

Slope Stability Analysis

Failure Models

The slope of an embankment, an excavation, a dam, a hill side, or other types can become unstable due to an increase in driving force, such as increase in surcharge, or a reduction in resistance, such as high porepressure. The failure surface of a slope failure in a 2D view can commonly be simplified as a circular shape, as shown in Figure 1, or a non-circular shape as shown in Figure 2.

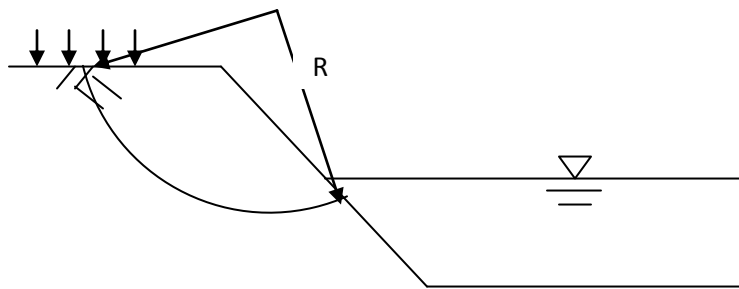


Figure 1- Circular Failure Surface

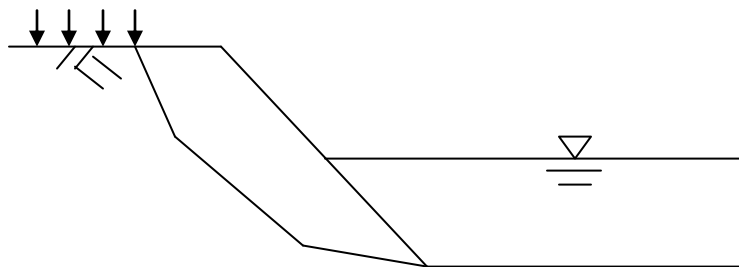


Figure 2 - Non-Circular Failure Surface

Method of Slices

For the convenience of slope stability analysis, the failure body of the slope is graphically divided into a number of vertical slices, as shown in Figure 3. The weight, driving force(s), and resistance of each slice can be calculated relatively easily. The total driving force and resistance are a combination of the driving force of each individual slice and combination of the resistance of each individual slice, respectively.

The factor of safety in a slope stability analysis is defined as the total resistance divided by the total driving force.

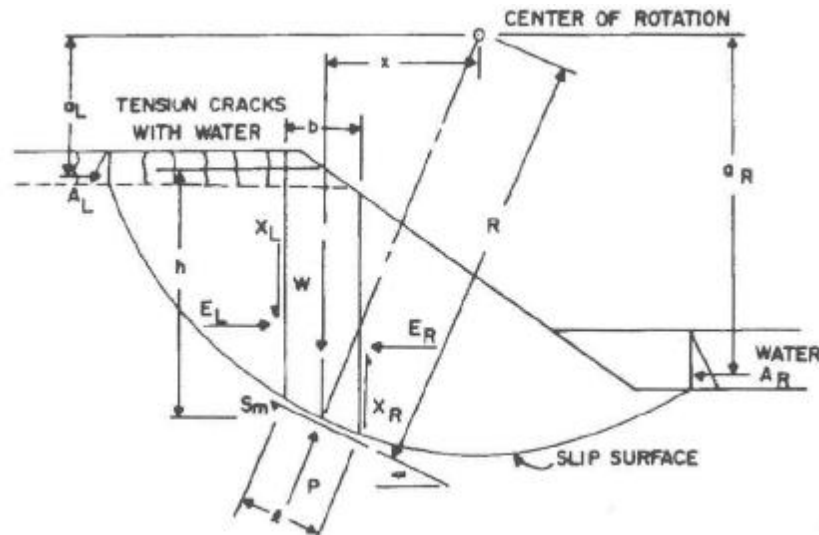


Figure 3 - Forces Acting of Method of Slice (after D.G. Fredlund)

The definition of each variable is as follows:

W = the total vertical force due to the mass of a slice of width ' b ' and height ' h '.

P = the total normal force on the base of a slice.

S_m = the shear force mobilized on the base of each slice.

E = the horizontal interslice normal forces.

X = the vertical interslice shear forces.

R = the radius or the moment arm associated with the mobilized shear force, S_m .

x = the horizontal distance from the centroid of each slice to the center of rotation.

a = the perpendicular distance from the resultant external water force to the center of rotation.

b = the width of a slice.

A = the resultant external water forces.

α = the angle between the tangent to the center of the base of each slice and the horizontal.

Bishop's Method

The Modified (or Simplified) Bishop's Method is an extension of the Method of Slices, and is used in Visual Slope for analyzing circular failures. This method assumes:

- forces on the sides of each slice are horizontal, and
- moments about the pivot point of a circular failure surface are in an equilibrium condition

The method has been shown to produce factor of safety values within a few percent of the "correct" values.

$$F_m = \frac{\sum [c' \cdot l + (P - u \cdot l) \tan(\varphi')]}{\sum W \cdot x \pm A \cdot a} R$$

Janbu's Method

The difficulty in analyzing a non-circular failure is that it is difficult to find a single point through which many of the force components act. Therefore, a moment equilibrium method - like Bishop's, used for circular surfaces, is no longer the most appropriate.

Janbu chose to use a force equilibrium method in the analysis. In the simplified Janbu's method, the interslice shear forces are assumed to be zero. The factor of safety can be calculated as:

$$F_f = \frac{\sum [c' \cdot l + (P - u \cdot l) \tan(\varphi')] \cos(\alpha)}{\sum P \sin(\alpha) \pm A}$$

Janbu's method is used in Visual Slope for non-circular failures.

Spencer's Method

In 1967, Spencer developed a complete equilibrium method known as Spencer's Method, which satisfies both force and moment equilibrium forces. As a result, the Factor of Safety calculated by this method should be more precise. Spencer's Method can also be adapted for use with non-circular slip surfaces, which is useful because many slides do not have circular failure surfaces. However, calculation of the factor of safety using Spencer's Method is an iteration process, which may result in non-convergence. Spencer's method is available in Visual Slope.

