CHAPTER 19

CONCRETE AND MECHANICALLY STABILIZED EARTH RETAINING WALLS

PART A—CONCRETE RETAINING WALLS

19.1 INTRODUCTION

The common types of concrete retaining walls and their uses were discussed in Chapter 11. The lateral pressure theories and the methods of calculating the lateral earth pressures were described in detail in the same chapter. The two classical earth pressure theories that have been considered are those of Rankine and Coulomb. In this chapter we are interested in the following:

1. Conditions under which the theories of Rankine and Coulomb are applicable to cantilever and gravity retaining walls under the active state.
2. The common minimum dimensions used for the two types of retaining walls mentioned above.
3. Use of charts for the computation of active earth pressure.
5. Drainage provisions for retaining walls.

19.2 CONDITIONS UNDER WHICH RANKINE AND COULOMB FORMULAS ARE APPLICABLE TO RETAINING WALLS UNDER ACTIVE STATE

Conjugate Failure Planes Under Active State

When a backfill of cohesionless soil is under an active state of plastic equilibrium due to the stretching of the soil mass at every point in the mass, two failure planes called conjugate rupture...
planes are formed. These are further designated as the inner failure plane and the outer failure plane as shown in Fig. 19.1. These failure planes make angles of $\alpha_i$ and $\alpha_0$ with the vertical. The equations for these angles may be written as (for a sloping backfill)

$$\alpha_i = \frac{90 - \phi + \epsilon - \beta}{2}$$  \hspace{1cm} (19.1a)

$$\alpha_0 = \frac{90 - \phi - \epsilon - \beta}{2}$$  \hspace{1cm} (19.1b)

where $\sin \epsilon = \frac{\sin \beta}{\sin \phi}$  \hspace{1cm} (19.2)

when $\beta = 0$, $\alpha_i = \frac{90 - \phi}{2} = 45^\circ - \frac{\phi}{2}$, $\alpha_0 = \frac{90 - \phi}{2} = 45^\circ - \frac{\phi}{2}$

The angle between the two failure planes $= 90^\circ - \alpha$

**Conditions for the Use of Rankine’s Formula**

1. Wall should be vertical with a smooth pressure face.
2. When walls are inclined, it should not come in the way of the formation of the outer failure plane. Figure 19.1 shows the formation of failure planes. Since the sloping face AB' of the retaining wall makes an angle $\alpha_w$, greater than $\alpha_o$, the wall does not interfere with the formation of the outer failure plane. The plastic state exists within wedge ACC'.

The method of calculating the lateral pressure on AB' is as follows.

1. Apply Rankine’s formula for the vertical section AB.
2. Combine $P_u$ with $W$, the weight of soil within the wedge $ABB'$, to give the resultant $P_R$.

Let the resultant $P_R$ in this case make an angle $\delta$, with the normal to the face of the wall. Let the maximum angle of wall friction be $\delta_m$. If $\delta > \delta_m$, the soil slides along the face AB' of the wall.

**Figure 19.1** Application of Rankine’s active condition to gravity walls
In such an eventuality, the Rankine formula is not recommended but the Coulomb formula may be used.

**Conditions for the Use of Coulomb's Formula**

1. The back of the wall must be plane or nearly plane.
2. Coulomb's formula may be applied under all other conditions where the surface of the wall is not smooth and where the soil slides along the surface.

In general the following recommendations may be made for the application of the Rankine or Coulomb formula without the introduction of significant errors:

1. Use the Rankine formula for cantilever and counterfort walls.
2. Use the Coulomb formula for solid and semisolid gravity walls.

In the case of cantilever walls (Fig. 19.2), $P_a$ is the active pressure acting on the vertical section $AB$ passing through the heel of the wall. The pressure is parallel to the backfill surface and acts at a height $H/3$ from the base of the wall where $H$ is the height of the section $AB$. The resultant pressure $P_R$ is obtained by combining the lateral pressure $P_a$ with the weight of the soil $W_s$ between the section $AB$ and the wall.

**19.3 PROPORTIONING OF RETAINING WALLS**

Based on practical experience, retaining walls can be proportioned initially which may be checked for stability subsequently. The common dimensions used for the various types of retaining walls are given below.

**Gravity Walls**

A gravity walls may be proportioned in terms of its height given in Fig. 19.3(a). The minimum top width suggested is 0.30 m. The tentative dimensions for a cantilever wall are given in Fig. 19.3(b) and those for a counterfort wall are given in Fig. 19.3(c).
19.4 EARTH PRESSURE CHARTS FOR RETAINING WALLS

Charts have been developed for estimating lateral earth pressures on retaining walls based on certain assumed soil properties of the backfill materials. These semi empirical methods represent a body of valuable experience and summarize much useful information. The charts given in Fig. 19.4 are meant to produce a design of retaining walls of heights not greater than 6 m. The charts have been developed for five types of backfill materials given in Table 19.1. The charts are applicable to the following categories of backfill surfaces. They are

1. The surface of the backfill is plane and carries no surcharge
2. The surface of the backfill rises on a slope from the crest of the wall to a level at some elevation above the crest.

The chart is drawn to represent a concrete wall but it may also be used for a reinforced soil wall. All the dimensions of the retaining walls are given in Fig. 19.4. The total horizontal and vertical pressures on the vertical section of A B of height $H$ are expressed as

$$P_h = \frac{1}{2} K_h H^2$$  (19.3)
### Table 19.1 Types of backfill for retaining walls

<table>
<thead>
<tr>
<th>Type</th>
<th>Backfill material</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Coarse-grained soil without admixture of fine soil particles, very permeable (clean sand or gravel)</td>
</tr>
<tr>
<td>2</td>
<td>Coarse-grained soil of low permeability due to admixture of particles of silt size</td>
</tr>
<tr>
<td>3</td>
<td>Residual soil with stones fine silty sand, and granular materials with conspicuous clay content</td>
</tr>
<tr>
<td>4</td>
<td>Very soft or soft clay, organic silts, or silty clays</td>
</tr>
<tr>
<td>5</td>
<td>Medium or stiff clay</td>
</tr>
</tbody>
</table>

Note: Numerals on the curves indicate soil types as described in Table 19.1.

For materials of type 5 computations of pressure may be based on the value of \( H \) 1 meter less than actual value.

**Figure 19.4** Chart for estimating pressure of backfill against retaining walls supporting backfills with a plane surface. (Terzaghi, Peck, and Mesri, 1996)
Figure 19.5  Chart for estimating pressure of backfill against retaining walls supporting backfills with a surface that slopes upward from the crest of the wall for limited distance and then becomes horizontal. (Terzaghi et al., 1996)
Concrete and Mechanically Stabilized Earth Retaining Walls

Soil type 4
Soil type 5

Note: Numerals on curves indicate the following slopes
No. Slope
1 1.5:1
2 1.75:1
3 2:1
4 3:1
5 6:1

Figure 19.5 Continued

\[ P_v = \frac{1}{2} K_v H^2 \quad (19.4) \]

Values of \( K_h \) and \( K_v \) are plotted against slope angle \( \beta \) in Fig. 19.4 and the ratio \( H_i/H \) in Fig. 19.5.

19.5 STABILITY OF RETAINING WALLS

The stability of retaining walls should be checked for the following conditions:

1. Check for sliding
2. Check for overturning
3. Check for bearing capacity failure
4. Check for base shear failure

The minimum factors of safety for the stability of the wall are:

1. Factor of safety against sliding = 1.5
2. Factor of safety against overturning = 2.0
3. Factor of safety against bearing capacity failure = 3.0

Stability Analysis

Consider a cantilever wall with a sloping backfill for the purpose of analysis. The same principle holds for the other types of walls.

Fig. 19.6 gives a cantilever wall with all the forces acting on the wall and the base, where

\[ \begin{align*}
P_a & = \text{active earth pressure acting at a height } H/3 \text{ over the base on section } AB \\
P_h & = P_a \cos \beta \\
P_v & = P_a \sin \beta \\
\beta & = \text{slope angle of the backfill}
\end{align*} \]
Chapter 19

(a) Forces acting on the wall

(b) Provision of key to increase sliding resistance

**Figure 19.6** Check for sliding

\[ W_s = \text{weight of soil} \]
\[ W_c = \text{weight of wall including base} \]
\[ W_t = \text{the resultant of } W_s \text{ and } W_c \]
\[ P_p = \text{passive earth pressure at the toe side of the wall.} \]
\[ F_R = \text{base sliding resistance} \]

**Check for Sliding (Fig. 19.6)**

The force that moves the wall = horizontal force \( P_h \)
The force that resists the movement is

\[ F_R = c_a B + R \tan \delta + P_p \]  

\[ R = \text{total vertical force} = W_s + W_c + P_v, \]

\[ \delta = \text{angle of wall friction} \]

\[ c_a = \text{unit adhesion} \]

If the bottom of the base slab is rough, as in the case of concrete poured directly on soil, the coefficient of friction is equal to \( \tan \phi \), \( \phi \) being the angle of internal friction of the soil.

The factor of safety against sliding is

\[ F_s = \frac{F_R}{P_h} \geq 1.5 \]  

(19.6)

In case \( F_s < 1.5 \), additional factor of safety can be provided by constructing one or two keys at the base level shown in Fig. 19.6b. The passive pressure \( P_p \) (Fig. 19.6a) in front of the wall should not be relied upon unless it is certain that the soil will always remain firm and undisturbed.

**Check for Overturning**

The forces acting on the wall are shown in Fig. 19.7. The overturning and stabilizing moments may be calculated by taking moments about point \( O \). The factor of safety against overturning is therefore

\[ F_o = \frac{\text{Sum of moments that resist overturning}}{\text{Sum of overturning moments}} = \frac{M_R}{M_o} \]  

(19.7a)
we may write (Fig. 19.7)

\[ F_o = \frac{W_i l_c + W_s l_s + P_s B}{P_h (H/3)} \]  \hspace{1cm} (19.7b)

where \( F_o \) should not be less than 2.0.

