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Design of Reinforced Soil Retaining Wall to Support Hill Slope

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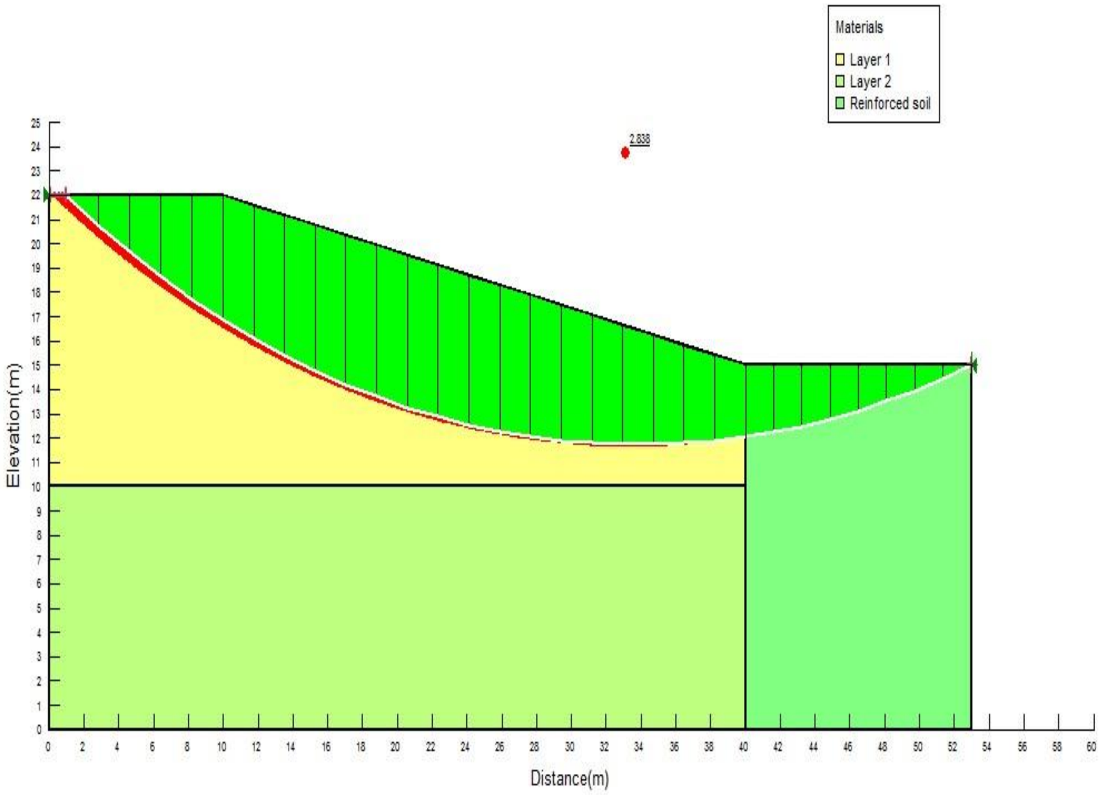

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TERMINOLOGY

A = Maximum Ground Acceleration Coefficient
AASHTO = American Association of State Highway and Transportation Officials
A_c= Design cross section area of the steel, defined as the original cross section area minus corrosion losses anticipated to occur during the design life of the wall
A_m= Maximum, wall Acceleration Coefficient at the centroid of the wall mass
c = Soil Cohesion
C = Reinforcement Effective Unit Perimeter; e.g., C = 2 for strips, grids, and sheets
E_c= thickness of the reinforcement at the end of the design life
E_n= Nominal thickness of the reinforcement at time of construction
E_R= sacrificial thickness of metal expected to be lost by uniform corrosion during the service life of the structure
e = eccentricity
F_y= Yield Stress of Steel
F^* = Pullout Resistance (or friction-bearing-interaction) Factor
F_g= Summation of Geosynthetic Resisting Force
F_H= Horizontal Earth Pressure Force
F_q= embedment (or surcharge) bearing capacity factor
F_T= total earth pressure force

FOS = overall factor of safety to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads
H = vertical wall or slope height
I= Importance factor
K_a= active lateral earth pressure coefficient
K_{af}= active lateral earth pressure coefficient of retained fill soil
k_h= horizontal seismic coefficient
k_v= vertical seismic coefficient
L = total length of reinforcement
L_a= length of reinforcement in the active zone
L_e= embedment or adherence length in the resisting zone behind the failure surface
MSEW = mechanically stabilized earth wall
N_c= dimensionless bearing capacity coefficient
N_q= dimensionless bearing capacity coefficient
N_γ= dimensionless bearing capacity coefficient
P_{AE} = seismic thrust
P_{IR}= horizontal seismic inertia force
P_r= pullout resistance of the reinforcement per unit width
q_n= net allowable bearing capacity
R=Response reduction factor
R_c= reinforcement coverage ratio b/sh
RSS = reinforced soil slope
S_t= spacing of transverse bar of grid reinforcements
t = thickness of the transverse bar of grid reinforcement
T_a= the design long term reinforcement tension load for the limit state, considering all time dependent strength losses over the design life period
T_{max}= maximum reinforcement tension
T_{MD}= dynamic increment of tensile load surface, in slope stability analysis
Z = zone factor
$\frac{Sa}{g}$= average response acceleration coefficient
α= a scale effect correction factor to account for a nonlinear stress reduction over the embedded length of highly extensible reinforcements, based on laboratory data
α_β= a bearing factor for passive resistance which is based on the thickness per unit width of the bearing member
β = surcharge slope angle (MSEW)
β = slope angle (RSS)
δ= wall friction angle
ξ= arc tan (K_h/ 1 - K_v)
γ_b= unit weight of the retained backfill
γ_f= unit weight of soil
γ_r= unit weight of the reinforced backfill
γ_w= unit weight of water
ϕ= the peak friction angle of the soil
ϕ_b= friction angle of retained fill
ϕ_{min}= minimum angle of shearing friction either between reinforced soil and reinforcement or the friction angle of the foundation soil
θ= the face inclination from a horizontal
λ= tractive shear stress
ρ= the soil-reinforcement interaction friction angle
σ'_v= the effective vertical stress at the soil-reinforcement interfaces

I. INTRODUCTION

1.1 GENERAL:

Retaining structures are essential elements of every highway design. Retaining structures are used not only for bridge abutments and wing walls but also for slope stabilization and to maximize right of-way for embankments. For many years, retaining structures were almost exclusively made of Reinforced Concrete and were designed as Gravity,Cantilever or Counterfort walls which are essentially rigid structures and cannot accommodate significant differential settlements unless founded on deep foundations. With increasing height of soil to be retained and poor subsoil conditions, the cost of construction ofReinforced Concrete retaining walls increases rapidly leading to the gradual increase in the overall project cost. Moreover construction or such type consumes time and space.

Hence in recent years, Civil Engineering practices alternative way of constructing a low cost structure that can be constructed within short time.This alternative way of construction is known as Reinforced Soil Retaining walls or Mechanically Stabilized Earth Wall (MSEW).The design, construction and monitoring techniques for Reinforced Soil retaining structures have evolved over the last few decades as a result of efforts by researchers, material suppliers and government agencies to improve some single aspect of the technology or the materials used.

1.2 MECHANICALLY STABILIZED EARTH WALL (MSEW):

MSEW is a generic term that includes reinforcedsoil (a term used when multiple layers of inclusions act as reinforcement in soils placed as fill). Reinforced Earth is a trademark for a specific reinforced soil system.MSEW structures are cost-effective alternatives for most applications where reinforced concrete or gravity type walls have traditionally been used to retain soil. These include bridge abutments and wing walls as well as areas where the right-of-way is restricted, such that an embankment or excavation with stable side slopes cannot be constructed. They are particularly suited to economical construction in steep-sided terrain, in ground subject to slope instability, or in areas where foundation soils are poor.

Uses of MSE walls include:

1) Temporary structures, which have been especially cost-effective for temporary detoursnecessary for highway reconstruction projects.
2) Reinforced soil dikes, which have been used for containment structures for water and waste impoundments around oil and liquid natural gas storage tanks. (The use of reinforced soil containment dikes is economical and can also result in savings of land because a vertical face can be used, which reduces construction time).
3) Dams and seawalls, including increasing the height of existing dams.
4) Bulk materials storage using sloped walls.

1.3 REINFORCED SOIL SLOPES (RSS):

RSS are a form of reinforced soil that incorporate planar reinforcing elements in constructed earth-sloped structures with face inclinations of less than 70 degrees.Reinforced Soil Slopes, are cost-effective alternatives for new construction where the cost of fill,right-of-way, and other considerations may make a steeper slope desirable. However, even if foundation conditions are satisfactory, slopes may be unstable at the desired slope angle. Existing slopes, natural or manmade, may also be unstable as is usually painfully obvious when they fail. Multiple layers of reinforcement may be placed in the slope during construction or reconstruction to reinforce the soil and provide increased slope stability. Reinforced slopes are a form of mechanically stabilized earth that incorporate planar reinforcing elements in constructed earth sloped structures with face inclinations of less than 70 degrees. Typically, geosynthetics are used for reinforcement.

Applications of reinforced slopes include:
1) Upstream/downstream face improvements to increased height of dams.
2) Permanent levees.
3) Temporary flood control structures.
4) Decreased bridge spans.
5) Temporary road widening for detours.
6) Prevention of surface sloughing during periods of saturation.
7) Embankment construction with wet, fine-grained soils.

1.4 METHODOLOGY:

Since the development of soil reinforcement concepts and their application to MSEW structure design, a number of design methods have been proposed, used, and refined. Current practice consists of determining the geometric and reinforcement requirements to prevent internal and external failure using limit equilibrium methods of analysis as follows:

- Designing for external stability.
- Designing for internal stability

a)*Designing for External Stability:*

As with classical gravity and semigravity retaining structures, four potential external failuremechanisms are usually considered in sizing reinforced soil retaining wall, as shown in **Fig: 1**they include:

- Sliding of the base.
- Limiting the location of the resultant of all forces (overturning).
- Bearing capacity.
- Deep seated stability (rotational slip-surface or slip along a plane of weakness).

