2.7 DESIGNING FOR SOIL REINFORCEMENT

This section continues the discussion of the use of geotextiles in the primary function of reinforcement. Since this was the topic of the preceding section involving road systems, it could easily have been incorporated into that section. However, this type of soil reinforcement raises a unique set of design issues, whereby the geotextile in horizontal layers and the interspersed soil form a *mechanically stabilized earth* (MSE) system rather than acting as a discrete material element. Three applications are involved here: (1) geotextile reinforced walls (facing angle $\geq 70°$ to the horizontal), (2) geotextile reinforced slopes (facing angle $<70°$ to the horizontal), and (3) geotextile reinforced foundations (also called basal reinforcement).

2.7.1 Geotextile Reinforced Walls

Background. Conventional gravity and cantilever wall systems made from masonry and concrete resist lateral earth pressure by virtue of their large mass. They act as rigid units and have served the industry well for centuries. However, a new era of retaining walls was introduced in the 1960s by H. Vidal with Reinforced Earth. Here metal strips extending from exposed facing panels back into the soil serve the dual role of anchoring the facing units and being restrained through frictional stresses mobilized between the strips and the backfill soil. The backfill soil both creates the lateral pressure and interacts with the strips to resist it. The walls are very flexible compared with conventional gravity structures. They offer many advantages, including significantly lower cost per square meter of exposed surface. A steady series of variations followed Vidal's steel strips, all of which can be put into the MSE wall category:

- Facing panels with metal strip reinforcement
- Facing panels with metal wire mesh reinforcement
- Solid panels with tieback anchors
- Anchored gabion walls
- Anchored crib walls
- Geotextile-reinforced walls (to be described here)
- Geogrid-reinforced walls (to be described in Chapter 3)

In all cases, the reinforced soil mass behind the wall facing is said to be mechanically stabilized earth and the wall system is generically called an MSE wall.

Construction Details. A critical factor in the successful functioning of a geotextile-reinforced MSE wall is proper construction, which is done on a planned sequential basis. Upon preparing an adequate soil foundation, which consists of removing unsuitable material and compacting in situ or replacement foundation soils, the wall itself is begun. There is no concrete footing with these walls, and the lowest geotextile layer is placed directly on the foundation soil. An iterative construction sequence, developed by the U.S. Forest Service (see Figure 2.43) can be summarized as follows. The resulting wall is referred to as a "wrap-around" MSE wall.

1. A wooden form of a height slightly greater than the individual soil layer thickness, called the *lift height*, is placed on the ground surface or on the previously placed lift after the first layer is completed. This form is nothing more than a series of metal L brackets with a continuous wooden brace board running along the face of the wall.
2. The geotextile is unrolled and positioned so that approximately 1.0 m extends over the top of the form and hangs free. If it is sufficiently wide, the geotextile can be unrolled parallel to the wall. In this way the geotextile's cross machine direction is oriented in the maximum stress direction. This will depend on the required design length and geotextile strength, which will be discussed later. If a single roll is not wide enough, two of them can be sewn together. The sewn strength is then a governing factor. Alternatively, the geotextile can be deployed perpendicular to the wall in full-width strips and adjacent roll edges can be overlapped or sewn. In this way the geotextile's machine direction is oriented in the maximum stress direction.
3. Backfill is now placed on the geotextile for 1/2 to 3/4 of its lift height and compacted. This is typically 200 to 400 mm and is done with lightweight construction equipment. The choice of backfill soil type is important. If it is angular gravel, drainage can easily occur but high installation damage to the geotextile must be considered. If it is fine-grained silts or clays, drainage cannot occur and hydrostatic pressures must be considered. This leaves sand, which the author considers the ideal backfill soil for MSE walls that are reinforced by geotextiles or geogrids.
4. A windrow is made 300 to 600 mm from the face of the wall with a road grader or is dug by hand. Care must be exercised not to damage the underlying geotextile.
5. The free end of the geotextile—that is, its "tail"—is then folded back over the wooden form into the windrow.
6. The remaining lift thickness of soil is then completed to the planned lift height and suitably compacted.
7. The wooden form is then removed from in front of the wall, and the metal brackets from beneath the lift, and the assembly is reset on top in preparation for the next higher lift. Note that it is usually necessary to have scaffolding in front of the wall when the wall is higher than 1.5 or 2.0 m.

