyn}}{F_{reduc}} \tag{25.23}$$

Designing against puncture requires estimating the force F_{stone} generated by the stone protruding into the geosynthetic due to a pressure p applied. The pressure p may be applied by a rolling truck, for example. Koerner (2012) proposed:

$$F_{stone} = p\,d_a^{\,2} S_1 S_2 S_3 \tag{25.24}$$

where d_a is the diameter of the penetrating stone, and S_1, S_2, and S_3 are the protrusion factor, the scale factor, and the shape factor respectively. The product $S_1S_2S_3$ varies from 0.65 in the most severe condition (angular large stone) to 0.01 in the most favorable condition (rounded small particles). The value of F_{stone} is then compared to the strength of the geosynthetic (Eq. 25.2) divided by the cumulative reduction factor.

Designing against tear starts by calculating the tension force generated in the geosynthetic when squeezed between two layers of soil. When the upper and lower layers of soil are subjected to a rolling truck, for example, the layers deflect and bend locally under the wheel load. During this bending, the geosynthetic trapped between the two soil layers is subjected to a tension force $F_{tension}$, which is given by (Giroud 1981):

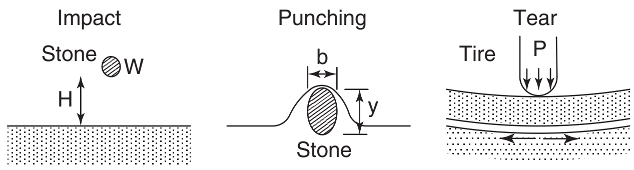


Figure 25.8 Modes of failure of geosynthetics in separation.

$$F_{tension} = 0.1 p d_a^{\,2} f(\varepsilon) = 0.025 p d_a^{\,2}\left(\frac{2y}{b} + \frac{b}{2y}\right) \tag{25.25}$$

where p is the pressure applied, d_a is the particle or stone diameter, $f(\varepsilon)$ is a function of the strain in the geosynthetic, y is the displacement into the stone void, and b is the width of the stone void (Figure 25.8). Then the value of $F_{tension}$ is compared to the strength of the geosynthetic (Eq. 25.2) divided by the cumulative reduction factor.

25.5 DESIGN OF LINERS AND COVERS

Liners are barriers placed at the bottom of landfills to prevent the waste and the liquid it generates from contaminating the soil and water below the waste. *Covers* are barriers placed on top of landfills to close them, prevent the waste from contaminating the surrounding environment, and prevent the gas it generates from escaping without control. Both liners and covers have evolved dramatically in the past 30 years, with most of the change taking place between 1980 and 1990.

Before 1980, only a compacted clay liner was required at the bottom of landfills (Figure 25.9). The leachate collection system was a layer of sand and gravel with perforated pipes; there was no leachate detection system and no secondary liner to decrease the probability of leaks through the liner. Nowadays, liners are double composite systems (Figure 25.10) with a leachate collection system, a primary liner, a leak detection system, and a secondary liner. The liner involves many layers: geosynthetic for the purpose of separation (geomembrane), barrier (geosynthetic clay liner), and drainage and leachate detection-collection system (geocomposite with geonet-geotextile-geomembrane) in addition

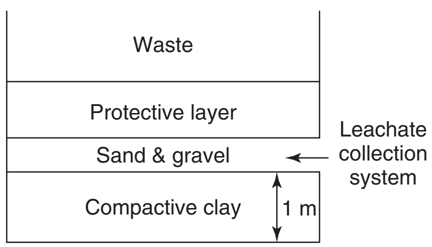


Figure 25.9 Early liner cross section.

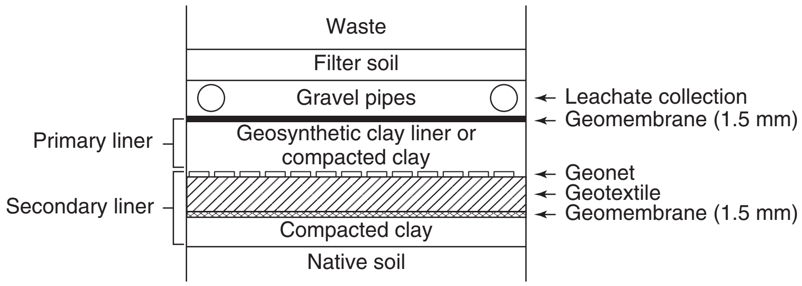


Figure 25.10 Example of a modern liner cross section.

to layers of compacted clay. The change from compacted clay liners to geosynthetic base liners was prompted by the fact that the compacted clay layer had to be quite thick (up to 1.5 m thick), thereby taking up space that could otherwise have been used for waste; and by the discovery that certain chemicals, such as organic solvent leachate, dramatically increase the hydraulic conductivity of clays. Given this, a compacted clay liner alone could not ensure that no leakage would occur.

The leakage through a liner is very rarely zero. It is measured in liters per hectares per day (lphd). A liter is 10^{-3} m^3 and an hectare is 10,000 m^2. The leakage varies with time as the landfill is being constructed and during its operation. Koerner (2012) defines stage 1 as the stage during construction, stage 2 as when considerable waste is placed, and stage 3 as when the final cover is placed. Furthermore, a distinction should be made between the different types of liners: geomembrane alone (GM), geomembrane over a compacted clay liner (GM/CCL), and geomembrane over a geosynthetic clay liners (GM/GCL). Based on the work of Othman et al. (1997) and Bonaparte et al. (2002), who gathered leakage rates for 289 landfills, Koerner gives the rates shown in Figure 25.11.

The geomembranes used in liners are typically made of high-density polyethylene (HDPE). Some global minimum recommendations for the survivability of geomembranes used in liners are presented in Table 25.3. Low severity refers to a careful manual placement with light loads on smooth ground, for example; very high severity refers to machine handling on rough, stiff ground under heavy loads.

Landfill covers are necessary so the waste does not contaminate the surrounding environment. These covers prevent rainwater from accessing the waste and keep the gas generated by the landfill from escaping into the atmosphere without control. An example of a cover cross section is shown in Figure 25.12. The layers involved include vegetation for a positive landscape impact, an erosion control geosynthetic, a top soil and cover soil layer for the plants to grow, a drainage layer (combination of geotextile/geonet/geomembrane), and then a second barrier (compacted clay or GCL), a gas collection layer, and the waste. The erosion control geosynthetic is often necessary because the top of the landfill is like a big hill, so the runoff water can erode the top soil in the cover. The drainage layer drains the rainwater away from the landfill. The barrier layer provides a second assurance that the water will not penetrate and also that the gas produced will not escape without control. The gas collection layer is necessary because most municipal landfills generate a lot of gas, mainly

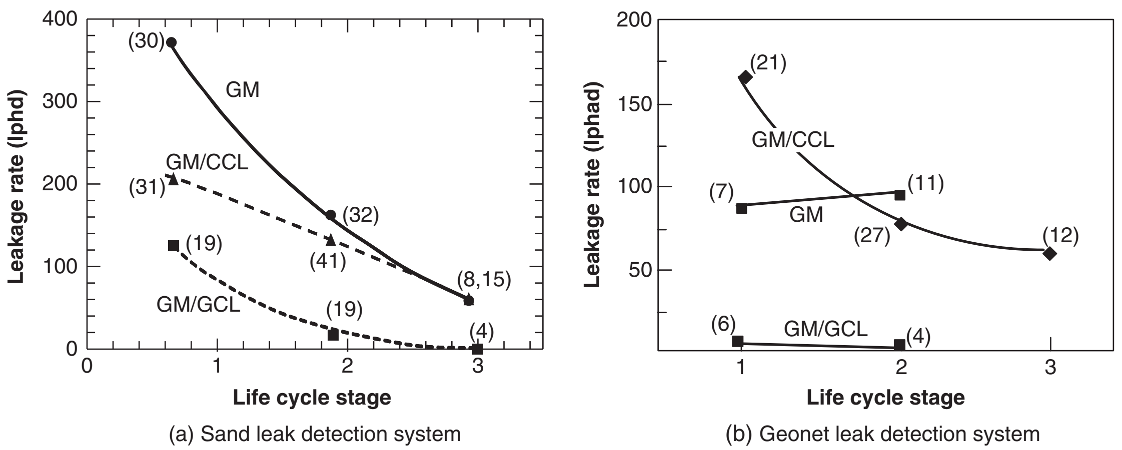


Figure 25.11 Leakage rates in landfill liners. (After Koerner 2012. Robert M. Koerner—Copyright Owner.)

Table 25.3 Minimum Requirements for Geomembrane Survivability

	Required Value Considering the Degree of Severity			
Property	Low	Medium	High	Very High
Thickness (mm)	0.63	0.75	0.88	1.00
Tensile strength (kN/m)	7	9	11	13
Tear resistance (N)	33	45	67	90
Puncture (N)	110	140	170	200
Impact resistance (J)	10	12	15	20

(After Koerner 2012)

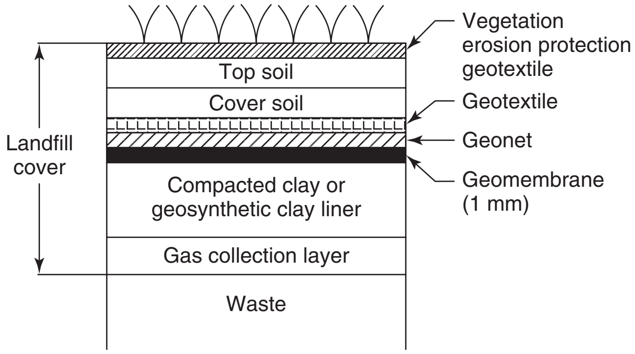


Figure 25.12 Example of a landfill cover. (After Koerner, 2012.)

methane (CH_4) and carbon dioxide (CO_2). This layer can be made of sand with perforated pipes that collect the gas; the perforated pipes are connected to risers (vertical unperforated pipes) that bring the gas to a collection point; alternatively, the gas may be burned and the combustion products released into the atmosphere (flare).

25.6 DESIGN FOR REINFORCEMENT

25.6.1 Road Reinforcement

The role of geosynthetics in road reinforcement is threefold: separation, reinforcement, and minimization of crack propagation. Geotextiles can be used for separation; geotextiles and geogrids can be used for reinforcement and mitigation of crack propagation. A distinction is made here between applications for unpaved roads, paved roads, and overlay of asphalt flexible pavements. Separation has already been addressed in section 25.4. It applies to the case of unpaved and paved roads, but rarely in the case of the overlay of asphalt flexible pavements.

Prevention of crack propagation applies to the case of asphalt flexible pavements only. Indeed, for overlay of asphalt pavements, the rolling surface may contain vertical cracks. Minimizing the chances that the crack will propagate from the lower cracked asphalt layer vertically through the overlay to the rolling surface can be achieved by using a thicker overlay asphalt layer or the combination of a geotextile and a thinner asphalt overlay. It is important to keep moisture from rising through the overlay. For this, the geotextile is first rendered impervious by impregnating it with bitumen; then it is placed on top of the old pavement; then the overlay is constructed. The concept is that the geotextile will provide horizontal reinforcement with significant tensile strength and contain the future increase of the crack growth.

The role of geosynthetics in road reinforcement is better suited to unpaved road than paved roads. The reason is that, on the one hand, geosynthetics tend to generate their resistance over a level of strain much larger than materials like asphalt and concrete and, on the other hand, unpaved roads deflect more under traffic load than paved roads. Thus, geosynthetics will contribute more to the capacity of unpaved roads than paved roads. The design concept is to calculate the pavement thickness with and without the geosynthetics layer and perform an economic analysis on the two options. The benefit of using the geosynthetic layer is derived by assuming that the pressure level on the rolling surface (tire pressure) can be increased due to the presence of the geosynthetics. Without the geosynthetics, the stress on the subgrade must be kept within the elastic limit, whereas with the geosynthetics, the stress on the subgrade can reach the bearing capacity of the subgrade, as failure will be prevented by the geosynthetics. Both geotextiles and geogrids can be used for this application. Details of these designs can be found in Koerner (2012).