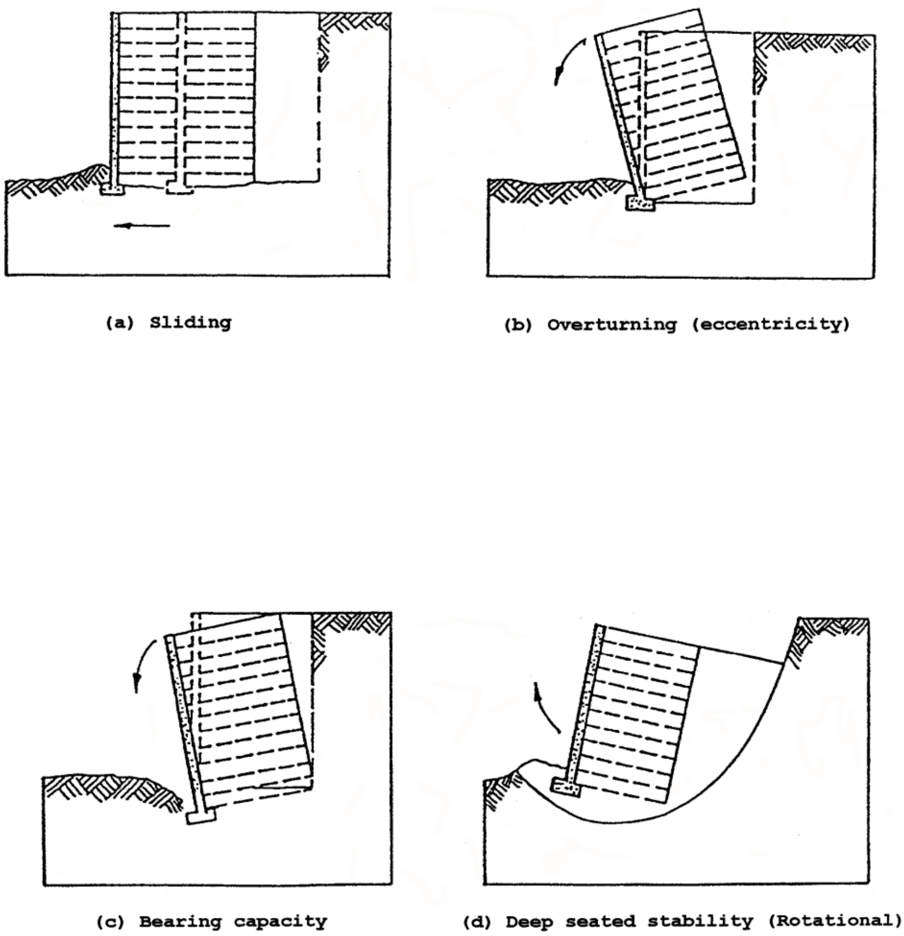


Figure 1. Potential external failure mechanisms for a MSE wall (*source:FHWA FHWA-00-043***)**

b) *Designing for internal stability:*

Internal failure of a MSE wall can occur in two different ways:

- The tensile forces (and, in the case of rigid reinforcements, the shear forces) in the inclusionsbecome so large that the inclusions elongate excessively or break, leading to largemovements and possible collapse of the structure. This mode of failure is called failure byelongation or breakage of the reinforcements.
- The tensile forces in the reinforcements become larger than the pullout resistance, i.e., theforce required to pull the reinforcement out of the soil mass. This, in turn, increases theshear stresses in the surrounding soil, leading to large movements and possible collapse ofthe structure. This mode of failure is called failure by pullout.

1.5 OBJECTIVES:

The main objectivesto be accomplished under this project is to Design a Reinforced Soil Retaining wall of height, 15m to support a 15^{o} inclined backfill soil.

1.6 SCOPE:

- Net bearing pressure is calculated on the basis of IS: 6403-1981.
- Dynamic active earth pressure coefficient is calculated on the basis of Mononobe-Okabe method.
- Coherent gravity (FHWA) has been used to analyse internal rapture.

II. LITERATURE REVIEW

2.1 GENERAL:

Reinforced soil structures, also known as mechanically stabilized earth(MSE) walls were introduced in civil engineering for more than Forty years ago(Vidal 1966). In principle, Reinforced Soil retaining walls can be considered as composite structures where the earth fill stability and capacity to retain backfill are improved by the reinforcements (Schlosser and Vidal 1969; Vidal 1969; Lee et al.1973; Ingold1982).

The modern methods of soil reinforcement for retaining wall construction were pioneered by the French architect and engineer Henri Vidal in the early 1960s. His research led to the invention and development of Reinforced Earth, a system in which steel strip reinforcement is used. The first wall to use this technology in the United States was built in 1972 on California State Highway 39, northeast of Los Angeles. In the last 25 years, more than 23,000 Reinforced Earth structures representing over 70 million m2 (750 million ft2) of wall facing have been completed in 37 countries. More than 8,000 walls have been built in the United States since 1972. The highest wall constructed in the United States was on the order of 30 meters (98 feet).

Geogrids for soil reinforcement were developed around 1980. The first use of geogrid in earth reinforcement was in 1981. Extensive use of geogrid products in the United States started in about 1983, and they now comprise a growing portion of the market.

2.2 REINFORCED SOIL CONCEPTS:

A reinforced soil mass is somewhat analogous to reinforced concrete in that the mechanical properties of the mass are improved by reinforcement placed parallel to the principal strain direction to compensate for soil's lack of tensile resistance. The improved tensile properties are a result of the interaction between the reinforcement and the soil. The composite material has the following characteristics:

1) Stress transfer between the soil and reinforcement takes place continuously along the reinforcement.

2) Reinforcements are distributed throughout the soil mass with a degree of regularity and must not be localized.

Stress Transfer Mechanisms:

Stresses are transferred between soil and reinforcement by friction and/or passive resistance depending on reinforcement geometry:

Friction develops at locations where there is a relative shear displacement and corresponding shear stress between soil and reinforcement surface

Passive resistance occurs through the development of bearing type stresses on "transverse" reinforcement surfaces normal to the direction of soil reinforcement relative movement.

Passive resistance is generally considered to be the primary interaction for rigid geogrids, barmat, and wire mesh reinforcements. The transverse ridges on "ribbed" strip reinforcement also provide some passive resistance

Mode of Reinforcement Action:

The primary function of reinforcements is to restrain soil deformations. In so doing, stresses are transferred from the soil to the reinforcement. These stresses are carried by the reinforcement in two ways: in tension or in shear and bending.

Tension is the most common mode of action of tensile reinforcements. All "longitudinal" reinforcing elements (i.e., reinforcing elements aligned in the direction of soil extension) are generally subjected to high tensile stresses. Tensile stresses are also developed in flexible reinforcements that cross shear planes.

Shear and Bending. "Transverse" reinforcing elements that have some rigidity, can withstand shear stress and bending moments.

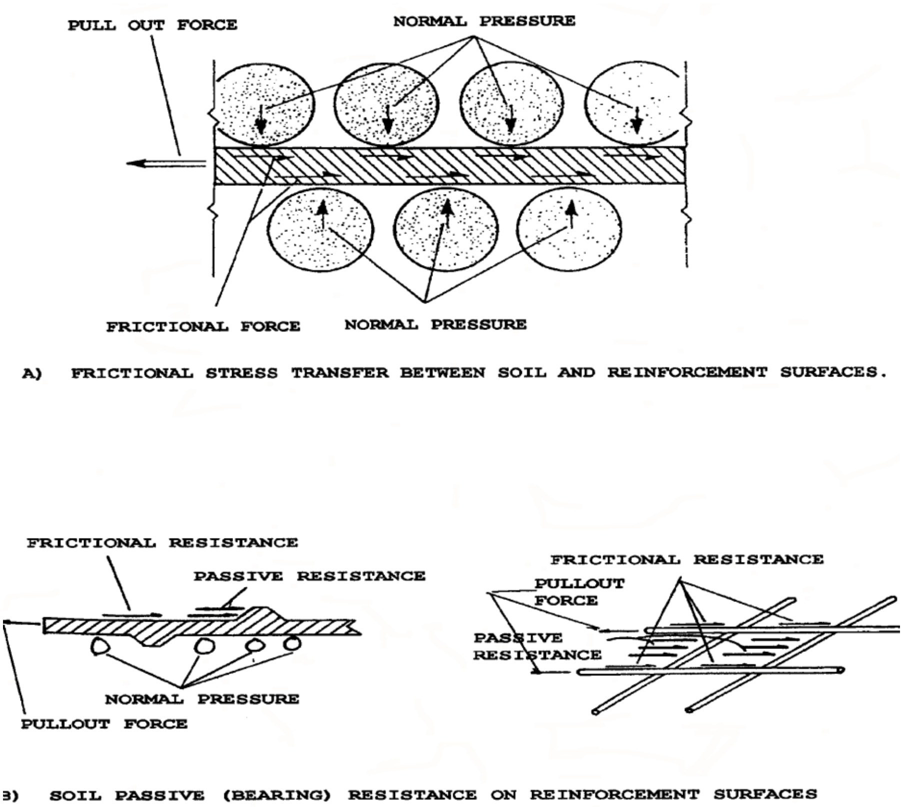


Figure 2.1 Stress transfer mechanisms for soil reinforcement.(*source: FHWA-00-043***)**

2.3 EXTERNAL STABILITY COMPUTATIONAL SEQUENCES ARE SCHEMATICALLY ILLUSTRATED AS FOLLOWS:

a) *Preliminary Sizing*

A preliminary length of reinforcement is chosen that should be greater of 0.7H and 2.5 m, where H is the design height of the structure. Structures with sloping surcharge fills or other concentrated loads, as in abutment fills, generally require longer reinforcements for stability, often on the order of 0.8H to as much as 1.1H.

b) *Earth Pressures for External Stability*

Stability computations for walls with a vertical face are made by assuming that the MSE wall mass acts as a rigid body with earth pressures developed on a vertical pressure plane arising from the back end of the reinforcements.

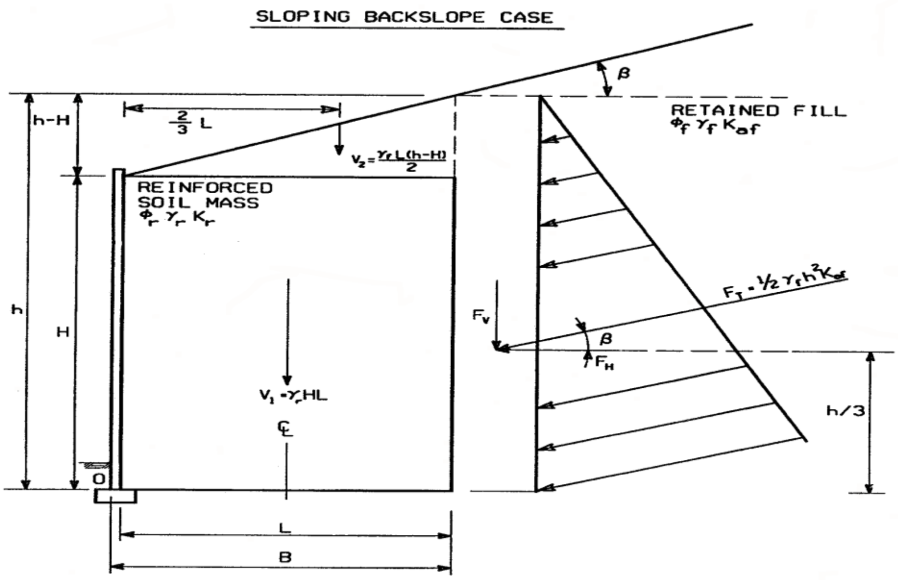
The active coefficient of earth pressure is calculated for vertical walls (defined as walls with a face batter of less than 8 degrees) and a horizontal back slope from:

$$Ka = tan^2(45 - \frac{\varphi}{2}) \quad (3)$$

For vertical wall with a surcharge slope from:

$$Ka = \cos\boldsymbol{\beta}\left[\frac{cos\beta - \sqrt{cos^2\beta - cos^2\varphi}}{cos\beta + \sqrt{cos^2\beta - cos^2\varphi}}\right] (4)$$

Whereβ = surcharge slope angle.



Note: For relatively thick facing elements (e.g., segmental concrete facing blocks) it may be desireable to include the facing dimensions and weight in sliding and overturning calculations (i.e. use "B" in lieu of "L").

Figure 2.2 External analysis: earth pressure/eccentricity; sloping backfill case.

Vertical Pressure Computations

Computations for vertical stresses at the base of the wall defined by the height h are shown on figure 4. It should be noted that the weight of any wall facing is typically neglected in the calculations. Calculation steps for the determination of a vertical bearing stress are:

1) Calculate:

F_T= ½ Kaf (φ, β) γf h2 (5)

2) Calculate eccentricity, e, of the resulting force on the base by summing the moments of the mass of the reinforced soil section about the center line of mass.Noting that R in figure 4 must equal the sum of the vertical forces on the reinforced fill, this condition yields:

$$e = \frac{F_T\,(cos\beta)\,h/3 - F_T\,(sin\beta)\,L/2 - V_2\,(L/6)}{V1+V2+F_T \sin\beta} \quad (6)$$

3) *e* must be less than L/6 in soil or L/4 in rock. If e is greater, than a longer length of renforcement is required

.