Reinforced Slope Design

A reinforced slope is commonly built for claiming more flat area with a steep slope, or repairing a failed slope. Geosynthetic, like geogrid or geotextile, is commonly used as reinforcement. If a reinforced slope is flatter than 70 degrees, it should be considered as a reinforced slope. Otherwise, it should be designed as a Mechanically Stabilized Earth Wall (MSE Wall). The advantages of reinforced slopes are:

- More cost effective compared with an MSE Wall.
- More tolerance for deformation.
- Slope face can be vegetated
-

A reinforced slope analysis is similar to a regular slope analysis, except for the consideration of direct sliding over the geogrid. In a reinforced slope analysis, the

resistance forces come not only from soil, but also from the reinforcement, as shown in Figure 4.

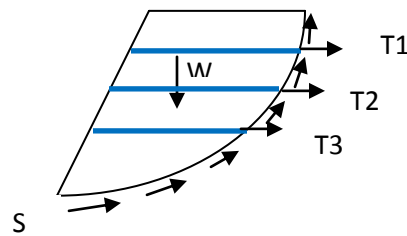


Figure 4 - Forces Acting on Failure Body

Failure Mechanisms

External (Global) Failure

If a failure surface occurs behind the reinforced zone as shown in Figure 5, it is considered as an external (global) failure. The solutions for an external (global) failure are commonly:

- Increase the length of reinforcement.
- Change to a flatter slope.
- Reduce surcharge.
- Improve the fill material.
- Reduce the water table, if applicable.

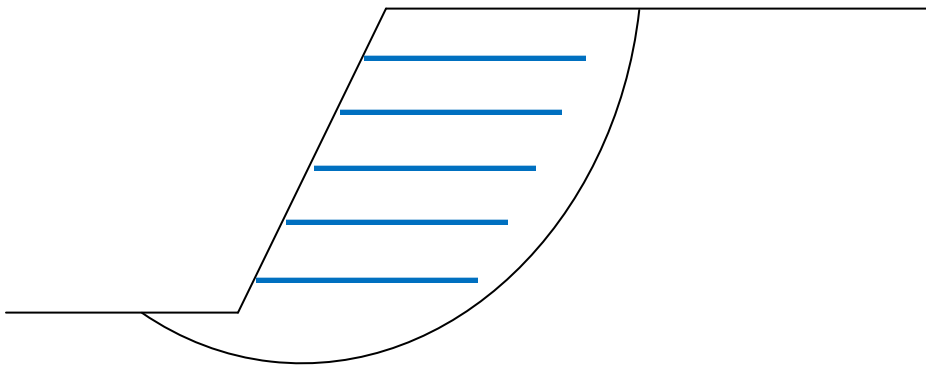


Figure 5 - Global Failure

Internal Failure

Pullout Failure

A pullout failure occurs, when the soil in the reinforced zone can not provide adequate holding force to the reinforcement. In a pullout failure, reinforcement can be pulled out

from the back slope of the failure body as shown in Figure 6, or the failure body can slip out from the front end of the reinforcement as shown in Figure 7. The solutions for a pullout failure are:

- Increase the length of reinforcement (for pullout from back cases).
- Use cohesionless soil to increase friction between the soil and reinforcement.
- Increase the layers of reinforcement
- Wrap geosynthetic over the face of the slope (for pullout from front cases)

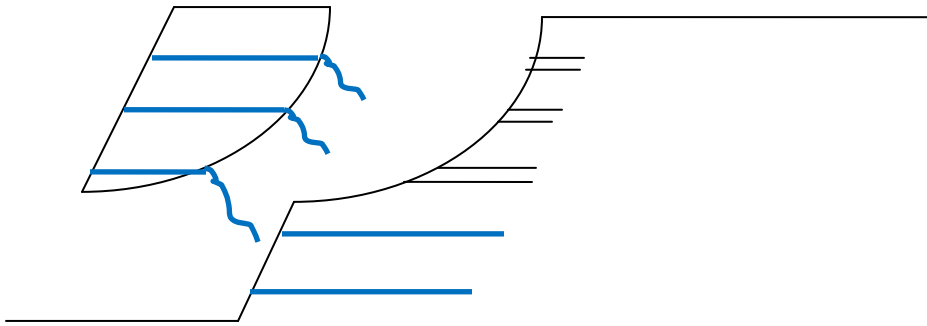


Figure 6 - Pullout from Back

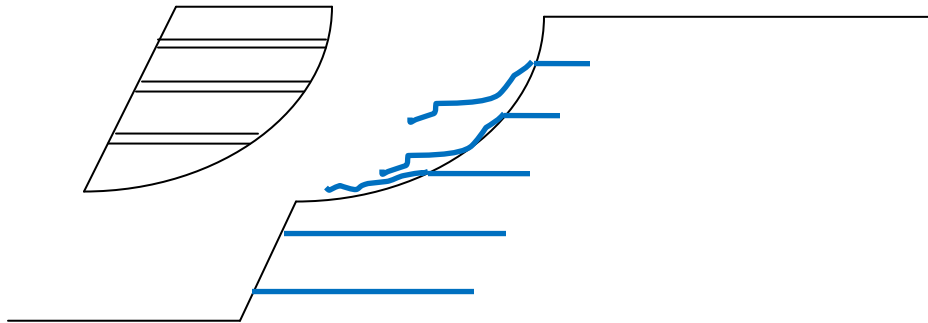


Figure 7 - Pullout from Front

Overstress Failure

An overstress failure occurs when the required reinforcement exceeds the strength of the reinforcement as shown in Figure 8. The solutions for an overstress failure are:

- Increase the strength of the reinforcement.
- Increase the layers of the reinforcement.

