Chapter 7 – Soil Reinforcement

7.1 Introduction

Preface

• Reinforcement of Soil can generally be subdivided into 2 categories: *Reinforced Soils* and *In-situ Reinforcement*. The latter is often termed “soil nailing”. You should be aware of its existence (see page 745-748 of Codutto) but we will not be doing any design problems with soil nailing. Many “proprietary” systems and products are on the market – just like geosynthetics, a good rule is “caveat emptor”!!

• Soil Reinforcement may be made with a number of materials:
  - Woven Geotextiles
  - Polymer Geogrids of Polyethylene (usually uniaxial) & polypropylene (usually biaxial)
  - Polyester and Fiberglass Geogrids (often knitted or stitched at junctions) and usually coated with a polymer such as polyethylene or PVC or with bitumen.
  - Steel Strips (the original “Reinforced Earth™”)
  - Welded wire mesh

• Reinforced Soil Structures Fall, Broadly, into 3 classes:
  - Mechanically-Stabilized Earth (MSE) Walls
  - Reinforced Slopes and Embankments
  - Reinforced Foundations

• Selection of Reinforcement should include an evaluation of candidate products for **SURVIVABILITY** during construction. In some instances, considerations of survivability will dictate a heavier/more durable/stronger/higher-modulus product than would be required by considering reinforcement alone. Thus, *Design by Function* means the requirement of the critical function (reinforcement) must be used in design, AND survivability should also be considered.

• The manufacturer’s specified properties for the product should be reduced to account for damage during and after construction.
### Table 5-1
**Construction Survivability Ratings**
(after AASHTO, 1990 and 1996)

<table>
<thead>
<tr>
<th>Site Soil CBR at Installation</th>
<th>&lt; 1</th>
<th>1 to 2</th>
<th>&gt; 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equipment Ground Contact Pressure (kPa)</td>
<td>&gt; 350</td>
<td>&lt; 350</td>
<td>&gt; 350</td>
</tr>
<tr>
<td>Cover Thickness² (compacted, mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100⁰M</td>
<td>NR</td>
<td>NR</td>
<td>H</td>
</tr>
<tr>
<td>150⁰</td>
<td>NR</td>
<td>NR</td>
<td>H</td>
</tr>
<tr>
<td>300</td>
<td>NR</td>
<td>H</td>
<td>M</td>
</tr>
<tr>
<td>450</td>
<td>H</td>
<td>M</td>
<td>M</td>
</tr>
</tbody>
</table>

**NOTES:**
1. Assume saturated CBR unless construction scheduling can be controlled.
2. Maximum aggregate size not to exceed one-half the compacted cover thickness
3. For low-volume, unpaved roads (ADT < 200 vehicles).
4. The 100 mm minimum cover is limited to existing road bases and is not intended for use in new construction.
5. Maximum aggregate size ≤ 30 mm
6. NR = NOT RECOMMENDED; L = LOW; M = MODERATE; and H = HIGH
### Table 5-2
**Physical Property Requirements**
(after AASHTO, 1990 and 1996)

<table>
<thead>
<tr>
<th>Survivability Level</th>
<th>Grab Strength* ASTM D 4632 (N)</th>
<th>Puncture Resistance* ASTM D 4833 (N)</th>
<th>Tear Strength* ASTM D 4533 (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&lt; 50% Geotextile Elongation&lt;sup&gt;1,2&lt;/sup&gt;</td>
<td>&gt; 50% Geotextile Elongation&lt;sup&gt;1,2&lt;/sup&gt;</td>
<td>&lt; 50% Geotextile Elongation</td>
</tr>
<tr>
<td>High (Class 1)</td>
<td>1400</td>
<td>900</td>
<td>500</td>
</tr>
<tr>
<td>Moderate (Class 2)</td>
<td>1100</td>
<td>700</td>
<td>400</td>
</tr>
<tr>
<td>Low (Class 3)</td>
<td>800</td>
<td>500</td>
<td>300</td>
</tr>
</tbody>
</table>

### Additional Requirements

**Apparent Opening Size**
1. < 50% soil passing 0.075 mm sieve, AOS < 0.6 mm
2. > 50% soil passing 0.075 mm sieve, AOS < 0.3 mm

**Permeability**

k of the geotextile > k of the soil
(permissivity x the nominal geotextile thickness)