4) Calculate the equivalent uniform vertical stress on the base, σv:

$$\sigma v = \frac{V1+V2+FT\ \sin\beta}{L-2e} \quad (7)$$

This approach, proposed originally by *Meyerhof*, assumes that eccentric loading results in a uniform redistribution of pressure over a reduced area at the base of the wall.This areais defined by a width equal to the wall width less twice the eccentricity as shown in **Fig. 5.**

5) Add the influence of surcharge and concentrated loads to σv, where applicable.

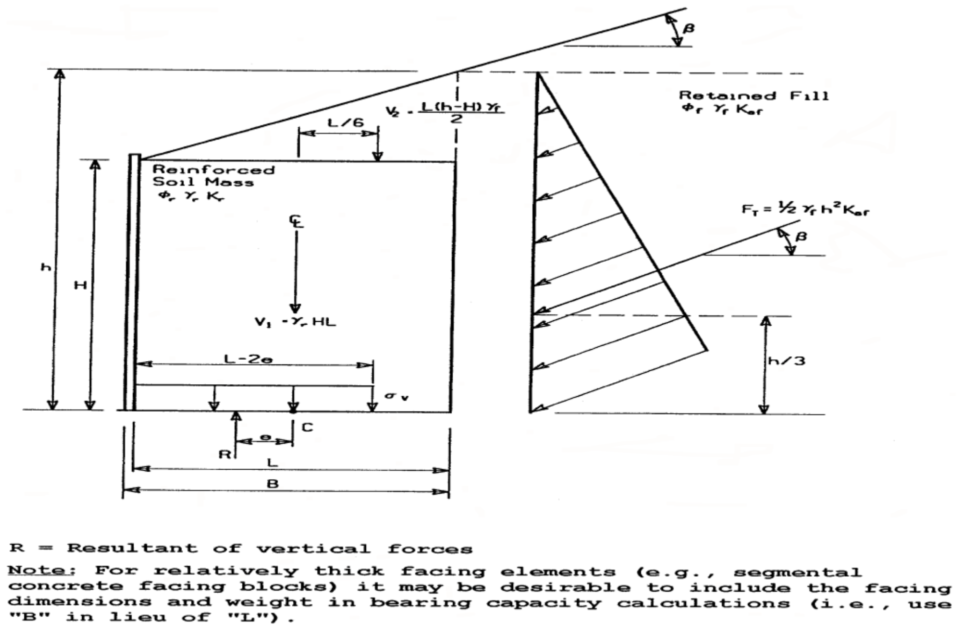


Figure 2.3 Calculation of vertical stress σv at the foundation level.

c) *Sliding Stability*

Check the preliminary sizing with respect to sliding at the base layer, which is the most critical depth as follows:

$FSsliding = \frac{\sum horizontal\ resisting\ forces}{\sum horizontal\ driving\ forces} = \frac{\sum PR}{\sum Pd} \geq 1.5$ (8)

Additional surcharge loads may include live and dead load surcharges

d)*Overturning*

One of the important results from any footing analysis is the ratio of the resisting moment to the overturning moment .This is referred to as the stability ratio or the safety factor for overturning. Most code require that this factor be greater or equal to 1.5.

$FS = \frac{\sum Resisting\ moment}{\sum Overturning\ moment}$ (9)

e)*Bearing Capacity Failure*

Two modes of bearing capacity failure exist, general shear failure and local shear failure. Local shear is characterized by a "squeezing" of the foundation soil when soft or loose soils exist below the wall.

General Shear:

To prevent bearing capacity failure, it is required that the vertical stress at the basecalculated with the Meyerhof-type distribution does not exceed the allowable bearing capacity of the foundation soil determined, considering a safety factor of 2.5 applied to the ultimate bearing capacity:

$\sigma v \leq qa = \frac{q_{ult}}{FS}$(10)

A lesser FS of 2.0 could be used if justified by a geotechnical analysis which calculates settlement and determines it to be acceptable.

Local Shear:

No theoretical solution is available for local shear failure.Local shear failure is common in the case of footings on loose sands orsoft clays.Shear strength parameters c_m and $\varphi_m$should be used in the bearing capacity equation and the bearing capacity factors are determined on the basis of φ_m instead of φ, where

$c_m = \frac{2}{3}c$ (11)

$\tan\varphi_m = \frac{2}{3} tan\varphi$ (12)

BEARING CAPACITY FACTORS

ϕ (Degrees)	N_c	N_q	N_γ
0	5·14	1·00	0·00
5	6·49	1·57	0·45
10	8·35	2·47	1·22
15	10·98	3·94	2·65
20	14·83	6·40	5·39
25	20·72	10·66	10·88
30	30·14	18·40	22·40
35	46·12	33·30	48·03
40	75·31	64·20	109·41
45	138·88	134·88	271·76
50	266·89	319·07	762·89

NOTE — For obtaining values of N'_c, N'_q and N'_γ, calculate $\phi' = \tan^{-1}(0.67 \tan\phi)$. Read N_c, N_q, and N_γ, from the Table corresponding to the value of ϕ' instead of ϕ which are values of N'_c, N'_q, N'_γ respectively.

Table 2.4 Bearing Capacity Factors

f)*Overall Stability*

Overall stability is determined using rotational or wedge analyses, as appropriate, which can be performed using a classical slope stability analysis method. The reinforced soil wall is considered as a rigid body and only failure surfaces completely outside a reinforced mass are considered. For simple structures with rectangular geometry, relatively uniform reinforcement spacing, and a near vertical face, compound failures passing both through the unreinforced and reinforced zones will not generally be critical. However, if complex conditions exist such as changes in reinforced soil types or reinforcementlengths, high surcharge loads, sloping faced structures, significant slopes at the toe or above the wall, or stacked structures, compound failures must be considered.If the minimum safety factor is less than the usually recommended minimum FS of 1.3, increase the reinforcement length or improve the foundation soil.

g) *Seismic Loading for external stability*

During an earthquake, the retained fill exerts a dynamic horizontal thrust,P_{AE}, on the MSE wall in addition to the static thrust. Moreover, the reinforced soil mass is subjected to a horizontal inertia force P_{IR} = MA_m, where M is the mass of the active portion of the reinforced wall section assumed at a base width of 0.5H, and $A_m$is the maximum horizontal acceleration in the reinforced soil wall.

Force P_{AE}can be evaluated by the pseudo-static Mononobe-Okabe analysis as shown in **Fig 5** and added to the static forces acting on the wall (weight, surcharge, and static thrust). The dynamic stability with respect to external stability is then evaluated. Allowable minimum dynamic safety factors are assumed as 75 percent of the static safety factors. The equation forP_{AE} was developed assuming a horizontal backfill, a friction angle of 30 degrees and may be adjusted for other soil friction angles using the Mononobe-Okabe method with the horizontal acceleration equal to $A_m$and vertical acceleration equal to zero.

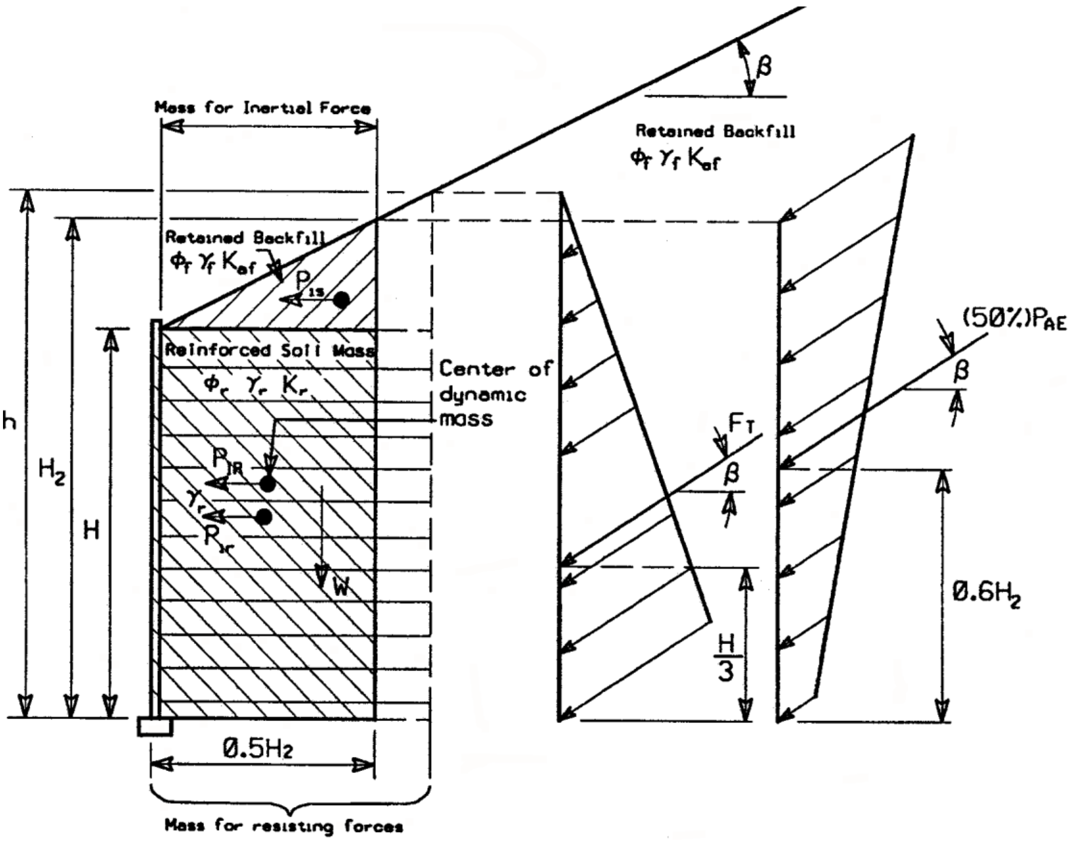


Figure 2.5 Seismic External Stability for Sloping Backfill Condition

2.4DESIGNING FOR INTERNAL STABILITY:

a)*Strength Properties*

Steel Reinforcement:

For steel reinforcements, the design life is achieved by reducing the cross-sectional area of the reinforcement used in design calculations by the anticipated corrosion losses over the design life period as follows:

$$E_c = E_n - E_R \qquad (13)$$

Where
E_c= the thickness of the reinforcement at the end of the design life,
E_n = the nominal thickness at construction, and
E_R= the sacrificial thickness of metal expected to be lost by uniform corrosion during service life of the structure

The allowable tensile force per unit width of reinforcement, Ta, is obtained as follows:

$$Ta = 0.55 \frac{Fy\,Ac}{b} \qquad \textit{for steel strips} \qquad (14)$$

And

$$Ta = 0.48\frac{Fy\,Ac}{b} \qquad \textit{for steel grids connected to concrete panels or blocks} \qquad (15)$$

(***Note**: 0.55 Fy may be used for steel grids with flexible facings (FHWA-00-043)*)

Where:
b = the gross width of the strip, sheet or grid
Fy = yield stress of steel
Ac = design cross section area of the steel, defined as the originalcross section area minus corrosion losses anticipated to occur during the design life of the wall.

b) *Lateral earth pressure coefficient:*
Recent research studies have indicated that the maximum tensile force is primarily related to the type of reinforcement in the MSE mass, which, in turn, is a function of the modulus, extensibility and density of reinforcement. Based on this research, a relationship between the type of the reinforcement and the overburden stress has been developed, and shown in **Fig 6**. The resulting K/Ka for inextensible reinforcement's ratio decreases from the top of wall to a constant value below 6 m (20 ft).