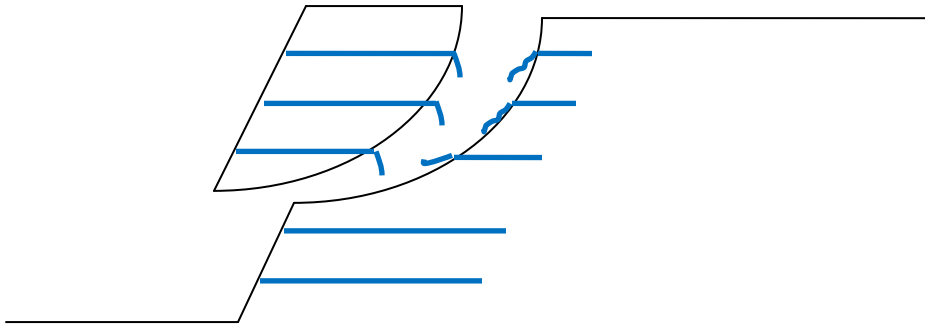


Figure 8 - Overstress Failure

Direct Slide over Geogrid

When geogrid layers are placed in a reinforced slope, weak interfaces are created between the geogrid layers and the soil layers directly above them. Therefore, the soil body above a layer of geogrid may slide over the geogrid as shown in Figure 9. The solution for direct sliding is to use material with a high friction angle.

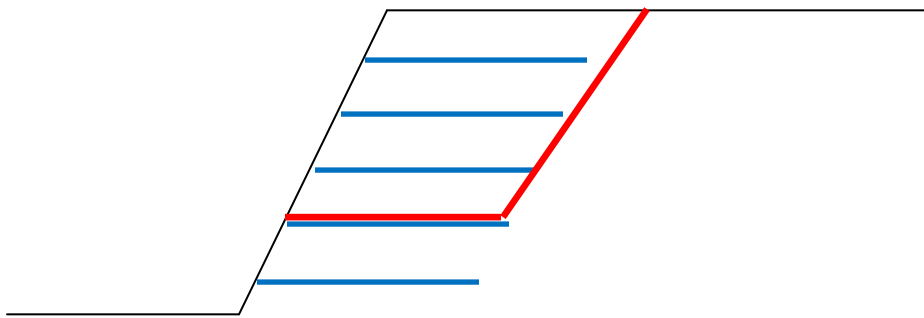


Figure 9 - Direct Sliding

Calculation of Reinforcement

The reinforcement that a geogrid can provide depends upon the holding forces provided by the surrounding soil and the strength of the geogrid itself. It should be noted that the reinforcement a layer of geosynthetic can provide is not a constant along the length of the geosynthetic because the holding force provided by the surrounding soil varies throughout the geosynthetic. For example, if a failure surface is close to the end of a geosynthetic, as failure surface A in Figure 10, the reinforcement from the geosynthetic is determined by the holding force from the back. On the other hand, if a failure surface is close to the front end of the geosynthetic, as is failure surface B in Figure 9, it is controlled by the holding force from the front. Therefore, prior to a reinforced slope analysis, reinforcement along a geosynthetic layer should be calculated.

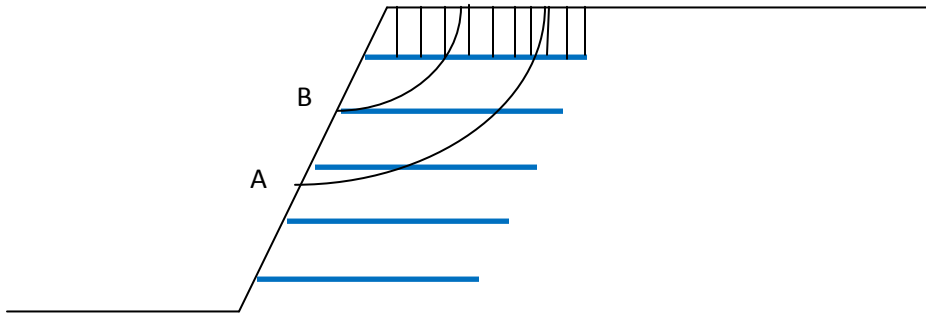


Figure 10

1. The holding force towards the back slope increases from the back end to the front end of a geosynthetic, as shown in Figure 11.

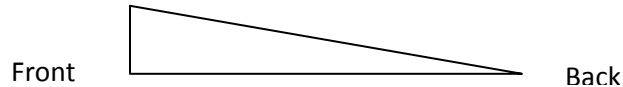


Figure 11 - Distribution of Holding Force towards Back

2. The strength of the geosynthetic of itself is a constant along the geosynthetic as shown in Figure 12.

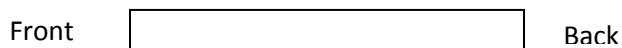


Figure 12 - Distribution of Strength

3. The holding force along a geosynthetic towards the front slope increases from the front end to back end of the geosynthetic. The holding force at the front end also depends on the front end holding condition. Figure 13 shows the holding force along the geosynthetic.

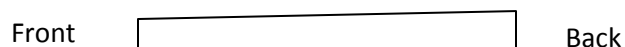


Figure 13 - Distribution of Holding Force towards Front

Overlay the three figures together. The overlap area, as shown in Figure 14, is the actual reinforcement a geosynthetic can provide along its length.

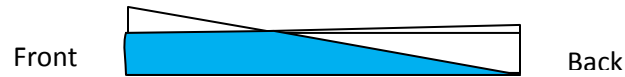


Figure 14 - Distribution of Actual Reinforcement

Soil Nailing Design

Soil nailing is used to reinforce and strengthen the existing ground by installing closely - spaced steel bars, called “nails”, in a slope or excavation as construction proceeds from the “top down”. This process creates a reinforced section that is itself stable and able to retain the ground behind it. As with MSE walls and reinforced slopes, the reinforcements are passive and develop their reinforcing action through nail-ground interactions as the ground deforms during and following construction. Nails work predominantly in tension but are considered by some to work also in bending/shear in certain circumstances. However, bending/shear is commonly not considered in design.

The analysis of a soil nailed slope is very similar to a geosynthetic reinforced slope analysis. In a soil nailing analysis, the resistance forces come not only from the soil, but also from the soil nails, as shown in Figure 15.