**Ultraviolet Degradation**

At 500 hours of exposure, 50% strength retained

**Geotextile Acceptance**

**Test Method**

- ASTM D 4751
- ASTM D 4491
- ASTM D 4355
- ASTM D 4759

### NOTES:

1. For the index properties, the first value of each set is for geotextiles which fail at less than 50% elongation, while the second value is for geotextiles which fail at greater than 50% elongation. Elongation is determined by ASTM D 4632.
2. Values shown are minimum roll average values. Strength values are in the weakest principal direction.
3. The values of the geotextile elongation do not relate to the allowable consolidation properties of the subgrade soil. These must be determined by a separate investigation.
4. AASHTO classification.
7.2 Reinforced Soil Structures

Mechanically-Stabilized Earth (MSE) Retaining Walls:

A large variety of design methods are available for the design of MSE walls or steep reinforced slopes. These depend, in part, upon the type of reinforcing used. Two methods will be presented here.

Consider first the “Limit Equilibrium” of a circular slip surface.
The Factor of Safety can therefore be expressed as:

\[
FS = \frac{\sum_{i=1}^{n} \left( (N_i - u_i \Delta x_i) \cdot \tan \Phi' + c' \Delta \ell_i \right) \cdot R + \sum_{i=1}^{m} T_i \cdot z_i}{\sum_{i=1}^{n} (W_i \sin \theta_i) R}
\]

we can simplify this to:

\[
FS = \frac{M_R + \sum_{i=1}^{m} T_i z_i}{M_D}
\]

where \(M_R\) = moments resisting due to soil strength and \(M_D\) = moments driving or causing failure due to gravity, seepage, seismic loads etc. \(T_i\) is the allowable reinforcement load, \(z_i\) is the appropriate moment arm and \(m\) the number of layers.

### 7.3 Limit Equilibrium Analysis

This approach is termed “Limit Equilibrium” Analysis and be carried out by hand calculations, but given the repetitive nature of the effort made to sum the forces and moments for each individual slice, stability analysis is much more efficiently carried out using a simple spreadsheet analysis or using specialty software.

It is important to be aware that Limit Equilibrium analysis tells you nothing about the displacements that may occur within and around the structure (in this it is analogous to the analysis of Bearing Capacity).

There is a variety of software available with which to carry out stability analysis for steep slopes or MSE walls. Much of this software originated in conventional analysis of natural (non-reinforced) slopes. A good example of software of this type is SLOPE/W™ from GeoSlope International of Calgary. Those of you taking CE466 next term will have the opportunity to use SLOPE/W™.
7.4 Reinforcement of Vertical Walls or steep slopes.

Failure modes:
- Sliding
- Overturning
- Bearing Capacity

External Stability
(as for any earth-retaining structure)

- Reinforcement Failure
- Pullout
- Failure of Reinforcement/Facing Connection

Internal Stability
(MSE retaining structures)

![Diagrams of failure modes]

(a) External stability

Spacing
Anchorage
Connections

(b) Internal stability

Figure 3.15 Elements of geogrid (or geotextile) reinforced wall design.
7.5 Jewell’s Method for geogrid reinforcement

1. Calculate allowable reinforcement load based on design strength and partial and global FS

2. Determine values of $K_{reqd}$, $(L/H)_{ovrl}$ and $(L/H)_{ds}$ from Jewell’s design charts, based on the height of the structure, strength of the soil, pore pressure conditions (i.e. drainage of backfill).

3. Calculate spacing at base of structure where the stresses are greatest:

   \[ S_v = \frac{T_{design}}{K_{reqd} \cdot \gamma \cdot z_{max}} \]

4. Determine the number of layers  \( n = H / S_v \)

5. Select reinforcement length

   If $(L/H)_{ovrl} > (L/H)_{ds}$ use const length = $(L/H)_{ovrl}$  otherwise use $(L/H)_{ds}$ or taper from $(L/H)_{ds}$ at toe to $(L/H)_{ovrl}$ at crest.
Figure 3.14(a) Steep reinforced slope design charts for \( r_u = u/y_z = 0.00 \), after Jewell [26].
Figure 3.14(b) Steep reinforced slope design charts for $r_u = u/\gamma z = 0.25$, after Jewell [26].
Figure 3.14(c) Steep reinforced slope design charts for $r_u = u/\gamma_z = 0.50$, after Jewell [26].
**Example 1:**

Design a 6 m high grid-reinforced retaining structure using a grid with an ultimate tensile strength of 144 kN/m and strength at 5% strain = 62 kN/m. Overall FS should be 1.4. Partial FS’s for the grids should be 4 (ultimate tensile failure) and 2 (excessive deformation at 5% strain). Assume free-draining backfill with $\gamma=18.5$ kN/m$^3$ and $\Phi'=32^\circ$. The structure needs to be essentially vertical ($\beta\geq85^\circ$).