25.6.2 Mechanically Stabilized Earth Geosynthetic Walls

Retaining walls (see Chapter 22) may be top-down walls, such as tieback walls; or bottom-up walls, such as gravity walls. Mechanically stabilized earth (MSE) walls are bottom-up walls, meaning that they are built starting at the bottom and going up until the top of the wall is completed. MSE walls are built by placing a layer of soil (say, 0.3 m thick), compacting it, then placing a layer of reinforcement, then a layer of soil and compacting it, placing a layer of reinforcement (geotextile or geogrids in this case), and so on to the top of the wall (Figure 25.13). These walls can be built in such

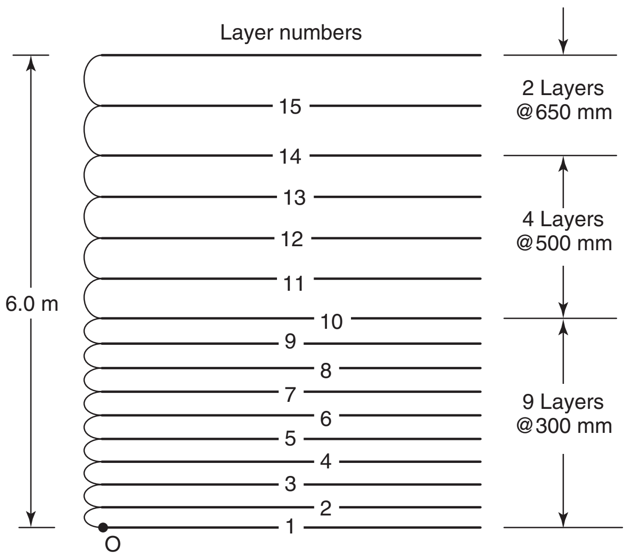


Figure 25.13 MSE wall with geosynthetics reinforcement.

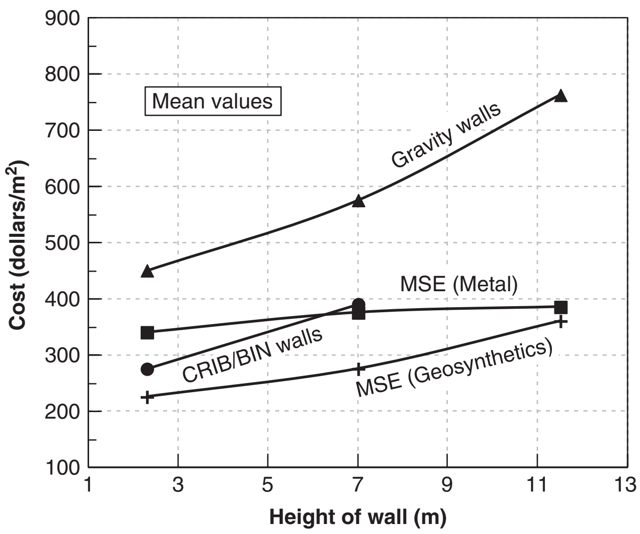


Figure 25.14 Cost of retaining walls. (From Koerner 2012.)

a fashion to heights reaching tens of meters and are less expensive than conventional gravity or cantilever retaining walls (Figure 25.14). The reinforcement can be made of rigid inclusions such as steel strips and steel wire mesh or flexible inclusions such as geosynthetics (geotextiles and geogrids). The front of the wall is covered with panels that are tied to the reinforcement. The design of geosynthetic MSE walls includes internal stability and external stability. The minimum length of reinforcement is set at 0.7 H where H is the height of the wall.

External Stability

Bearing capacity at the base of the wall, general slope stability of the wall and the slope within which it rests, sliding of the wall mass, and overturning of the wall mass are all external stability issues. Commonly used factors of safety for a global factor of safety approach and for each one of those failure modes are presented in Table 25.4. Average load and resistance factors for an LRFD approach are also shown in Table 25.4. Bearing capacity and slope stability are dealt with in Chapters 17 and 19 respectively. Sliding of the wall mass can be addressed through a factor of safety expressed as:

$$F_{sliding} = \frac{\sum \text{Resisting forces}}{\sum \text{Driving forces}} = \frac{W \tan \varphi'}{P_a} \tag{25.26}$$

where W is the weight of the wall mass per unit length of wall, φ' is the friction angle of the interface at the bottom of the wall, and P_a is the horizontal force per unit length of wall due to the active earth pressure against the back of the wall (see Chapter 22).

Alternatively, for the LRFD approach the equation is:

$$\gamma P_a = \varphi W \tan \varphi' \tag{25.27}$$

where γ is the load factor and φ is the resistance factor.

Overturning of the wall is addressed through a factor of safety expressed as a ratio of moments. The moments are

Table 25.4 Some Possible Load and Resistance Factors for External and Internal Stability of Geosynthetic MSE Wall

				Load Factor	
Stability	Design Issue	Factor of Safety	Resistance Factor	Dead	Live
External Stability	Bearing capacity	2	0.5	1.25	1.75
	Slope stability	1.5	0.7	1.25	1.75
	Sliding	1.5	0.85	1.25	1.75
	Overturning	2	0.7	1.25	1.75
Internal Stability	Pull-out	1.5	0.9	1.5	1.75
	Breakage	1.5	0.9	1.5	1.75

taken around the bottom of the front of the wall (point O on Figure 25.13):

$$F_{overturning} = \frac{\sum \text{Resisting moments}}{\sum \text{Driving moments}} = \frac{Wd}{P_a x_a} \tag{25.28}$$

where d is the moment arm of the weight W of the wall and x_a is the moment arm of the active earth pressure force P_a.

Alternatively, for the LRFD approach the equation is:

$$\gamma\, P_a\, x_a = \varphi\, W\, d \tag{25.29}$$

where γ is the load factor and φ is the resistance factor.

Internal Stability

Pull-out capacity and yield of the geosynthetic reinforcement are the two aspects of internal stability of an MSE wall with geosynthetic reinforcement. Commonly used factors of safety for a global factor of safety approach and for each one of those failure modes are presented in Table 25.4. Average load and resistance factors for an LRFD approach are also shown in Table 25.4. Pull-out capacity requires an understanding of the load distribution in the reinforcement. Figure 25.15 shows the variation of the tension load T (kN/m) in the reinforcement as a function of the distance from the front of the wall.

At the wall facing, the load T in the reinforcement is very small, and then it increases as the instability of the wedge of soil near the wall is transferred into the geosynthetic as a tension force T (kN/m). At a distance $L_{\max}$ from the front, the tension T reaches a maximum $T_{\max}$. Beyond $T_{\max}$, the tension decreases and reaches zero at a certain distance from the front. This distance must be less than the actual length L of the reinforcement, or significant deformations and possibly failure will occur. The true embedment or anchoring length L_a available to resist the active pressure force against the wall is $L - L_{\max}$. The design requires a knowledge of $L_{\max}$, which is to be ignored in the length required to resist $T_{\max}$. The force $T_{\max}$ is calculated as:

$$T_{\max} = s_v \sigma_{ah} \tag{25.30}$$

where $T_{\max}$ is the maximum line load (kN/m) to be resisted by the geosynthetic layer at a depth z, s_v is the vertical spacing between reinforcement layers at the depth z, and σ_{ah} is the total horizontal active stress at the depth z. The stress σ_{ah} is discussed in Chapter 22.

Now that we have calculated the load, we need to find the length of reinforcement that will safely carry the load without pulling out of the soil. The pull-out line capacity T_{pullout} (kN/m) of the geosynthetic layer is given by:

$$T_{\text{pull out}} = 2 f_{\max} L_a \tag{25.31}$$

where $f_{\max}$ is the maximum shear stress that can be developed on both sides of the interface between the geosynthetic and the soil, and L_a is the anchoring length. Recall that L_a is the length beyond the failure wedge. The shear stress $f_{\max}$ is evaluated as follows:

$$f_{\max} = \sigma_v' \tan \delta \tag{25.32}$$

where σ_v' is the vertical effective stress on the geosynthetic layer at depth z (including any effect from surcharge or load at the ground surface), and $\tan\delta$ is the coefficient of friction between the soil and the geosynthetic. Then the ratio between the load $T_{\max}$ and the resistance T_{pullout} must satisfy a factor of safety F (Table 25.4):

$$T_{\text{pull out}} = F \times T_{\max} \tag{25.33}$$

and the required safe length L_a of the geosynthetic sheet is given by:

$$L_a = \frac{F s_v \sigma_{ah}}{2\sigma_v' \tan \delta} \tag{25.34}$$

In the simple case where $\sigma_{ah} = K_a\, \sigma_v'$, where $K_a = 0.33$, $F = 2$, and $\tan \delta = 0.5$, then L_a is equal to 0.66 s_v, which is quite small for normal vertical spacing of 0.3 to 0.5 meters.

A load and resistance factor approach would consist of replacing Eq. 25.34 by:

$$\gamma 2 L_a \sigma_v' \tan \delta = \varphi s_v \sigma_{ah} \tag{25.35}$$

where γ is the load factor and φ is the resistance factor.

Note that the anchoring length L_a is constant with depth. The reason is that as the load increases with depth, so does the resistance. In practice, the minimum embedment length is set at 1 m. The distance $L_{\max}$ required to develop the load

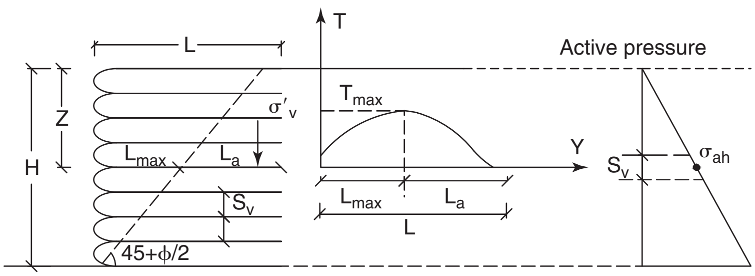


Figure 25.15 Load in the reinforcement. (After Theisen, 1992.)

in the reinforcement layer is taken as the width of the active wedge (see Chapter 22 and Figure 25.15):

$$L_{\max} = (H - z)\tan\left(45 - \frac{\varphi'}{2}\right) \tag{25.36}$$

Therefore, the final length of the geosynthetic layer L is:

$$L = (H - z)\tan\left(45 - \frac{\varphi'}{2}\right) + \frac{\frac{\gamma}{\varphi} s_v \sigma_{ah}}{2\sigma'_v \tan\delta} \tag{25.37}$$

For construction simplicity, the length L is often kept constant for the entire wall. Because the length L is largest at the top of the wall, practically the length of reinforcement is taken as:

$$L = H\tan\left(45 - \frac{\varphi'}{2}\right) + \frac{\frac{\gamma}{\varphi} s_v \sigma_{ah}}{2\sigma'_v \tan\delta} \tag{25.38}$$

where σ_{ah} and σ'_v are calculated at the depth of the first reinforcement layer.

Yield of the geosynthetic layer is the next design issue. We must make sure that the geosynthetic can safely carry the load $T_{\max}$ without yielding or rupturing. For this, we need to find the allowable tensile resistance of the geosynthetic layer T_{allow}. This allowable tensile resistance T_{allow} is obtained from the measured ultimate tensile resistance T_{ult} and given by:

$$T_{allow} = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_{CBD}} \tag{25.39}$$

where T_{allow} and T_{ult} are the allowable and ultimate resistance of the geosynthetic layer (kN/m); and RF_{ID}, RF_{CR}, and RF_{CBD} are strength reduction factors that take into account installation damage, creep, and chemical and biological factors.