The simplified approach used herein was developed in order to avoid iterative design procedures required by some of the complex refinements of the available methods *i.e*., thecoherent gravity method (AASHTO, 1994 Interims) and the structure stiffness method (FHWA RD 89-043). The *simplified coherent gravity* method is based on the same empirical data used to develop these two methods.

This graphical figure was prepared by back analysis of the lateral stress ratio K from available field data where stresses in the reinforcements have been measured and normalized as a function of an active earth pressure coefficient, Ka. The ratios shown on figure 2 correspond to values representative of the specific reinforcement systems that are known to give satisfactory results assuming that the vertical stress is equal to the weight of theoverburden (γH).

The lateral earth pressure coefficient K is determined by applying a multiplier to the active earth pressure coefficient. The active earth pressure coefficient is determined using a Coulomb earth pressure relationship

$$Ka=\cos\boldsymbol{\beta}\left[\frac{\cos\beta-\sqrt{\cos^2\beta-\cos^2\varphi}}{\cos\beta+\sqrt{\cos^2\beta-\cos^2\varphi}}\right] \quad (16)$$

Where β = surcharge slope angle.

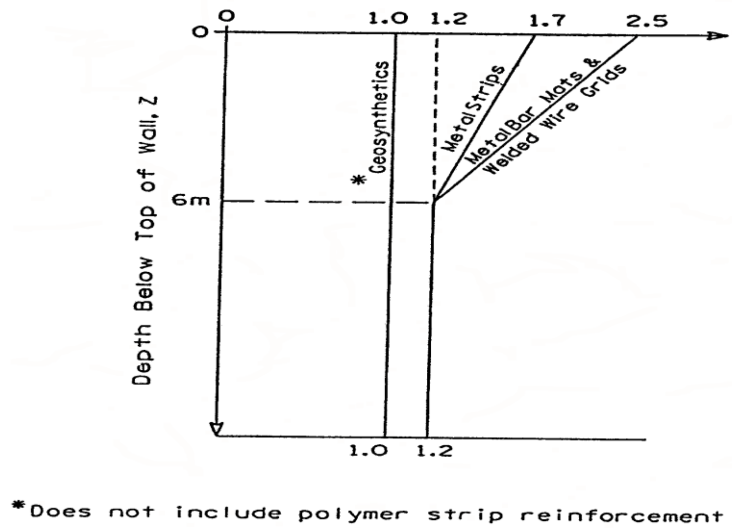


Figure 2.6 Variation of stress ratio with depth in a MSE wall.

c)*Pullout*

The pullout resistance, Pr, of the reinforcement per unit width of reinforcement is given by:

$P_r = F^* .\alpha .\sigma'_v .L_e. C$ (17)

Where: L_e. C = the total surface area per unit width of the reinforcement in the resistive zone behind the failure surface

L_e = the embedment or adherence length in the resisting zone
behind the failure surface

C = the reinforcement effective unit perimeter; e.g., C = 2 forstrips, grids, and sheets

F^* = the pullout resistance (or friction-bearing-interaction) factor

α = a scale effect correction factor to account for a nonlinearstress reduction over the embedded length of highlyextensible reinforcements, based on laboratory data (generally1.0 for metallic reinforcements and 0.6 to 1.0 for geosyntheticreinforcements, see table 5).

σ'_v= the effective vertical stress at the soil-reinforcementinterfaces.

The correction factor α depends, therefore, primarily upon the strain softening of the compacted granular backfill material, the extensibility and the length of the reinforcement. For inextensible reinforcement, α is approximately 1, but it can be substantially smaller than 1 for extensible reinforcements. The α factor (a scale correction factor) can be obtained from pullout tests on reinforcements with different lengths as presented in appendix A or derived using analytical or numerical load transfer models which have been "calibrated" through numerical test simulations. In the absence of test data, α = 0.8 for geogrids and α = 0.6 for geotextiles (extensible sheets) is recommended (see **table 5**).

For any reinforcement,F* can be estimated using the general equation:

F* = Passive Resistance + Frictional Resistance
Or,
$$F^* = F_q \,.\alpha\beta + \tan \rho \qquad (18)$$

Where: Fq = the embedment (or surcharge) bearing capacity factor

αβ = a bearing factor for passive resistance which is based on the thickness per unit width of the bearing member.

ρ = the soil-reinforcement interaction friction angle.

The pullout capacity parameters for equation 18 are summarized in **Table 2**.

Table 2. Summary of pullout capacity design parameters.

Reinforcement type	S_{opt}	Grid spacing	Tan ρ	F_q	α_β	α Default Value
Inextensible strips		NA	Obtain Tan ρ from tests, or use default values	NA	NA	1.0
Inextensible grids(bar mats and welded wire)	$\frac{t(F_q)}{(2Tan\emptyset)}$	$S_t \le S_{opt}$	Obtain Tan ρ from tests	NA	NA	1.0
	$\frac{t(F_q)}{(2Tan\emptyset)}$	$S_t > S_{opt}$	NA	Obtain F_q from tests or use default values	$\frac{t}{2S_t}$	1.0
Extensible grids:						
(Min. grid opening)/d_{50}>1	$\frac{t(F_q)}{(2Tan\emptyset)}$	$S_t \le S_{opt}$	Obtain Tan ρ from tests	NA	NA	0.8
	$\frac{t(F_q)}{(2Tan\emptyset)}$	$S_t > S_{opt}$	NA	Obtain F_q from tests, or use default values	$\frac{(f_b\ t)}{(2S_t)}$	0.8
(Min.grid opening)/d_{50}<1		NA	Obtain Tan ρ from tests	NA	NA	0.8
Extensible sheets		NA	Obtain Tan ρ from tests	NA	NA	0.6

NOTES:
It is acceptable to use the empirical values provided in or referenced by this table to determine F* in the absence of product and backfill specific test data, provided granular backfill as specified in Article 7.3.6.3 of Division II of 1996 AASHTO Standard Specifications for Highway Bridges is used and Cu≥4. For backfill outside these limits, tests must be run.
Pullout testing to determine α is recommended if α shown in table is less than 1.0. These values of α represent highly extensible geosynthetics.
For grids where Tan ρ is applicable, apply Tan ρ to the entire surface area of the reinforcement sheet (i.e., soil and grid), not just the surface area of the grid elements.
NA means "not applicable." φ is the soil friction angle. ρis the interface friction angle mobilized along the reinforcement. Sopt is the optimum transverse grid element spacing to mobilize maximum pullout resistance as obtained from pullout tests (typically 150 mm or greater). St is the spacing of the transverse grid elements. t is the thickness of the transverse elements. Fq is the embedment (or surcharge) bearing capacity factor. $\alpha_\beta$is a structural geometric factor for passive resistance. fb is the fraction of the transverse member on which bearing can be fully developed (typically ranging from 0.6 to 1.0) as obtained from an evaluation of the bearing surface shape. d50 is the backfill grain size at 50% passing by weight. αis the scale effect correction factor.

d)*Seismic loading for internal stability*

Seismic loads produce an inertial forceP_I acting horizontally, in addition to the existing static forces.

This force will lead to incremental dynamic increases in the maximum tensile forces in the reinforcements. It is assumed that the location and slope of the maximum tensile force line does not change during seismic loading.

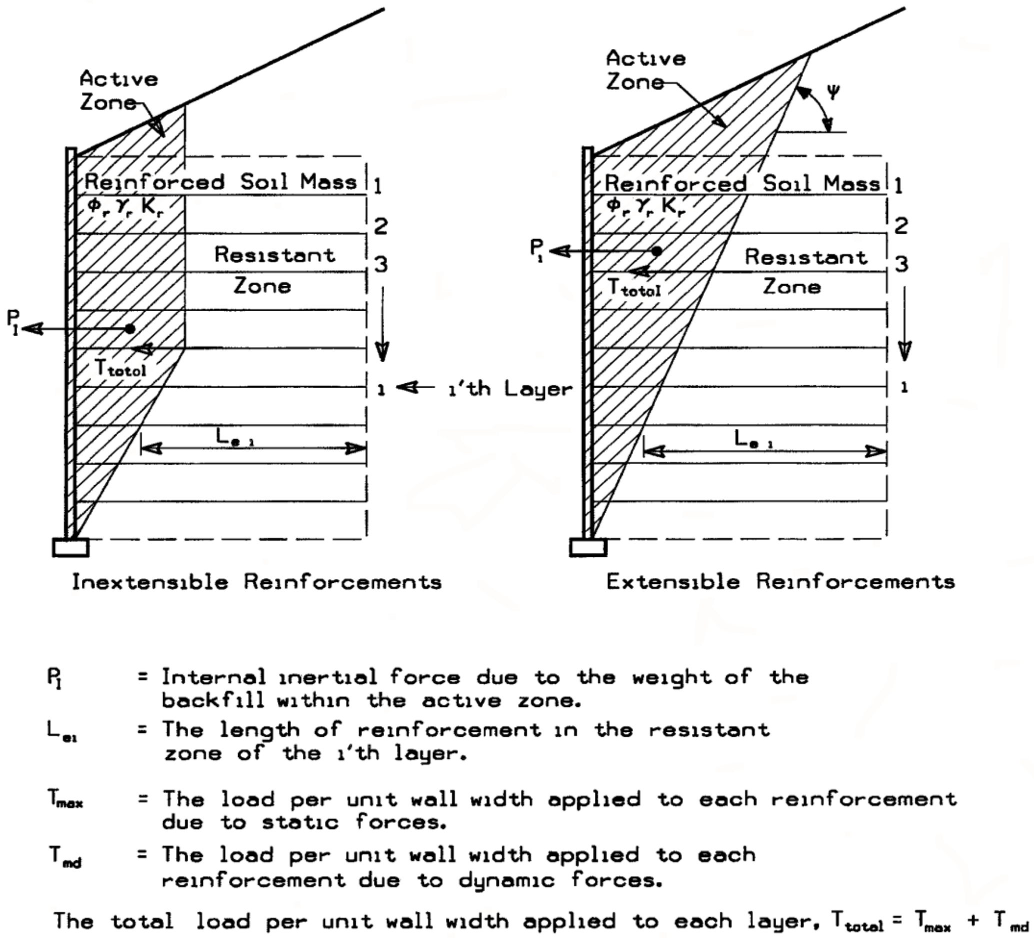


Figure 7. Seismic internal stability of a MSE wall(*source: FHWA-00-043***).**

III. DESIGN METHODOLOGY

This chapter discusses about the design steps to be followed inorder accomplish the objectives of the project.

3.1 SEQUENTIAL STEPS FOR EXTERNAL STABILITY:

3.1.1. Sliding Stability

The calculation steps for an MSE wall with a sloping backfill are (Fig. 4):

(1) Calculate thrust:
$F_p = Kaf(\varphi, \beta)\ ½\ \gamma_f h^2$ (19)

where $h = H + L \tan\beta$

(2) Calculate the driving force:

$P_d = F_H = F_p \cos\beta$. (20)

(3) Calculate the factor of safety with respect to sliding and check if it is greater than the required value, using **Eqn. (8).**

(4) Ifnot; Increase the reinforcement length, L, and repeat the calculations

3.1.2 Overturning

$$FS = \frac{\sum Resisting\ moment}{\sum Overturning\ moment} \quad (21)$$

3.1.3. Bearing Pressure

Calculation steps are as follows:

(1) Obtain the eccentricity e of the resulting force at the base of the wall. Remember that under preliminary sizing if the eccentricity exceeded L/6, the reinforcement length at the base was increased.