**Check for Bearing Capacity Failure (Fig. 19.8)**

In Fig. 19.8, \( W_i \) is the resultant of \( W_s \) and \( W_c \). \( P_{R_h} \) is the resultant of \( P_a \) and \( W_i \) and meets the base at \( m \). \( R \) is the resultant of all the vertical forces acting at \( m \) with an eccentricity \( e \). Fig. 19.8 shows the pressure distribution at the base with a maximum \( q_t \) at the toe and a minimum \( q_h \) at the heel.

An expression for \( e \) may be written as

\[ e = \frac{B}{2} \left( \frac{M_R - M_o}{\Sigma V} \right) \]  \hspace{1cm} (19.8a)

where \( R = \Sigma V = \) sum of all vertical forces

---

**Figure 19.8** Stability against bearing capacity failure
The values of $q_t$ and $q_h$ may be calculated by making use of the equations

$$q_t = \frac{R}{B} \left[ 1 + \frac{6e}{B} \right] = q_a \left[ 1 + \frac{6e}{B} \right] \quad (19.8b)$$

$$q_h = q_a \left[ 1 - \frac{6e}{B} \right] \quad (19.8c)$$

where, $q_a = R/B =$ allowable bearing pressure.

Equation (19.8) is valid for $e \leq B/6$. When $e = B/6$, $q_t = 2q_a$ and $q_h = 0$. The base width $B$ should be adjusted to satisfy Eq. (19.8). When the subsoil below the base is of a low bearing capacity, the possible alternative is to use a pile foundation.

The ultimate bearing capacity $q_u$ may be determined using Eq. (12.27) taking into account the eccentricity. It must be ensured that

$$q_t \leq \frac{q_u}{F_j} = \frac{q_u}{3}$$

**Base Failure of Foundation (Fig. 19.9)**

If the base soil consists of medium to soft clay, a circular slip surface failure may develop as shown in Fig. 19.9. The most dangerous slip circle is actually the one that penetrates deepest into the soft material. The critical slip surface must be located by trial. Such stability problems may be analyzed either by the method of slices or any other method discussed in Chapter 10.

![Figure 19.9 Stability against base slip surface shear failure](image-url)
Drainage Provision for Retaining Walls (Fig. 19.10)

The saturation of the backfill of a retaining wall is always accompanied by a substantial hydrostatic pressure on the back of the wall. Saturation of the soil increases the earth pressure by increasing the unit weight. It is therefore essential to eliminate or reduce pore pressure by providing suitable drainage. Four types of drainage are given in Fig. 19.10. The drains collect the water that enters the backfill and this may be disposed of through outlets in the wall called weep holes. The graded filter material should be properly designed to prevent clogging by fine materials. The present practice is to use geotextiles or geogrids.

The weep holes are usually made by embedding 100 mm (4 in.) diameter pipes in the wall as shown in Fig. 19.10. The vertical spacing between horizontal rows of weep holes should not exceed 1.5 m. The horizontal spacing in a given row depends upon the provisions made to direct the seepage water towards the weep holes.

Figure 19.10 Diagram showing provisions for drainage of backfill behind retaining walls: (a) vertical drainage layer (b) inclined drainage layer for cohesionless backfill, (c) bottom drain to accelerate consolidation of cohesive backfill, (d) horizontal drain and seal combined with inclined drainage layer for cohesive backfill (Terzaghi et al., 1996)
Example 19.1
Figure Ex. 19.1(a) shows a section of a cantilever wall with dimensions and forces acting thereon. Check the stability of the wall with respect to (a) overturning, (b) sliding, and (c) bearing capacity.

Solution
Check for Rankine’s condition
From Eq. (19.1b)
\[
\alpha_o = \frac{90 - \phi - \epsilon - \beta}{2} - \frac{\sin \epsilon}{\sin \phi} = \frac{\sin 15^\circ}{\sin 30^\circ} = 0.5176
\]
where \(\sin \epsilon = \frac{\sin \beta}{\sin \phi} = \frac{\sin 31^\circ}{\sin 30^\circ} = 0.5176\)

or \(\epsilon \approx 31^\circ\)

\[
\alpha_o = \frac{90 - 30}{2} - \frac{31 - 15}{2} = 22^\circ
\]

The outer failure line \(AC\) is drawn making an angle 22° with the vertical \(AB\). Since this line does not cut the wall Rankine’s condition is valid in this case.

Figure Ex. 19.1(a)
Rankine active pressure

Height of wall = \( AB = H = 7.8 \) m (Fig. Ex. 19.1(a))

\[
P_a = \frac{1}{2} \gamma H^2 K_A
\]

where \( K_A = \tan^2 (45^\circ - \phi / 2) = \frac{1}{3} \)

substituting

\[
P_a = \frac{1}{2} \times 18.5 \times (7.8)^2 \times \frac{1}{3} = 187.6 \text{ kN/m of wall}
\]

\[
P_h = P_a \cos \beta = 187.6 \cos 15^\circ = 181.2 \text{ kN/m}
\]

\[
P_v = P_a \sin \beta = 187.6 \sin 15^\circ = 48.6 \text{ kN/m}
\]

Check for overturning

The forces acting on the wall in Fig. Ex. 19.1(a) are shown. The overturning and stabilizing moments may be calculated by taking moments about point \( O \).

The whole section is divided into 5 parts as shown in the figure. Let these forces be represented by \( w_1, w_2, \ldots w_5 \) and the corresponding lever arms as \( l_1, l_2, \ldots l_5 \). Assume the weight of concrete \( \gamma_c = 24 \text{ kN/m}^3 \). The equation for the resisting moment is

\[
M_R = w_1 l_1 + w_2 l_2 + \ldots w_5 l_5
\]

The overturning moment is

\[
M_O = P_h \frac{H}{3}
\]

The details of calculations are tabulated below.

<table>
<thead>
<tr>
<th>Section No.</th>
<th>Area (m²)</th>
<th>Unit weight (kN/m³)</th>
<th>Weight (kN/m)</th>
<th>Lever arm (m)</th>
<th>Moment (kN-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.20</td>
<td>18.5</td>
<td>22.2</td>
<td>3.75</td>
<td>83.25</td>
</tr>
<tr>
<td>2</td>
<td>18.75</td>
<td>18.5</td>
<td>346.9</td>
<td>3.25</td>
<td>1127.40</td>
</tr>
<tr>
<td>3</td>
<td>3.56</td>
<td>24.0</td>
<td>85.4</td>
<td>2.38</td>
<td>203.25</td>
</tr>
<tr>
<td>4</td>
<td>3.13</td>
<td>24.0</td>
<td>75.1</td>
<td>1.50</td>
<td>112.65</td>
</tr>
<tr>
<td>5</td>
<td>0.78</td>
<td>24.0</td>
<td>18.7</td>
<td>1.17</td>
<td>21.88</td>
</tr>
</tbody>
</table>

\[ P_v = 48.6 \]  
\[ \Sigma_i = 596.9 \]

\[ \Sigma_y = 1779.3 = M_R \]

\[ M_O = 181.2 \times 2.6 = 471.12 \text{ kN-m} \]

\[ F_s = \frac{M_R}{M_O} = \frac{1779.3}{47112} = 3.78 > 2.0 \quad \text{--- OK.} \]

Check for sliding (Fig. 19.1a)

The force that resists the movement as per Eq. (19.5) is

\[ F_R = c_o B + R \tan \delta + P_p \]
Concrete and Mechanically Stabilized Earth Retaining Walls

where \( B = \text{width} = 4.75 \text{ m} \)

\( c_a = \alpha c_u \), \( \alpha = \text{adhesion factor} = 0.55 \) from Fig. 17.15

\( R = \text{total vertical force} \Sigma \gamma = 596.9 \text{ kN} \)

For the foundation soil:

\( \delta = \text{angle of wall friction} = \phi = 25^\circ \)

From Eq. (11.45c)

\[
P_p = \frac{1}{2} \gamma h^2 K_p + 2c h \sqrt{K_p}
\]

where \( h = 2 \text{ m}, \gamma = 19 \text{ kN/m}^3, c = 60 \text{ kN/m}^2 \)

\( K_p = \tan^2 (45^\circ + \phi/2) = \tan^2 (45^\circ + 25/2) = 2.46 \)

substituting

\[
P_p = \frac{1}{2} \times 19 \times 2^2 \times 2.46 + 2 \times 60 \times 2 \times \sqrt{2.46} = 470 \text{ kN/m}
\]

\( B/2 \)

\( B/2 \)

\( e = 0.183 \text{ m} \)

\( \gamma \)

\( \delta \)

\( \phi \)

\( \alpha \)

\( c_u \)

\( \Sigma \gamma \)

\( \gamma \)

\( c \)

\( h \)

\( K_p \)

\( \tan \)

\( 45^\circ \)

\( \phi/2 \)

\( 2.46 \)

\( 470 \text{ kN/m} \)

\( \text{Figure Ex. 19.1(b)} \)
\[ F_R = 60 \times 4.75 + 596.9 \tan 25^\circ + 470 = 285 + 278 + 470 = 1033 \text{ kN/m} \]

\[ P_h = 181.2 \text{ kN/m} \]

\[ F_s = \frac{F_R}{P_h} = \frac{1033}{181.2} = 5.7 > 1.5 \quad \text{- OK.} \]

Normally the passive earth pressure \( P_p \) is not considered in the analysis. By neglecting \( P_p \), the factor of safety is

\[ F_s = \frac{1033 - 470}{181.2} = \frac{563}{181.2} = 3.1 > 1.5 \quad \text{- OK} \]

Check for bearing capacity failure (Fig. 19.1b)

From Eq. (19.8b and c), the pressures at the toe and heel of the retaining wall may be written as

\[ q_t = \frac{R}{B} \left[ 1 + \frac{6e}{B} \right] \]

\[ q_h = \frac{R}{B} \left[ 1 - \frac{6e}{B} \right] \]

where \( e \) = eccentricity of the total load \( R (= \Sigma V) \) acting on the base. From Eq. (19.8a), the eccentricity \( e \) may be calculated.

\[ e = \frac{B}{2} \left( \frac{M_R - M_o}{R} \right) = \frac{4.75}{2} \left( \frac{1779.3 - 471.12}{596.9} \right) = 0.183 \text{ m} \]

Now \( q_t = \frac{596.9}{4.75} \left( 1 + \frac{6 \times 0.183}{4.75} \right) = 154.7 \text{ kN/m}^2 \)

\[ q_h = \frac{596.9}{4.75} \left( 1 - \frac{6 \times 0.183}{4.75} \right) = 96.6 \text{ kN/m}^2 \]

The ultimate bearing capacity \( q_u \) may be determined by Eq. (12.27). It has to be ensured that

\[ q_t \leq \frac{q_u}{F_s} \]

where \( F_s = 3 \)
19.6 GENERAL CONSIDERATIONS

Reinforced earth is a construction material composed of soil fill strengthened by the inclusion of rods, bars, fibers or nets which interact with the soil by means of frictional resistance. The concept of strengthening soil with rods or fibers is not new. Throughout the ages attempts have been made to improve the quality of adobe brick by adding straw. The present practice is to use thin metal strips, geotextiles, and geogrids as reinforcing materials for the construction of reinforced earth retaining walls.

