The design of reinforced soil retaining walls using TENAX geogrids

Design Manual

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Reinforced soil retaining walls using TENAX geogrids

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1. Introduction

This manual provides a design methodology to include TENAX geogrid reinforcements into the soil for the purpose of building retaining walls having vertical face or nearly vertical face (> 80°). The inclusion of the geogrid reinforcements into the soil creates a reinforced composite structure, which has the ability to resist to high compressive and tensile stresses. The geogrid reinforcements improve the soil properties by preventing tensile failures.

The reinforced soil technique is a thousand of years old concepts, and has experienced different types of reinforcing materials, from bamboo to steel strips, from tree branches to geogrids. The synthetic types of reinforcement have the advantage of providing greater durability, strength, proven experience and finally a more theoretical design approach.

The major advantages of using TENAX geogrids for designing and building a reinforced soil retaining wall are that the construction is quite simple and rapid, experienced craftsmen are not required and the final overall structure is flexible and ductile allowing differential settlements of the base without failure.

This design manual covers the safe and economic design of vertical soil reinforced structures by the mean of TENAX geogrids.

2. TENAX Geogrids

TENAX geogrids are continuous grid structures with oval apertures, manufactured with the best durable polymers by a process of extrusion and longitudinal orientation. TENAX geogrids are made of high density polyethylene and they are chemically inert, unaffected by the U.V. rays and fully resistant to aging in the soil environment.

The TENAX geogrids are an integral structure without any weak point. The product is manufactured in a continuous process so that the geogrid junctions are extruded. Therefore the final product has the cross directional bars integrally connected to the longitudinal strands to form a geogrid with a monolithic structure that receives and transfers stresses to the reinforced soil by both passive resistance and friction mechanisms.

The soil stresses are transferred to the TENAX geogrid through bearing against the cross directional bars of the soil interlocked into the openings of the geogrids, and through shear at the soil-geogrid interface.

The high strength longitudinal strands have been engineered to provide high long term tensile forces for the overall life of the structure and high tensile modulus at low strain in order to be fully compatible with the soil modulus.

TENAX geogrids have been heavily tested for mechanical and performance properties in laboratories all over the world. TENAX geogrids have been installed in hundreds of retaining wall applications, showing superior performances, easiness of installation and savings when compared to any other wall systems.

3. Reinforced soil wall design theory

This design manual follows the design methodology of the “Tied back wedge analysis”, which yields the best evaluation of the behavior of soil-geogrid structures. This method is a safe and economical method that has been recommended by many authors.

The “Tied back wedge method” analyses the overall structure using the Limit Equilibrium approach, which allows the engineer to verify the “distance” from the failure point.

The design procedure consists of analyzing all the different types of possible failures in four consecutive steps (Figure 1):

a) External stability analysis

The reinforced soil-geogrid volume is assumed to act as a rigid block. This block is subject to the conventional retaining wall failure mechanisms such as: Sliding, Overturning and Bearing Capacity failure.

This design step will identify the dimensions of the area to be reinforced.
b) Internal stability analysis

This analysis is performed on the soil-geogrid volume to determine the required geogrid tensile strength, the minimum required number of geogrid layers and the minimum required length to ensure a rigid behavior in the reinforced block. Typical internal stability analyses are: the Geogrid layers layout, the Geogrid Overtension failure and the Geogrid Pullout failure.

c) Local stability analysis

This analysis is carried out for Segmental Retaining Walls to ensure that the column of concrete block units remains intact without bulging; local stability analysis are: facing connection, bulging and maximum unreinforced height.

d) Global stability analysis

This analysis is performed on the overall structure including the retained backfill and the foundation soil. This analysis should be performed according to the classical slope stability procedures, such as Bishop’s modified method of slices. The minimum recommended safety factor for this analysis ranges between 1.3 and 1.5.

Chapter 12, in the following describes in details the global stability analysis.

4. Definition of wall geometry

The wall geometry is defined by several parameters including the total height (h) the embedment height (d), the top slope angle (β), the surcharge load distribution (q).

The required wall embedment depth is determined according to the specific site conditions such as: the depth of the frost penetration, the type of slope at the wall toe, the presence of any shrink-swell clay soil in the foundation, the seismic activity of the area. The required depth is usually 0.50 m up to a value that is approximately 10% of the wall exposed height.

If the embedded height is kept exposed during the construction of the wall and then finally covered, the embedded height shall be added to the exposed wall height to calculate the total wall height (h). Unless the embedded height is immediately covered before reaching the top of the wall, then the exposed height is the total height. The total height of the wall is the design height that is used for calculations performed in this manual.
Reinforced soil retaining walls using TENAX geogrids

The presence of the top sloping backfill is taken into account by the calculation of the appropriate active earth pressure coefficient ($K_a$) and by using the height of the wall at the end of the reinforcement layers in the external backfill stability analysis. The top backfill slope highly influences the required length and number of the geogrids, and sometime is more convenient and safe to increase the wall height in order to decrease the top slope angle. This angle shall always be smaller than the fill soil friction angle, else this area must be reinforced with geogrids. This design procedure is however accurate for top slope angle lower than 20 deg. When a wall is in presence of a very long sloped backfill, a global stability analysis must be accurately performed. The surcharge loads are vertical and considered uniformly distributed over the overall length of the top surface. They range typically between 5 and 20 kPa. Point and line loads are more complex to handle and they are not discussed at this time in this manual.

The facing system is one of the key factors in designing a reinforced soil retaining wall. The facing system to be used with a reinforcing geogrid must be selected not only for its functionality, aesthetic, cost, easiness of installation, longevity, but very important is the type of connection system to the geogrids. The facing system must be placed on a solid base, such as a leveled reinforced concrete slab or a highly compacted free drainage crushed stone base. The thickness of the above bases ranges from 0.15 to 0.40 m for the concrete slab and from 0.30 to 0.60 m for the crushed stone base.

5. TENAX Nuraghe Retaining Wall System

TENAX NURAGHE is a segmental retaining wall System composed of concrete blocks, specifically developed for the face of the structure, and of geogrids for soil reinforcement (see Figure 2).

The concrete blocks, placed on well-compacted soil don't require any use of mortar, but each block is fixed to the adjacent ones, just thanks to their peculiar shape. The concrete blocks usually present passing holes, which are filled with soil when installed, increasing in this way the weight of the block and allowing the anchorage of TENAX geogrids.

The face of the blocks may be curved better aesthetical finishing. The blocks are self stable by gravity without soil pressure; therefore they don't require any form work for installation, since the blocks themselves act as form work for the construction of the geogrids reinforced soil block.

The TENAX Nuraghe retaining wall system can be built with any of the concrete blocks specifically developed for this application. In particular TENAX Nuraghe blocks shows in Figure 2 have been engineered for the use in association with TENAX geogrids.