(2) Calculate the vertical stress σ_v at the base assuming Meyerhof-type distribution
$\sigma_v = \frac{V1+V2+FT\ sin\beta}{L-2e}$ (22)

(3) Determine the net bearing capacity q_{nu} using classical soil mechanicsmethods:

$$q_{nu} = cN_c S_c d_c i_c + \gamma_1 (N_q - 1) S_q d_q i_q r_w + 0.5\gamma_2 B N_\gamma S_\gamma d_\gamma i_\gamma r'w' \quad (23)$$

Where,

$$S_c = S_q = S_\gamma = 1\ for\ strip\ footing$$

$$d_c = 1 + 0.2\frac{D_f}{B} tan\left(45 + \frac{\varphi}{2}\right)$$

$$d_q = d_\gamma = 1 + 0.1\frac{D_f}{B} tan\left(45^\circ + \frac{\varphi}{2}\right) for\ \varphi > 10^\circ$$

$$= 1\ for\ \varphi < 10^\circ$$

$$i_c = i_q = \left(1 - \frac{\alpha}{90}\right)^2.$$

$$i_\gamma = \left(1 - \frac{\alpha}{\varphi}\right)^2 \alpha\ in\ degrees$$

Values of $N_c, N_q \& N_\gamma$ is obtained from **table 1**.

3.1.4 Overall Slope Stability

Bishop's simplified method

In the 1950's Professor Bishop at Imperial College in London devised a method which included interslice normal forces, but ignored the interslice shear forces. Bishop developed an equation for the normal at the slice base by summing slice forces in the vertical direction. The consequence of this is that the base normal becomes a function of the factor of safety. This in turn makes the factor of safety equation nonlinear (that is, FS appears on both sides of the equation) and an iterative procedure is consequently required to compute the factor of safety.

A simple form of the Bishop's Simplified factor of safety equation in the absence of any pore-water pressure is:

$$FS=\frac{1}{\Sigma W \sin\alpha}\Sigma\left[\frac{c\beta+W\tan\phi'-\frac{c\beta}{FS}\sin\alpha\tan\phi'}{m\alpha}\right] \quad (24)$$

FS is on both sides of the equation as noted above. The equation is not unlike the Ordinary factor of safety equation except for the m_a term, which is defined as:

$$m_a = \cos\alpha + \frac{\sin\alpha\tan\phi'}{FS} \quad (25)$$

To solve for the Bishop's Simplified factor of safety, it is necessary to start with a guess for FS. In SLOPE/W, the initial guess is taken as the Ordinary factor of safety. The initial guess for FS is used to compute $m_\alpha$and then a new FS is computed. Next the new FS is used to compute $m_\alpha$and then another new FS is computed.

The procedure is repeated until the last computed FS is within a specified tolerance of the previous FS. Fortunately, usually it only takes a few iterations to reach a converged solution. Now if we examine the slice free body diagrams and forces polygons for the same slices as for the Ordinary method above, we see a marked difference (**Fig 3.1**). The force polygon closure is now fairly good with the addition of the interslice normal forces. There are no interslice shear forces, as assumed by Bishop, but the interslice normal forces are included.

In a factor of safety versus lambda plot, as in **Fig 3.2**, the Bishop's Simplified factor of safety falls onthe moment equilibrium curve where lambda is zero (*FS* = 1.36). Recall that

$X = E\lambda f(x)$

The interslice shear is not included by making lambda zero.

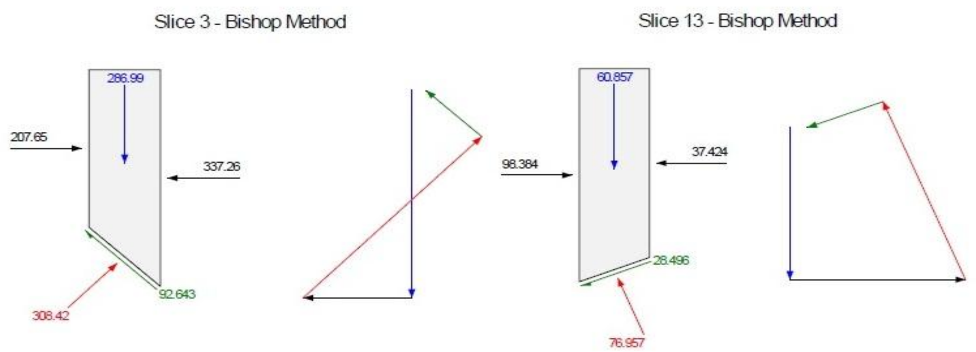


Figure 3.1: Free body diagram and force polygon for the Bishop's Simplified method (*source: Geoslope 2012*)

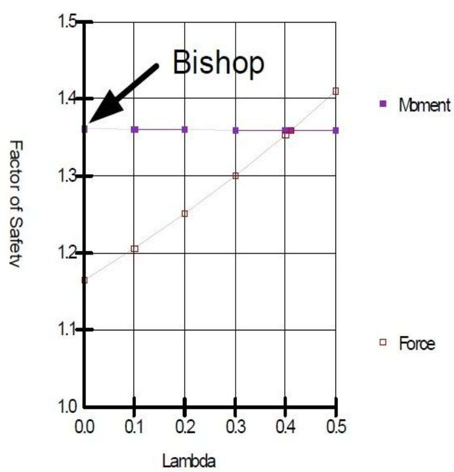


Figure 3.2: Bishop's Simplified factor of safety (*source: Geoslope 2012*)

In this case the moment factor of safety (*Fm*) is insensitive to the interslice forces. The reason for this, as discussed in the previous chapter, is that no slippage is required between the slices for the sliding mass to rotate. This is not true for force equilibrium and thus the force factor of safety (*Ff*) is sensitive to the interslice shear.

3.1.5 Seismic Loading

The seismic external stability evaluation is performed as follows:

- Calculate themaximum ground acceleration coefficient A:

$A=\frac{Z\ I\ S_a}{2Rg}$(26)

- Calculate the maximum acceleration Am developed in the wall:

Am = (1.45 - A) A(27)

where:Am = max. wall acceleration coefficient at the centroid of the wall mass.

- Add to the static forces (see figure 5) acting on the structure, 50 percent of theseismic thrust P_{AE} and the full inertial forceP_{IR} . The reduced PAE is used becausethese two forces are unlikely to peak simultaneously.

- For structures with sloping backfills, the inertial force (P_{IR}) and the dynamichorizontal thrust (P_{AE}) shall be based on a height H2 near the back of the wall massdetermined as follows:

$H2=H+\frac{tan\beta.0.5H}{(1-0.5tan\beta)}$(27)

P_{AE}may be adjusted for sloping backfills using Mononobe-Okabe method, with thehorizontal acceleration kh equal to Am and kv equal to zero. A height of H2 should beused to calculate P_{AE}in this case.P_{IR} for sloping backfills should be calculated as follows:

PIR=Pir+Pis(28)
Pir= 0.5 Am γfH2H (29)
Pis= 0.125 Am γf(H2)2 tan β (30)
And
PAE = 0.5 γf(H2)2 ΔKAE (sloping backfill) (31)

Where P_{ir} is the inertial force caused by acceleration of the reinforced backfill and P_{is} is the inertial force caused by acceleration of the sloping soil surcharge above the re-inforced backfill, with the width of mass contributing to P_{IR} equal to 0.5H2. Pir acts at the combined centroid ofP_{ir} and Pis as shown in figure. The total seismic earth pressure coefficient K_{AE}based on the Mononobe-Okabe general expression is computed from:

$$K_{AE}=\frac{\cos2\ (\varphi-\Psi-\alpha)}{\cos\Psi\ cos^2\alpha\cos(\delta+\alpha+\Psi)\left(1+\sqrt{\frac{\sin\ (\varphi+\delta)\sin(\varphi-\Psi-\beta)}{\cos\ (\delta+\alpha+\Psi)\cos(\beta-\alpha)}}\right)^2} \quad (32)$$

Where:

K_v = coefficient of vertical acceleration of soil wedge
K_h =Coefficient of horizontal acceleration of soil wedge
Ψ = tan^{-1} (Kh/1 - Kv)φ = friction angle of backfill
δ =friction angle at wall-backfill interface
α = angle between inner face of wall and vertical
β= backfill slope with respect to horizontal

To complete design:

- Evaluate sliding stability, eccentricity and bearing capacity as detailed in the previoussections.
- Check that the computed safety factors are equal to or greater than 75 percent of theminimum static safety factors, and that the eccentricity falls within L/3 for both soi l and rock.

3.2 SEQUENTIAL STEPS FOR INTERNAL STABILITY:

3.2.1 Rapture

Calculations steps are as follows:

(1) Calculate at each reinforcement level the horizontal stresses σH along the potentialfailure line from the weight of the retained fill γrZ plus, if present, uniform surcharge loads q concentrated surcharge loads $\Delta\sigma_v$.

$\sigma H = K_r \sigma_v$(33)

Where,
$\sigma_v = \gamma_r Z + \sigma_2$

$K_r = K(z)$

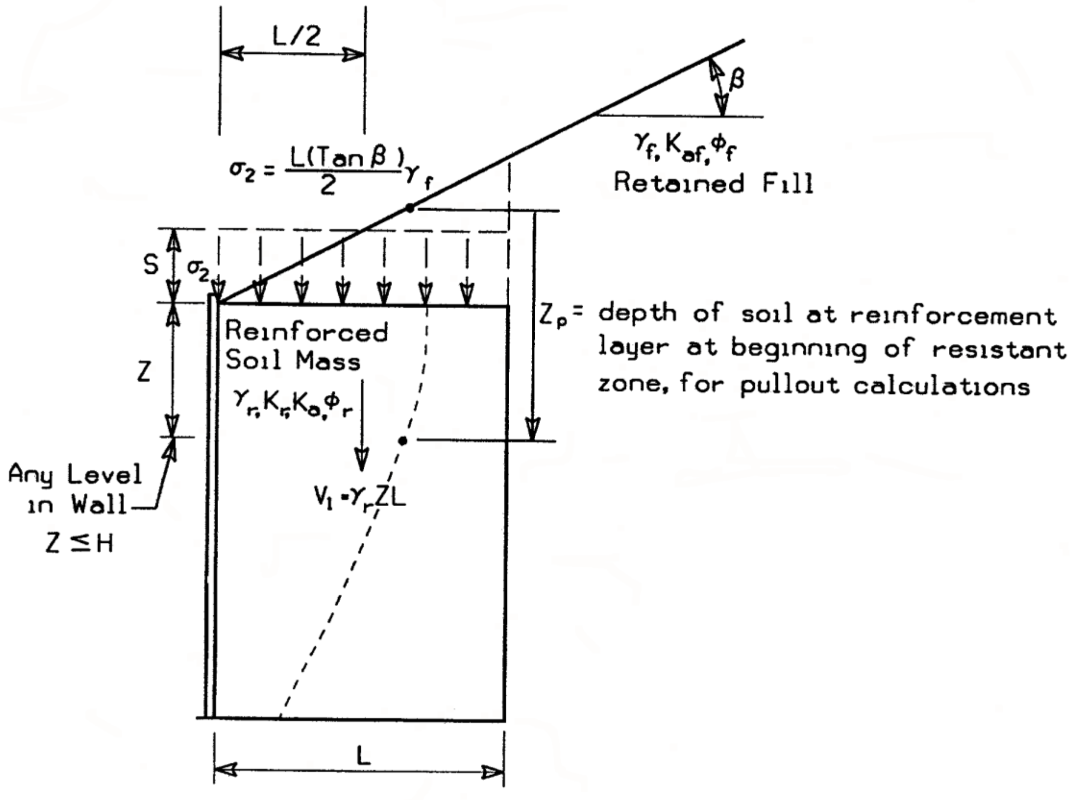


Figure 8. Calculation of vertical stress for sloping backslope conditions.