1. Calculate allowable reinforcement strength

$$T_{\text{allow}} = \frac{144 \text{ kN/m}}{4} = 36 \text{ kN/m}$$ for tensile failure OR

$$T_{\text{allow}} = \frac{62 \text{ kN/m}}{2} = 31 \text{ kN/m}$$ at 5% strain  $\leftarrow$ use this (lower)

So applying the global factor for design, $T_{\text{des}} = \frac{31 \text{ kN/m}}{1.4} = 22 \text{ kN/m}$

2. Determine the necessary values from the appropriate design chart. Since the backfill is assumed to be free-draining, $u \approx 0$.

Define $r_u = \frac{u}{\gamma \cdot z}$ as shown in Fig 3.13 on page 6 of these notes.

Since $u=0$, $r_u=0$ $\Rightarrow$ use Chart 1.

This gives: $K_{\text{req}}=0.31$ ($K_{\text{req}}$ will be used to determine the overall amount of reinforcement required).

$$(L/R/H)_{\text{ovrl}} = 0.58$$ ($L/R/H)_{\text{ovrl}}$ will be used to determine the length of reinforcement required for overall stability

$$(L/R/H)_{\text{dsl}} = 0.19$$ ($L/R/H)_{\text{ds}}$ will be used to determine the length of reinforcement required against base sliding

3. Calculate the spacing $S_V$ required near the base where the stresses are greatest:

$$S_V = \frac{T_{\text{des}}}{K_{\text{req}} \cdot \gamma \cdot z_{\text{max}}} = \frac{22 \text{ kN/m}}{0.31 \cdot 18.5 \text{ kN/m}^3 \cdot 6 \text{ m}} = 0.64 \text{ m}$$

0.6m (2 ft is an appropriate reinforcement spacing – anything less than 1 or 1.5 feet starts to slow down construction excessively. If you calculate $S_V<0.3$ m, use stronger reinforcement, better backfill, flatten the slope etc.

It is simple to keep the reinforcement spacing constant (although the total amount of reinforcing is somewhat more than the minimum allowable). This is probably best practice, unless you are using excessively expensive reinforcement.

So $n = H / S_V = 6 \text{ m} / 0.6 \text{ m} \text{ i.e. 10 layers}$
4. Select reinforcement length:

If \((L/H)_{\text{ovrl}} > (L/H)_{\text{ds}}\) use const length = \((L/H)_{\text{ovrl}}\) otherwise use \((L/H)_{\text{ds}}\) or taper from \((L/H)_{\text{ds}}\) at toe to \((L/H)_{\text{ovrl}}\) at crest.

\((L/H)_{\text{ovrl}} = 0.58 > (L/H)_{\text{ds}} = 0.19\). So \(L = 0.58H = 3.5\) m for all reinforcing layers over the entire height.

**Example 2:**

Consider the same 6 m high grid-reinforced retaining structure. The available backfill is poorer quality, you estimate that after compaction, \(\gamma = 19\) kN/m\(^3\) and \(\Phi' \leq 30^\circ\) and, even more importantly, it has sufficient fines content that you anticipate there will be some pore-pressure buildup behind the wall. At the base of the wall, you estimate that the total head may be \(1/2\) the height of the wall.

Can you build the wall with the same reinforcing? If not, and if that is the strongest reinforcing that is readily available, how much would you have to flatten the slope? If you had to keep the slope \(\approx\) vertical, what strength reinforcing would you need?

1. First consider the pore pressure. At the base of the wall, \(u/\gamma_{W} = H/2 = 3\) m. So \(u/\gamma_{z} = 3m \cdot 9.8\) kN/m\(^3\) / \((19\) kN/m\(^3\) \cdot 6) = \(r_{U} = 0.26\). **Use Chart 2 \((r_{U} = 0.25)\)**

2. From chart 2, calculate \(K_{\text{req}}\) and \((L/R)\) using \(\Phi' \leq 30^\circ\), say \(29^\circ\)

\(K_{\text{req}} = 0.51\) (minimum amount of reinforcement required).

\((L_{R}/H)_{\text{ovrl}} = 0.72\) (length of reinforcement required for overall stability

\((L_{R}/H)_{\text{dsl}} = 0.5\) (length of reinforcement required against base sliding

3. As before, \(S_{V} = \frac{T_{\text{des}}}{K_{\text{req}} \cdot \gamma \cdot z_{\text{max}}} = \frac{22\) kN/m}{0.51 \cdot 19\) kN/m\(^3\) \cdot 6 m} = 0.38 m This represents a pretty thin lift of backfill. You could probably go with this spacing for a few layers at the base and then go to \(S_{V} = 0.6\) m as you build up.