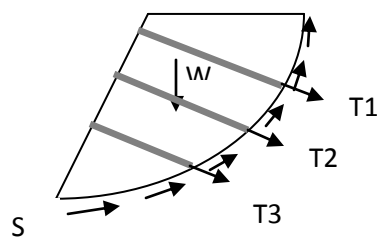


Figure 15 - Forces Acting on Failure Body

Except for the lack of direct sliding failure, the failure mechanisms of a geosynthetic reinforced slope and a soil nail reinforced slope are the same, including external (global) failure and internal failure. Internal failure includes pullout failure and overstress failure.

An increment of reinforcement along a section of a soil nail is:

$$\Delta T = \Delta x \cdot \pi d \cdot \sigma$$

Where:

ΔT = increment of reinforcement.

Δx = section length of soil nail.

d = diameter of hole.

σ = bond strength of soil.

Calculation of reinforcement along a soil nail is similar to that along a geosynthetic. Therefore, please refer to the reinforced slope section. The following tables show the bond strength for different types of soil (after FHWA soil nail design manual).

ULTIMATE BOND STRESS – CHHESIONLESS SOILS

Construction Method	Soil Type	Unit Ultimate Bond Stress kN/m^2 (psi)
Open Hole	Non-plastic silt	20-30 (3.0-4.5)
	Medium dense sand and silty sand/sandy silt	50-75 (7.0-11.0)
	Dense silty sand and gravel	80-100 (11.5-14.5)
	Very dense silty sand and gravel	120-240 (17.5-34.5)
	Loess	25-75 (3.5-11.0)

ULTIMATE BOND STRESS – COHESIVE SOILS

Construction Method	Soil Type	Unit Ultimate Bond Stress kN/m^2 (psi)
Open Hole	Stiff Clay	40-60 (6.0-8.5)
	Stiff Clayey Silt	40-100 (6.0-14.5)
	Stiff Sandy Clay	100-200 (16.5-29.0)

ULTIMATE BOND STRESS - ROCK

Construction Method	Soil Type	Unit Ultimate Bond Stress kN/m^2 (psi)
Rotary Drilled	Marl/Limestone	300-400 (43.5-58.0)
	Phillite	100-300 (14.5-43.5)
	Chalk	500-600 (72.0-86.5)
	Soft Dolomite	400-600 (58.0-86.5)
	Fissured Dolomite	600-1000 (86.5-144.5)
	Weathered Sandstone	200-300 (29.0-43.5)
	Weathered Shale	100-150 (14.5-21.5)
	Weathered Schist	100-175 (14.5-25.5)
	Basalt	500-600 (72.0-86.5)

Segmental Retaining Wall Design

Segmental retaining walls (SRW's), also called mechanically stabilized earth (MSE) walls, are gravity retaining walls with an expanded width created by a geosynthetic reinforced (infill) soil mass located behind a column of dry-stacked units. The dry-stacked column of SRW units and the geosynthetic reinforced (infill) soil zone act together to resist the destabilizing forces generated by the retained soil (backfill) and surcharge loading. Figure 16 shows a typical section of a segmental retaining wall.

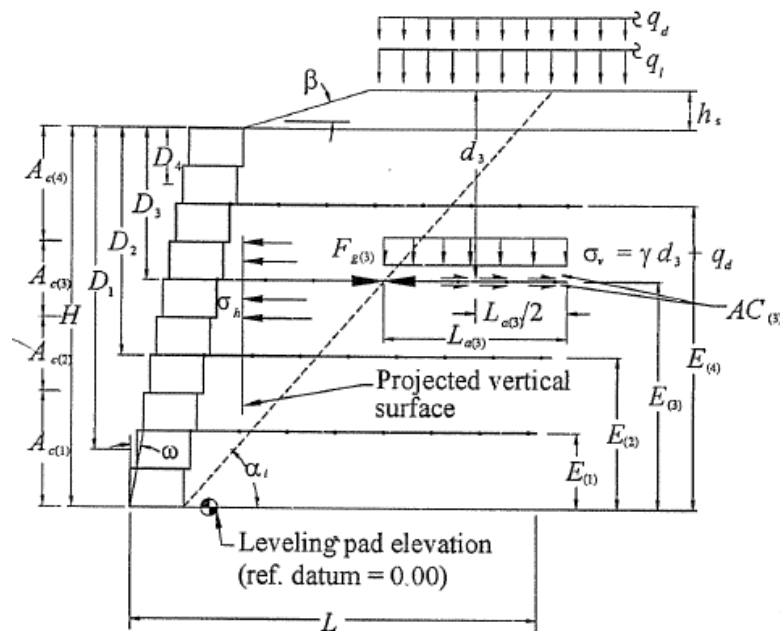


Figure 16 - Typical Section of SRW (after NCMA Design Manual)

There are two main design methods for segmental retaining walls. One is from National Concrete Masonry Association (NCMA), and the other is from American Association of State Highway and Transportation Officials (AASHTO). The two methods, in general, are very close. Visual Slope can use both methods for design.

Failure Mechanisms

External Stability

External stability analyses examine the stability of the mass formed by the facing units and reinforced soil zone with respect to active earth force generated by self-weight of the retained soil and distributed surcharge pressures beyond the reinforced zone. These analyses determine the minimum length of geosynthetic reinforcement by checking:

Base Sliding

An SRW moves outward along the base of the reinforced soil mass due to insufficient shear resistance in the soil as shown in Figure 17.