(2) Calculate the maximum tension Tmax in each reinforcement layer per unit width of wall based on the vertical spacing Sv from:

$T\text{max} = \sigma_H . S_v$ (34)

(3) Calculate internal stability with respect to breakage of the reinforcement. Stability with respect to breakage of the reinforcements requires that

$T_a \geq \frac{T_{max}}{R_c}$ (35)

Where Rc is the coverage ratio b/Sh,

b= gross width of the reinforcing element
Sh= center to center horizontal spacing between reinforcements

3.2.2Pullout

The pullout resistance, Pr, of the reinforcement per unit width of reinforcement is calculated by using **Eqn (17)**

$Pr = F^* .\alpha .\sigma'v \ .Le \ .C$

3.2.3 Seismic Loading
Calculation steps for internal stability analyses with respect to seismic loading are as follows:
(1) Calculate the maximum acceleration in the wall and the force PI per unit width acting above the base:
$P_I = A_m W_A$(36)
$A_m = (1.45 - A) A$ (37)

where: $W_A$is the weight of the acactive zone;
A is the acceleration coefficient and
A_m may be reduced based on the permissible lateral movement

(2) Calculate the total maximum static load applied to the reinforcements horizontal Tmax as follows:

Calculate horizontal stress σHusing K coefficient (previously discussed)

$$\sigma H = K\sigma v = K\gamma Z \quad (38)$$

Calculate the maximum tensile load component Tmaxper unit width:

$Tmax = Sv\sigma H$(39)

(3) Calculate the dynamic increment Tmddirectly induced by the inertia force PI in the reinforcements by distributing PI in the different reinforcements proportionally to their "resistant area" (Le) on a load per unit wall width basis. This leads to:

$T\,md = P_I \frac{L_{ei}}{\sum_{i=1}^{n}(L_{ei})}$(40)
which is the resistant length of the reinforcement at level i divided by the sum of theresistant length for all reinforcement levels.

(4) The maximum tensile force is:

$$T\,total = T\max + Tmd \quad (41)$$

Check stability with respect to breakage and pullout of the reinforcement, withseismic safety factors of 75 percent

3.2.4 Fundamental Time Period:

Richardson and Lee proposed that the fundamental period, T, of MSE walls constructed with steel strip reinforcement can be estimated empirically using **Eqn. (41).**

$T=0.020H$ *to* $0.033H$ (41) where H is the height of the wall in meters and T gives you the natural period of the wall in seconds. Converting the height of the MSE wall model to meters and multiplying by 0.03 gives the result of T= 0.13s which matches what was found by LS-Dyna. The fundamental frequency of the MSE walls studied by Hatami and Bathurst were found to have frequencies of 32.0 to 52.2 rad/s^2using this relationship.

3.3 Introduction to Geostudio-8.15.5.11777(SLOPE/W)

SLOPE/W is one component in a complete suite of geotechnical products called GeoStudio. SLOPE/W, in one form, or another has been on the market since 1977. The initial code was developed by Professor D.G. Fredlund at the University of Saskatchewan. The first commercial version was installed on mainframe computers and users could access the software through software bureaus. Then in the 1980s when Personal Computers (PCs) became available, the code was completely re-written for the PC environment. The software was renamed as SLOPE/W from PC-SLOPE to reflect the Microsoft Windows environment and that it now had a graphical user interface. SLOPE/W was the very first geotechnical software product available commercially for analyzing slope stability.

One of the powerful features of this integrated approach is that it opens the door to types of analyses of a much wider and more complex spectrum of problems, including the use of finite element computed pore-water pressures and stresses in a stability analysis. Not only does an integrated approach widen the analysis possibilities, it can help overcome some limitations of the purely limit equilibrium formulations.

3.3.1 Method Basics

Many different solution techniques for the method of slices have been developed over the years. Basically, all are very similar. The differences between the methods are depending on: what equations of statics are included and satisfied and which interslice forces are included and what is the assumed relationship between the interslice shear and normal forces?

In this project Bishop's simplified method (**Refer clause 3.1.4**) is used to calculate the critical circle using Factor of Safety approach.

IV. DETAIL DESIGN

4.1 PROBLEM STATEMENT

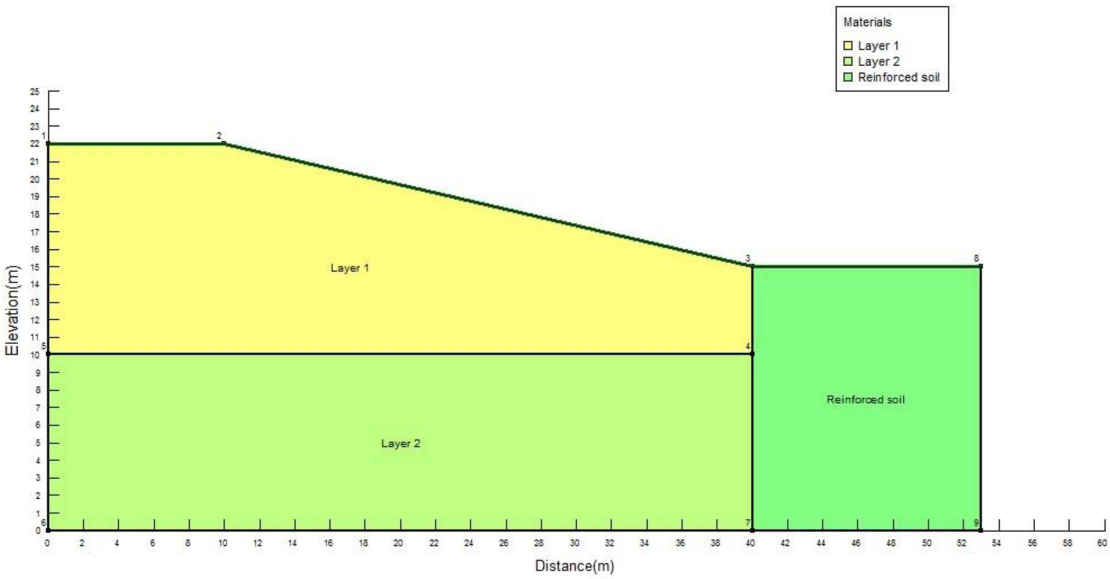


Figure 4.1: Soil layer along with the Reinforced Soil

For a vertical back reinforced soil retaining wall, following properties is considered:-

1. Engineering properties for retained soil(**Fig. 4.1**):

G=2.68,β=15°
Layer 1;
ϕ=26°, γ =16kN/m^3, c=3kN/m^3, e=0.35
Layer 2;
ϕ =30°, γ =18kN/m^3, c=5kN/m^3, e=0.4

2. Engineering properties for Reinforced Soil:

ϕ=34˚, γ=20 kN/m^3
3. Design height of the wall:
H=15m
4. Type of Reinforcement:
Inextensible welded wire of 11.5 mm diameter
$F_{y=}$450 MPa
Longitudinal wire spacing=150 mm
Transverse wire spacing=230 mm

4.2EXTERNAL STABILITY:

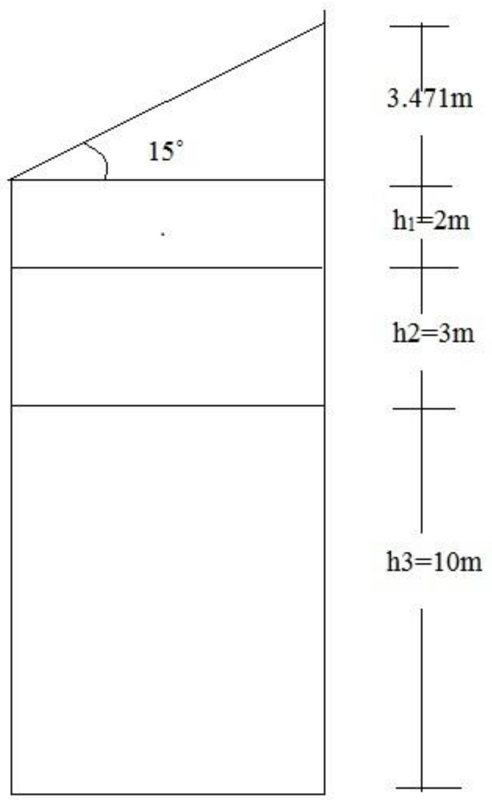


Figure 4.2 Details of the Backfill Soil

Height of the wall= 15m
Slope of Backfill (β) = 15˚
Properties of layer 1 backfill soil:
Submerged unit weight of soil:
$(\gamma'_{1-b}) = (\frac{G-1}{1+e}).\gamma w$
$= (\frac{2.68-1}{1+0.4}) * 9.81$
=11.772kN/m^3
Active Earth pressure Coefficient, Ka_1=0.624

Properties of layer 2 backfill soil:
Submerged unit weight of soil
(γ'_{2-b})=12.208kN/m^3
Activeearth pressure coefficient, K_{a2}= 0.386

Properties of reinforced soil: Active Earth pressure Coefficient, $(K_r) = \frac{1-\sin(\phi r)}{1+\sin(\phi r)}$

$=0.282$

4.2.1 Lateral Thrust:

$\sigma_1 = \gamma_{1\text{-}b}.K_{a1}.h_1$

$\sigma_1 = 16\times0.624\times5.471$

$=54.622\text{kN}/m^2$

$\sigma_2 = 11.772\times0.624\times3$

$=22.0371\text{kN}/m^2$

$\sigma_3 = 12.208\times0.386\times10$

$=47.122\text{kN}/m^2$

$P_1 = \frac{1}{2}\times\sigma 1\times h1$

$= 149.419\text{kN}$

$P_2 = \sigma_1\times13$

$=710.092\text{kN}$

$P_3 = \frac{1}{2}\times\sigma_2\times h2$

$=33.0557\text{kN}$

$P_4 = \sigma_2\times h_3$

$=220.371\text{kN}$

P5 $P_5 = \frac{1}{2}\times\sigma_3\times h3$

$=235.611\text{kN}$

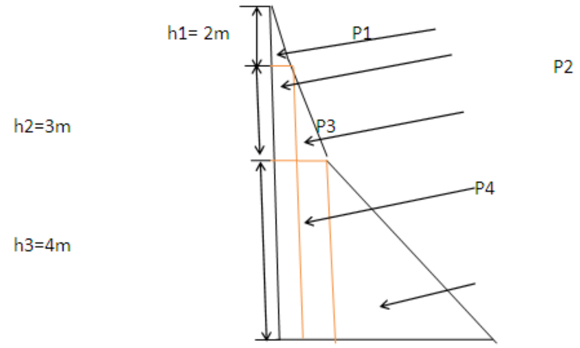


Figure 4.3 pressure diagram

Total lateral thrust, $P=P_1+P_2+P_3+P_4+P_5$

$=1348.554\text{kN}$

4.2.2 Overturning Moment:

$M_1=P_1\times(13+\frac{5.471}{3})$

$=2214.949\text{kNm/m}$

$M_2=P_2\times(\frac{13}{2})$

$=4615.598\text{kNm/m}$

$M_3=P_3\times(10+\frac{3}{3})$

$=363.6135\text{kNm/m}$

$M_4=P_4\times(\frac{10}{2})$

$=1101.859\text{kNm/m}$

$M_5=P_5\times(\frac{10}{3})$

$=785.381\text{kNm/m}$

Total overturning moment, $M_o= M_1+M_2+M_3+M_4+M_5$

$=9081.401\text{kNm/m}$

Factor of safety:

Length of reinforced block=0.7H (*min*) (***FHWA***)