These strength reduction factors vary between 1 and 2 depending on the application (Koerner 2012), and average 1.55, 2.15, and 1.32 respectively. Note that once combined, these reduction factors lead to using an allowable tension that is about 20% of the measured ultimate tensile strength T_{ult} of the geosynthetic. The required ultimate strength T_{ult} of the geosynthetic is such that:

$$T_{allow} = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_{CBD}} = T_{\max}\frac{\gamma}{\varphi} \quad \text{or}$$

$$T_{ult} = T_{\max}\frac{\gamma}{\varphi} \times RF_{ID} \times RF_{CR} \times RF_{CBD} \tag{25.40}$$

We must also make sure that there is no slip at the location of the overlap between geosynthetic layers. Thus, the overlap distance must satisfy Eq. 25.35. Because the overlap is located near the wall where the tension load is less than T_{max}, a length equal to ½ L_a is typically used for the required overlap distance.

25.6.3 Reinforced Slopes

A distinction must be made here between manmade bottom-up slopes (e.g., embankments and dams) and natural slopes or top-down slopes (e.g., hillside slopes and cuts). In the first case, it is possible to install geosynthetic reinforcement layers (geotextiles or geogrids) as the slope is built. In the second case, it is not possible to use geosynthetics as reinforcement; however, natural slopes and cuts can be reinforced with geosynthetics by covering the slope with a geosynthetic layer and anchoring the cover deeply beyond the failure plane. For bottom-up slopes, the factor of safety of the slope with reinforcement is calculated as presented in section 19.14.

25.6.4 Reinforced Foundations and Embankments

Geosynthetics can be placed below a foundation or embankment to improve its carrying capacity and performance. Geotextiles and geogrids are most commonly used this way. The main design issues are bearing capacity, settlement, and anchoring length.

Bearing capacity. Bearing capacity is improved because the failure plane has to pull on the geosynthetic layer. The degree of improvement is analyzed by the same method used for slope stability analysis once a circular surface has been chosen. For example, consider a strip footing of width B at the surface of a soft clay with an undrained shear strength s_u (Figure 25.16). A geosynthetic layer is placed at a depth of $B/2$ with a tensile strength T kN/m. Let's find the value of T required to increase the bearing capacity by a factor of 2. At failure and for a circular failure surface as shown on Figure 25.16, moment equilibrium around O gives:

$$p_u B\frac{B}{2} = s_u \pi BB + TB \quad \text{or } p_u = 2\pi s_u + 2\frac{T}{B} \tag{25.41}$$

Note that it is assumed here that the geosynthetic is flexible and that it will deform at the intersection with the failure surface in such a way that it will become tangential to the failure circle. For the geosynthetic layer to double the bearing capacity, we must have:

$$\frac{2\pi s_u + 2\frac{T}{B}}{2\pi s_u} = 2 \quad \text{or} \quad T = \pi s_u B \tag{25.42}$$

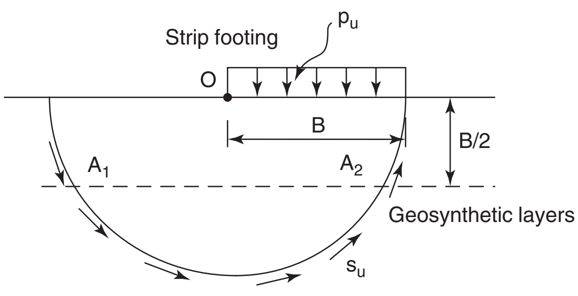


Figure 25.16 Foundation reinforcement.

For a 2 m wide strip footing and $s_u = 120$ kPa, the value of T is 240 kN/m. This is a very high allowable tensile strength for a geosynthetic and several layers would have to be used to safely achieve this level of tensile strength. However, for $s_u = 40$ kPa, the value of T is 80 kN/m, which can be achieved with one layer.

Settlement. Settlement is also affected by the presence of a geosynthetic layer. At small displacements, the contribution is limited, as the geosynthetic typically has to deform enough to make a difference. The magnitude of settlement necessary for this contribution to be significant is more consistent with embankments than foundations. Indeed, larger settlements are more readily accommodated by flexible embankments than by rigid foundations. Figure 25.17 shows an embankment with a width L and a geosynthetic layer at the bottom of it. If it is assumed that the embankment settles s at its center, that the deflected shape of the bottom of the embankment is an arc of a circle, and that the circle passes through the ends of the embankment, then the relationship between the settlement s and the radius R of the circle is:

$$(R-s)^2 + \frac{L^2}{4} = R^2 \text{ or (neglecting higher-order terms)}$$

$$R = \frac{L^2}{8s} \tag{25.43}$$

where L is the width of the embankment.

Then the geosynthetic stretches from an initial length L to a deformed length L':

$$L' = 2R\ Arc\ \sin\frac{L}{2R} \tag{25.44}$$

which leads to a strain ε in the geosynthetic of:

$$\varepsilon = \frac{Arc\ \sin(4s/L)}{4s/L} - 1 \tag{25.45}$$

and a tension T equal to:

$$T = E\varepsilon \tag{25.46}$$

where E is the geosynthetic modulus (section 25.3.1).

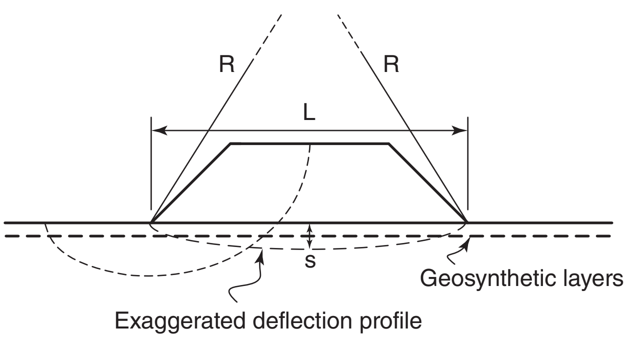


Figure 25.17 Embankment reinforcement.

The geotextile or geogrid required needs to have a tensile strength much higher than T because of the reduction factors (Eq. 25.39).

Anchoring length. The geosynthetic layer must extend far enough beyond the edges of the embankment or the foundation to ensure that it will not pull out when loaded. The anchor length is calculated as presented in Eq. 25.7. The anchor length can be shortened by wrapping the geosynthetic around large embedded stones or timber cribbing.

25.7 DESIGN FOR FILTRATION AND DRAINAGE

Filtration refers to the case in which water is flowing perpendicular to the plane of the geosynthetic; *drainage* is the case in which water flows in the direction of the geosynthetic. The design of a geosynthetic filter or drain (mostly nonwoven, needle-punched geotextile) has two aspects: water passage and soil retention. The problem is that for water conveyance, the geotextile should have large openings, whereas for soil retention it should have small openings. A compromise must be found.

For filtering, the required permittivity for water passage is calculated:

$$\Psi_{req} = \frac{k}{t} = \frac{q}{\Delta h\, A} \tag{25.47}$$

where ψ_{req} is the permittivity, k is the hydraulic conductivity, t is the thickness of the geotextile, q is the flow through the flow net to be handled by the geotextile, Δh is the drop of total head through the flow net, and A is the cross-sectional area perpendicular to the flow.

The steps include drawing a flow net, calculating the flow q through the flow net, determining the required permittivity from Eq. 25.47, and seeking the geotextile that satisfies this requirement. Note that the permittivity of the geotextile has to be corrected for reduction factors as follows:

$$\Psi_{allow} = \frac{\Psi_{ult}}{(RF_{SCB} \times RF_{CR} \times RF_{IN} \times RF_{CC} \times RF_{BC})} \tag{25.48}$$

where ψ_{allow} is the permittivity that can be used in design, ψ_{ult} is the permittivity quoted by the manufacturer, RF_{SCB} is the reduction factor for soil clogging and blinding, RF_{CR} is the reduction factor for creep reduction of void space, RF_{IN} is the reduction factor for adjacent materials intruding into the geotextile void space, RF_{CC} is the reduction factor for chemical clogging, and RF_{BC} is the reduction factor for biological clogging.

These reduction factors vary between 1 and 10 depending on the application, and average 4.41 (RF_{SCB}), 1.83 (RF_{CR}), 1.1 (RF_{IN}), 1.25 (RF_{CC}), and 2.2 (RF_{BC}). As can be seen, multiplying all these factors leads to using an allowable permittivity that is a very small fraction of the ultimate value. In addition, a regular factor of safety is applied as follows:

$$F = \frac{\Psi_{allow}}{\Psi_{req}} \tag{25.49}$$

For critical applications, this factor of safety should be as high as 5 to 10.

For filtering, the problem of soil retention occurs when the water flows from a soil with fines into a much coarser soil or an open space. In this case the fines may wash out through the coarser soil and follow the water flow. The filter ensures that the transition from fine-grained to coarse-grained is gradual and lets the water go through while retaining the fines of the soil. The design makes use of the geotextile AOS. (Recall from section 25.3.1 that the AOS is the apparent opening size, defined as the diameter of glass beads corresponding to 95% retained by weight.) Typical AOS values range from 0.01 mm to 0.5 mm. A simple criterion for the opening of the geotextile is (Carroll 1983):

$$O_{95} < 2.5D_{85} \tag{25.50}$$

where O_{95} is the AOS of the geotextile and D_{85} is the particle size corresponding to 85% passing by weight of the soil to be protected. More detailed criteria for geosynthetic filters have been developed (e.g., Luettich et al. 1992; Koerner 2012; Giroud 2010). In particular, Giroud (2010) proposed two new criteria based on porosity and thickness in addition to water conveyance and soil retention. The porosity criterion ensures that the geotextile has enough openings per unit area and makes a clear distinction between woven and nonwoven geotextiles. The thickness criterion recognizes that, unlike granular filters, the opening size of a geotextile filter depends on its thickness.

For drainage, the water flows in the direction of the geosynthetic. The design of a geosynthetic for drainage purposes follows an approach similar to that used in the design for filtering. Instead of permittivity, however, we use transmissivity in this case, defined as:

$$\Theta_{req} = kt = \frac{q}{iw} \tag{25.51}$$

where Θ_{req} is the transmissivity required, k is the in-plane hydraulic conductivity, t is the thickness of the geosynthetic, q is the water flow to be handled, i is the hydraulic gradient, and w is the width of the geosynthetic.

Although geosynthetic have very useful applications in filtering and drainage, they provide limited flow capacity for water conveyance (10^{-8} to 10^{-6} m^3/s per meter of geosynthetic).

25.8 DESIGN FOR EROSION CONTROL

Geosynthetics have been used for decades in the field of erosion control. There are several application domains, including geosynthetic filters under rip rap, geosynthetics to facilitate revegetation, and silt fences.

Filters

Rip rap is often placed to prevent erosion when high water velocities affect the ground surface. Sizing the rip rap consists of finding out the highest water velocity to be handled and choosing the rip-rap size accordingly. Figure 23.8 can be used for rip-rap size selection. In addition, one must check that the rock itself is not degradable over time when subjected to wet-dry cycles. Once the rip rap is chosen, it is very important to place a geosynthetic layer between the soil to be protected and the rip rap. If such a layer is not placed, the soil can erode from underneath the rip rap. The rip rap will not move downstream, but will sink into the soil below and not prevent erosion. The geosynthetic layer has two functions: soil retention to prevent the soil underneath from eroding away, and water conveyance to prevent compression water stresses from developing in the underlying soil. These water stresses would weaken the soil and lead to failure (e.g., slope instability). Therefore, the geosynthetic must be a filter, and geotextiles are best suited for this purpose. The design of the filter follows the same rules as those discussed in section 25.7.