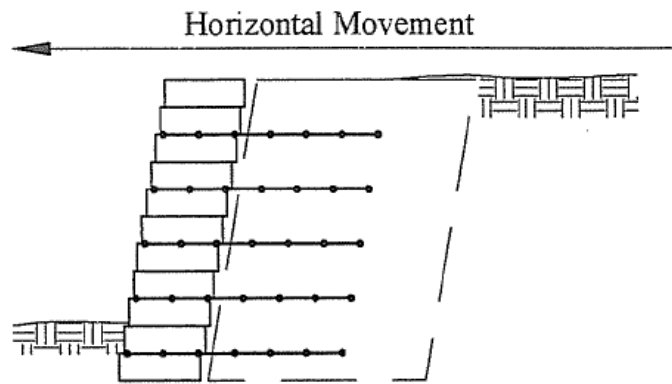


Figure 17 - Base Sliding (after NCMA Design Manual)

Overturning

The reinforced soil of an SRW rotates outward about the toe of the wall as shown in Figure 18.

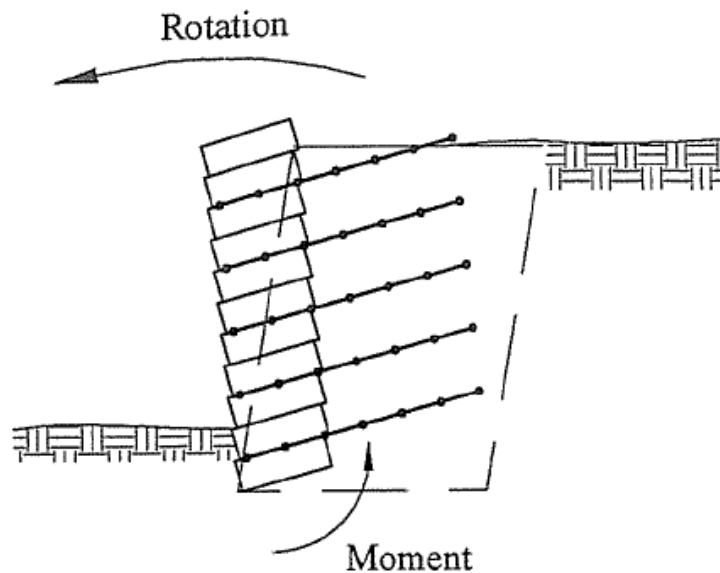


Figure 18 - Overturning (after NCMA Design Manual)

Bearing Capacity

Bearing capacity and foundation settlement should be evaluated by the project geotechnical engineer. The evaluation bearing capacity and settlement is similar to those for a strip footing.

Internal Stability

Tensile Overstress

When the applied tensile stress in the geosynthetic reinforcement exceeds and established, product-specific, allowable working stress level as shown Figure 19.

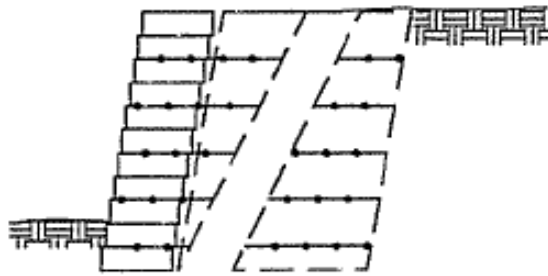


Figure 19 - Tensile Overstress (after NCMA Design Manual)

Pullout

If the soil in the reinforced zone cannot provide adequate holding force to the geosynthetic, excessive movement of the geosynthetic through the reinforced soil zone will occur without rupture of the reinforcement as shown in Figure 20.

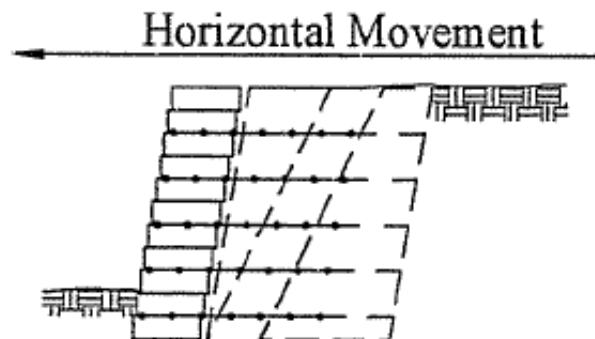


Figure 20 - Pullout (after NCMA Design Manual)

Internal Sliding

Geosynthetic reinforcement layers may create preferred planes of sliding at elevations along the height of the wall as illustrated in Figure 21.

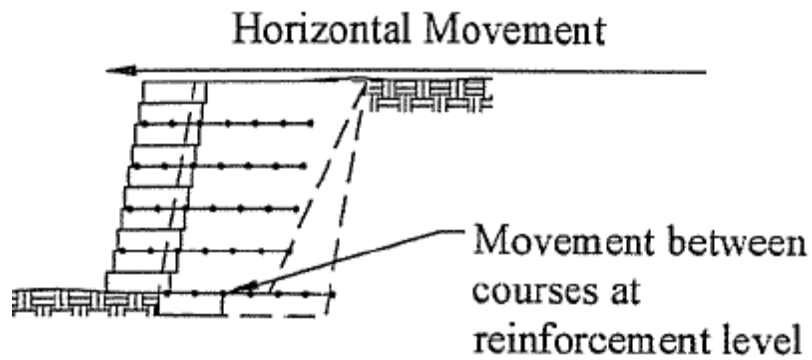


Figure 21 - Internal Sliding (after NCMA Design Manual)

Facing Stability

Facing Connection

The interface between the geosynthetic reinforcement and wall facing unit at each reinforcement placement elevation must have sufficient connection strength to preclude rupture or slippage of the reinforcement due to the applied tensile force as illustrated in Figure 22.

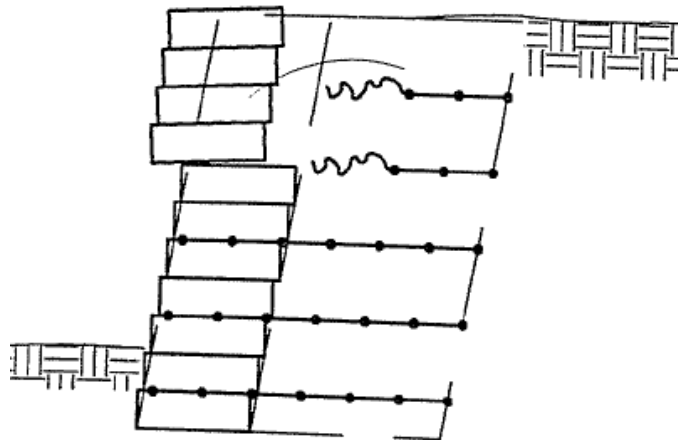


Figure 22 - Facing Connection (after NCMA Design Manual)

Crest Topping

The wall units above the highest reinforcement placement elevation must be examined to ensure that they will perform as a free standing retaining wall. Sliding and overturning must be examined as a unreinforced SRW as illustrated in Figure 23.