The units have a notch 30 mm high on the top and the bottom faces. These notches increase the shear connection between successive vertical courses and help to ensure alignment during wall construction. On the block sides the blocks are connected by a male-female joint that allow them to rotate on the vertical axe thus allowing to follow low radius contours.

The two internal cavities must be filled with granular free drainage soil to allow water to pass through and full interlock with the geogrid.
6. Soil Characteristics

The geotechnical characteristics of the soils are defined by the moist unit weight, the angle of internal friction and the cohesion. These soil characteristics must be identified for the reinforced, the backfill and the foundation soils.

In the computation of the lateral soil stresses, the reinforced and backfill soil cohesion is neglected for safety considerations.

One of the main advantages of using TENAX geogrids for soil reinforcement is that they can be used with all the available on-site fill, from granular to fine soil. However special care shall be taken when working with a non free-draining soil. The pattern of the ground water must be identified and corrected if inside or near the reinforced volume. A drainage system shall be provided in the back of the reinforced area. This drainage system may be composed of a geocomposite layer, such as the TENAX TNT, and a collector pipe system or a free-drainage granular soil between two layers of nonwoven filter fabric. The system shall be designed in order to prevent the formation of any possible hydrostatic pressure. Additional drainage must be provided at the wall face, if the wall has been designed with impermeable facing units. In presence of water flow or runoff on the surface of the wall, a positive drainage of the wall face must be designed. A thick topsoil cover is a good solution to seal off major surface infiltration problems.

7. TENAX geogrid design characteristics

The key factors in soil reinforcement are the tensile strength of the reinforcing layers and its ability to transfer and receive stresses to and from the surrounding soil. TENAX geogrids have been engineered to interlock with the soil and to create a distribution of bearing members inside the soil structure that allow the soil to be reinforced. These bearing members are the geogrid cross directional bars. These bars are integrally connected with the longitudinal strands in order to transfer fully the soil stresses to the geogrid: no movements are possible between the bars and the strands. TENAX geogrids have a tensile junction strength that is always much higher than the design strength.

TENAX geogrids provide high coefficients of direct sliding and pullout in every soil, from fine to granular, from cohesive to frictional. These characteristics allow to build reinforced soil retaining walls having a smaller required reinforced length and to save in time and money during the excavation, compaction, earthmoving and installation.

The long term design strength of TENAX geogrid is established by intensive constant load tensile tests. These tests run for more than 10,000 hours and their results are extrapolated to a service life of more than 100 years.

The coefficient of soil-geogrid direct sliding \( C_{ds} \) is determined through intensive testing in a direct shear box apparatus of 0.30 m by 0.30 m of contact area. In this shear box, the performances of all the TENAX geogrids and all the representative types of soil have been analyzed and tested under different vertical stresses.

The results are expressed with a coefficient of sliding for every major class of soil.

Similar type of testing has been performed in a large pullout box to determine the pullout soil-geogrid coefficient \( C_{po} \).

When designing a reinforced soil retaining wall, it is important to distribute the reinforcing layers for the full height of the structure, having reinforcing layers spaced typically no more than 1.0 m from each other, otherwise it is possible to have areas not properly reinforced. The spacing between two geogrids layers increases with the quality and the grain size of the fill soil. For example, if we want to reinforce a poor soil, the engineer shall not select the strongest geogrid, but many layers of a lighter one because a higher number of reinforcing layers will provide better overall geogrid-soil interaction.

Sometime, the facing system causes the geogrid cover ratio to be lower than 100%. It might be the case of a timber retaining wall having the vertical posts behind the wall face. The geogrid cover ratio is the ratio between the area covered with geogrid and the total horizontal area to be reinforced.

This ratio shall be always greater than 75% in order to get the best performances.

The global sliding coefficient \( C_s \) for a geogrid failure plane is a function of the geogrid coverage:

\[
C_s = 1 - R_s \cdot (1 - C_{ds})
\]  

(1)

where:

\( C_s \) is the global sliding coefficient;
**Reinforced soil retaining walls using TENAX geogrids**

*Rc* is the geogrid coverage ratio; 

*Cs* is the soil-geogrid direct sliding coefficient.

For preliminary design with TENAX geogrid, we suggest to use the coefficients listed in Table 1, determined through intensive testing using different classes of soil.

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>0.90</td>
<td>1.00</td>
</tr>
<tr>
<td>Sand</td>
<td>0.85</td>
<td>0.95</td>
</tr>
<tr>
<td>Silt</td>
<td>0.75</td>
<td>0.85</td>
</tr>
<tr>
<td>Clay</td>
<td>0.70</td>
<td>0.80</td>
</tr>
</tbody>
</table>

Tab. 1a - Typical soil-geogrid direct sliding coefficient *Cds*, for TENAX mono-oriented geogrids.

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>0.90</td>
<td>1.50</td>
</tr>
<tr>
<td>Sand</td>
<td>0.85</td>
<td>1.20</td>
</tr>
<tr>
<td>Silt</td>
<td>0.75</td>
<td>1.00</td>
</tr>
<tr>
<td>Clay</td>
<td>0.70</td>
<td>0.90</td>
</tr>
</tbody>
</table>

Tab. 1b - Typical soil-geogrid pullout coefficient *Cpo*, for TENAX mono-oriented geogrids.

The coefficients in Table 1 shall be used to determine the resistant shear stresses with the following equations:

\[
\tau_{ds} = \sigma'_s \cdot C_{ds} \cdot \tan \phi'
\]

(2)

\[
\tau_{po} = 2 \cdot \sigma'_s \cdot C_{po} \cdot \tan \phi'
\]

(3)

### 8. Recommended safety factors

Different safety factors must be used to analyze and set up the distance from the “at failure” conditions, according to the “limit equilibrium theory”.

The recommended Safety Factor to design a typical reinforced soil vertical retaining wall with TENAX geogrids are listed in Table 2:

<table>
<thead>
<tr>
<th>Factor</th>
<th>Safety Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>FS Global Stability</td>
<td>1.30 ÷ 1.50</td>
</tr>
<tr>
<td>FS Wall Sliding</td>
<td>1.50</td>
</tr>
<tr>
<td>FS Wall Overtaking</td>
<td>2.00</td>
</tr>
<tr>
<td>FS Bearing Capacity Failure</td>
<td>2.00</td>
</tr>
<tr>
<td>FS Geogrid Overtension</td>
<td>1.50</td>
</tr>
<tr>
<td>FS Geogrid Pull-out Resistance</td>
<td>1.50</td>
</tr>
<tr>
<td>FS Facing Shear</td>
<td>1.00 ÷ 1.50</td>
</tr>
<tr>
<td>FS Geogrid Connection</td>
<td>1.00 ÷ 1.50</td>
</tr>
</tbody>
</table>

Tab. 2 - Recommended Factor of Safety

These safety factors must be tuned according to the specific site conditions such as the wall geometry, the soil types, the construction procedures, the overall life of the project and the criticality of the structure.