When completed, this sequence provides walls similar to those shown in Figure 2.44. The exposed face of the wall must now be covered to prevent the geotextile's weakening due to UV exposure (recall Section 2.3.6) and possible vandalism. Bituminous emulsions or other asphalt products have been used for covering wall faces and have the advantage of being flexible, as are the walls themselves. Unfortunately, oxidation of the bitumen causes deterioration after a few years, and it must be periodically reapplied. Alternatively, the surface of wrap-around geotextile walls can be covered with shotcrete (wet-mixed cement/sand/water paste with air supplied at the nozzle) or gunite (dry cement/sand mix

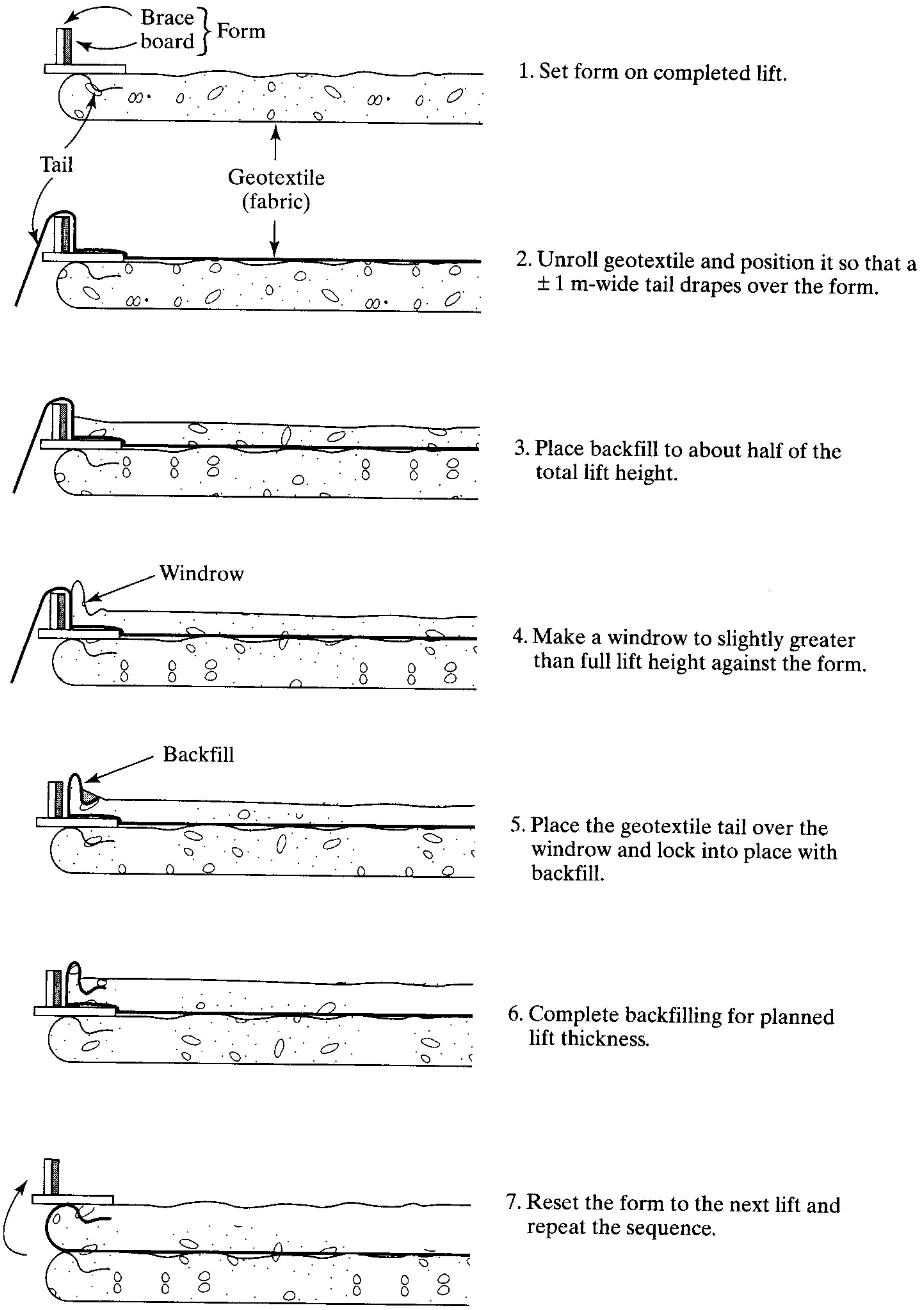


Figure 2.43 Construction sequence for geotextile wrap-around walls followed by U.S. Forest Service.