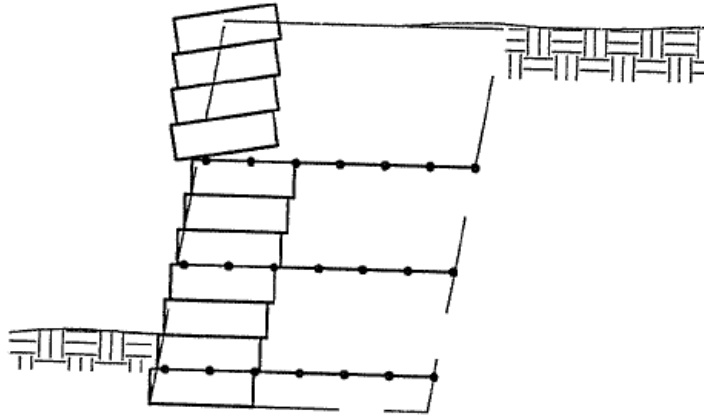


Figure 23 - Crest Topping (after NCMA Design Manual)

Slope Stability

Internal Compound Stability

Internal compound stability analysis is the evaluation of failure surfaces passing the reinforced zone as shown in Figure 24.

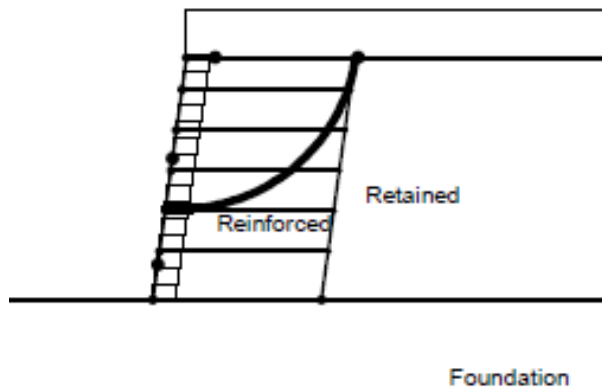


Figure 24 - Internal Compound Stability

Global Slope Stability

Global slope stability analysis is the evaluation of the failure surfaces behind the reinforced zone as shown in Figure 5.

For all the detailed calculations, please see NCMA Design Manual for Segmental Retaining Walls 3rd Edition and AASHTO 2002 MSE Wall Design Guideline.

Shoring Design

Shoring is a term used to describe a retaining wall that functions to retain earth, water, and adjacent structures when an excavation is required. Shoring design can be a very

complicated matter. The designer has to content with many unknowns and factors that influence the behavior of the excavation shoring. Typically, shoring systems can be divided into three types according to their supporting types – cantilevered, single point braced, and multi-point braced. The following sections will describe:

- How to determine the pressures acting on a retaining wall.
- How to design a cantilevered retaining wall.
- How to design a single-brace retaining wall.
- How to design a multi-brace retaining wall.

LATERAL PRESSURES ON SHEET PILE WALLS

Earth Pressure

Coulomb theory is used in Visual Slope for calculation of active and passive earth pressures for cantilevered and single-point braced walls. Active and passive pressures are calculated as follows.

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

$$K_p = \frac{\cos^2 \phi}{\cos \delta \left[1 - \sqrt{\frac{\sin(\phi + \delta) \sin(\phi + \beta)}{\cos \delta \cos \beta}} \right]^2}$$

Where

K_a = Active earth pressure coefficient.

K_p = Passive earth pressure coefficient.

ϕ = Soil internal friction angle.

β = Backfill angle with respect to horizontal.

δ = Wall friction angle.

For multi-brace walls, earth pressures are calculated as follows:

$$P = 0.65 \gamma H K_a$$

Where

P = Earth pressure.

γ = Unit weight of soil.

H = Wall Height.

The shapes of the earth pressure are:

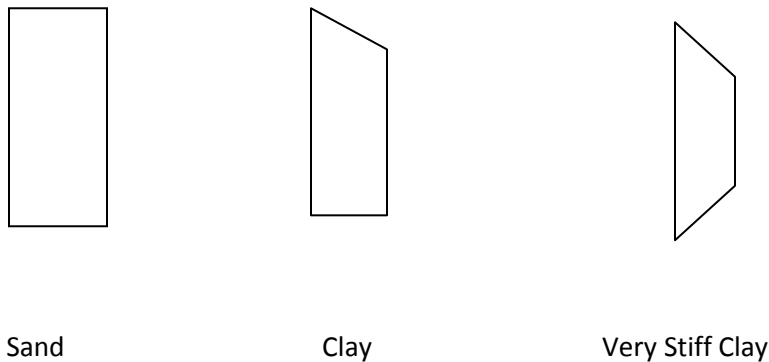


Figure 25 - Earth Pressure Diagram for Multi-point Braced Walls

Surcharge Pressure

Uniform Surcharge

When a uniformly distributed surcharge is applied at the surface, the vertical pressures at all depths in the soil are increased equally. Without the surcharge the vertical pressure at any depth h would be γh , where γ is the unit weight of the soil. When a surcharge of intensity q (force/area) is added, the vertical pressures at depth h become $\gamma h + q$. The lateral pressure σ_H , due to the uniform surcharge q , is equal to Kq , as shown in Figure 26 below.