A new era of retaining walls with reinforced earth was introduced by Vidal (1969). Metal strips were used as reinforcing material as shown in Fig. 19.11(a). Here the metal strips extend from the panel back into the soil to serve the dual role of anchoring the facing units and being restrained through the frictional stresses mobilized between the strips and the backfill soil. The backfill soil creates the lateral pressure and interacts with the strips to resist it. The walls are relatively flexible compared to massive gravity structures. These flexible walls offer many advantages including significant lower cost per square meter of exposed surface. The variations in the types of facing units, subsequent to Vidal’s introduction of the reinforced earth walls, are many. A few of the types that are currently in use are (Koerner, 1999)

Figure 19.11(a) Component parts and key dimensions of reinforced earth wall
(Vidal, 1969)
1. Facing panels with metal strip reinforcement
2. Facing panels with wire mesh reinforcement
3. Solid panels with tie back anchors
4. Anchored gabion walls
5. Anchored crib walls

(b) Line details of a reinforced earth wall in place

(c) Front face of a reinforced earth wall under construction for a bridge approach fill using patented precast concrete wall face units

Figure 19.11(b) and (c) Reinforced earth walls (Bowles, 1996)
6. Geotextile reinforced walls
7. Geogrid reinforced walls

In all cases, the soil behind the wall facing is said to be *mechanically stabilized earth* (MSE) and the wall system is generally called an MSE wall.

The three components of a MSE wall are the facing unit, the backfill and the reinforcing material. Figure 19.11(b) shows a side view of a wall with metal strip reinforcement and Fig. 19.11(c) the front face of a wall under construction (Bowles, 1996).

Modular concrete blocks, currently called segmental retaining walls (SRWS, Fig. 19.12(a)) are most common as facing units. Some of the facing units are shown in Fig. 19.12. Most interesting in regard to SRWS are the emerging block systems with openings, pouches, or planting areas within them. These openings are soil-filled and planted with vegetation that is indigenous to the area (Fig. 19.12(b)). Further possibilities in the area of reinforced wall systems could be in the use of polymer rope, straps, or anchor ties to the facing in units or to geosynthetic layers, and extending them into the retained earth zone as shown in Fig. 19.12(c).

A recent study (Koerner 2000) has indicated that geosynthetic reinforced walls are the least expensive of any wall type and for all wall height categories (Fig. 19.13).

### 19.7 BACKFILL AND REINFORCING MATERIALS

**Backfill**

The backfill, is limited to cohesionless, free draining material (such as sand), and thus the key properties are the density and the angle of internal friction.

---

**Figure 19.12** Geosynthetic use for reinforced walls and bulkheads (Koerner, 2000)
Reinforcing material

The reinforcements may be strips or rods of metal or sheets of geotextile, wire grids or geogrids (grids made from plastic).

Geotextile is a permeable geosynthetic comprised solely of textiles. Geotextiles are used with foundation soil, rock, earth or any other geotechnical engineering-related material as an integral part of a human made project, structure, or system (Koerner, 1999). AASHTO (M288-96) provides (Table 19.2) geotextile strength requirements (Koerner, 1999). The tensile strength of geotextile varies with the geotextile designation as per the design requirements. For example, a woven slit-film polypropylene (weighing 240 g/m²) has a range of 30 to 50 kN/m. The friction angle between soil and geotextiles varies with the type of geotextile and the soil. Table 19.3 gives values of geotextile friction angles (Koerner, 1999).

The test properties represent an idealized condition and therefore result in the maximum possible numerical values when used directly in design. Most laboratory test values cannot generally be used directly and must be suitably modified for in-situ conditions. For problems dealing with geotextiles the ultimate strength \( T_u \) should be reduced by applying certain reduction factors to obtain the allowable strength \( T_a \) as follows (Koerner, 1999).

\[
T_a = T_u \left( \frac{1}{RF_{ID} \times RF_{CR} \times RF_{CD} \times RF_{BD}} \right)
\]

(19.9)

where

- \( T_a \) = allowable tensile strength
- \( T_u \) = ultimate tensile strength
- \( RF_{ID} \) = reduction factor for installation damage
- \( RF_{CR} \) = reduction factor for creep
- \( RF_{BD} \) = reduction factor for biological degradation and
- \( RF_{CD} \) = reduction factor for chemical degradation

Typical values for reduction factors are given in Table 19.4.
Table 19.2  AASHTO M288-96 Geotextile strength property requirements

<table>
<thead>
<tr>
<th>Test methods</th>
<th>Units</th>
<th>Elongation</th>
<th>Elongation</th>
<th>Elongation</th>
<th>Elongation</th>
<th>Elongation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>&lt; 50 %</td>
<td>≥ 50 %</td>
<td>&lt; 50 %</td>
<td>≥ 50 %</td>
<td>&lt; 50 %</td>
</tr>
<tr>
<td>Strength</td>
<td>N</td>
<td>1400</td>
<td>900</td>
<td>1100</td>
<td>700</td>
<td>800</td>
</tr>
<tr>
<td>N</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seam strength</td>
<td>N</td>
<td>1200</td>
<td>810</td>
<td>990</td>
<td>630</td>
<td>720</td>
</tr>
<tr>
<td>N</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Puncture strength</td>
<td>N</td>
<td>500</td>
<td>350</td>
<td>400</td>
<td>250</td>
<td>300</td>
</tr>
<tr>
<td>N</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MARV tear strength</td>
<td>kPa</td>
<td>3500</td>
<td>1700</td>
<td>2700</td>
<td>1300</td>
<td>2100</td>
</tr>
</tbody>
</table>

Measured in accordance with ASTM D4632. Woven geotextiles fail at elongations (strains) < 50%, while nonwovens fail at elongation (strains) > 50%.

When sewn seams are required. Overlap seam requirements are application specific.

Required MARV tear strength for woven monofilament geotextiles is 250 N.
### Table 19.3  Peak soil-to-geotextile friction angles and efficiencies in selected cohesionless soils*

<table>
<thead>
<tr>
<th>Geotextile type</th>
<th>Concrete sand ( (\phi = 30^\circ) )</th>
<th>Rounded sand ( (\phi = 28^\circ) )</th>
<th>Sity sand ( (\phi = 26^\circ) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Woven, monofilament</td>
<td>26° (84 %)</td>
<td>24° (77 %)</td>
<td>26° (84 %)</td>
</tr>
<tr>
<td>Woven, slit-film</td>
<td>24° (77 %)</td>
<td>24° (84 %)</td>
<td>23° (87 %)</td>
</tr>
<tr>
<td>Nonwoven, heat-bonded</td>
<td>30° (100 %)</td>
<td>26° (92 %)</td>
<td>25° (96 %)</td>
</tr>
<tr>
<td>Nonwoven, needle-punched</td>
<td>30° (100 %)</td>
<td>26° (92 %)</td>
<td>25° (96 %)</td>
</tr>
</tbody>
</table>

* Numbers in parentheses are the efficiencies. Values such as these should not be used in final design. Site specific geotextiles and soils must be individually tested and evaluated in accordance with the particular project conditions: saturation, type of liquid, normal stress, consolidation time, shear rate, displacement amount, and so on. (Koerner, 1999)

### Table 19.4  Recommended reduction factor values for use in Eq. (19.9)

<table>
<thead>
<tr>
<th>Application Area</th>
<th>Range of Reduction Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Installation Damage</td>
</tr>
<tr>
<td>Separation</td>
<td>1.1 to 2.5</td>
</tr>
<tr>
<td>Cushioning</td>
<td>1.1 to 2.0</td>
</tr>
<tr>
<td>Unpaved roads</td>
<td>1.1 to 2.0</td>
</tr>
<tr>
<td>Walls</td>
<td>1.1 to 2.0</td>
</tr>
<tr>
<td>Embankments</td>
<td>1.1 to 2.0</td>
</tr>
<tr>
<td>Bearing capacity</td>
<td>1.1 to 2.0</td>
</tr>
<tr>
<td>Slope stabilization</td>
<td>1.1 to 1.5</td>
</tr>
<tr>
<td>Pavement overlays</td>
<td>1.1 to 1.5</td>
</tr>
<tr>
<td>Railroads (filter/sep.)</td>
<td>1.5 to 3.0</td>
</tr>
<tr>
<td>Flexible forms</td>
<td>1.1 to 1.5</td>
</tr>
<tr>
<td>Silt fences</td>
<td>1.1 to 1.5</td>
</tr>
</tbody>
</table>

* The low end of the range refers to applications which have relatively short service lifetimes and / or situations where creep deformations are not critical to the overall system performance. (Koerner, 1999)

### Table 19.5  Recommended reduction factor values for use in Eq. (19.10) for determining allowable tensile strength of geogrids