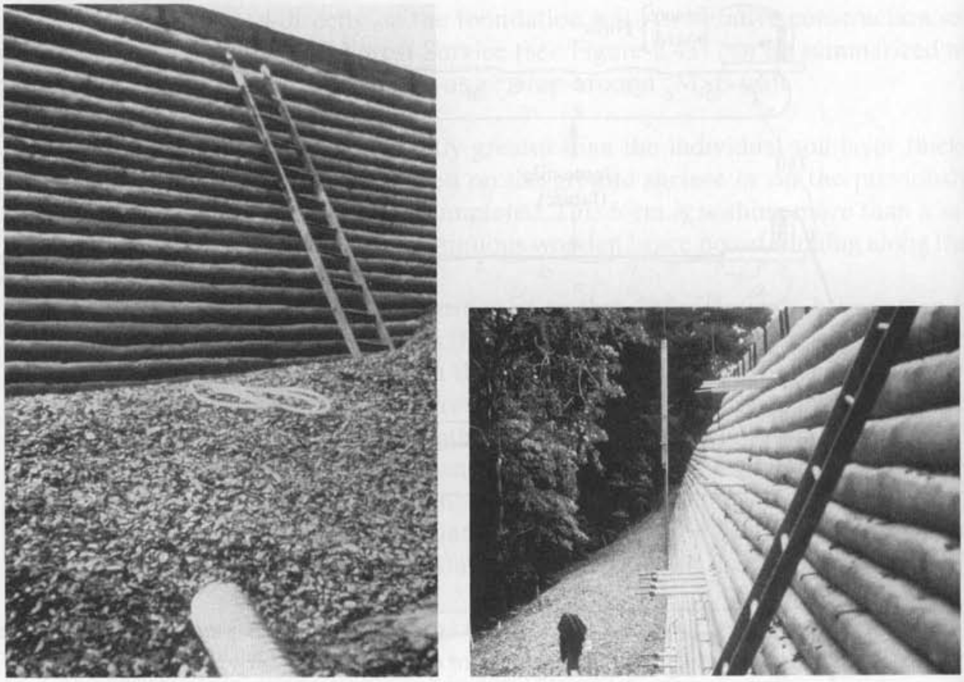


Figure 2.44 Geotextile wrap-around walls. (Compliments of Crown Zellerbach Corp.)

with water and air supplied at the nozzle). A wire mesh anchored between the geotextile layers may be necessary to keep the coating adhered to the vertical face of the wall. Still further, a precast concrete or cast-in-place concrete facing can be used, but only after deformation equilibrium of the wrap-around wall has occurred.

Design Methods. There are two somewhat different approaches to the design of geotextile walls: the method used by Broms [87] and the one used by the U.S. Forest Service, as discussed in Steward et al. [88] and Whitcomb and Bell [89]. The latter method will be followed in this book. This method follows the work that Lee et al. [90] did on reinforced earth walls with metallic strip reinforcement and was originally adapted to geotextile walls by Bell et al. [91]. The design progresses in parts, as follows:

- Internal stability is first addressed to determine geotextile spacing, geotextile length, and overlap distance.
- External stability against overturning, sliding, and foundation failure is investigated and the internal design verified or modified accordingly.
- Miscellaneous considerations, including wall facing details and external drainage, are completed.

To determine the geotextile layer separation distances, earth pressures are assumed to be linearly distributed using Rankine active "earth pressure" conditions for the soil backfill and "at rest" conditions for the surcharge. A prediction conference at the Canadian Royal Military College, however, showed that the entire design to be presented here is quite conservative (see Jarrett and McGown [92]). Therefore, active earth pressure (K_a) conditions will be used throughout. An even less conservative approach would be to use a Coulomb analysis for the earth pressure values. This is the approach used in several computer programs and will be discussed later. Boussinesq elastic theory for live loads on the soil backfill is used. As shown in Figure 2.45, the following earth pressures result:

$$\sigma_{hs} = K_a \gamma z \tag{2.42}$$

$$\sigma_{hq} = K_a q \tag{2.43}$$

$$\sigma_{hl} = P\frac{x^2 z}{R^5} \tag{2.44}$$

$$\sigma_h = \sigma_{hs} + \sigma_{hq} + \sigma_{hl} \tag{2.45}$$

where

σ_{hs} = lateral pressure due to soil,
K_a = $\tan^2(45 - \phi/2)$ = coefficient of active earth pressure, where
ϕ = angle of shearing resistance of backfill soil,
γ = unit weight of backfill soil,
z = depth from ground surface to layer in question,

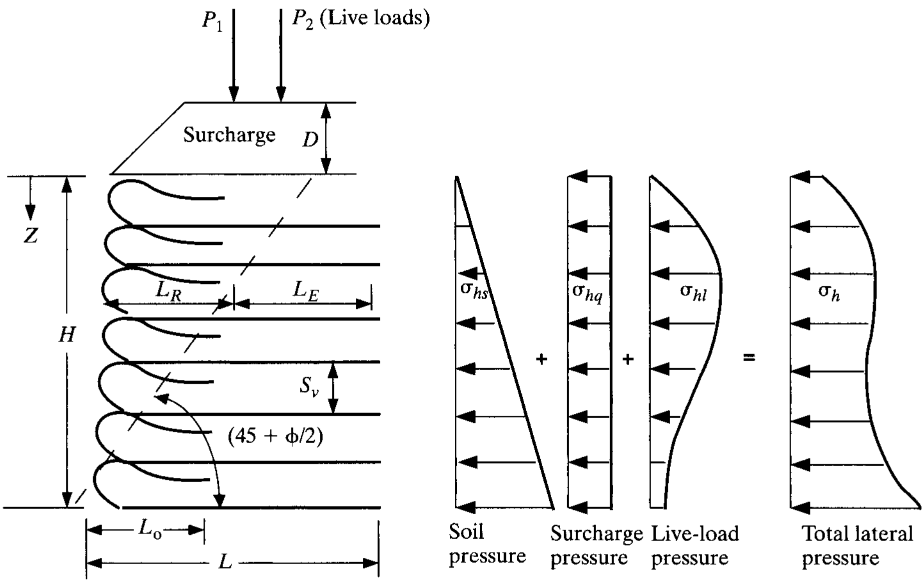


Figure 2.45 Earth pressure concepts and theory for geotextile wall design.