### 9. Design procedure for external stability analysis

The design of a geogrid reinforced soil wall is performed using the “Tied back wedge method”. This analysis is based preferably on the Rankine theory of earth pressure and stress distribution. This distribution is believed to be the one that better represents the behavior of the TENAX geogrids reinforced soil retaining wall.

During the external stability analysis, the passive resistance of the foundation soil at the wall toe and the backfill vertical driving forces are neglected and considered equal to zero for easiness and safety.

Before starting the design, the following information shall be known or determined (see Figure 3).
Reinforced soil retaining walls using TENAX geogrids

A) Input of wall geometry:

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Total wall height</td>
<td>h</td>
<td>(m)</td>
</tr>
<tr>
<td>Vertical surcharge load</td>
<td>q</td>
<td>(kPa)</td>
</tr>
<tr>
<td>Top slope angle</td>
<td>( \beta )</td>
<td>(deg)</td>
</tr>
<tr>
<td>Backfill wall height</td>
<td>H</td>
<td>(m)</td>
</tr>
<tr>
<td>Soil Compaction lift</td>
<td>s</td>
<td>(m)</td>
</tr>
<tr>
<td>Maximum geogrid spacing</td>
<td>M</td>
<td>(m)</td>
</tr>
<tr>
<td>First geogrid elevation</td>
<td>h(_1)</td>
<td>(m)</td>
</tr>
<tr>
<td>Facing system angle</td>
<td>( \omega )</td>
<td>(deg)</td>
</tr>
<tr>
<td>Inclination of wall base</td>
<td>( \alpha )</td>
<td>(deg)</td>
</tr>
</tbody>
</table>

B) Input of characteristics of the reinforced (r), backfill (b) and foundation (f) soils:

<table>
<thead>
<tr>
<th></th>
<th>( \gamma_r, \gamma_b, \gamma_f )</th>
<th>(kN/m(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Internal friction angles</td>
<td>( \phi_r, \phi_b, \phi_f )</td>
<td>(deg)</td>
</tr>
<tr>
<td>Cohesions</td>
<td>( \sigma_r, \sigma_b, \sigma_f )</td>
<td>(kPa)</td>
</tr>
<tr>
<td>Wall-soil friction angle</td>
<td>( \delta )</td>
<td>(deg)</td>
</tr>
</tbody>
</table>

C) Input of the design characteristics of TENAX geogrids:

<table>
<thead>
<tr>
<th>TENAX geogrid type</th>
<th>( t_1, t_2 )</th>
<th>(-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Long term design strength</td>
<td>LTDS(_1), LTDS(_2)</td>
<td>(kN/m)</td>
</tr>
<tr>
<td>Geogrid pullout coefficient</td>
<td>( C_{po} )</td>
<td>(-)</td>
</tr>
<tr>
<td>Geogrid sliding coefficient</td>
<td>( C_{sl} )</td>
<td>(-)</td>
</tr>
<tr>
<td>Geogrid coverage ratio</td>
<td>( R_c )</td>
<td>(-)</td>
</tr>
</tbody>
</table>

Fig. 3 - Wall geometry and soil data
9.1 Calculation of earth pressure coefficient

The active earth pressure coefficient \( K_a \) for a vertical retaining wall having a sloping top backfill angle \( \beta \) is given by:

\[
K_a = \frac{\cos \beta \cdot \cos \beta - \sqrt{[\cos \beta]^2 - (\cos \phi)^2}}{\cos \beta + \sqrt{[\cos \beta]^2 - (\cos \phi)^2}}
\]

(according to Rankine’s Theory)

or by:

\[
K_a = \frac{\cos^2 (\phi + \omega + \alpha)}{\cos^2 (\omega + \alpha) \cdot \cos (\omega + \alpha - \delta) \cdot \left[ 1 + \frac{\sin (\phi + \delta) \cdot \sin (\phi - \beta)}{\cos (\omega + \alpha - \delta) \cdot \cos (\omega + \alpha + \beta)} \right]}
\]

(according to Coulomb’s Theory)

Several authorities, including FHWA and AASHTO, recommend Rankine’s theory for internal stability analysis, while Coulomb's theory is recommended for external stability analysis by FHWA. Coulomb's theory gives the possibility to take into account the real geometry of the wall, including facing batter \( \omega \) and inclination of base \( \alpha \); furthermore, Rankine's earth theory has been shown to over estimate the lateral earth pressure (Simac et al., 1993).

The active earth pressure coefficient must be calculated for both the reinforced \( K_{ar} \) and backfill \( K_{ab} \) soil if they have different internal friction angles \( \phi_i \) and \( \phi_b \).

In order to provide always a conservative design, the Rankine’s earth pressure coefficient, from Eq. (4a), has been mostly used by TENAX Engineers.

Reference is now made to Figure 4.

The backfill wall height \( H \) for a top sloping wall is given by:

\[
H = h + L \cdot \tan \beta
\]

and the lateral active forces due to the backfill soil \( F_{db} \), the surcharge loads \( F_{qb} \) and the sum of the above \( F_{tb} \) are given by:

\[
F_{db} = \frac{1}{2} K_{ab} \cdot \gamma_b \cdot H^2
\]

\[
F_{qb} = q \cdot K_{ab} \cdot H
\]

\[
F_{tb} = F_{db} + F_{qb}
\]

or generally the above forces at a given vertical elevation \( y \) from the wall base are expressed by the following formulas:

\[
F_{db(y)} = \frac{1}{2} K_{ab} \cdot \gamma_b \cdot (H - y)^2
\]

\[
F_{qb(y)} = q \cdot K_{ab} \cdot (H - y)
\]

\[
F_{tb(y)} = F_{db(y)} + F_{qb(y)}
\]
The horizontal component of the above forces can be computed according to Rankine’s Earth Pressure Theory and Coulomb’s Earth Pressure Theory.

In the first case, the force due to soil weight and the force due to surcharge are inclined of the angle $E$ of the slope over the wall; the horizontal component is given by:

\[
F_{dbh} = F_{db} \cdot \cos \beta \tag{12}
\]

\[
F_{qbh} = F_{qb} \cdot \cos \beta \tag{13}
\]

In the second case the above forces are inclined of an angle $(\delta - \omega - \alpha)$; the horizontal component is given by:

\[
F_{dbh} = F_{db} \cdot \cos(\delta - \omega - \alpha) \tag{14}
\]

\[
F_{qbh} = F_{qb} \cdot \cos(\delta - \omega - \alpha) \tag{15}
\]

The total horizontal force $F_{bh}$ is:

\[
F_{bh} = F_{dbh} + F_{qbh} \tag{16}
\]

The global soil-geogrid direct sliding coefficient ($C_g$) is calculated using Eq. (1).