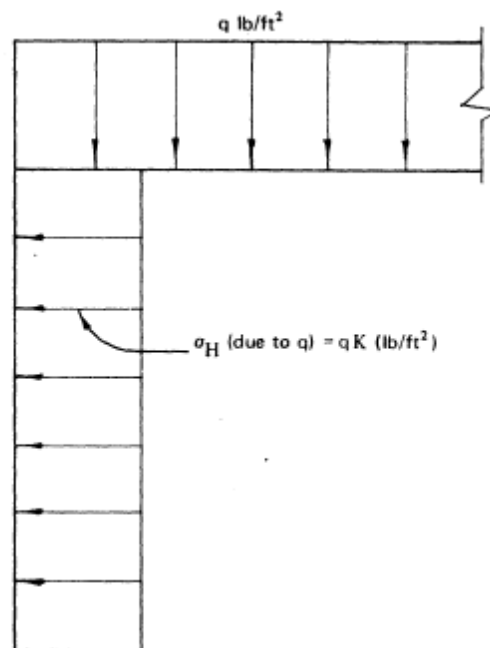
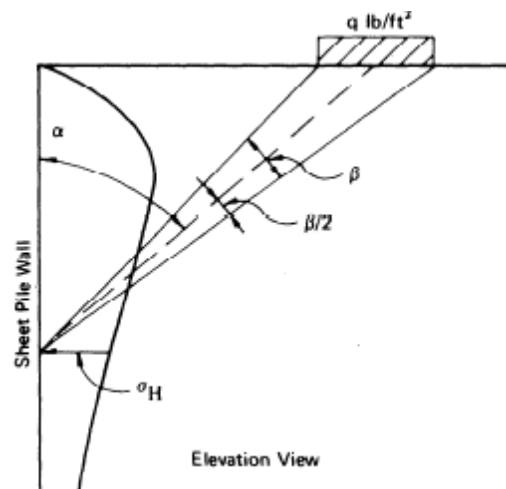


Figure 26 - Lateral pressure due to uniform surcharge

The K value is either the active coefficient K_a or the passive coefficient K_p depending upon whether the wall tends to move away from or toward the surcharge area. The uniform lateral pressure due to the surcharge is then added to the lateral dead weight earth pressures as described in previous sections.

Strip Loads

Highways and railroads are examples of strip loads. When they are parallel to a sheet pile wall, the lateral pressure distribution on the wall may be calculated as shown in Figure 27.



$$\sigma_H = \frac{2q}{\pi} [\beta - \sin\beta \cos 2\alpha]$$

Figure 27 - Lateral pressure due to strip load (Boussinesq equation modified by experiment) (after Teng)

Water Pressure

If a sheet pile structure is driven in granular soil with fairly uniform permeability, the unbalanced water pressure may be approximated by the trapezoid in Figure 28(b). If the permeability of the soil varies greatly in the vertical direction, a flow net should be used to determine the unbalanced pressure as shown in Figure 28(a).

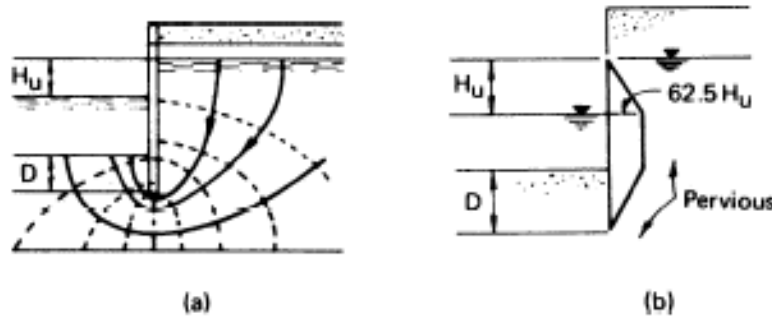


Figure 28 - Hydrostatic and seepage pressures (after Terzaghi22)

Retaining Wall Calculations

Cantilevered Walls

In the case of a cantilevered wall, sheet piling or a soldier pile is driven or drilled to a sufficient depth into the ground to become fixed as a vertical cantilever in resisting the lateral active earth pressure. This type of wall is suitable for moderate height. Walls designed as cantilevers usually undergo large lateral deflections and are readily affected by scour and erosion in front of the wall. Since the lateral support for a cantilevered wall comes from passive pressure exerted on the embedded portion, penetration depths can be quite high, resulting in excessive stresses and severe yield. Therefore, cantilevered walls are restricted to a maximum height of approximately 15 feet.

The computation of a cantilevered wall in Visual Slope is according to the following steps.

1. Use numerical calculations to determine the embedment of the pile, O , where sum of moments acting on the pile equals to zero. From that point down, the directions of active and passive earth pressures reverse as shown in Figure 29.
2. Increase the embedment till horizontal force equilibrium reaches.
3. Increase the embedment till no piping will occur.
4. Increase the embedment according to the specified factor of safety.
5. Calculation of internal forces and deflection.

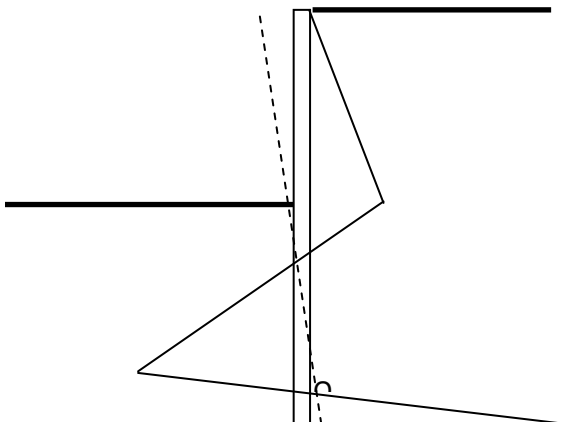


Figure 29 - Cantilevered Wall

Single-Brace Walls

A single-brace wall, as shown in Figure 30, is supported by two means: passive pressure acting on the front of the wall below the dredge line and the force from the brace above the dredge line.

The computation of a single-point braced wall in Visual Slope is according to the following steps.