σ_{hq} = lateral pressure due to surcharge load,
q = $\gamma_q D$ = surcharge load on ground surface, where,
γ_q = unit weight of surcharge soil, and
D = depth of surcharge soil,
σ_{hl} = lateral pressure due to live load,
P = concentrated live load on backfill surface,
x = horizontal distance load is away from wall,
R = radial distance from load point on wall where pressure is being calculated, and
σ_h = total, or cumulative, lateral earth pressure on wall.

The calculations of σ_{hs} and σ_{hq} are quite straightforward, but σ_{hl} presents problems, particularly for multiwheeled truck loads where superposition of each wheel must be performed. Figure 2.46 greatly aids in such calculations.

By taking a free body at any depth in the total lateral pressure diagram and then summing the forces in the horizontal direction, we obtain the equation for the lift thickness:

$$\sigma_h S_v = \frac{T_{\text{allow}}}{\text{FS}}$$

$$S_v = \frac{T_{\text{allow}}}{\sigma_h \, \text{FS}} \tag{2.46}$$

where

S_v = vertical spacing (lift thickness),
T_{allow} = allowable stress in the geotextile (recall equation 2.24 and Table 2.11),
σ_h = total lateral earth pressure at depth considered, and
FS = factor of safety (use 1.3 to 1.5 when using T_{allow} as determined above).

The same free-body approach can be taken for obtaining the length of embedment of the geotextile layers in the anchorage zone, L_e. Note that when these values are obtained they must be added to the nonacting lengths (L_R) of geotextile within the active zone for the total geotextile lengths (L); that is,