<table>
<thead>
<tr>
<th>Application Area</th>
<th>(RF_{ID})</th>
<th>(RF_{CR})</th>
<th>(RF_{CD})</th>
<th>(RF_{BD})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unpaved roads</td>
<td>1.1 to 1.6</td>
<td>1.5 to 2.5</td>
<td>1.0 to 1.5</td>
<td>1.0 to 1.1</td>
</tr>
<tr>
<td>Paved roads</td>
<td>1.2 to 1.5</td>
<td>1.5 to 2.5</td>
<td>1.1 to 1.6</td>
<td>1.0 to 1.1</td>
</tr>
<tr>
<td>Embankments</td>
<td>1.1 to 1.4</td>
<td>2.0 to 3.0</td>
<td>1.1 to 1.4</td>
<td>1.0 to 1.2</td>
</tr>
<tr>
<td>Slopes</td>
<td>1.1 to 1.4</td>
<td>2.0 to 3.0</td>
<td>1.1 to 1.4</td>
<td>1.0 to 1.2</td>
</tr>
<tr>
<td>Walls</td>
<td>1.1 to 1.4</td>
<td>2.0 to 3.0</td>
<td>1.1 to 1.4</td>
<td>1.0 to 1.2</td>
</tr>
<tr>
<td>Bearing capacity</td>
<td>1.2 to 1.5</td>
<td>2.0 to 3.0</td>
<td>1.1 to 1.6</td>
<td>1.0 to 1.2</td>
</tr>
</tbody>
</table>
Geogrid
A geogrid is defined as a geosynthetic material consisting of connected parallel sets of tensile ribs with apertures of sufficient size to allow strike-through of surrounding soil, stone, or other geotechnical material (Koerner, 1999).

Geogrids are matrix like materials with large open spaces called apertures, which are typically 10 to 100 mm between the ribs, called longitudinal and transverse respectively. The primary function of geogrids is clearly reinforcement. The mass of geogrids ranges from 200 to 1000 g/m$^2$ and the open area varies from 40 to 95 %. It is not practicable to give specific values for the tensile strength of geogrids because of its wide variation in density. In such cases one has to consult manufacturer’s literature for the strength characteristics of their products. The allowable tensile strength, $T_a$, may be determined by applying certain reduction factors to the ultimate strength $T_U$ as in the case of geotextiles. The equation is

$$T_a = T_U \left( \frac{1}{RF_{ID} \times RF_{CR} \times RF_{BD} \times RF_{CD}} \right)$$

(19.10)

The definition of the various terms in Eq (19.10) is the same as in Eq. (19.9). However, the reduction factors are different. These values are given in Table 19.5 (Koerner, 1999).

Metal Strips
Metal reinforcement strips are available in widths ranging from 75 to 100 mm and thickness on the order of 3 to 5 mm, with 1 mm on each face excluded for corrosion (Bowles, 1996). The yield strength of steel may be taken as equal to about 35000 lb/in$^2$ (240 MPa) or as per any code of practice.

19.8 CONSTRUCTION DETAILS
The method of construction of MSE walls depends upon the type of facing unit and reinforcing material used in the system. The facing unit which is also called the skin can be either flexible or stiff, but must be strong enough to retain the backfill and allow fastenings for the reinforcement to be attached. The facing units require only a small foundation from which they can be built, generally consisting of a trench filled with mass concrete giving a footing similar to those used in domestic housing. The segmental retaining wall sections of dry-laid masonry blocks, are shown in Fig. 19.12(a). The block system with openings for vegetation is shown in Fig. 19.12(b).

The construction procedure with the use of geotextiles is explained in Fig. 19.14(a). Here, the geotextile serve both as a reinforcement and also as a facing unit. The procedure is described below (Koerner, 1985) with reference to Fig. 19.14(a).

1. Start with an adequate working surface and staging area (Fig. 19.14a).
2. Lay a geotextile sheet of proper width on the ground surface with 4 to 7 ft at the wall face draped over a temporary wooden form (b).
3. Backfill over this sheet with soil. Granular soils or soils containing a maximum 30 percent silt and/or 5 percent clay are customary (c).
4. Construction equipment must work from the soil backfill and be kept off the unprotected geotextile. The spreading equipment should be a wide-tracked bulldozer that exerts little pressure against the ground on which it rests. Rolling equipment likewise should be of relatively light weight.
5. When the first layer has been folded over the process should be repeated for the second layer with the temporary facing form being extended from the original ground surface or the wall being stepped back about 6 inches so that the form can be supported from the first layer. In the latter case, the support stakes must penetrate the fabric.

6. This process is continued until the wall reaches its intended height.

7. For protection against ultraviolet light and safety against vandalism the faces of such walls must be protected. Both shotcrete and gunite have been used for this purpose.

Figure 19.14(b) shows complete geotextile walls (Koerner, 1999).
19.9 DESIGN CONSIDERATIONS FOR A MECHANICALLY STABILIZED EARTH WALL

The design of a MSE wall involves the following steps:

1. Check for internal stability, addressing reinforcement spacing and length.
2. Check for external stability of the wall against overturning, sliding, and foundation failure.

The general considerations for the design are:

1. Selection of backfill material: granular, freely draining material is normally specified. However, with the advent of geogrids, the use of cohesive soil is gaining ground.
2. Backfill should be compacted with care in order to avoid damage to the reinforcing material.
3. Rankine's theory for the active state is assumed to be valid.
4. The wall should be sufficiently flexible for the development of active conditions.
5. Tension stresses are considered for the reinforcement outside the assumed failure zone.
6. Wall failure will occur in one of three ways
Figure 19.15 Principles of MSE wall design

(a) Reinforced earth-wall profile with surcharge load

(b) Lateral pressure distribution diagrams
Figure 19.16 Typical range in strip reinforcement spacing for reinforced earth walls (Bowles, 1996)

7. Surcharges are allowed on the backfill. The surcharges may be permanent (such as a roadway) or temporary.
   a. Temporary surcharges within the reinforcement zone will increase the lateral pressure on the facing unit which in turn increases the tension in the reinforcements, but does not contribute to reinforcement stability.
   b. Permanent surcharges within the reinforcement zone will increase the lateral pressure and tension in the reinforcement and will contribute additional vertical pressure for the reinforcement friction.
   c. Temporary or permanent surcharges outside the reinforcement zone contribute lateral pressure which tends to overturn the wall.

8. The total length $L$ of the reinforcement goes beyond the failure plane $AC$ by a length $L_e$. Only length $L_e$ (effective length) is considered for computing frictional resistance. The length $L_R$ lying within the failure zone will not contribute for frictional resistance (Fig. 19.15a).

9. For the propose of design the total length $L$ remains the same for the entire height of wall $H$. Designers, however, may use their discretion to curtail the length at lower levels. Typical ranges in reinforcement spacing are given in Fig. 19.16.

19.10 DESIGN METHOD

The following forces are considered:

1. Lateral pressure on the wall due to backfill
2. Lateral pressure due to surcharge if present on the backfill surface.
3. The vertical pressure at any depth \( z \) on the strip due to
   a) overburden pressure \( p_o \) only
   b) overburden pressure \( p_o \) and pressure due to surcharge.

**Lateral Pressure**

**Pressure due to Overburden**

Lateral earth pressure due to overburden

\[
\text{At depth } z \quad p_a = p_o z K_A = \gamma z K_A \tag{19.11a}
\]

\[
\text{At depth } H \quad p_a = p_o H K_A = \gamma H K_A \tag{19.11b}
\]

Total active earth pressure

\[
p_a = \frac{1}{2} \gamma H^2 K_A \tag{19.12}
\]

**Pressure Due to Surcharge (a) of Limited Width, and (b) Uniformly Distributed**

(a) From Eq. (11.69)

\[
q_h = \frac{2q}{\pi} (\beta - \sin \beta \cos 2\alpha) \tag{19.13a}
\]

(b) \( q_h = q H K_A \) \tag{19.13b}

Total lateral pressure due to overburden and surcharge at any depth \( z \)

\[
p_h = p_a + q_h = (\gamma z K_A + q_h) \tag{19.14}
\]

**Vertical pressure**

Vertical pressure at any depth \( z \) due to overburden only

\[
p_o = \gamma z \tag{19.15a}
\]

due to surcharge (limited width)

\[
\Delta q = \frac{q B}{B + z} \tag{19.15b}
\]

where the 2:1 (2 vertical : 1 horizontal) method is used for determining \( \Delta q \) at any depth \( z \).

Total vertical pressure due to overburden and surcharge at any depth \( z \)

\[
\overline{p}_o = p_o + \Delta q \tag{19.15c}
\]

**Reinforcement and Distribution**

Three types of reinforcements are normally used. They are

1. Metal strips
2. Geotextiles
Galvanized steel strips of widths varying from 5 to 100 mm and thickness from 3 to 5 mm are generally used. Allowance for corrosion is normally made while deciding the thickness at the rate of 0.001 in. per year and the life span is taken as equal to 50 years. The vertical spacing may range from 20 to 150 cm (8 to 60 in.) and can vary with depth. The horizontal lateral spacing may be on the order of 80 to 150 cm (30 to 60 in.). The ultimate tensile strength may be taken as equal to 240 MPa (35,000 lb/in.$^2$). A factor of safety in the range of 1.5 to 1.67 is normally used to determine the allowable steel strength $f_a$.

Figure 19.16 depicts a typical arrangement of metal reinforcement. The properties of geotextiles and geogrids have been discussed in Section 19.7. However, with regards to spacing, only the vertical spacing is to be considered. Manufacturers provide geotextiles (or geogrids) in rolls of various lengths and widths. The tensile force per unit width must be determined.

### Length of Reinforcement

From Fig. 19.15(a)

$$L = L_R + L_e = L_R + L_1 + L_2$$

where 
- $L_R = (H - z) \tan(45° - \phi/2)$
- $L_e =$ effective length of reinforcement outside the failure zone
- $L_1 =$ length subjected to pressure $(p_o + \Delta q) = \bar{p}_o$
- $L_2 =$ length subjected to $p_o$ only.