Revegetation

Erosion on the slopes of embankments, dams, levees, and river banks can be minimized by strong and thick vegetation. The problem is that it takes time for appropriate vegetation to grow and become dense and deeply rooted. To help fix the vegetation, geosynthetics can be used. A distinction is made between temporary erosion and revegetation materials (TERMS) and permanent erosion and revegetation materials (PERMS). TERMS are completely biodegradable (hay straws, mulches) or partially biodegradable (hydraulic mulch geofibers, erosion control blankets). PERMS include turf reinforcement mats (TRMs) and vegetated geocellular containment systems (GCSs). The geosynthetics used in PERMS have openings to let the vegetation and roots grow through and some filtering capability. After the seeds are sown, the vegetation grows and gets entangled with the geosynthetic, which provides reinforcement to the root system. Figure 25.18 gives a range of velocities that can be resisted by various forms of

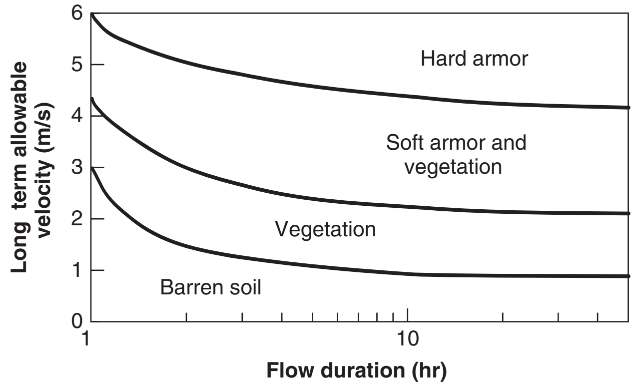


Figure 25.18 Allowable velocities for erosion control measures. (After Theisen 1992.)

armoring, including the soft armor with vegetation discussed here.

Silt Fences

Water flowing on barren soil along roadways or construction sites erodes the soil. To prevent this erosion, silt fences are often placed to let the water go through but stop and collect the silt-size particles that would otherwise flow downstream. Silt fences consist of a geosynthetic (most often woven geotextile) placed above ground by attaching it to vertical posts driven in the soil (Figure 25.19). The silt fence catches and retains the fine soil particles yet lets the water flow through. The water flow is typically quite shallow compared to a river flow, but the velocity can be high on steep slopes. The following design issues must be addressed: maximum length of slope between fences, runoff flow rate, sediment flow rate, height of fence, spacing and strength of fence posts, and geotextile selection. The maximum length of slope L_{max} that can be handled by one silt fence may be estimated by (Koerner 2012):

$$L_{max}(\mathrm{m}) = 36.2\, e^{-11.1 \tan\alpha} \tag{25.52}$$

where α is the slope angle.

If the length of slope to be protected is longer than that, a sequence of silt fences separated by a distance less than L_{max} is used. The runoff flow rate Q is tied to the recurrence interval of the rainfall selected (often taken as the 10-year flow) and is given by:

$$Q\,(\mathrm{m^3/hr}) = C \times I\,(\mathrm{m/hr}) \times A(\mathrm{m^2}) \tag{25.53}$$

where C is a dimensionless coefficient taken as 0.5 for barren soil, I is the rainfall intensity, and A is the drainage area. The weight of soil accumulated per unit area of soil drained and per unit time behind the silt fence can be estimated by using the Uniform Soil Loss Equation (USLE; Wishmeier and Smith 1960):

$$E\ (\mathrm{kN/km^2.yr}) = 10 \times R \times K \times LS \times C \times P \tag{25.54}$$

where R is the dimensionless rainfall coefficient, K is the dimensionless soil erodibility factor, LS is the dimensionless length of slope or gradient factor, C is the dimensionless vegetation cover factor, and P is the dimensionless conservation practice factor. These factors are given in Wishmeier and Smith (1960). The USLE equation has shortcomings and does not apply to channel and gully flow. Nevertheless, it provides a first estimate.

The height H of the silt fence can be calculated by finding out the volume V of water and soil that can be retained by the fence over a 1 m width of fence:

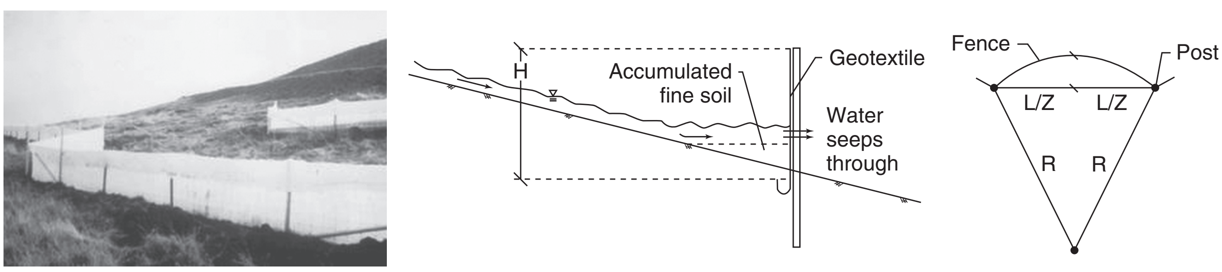
$$V\,(\mathrm{m^3}) = Qt = H\left(\frac{H}{\tan\alpha}\right) \times 1\ \mathrm{m} \tag{25.55}$$

where Q is obtained from Eq. 25.53, t is the duration of the rainstorm, H is the height of the fence, and α is the angle of the slope on which the water flows.

Silt fences are usually 0.3 to 0.9 m high. Then the spacing of the posts retaining the fence is chosen. This spacing is usually between 1 and 3 m. The load on the fence due to the water pressure can then be calculated to obtain the lateral load and maximum bending moment on the posts. This bending moment is in the range of 5 to 30 kN.m. The last step is to calculate the tensile load in the fence material. If it is assumed that the fence deflects an amount s in an arc of circle under the average water pressure of $0.5\gamma_w$H behind the fence, the tension in the geosynthetic is given by the following equation:

$$T = \frac{\gamma_w H L^2}{16s\left(\dfrac{Arc\sin\dfrac{4s}{L}}{\dfrac{4s}{L}}\right)} \quad \text{or } T = \frac{\gamma_w H L^2}{16s} \text{ if 4s/L is small} \tag{25.56}$$

where T is the tension per meter of geotextile, γ_w is the unit weight of water, H is the height of the fence, L is the distance between posts, and s is the horizontal deflection of the fence at midspan (Figure 25.19). Typical values of T range from 5 to 30 kN/m.



(a) Silt fences (b) Cross section (c) Plan view

Figure 25.19 Silt fences. (a: Courtesy of Robert Koerner, 2012)

25.9 OTHER DESIGN APPLICATIONS

25.9.1 Lightweight Fills

Lightweight fills are most commonly built of geofoam blocks (Horvath 1994; Saye et al. 2000). The unit weight of the geofoam blocks is at most 10% of the unit weight of soil. Therefore, the pressure on the native soil and the associated settlement can be reduced significantly. Note that a pavement layer still has to be constructed with heavier materials (granular base course for drainage and asphalt rolling layer) on top of a geofoam embankment. Of course, the compression of the geofoam must be added to the settlement of the soil below, but considering the typical application (embankment on soft soils), that compression is most of the time negligible compared to the settlement of the soil below. Overall, 80% reduction in settlement is not uncommon.

25.9.2 Compressible Inclusions

Another notable application of compressible inclusions is the case of geofoam blocks behind retaining walls to decrease the earth pressure. This type of solution is particularly useful for walls that cannot tolerate much lateral deflection without damaging the structure. This is the case of basement walls and bridge abutments in shrink-swell soil areas. The pressure-absorbing layer may be 50 to 600 mm thick and tends to decrease the pressure as shown in Figure 25.20. The thicker the geofoam layer is, the lower the pressure is likely to be. Note also that the pressure distribution is altered toward the bottom of the wall where the geofoam is most effective. Another application is to mitigate seismically induced pressures (Athanasopoulos 2007).

25.9.3 Thermal Insulation

Geofoams are among the best temperature insulators. Recall from Chapter 16, Eq. 16.8 that the R factor of an insulator is a measure of the resistance to temperature propagation and is given by:

$$R = \frac{dx}{k_t} \tag{25.57}$$

where dx is the thickness of the insulating layer in meters and k_t is the thermal conductivity of the insulating material in watts per degree Kelvin per meter (W/K.m). Therefore, the R rating is expressed in m^2.K/W. Because the degree Kelvin is equal to the degree Celsius, the R rating has the same value in m^2.K/W and in m^2.C/W.

Table 25.5 shows that geofoam blocks have some of the highest R ratings of any materials. Within the geofoam range of R values, extruded polystyrene (XPS) has higher R values than expanded polystyrene (EPS). Styrofoam coffee cups are made of XPS. Once the R factor is known, the heat flow can be calculated:

$$\frac{dQ}{dt} = k_t A \frac{dT}{dx} = \frac{A}{R} dT \tag{25.58}$$

where dQ is the amount of heat (J) flowing in a time dt (s), k_t is the thermal conductivity (J/s.K.m or W/K.m), A is the area perpendicular to the heat flow (m^2), dT is the change in temperature (K), dx is the length over which the change of temperature is occurring (m), and R is the thermal resistance or R factor (m^2.K/W).

The applications include insulation under a house on permafrost to avoid ground thawing, or under a refrigerated building to avoid ground freezing.

25.9.4 Geosynthetics and Landfill Slopes

Modern landfill liners are made of many different layers, including geosynthetics. These geosynthetic layers, particularly GCLs, can represent planes of lower shear strength where slope failure can develop. This issue should be addressed at the time the liner is designed, together with a plan and possible restrictions on where and how high the waste can be piled up at one location. This topic is addressed in Chapter 26.

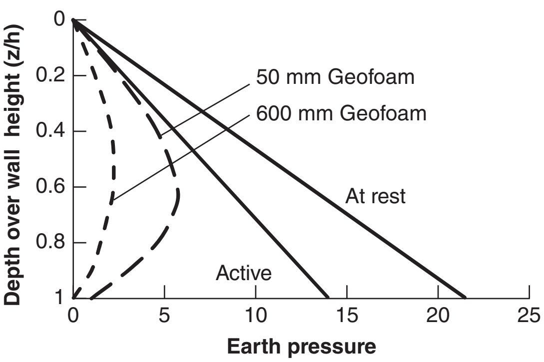


Figure 25.20 Decrease in earth pressure by compressible inclusions. (After Horvath 1997)

Table 25.5 R Factor for Various Materials

Material	R Factor (M^2.K/W or M^2.C/W)
Steel	0.022
Ice	0.45
Concrete	0.95
Glass	1.25
Water (25°C)	1.64
Glass wool	23.8
Air (25°C)	38.5
Geofoam blocks	25 to 40

(After Koerner 2012)

PROBLEMS

25.1 A geosynthetic is placed on the ground surface and stones are to be placed on top of it. The maximum diameter of the stones is 60 mm and the drop height from the truck is 1.5 m. The soil below the geosynthetic is medium stiff with a soil support reduction factor of $F_s = 15$; the cumulative reduction factor for the geosynthetic is $F_r = 5$. What is the impact strength required of the geosynthetic to safely handle the impact loading?

25.2 A 0.5 m thick layer of base course has been placed on top of a geotextile. Trucks with tire pressures equal to 600 kPa will travel on top of the base course during construction. The stones are 60 mm in diameter and fairly sharp, such that the product $S_1S_2S_3$ in Eq. 25.24 is equal to 0.3. If the geotextile strength reduction factor is 4.5 (Eq. 25.39), what is the required ultimate strength of the geotextile to safely avoid puncture?