$$L = L_e + L_R \tag{2.47}$$

where

$$L_R = (H - z)\tan\left(45 - \frac{\phi}{2}\right) \tag{2.48}$$

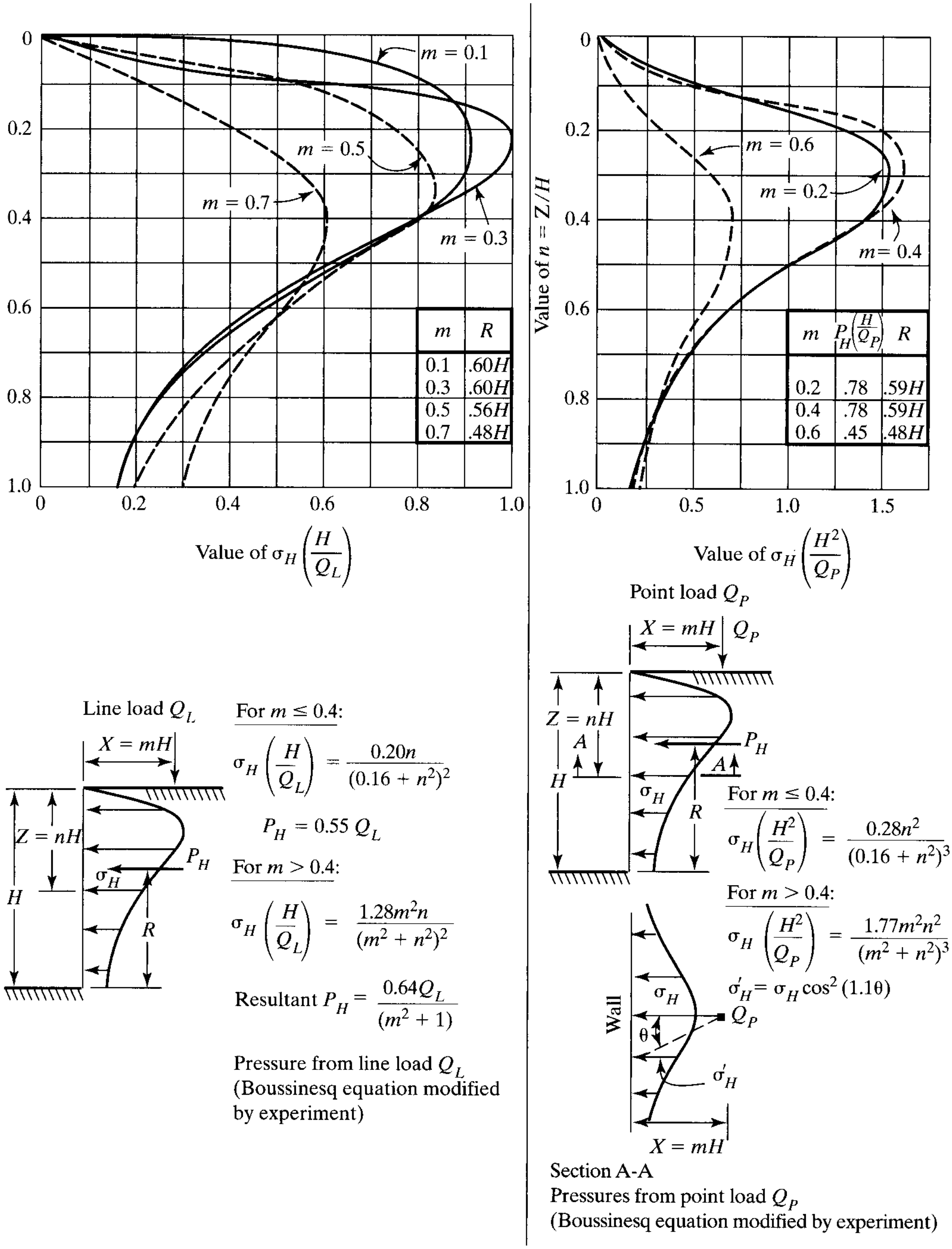


Figure 2.46 Lateral earth pressure due to a surface load. Left side is for line load; right side is for point load. (After NAVFAC [93])

and

$$\begin{aligned} S_v \sigma_h \,\mathrm{FS} &= 2\tau L_e \\ &= 2(c_a + \sigma_v \tan \delta) L_e \\ &= 2(c_a + \gamma Z \tan \delta) L_e \\ L_e &= \frac{S_v \sigma_h \,\mathrm{FS}}{2(c_a + \gamma Z \tan \delta)} \end{aligned} \tag{2.49}$$

where

τ = shear strength of the soil to the geotextile,
L_e = required embedment length (minimum is one meter),
S_v = vertical spacing (lift thickness),
σ_h = total lateral pressure at depth considered,
FS = factor of safety,
c_a = soil adhesion between soil and geotextile (zero if granular soil is used),
γ = unit weight of backfill soil,
Z = depth from ground surface, and
δ = angle of shearing resistance (friction) between soil and geotextile.

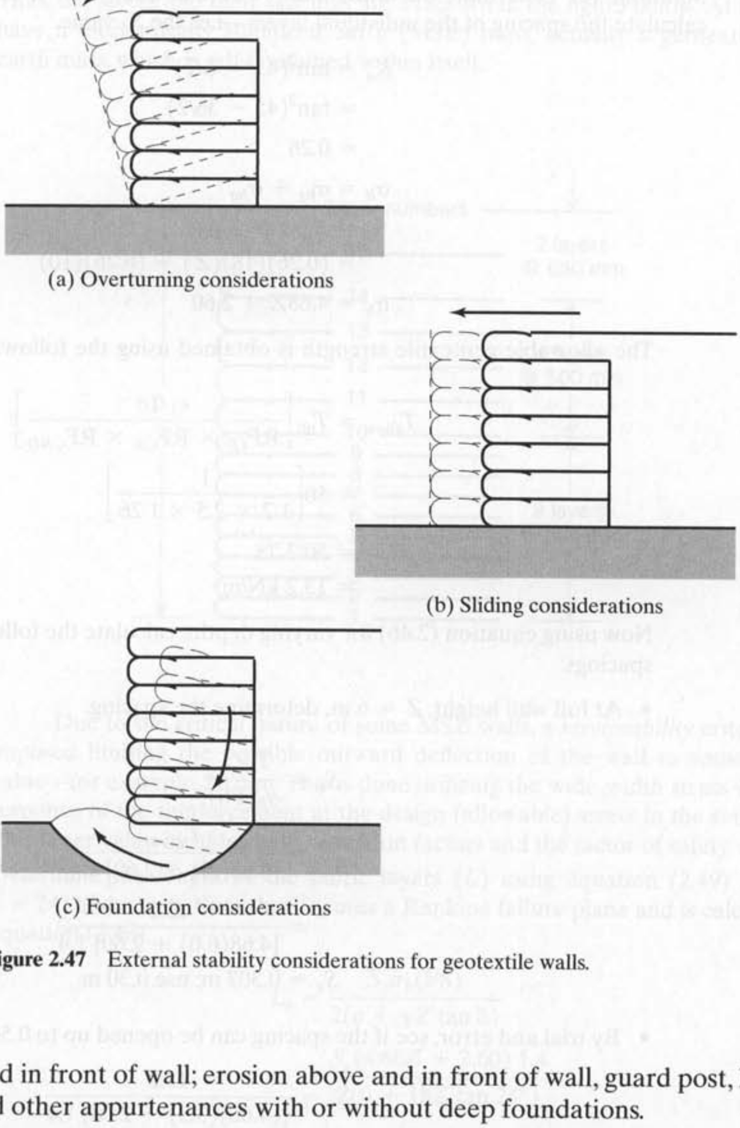
Finally, the overlap distance L_o is obtained in a manner similar to that above with a few exceptions—namely, that the distance Z should be measured to the middle of the layer and σ_h is not as large as that illustrated in Figure 2.45. It is reasonably well-established that the stress in reinforcement elements is maximum near the failure plane and falls off sharply to either side [94]. As an approximation, $0.5\sigma_h$ will be used, which results in the following equation:

$$L_o = \frac{S_v \sigma_h \,\mathrm{FS}}{4(c_a + \gamma Z \tan \delta)} \tag{2.50}$$

where L_o is the required overlap length (minimum is one meter).