=10.5m

Length is taken as (L) 13m,

$h=H+L\tan(\beta)$

$=15+13\tan(15) =18.471\text{m}$

$h'=18.471-15=3.471\text{m}$

$v_1= \gamma_r \times L\times H$ $=3900\text{kN}/m$

$v_2=\frac{\gamma 1-b\times L\times h'}{2}$

$=360.984\text{kN}/m$

P=1348.554$cos(15)$=1302.603kN/m
Resistance to Lateral Sliding =(v_1+v_2+$Psin(\beta))tan(\phi_r)$
=3109.447kN/m
$FOS_{sliding}=\frac{resistance}{lateral\ thrust}=\frac{3109.447}{1302.603}$
=2.22 >1.5 *Ok*
m_1= 20×15×13×$\frac{13}{2}$=16900kNm/m
m_2= 360.984×$\frac{2}{3}$×13=3130.092kNm/m
m_p=1348.554$sin(15)$×$\frac{13}{2}$=2268.625kNm/m
Resisting moment, $M_r=m_1+m_2+m_p$
=22298.72kNm/m
Overturning Moment, M_o= 9081.401cos (15)
= 8771.725kNm/m
$FOS_{overturning}=\frac{Mr}{Mo}=\frac{22298.72}{8771.725}$ =2.542> 2
Ok

4.2.3 Bearing Pressure on Foundation Soil:

Angle of friction for the 2nd layer, $\phi_{2\text{-}b}$=30°
IS:6403-1981
Bearing capacity factors: N_c=34.169, N_q=23.026, N_γ=31.744,
Depth factor : d_c =1.04, d_q=1.0198, d_γ=1.0198
Inclination factor : i_c=0.774, i_q=0.774, i_γ=0.387
Shape factor : S_c=1, S_q=1, S_γ=1 (*strip footing*)

$$Q_{nu}= cN_cS_cd_ci_c + \gamma_1(N_q-1)S_qd_qi_qr_w + 0.5\gamma_2BN_\gamma S_\gamma d_\gamma i_\gamma r'w'$$

=1742.405kN/m^2

$Q_{ns}=\frac{1742.405}{2.5}$ =696.9619kN/m^2

Eccentricity (e) =1.07 <$\frac{L}{6}$
Bearing pr.(σ_{vb}) = 425.025kN/m^2<Q_{ns}

4.2.4 Seismic Loading:

Zone (Z) =0.36
Importance factor (I)= 1
Response reduction factor (R)=3
Spectral acceleration coefficient(Sa/g)=2.5

Base acceleration coefficient (A)=$\frac{ZI}{2R}\frac{Sa}{g}$
=0.15

Average acceleration in the soil (A_m) =0.195
H_2= 17.3205m

$K_h=A_m$=0.15, $K_v=\frac{2}{3}k_h$ =0.1

$\phi_{1\text{-}b}$=20, $\delta_1=\frac{2}{3}\phi_{1\text{-}b}$=13.33, α=0°

$\phi_{2\text{-}b}$=30, δ_2=20

$\theta=tan^{-1}(\frac{Kh}{1.Kv})$ =9.462

K_{ae1}=0.716, K_{ae2}=0.607

$\Delta K_{ae1} = K_{ae1}\text{-}K_{a1}$
=0.092

ΔK_{ae2}=0.221

Dynamic horizontal thrust, $P_{ae1} = 0.5.\gamma_{1\text{-}b}.H2^2.\Delta K_{ae1}$
=18.552kN/m

$P_{ae2} = 0.5.\gamma_{2\text{-}b}.H2^2.\Delta K_{ae2}$
=134.898kN/m

$P_{ae} = P_{ae1}+P_{ae2}$
=153.45kN/m

Inertial force
$P_{IR} = P_{ir} + P_{is}$
$P_{ir} = 0.5\ Am\ \gamma r H_2 H$
= 31.204kN/*m*
$P_{is} = 0.125\ Am\ \gamma_{1\text{-}b}(H_2)2\ \tan\beta$
=506.347kN/*m*
P_{IR} =537.551kN/*m*

4.2.4.1 Factor of Safety

Sliding:

Total thrust, $P' = P+P_{IR}+0.5P_{ae}$
= 1348.554+537.551+0.5*153.45
= 1962.831kN/m
Resistance= $(v_1+v_2+P'\sin(\beta))(\phi_r)$
=3216.667kN/m
Fossliding =$\frac{Resistance}{total\ thrust}$ =1.63 >1.125 *Ok*

Overturning moment:

Overturning moment, $M_o' = M_o+P_{ae}*0.6H_2+P_{IR}*0.5H_2$
=9081.401cos(15)+76.725*0.6*17.32+537.551*0.5*17.32
=14221.43kNm/m
$m_p' = P'\sin(\beta)*\frac{13}{2}$

Resisting moment, $Mr = m_1+m_2+m_p'$
=23331.97kNm/m

$FOS_{overturning} = \frac{Mr}{Mo'} = \frac{23331.97}{14221.43}$ =1.6406 >1.5 *Ok*

4.2.5 Bearing Pressure on Foundation Soil:

v_1=3900kN/m;v_2 =360.984kN/m; P'=1962.831kN/m
h=18.483m
Eccentricity, e= 1.59<$\frac{L}{6}$
Bearingpressure .(σ_{vb}) = 485.648kN/m^2< Q_{ns}=696.961kN/m^2

4.3SPACING OF REINFORCEMENT:

Design Allowable strength (Ta) =0.48$\frac{FyAc}{b}$
F_y=450Mpa

A_c=(no. of longitudinal bars).$\frac{\pi.D'^2}{4}$
D=12.8mm

For corrosion:
Zinc loss = 15 μm (first 2 years)
= 4 μm (thereafter)
Carbon steel loss = 12 μm

Service life of Zinc coating (86 µm) is:
Life = 2 yrs. + 86 - 2 (15) = 2 years + 14 years = 16 years

The base carbon steel will lose section for:
75 years - 16 years = 59 years at a rate of 12 µm/year/side.
Therefore, the anticipated loss is:
ER = 12(59)2 = 1.416 mm and
Diameter at the end of design life D' = 11.5- 1.416
= 10.084mm

A_c= 3×$\frac{\pi\times D'^2}{4}$ =239.594 mm^2
Transverse wire spacing (b) =230mm
Longitudinal wire spacing =150mm

Ta=0.48×$\frac{450\times239.594}{3\times230}$

=74.965kN/m
Ka=0.282; σvb=485.648kN/m^2
Sv(spacing)=$\frac{74.965}{485.648\times0.282}$ =0.547m
Hence at 15m depth the spacing that can be provided =500mm

Ft	H	h'	v2	v1	e	h1	σvb	sv
1962.831	15	18.471	360.984	3900	1.5913	3.471	485.7704	0.547244
1758.824	14	17.471	360.984	3640	1.3807	3.471	435.2334	0.610787
1565.675	13	16.471	360.984	3380	1.1787	3.471	389.5823	0.682359
1383.383	12	15.471	360.984	3120	0.985	3.471	348.0537	0.763776
1211.948	11	14.471	360.984	2860	0.7995	3.471	310.0266	0.857459
1051.369	10	13.471	360.984	2600	0.6215	3.471	274.9904	0.966706
901.647	9	12.471	360.984	2340	0.4503	3.471	242.5203	1.096135
762.786	8	11.471	360.984	2080	0.285	3.471	212.2594	1.252406
634.778	7	10.471	360.984	1820	0.1237	3.471	183.9045	1.445506
517.625	6	9.471	360.984	1560	-0.036	3.471	157.1964	1.691101
411.331	5	8.471	360.984	1300	-0.199	3.471	131.9124	2.015239
315.184	4	7.471	360.984	1040	-0.374	3.471	107.8408	2.465068
231.719	3	6.471	360.984	780	-0.574	3.471	84.8869	3.131637
160.936	2	5.471	360.984	520	-0.834	3.471	62.90213	4.226168
102.711	1	4.471	360.984	260	-1.246	3.471	41.7984	6.359932

Table 4.1 Calculation of Eccentricity

4.4 INTERNAL STABILITY:

4.4.1 Internal Stability with Respect to Rupture of the Reinforcement

a) Tensile force due to inertial effects:

$$H1=H+\frac{\tan(\beta)*0.3H}{1-0.3\tan(\beta)}$$

H1=16.311m ; h1=6m; h2=9 m

Total vertical force from the wedge

$R_v= 0.3H*h_1*\gamma r+\frac{1}{2}*h_2*0.3H*\gamma_r$

=0.3*15*6*20+0.5*9*0.3*15*20

=945kN/m

Inertial force generated, $P_I= R_v*A_m$

=945*0.195

=184.275kN/m

Distribution of the above forces into each reinforcement layer is proportional to the anchorage length of each layer

$T_{md}= P_I*\frac{Lei}{\Sigma Lei}$

$L_{ei}=L_e-\frac{(H-zi)}{tan\ (45+\frac{\emptyset}{2})}$

L_{ei} for a depth of 15m

$L_{ei}=13-\frac{15-14.75}{tan\ (45+\frac{34}{3})}$

=12.867 m

$T_{md}(15)$= 184.275*$\frac{12.867}{202.507}$

=11.708kN/m

Layer	***Z***	**L_{ei}**	**T_{md}**
21	0.25	8.1067	7.3768
20	0.75	8.1067	7.3768
19	1.50	8.1067	7.3768
18	2.25	8.1067	7.3768
17	3.00	8.1067	7.3768
16	3.75	8.1067	7.3768
15	4.50	8.1067	7.3768
14	5.25	8.1067	7.3768
13	6.00	8.1067	7.3768
12	6.75	8.6133	7.8378
11	7.50	9.0122	8.2007
10	8.25	9.4109	8.5636
9	9.00	9.8096	8.9265
8	9.75	10.208	9.2893
7	10.5	10.607	9.6522
6	11.25	11.006	10.015
5	12.00	11.404	10.378
4	12.75	11.803	10.740
3	13.5	12.202	11.103
2	14.25	12.601	11.466
1	14.75	12.867	11.708
	ΣL_{ei}=	202.5074	184.275

b) Tensile force in reinforcement due to self-wt.:

$T_{max}= \sigma_h\times s_v$

$\sigma_h=K\times\sigma_v$

$\sigma_v=\gamma_r Z + \sigma_2$

$\sigma_2=\frac{Ltan(\beta)}{2}\gamma b$

For *15m* depth,

$\sigma_2 = \frac{13tan\ (15)}{2} \times 16$
$=27.866 kN/m^2$

$\sigma_v = 20 \times 14.75 + 27.866$
$=322.866 kN/m^2$

$K=0.282 \times 1.2$ (**Fig.6**)
$=0.3384$
$\sigma_h = 0.338 \times 322.866 = 109.128 kN/m^2$
$T_{max} = 109.128 \times 0.5$
$=54.564$ kN/m
The maximum tensile force Ttotal =Tmax+Tmd
=0.75(54.564+11.501)kN/m
=49.548kN/m

Ta=74.95kN/m

$\frac{Ta}{\text{Ttotal}} = \frac{74.95}{49.54} = 1.512 > 1$ ok

T_{md}	K	σ_v	S_v	T_{max}	Total-tensile force	FOS-$_{against\ rupture}$>1
7.601	0.705	32.76	0.5	11.55	14.364	5.217
7.601	0.673	42.76	0.62	18.01	19.212	3.901
7.601	0.619	57.76	0.75	26.85	25.842	2.9002
7.601	0.577	72.76	0.75	31.53	29.352	2.553
7.601	0.524	87.76	0.75	34.54	31.610	2.371
7.601	0.478	102.7	0.75	36.90	33.381	2.245
7.601	0.436	117.7	0.75	38.58	34.638	2.163
7.601	0.393	132.7	0.75	39.22	35.122	2.133
7.601	0.338	147.7	0.75	37.50	33.829	2.215
7.703	0.338	162.7	0.75	41.31	36.760	2.038
8.059	0.338	177.7	0.75	45.11	39.883	1.879
8.416	0.338	192.7	0.75	48.92	43.005	1.742
8.773	0.338	207.7	0.75	52.73	46.128	1.624
9.129	0.338	222.7	0.75	56.53	49.251	1.521
9.486	0.338	237.7	0.75	60.34	52.373	1.431
9.843	0.338	252.7	0.75	64.15	55.496	1.351
10.199	0.338	267.7	0.75	67.95	58.619	1.278
10.556	0.338	282.7	0.75	71.76	61.742	1.213
10.913	0.338	297.7	0.75	75.57	64.864	1.155
11.269	0.338	312.7	0.62	66.15	58.065	1.291
11.507	0.338	322.7	0.5	54.61	49.589	1.511