To simplify the computation process for determining the external stability safety factors, the following constants are defined:

- The reinforced ($W_r$) and backfill ($W_b$) soil volume weight per unit width:

\[
W_r = L \cdot h \cdot \gamma_r \tag{17}
\]

\[
W_b = \frac{1}{2} \gamma_r \cdot L^2 \cdot \tan \beta \tag{18}
\]

- The total vertical surcharge load per unit width ($Q$):

\[
Q = q \cdot L \tag{19}
\]
9.2 Wall sliding analysis along the base of the wall

The shear strength of the reinforced and foundation soils must be large enough to resist the horizontal stresses applied to the reinforced block by the backfill soil and by the external loads. A trial geogrid length and a first geogrid elevation shall be selected. The geogrid length shall be greater than the 60% of the wall height and the first geogrid elevation is typically between 0 and 0.40 m.

The safety factor against sliding failure \( FS_s \) along the wall base is given by:

\[
FS_s = \left( \frac{W_r + W_h + Q}{F_{shb}} \right) \cdot \tan \phi_k
\]

where:

\[
\phi_k = \min(\phi_r, \phi_f)
\]

If the wall is embedded since the starting of the construction, then

\[
\phi_k = \phi_f \quad (21 \text{ bis})
\]

9.3 Wall sliding analysis along the first geogrid layer

The sliding analysis shall be performed also at the first geogrid layer elevation \( (h_1) \) in order to verify if the reinforcing length is appropriate. To perform this analysis, we shall take into consideration the global soil/geogrid interaction coefficient.

The safety factor against sliding failure \( FS_s \) along the first geogrid layer is given by:

\[
FS_s = \left( \frac{W_r + W_h + Q - W_i}{F_{shb}(h_1)} \right) \cdot \tan \phi_k \cdot C_y
\]

where \( W_i \) is the reinforced soil weight per unit width between the wall base and the first geogrid layer and \( F_{shb}(h_1) \) is obtained from eq. (9) ÷ (16) with \( y = h_1 \).

\[
W_i = L \cdot h_1 \cdot \gamma
\]

9.4 Wall overturning analysis around the toe

The safety factor against overturning failure \( FS_o \) around the wall toe is determined by comparing the resisting moments, due to the soil weight \( (W_r) \) and the surcharge loads \( (W_h) \), and the driving moments due to the backfill driving forces \( (F_{shb}) \) e \( (F_{qshb}) \) (Figure 4). If the computed safety factor \( FS \) is lower than the required one, then the trial length shall be increased.

\[
FS_o = \left( \frac{3 \cdot W_r + 3 \cdot Q + 4 \cdot W_h}{2 \cdot F_{shb} + 3 \cdot F_{qshb}} \right) \cdot \frac{L}{H}
\]

9.5 Bearing capacity analysis at the wall base

The safety factor against the bearing capacity failure \( FS_b \) is computed according to the Meyerhof distribution theory. This theory indicates that the distribution of stresses on the base can be assumed as a uniform distribution over the effective length \( L' \) (Figure 4) given by:

\[
L' = L - 2 \cdot e
\]

where " \( e \) " is the eccentricity of the resulting force at the base of the wall.
"e" shall be lower than the geogrid length divided by 6 to prevent tensioning stresses at the base (in this case the resultant force on the base falls within the inertia core of the base itself):

\[
e = \frac{2 \cdot F_{abh} + 3 \cdot F_{reh}}{6 \cdot (W_r + W_b + Q)} < \frac{L}{6}
\]

(26)

The ultimate foundation soil bearing capacity \( Q_{ult} \) according to Meyerhof theory is given by the following formula (where usually the wall embedded height (d) is considered equal to zero) and \( N_q \), \( N_c \) and \( N_f \) are expressed according to the classical geotechnical theory (Vesic):

\[
Q_{ult} = N_c \cdot c_f + 0.5 \cdot N_q \cdot (L - 2 \cdot e) \cdot \gamma_f + d \cdot \gamma_f \cdot N_q
\]

(27)

\[
N_q = e^{\pi \cdot \tan \phi} \cdot \tan \left( \frac{\pi}{4} + \frac{\phi}{2} \right)
\]

(28)

\[
N_c = (N_q - 1) \cdot \cot \phi
\]

(29)

\[
N_f = 2 \cdot (N_q + 1) \cdot \tan \phi
\]

(30)

The applied vertical pressure \( Q_a \) at the wall base is:

\[
Q_a = \frac{W_r + W_b + Q}{L - 2 \cdot e}
\]

(31)

The bearing capacity safety factor \( FS_b \) is computed by comparing the ultimate bearing capacity with the applied vertical pressure \( Q_a \).

\[
FS_b = \frac{Q_{ult}}{Q_a}
\]

(32)

9.6 Final verification for the external stability analysis

The safety factors computed with equations (20), (22), (24) and (32) shall be greater than the minimum ones required by the design engineer. The recommended safety factors to design a reinforced soil retaining wall are listed in Table 2.

Moreover it is suggested that also equation (26) shall be verified.

If any equation is not verified then the following suggestions to improve the design can be followed:

A) Increase the geogrid reinforcing length.
B) Reduce the top slope angle by increasing the wall height.
C) Select a better frictional fill soil.
D) Select a heavier fill soil.
E) Increase the wall embedded height.

10. Design procedure for internal stability analysis

To provide the internal stability to the reinforced block that we have designed during the external stability analysis, the geogrid layers must be able to withstand, without overtensioning, all the tensile stresses induced by the fill soil behind the vertical wall face and the surcharge loads. The internal stability analysis will determine the type(s) and the number of required geogrids and it will verify if the reinforcing layers length is appropriate to resist to the pullout forces.
10.1 Tensile overtension failure analysis

A geogrid layout is defined and analyzed for overtension failure. The failure surfaces are assumed to be, according to Rankine's theory, along planes inclined \( 45 - \frac{\phi}{2} \) from the vertical and passing through the toe of the wall and through every mid point elevations between two geogrid layers at the wall face. This, according to Cristopher et Al., 1989, is accurate for vertical walls having the top slope angle between 0 and 20 deg (Figure 5). The elevation of the geogrid layers shall be a multiple of the compaction lift thickness or of the facing unit height if present. This is done to facilitate and speed up the construction procedure and to reduce the construction costs.