Next, we must consider external stability of the geotextile reinforced MSE mass, which includes overturning, sliding, and foundation failures. These are illustrated in Figure 2.47. These features are common to all wall systems and can be treated exactly the same way as with gravity or crib walls. They are generally site-specific insofar as calculations are concerned. In general, it is recommended that for overturning and foundation-bearing capacity the FS value $\geq$ 2.0 and for sliding the FS-value $\geq$ 1.5.

The miscellaneous considerations that generally must be addressed are facing details; facing connections (if applicable); seaming methods (if necessary); drainage behind,



(a) Overturning considerations

(b) Sliding considerations

(c) Foundation considerations

Figure 2.47 External stability considerations for geotextile walls.

beneath and in front of wall; erosion above and in front of wall, guard post, light posts, fencing and other appurtenances with or without deep foundations.

Example 2.17

Design a 6-m-high wrap-around type of geotextile wall that is to carry a storage area of equivalent dead load of 10 kPa. The wall is to be backfilled with a granular soil (SP) having properties of $\gamma = 18\ \mathrm{kN/m^3}$, $\phi = 36°$, and $c = 0$. A woven slit-film geotextile with warp (machine) direction ultimate wide-width tensile strength of 50 kN/m and friction angle with granular soil of $\delta = 24°$ (see again Table 2.5) is intended to be used in its construction. The orientation of the geotextile is perpendicular to the wall face and the edges are to be overlapped or sewn to handle the weft (cross machine) direction. A factor of safety of 1.4 is to be used along with site-specific reduction factors.

Solution: (a) Determine the horizontal pressure as a function of the depth Z in order to calculate the spacing of the individual layers—i.e., the S_v value.

$$\begin{aligned} K_a &= \tan^2(45 - \phi/2) \\ &= \tan^2(45 - 36/2) \\ &= 0.26 \\ \sigma_h &= \sigma_{hs} + \sigma_{hq} \\ &= K_a\gamma z + K_a q \\ &= (0.26)(18)(Z) + (0.26)(10) \\ \sigma_h &= 4.68Z + 2.60 \end{aligned}$$

The allowable geotextile strength is obtained using the following reduction factors:

$$\begin{aligned} T_{\text{allow}} &= T_{\text{ult}}\left[\frac{1}{\text{RF}_{ID} \times \text{RF}_{CR} \times \text{RF}_{CBD}}\right] \\ &= 50\left[\frac{1}{1.2 \times 2.5 \times 1.26}\right] \\ &= 50/3.78 \\ &= 13.2 \text{ kN/m} \end{aligned}$$

Now using equation (2.46) for varying depths, calculate the following geotextile layer spacings:

- At full wall height, $Z = 6$ m, determine the spacing.

$$\begin{aligned} S_v &= \frac{T_{\text{allow}}}{\sigma_h(\text{FS})} \\ &= \frac{T_{\text{allow}}}{[4.68(Z) + 2.60]\,1.4} \\ &= \frac{13.2}{[4.68(6.0) + 2.60]\,1.4} \\ S_v &= 0.307 \text{ m; use } 0.30 \text{ m} \end{aligned}$$

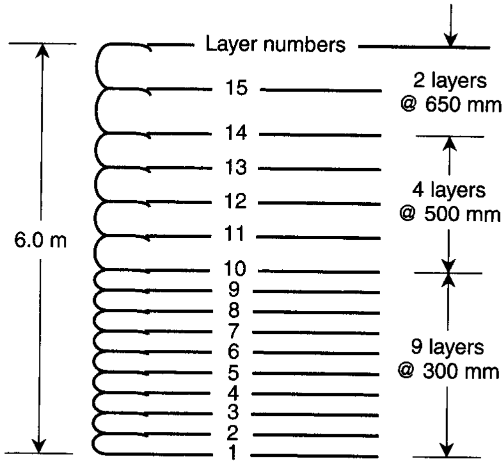
- By trial and error, see if the spacing can be opened up to 0.50 m at $Z = 3.3$ m.

$$\begin{aligned} S_v &= \frac{13.2}{[(4.68)(3.3) + 2.60]\,1.4} \\ S_v &= 0.52 \text{ m; use } 0.50 \text{ m} \end{aligned}$$

- By trial and error, see if the spacing can be further opened up to 0.65 m at $Z = 1.3$ m.

$$\begin{aligned} S_v &= \frac{13.2}{[(4.68)(1.3) + 2.60]\,1.4} \\ S_v &= 1.08 \text{ m; use } 0.65 \text{ m} \end{aligned}$$

Thus, the layers and their spacings are as shown in the figure below. At this point we have a mechanically stabilized earth (MSE) mass, actually a geotextile stabilized earth mass, which is self-contained within itself.