25.3 A landfill owner is considering replacing a 1 m thick layer of compacted clay with a 15 mm thick GCL as part of the design of a new landfill liner. The landfill has an area of 7.5 hectares and the fee collected per cubic meter of waste is \$90. How much additional income does the owner stand to collect from the saving in the thickness of the liner?

25.4 A geosynthetic clay liner and a compacted clay liner are being compared. The GCL is 15 mm thick and has a hydraulic conductivity of 10^{-11} m/s; the CCL is 500 mm thick and has a hydraulic conductivity of 10^{-9} m/s. The water level is 1 m above the top of the liner and the pressure head is assumed to be zero on the bottom side of the liner. Calculate the amount of water going through the GCL and the CCL.

25.5 A geosynthetic clay liner has a bentonite clay layer with the following shear strength characteristics: $c' = 0$ and $\varphi' = 10^\circ$. It is placed on the side slope of a landfill that has an 18° angle with the horizontal. The plan is to cover the GCL uniformly with 20 m of waste weighing 10 kN/m^3. What cohesion c' must be developed by needle-punching in the GCL to have a factor of safety of 1.5 against failure in the bentonite? Use the infinite slope equation from section 19.3.

25.6 Design a 6 m high geosynthetic-reinforced MSE wall. The vertical spacing between geosynthetic layers is 0.5 m, the backfill is sand with a unit weight of 20 kN/m^3 and a friction angle of 34°, a surcharge of 20 kN/m^2 is applied on the ground surface at the top of the wall, and the geosynthetic is a geogrid. The soil on which the wall is being built is a very stiff clay with an undrained shear strength s_u equal to 100 kN/m^2 and a friction angle of 25°. The soil behind the wall is a sandy clay with a unit weight of 20 kN/m^3 and a friction angle of 30°. Assume reasonable values for all other parameters needed for the design.

25.7 A layer of geotextile is placed 1 m below a 2 m wide strip footing. The footing rests on the surface of a loose sand with a friction angle equal to 30° and a pressuremeter limit pressure of 500 kPa within a depth equal to one footing width below the footing. The geogrid has an ultimate tensile strength of 100 kN/m.

a. Calculate the percent increase in ultimate bearing capacity between the case of no geogrid and the case with geogrid.
b. If the geogrid has a global friction factor K (Eq. 25.7) of 0.3, what length of geogrid is required to safely anchor the geogrid on each side of the footing?

25.8 A 30 m wide, 7 m high embankment is placed on soft clay with a geotextile between the surface of the soft clay and the embankment fill. The purpose of the geotextile is to increase the bearing capacity and reduce the settlement reduction, but it is also used for separation, drainage, and filtering. The bottom of the embankment settles along an arc of circle with 1 m of settlement at the center and a negligible amount at the edges. What will be the tension load in the geotextile if its modulus is 500 kN/m?

25.9 A geotextile has an ultimate tensile strength of 100 kN/m and a maximum flow rate capacity of 8×10^{-7} m^3/s per meter of geotextile. What are reasonable values of the allowable tensile strength and allowable flow rate for this geotextile?

25.10 A construction site has a 30 m long erodible slope with an angle of 6°. Silt fences are required.

a. How many silt fences are needed?
b. If the 10-year rainstorm generates 100 mm/hr, what is the flow rate to be handled per meter of width of the fence?
c. Calculate the height of the fence so that it can safely handle two 10-year storms each lasting 3 hours.
d. Posts are placed every 3 m and the fence is allowed to deflect 0.2 m at its center. Estimate the tension in the fence fabric.

25.11 Derive Eq. 25.56 for silt fences.

25.12 A 7 m high, 60 m wide embankment is to be built on a layer of soft clay with a water table at the ground surface. The soft clay is 5 m thick and the increase in stress in the clay layer can be taken as the pressure under the embankment because the clay layer is thin compared to the width of the embankment. The clay layer has the following consolidation characteristics: $e_o = 1.1$, $\gamma = 19$ kN/m^3, $C_c = 0.5$. Two options are considered for the embankment fill: soil fill and geofoam fill. The soil fill has a unit weight of 20 kN/m^3 and the geofoam fill 2 kN/m^3. What will be the settlement of the embankment in each case? Which fill type will have the shortest time to reach 90% consolidation?

25.13 A building refrigerated at -5°C is being designed on a soil with a high water table. The concern is the cost of the power (watts) to maintain the difference in temperature across the foundation. Two alternatives are considered. The first consists of a relatively inexpensive 100 mm thick concrete slab on grade on top of the soil. The second one consists of the same slab on grade on top of a 150 mm thick geofoam. The concrete has a thermal resistance R value equal to 0.9 m^2.C/W per meter of thickness and the geofoam 35 m^2.C/W per meter of thickness. Calculate the amount of power required in each case to maintain the difference in temperature at -5°C above the slab and 0°C on top of the soil.

25.14 A Styrofoam coffee cup holds coffee at 80°C. Your hand holding the coffee cup is at 30°C. Assuming a steady-state heat transfer in the cup wall, what is the R rating per meter of the Styrofoam if the amount of heat released from the coffee cup through the wall of the cup is 45 W? What is the thermal conductivity of the Styrofoam if the cup wall is 1.5 mm thick?

Problems and Solutions

Problem 25.1

A geosynthetic is placed on the ground surface and stones are to be placed on top of it. The maximum diameter of the stones is 60 mm and the drop height from the truck is 1.5 m. The soil below the geosynthetic is medium stiff with a soil support reduction factor of $F_s = 15$; the cumulative reduction factor for the geosynthetic is $F_r = 5$. What is the impact strength required of the geosynthetic to safely handle the impact loading?

Solution 25.1

We assume that the stone has a unit weight of 26 kN/m^3:

$$E_{stone} = Wh = \frac{\pi d^3}{6}\gamma h = \frac{\pi \times 0.06^3}{6} \times 26000 \times 1.5 = 4.41 \text{ J}$$

$$\frac{E_{stone}}{F_{soil}} = \frac{4.41}{15} = 0.294$$

We must satisfy:

$$\frac{E_{stone}}{F_{soil}} \leq \frac{E_{geosyn}}{F_{reduc}}$$

$$E_{geosyn} \geq 0.294 \times 5 = 1.47 \text{ J}$$

The geosynthetic must have an impact strength at least equal to 1.47 J.

Problem 25.2

A 0.5 m thick layer of base course has been placed on top of a geotextile. Trucks with tire pressures equal to 600 kPa will travel on top of the base course during construction. The stones are 60 mm in diameter and fairly sharp, such that the product $S_1S_2S_3$ in Eq. 25.24 is equal to 0.3. If the geotextile strength reduction factor is 4.5 (Eq. 25.39), what is the required ultimate strength of the geotextile to safely avoid puncture?

Solution 25.2

$$F_{stone} = p\, d_a{}^2 S_1 S_2 S_3 = (600 + 0.5 \times 20) \times 0.06^2 \times 0.3 = 0.659 \text{ kN}$$

$$S_t = \frac{P}{\pi d} = \frac{0.659}{0.06\pi} = 3.5\ \frac{\text{kN}}{\text{m}}$$

$$T_{ultimate} = T_{allowable} \times RF = 3.5 \times 4.5 = 15.8\ \frac{\text{kN}}{\text{m}}$$

In this situation, a geotextile rated at 15.8 kN/m is needed.

Problem 25.3

A landfill owner is considering replacing a 1 m thick layer of compacted clay with a 15 mm thick GCL as part of the design of a new landfill liner. The landfill has an area of 7.5 hectares and the fee collected per cubic meter of waste is \$90. How much additional income does the owner stand to collect from the saving in the thickness of the liner?

Solution 25.3

The change in height after replacing the compacted clay layer with the GCL:

$$\begin{aligned}\Delta H &= 1 - 0.015\\ &= 0.985 \text{ m}\end{aligned}$$

For a landfill area of 7.5 hectares and a fee of \$90 per m^3:

$$\begin{aligned}\text{Additional income per hectare} &= 90 \times 10{,}000 \times 0.985\\ &= \$886{,}500/\text{ha}\\ \text{Total income} &= \frac{\$886{,}500}{\text{ha}} \times 7.5 \text{ ha} = \$6{,}648{,}750\end{aligned}$$

Problem 25.4

A geosynthetic clay liner and a compacted clay liner are being compared. The GCL is 15 mm thick and has a hydraulic conductivity of 10^{-11} m/s; the CCL is 500 mm thick and has a hydraulic conductivity of 10^{-9} m/s. The water level is 1 m above the top of the liner and the pressure head is assumed to be zero on the bottom side of the liner. Calculate the amount of water going through the GCL and the CCL.

Solution 25.4

Using Darcy's law, and assuming that the hydraulic gradient is the total head divided by the thickness of the GCL and a flow through a unit area, the amount of water through the GCL is:

$$\begin{aligned}q &= kiA\\ &= (1 \times 10^{-11})\left(\frac{1.015}{0.015}\right)(1 \times 1)\\ q &= 7 \times 10^{-10} \text{ m}^3/\text{s}\end{aligned}$$

Using the same procedure, the flow rate through the CCL is:

$$\begin{aligned}q &= kiA\\ &= (1 \times 10^{-9})\left(\frac{1.500}{0.500}\right)(1 \times 1)\\ q &= 3 \times 10^{-9} \text{ m}^3/\text{s}\end{aligned}$$

Problem 25.5

A geosynthetic clay liner has a bentonite clay layer with the following shear strength characteristics: $c' = 0$ and $\varphi' = 10^{\circ}$. It is placed on the side slope of a landfill that has an 18° angle with the horizontal. The plan is to cover the GCL uniformly with 20 m of waste weighing 10 kN/m^3. What cohesion c' must be developed by needle-punching in the GCL to have a factor of safety of 1.5 against failure in the bentonite? Use the infinite slope equation from section 19.3.

Solution 25.5

Figure 25.1s shows the illustration of the infinite slope. Note that W is the weight of the wedge, T is the shear force, and N is the normal force. H is the height of the waste, L is the length of the wedge, and β is the inclination of the slope.

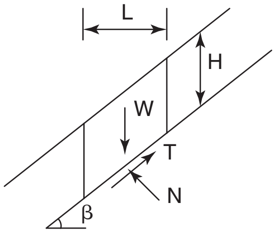


Figure 25.1s Illustration of infinite slope.

Based on the equilibrium condition and the definition of factor of safety, FS can be calculated:

$$FS = \frac{\tan\varphi'}{\tan\beta} + \frac{c'}{\gamma H \sin\beta\cos\beta}$$

Here, γ is the unit weight of the waste and φ' is the friction angle.

To achieve a factor of safety of 1.5, the cohesion developed by needle-punching has to satisfy the following equation:

$$1.5 = \frac{\tan 10^\circ}{\tan 18^\circ} + \frac{c'}{10 \times 20 \times \sin 18^\circ \cos 18^\circ}$$

Therefore, the cohesion developed by needle-punching must be $c' = 56\,\text{kPa}$.

Problem 25.6

Design a 6 m high geosynthetic-reinforced MSE wall. The vertical spacing between geosynthetic layers is 0.5 m, the backfill is sand with a unit weight of 20 kN/m^3 and a friction angle of 34^o, a surcharge of 20 kN/m^2 is applied on the ground surface at the top of the wall, and the geosynthetic is a geogrid. The soil on which the wall is being built is a very stiff clay with an undrained shear strength s_u equal to 100 kN/m^2 and a friction angle of 25^o. The soil behind the wall is a sandy clay with a unit weight of 20 kN/m^3 and a friction angle of 30^o. Assume reasonable values for all other parameters needed for the design.