![Geogrid overtension analysis](image)

The design force \( P \) is generally ruled by the resistance of the reinforcement or by the force in the geogrid corresponding to the maximum deformations compatible with serviceability. The allowable Resistance of a geogrid is determined as a fraction of the Long Term Design Strength \( \text{LTDS} \) by means of a Partial Safety Factor \( \frac{f_{\text{s\_total}}}{\text{LTDS}} \):

\[
T_{\text{all}} = \frac{\text{LTDS}}{f_{\text{s\_total}}}
\]  

where:

\[
\text{LTDS} = T_{CR} = \text{design tensile strength (Ultimate limit state) according to Creep Rupture Analysis (Fig. 9)}
\]

or;

\[
\text{LTDS} = T_{CS} = \text{design tensile strength (Serviceability limit state) according to Creep Strain Analysis (Fig. 10)}
\]

\[
f_{\text{s\_total}} = (f_{\text{construction}} \cdot f_{\text{chemical}} \cdot f_{\text{biological}} \cdot f_{\text{junction}})
\]

The design strength \( P \) is determined by applying a further global Safety Factor \( FS_g \) to the allowable resistance \( T_{\text{all}} \). Depending on the importance and the design life of the structure; this value ranges between 1.30 ÷1.50.

\[
P = \frac{T_{\text{all}}}{FS_g}
\]

Figure 6 provides the Creep Rupture Regression curve \( T_{CR} \) at 20°C for TENAX TT SAMP geogrids as function of time determined at a different level of load % and temperature.
Reinforced soil retaining walls using TENAX geogrids

Figure 7 shows the Isochronous creep curves from which it is possible to determine the geogrid design strength \( (T_{CS}) \) as function of time and long term "in-air" strain for TENAX TT SAMP geogrids.

![Isochronous Creep Curves](image)

Fig. 6 - Creep Rupture Regression curve \( (T_{CR}) \) at 20°C determined at a different level of load % and temperature.

![Creep Rupture Regression Curve](image)

Fig. 7 - Isochronous Creep Strain Curves for TENAX TT SAMP Geogrids at 20°C.

The LTDS is a function of the creep phenomena of the geogrids, temperature and time; it is determined after creep tests (Montanelli & Rimoldi, 1993). In Table 3 are given the suggested Long Term Design Strength (LTDS) at 20°C based on the assessment performed by I.T.C. in their Technical Agrément certificate N. 580/02 (2002).

<table>
<thead>
<tr>
<th>Geogrid Type</th>
<th>( T_{CS} ) (kN/m)</th>
<th>Design Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>up to 60 Years</td>
<td>up to 120 Years</td>
</tr>
<tr>
<td>TT 045 SAMP</td>
<td>19.6</td>
<td>18.5</td>
</tr>
<tr>
<td>TT 060 SAMP</td>
<td>26.4</td>
<td>24.6</td>
</tr>
<tr>
<td>TT 090 SAMP</td>
<td>39.6</td>
<td>36.9</td>
</tr>
<tr>
<td>TT 120 SAMP</td>
<td>52.8</td>
<td>49.2</td>
</tr>
<tr>
<td>TT 160 SAMP</td>
<td>70.4</td>
<td>65.6</td>
</tr>
</tbody>
</table>

Tab. 3 - LTDS in kN/m for different geogrids at 20°C.
Table 4 provides the values of $T_{CR}$ for the Tenax geogrid at different temperatures at 120 years.

<table>
<thead>
<tr>
<th>Geogrid Type</th>
<th>Temperature 10°C</th>
<th>Temperature 20°C</th>
<th>Temperature 30°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>TT 045 SAMP</td>
<td>22.4</td>
<td>21.2</td>
<td>20.8</td>
</tr>
<tr>
<td>TT 060 SAMP</td>
<td>29.9</td>
<td>28.3</td>
<td>27.7</td>
</tr>
<tr>
<td>TT 090 SAMP</td>
<td>44.8</td>
<td>42.4</td>
<td>41.6</td>
</tr>
<tr>
<td>TT 120 SAMP</td>
<td>59.8</td>
<td>56.5</td>
<td>55.4</td>
</tr>
<tr>
<td>TT 160 SAMP</td>
<td>79.7</td>
<td>75.4</td>
<td>73.9</td>
</tr>
</tbody>
</table>

Tab. 4 - $T_{CR}$ in kN/m for Tenax geogrid at different temperatures at 120 years

The biological and chemical Safety Factors for TENAX TT SAMP geogrids are equal to 1.00 for all typical conditions found in natural soil; the manufacturing technology and polymer used for Tenax TT SAMP geogrids are such to guarantee any aging in consequence of chemical and biological aggression. The geogrid are made with high quality polyethylene (HDPE) the most inert polymer type and therefore are chemically and biologically resistant. Tests results performed on TENAX TT SAMP geogrids at Geosyntec Laboratory (1991) in USA, using the E.P.A. 9090 Test Method have shown that the HDPE extruded geogrids are not damaged by any synthetic leachate at typical temperature condition found in soil. Furthermore, the TENAX geogrids have high resistant to micro-organisms attack (aerobic and anaerobic bacterium) and macro-organisms (rodent and termite).

The total factor $f_{S_{total}}$ shall be obtained by multiplying several Partials Factors of Safety (Koerner, 1994) to account for several possible aging factors (eq. 18). The biological and chemical Safety Factors for TENAX TT SAMP geogrids are equal to 1.00 for all typical conditions found in natural soil; the manufacturing technology and polymer used for Tenax TT SAMP geogrids are such to guarantee any aging in consequence of chemical and biological aggression. The geogrid are made with high quality polyethylene (HDPE) the most inert polymer type and therefore are chemically and biologically resistant. Tests results performed on TENAX TT SAMP geogrids at Geosyntec Laboratory (1991) in USA, using the E.P.A. 9090 Test Method have shown that the HDPE extruded geogrids are not damaged by any synthetic leachate at typical temperature condition found in soil. Furthermore, the TENAX geogrids have high resistant to micro-organisms attack (aerobic and anaerobic bacterium) and macro-organisms (rodent and termite).

$\frac{R_j}{LTDS} \geq 1.50 \Rightarrow f_{S_{junction}} = 1.00$ (36)

Furthermore, the tensile creep tests are performed by applying the load thru the junctions, that is by clamping the specimen on the junctions; therefore thru testing we can verify that even in the long term the junction strength is always greater than the design strength: therefore the junction Safety Factor $f_{S_{junction}}$ shall be assumed equal to 1.0. Table 6 reports the mechanical characteristics for Tenax TT SAMP geogrids types.