Due to the critical nature of some MSE walls, a *serviceability* criterion can be imposed limiting the possible outward deflection of the wall to some acceptable value—for example, 20 mm. This is done utilizing the wide-width stress versus strain response of the reinforcement at the design (allowable) stress in the reinforcement. This latter value includes both reduction factors and the factor of safety.

(b) Determine the length of the fabric layers (L) using equation (2.49) for L_e with $\delta = 24°$ and $c = 0$. Note that L_R uses a Rankine failure plane and is calculated from equation (2.48).

$$L_e = \frac{S_v\sigma_h(\text{FS})}{2(c + \gamma Z \tan\delta)}$$

$$= \frac{S_v(4.68Z + 2.60)\,1.4}{2(0 + 18Z\tan 24°)}$$

$$L_e = \frac{S_v(6.55Z + 3.64)}{16.0Z}, \text{ and}$$

$$L_R = (H - Z)\tan\left(45 - \frac{36}{2}\right)$$

$$L_R = (6.0 - Z)(0.509)$$

Layer No.	Depth, z (m)	Spacing, S_v (m)	L_e (m)	L_e min (m)	L_R (m)	L_{calc} (m)	L_{spec} (m)
15	0.65	0.65	0.49	1.0	2.72	3.72	Use 4.0
14	1.30	0.65	0.38	1.0	2.39	3.39	
13	1.80	0.50	0.27	1.0	2.14	3.14	
12	2.30	0.50	0.26	1.0	1.88	2.88	Use 3.0
11	2.80	0.50	0.25	1.0	1.63	2.63	
10	3.30	0.50	0.24	1.0	1.37	2.37	
9	3.60	0.30	0.14	1.0	1.22	2.22	
8	3.90	0.30	0.14	1.0	1.07	2.07	
7	4.20	0.30	0.14	1.0	0.92	1.92	Use 2.0
6	4.50	0.30	0.14	1.0	0.76	1.76	
5	4.80	0.30	0.14	1.0	0.61	1.61	
4	5.10	0.30	0.14	1.0	0.46	1.46	
3	5.40	0.30	0.14	1.0	0.31	1.31	
2	5.70	0.30	0.14	1.0	0.15	1.15	
1	6.00	0.30	0.13	1.0	0.00	1.00	

Note that the calculated L_e values are very small (this is typically the case with geotextile walls) and the minimum value of 1.0 m should be used. When this is added to L_R for the total length, you should round up to a even number of meters. Also, the important consideration of total geotextile width must be addressed. Three cases can be envisioned.

- *Case 1*: If the geotextile rolls are wide enough, they can be deployed parallel to the wall, and the weft or cross machine direction is the important property insofar as its wide width strength is concerned. Although this is possible for the lower fabric layers, it is not for the uppermost, since, $4.0 + 0.65 + 1.0 = 5.65$ m, which is wider than many commercially available geotextiles.
- *Case 2*: Alternatively, two adjacent rolls of fabric can be used parallel to the wall, but a sewn seam, or large overlap, must be used for the uppermost layers. If sewn seams are used, an appropriate reduction factor must be used.
- *Case 3*: The fabric layers can be deployed perpendicular to the wall, thereby utilizing their warp or machine direction wide-width strength in the major principal stress direction. This was the case posed in this example. This requires sewn seams, or overlaps, in the opposite direction. However, in this (the minor principal stress) direction the required forces are significantly lower—for example, 33 to 50% of the major principal stress direction.

(c) Check the overlap length L_o, to see if it is less than the 1.0 m recommended value using equation (2.50):

$$L_o = \frac{S_v \sigma_h (\text{FS})}{4(c_a + \gamma Z \tan \delta)}$$

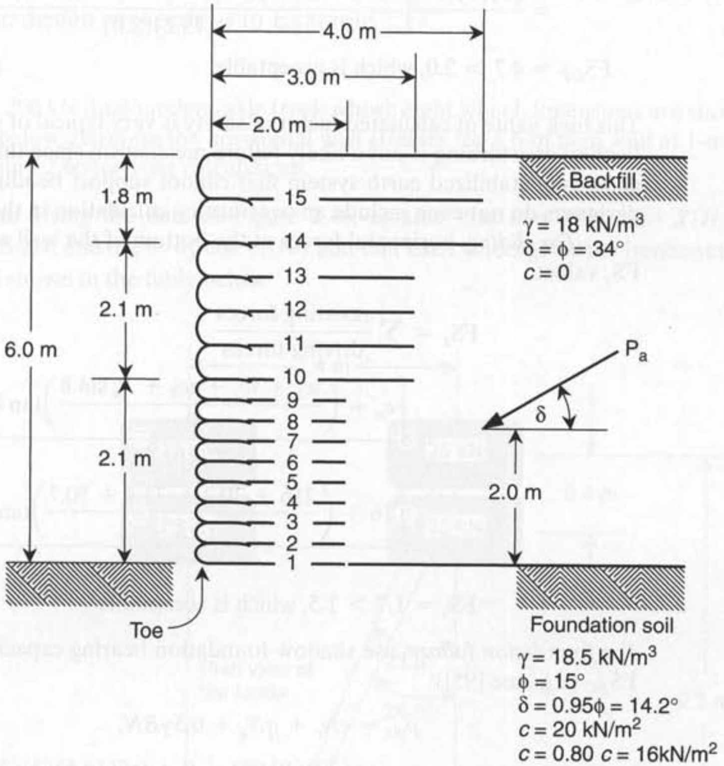
$$= \frac{S_v[4.68(Z) + 2.60]\,1.4}{4[0 + (18)Z \tan 24°]}$$