Solution 25.6 (Figure 25.2s)

$$\text{H} = 6\text{ m}$$

$$\text{S}_v = 0.5\text{ m}$$

$$\Delta\sigma_v = 20\text{ kN/m}^2$$

Use a minimum reinforcement length $L = 4.2$ m as the length-to-height ratio of the reinforced wall (should be no less than 0.7).

Assume that the first layer of geosynthetics is placed at 0.25 m from the finished grade.

Consider the ultimate strength resistance of the geosynthetic (T_{ult}) as 170 kN/m.

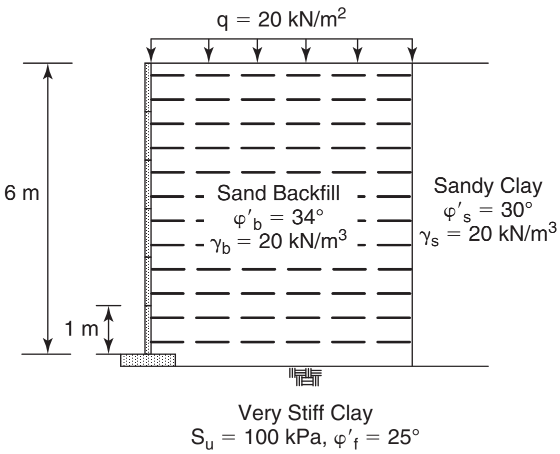


Figure 25.2s Retaining wall.

a. External stability, earth pressures.

$$k_a = \frac{1 - \sin\varphi_b}{1 + \sin\varphi_b} = \frac{1 - \sin 30}{1 + \sin 30} = 0.33$$

Then the active load generated by the horizontal soil pressure P_{a1} and the traffic surcharge P_{a2} can be computed as:

$$P_{a1} = \frac{K_a \times \gamma_s \times H^2}{2} = \frac{0.333 \times 20 \times (6)^2}{2} = 120\ \text{kN/m}$$

- Located 2 m above the bottom of the wall ($x_{a1} = 2$ m, as shown in Figure 25.3s).

$$P_{a2} = K_a \times q \times H = 0.333 \times 20 \times 6 = 40\ \text{kN/m}$$

- Located 3 m above the bottom of the wall ($x_{a2} = 3$ m as shown in Figure 25.3s).

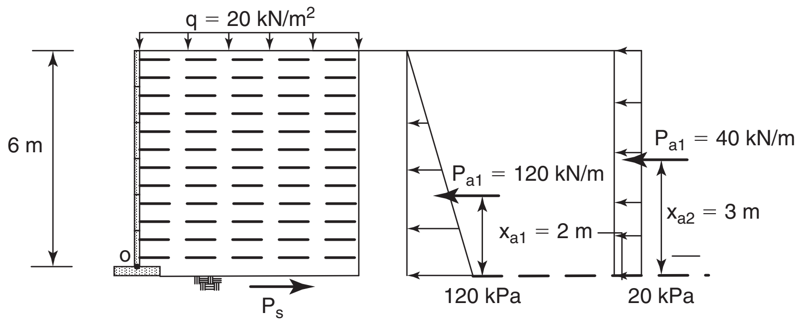


Figure 25.3s Pressure diagram on retaining walls.

We can now calculate the sliding and overturning stability (ignoring the traffic surcharge).

b. External stability, sliding analysis.
   Using the LRFD approach and no traffic surcharge:

$$\varphi W \tan\varphi'_f \geq \gamma P_{a1} \quad \text{or} \quad \varphi\gamma_b HL \tan\varphi'_f \geq \gamma P_{a1}$$

Using φ as 0.85, γ as 1.25, and L as 4.2 m, we have:

$$0.85 \times 20 \times 6 \times 4.2 \times \tan 25 \geq 1.25 \times 120$$
$$428.4\ \text{kN/m} \geq 150\ \text{kN/m} \quad \therefore \text{OK}$$

Using the LRFD approach and the traffic surcharge:

$$\varphi W \tan\varphi'_f \geq \gamma_1 P_{a1} + \gamma_2 P_{a2} \quad \text{or} \quad \varphi\gamma_b HL \tan\varphi'_f \geq \gamma P_{a1}$$

Using φ as 0.85, γ as 1.25 for the dead load and γ as 1.75 for the live load, and L as 4.2 m, we have:

$$0.85 \times 20 \times 6 \times 4.2 \times \tan 25 \geq 1.25 \times 120 + 1.75 \times 40$$
$$428.4\ \text{kN/m} \geq 220\ \text{kN/m} \quad \therefore \text{OK}$$

c. External stability, overturning analysis.
Overturning around the toe (point O) of the wall with no traffic surcharge:

$$\frac{\varphi WL}{2} \geq \frac{\gamma P_{a1} H}{3} \quad \text{or} \quad \frac{\varphi \gamma_b H L^2}{2} \geq \frac{\gamma P_{a1} H}{3} \quad \text{or} \quad \frac{0.85 \times 20 \times 6 \times 4.2^2}{2} \geq \frac{1.25 \times 120 \times 6}{3}$$
$$899.6 \text{ kN} \geq 300 \text{ kN} \quad \therefore \text{OK}$$

Overturning around the toe (point O) of the wall with traffic surcharge:

$$\frac{\varphi (W + qL)L}{2} \geq \frac{\gamma_1 P_{a1} H}{3} + \frac{\gamma_2 P_{a2} H}{2} \quad \text{or}$$
$$\frac{0.85(20 \times 6 \times 4.2 + 20 \times 4.2)4.2}{2} \geq \frac{1.25 \times 120 \times 6}{3} + \frac{1.75 \times 40 \times 6}{2}$$
$$1076 \text{ kN} \geq 510 \text{ kN} \quad \therefore \text{OK}$$

d. External stability, bearing capacity analysis.
The eccentricity of the wall applied forces can be calculated as:

$$W \times e + q \times L \times e = M_{ov} \quad \text{or} \quad (W + q \times L) \times e = P_{a1} \times x_{a1} + P_{a2} \times x_{a2}$$
$$e = \frac{P_{a1} \times x_{a1} + P_{a2} \times x_{a2}}{(W + q \times L)} = \frac{120 \times 2 + 40 \times 3}{(20 \times 6 \times 4.2 + 20 \times 4.2)} = 0.61 \text{ m}$$

This eccentricity cannot be outside of the central one-third of the footing, which is:

$$e \leq \frac{L}{6} = \frac{4.2}{6} = 0.7 \text{ m} \quad \therefore \text{OK}$$

This means that there is no tension underneath the footing.
The active length, according to Meyerhof's distribution, is:

$$L_{active} = L - 2 \times e = 4.2 - 2 \times 0.61 = 2.98 \text{ m}$$

The bearing pressure is:

$$p = (\gamma_b \times H + q) \times \frac{L}{L_{active}} = (20 \times 6 + 20) \times \frac{4.2}{2.98} = (169.1)_{weight} + (28.2)_{traffic} = 197.3 \text{ kPa}$$

The bearing capacity of the existing soil can be calculated according to the Skempton chart:

$$q_u = N_c S_u + \gamma_b D = 7.5 \times 100 = 750 \text{ kPa}$$

Checking for bearing capacity failure:

$$\varphi \times q_{bc} \geq \gamma_1 p_1 + \gamma_2 p_2 \quad \text{or} \quad 0.5 \times 750 \geq 1.25 \times 169.1 + 1.75 \times 28.2$$
$$375 \text{ kPa} \geq 260.7 \text{ kPa} \quad \therefore \text{OK}$$

e. Internal stability, pull-out failure.
No traffic surcharge:

$$T_{\max} = s_v \sigma_{ah} = s_v k_a \sigma'_v$$
$$k_a = \frac{1 - \sin \varphi_r}{1 + \sin \varphi_r} = \frac{1 - \sin 34}{1 + \sin 34} = 0.283$$

Note that for an MSE wall built with geosynthetics, the k_r and k_a ratio are the same according to AASHTO LRFD (Figure 25.4s).

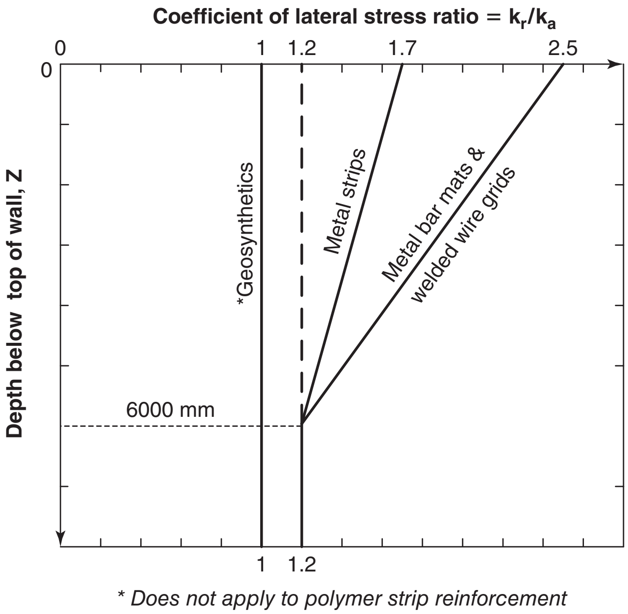


Figure 25.4s Coefficient of lateral stress ratio.

The results of T_{max} at different heights are shown in Table 25.1s. T_{max1} is due to the soil weight ($s_v\ \sigma_{ah}$, active earth pressure) and T_{max2} is due to the traffic surcharge ($s_v\ k_a \times 20\text{kN/m}^2$).

Table 25.1s Summary of Calculation of T_{max}

Layer No.	Depth (m)	k_a	k_r	σ_v (kPa)	σ_{ah} (kPa)	T_{max1} (kN/m)	T_{max2} (kN/m)	$T_{max\text{-}total}$ (kN/m)
1	0.25	0.283	0.283	5.0	1.414	0.71	2.83	3.53
2	0.75	0.283	0.283	15.0	4.241	2.12	2.83	4.95
3	1.25	0.283	0.283	25.0	7.068	3.53	2.83	6.36
4	1.75	0.283	0.283	35.0	9.895	4.95	2.83	7.77
5	2.25	0.283	0.283	45.0	12.722	6.36	2.83	9.19
6	2.75	0.283	0.283	55.0	15.549	7.77	2.83	10.60
7	3.25	0.283	0.283	65.0	18.376	9.19	2.83	12.02
8	3.75	0.283	0.283	75.0	21.204	10.60	2.83	13.43
9	4.25	0.283	0.283	85.0	24.031	12.02	2.83	14.84
10	4.75	0.283	0.283	95.0	26.858	13.43	2.83	16.26
11	5.25	0.283	0.283	105.0	29.685	14.84	2.83	17.67
12	5.75	0.283	0.283	115.0	32.512	16.26	2.83	19.08

Using the ultimate limit state procedure, we have:

$$\gamma_1 T_{max\,1} + \gamma_2 T_{max\,2} = \phi T_{pullout}$$

The active length of the reinforcement strip required to resist the pull-out load is:

$$T_{pullout} = \frac{\gamma_1 T_{\max 1} + \gamma_2 T_{\max 2}}{\varphi}$$

$$L_a = \frac{T_{pullout}}{2 \times f_{\max} \times b} = \frac{(\gamma_1 \sigma'_{ah} \times s_v) + (\gamma_2 k_a q \times s_v)}{2 \times \varphi \times \sigma'_v \times \tan\delta}$$

$$L_{\max} = (H - z) \times \tan\left(45 - \frac{\varphi}{2}\right)$$

$$L_{total} = L_{\max} + L_a = (H - z) \times \tan\left(45 - \frac{\varphi}{2}\right) + \frac{(\gamma_1 \sigma'_{ah} \times s_v) + (\gamma_2 k_a q \times s_v)}{2 \times \varphi \times \sigma'_v \times \tan\delta}$$