<table>
<thead>
<tr>
<th>Geogrid Type</th>
<th>Tensile strength (kN/m)</th>
<th>Elongation at peak (%)</th>
<th>Junction strength (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TT 045</td>
<td>45</td>
<td>11.5</td>
<td>36</td>
</tr>
<tr>
<td>TT 060</td>
<td>60</td>
<td>13.0</td>
<td>50</td>
</tr>
<tr>
<td>TT 090</td>
<td>90</td>
<td>13.0</td>
<td>80</td>
</tr>
<tr>
<td>TT 120</td>
<td>120</td>
<td>13.0</td>
<td>110</td>
</tr>
<tr>
<td>TT 160</td>
<td>160</td>
<td>13.0</td>
<td>130</td>
</tr>
</tbody>
</table>

Tab. 6 - Junction strength for different geogrid type

When soil, especially crushed gravel, is spread on geogrids and is compacted, geogrids suffer damages due to local punctures, indentations, abrasions, cuttings and splitting inferred by the aggregate. Every type of geogrid suffers a different degree of damage which can be assessed by tensile tests performed on both damaged and control (undamaged) products. On this subject extensive independent test programs have been performed in the UK for evaluating the residual tensile strength of different geosynthetics after a full scale compaction damage trial. In example, full scale compaction damage trials were performed by the TRRL (Transport Road Research Lab) following the procedure set by Watts and Brady (1990) and tensile tests were performed both on the
original and damaged specimens by independent laboratories. The results of these tests for several geogrids and soil type are summarized in the following Table 7.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>( \varnothing ) max. of the particles</th>
<th>( f_{s_{\text{construction}}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silt and Clay</td>
<td>&lt; 0.06 mm</td>
<td>1.00</td>
</tr>
<tr>
<td>Pulverized fuels ash</td>
<td>variable</td>
<td>1.00</td>
</tr>
<tr>
<td>Fine and medium sand</td>
<td>0.06 - 0.6 mm</td>
<td>1.00</td>
</tr>
<tr>
<td>Coarse sand and fine gravel</td>
<td>0.6 - 6 mm</td>
<td>1.00</td>
</tr>
<tr>
<td>Gravel</td>
<td>6 - 40 mm</td>
<td>1.00</td>
</tr>
<tr>
<td>Ballast, sharp stones</td>
<td>&lt; 75 mm</td>
<td>1.03</td>
</tr>
<tr>
<td></td>
<td>&lt; 125 mm</td>
<td>1.07</td>
</tr>
</tbody>
</table>

Tab. 7 - \( f_{s_{\text{construction}}} \) for different types of soil

The total active force \( F_r \) in the reinforced soil due to the active soil wedge and the surcharge loads at the wall base elevation is given by:

\[
F_r = (0.5 \cdot \gamma_r \cdot h + q) \cdot h \cdot K_w
\]  \hspace{1cm} (37)

and the total active force \( F_r(y) \) in the reinforced soil at the mid point elevation \( m_i \) between two geogrid layers is the following:

\[
F_r(m_i) = \left[ 0.5 \cdot \gamma_r \cdot (h - m_i) + q \right] \cdot (h - m_i) \cdot K_w
\]  \hspace{1cm} (38)

The horizontal component \( F_{hr} \) and \( F_{hr}(m_i) \) are obtained as shown in the previous chapter, according to Rankine’s or Coulomb’s Theory.

The minimum required number \( N_{\text{min}} \) of reinforcing layers to provide the internal stability is:

\[
N_{\text{min}} = \left( \frac{F_{hr} \cdot F_{S_t}}{P \cdot R_t} \right)_{\text{round up}}
\]  \hspace{1cm} (39)

where \( F_{S_t} \) is the safety factor against overtension failure and \( R_t \) is the geogrid area coverage ratio. The minimum geogrid number is theoretically enough to reinforce the fill soil, but since the geogrids are placed at an elevation which is a multiple of the compaction spacing lift, and not where the reinforcement strength is optimal, then typically this number must be increased.

The geogrid layers must be spaced along the height of the wall taking into consideration that, at the bottom, the horizontal stresses are greater, so the required spacing lift shall be smaller than the one near the top of the wall. For full height facing panel wall, the first geogrid layer shall be placed at an elevation higher than the base elevation to provide higher resisting moment against tilting of the wall face. On the other hand, for concrete block facing walls, placing a geogrid layer at the base will enhance the stability and will increase the foundation bearing capacity.

The following compaction lift numbers \( (n_i) \) between two geogrid layers are selected:

\[
n_1, n_2, n_3, ..., n_i, ..., n_n \text{ where usually: } n_i \leq n_{i+1}
\]  \hspace{1cm} (40)

The geogrid layers elevations \( (h_i) \) and the mid point elevations \( (m_i) \) between two geogrids are then calculated:

\[
h_1, h_2, h_3, ..., h_i, ..., h_n \quad h_i = \sum_{n=1}^{i} n_s \cdot s
\]  \hspace{1cm} (41)

\[
m_1, m_2, m_3, ..., m_i, ..., m_n \quad m_i = \frac{h_i + h_{i+1}}{2}
\]  \hspace{1cm} (42)

where \( h_0 = m_0 = 0 \).
We assume that a single geogrid layer receive a total lateral horizontal force \( F_{g_{ij}} \) that is equal to the difference of
the Rankine’s horizontal active force computed at the two mid point elevations between the geogrid into
consideration and the above and below ones:
\[
F_{g_{ij}} = F_{hr}(m_{i+1}) - F_{hr}(m_i)
\]  
(43)

The safety factor against the geogrid layer overtension \( FS_{g_{ij}} \) is then calculated by:
\[
FS_{g_{ij}} = \frac{F_{g_{ij}}}{P}
\]  
(44)

The safety factor against geogrid layer overtension shall be calculated for all the geogrid layers taking into
consideration each elevation \( h_i \) and type \( t_j \). When all of the above safety factors against overtension failure
\( FS_{g_{ij}} \) are greater than the required safety factor \( FS_{ij} \) than the wall is properly reinforced in respect of the
overtension failure analysis. If any of the above safety factors is not high enough, then the design shall be
modified following these suggestions:
A) Reduce the geogrid spacing
B) Increase the number of geogrid layers
C) Use a TENAX geogrid having higher long term tensile strength
D) Use better frictional fill soil
E) Combination of the above points from A) to D) or other possibilities allowed by the specific site conditions.

10.2 Geogrid pullout failure analysis

Once the geogrid layout has been finally determined the geogrid pullout failure analysis is performed to verify
that the geogrid lengths are appropriate to carry the design loads \( F_{g_{ij}} \). The Rankine’s failure line passing through
the wall toe divides the geogrid lengths in two portions, one near the wall face in the active wedge \( L_{ai} \), the other
embedded behind in the resisting area \( L_{ei} \) as shown in Figure 8. Experimental evidence (Christopher et al, 1989)
shows that, for vertical walls with "extensible" reinforcements, like geogrids, the failure line is very close to the
Rankine’s one.