To avoid small \(S_{V}\), slope would have to be flattened and for \(S_{V} = 0.6\) m, need

\[K_{\text{req}} = \frac{T_{\text{des}}}{S_{V} \cdot \gamma \cdot z_{\text{max}}} = \frac{22\) kN/m}{0.6 m \cdot 19\) kN/m\(^3\) \cdot 6 m} = 0.32\] from Chart 2, \(\beta < 70^\circ\)

4. To keep the reinforcement spacing to 0.6 m and the wall vertical, we would need \(T_{\text{des}} = S_{V} \cdot K_{\text{req}} \cdot \gamma \cdot z_{\text{max}} = 0.6 m \cdot 0.51 \cdot 19\) kN/m\(^3\) \cdot 6 m = 35 kN/m We would need to find a grid with \(T_{\text{allow}} = 35\) kN/m, or \(T_{5\%} \geq 70\) kN/m AND \(T_{\text{ULT}} \geq 140\) kN/m.
7.6 “Generic” method for reinforced soil structures

Consider the geotextile-reinforced structure shown below: Remember, the facing may be a simple lapping of geotextiles, or may be something more resistant to vandalism. Always consider drainage through the facing.
**Internal Stability**

We must consider first the “internal stability” of the MSE structure. That is to say, we consider the forces within the *reinforced mass of soil*.

**External Stability**

We must next consider the “external stability” of the MSE structure. Using the same techniques that we used for concrete retaining walls, we consider the That is to say, we consider the external forces acting on the *reinforced mass of soil*.

As before, we consider **Sliding**, **Rotation** and **Bearing Capacity**.
INTERNAL STABILITY

1. Determine the active earth pressure distribution on the wall:

\[ \sigma_{XA} = K_A \cdot \gamma_1 \cdot z \]

We will ignore pore pressure, for now, assuming that the wall will be completely drained. Usually, for MSE walls, one uses well-draining soils; including pore pressures in reinforced soil structures renders the problems quite complex and usually amenable to solution only by means of numerical methods.

2. The allowable strength of the geotextile reinforcement is \( \sigma_G \) (kN/m)

We can determine the required vertical spacing of the reinforcement at any depth using:

\[ S_V = \frac{\sigma_G}{\sigma_{XA} \cdot FS_B} = \frac{\sigma_G}{K_A \cdot \gamma_1 \cdot z \cdot FS_B} \]

FS\(_B\) is the Factor of Safety against failure of the reinforcement itself. This is usually = 1.4 - 1.5.

3. Determine the length of each layer of reinforcement using \( L = \ell_r + \ell_e \)

where:

\[ \ell_r = \frac{H - z}{\tan(45^\circ + \phi/2)} \]

\[ \ell_e = \frac{S_V \cdot K_A \cdot \gamma_1 \cdot z \cdot [FS_P]}{2 \cdot \gamma_1 \cdot z \cdot \tan \Phi_{if}} \]

where \( \ell_e \) = the length of embedment

\[ = \frac{S_V \cdot K_A \cdot [FS_P]}{2 \cdot \tan \Phi_{if}} \]

(\( \Phi_{if} \) = the interface shear strength, \( \approx \frac{1}{2} \phi' \) to \( \frac{2}{3} \phi' \) (usually, but be careful)

you may want to carry out direct-shear testing for this interface strength with your site soil and the various reinforcing products under consideration.)
4. Determine the required lap length $\ell_\ell$ using:

$$\ell_\ell = \frac{S_V \cdot \sigma_{x_A} \cdot FS_P}{4 \cdot \sigma_z \cdot \tan \Phi_{if}}$$

Note that $\ell_\ell$ should be maintained at least 1 m.

**EXTERNAL STABILITY**

Check FS against overturning, sliding and Bearing Capacity failure.

5. OVERTURNING

The overturning moment is calculated about the toe and $= P_A \cdot z'$

where the active thrust is $P_A = \int K_A \cdot \sigma_z \, dz$ and $P_A$ acts at $z'$ above the toe.

Remember to include any surcharge loads (acting on the surface) in the calculation of $\sigma_z$ and thus $P_A$.