1. Take moment about point "O" where the brace located. Increase the pile embedment until the sum of the moment equals to zero ($\Sigma M=0$).
2. Assume the sum of horizontal forces ($\Sigma F=0$) equals to zero for calculating brace reaction.
3. Increase the embedment till no piping will occur.
4. Increase the embedment according to the specified factor of safety.
5. Calculation of internal forces and deflection.

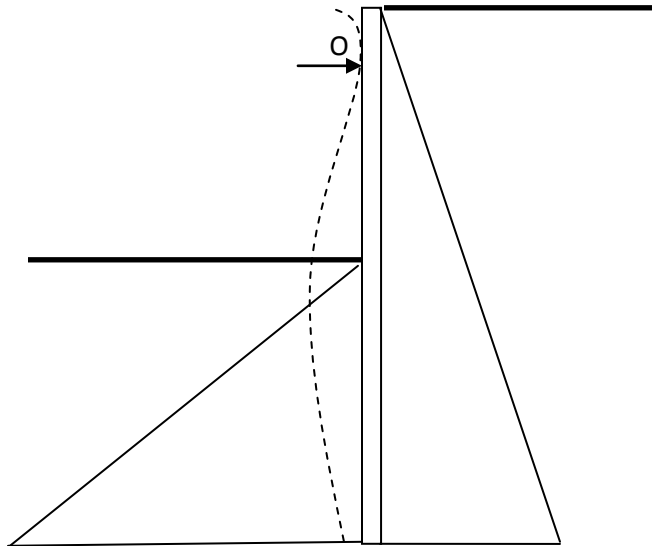


Figure 30 - Single-Brace Wall

Multi-Brace Walls

When the pressure behind a wall is high, especially in deep excavations, a cantilevered wall or a single-brace wall may not be feasible or cost effective. A multi-brace wall becomes necessary. A multi-brace wall can be with or without embedment if piping is not a concern.

A typical pressure diagram of multi-brace wall is illustrated in Figure 30. Due to the effect of multi-bracing, earth pressure diagrams of this type of wall are mostly rectangular or trapezoidal depending to the soil type. This type of earth pressure diagram can be easily divided into smaller components to simplify calculation.

In design of a multi-brace wall, the wall is divided into several spans. As shown in Figures 31 and 32.

- a) Cantilever span (from top of wall to second brace). If a multi-braced wall is without the embedment, the span from the dredge line to the second brace from the bottom is also a cantilever span.
- b) Beam span (between two adjacent braces) is simply supported beam. The reaction force and moment applied on each span are different and they needed to be determined individually for each span. It should be noted that the last beam span ends at the base of excavation, where it is assumed to be the last hinge.
- c) Embedment span (from the dredge line to the tip of pile). Since the penetration of the pile is unknown, the embedment will be calculated.

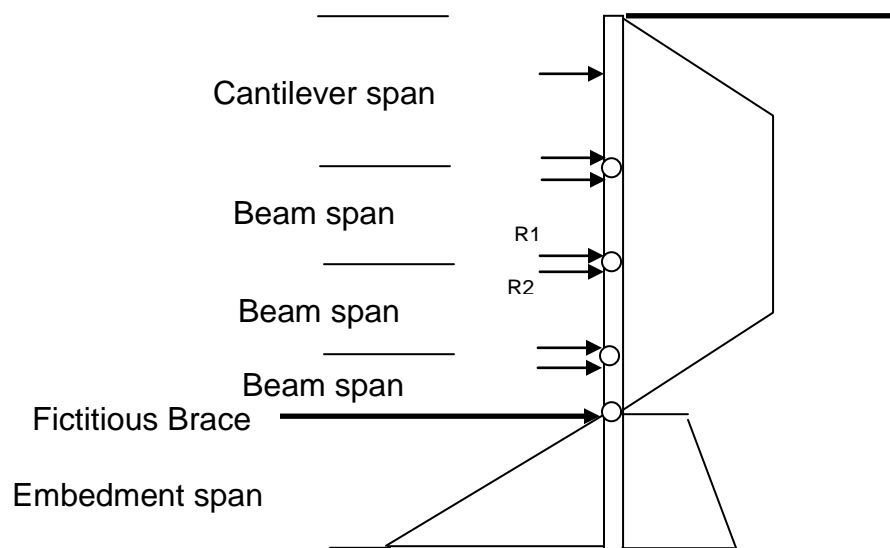


Figure 30 - Multi-Brace Wall with Embedment

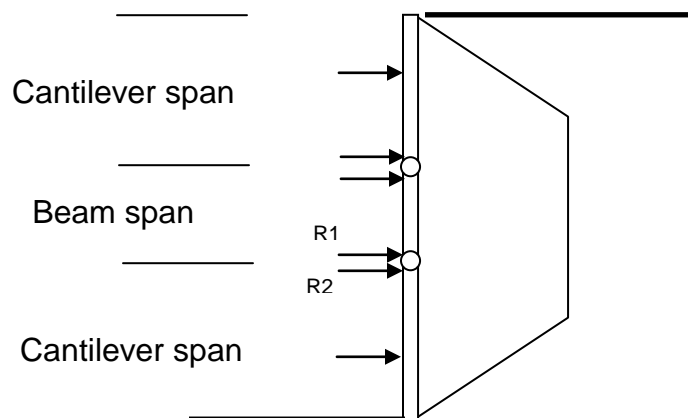


Figure 30 - Multi-Brace Wall without Embedment

The procedures for analyzing a multi-braced wall are:

1. Calculate the brace reaction(s) in cantilever span(s) and the cantilever moment(s).
2. Use the equilibrium method to calculate the driving (reaction) force of each brace against the load applied from the two closest spans. As illustrated in Figures 30 and 31, the total driving force of a brace is the sum of R_1 from the upper span (Beam Span 1) and R_2 from the lower span (Beam Span 2).
3. Determine the embedment span, if apply, by taking the moment about the fictitious brace level where the embedment span begins. Use numerical calculations to get the embedment, where moment becomes zero.
4. Increase the embedment till no piping will occur.
5. Increase the embedment according to the specified factor of safety.
6. Calculation of internal forces and deflection