### Strip Tensile Force at any Depth $z$

The equation for computing $T$ is

$$T = p_h \times h \times s / \text{strip} = (\gamma z K_A + q_h) h \times s$$

(19.17a)

The maximum tie force will be

$$T(\max) = (\gamma H K_A + q_{bh\max}) h \times s$$

(19.17b)

where
- $p_h = \gamma z K_A + q_h$
- $q_h =$ lateral pressure at depth $z$ due to surcharge
- $q_{bh\max} = q_h$ at depth $H$
- $h =$ vertical spacing
- $s =$ horizontal spacing

$$T = P_a + P_q$$

(19.18)

where $P_a = 1/2 \gamma H^2 K_A$—Rankine’s lateral force
- $P_q =$ lateral force due to surcharge

### Frictional Resistance

In the case of strips of width $b$ both sides offer frictional resistance. The frictional resistance $F_R$ offered by a strip at any depth $z$ must be greater than the pullout force $T$ by a suitable factor of safety. We may write

$$F_R = 2b[(p_o + \Delta q)L_1 + p_o L_2] \tan \delta \leq TF_s$$

(19.19)

or

$$F_R = 2b[\bar{p}_o L_1 + p_o L_2] \tan \delta \leq TF_s$$

(19.20)

where $F_s$ may be taken as equal to 1.5.
The friction angle $\delta$ between the strip and the soil may be taken as equal to $\phi$ for a rough strip surface and for a smooth surface $\delta$ may lie between 10 to 25°.

**Sectional Area of Metal Strips**

Normally the width $b$ of the strip is assumed in the design. The thickness $t$ has to be determined based on $T$ (max) and the allowable stress $f_a$ in the steel. If $f_y$ is the yield stress of steel, then

$$f_a = \frac{f_y}{F_s(\text{steel})}$$  \hspace{1cm} (19.21)

Normally $F_s(\text{steel})$ ranges from 1.5 to 1.67. The thickness $t$ may be obtained from

$$t = \frac{T(\text{max})}{bf_a}$$  \hspace{1cm} (19.22)

The thickness of $t$ is to be increased to take care of the corrosion effect. The rate of corrosion is normally taken as equal to 0.001 in/yr for a life span of 50 years.

**Spacing of Geotextile Layers**

The tensile force $T$ per unit width of geotextile layer at any depth $z$ may be obtained from

$$T = p_hh = (\gamma z K_A + q_h)h$$  \hspace{1cm} (19.23)

where $q_h$ = lateral pressure either due to a stripload or due to uniformly distributed surcharge.

The maximum value of the computed $T$ should be limited to the allowable value $T_a$ as per Eq. (19.9). As such we may write Eq. (19.23) as

$$T_a = TF_s = (\gamma z K_A + q_h)hF_s$$  \hspace{1cm} (19.24)

or

$$h = \frac{T_a}{(\gamma z K_A + q_h)F_s} = \frac{T_a}{p_hF_s}$$  \hspace{1cm} (19.25)

where $F_s$ = factor of safety (1.3 to 1.5) when using $T_a$.

Equation (19.25) is used for determining the vertical spacing of geotextile layers.

**Frictional Resistance**

The frictional resistance offered by a geotextile layer for the pullout force $T_a$ may be expressed as

$$F_R = 2[\gamma z(\Delta q)L_1 + \gamma z L_z] \tan \delta \geq T_a F_s$$  \hspace{1cm} (19.26)

Equation (19.26) expresses frictional resistance per unit width and both sides of the sheets are considered.

**Design with Geogrid Layers**

A tremendous number of geogrid reinforced walls have been constructed in the past 10 years (Koerner, 1999). The types of permanent geogrid reinforced wall facings are as follows (Koerner, 1999):

1. **Articulated precast panels** are discrete precast concrete panels with inserts for attaching the geogrid.
2. **Full height precast panels** are concrete panels temporarily supported until backfill is complete.
3. **Cast-in-place concrete panels** are often wrap-around walls that are allowed to settle and, after 1/2 to 2 years, are covered with a cast-in-place facing panel.
4. **Masonry block facing walls** are an exploding segment of the industry with many different types currently available, all of which have the geogrid embedded between the blocks and held by pins, nubs, and/or friction.

5. **Gabion facings** are polymer or steel-wire baskets filled with stone, having a geogrid held between the baskets and fixed with rings and/or friction.

The frictional resistance offered by a geogrid against pullout may be expressed as (Koerner, 1999)

\[
F_R = 2C_i C_r l_p p_o \tan \phi \geq TF_s
\]  

where \( C_i \) = interaction coefficient = 0.75 (may vary)

\( C_r \) = coverage ratio = 0.8 (may vary)

All the other notations are already defined. The spacing of geogrid layers may be obtained from

\[
h = \frac{T_o C_r}{P_h}\]  

where \( p_h \) = lateral pressure per unit length of wall.

### 19.11 EXTERNAL STABILITY

The MSE wall system consists of three zones. They are

1. The reinforced earth zone.
2. The backfill zone.

![Diagram of MSE wall system](image)

- (a) Overturning considerations
- (b) Sliding considerations
- (c) Foundation considerations

**Figure 19.17** External stability considerations for reinforced earth walls
3. The foundation soil zone.

The reinforced earth zone is considered as the wall for checking the internal stability whereas all three zones are considered for checking the external stability. The soils of the first two zones are placed in layers and compacted whereas the foundation soil is a normal one. The properties of the soil in each of the zones may be the same or different. However, the soil in the first two zones is normally a free draining material such as sand.

It is necessary to check the reinforced earth wall (width = B) for external stability which includes overturning, sliding and bearing capacity failure. These are illustrated in Fig. 19.17. Active earth pressure of the backfill acting on the internal face AB of the wall is taken in the stability analysis. The resultant earth thrust $P_a$ is assumed to act horizontally at a height $H/3$ above the base of the wall. The methods of analysis are the same as for concrete retaining walls.

**Example 19.2**

A typical section of a retaining wall with the backfill reinforced with metal strips is shown in Fig. Ex. 19.2. The following data are available:

- Height $H = 9$ m; $b = 100$ mm; $t = 5$ mm; $f_y = 240$ MPa; $F_s$ for steel = 1.67; $F_s$ on soil friction = 1.5; $\phi = 36^\circ$; $\gamma = 17.5$ kN/m$^3$; $\delta = 25^\circ$; $h \times s = 1 \times 1$ m.

Required:

(a) Lengths $L$ and $L_e$ at varying depths.

(b) The largest tension $T$ in the strip.

![Figure Ex. 19.2](image-url)
(c) The allowable tension in the strip.
(d) Check for external stability.

**Solution**

From Eq. (19.17a), the tension in a strip at depth $z$ is

$$T = \gamma z K_A s$$

where $\gamma = 17.5\ kN/m^3$, $K_A = \tan^2(45° - 36/2) = 0.26$, $s = 1m$; $h = 1\ m$.

Substituting

$$T = 17.5 \times 0.26 (1) \ [1] \ z = 4.55z\ kN/strip.$$  

$$L_e = \frac{F_e T}{2 \gamma z b \tan \delta} = \frac{1.5 \times 4.55z}{2 \times 17.5 \times 0.1 \times 0.47 \times z} = 4.14\ m$$

This shows that the length $L_e = 4.14\ m$ is a constant with depth. Fig. Ex. 19.2 shows the positions of $L_e$ for strip numbers 1, 2, ... 9. The first strip is located 0.5\ m below the backfill surface and the 9th at 8.5\ m below with spacings at 1\ m apart. Tension in each of the strips may be obtained by using the equation $T = 4.55z$. The total tension $\Sigma T$ as computed is

$$\Sigma T = 184.29\ kN/m\ since\ s = 1\ m.$$  

As a check the total active earth pressure is

$$P_a = \frac{1}{2} \gamma H^2 K_A = \frac{1}{2} \times 17.5 \times 9^2 \times 0.26 = 184.28\ kN/m = \Sigma T.$$  

The maximum tension is in the 9th strip, that is, at a depth of 8.5\ m below the backfill surface.

Hence

$$T = \gamma z K_A s h = 17.5 \times 8.5 \times 0.26 \times 1 \times 1 = 38.68\ kN/strip.$$  

The allowable tension is

$$T_a = f_ad$$

where $f_a = \frac{240 \times 10^3}{1.67} = 143.7 \times 10^3\ kN/m^2$.

Substituting $T_a = 143.7 \times 10^3 \times 0.005 \times 0.1 = 72\ kN > T$ – OK.

The total length of strip $L$ at any depth $z$ is

$$L = L_R + L_e = (H - z) \tan (45° - \phi/2) + 4.14 = 0.51 (9 - z) + 4.14\ m$$

where $H = 9\ m$.

The lengths as calculated have been shown in Fig. Ex. 19.2. It is sometimes convenient to use the same length $L$ with depth or stepped in two or more blocks or use a linear variation as shown in the figure.

**Check for External Stability**

**Check of bearing capacity**

It is necessary to check the base of the wall with the backfill for the bearing capacity per unit length of the wall. The width of the wall may be taken as equal to 4.5\ m (Fig. Ex. 19.2). The procedure as explained in Chapter 12 may be followed. For all practical purposes, the shape, depth, and inclination factors may be taken as equal to 1.
Check for sliding resistance

\[ F_s = \frac{\text{Sliding resistance } F_R}{\text{Driving force } P_a} \]

where \( F_R = W \tan \delta = \frac{4.5 + 8.5}{2} \times 17.5 \times 9 \tan 36^\circ \)
\[ = 1024 \times 0.73 = 744 \text{ kN} \]

where \( \delta = \phi = 36^\circ \) for the foundation soil, and \( W = \) weight of the reinforced wall

\( P_a = 184.28 \text{ kN} \)

\[ F_s = \frac{744}{184.28} = 4 > 1.5 \quad \text{OK} \]

Check for overturning

\[ F_s = \frac{M_R}{M_o} \]

From Fig. Ex. 19.2 taking moments of all forces about \( O \), we have

\[ M_R = 4.5 \times 9 \times 17.5 \times \frac{4.5}{2} + \frac{1}{2} \times 9 \times (8.5 - 4.5) (4.5 \times \frac{4}{3}) \times 17.5 \]
\[ = 1595 + 1837 = 3432 \text{ kN-m} \]

\[ M_o = P_a \times \frac{H}{3} = 184.28 \times \frac{9}{3} = 553 \text{ kN-m} \]

\[ F_s = \frac{3432}{553} = 6.2 > 2 \quad \text{OK} \]

Example 19.3

A section of a retaining wall with a reinforced backfill is shown in Fig. Ex. 19.3. The backfill surface is subjected to a surcharge of 30 kN/m². Required:

(a) The reinforcement distribution.
(b) The maximum tension in the strip.
(c) Check for external stability.