However, in construction practice, the length is often taken as constant throughout the height of the wall. The longest value of L_{total} is at the top of the wall ($z = 0$). Then:

$$L_{total} = H \times \tan\left(45 - \frac{\varphi}{2}\right) + \frac{(\gamma_1 \sigma'_{ah} \times s_v) + (\gamma_2 k_a q \times s_v)}{2 \times \varphi \times \sigma'_v \times \tan\delta}$$

The resistance (φ) and load factor (γ) are taken as 0.9 and 1.5, respectively. The coefficient of friction ($\tan\delta$) is computed according to AASHTO LRFD using Figure 25.5s. Based on this figure, the friction factor is equal to the tangent of the friction angle of the reinforced backfill (φ_r). Therefore:

$$F^* = \tan\delta = \tan\varphi_b = 0.6745$$

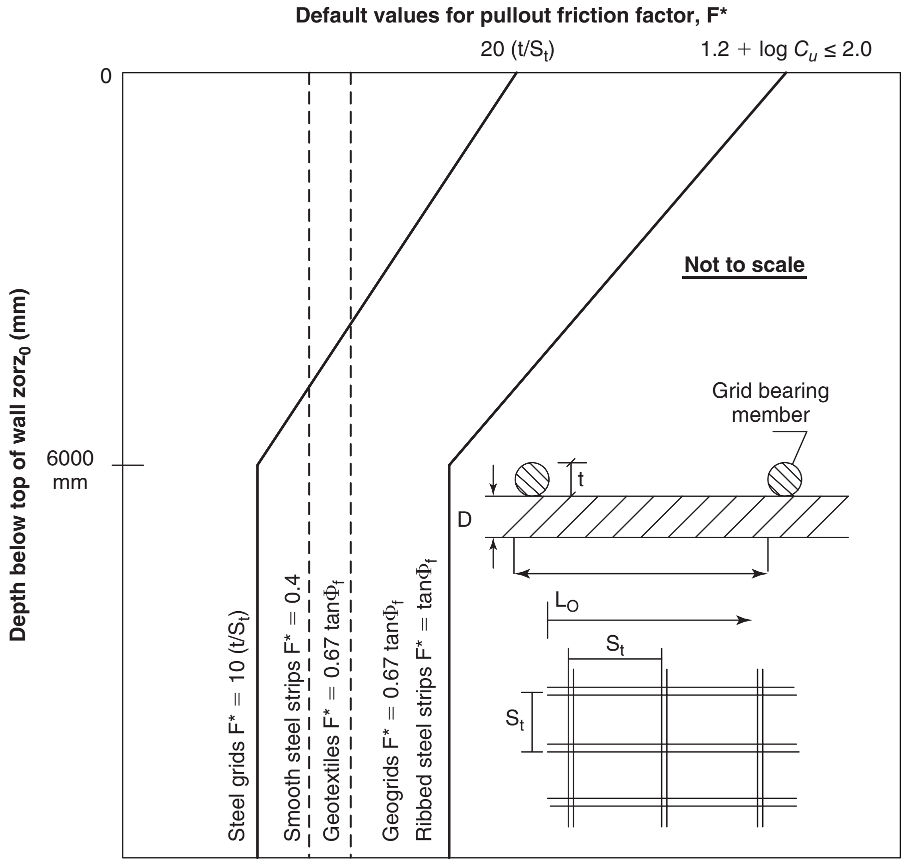


Figure 25.5s Default values for pull-out friction factor.

$$L_{total} = H \times \tan\left(45 - \frac{\varphi}{2}\right) + \frac{(\gamma_1 \sigma'_{ah} \times s_v) + (\gamma_2 k_a q \times s_v)}{2 \times \varphi \times \sigma'_v \times \tan\delta}$$

$$L_{total} = 6 \times \tan\left(45 - \frac{34}{2}\right) + \frac{(1.5 \times 0.283 \times 20 \times 0.25 \times 0.5) + (1.75 \times 0.283 \times 20 \times 0.5)}{2 \times 0.9 \times 5 \times 0.6745}$$

$$L_{total} = 3.19 \text{ m} + 0.17 \text{ m} + 0.81 \text{ m} = 4.18 \text{ m}$$

However, the required length of reinforced soil mass is 0.7 H or 4.2 m.

f. Internal stability, yield of reinforcement.

No traffic surcharge:

$$T_{allow} = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_{CBD}}$$

Consider RF_{ID}, RF_{CR}, and RF_{CBD} as 1.55, 2.15, and 1.32, respectively:

$$T_{allow} = \frac{T_{ult}}{1.55 \times 2.15 \times 1.32} = \frac{T_{ult}}{4.4} = \frac{170 \text{ kN/m}}{4.4} = 38.6 \text{ kN/m}$$

$$\varphi\, T_{allow} = 34.7 \text{ kN/m}$$

Using the ultimate limit state analysis, we have:

$$\gamma_1 T_{\max 1} + \gamma_2 T_{\max 2} = \phi T_{allow}$$

$$T_{allow} = \frac{\gamma_1 T_{\max 1} + \gamma_2 T_{\max 2}}{\varphi}$$

$$T_{allow} \geq \frac{1.5 \times s_v \times \sigma_{ah}}{0.9} + \frac{1.75 \times s_v \times k_a q}{0.9}$$

The maximum horizontal strength required is at the bottom of the wall, so we will check that layer of soil reinforcement:

$$T_{allow} \geq \frac{1.5 \times 0.5 \times 32.5}{0.9} + \frac{1.75 \times 0.5 \times 0.283 \times 20}{0.9} \quad \text{or} \quad T_{allow} \geq 27 + 5.5 = 32.5 \text{ kN/m}$$

$$T_{allow} = 38.6 \text{ kN/m} > 32.5 \text{ kN/m}$$

$$\varphi T_{allow} = 34.7 \text{ kN/m} > \gamma T_{\max\text{-}total} = 29.3 \text{ kN/m}$$

Here we compare either the factored or the unfactored resistance to the factored or unfactored loads. Detail calculations are shown in Table 25.2s.

Table 25.2s Summary of Calculation for Strength

Layer No.	Depth (m)	$T_{max\text{-}total}$ (kN/m)	L_{total} (m)	T_{ult} (kN/m)	T_{allow} (kN/m)	φT_{allow} (kN/m)	$\gamma T_{max\text{-}total}$	Check
1	0.25	3.53	4.18	170.0	38.6	34.8	6.0	OK
2	0.75	4.95	3.64	170.0	38.6	34.8	8.1	OK
3	1.25	6.36	3.53	170.0	38.6	34.8	10.2	OK
4	1.75	7.77	3.48	170.0	38.6	34.8	12.4	OK
5	2.25	9.19	3.46	170.0	38.6	34.8	14.5	OK
6	2.75	10.60	3.44	170.0	38.6	34.8	16.6	OK
7	3.25	12.02	3.43	170.0	38.6	34.8	18.7	OK
8	3.75	13.43	3.42	170.0	38.6	34.8	20.9	OK
9	4.25	14.84	3.41	170.0	38.6	34.8	23.0	OK
10	4.75	16.26	3.41	170.0	38.6	34.8	25.1	OK
11	5.25	17.67	3.40	170.0	38.6	34.8	27.2	OK
12	5.75	19.08	3.40	170.0	38.6	34.8	29.3	OK

Problem 25.7

A layer of geotextile is placed 1 m below a 2 m wide strip footing. The footing rests on the surface of a loose sand with a friction angle equal to 30° and a pressuremeter limit pressure of 500 kPa within a depth equal to one footing width below the footing. The geogrid has an ultimate tensile strength of 100 kN/m.

a. Calculate the percent increase in ultimate bearing capacity between the case of no geogrid and the case with geogrid.
b. If the geogrid has a global friction factor K (Eq. 25.7) of 0.3, what length of geogrid is required to safely anchor the geogrid on each side of the footing?

Solution 25.7

Figure 25.6s shows the foundation without geogrid. The ultimate bearing capacity of the foundation without geogrid is equal to p_L:

$$p_{u1} = p_L$$

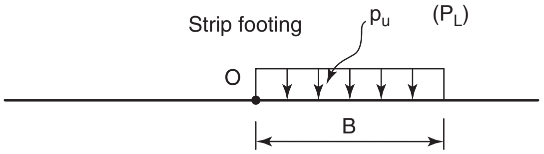


Figure 25.6s Illustration of foundation failure without geogrid.

Figure 25.7s shows the foundation with geogrid. The bearing capacity is calculated as:

$$p_{u2} = p_L + \frac{2T}{B}$$

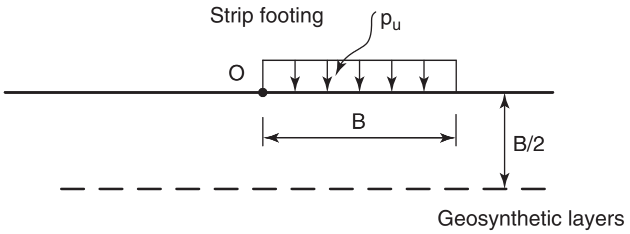


Figure 25.7s Illustration of foundation failure with geogrid.

In this problem, $p_L = 500$ kPa, B = 2 m, and T = 100 kN/m; hence:

$$p_{u1} = P_L = 500 \text{ kPa}$$
$$p_{u2} = P_L + \frac{2T}{B} = 500 + \frac{2 \times 100}{2} = 600 \text{ kPa}$$

Using the geogrid improves the ultimate bearing capacity by 20%.
The required length of geogrid can be calculated:

$$F_{us} = 2L_e K \sigma'_v \tan\phi'$$

Assume that the soil unit weight is 20 kN/m^3. The geogrid is buried 1 m beneath the foundation; therefore:

$$\sigma'_v = \gamma h = 20 \times 1 = 20 \text{ kPa}$$
$$L_e = \frac{F_{us}}{2K\sigma'_v \tan\phi'} = \frac{100}{2 \times 0.3 \times 20 \times \tan 30^\circ} = 14.4 \text{ m}$$

So, the length of geogrid required to safely anchor the geogrid on each side of the footing is 14.4 m.

Problem 25.8

A 30 m wide, 7 m high embankment is placed on soft clay with a geotextile between the surface of the soft clay and the embankment fill. The purpose of the geotextile is to increase the bearing capacity and reduce the settlement reduction, but it is also used for separation, drainage, and filtering. The bottom of the embankment settles along an arc of circle with 1 m of settlement at the center and a negligible amount at the edges. What will be the tension load in the geotextile if its modulus is 500 kN/m?

Solution 25.8

$$\text{Modulus, } E = 500 \text{ kN/m}$$

$$\text{Settlement, } s = 1 \text{ m}$$

$$\text{Width of embankment, } L = 30 \text{ m}$$

Radius R of the circle is:

$$R = \frac{L^2}{8s} = \frac{30^2}{8 \times 1} = 112.5 \text{ m}$$

Deformed length L′:

$$L' = 2RArc\sin\frac{L}{2R} = 30.09 \text{ m}$$

Strain ε in the geosynthetic:

$$\varepsilon = \frac{Arc\sin\left(\frac{4s}{L}\right)}{\frac{4s}{L}} - 1 = \frac{L' - L}{L} = \frac{30.09 - 30}{30} = 0.003$$

Tension T equal to:

$$T = E\varepsilon = 500 \times 0.003 = 1.5 \text{ kN/m}$$

Problem 25.9

A geotextile has an ultimate tensile strength of 100 kN/m and a maximum flow rate capacity of 8×10^{-7} m^3/s per meter of geotextile. What are reasonable values of the allowable tensile strength and allowable flow rate for this geotextile?