The failure surface can be defined by a plane passing through the wall toe and inclined of an angle equal to
\[
\frac{45 - \phi_r}{2}
\]
from the vertical. The active geogrid lengths \( L_{ai} \) and the embedded geogrid lengths \( L_{ei} \) are given
by:
\[
L_{ai} = h_i \cdot \tan(45^\circ - \phi_r / 2)
\]  
(45)
\[
L_{ei} = L_i - L_{ai}
\]  
(46)

where \( L_i \) is the specific length of the geogrid layer at the elevation \( h_i \).
The pullout resistant force is provided by shear stresses between the soil and the geogrid and by passive resistance between the soil, interlocked into the geogrid apertures, and the cross directional geogrid bars. The geogrid pullout properties are expressed by the soil-geogrid pullout coefficient ($\phi_{po}$).

The pullout forces ($P_{ri}$) on the embedded reinforcement lengths ($L_{ei}$) are given by:

$$P_{ri} = 2 \cdot C_{po} \cdot L_{ei} \cdot \sigma_{vi} \cdot \tan \phi_r$$  \hspace{1cm} (47)

where:

$$\sigma_{vi} = (h - h_i) \cdot \gamma_r + q + \frac{W_{ei}}{L_{ei}}$$  \hspace{1cm} (48)

and:

$$W_{ei} = 0.5 \cdot (L_{oi} + L_i) \cdot \tan \beta \cdot \gamma_r \cdot L_{ei}$$  \hspace{1cm} (49)

The geogrid length $L_{ei}$ shall always be kept equal to or greater than the reinforced block length $L$ at the wall base when the geogrid elevation is lower than two thirds of the wall height. Then for higher geogrid elevations, the reinforcing length can be carefully reduced. The pullout safety factor $FS_{pi}$ for every geogrid layer is computed as following:

$$FS_{pi} = \frac{P_{ri}}{F_{gi}}$$  \hspace{1cm} (50)

All the pullout safety factors for all geogrid layers shall be greater than the one required by the design engineer. If they are not high enough then the following adjustments shall be made:

A) Increase all the geogrid lengths.
B) Increase the geogrid lengths where the pullout safety factors $FS_{pi}$ are low.
C) Reduce the active horizontal stress on the geogrid layer by reducing the spacing between geogrids.
D) Increase the vertical stress $\sigma_r$ on the geogrid layers by decreasing the geogrids elevations.
11. **Local stability of segmental retaining wall units**

When the wall is built using segmental units for the face, such as concrete blocks, then additional analysis shall be performed to ensure that the stability of the face is accomplished.

Before starting the design, the following additional information shall be known or determined:

- Block unit height (m) \( H_u \)
- Block unit depth (m) \( B_u \)
- Distance of the gravity center from the face (m) \( G_u \)
- Inclination of segmental retaining wall (deg) \( \alpha \)
- Inclination of wall base (deg) \( \omega \)
- Apparent minimum shear resistance (peak and serviceability) between block units (kN/m) \( a_u, a'_u \)
- Apparent minimum shear resistance (peak and serviceability) between block unit and geogrid (kN/m) \( a_{cs}, a'_{cs} \)
- Apparent friction angle (peak and serviceability) between block units (deg) \( \lambda_u, \lambda'_u \)
- Apparent friction angle (peak and serviceability) between block unit and geogrid (deg) \( \lambda_{cs}, \lambda'_{cs} \)
- Maximum connection strength (peak and serviceability) between reinforcement and block unit (kN/m) \( S_{c(max)}, S'_{c(max)} \)

The values of the apparent shear resistance between block units and between block units and the reinforcement are determined with direct shear of tests which allow to define a relation between interface shear strength for stacked segmental units and normal pressure. Tests are carried out with or without a geogrid between layers of concrete units. The bottom layer of concrete blocks is laterally restrained, while the top layer is subjected to a constant vertical pressure.

The interface is sheared at a constant rate of displacement until failure occurs.

The apparent shear resistance is recorded both at peak and after 10 mm of displacement. The resulting shear stresses are plotted versus the applied vertical stress and are linearly interpolated to determine the apparent friction angle.

Visual identification of the structural performance of the composite system of segmental retaining wall units, soil and geogrid is largely determined by local stability and influenced by construction procedures. There must be sufficient connection strength and stiffness between the wall unit and geosynthetic reinforcement; in addition the geosynthetic reinforcements should be vertically spaced so that the lateral forces are safely below the shear capacity of the wall units.

Before analyzing the three main failure modes, it is necessary to define the “hinge height” concept.

The hinge height \( H_h \) is related to the maximum number of block units that can be stacked in an isolated column at a facing inclination of \( (\omega + \alpha) \) without toppling (Figure 9).  

The hinge height \( H_h \) is determined by the summing moments about the heel of the wall base and when \( (\omega + \alpha) > 0 \) must be calculated as:

\[
H_h = 2 \cdot \left( B_u - G_u - 0.5 \cdot H_u \cdot \tan \alpha \right) \cdot \cos \alpha \right) / \tan(\omega + \alpha)
\]

(51)

where:

- \( B_u \) = wall unit width
- \( H_u \) = wall unit height
- \( G_u \) = distance of the gravity center of the block unit filled with soil from the face.
The weight per unit width of the column of block units used in the following chapters is given by:

\[ W_w = H_h \cdot \gamma_u \cdot B_u \]  

(52)

where \( \gamma_u \) = weight per unit volume of block unit filled with soil.

### 11.1 Facing connection strength

The facing connection between the reinforcements and the block units at each geogrid elevation \( h_i \) must have sufficient connection strength to preclude slippage of the reinforcement due to the applied tensile forces \( F_g(i) \).

This is a conservative evaluation of the geogrid connection performance, since the maximum applied tensile load \( F_g(i) \) in the reinforcement occurs at its intersection with the internal failure surface and not at the back of the wall, except near the toe of the wall.

The geogrid connection strength \( S_c(i) \) at each reinforcement elevation \( h_i \) is influenced by the weight of the wall units \( W_w(i) \) acting on the interface (Figure 10) and can be expressed by:

\[ S_c(i) = a_{cs} + W_w(i) \cdot \cos \alpha \cdot \tan \lambda_{cs} \]  

(53)
$S_s(i)$ shall not be greater than $S_s^{\text{max}}$.

Either the peak or serviceability state parameter may be used, depending upon the criticality of the structure. The adequacy of the connection strength at any geogrid layer elevation is determined by comparing it to the maximum applied reinforcement tensile load $F_g(i)$ using the following expression:

$$FS_{cs}(i) = \left[ S_s(i) \cdot \cos \alpha \right] / F_g(i)$$

(54)

where $FS_{cs}(i)$ is the geogrid connection strength safety factor.

The magnitude of $FS_{cs}(i)$ can be increased by reducing the vertical spacing of the reinforcements.