Given: \( b = 100 \text{ mm}, t = 5 \text{ mm}, f_a = 143.7 \text{ MPa}, c = 0, \phi = 36^\circ, \delta = 25^\circ, \gamma = 17.5 \text{ kN/m}^3, s = 0.5 \text{ m}, \text{ and } h = 0.5 \text{ m}. \)

Solution

From Eq. (19.17a)

\[ T = (\gamma z K_a + q_h) h \times s = (p_a + q_h) A_c \]

where \( \gamma = 17.5 \text{ kN/m}^3, K_a = 0.26, A_c = h \times s = (0.5 \times 0.5) \text{ m}^2 \)

From Eq. (19.13a)

\[ q_h = \frac{2q_s}{\pi} [\beta \sin \beta \cos 2\alpha] \]
Refer to Fig. Ex. 19.3 for the definition of $\alpha$ and $\beta$.

$q_s = 30 \text{ kN/m}^2$

The procedure for calculating length $L$ of the strip for one depth $z = 1.75 \text{ m}$ (strip number 4) is explained below. The same method is valid for the other strips.

**Strip No. 4. Depth $z = 1.75 \text{ m}$**

$$p_a = \gamma z K_A = 17.5 \times 1.75 \times 0.26 = 7.96 \text{ kN/m}^2$$

From Fig. Ex. 19.3, $\beta = 19.07^\circ = 0.3327 \text{ radians}$

$\alpha = 29.74^\circ$

$$q_s = 30 \text{ kN/m}^2$$

$$q_h = \frac{2 \times 30}{3.14} \left(0.3327 - \sin 19.07^\circ \cos 59.5^\circ\right) = 3.19 \text{ kN/m}^2$$

Figure Ex. 19.3 shows the surcharge distribution at a 2 (vertical) to 1 (horizontal) slope. Per the figure at depth $z = 1.75 \text{ m}$, $L_4 = 1.475 \text{ m}$ from the failure line and $L_R = (H - z) \tan (45^\circ - \phi/2) = 2.75 \tan (45^\circ - 36^\circ/2) = 1.4 \text{ m}$ from the wall to the failure line. It is now necessary to determine $L_2$ (Refer to Fig. 19.15a).
Chapter 19

Now \( T = (7.96 + 3.19) \times 0.5 \times 0.5 = 2.79 \text{ kN/strip}. \)

The equation for the frictional resistance per strip is

\[
F_R = 2b (\gamma c + \Delta q) L_1 \tan \delta + (\gamma z L_2 \tan \delta) 2b
\]

From the 2:1 distribution \( \Delta q \) at \( z = 1.75 \text{ m} \) is

\[
\Delta q = \frac{Q}{B + z} = \frac{30 \times 1}{1 + 1.75} = 10.9 \text{ kN/m}^2
\]

\( p_o = 17.5 \times 1.75 = 30.63 \text{ kN/m}^2 \)

Hence \( \overline{p_o} = 10.9 + 30.63 = 41.53 \text{ kN/m}^2 \)

Now equating frictional resistance \( F_R \) to tension in the strip with \( F_s = 1.5 \), we have

\( F_R = 1.5 T \). Given \( b = 100 \text{ mm}. \) Now from Eq. (19.20)

\[
F_R = 2b \tan \delta (\overline{p_o} L_1 + p_o L_2) = 1.5 T
\]

Substituting and taking \( \delta = 25^\circ \), we have

\[
2 \times 0.1 \times 0.47 \{41.53 \times 1.475 + 30.63 L_2\} = 1.5 \times 2.79
\]

Simplifying

\[
L_2 = -0.546 \text{ m} = 0
\]

Hence \( L_e = L_1 + 0 = 1.475 \text{ m} \)

\( L = L_R + L_e = 1.4 + 1.475 = 2.875 \text{ m} \)

\( L \) can be calculated in the same way at other depths.

Maximum tension \( T \)

The maximum tension is in strip number 9 at depth \( z = 4.25 \text{ m} \)

Allowable \( T_a = f_{ta} b t = 143.7 \times 10^3 \times 0.1 \times 0.005 = 71.85 \text{ kN} \)

\( T = (\gamma z K_A + q_h) sh \)

where \( \gamma z K_A = 17.50 \times 4.25 \times 0.26 = 19.34 \text{ kN/m}^2 \)

\( q_h = 0.89 \text{ kN/m}^2 \) from equation for \( q_h \) at depth \( z = 4.25 \text{ m} \).

Hence \( T = (19.34 + 0.89) \times 1/2 	imes 1/2 = 5.05 \text{ kN/strip} < 71.85 \text{ kN} - \text{OK} \)

---

**Example 19.4 (Koerner, 1999)**

Figure Ex. 19.4 shows a section of a retaining wall with geotextile reinforcement. The wall is backfilled with a granular soil having \( \gamma = 18 \text{ kN/m}^3 \) and \( \phi = 34^\circ \).

A woven slit-film geotextile with warp (machine) direction ultimate wide-width strength of 50 kN/m and having \( \delta = 24^\circ \) (Table 19.3) 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 to handle the weft direction. A factor of safety of 1.4 is to be used along with site-specific reduction factors (Table 19.4).

Required:

(a) Spacing of the individual layers of geotextile.
(b) Determination of the length of the fabric layers.
Concrete and Mechanically Stabilized Earth Retaining Walls

(c) Check the overlap.

(d) Check for external stability.

The backfill surface carries a uniform surcharge dead load of 10 kN/m²

Solution

(a) The lateral pressure \( p_h \) at any depth \( z \) is expressed as

\[
p_h = p_a + q_h
\]

where \( p_a = \gamma z K_A \), \( q_h = q K_A \), \( K_A = \tan^2 (45° - 36°/2) = 0.26 \)

Substituting

\[
p_h = 18 \times 0.26 z + 0.26 \times 10 = 4.68 z + 2.60
\]

From Eq. (19.9), the allowable geotextile strength is

\[
T_a = T_u \times \frac{1}{RF_{ID} \times RF_{CR} \times RF_{CD} \times RF_{BD}}
\]

\[
= 50 \times \frac{1}{1.2 \times 2.5 \times 1.15 \times 1.1} = 13.2 \text{ kN/m}
\]
From Eq. (19.17a), the expression for allowable stress in the geotextile at any depth $z$ may be expressed as

$$T = T_a = p_h h F_s$$

$$h = \frac{T_a}{p_h F_s}$$

where $h =$ vertical spacing (lift thickness)

$T_a =$ allowable stress in the geotextile

$p_h =$ lateral earth pressure at depth $z$

$F_s =$ factor of safety = 1.4

Now substituting

$$h = \frac{13.2}{[4.68(z) + 2.60]1.4} = \frac{13.2}{6.55(z) + 3.64}$$

At $z = 6\text{m}$,

$$h = \frac{13.2}{6.55 \times 6 + 3.64} = 0.307 \text{ m or say 0.30 m}$$

At $z = 3.3\text{m}$,

$$h = \frac{13.2}{6.55 \times 3.3 + 3.64} = 0.52 \text{ m or say 0.50 m}$$

At $z = 1.3\text{m}$,

$$h = \frac{13.2}{6.55 \times 1.3 + 3.64} = 1.08 \text{ m, but use 0.65 m for a suitable distribution.}$$

The depth 3.3 m or 1.3 m are used just as a trial and error process to determine suitable spacings. Figure Ex. 19.4 shows the calculated spacings of the geotextiles.

(b) *Length of the Fabric Layers*

From Eq. (19.26) we may write

$$L_e = \frac{T F_s}{2 \gamma z \tan \delta} = \frac{p_h h F_s}{2 \gamma z \tan \delta} = \frac{h (4.68z + 2.60)1.4}{2 \times 18 \ z \tan 24^\circ} = L_e = \frac{h (6.55z + 3.64)}{16z}$$

From Fig. (19.15) the expression for $L_R$ is

$$L_R = (H - z) \tan (45^\circ - \phi/2) = (H - z) \tan (45^\circ - 36/2) = (6.0 - z) (0.509)$$

The total length $L$ is

$$L = L_R + L_e$$
The computed $L$ and suggested $L$ are given in a tabular form below.