Solution 25.9

$$\text{Ultimate tensile strength, } T_{ult} = 100 \text{ kN/m}$$

$$\text{Maximum flow rate, } q_{ult} = 8 \times 10^{-7} \text{m}^3/\text{s per meter of geotextile.}$$

The strength reduction factors take into account installation damage ID, creep CR, and chemical and biological degradation CBD. They are RFID, RFCR, and RFCBD. They average respectively 1.55, 2.15, and 1.32:

$$T_{allow} = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_{CBD}} = \frac{100}{1.55 \times 2.15 \times 1.32} = 23.05 \text{ kN/m}$$

The flow reduction factors take into account soil clogging and blinding, creep reduction of void space, adjacent materials intruding into the geotextile void space, chemical clogging, and biological clogging. They are RF_{SCB}, RF_{CR}, RF_{IN}, RF_{CC}, and RF_{BC}. Their respective average values are: 4.41 (RF_{SCB}), 1.83 (RF_{CR}), 1.1 (RF_{IN}), 1.25 (RF_{CC}), and 2.2 (RF_{BC}).

$$q_{allow} = \frac{q_{ult}}{(RF_{SCB} \times RF_{CR} \times RF_{IN} \times RF_{CC} \times RF_{BC})} = \frac{8 \times 10^{-7}}{4.41 \times 1.83 \times 1.1 \times 1.25 \times 2.2} = \frac{8 \times 10^{-7}}{24.41}$$
$$= 0.33 \times 10^{-7} \text{m}^3/\text{s per meter of geotextile}$$

Problem 25.10

A construction site has a 30 m long erodible slope with an angle of 6°. Silt fences are required.

a. How many silt fences are needed?
b. If the 10-year rainstorm generates 100 mm/hr, what is the flow rate to be handled per meter of width of the fence?
c. Calculate the height of the fence so that it can safely handle two 10-year storms each lasting 3 hours.
d. Posts are placed every 3 m and the fence is allowed to deflect 0.2 m at its center. Estimate the tension in the fence fabric.

Solution 25.10

a. Number of silt fences:

$$L_{\max}\,(\text{m}) = 36.2e^{-11.1\tan\alpha}$$
$$\alpha = 6^\circ \Rightarrow L_{\max} = 11.3\text{ m} \Rightarrow \text{For 30 m long slope, 3 silt fences are needed}$$

b. Flow rate per meter of fence:

$$Q(\text{m}^3/\text{hr}) = C \times I\,(\text{m/hr}) \times A(\text{m}^2)$$
$$\left.\begin{array}{l} C = 0.5 \\ I = 0.1 \\ A = 1 \times 11.3 \end{array}\right\} \Rightarrow Q = 0.565\,(\text{m}^3/\text{hr/m of fence})$$

c. Height of fence:

$$V(\text{m}^3) = Qt = H\left(\frac{H}{tan\alpha}\right) \times 1(\text{m})$$

The time t (duration of 10 yr rain storm) is 3 hours

$$0.565 \times 3 \times 2 = H\left(\frac{H}{\tan 6^\circ}\right) \Rightarrow H = 0.6\text{ m}$$

d. Tension in the fence geosynthetic fabric:

$$\frac{0.2}{3} = 0.067\text{, small enough to use the simplified equation :}$$
$$T = \frac{\gamma_w H L^2}{16s} \Rightarrow T = \frac{9.81 \times 0.6 \times 3^2}{16 \times 0.2} = 16.55\text{ kN/m}$$

Problem 25.11

Derive Eq. 25.56 for silt fences.

Solution 25.11

The average pressure on the fence is:

$$p = \frac{1}{2}\gamma_w H^2$$

The corresponding load on the fence is pL, where L is the length between posts. The resistance comes from the tension T in the fence geosynthetic. The component of T in the direction of the load is $T\sin\alpha$ (Figure 25.8s). For equilibrium:

$$pL = 2T\sin\alpha$$

In triangle OAC (Figure 25.8s), $\sin\alpha$ is given by:

$$\sin\alpha = \frac{L/2}{R}$$

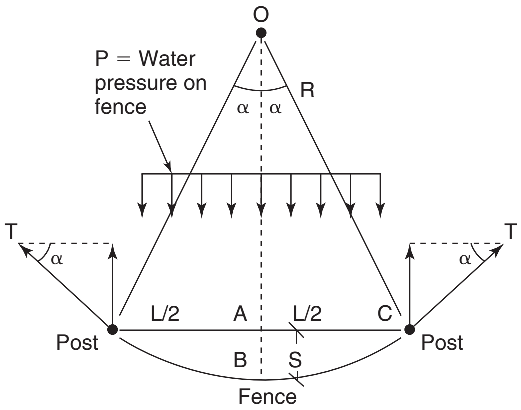


Figure 25.8s Plan view of deformed silt fence.

So,

$$T = pR$$

The radius R is given by using triangle OAC (Figure 25.8s):

$$R^2 = (R - s)^2 + \left(\frac{L}{2}\right)^2$$

Because s is small compared to R and L, this gives:

$$R = \frac{L^2}{8s}$$

Then:

$$T = p\frac{L^2}{8s} = \frac{\gamma_w H L^2}{16s}$$

Problem 25.12

A 7 m high, 60 m wide embankment is to be built on a layer of soft clay with a water table at the ground surface. The soft clay is 5 m thick and the increase in stress in the clay layer can be taken as the pressure under the embankment because the clay layer is thin compared to the width of the embankment. The clay layer has the following consolidation characteristics: $e_o = 1.1$, $\gamma = 19$ kN/m^3, $C_c = 0.5$. Two options are considered for the embankment fill: soil fill and geofoam fill. The soil fill has a unit weight of 20 kN/m^3 and the geofoam fill 2 kN/m^3. What will be the settlement of the embankment in each case? Which fill type will have the shortest time to reach 90% consolidation?

Solution 25.12

Using the consolidation theory:

$$s = H\frac{C_c}{1 + e_0}\log\frac{\sigma'_{ov} + \Delta\sigma_v}{\sigma'_{ov}}$$

a. Option 1: Soil fill.
   The increase in stress in the clay layer is:

$$\Delta\sigma_v = 20 \times 7 = 140\,\text{kPa}$$

The initial effective stress in the middle of the clay layer is:

$$\sigma'_{ov} = 2.5 \times 19 - 2.5 \times 9.81 = 23\,\text{kPa}$$

Therefore:

$$s = 5\frac{0.5}{1+1.1}\log\frac{23+140}{23} = 1.01\text{ m}$$

b. Option 2: Geofoam.
The increase in stress in the clay layer is:

$$\Delta\sigma_v = 2\times 7 = 14\text{ kPa}$$

The initial effective stress in the middle of the clay layer is still:

$$\sigma'_{ov} = 2.5\times 19 - 2.5\times 9.81 = 23\text{ kPa}$$

Therefore:

$$s = 5\frac{0.5}{1+1.1}\log\frac{23+14}{23} = 0.25\text{ m}$$

So, the settlement is reduced by a factor of 4 but the time to reach 90% consolidation is unchanged; the time required for the settlement to take place does not depend on the stress level, but rather on the drainage length and the properties of the compressing layer:

$$t = T_v\frac{H_{dr}^2}{c_v}$$

Problem 25.13

A building refrigerated at -5°C is being designed on a soil with a high water table. The concern is the cost of the power (watts) to maintain the difference in temperature across the foundation. Two alternatives are considered. The first consists of a relatively inexpensive 100 mm thick concrete slab on grade on top of the soil. The second one consists of the same slab on grade on top of a 150 mm thick geofoam. The concrete has a thermal resistance R value equal to 0.9 m^2.C/W per meter of thickness and the geofoam 35 m^2.C/W per meter of thickness. Calculate the amount of power required in each case to maintain the difference in temperature at -5°C above the slab and 0°C on top of the soil.

Solution 25.13

The heat flow is defined as:

$$\frac{\Delta Q}{\Delta t}(W) = kA\frac{\Delta T}{\Delta x}$$

Where:

$$\text{k} = \text{Material thermal conductivity}\,(\text{W/m.}^\circ\text{C})$$

$$\text{A} = \text{Cross-sectional area}$$

$$\Delta\text{T}/\Delta\text{x} = \text{Temperature gradient}$$

In terms of thermal resistance, the preceding equation can be written as:

$$\frac{\Delta Q}{\Delta t}(\text{W}) = \frac{A\Delta T}{R}$$

where R ($m^2.^\circ$C/W) is the thermal resistance per meter thickness of the material. Note that the thermal resistance of a layered system is equal to the sum of each layer's thermal resistance. Assuming a unit area of the slab ($A = 1\text{ m}^2$), for the case of the concrete slab only, the heat flow is:

$$\frac{\Delta Q}{\Delta t}(\text{W}) = \frac{1\times 5}{0.9\times 0.1} = 55.5\text{ Watt}$$

For the case of the concrete slab + 150 mm of geofoam:

$$\frac{\Delta Q}{\Delta t}(\text{W}) = \frac{1\times 5}{(0.9\times 0.1 + 35\times 0.15)} = 0.93\text{ Watt}$$

The use of 150 mm of geofoam can reduce the power usage by 98.3%.

Problem 25.14

A Styrofoam coffee cup holds coffee at 80°C. Your hand holding the coffee cup is at 30°C. Assuming a steady-state heat transfer in the cup wall, what is the R rating per meter of the Styrofoam if the amount of heat released from the coffee cup through the wall of the cup is 45 W? What is the thermal conductivity of the Styrofoam if the cup wall is 1.5 mm thick?

Solution 25.14

Assume a coffee cup with an internal radius $r_1 = 40$ mm, an external radius $r_2 = 41.5$ mm and a height of 160 mm.

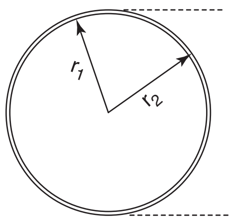


Figure 25.9s Coffee cup dimensions.

At steady state, the amount of heat Q (W) released from the coffee cup, assuming that it is a long hollow cylinder, can be calculated as follows:

$$Q(\mathrm{W}) = -kA\frac{\Delta T}{r_1 \times \ln\left(\frac{r_2}{r_1}\right)} = k2\pi r_1 L\frac{\Delta T}{r_1 \times \ln\left(\frac{r_2}{r_1}\right)}$$

with A (m^2) $= 2\pi r_1 L$ as the internal surface area of the cup.

k (W/m°C) is the thermal conductivity of the cup wall.

ΔT is the temperature difference between the inside and outside faces of the cup wall.

The preceding equation can be rewritten in terms of thermal resistance:

$$Q(\mathrm{W}) = -2\pi L\frac{\Delta T}{R}$$

where R ($m^{2\circ}$C/W/m) is the thermal resistance per meter of the cup wall and is equal to:

$$R = \frac{\ln\left(\frac{r_2}{r_1}\right)}{k}$$

Based on the data given in the problem statement, the thermal resistance of the Styrofoam is:

$$R = -2\pi L\frac{\Delta T}{Q} = -2\pi \times 0.16 \times \frac{(30-80)}{45} = 1.117\ \mathrm{m^2.^\circ C/W/m}$$

The thermal conductivity of the Styrofoam is then:

$$k(\mathrm{W/m.^\circ C}) = \frac{\ln\left(\frac{r_2}{r_1}\right)}{R} = \frac{\ln\left(\frac{0.0415}{0.04}\right)}{1.117} = 0.033$$