### 11.2 Resistance to face bulging

Bulging occurs when a wall unit does not maintain its relative position with respect to the block unit above or below it. The relative position of one course to the others is maintained by shear resistance. Therefore, for soil reinforced walls, all units must have sufficient shear strength to resist the theoretical horizontal earth pressure being applied between the layers of reinforcement. Resistance to bulging is determined by the magnitude of applied lateral pressure, vertical spacing of reinforcement and shear capacity between wall units.

With reference to Figure 11, the shear capacity $V_h(i)$ acting at any interface is controlled by the weight of wall units $W_w(i)$ acting on the interface and implementing the hinge height.

$$V_h(i) = a_u + W_w(i) \cdot \cos \alpha \cdot \tan \lambda + W_w(i) \cdot \sin \alpha$$

(55)

Either the peak or serviceability state parameter may be utilized, depending upon the criticality of the structure. The total horizontal earth forces $F_{tb}(i)$ are calculated using equation (24) for each intermediate wall height $m_i$.

The maximum applied tensile force in each reinforcement $F_g(i)$ may be taken as that calculated by equation (28).

$$FS_{fb}(i) = (V_h \cdot \cos \alpha) / \left[ F_{tb}(i) \left[ F_g(i+1) + F_g(i+2) + \ldots + F_g(n) \right] \right]$$

(56)

where $FS_{fb}(i)$ is the face bulging factor of safety.

The magnitude of $FS_{fb}(i)$ may be increased by either changing the vertical spacing and increasing the number of layers of geogrid.

![Fig. 11 - Face bulging.](image-url)
11.3 Maximum unreinforced number of block units

The block units above the highest reinforcement elevation must be examined to ensure they will perform as a freestanding retaining wall. The examination of the upper unreinforced wall height for sliding and overturning failure modes is done in the same manner as the conventional walls analysis. Considering the horizontal component $F_{bh}$ of the active earth force $F_b$ acting on the unreinforced wall height, and the shear capacity $V_h(i)$ acting on the last reinforcement layer, we have:

$$FS_{sc} = \frac{V_h}{F_{bh}}$$

The resistance to overturning about the toe is evaluated by calculating a factor of safety $FS_o$ as the ratio of the sum of the resisting moments to the sum of the driving moments with respect to the toe of the unreinforced wall:

$$FS_o = \frac{M_r}{M_d}$$

If unacceptable values of $FS_{sc}$ and $FS_o$ are identified, it is possible to reduce the unreinforced wall height by incorporating an additional reinforcement layer near the top of the wall having a length of about 70% of the height.

12. Global stability analysis

The general mass movement of a wall structure and adjacent soil mass is called a global stability failure. With reference to Figure 10, the factor of safety for reinforced soil retaining wall $FS_{gl}$ can be computed as:

$$FS_{gl} = \left( M_s + M_g \right) / M_o = FS_o + M_g / M_o$$

where:

- $M_s$: stabilizing moment due to soil shear resistance;
- $M_g$: stabilizing moment due to geogrid tensile forces;
- $M_o$: destabilizing moment;
- $FS_o$: Factor of Safety for the unreinforced wall (without geogrid);
- $M_s$, $M_g$, and $FS_o$ can be computed using the modified Bishop Method of slices or any other suitable method (Janbu, Morgensten-Price, etc.).

The stabilizing moment due to geogrids tensile strength can be computed as:

$$M_g = \sum_i F_{ai}(x) \cdot b_i$$

where:

- $F_{ai}(x)$ = tensile force in the i the geogrid layer, at the point where the failure surface cut the geogrid;
- $b_i$ = arm of the geogrid tensile force.

Global stability Safety Factor shall typically range between 1.3 and 1.5.

Figure 12 shows the envelope of $F_{ai}(x)$ which is usually adopted for this calculation (Rimoldi & Ricciuti, 1992).
13. Design Example

We have to design retaining wall using TENAX geogrid for soil reinforcement. The wall is 6.0 m high, the maximum surcharge load on the crest is 10 kN/m². The design life is 120 years. The wall geometry is defined by several parameters including the total height (h) the embedment height (d), the top slope angle (β), the surcharge load distribution (q) (see figure 13).

The surcharge loads are vertical and considered uniformly distributed over the overall length of the top surface. The data of the problem are summarized in Figures 14, 15 and 16.

This design example is solved using the TNXWall Design Software; TNXWall is a software package developed by the TENAX Geosynthetics Design for designing reinforced soil retaining walls incorporating HDPE monoriented geogrids.

The software performs an external stability analysis to determine the dimension of the area to be reinforced, an internal stability analysis to determine the required geogrid tensile strength, the minimum required number of geogrid layers and the minimum required length to ensure a rigid behavior in the reinforced block.
Reinforced soil retaining walls using TENAX geogrids

The following figure extracted from TNXWall design software shows the slope and retaining wall characteristics needed for input in the design.

![Wall geometry input data](image)

**Fig. 14 - Wall geometry input data**

The next figure extracted from TNXWall software shows the soil characteristics identified for the design of the reinforced, the backfill and the foundation soils and the geogrids selected between different types of reinforcement available.

![Soil characteristics input area](image)

**Fig. 15 - Soil characteristics**

The following figure extracted from TNXWall design software shows the safety factors suggested to design a typical reinforced soil vertical retaining wall with Tenax geogrids. These safety factors must be tuned according to the specific site conditions such as the wall geometry, the soil types, the construction procedures, the overall life of the project, the criticality of the structure and existing codes.
Utilizing TNXWall software, the results relating to “Design procedure for external stability analysis” (paragraph 9) are contained in the following figures.

For this analysis, the reinforced soil-geogrid volume is assumed to act as a rigid block. This block is subject to the conventional retaining wall failure mechanisms such as: Sliding, Overturning and Bearing Capacity failure. This design step will identify the dimensions of the area to be reinforced.

During the external stability analysis, the passive resistance of the foundation soil at the wall toe and the backfill vertical driving forces are neglected and considered equal to zero for easiness and safety.

Fig. 16 - Coefficient and Safety Factor input data

Fig. 17 - External stability analysis (1)
Utilizing TNXWall design software, the results relating to “Design procedure for internal stability analysis” (paragraph 10) are contained in the following figures. This analysis is performed on the soil-geogrid volume to determine the required geogrid tensile strength, the minimum required number of geogrid layers and the minimum required length to ensure a rigid behavior in the reinforced block. Typical internal stability analyses are: the Geogrid layers layout, the Geogrid Overtension failure and the Geogrid Pullout failure.
The following figure of the TNXWall design software allows the determination of the required geogrid quantity for the overall project.

The final geogrid layout is given below.
Fig. 23 - Final geogrid layout
14. References