<table>
<thead>
<tr>
<th>Layer No</th>
<th>Depth $z$ (m)</th>
<th>Spacing $h$ (m)</th>
<th>$L_e$ (m)</th>
<th>$L_e$ (min) (m)</th>
<th>$L_R$ (m)</th>
<th>$L$ (cal) (m)</th>
<th>$L$ (suggested) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.65</td>
<td>0.65</td>
<td>0.49</td>
<td>1.0</td>
<td>2.72</td>
<td>3.72</td>
<td>4.0</td>
</tr>
<tr>
<td>2</td>
<td>1.30</td>
<td>0.65</td>
<td>0.38</td>
<td>1.0</td>
<td>2.39</td>
<td>3.39</td>
<td>–</td>
</tr>
<tr>
<td>3</td>
<td>1.80</td>
<td>0.50</td>
<td>0.27</td>
<td>1.0</td>
<td>2.14</td>
<td>3.14</td>
<td>–</td>
</tr>
<tr>
<td>4</td>
<td>2.30</td>
<td>0.50</td>
<td>0.26</td>
<td>1.0</td>
<td>1.88</td>
<td>2.88</td>
<td>3.0</td>
</tr>
<tr>
<td>5</td>
<td>2.80</td>
<td>0.50</td>
<td>0.25</td>
<td>1.0</td>
<td>1.63</td>
<td>2.63</td>
<td>–</td>
</tr>
<tr>
<td>6</td>
<td>3.30</td>
<td>0.50</td>
<td>0.24</td>
<td>1.0</td>
<td>1.37</td>
<td>2.37</td>
<td>–</td>
</tr>
<tr>
<td>7</td>
<td>3.60</td>
<td>0.30</td>
<td>0.14</td>
<td>1.0</td>
<td>1.22</td>
<td>2.22</td>
<td>–</td>
</tr>
<tr>
<td>8</td>
<td>3.90</td>
<td>0.30</td>
<td>0.14</td>
<td>1.0</td>
<td>1.07</td>
<td>2.07</td>
<td>–</td>
</tr>
<tr>
<td>9</td>
<td>4.20</td>
<td>0.30</td>
<td>0.14</td>
<td>1.0</td>
<td>0.92</td>
<td>1.92</td>
<td>2.0</td>
</tr>
<tr>
<td>10</td>
<td>4.50</td>
<td>0.30</td>
<td>0.14</td>
<td>1.0</td>
<td>0.76</td>
<td>1.76</td>
<td>–</td>
</tr>
<tr>
<td>11</td>
<td>4.80</td>
<td>0.30</td>
<td>0.14</td>
<td>1.0</td>
<td>0.61</td>
<td>1.61</td>
<td>–</td>
</tr>
<tr>
<td>12</td>
<td>5.10</td>
<td>0.30</td>
<td>0.14</td>
<td>1.0</td>
<td>0.46</td>
<td>1.46</td>
<td>–</td>
</tr>
<tr>
<td>13</td>
<td>5.40</td>
<td>0.30</td>
<td>0.14</td>
<td>1.0</td>
<td>0.31</td>
<td>1.31</td>
<td>–</td>
</tr>
<tr>
<td>14</td>
<td>5.70</td>
<td>0.30</td>
<td>0.14</td>
<td>1.0</td>
<td>0.15</td>
<td>1.15</td>
<td>–</td>
</tr>
<tr>
<td>15</td>
<td>6.00</td>
<td>0.30</td>
<td>0.13</td>
<td>1.0</td>
<td>0.00</td>
<td>1.00</td>
<td>–</td>
</tr>
</tbody>
</table>

It may be noted here that the calculated values of $L_e$ are very small and a minimum value of 1.0 m should be used.

(c) Check for the overlap
When the fabric layers are laid perpendicular to the wall, the adjacent fabric should overlap a length $L_o$. The minimum value of $L_o$ is 1.0 m. The equation for $L_o$ may be expressed as

$$L_o = \frac{h p_\delta F_s}{2 \times 2 \gamma z \tan \delta} = \frac{h [4.68(z) + 2.60] 1.4}{4 \times 18(z) \tan 24°}$$

The maximum value of $L_o$ is at the upper layer at $z = 0.65$. Substituting for $z$ we have

$$L_o = \frac{0.65 [4.68(0.65) + 2.60] 1.4}{4 \times 18(0.65) \tan 24°} = 0.25 \text{ m}$$

Since this value of $L_o$ calculated is quite low, use $L_o = 1.0$ m for all the layers.

(d) Check for external stability
The total active earth pressure $P_a$ is

$$P_a = \frac{1}{2} \gamma H^2 K_a = \frac{1}{2} \times 18 \times 6^2 \times 0.28 = 90.7 \text{ kN/m}$$

$$F_s = \frac{\text{Resisting moment } M_R}{\text{Driving moment } M_o} = \frac{W_1 l_1 + W_2 l_2 + W_3 l_3 + P_{s4}}{P_a (H/3)}$$

where $W_1 = 6 \times 2 \times 18 = 216 \text{ kN}$ and $l_1 = 2/2 = 1 \text{ m}$

$W_2 = (6 - 2.1) \times (3 - 2) \times (18) = 70.2 \text{ kN}$, and $l_2 = 2.5 \text{ m}$

$W_3 = (6 - 4.2) \times (4 - 3) \times (18) = 32.4 \text{ kN}$ and $l_3 = 3.5 \text{ m}$
Check for sliding

\[ F_s = \frac{213 \times (1) + 70.2 \times (2.5) + 32.4 \times (3.5)}{90.7 \times (2)} = 2.78 > 2 \text{ - OK} \]

Check for a foundation failure

Consider the wall as a surface foundation with \( D_f = 0 \). Since the foundation soil is cohesionless, we may write

\[ q_u = \frac{1}{2} \gamma B N_y \]

Use Terzaghi's theory. For \( \phi = 34^\circ \), \( N_y = 38 \), and \( B = 2m \)

\[ q_u = \frac{1}{2} \times 18 \times 2 \times 38 = 684 \text{ kN/m}^2 \]

The actual load intensity on the base of the backfill

\[ q(\text{actual}) = 18 \times 6 + 10 = 118 \text{ kN/m}^2 \]

\[ F_s = \frac{684}{118} = 5.8 > 3 \text{ which is acceptable} \]

Example 19.5 (Koerner, 1999)

Design a 7m high geogrid-reinforced wall when the reinforcement vertical maximum spacing must be 1.0 m. The coverage ratio is 0.80 (Refer to Fig. Ex. 19.5). Given: \( T_u = 156 \text{ kN/m}, C_r = 0.80, \) \( C_i = 0.75 \). The other details are given in the figure.

Solution

Internal Stability

From Eq. (19.14)

\[ p_h = (\gamma z K_A + q_h) = \gamma z K_A + q_s K_A \]

\[ K_A = \tan^2(45^\circ - \phi/2) = \tan^2(45^\circ - 32/2) = 0.31 \]

\[ p_h = (18 \times z \times 0.31) + (15 \times 0.31) = 5.58z + 4.65 \]
1. For geogrid vertical spacing.

Given $T_u = 156 \text{kN/m}$

From Eq. (19.10) and Table 19.5, we have

$$T_u = T_u \left[ \frac{1}{RF_{ID} \times RF_{CR} \times RF_{BD} \times RF_{CD}} \right]$$

$$T_u = 156 \left[ \frac{1}{1.2 \times 2.5 \times 1.3 \times 1.0} \right] = 40 \text{kN/m}$$

But use $T_{design} = 28.6 \text{kN/m}$ with $F_r = 1.4$ on $T_u$

From Eq. (19.28)

$$T_{design} = \frac{h p_h}{C_r}$$
\[ 28.6 = h \frac{5.58z + 4.65}{0.8} \]

or \[ h = \frac{22.9}{5.58z + 4.65} \]

Maximum depth for \( h = 1 \) m is

\[ 1.0 = \frac{22.9}{5.58z + 4.65} \text{ or } z = 3.27 \text{ m} \]

Maximum depth for \( h = 0.5 \) m

\[ 0.5 = \frac{22.9}{5.58z + 4.65} \text{ or } z = 7.37 \text{ m} \]

The distribution of geogrid layers is shown in Fig. Ex. 19.5.

2. Embedment length of geogrid layers.

From Eqs (19.27) and (19.24)

\[ 2C_1C_sL_eF_o \tan \phi = T_H F_s = p_h h F_s \]

Substituting known values

\[ 2 \times 0.75 \times 0.8 \times (L_e) \times 18 \times (z) \tan 32^\circ = h (5.58z + 4.65) 1.5 \]

Simplifying \( L_e = \frac{(0.62z + 0.516)h}{z} \)

The equation for \( L_R \) is

\[ L_R = (H - z) \tan(45^\circ - \phi/2) = (7 - z) \tan(45^\circ - 32/2) \]

\[ = 3.88 - 0.554(z) \]

From the above relationships the spacing of geogrid layers and their lengths are given below.

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Depth (m)</th>
<th>Spacing h (m)</th>
<th>( L_e ) (m)</th>
<th>( L_e ) (min) (m)</th>
<th>( L_R ) (m)</th>
<th>L (cal) (m)</th>
<th>L (required) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.75</td>
<td>0.75</td>
<td>0.98</td>
<td>1.0</td>
<td>3.46</td>
<td>4.46</td>
<td>5.0</td>
</tr>
<tr>
<td>2</td>
<td>1.75</td>
<td>1.00</td>
<td>0.92</td>
<td>1.0</td>
<td>2.91</td>
<td>3.91</td>
<td>5.0</td>
</tr>
<tr>
<td>3</td>
<td>2.75</td>
<td>1.00</td>
<td>0.81</td>
<td>1.0</td>
<td>2.36</td>
<td>3.36</td>
<td>5.0</td>
</tr>
<tr>
<td>4</td>
<td>3.25</td>
<td>0.50</td>
<td>0.39</td>
<td>1.0</td>
<td>2.08</td>
<td>3.08</td>
<td>5.0</td>
</tr>
<tr>
<td>5</td>
<td>3.75</td>
<td>0.50</td>
<td>0.38</td>
<td>1.0</td>
<td>1.80</td>
<td>2.80</td>
<td>5.0</td>
</tr>
<tr>
<td>6</td>
<td>4.25</td>
<td>0.50</td>
<td>0.37</td>
<td>1.0</td>
<td>1.52</td>
<td>2.52</td>
<td>5.0</td>
</tr>
<tr>
<td>7</td>
<td>4.75</td>
<td>0.50</td>
<td>0.36</td>
<td>1.0</td>
<td>1.25</td>
<td>2.25</td>
<td>5.0</td>
</tr>
<tr>
<td>8</td>
<td>5.25</td>
<td>0.50</td>
<td>0.36</td>
<td>1.0</td>
<td>0.97</td>
<td>1.97</td>
<td>5.0</td>
</tr>
<tr>
<td>9</td>
<td>5.75</td>
<td>0.50</td>
<td>0.36</td>
<td>1.0</td>
<td>0.69</td>
<td>1.69</td>
<td>5.0</td>
</tr>
<tr>
<td>10</td>
<td>6.25</td>
<td>0.50</td>
<td>0.35</td>
<td>1.0</td>
<td>0.42</td>
<td>1.42</td>
<td>5.0</td>
</tr>
<tr>
<td>11</td>
<td>6.75</td>
<td>0.50</td>
<td>0.35</td>
<td>1.0</td>
<td>0.14</td>
<td>1.14</td>
<td>5.0</td>
</tr>
</tbody>
</table>
External Stability
(a) Pressure distribution

\[ P_a = \frac{1}{2} \gamma H^2 K_A = \frac{1}{2} \times 17 \times 7^2 \tan^2(45^\circ - 30/2) = 138.8 \text{ kN/m} \]

\[ P_q = q_s K_A H = 15 \times 0.33 \times 7 = 34.7 \text{ kN/m} \]