The resisting moments include the weight of the MSE structure (as well as surcharge that may be applied to the upper surface of the structure).

$$M_R = \sum W_i \cdot x_i \quad \text{where} \quad x_i = \frac{1}{2} L_i$$
6. **SLIDING**

\[ FS = \{ \sum W_i \} \tan \Phi_{if} / P_A \]

7. Check for ultimate bearing capacity failure using the strength parameters for the foundation soil, \( \Phi_2, c_2 \).  

\[ q_{ult} = c_2 N_c + \frac{1}{2} \gamma_2 L_2' N_\gamma \]

Where \( L_2' = L_2 - 2e \), \( e = \) eccentricity \[ \frac{L_2}{2} - \frac{M_R - M_O}{\sum W_i + \text{surcharge (if any)}} \]

Vertical stress at \( z=H \) is \( \sigma_z(H) = \gamma_1 \cdot H + \text{surcharge (if any)} \)

\[ FS_{(BC)} = q_{ult} / \sigma_z(H) \]

Generally, \( FS \) should be >3 for overturning and sliding and 3-5 for B.C.

**Example 3:**

Design a 7 m high geotextile-reinforced retaining structure using a fabric with an ultimate tensile strength of 210 kN/m and strength at 5% strain = 70 kN/m.

FS should be 1.5 for reinforcement failure, pullout etc. Partial FS’s for the reinforcement should be 4 (ultimate tensile failure) and 2 (excessive deformation at 5% strain).

Assume good-quality free-draining backfill with \( \gamma=18 \text{ kN/m}^3 \) and \( \Phi'=33^\circ \). The structure needs to be essentially vertical.

1. Calculate allowable reinforcement strength

\[ T_{allow} = 210 \text{ kN/m} / 4 = 55 \text{ kN/m for tensile failure} \quad \text{OR} \]

\[ T_{allow} = 70 \text{ kN/m} / 2 = 35 \text{ kN/m at 5\% strain} \leftarrow \text{use this (lower)} \]

\[ K_A = \frac{1 - \sin \Phi}{1 + \sin \Phi} = 0.295 \]

2. Determine min. required SV (at base of wall where stresses are highest).

\[ S_V = \frac{\sigma_G}{\sigma_{x_A} \cdot FS_B} = \frac{35 \text{ kN/m}}{K_A \cdot \gamma_1 \cdot z \cdot FS_B} = \frac{35 \text{ kN/m}}{0.295 \cdot 7 \text{ m} \cdot 18.5 \text{ kN/m}^3 \cdot 1.5} = 0.6 \text{m} \]

Similarly, at mid-height, \( S_V = 1.2 \text{ m (at most). Can keep SV const or increase as you build up; usually you would use one spacing for the first bunch of layers, then increase the spacing to a new value for the remainder.} \]
3. Determine reinforcement length.

\[ \ell_r = \frac{H - z}{\tan(45° + \Phi/2)} \]

and depends only on height of wall and \( \Phi' \) of backfill at crest, \( \ell_r = 7\text{m} / \tan 61.5° = 3.8 \text{ m} \), at base \( \ell_r = 0 \)

Embedment Length, use \( \Phi_{if} = (\frac{1}{2} \text{ to } \frac{2}{3}) \Phi' \approx 20° \)

\[ \ell_e = \frac{S_V \cdot K_A \cdot z \cdot [FS_P]}{2 \cdot \gamma_1 \cdot \tan \Phi_{if}} = \frac{S_V \cdot K_A \cdot [FS_P]}{2 \cdot 	an \Phi_{if}} = \frac{0.6 \text{m} \cdot 0.295 \text{m} \cdot 1.5}{2 \cdot \tan 20°} = 0.4 \text{ m} \]

and similarly at mid-height, for \( SV=1.2 \text{ m}, \ \ell_e = 0.8 \text{m} \)

4. Check required overlap

\[ \ell_\ell = \frac{S_V \cdot \sigma_{x,A} \cdot FS_P}{4 \cdot \sigma_z \cdot \tan \Phi_{if}} = \frac{0.6 \text{m} \cdot 0.295 \cdot 7 \text{m} \cdot 18.5 \text{ kN/m}^3 \cdot 1.5}{4 \cdot 7 \text{m} \cdot 18.5 \text{ kN/m}^3 \cdot \tan(20°)} = 0.2 \text{ m} \text{ at base} \]

and similarly, at mid-height, for \( SV=1.2 \text{ m}, \ \ell_\ell = 0.4 \text{m} \). Use 1 m throughout.

5. Then External Stability is checked (overturning, sliding and B.C.) using our standard geotechnical approach.