This is maximum at the upper layer at $z = 0.65$ m:

$$L_o = \frac{0.65[4.68(0.65) + 2.60]\,1.4}{4[0 + (18)(0.65)\tan 24°]}$$

$$= 0.25 \text{ m; use 1.0 m throughout}$$

The solution at this point in the design, appears as in the figure below.



(d) Since the internal stability of the wall has been provided for, focus now shifts to external stability. Standard geotechnical engineering concepts are used to analyze overturning, sliding, and bearing capacity. See the above figure where

$$K_a = \tan^2(45 - \phi/2) = \tan^2(45 - 34/2)$$
$$= 0.28$$
$$P_a = 0.5\gamma H^2 K_a$$
$$= 0.5(18)(6)^2(0.28)$$
$$= 90.7 \text{ kN/m}$$
$$P_a \cos 34 = 75.2 \text{ kN/m}$$
$$P_a \sin 34 = 50.7 \text{ kN/m}$$

For overturning, moments are taken about the toe of the wall to generate a FS_{OT} value.

$$
\begin{aligned}
FS_{OT} &= \sum \frac{\text{resisting moments}}{\text{driving moments}} \\
&= \frac{w_1x_1 + w_2x_2 + w_3x_3 + P_a \sin \delta(4)}{P_a \cos \delta(2)} \\
&= \frac{(6)(2)(18)(1) + (3.9)(1)(18)(2.5) + (1.8)(1.0)(18)(3.5) + (50.7)(4)}{(75.2)(2.0)} \\
FS_{OT} &= 4.7 > 2.0, \text{ which is acceptable}
\end{aligned}
$$

This high value of calculated factor of safety is very typical of walls of this type. Even further, overturning is not a likely failure mechanism since this is a very flexible mechanically stabilized earth system that cannot support bending stresses. Thus many designers do not even include an overturning calculation in the design process.

For sliding, horizontal forces at the bottom of the wall are summed to obtain a FS_s value:

$$
\begin{aligned}
FS_s &= \sum \frac{\text{resisting forces}}{\text{driving forces}} \\
&= \frac{\left[c_a + \left(\dfrac{w_1 + w_2 + w_3 + P_a \sin \delta}{2}\right)\tan \delta\right]2}{P_a \cos \delta} \\
&= \frac{\left[16 + \left(\dfrac{216 + 70.2 + 32.4 + 50.7}{2}\right)\tan 14.2\right]2}{75.2} \\
FS_s &= 1.7 > 1.5, \text{ which is acceptable}
\end{aligned}
$$

For foundation failure, use shallow-foundation bearing capacity theory to determine FS_{BC} (e.g., see [95]):

$$
\begin{aligned}
p_{\text{ult}} &= cN_c + qN_q + 0.5\gamma BN_\gamma \\
&= (20)(10.98) + 0 + 0.5(18.5)(2)(2.65) \\
&= 219.6 + 49.0 \\
&= 269 \text{ kN/m}^2 \\
p_{\text{act}} &= (18)(6) + (10) \\
&= 118 \text{ kN/m}^2 \\
FS_{BC} &= \frac{p_{\text{ult}}}{p_{\text{act}}} \\
&= \frac{269}{118} \\
FS_{BC} &= 2.3 > 2.0, \text{ which is acceptable}
\end{aligned}
$$

Both internal and external designs are now complete. The wall uses 15 layers of fabric (the lowest nine at 0.30 m spacing; the middle four at 0.50 m spacing; the upper two at 0.65 m spacing). The fabric lengths are 3.3 m (2 + 0.3 + 1) at the lowest level, 4.5 m (3 + 0.5 + 1) at the intermediate level, and 5.65 m (4 + 0.65 + 1) at the upper level.

Although this example illustrates the design of a wrap-around geotextile retaining wall design for static loads, it does *not* take into account the incorporation of live loads as produced by traffic. Example 2.18 will illustrate how this is done, but just to the point of calculating the additional horizontal stress distribution against the wall. Beyond this, the design proceeds as in Example 2.17.

Example 2.18

For the 200 kN dual-tandem-axle truck whose eight wheel dimensions are shown in the diagram below, calculate the horizontal wall stresses for a 6-m high wall at 1-m increments. Use Figure 2.46 for your calculations.

Solution: Using the data in Figure 2.46, assume that $n = Z/H$, $m = X/H$, $H = 6.0$ m, $Q_p = 25$ kN, and $\sigma'_h = \sigma_h \cos^2(1.1\theta)$ and that each wheel gives the horizontal stresses (in kN/m^2) shown in the table below.