Total \( = 173.5 \text{ kN/m} \)

1. Check for sliding (neglecting effect of surcharge)

\[ F_s = W \tan \delta = \gamma \times H \times L \tan 25^\circ = 18 \times 7 \times 5.0 \times 0.47 = 293.8 \text{ kN/m} \]

\[ P = P_a + P_q = 173.5 \text{ kN/m} \]

\[ F_s = \frac{293.8}{173.5} = 1.69 > 15 \text{ OK} \]

2. Check for overturning

Resisting moment \[ M_R = W \times \frac{L}{2} = 18 \times 7 \times 5 \times \frac{5}{2} = 1575 \text{ kN} \cdot \text{m} \]

Overturning moment \[ M_O = P_a \times \frac{H}{3} + P_q \times \frac{H}{2} \]

or \[ M_O = 138.8 \times \frac{7}{3} + 34.7 \times \frac{7}{2} = 445.3 \text{ kN} \cdot \text{m} \]

\[ F_s = \frac{1575}{445.3} = 3.54 > 2.0 \text{ OK} \]

3. Check for bearing capacity

Eccentricity \[ e = \frac{M_O}{W + q_s L} = \frac{445.3}{18 \times 7 \times 5 + 15 \times 5} = 0.63 \]

\[ e = 0.63 < \frac{L}{6} = \frac{5}{6} = 0.83 \text{ Ok} \]

Effective length \[ = L - 2e = 5 - 2 \times 0.63 = 3.74 \text{m} \]

Bearing pressure \[ = \left[ 18 \times 7 + 15 \right] \left( \frac{5}{3.74} \right) = 189 \text{ kN/m}^2 \]

\[ F_s = \frac{600}{189} = 3.17 > 3.0 \text{ OK} \]

19.12 EXAMPLES OF MEASURED LATERAL EARTH PRESSURES
Backfill Reinforced with Metal Strips
Laboratory tests were conducted on retaining walls with backfills reinforced with metal strips (Lee et al., 1973). The walls were built within a 30 in. x 48 in. x 2 in. wooden box. Skin elements
were made from 0.012 in aluminum sheet. The strips (ties) used for the tests were 0.155 in wide and 0.0005 in thick aluminum foil. The backfill consisted of dry Ottawa No. 90 sand. The small walls built of these materials in the laboratory were constructed in much the same way as the larger walls in the field. Two different sand densities were used: loose, corresponding to a relative density, $D_r = 20\%$, and medium dense, corresponding to $D_r = 63\%$, and the corresponding angles of internal friction were $31^\circ$ and $44^\circ$ respectively. SR-4 strain gages were used on the ties to determine tensile stresses in the ties during the tests.

Examples of the type of earth pressure data obtained from two typical tests are shown in Fig. 19.18. Data in Fig. 19.18(a) refer to a typical test in loose sand whereas data in 19.18(b) refer to test in dense sand. The ties lengths were different for the two tests. For comparison, Rankine lateral earth pressure variation with depth is also shown. It may be seen from the curve that the measured values of the earth pressures follow closely the theoretical earth pressure variation up to two thirds of the wall height but fall off comparatively to lower values in the lower portion.

Field Study of Retaining Walls with Geogrid Reinforcement

Field studies of the behavior of geotextile or geogrid reinforced permanent wall studies are fewer in number. Berg et al., (1986) reported the field behavior of two walls with geogrid reinforcement. One wall in Tucson, Arizona, 4.6 m high, used a cumulative reduction factor of 2.6 on ultimate strength for allowable strength $T_a$ and a value of 1.5 as a global factor of safety. The second wall was in Lithonia, Georgia, and was 6 m high. It used the same factors and design method. Fig. 19.19 presents the results for both the walls shortly after construction was complete. It may be noted that the horizontal pressures at various wall heights are overpredicted for each wall, that is, the wall designs that were used appear to be quite conservative.
Concrete and Mechanically Stabilized Earth Retaining Walls

877

Tolerances
soil weight = 6%
Field measurement = 20%

Field measurements

Rankine lateral pressure

Lateral pressure $\sigma_h$ (kPa)

10 20 30 40

Height of fill above load cell (m)

(a) Results of Tucson, Arizona, wall

(b) Results of Lithonia, Georgia, wall

Figure 19.19 Comparison of measured stresses to design stresses for two geogrid reinforced walls (Berg et al., 1986)

19.13 PROBLEMS

19.1 Fig. Prob. 19.1 gives a section of a cantilever wall. Check the stability of the wall with respect to (a) overturning, (b) sliding, and (c) bearing capacity.

Foundation soil
$\phi = 20^\circ$  $c = 30$ kN/m$^2$
$\gamma = 18.5$ kN/m$^3$

Figure Prob. 19.1

Foundation soil
$\gamma = 115$ lb/ft$^3$  $\phi = 36^\circ$
$\gamma_c = 150$ lb/ft$^3$

Figure Prob. 19.3
19.2 Check the stability of the wall given in Prob. 19.1 for the condition that the slope is horizontal and the foundation soil is cohesionless with $\phi = 30^\circ$. All the other data remain the same.

19.3 Check the stability of the cantilever wall given in Fig. Prob. 19.3 for (a) overturning, (b) sliding, and (c) bearing capacity failure.

19.4 Check the stability of the wall in Prob. 19.3 assuming (a) $\beta = 0$, and (b) the foundation soil has $c = 300$ lb/ft$^2$, $\gamma = 115$ lb/ft$^3$, and $\phi = 26^\circ$.

19.5 Fig. Prob. 19.5 depicts a gravity retaining wall. Check the stability of the wall for sliding, and overturning.

19.6 Check the stability of the wall given in Fig. Prob. 19.6. All the data are given on the figure.

19.7 Check the stability of the gravity wall given in Prob. 19.6 with the foundation soil having properties $\phi = 30^\circ$, $\gamma = 110$ lb/ft$^3$ and $c = 500$ lb/ft$^2$. All the other data remain the same.

19.8 Check the stability of the gravity retaining wall given in Fig. Prob. 19.8.

19.9 Check the stability of the gravity wall given in Prob. 19.8 for Coulomb’s condition. Assume $\delta = 2/3\phi$.

19.10 A typical section of a wall with granular backfill reinforced with metal strips is given in Fig. Prob. 19.10. The following data are available.

$H = 6$ m, $b = 75$ mm, $t = 5$ mm, $f_s = 240$ MPa, $F_s$ for steel = 1.75, $F_s$ on soil friction = 1.5.

The other data are given in the figure. Spacing: $h = 0.6$ m, and $s = 1$ m.

Required
(a) Lengths of tie at varying depths 
(b) Check for external stability

19.11 Solve the Prob. 19.10 with a uniform surcharge acting on the backfill surface. The intensity of surcharge is 20 kN/m$^2$.

19.12 Figure Prob. 19.12 shows a section of a MSE wall with geotextile reinforcement.

![Figure Prob. 19.5](image1)

![Figure Prob. 19.6](image2)
Concrete and Mechanically Stabilized Earth Retaining Walls

879

\[ \gamma = 115 \text{ lb/ft}^2 \]
\[ \phi = 35^\circ \]
\[ \gamma_c (\text{concrete}) = 150 \text{ lb/ft}^3 \]

Foundation soil: \( \gamma = 120 \text{ lb/ft}^3, \phi = 36^\circ \)

Figure Prob. 19.8

Required:
(a) Spacing of the individual layers of geotextile
(b) Length of geotextile in each layer
(c) Check for external stability

19.13 Design a 6 m high geogrid-reinforced wall (Fig. Prob. 19.13), where the reinforcement maximum spacing must be at 1.0 m. The coverage ratio \( C_r = 0.8 \) and the interaction coefficient \( C_i = 0.75 \), and \( T_a = 26 \text{ kN/m. (} T_{\text{design}}' \)

Given: Reinforced soil properties: \( \gamma = 18 \text{ kN/m}^3, \phi = 32^\circ \)
Foundation soil: \( \gamma = 17.5 \text{ kN/m}^3, \phi = 34^\circ \)

Metal strips \( b = 75 \text{ mm, } t = 5 \text{ mm} \)

Figure Prob. 19.10
Geotextile reinforcement

Backfill
Granular soil
\[ \gamma = 17.5 \text{kN/m}^3 \]
\[ \phi = 35^\circ \]
\[ c = 0 \]

Wall
\[ \phi = 36^\circ, \, \delta = 24^\circ \]
\[ c = 0 \]
\[ T_a = 12 \text{kN/m} \]
\[ \gamma = 17.5 \text{kN/m}^3 \]
\[ F_r = 1.5 \]

Foundation soil: \( \gamma = 18.5 \text{kN/m}^3 \), \( \phi = 36^\circ \).

Figure Prob. 19.12

Geogrid reinforcement

Backfill
\[ \gamma = 18 \text{kN/m}^3 \]
\[ \phi = 32^\circ \]
\[ \gamma = 18 \text{kN/m}^3 \]

\[ \gamma = 17.5 \text{kN/m}^3 \] \( \phi = 34^\circ \)

Figure Prob. 19.13