4.4.2 Internal Stability with Respect to Pull-out Failure of Reinforcement

$Pr = F^*.\alpha\ .\sigma v Le.\ C$
$\alpha = 1$
$C = 2$
$\sigma_v = 322.866 kN/m^2$ (at a depth of 15m)
$L_e = 12.867$ m
$F^* = Fq\ .\ \alpha\beta$
40 $\alpha\beta$ = 40 (t/2St) = 20 (t/St) at the top of the structure
20 $\alpha\beta$ = 20 (t/2St) = 10 (t/St) at a depth of 6 m (20 ft) and below
S_t = 230mm; t=10.084 mm

layer	Z	F*	α	σ_v	L_{ei}	c	Pr	total force
21	0.25	0.701	1	32.768	8.5	2	347.353	14.364
20	0.75	0.701	1	42.768	8.5	2	453.356	19.212
19	1.5	0.701	1	57.768	8.5	2	612.362	25.842
18	2.25	0.701	1	72.768	8.5	2	771.368	29.352
17	3	0.701	1	87.768	8.5	2	930.374	31.610
16	3.75	0.701	1	102.768	8.5	2	1089.37	33.381
15	4.5	0.701	1	117.768	8.5	2	1248.38	34.638
14	5.25	0.701	1	132.768	8.5	2	1407.39	35.122
13	6	0.701	1	147.768	8.5	2	1566.39	33.829
12	6.75	0.350	1	162.768	8.613	2	874.204	36.760
11	7.5	0.350	1	177.768	9.012	2	998.971	39.883
10	8.25	0.350	1	192.768	9.410	2	1131.19	43.005
9	9	0.350	1	207.768	9.809	2	1270.88	46.128
8	9.75	0.350	1	222.768	10.208	2	1418.03	49.251
7	10.5	0.350	1	237.768	10.607	2	1572.64	52.373
6	11.25	0.350	1	252.768	11.006	2	1734.70	55.496
5	12	0.350	1	267.768	11.404	2	1904.23	58.619
4	12.75	0.350	1	282.768	11.803	2	2081.22	61.742
3	13.5	0.350	1	297.768	12.202	2	2265.66	64.864
2	14.25	0.350	1	312.768	12.601	2	2457.57	58.065
1	14.75	0.350	1	322.768	12.867	2	2589.65	49.589

For,

F*=0.876 at top of the structure

F*=0.438 at 6m and below

F* should be reduced to 80 percent of the static value:

F*=0.438×0.8=0.3504

Pulloutresistance for 15m depth=0.3504×1×322.866×12.867×2

=2911.345 kN/m

Ttotal =49.548kN/m

Ttotal $\leq \frac{2911.345}{0.75*1.5}$

Safety factor against pullout = 1.5< 2589.65kN/m *Ok*

4.5 SLOPE STABILITY ANALYSIS USING GEOSTUDIO-8.15.5.11777 (SLOPE/W)

Soil properties of the slope (***Refer to Clause 4.1***)

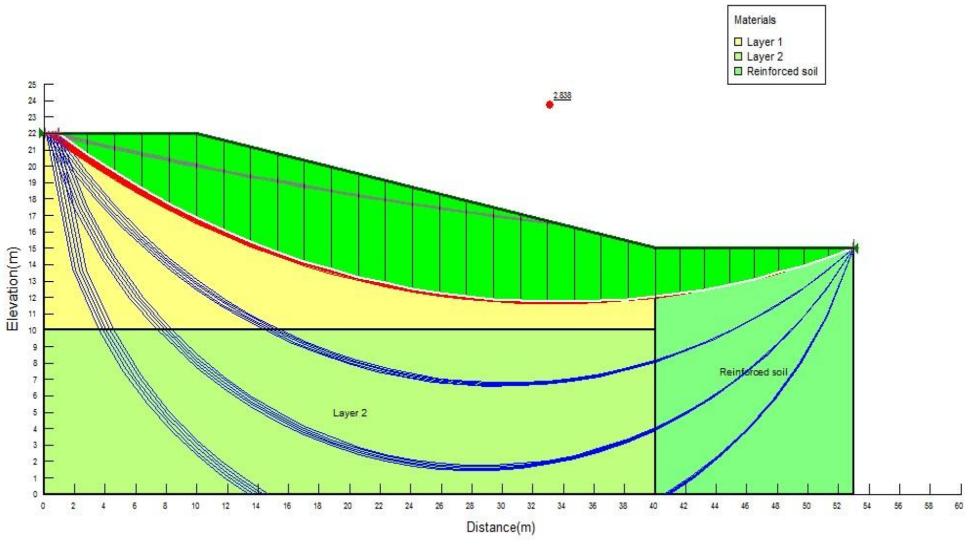


Figure 4.4 Slip circles of the slope

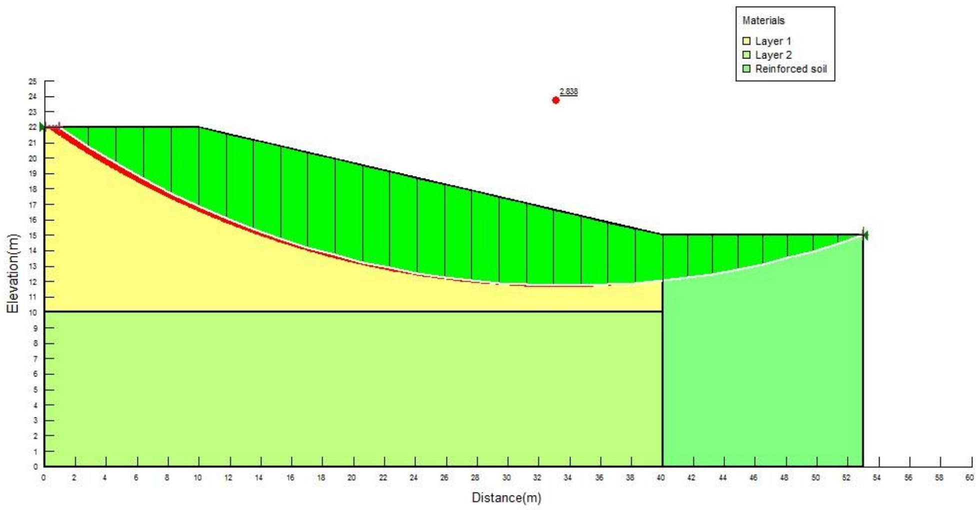


Figure 4.4 Critical Slip circles and Safe Zone

After calculations the factor of safety is found to be 2.843.

4.5.1 Detail Analysis

File Information

File Version: 8.15
Revision Number: 1
Date: 5/12/2020
Time: 7:50:09 PM
Tool Version: 8.15.5.11777
File Name: 4th trial design including Reinforced soil.gsz
Directory: F:\PROJECT\GEO STUDIO\
Last Solved Date: 5/11/2016
Last Solved Time: 7:50:16 PM

Project Settings

Length(L) Units: Meters
Time(t) Units: Seconds
Force(F) Units: Kilonewtons
Pressure(p) Units: kPa
Strength Units: kPa
Unit Weight of Water: 9.807 kN/m³

View: 2D
Element Thickness: 1
Analysis Settings
Slope Stability
Kind: SLOPE/W
Method: Bishop
Settings
PWP Conditions Source: (none)
Slip Surface
Direction of movement: Left to Right
Use Passive Mode: No
Slip Surface Option: Entry and Exit
Critical slip surfaces saved: 1
Resisting Side Maximum Convex Angle: 1 °
Driving Side Maximum Convex Angle: 5 °
Optimize Critical Slip Surface Location: No
Tension Crack
Tension Crack Option: (none)
F of S Distribution
F of S Calculation Option: Constant
Advanced
Number of Slices: 30
F of S Tolerance: 0.001
Minimum Slip Surface Depth: 0.1 m

Materials
Layer 1
Model: Mohr-Coulomb
Unit Weight: 16 kN/m³
Cohesion': 3 kPa
Phi': 20 °
Phi-B: 0 °
Layer 2
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion': 5 kPa
Phi': 30 °
Phi-B: 0 °
Reinforced soil
Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 34 °
Phi-B: 0 °
Slip Surface Entry and Exit
Left Projection: Range
Left-Zone Left Coordinate: (0.12, 22) m
Left-Zone Right Coordinate: (1, 22) m
Left-Zone Increment: 4
Right Projection: Point
Right Coordinate: (53, 15) m
Right-Zone Increment: 4
Radius Increments: 4
Slip Surface Limits
Left Coordinate: (0, 22) m
Right Coordinate: (53, 15) m
Points

	X (m)	Y (m)
Point 1	0	22

Point 2	10	22
Point 3	40	15
Point 4	40	10
Point 5	0	10
Point 6	0	0
Point 7	40	0
Point 8	53	15
Point 9	53	0

Regions

	Material	Points	Area (m²)
Region 1	Layer 1	1,2,3,4,5	375
Region 2	Layer 2	5,6,7,4	400
Region 3	Reinforced soil	3,4,7,9,8	195

Current Slip Surface
Slip Surface: 12
F of S: 2.849
Volume: 214.43265 m³
Weight: 3,521.5875 kN
Resisting Moment: 91,651.551 kN-m
Activating Moment: 32,172.161 kN-m
F of S Rank (Analysis): 3 of 25 slip surfaces
F of S Rank (Query): 3 of 25 slip surfaces
Exit: (53, 15) m
Entry: (0.56, 22) m
Radius: 58.126236 m
Center: (33.628258, 69.803238) m
Slip Slices

	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
Slice 1	1.504	21.373825	0	8.5921243	3.1272775	3
Slice 2	3.392	20.172541	0	26.532604	9.657078	3
Slice 3	5.28	19.069954	0	43.207741	15.726332	3
Slice 4	7.168	18.059738	0	58.672386	21.355002	3
Slice 5	9.056	17.136567	0	72.973739	26.560269	3
Slice 6	10.882353	16.32087	0	82.634033	30.076328	3
Slice 7	12.647059	15.604038	0	87.707532	31.922931	3
Slice 8	14.411765	14.953338	0	91.798097	33.411775	3
Slice 9	16.176471	14.366439	0	94.928606	34.551187	3
Slice 10	17.941176	13.841331	0	97.118273	35.348161	3
Slice 11	19.705882	13.376287	0	98.382958	35.808468	3

Slice 12	21.470588	12.969831	0	98.735417	35.936753	3
Slice 13	23.235294	12.620712	0	98.185506	35.736602	3
Slice 14	25	12.327884	0	96.740319	35.210596	3
Slice 15	26.764706	12.090489	0	94.4043	34.360355	3
Slice 16	28.529412	11.907846	0	91.179307	33.186554	3
Slice 17	30.294118	11.779435	0	87.06464	31.688937	3
Slice 18	32.058824	11.704899	0	82.057031	29.866317	3
Slice 19	33.823529	11.684028	0	76.150604	27.716553	3
Slice 20	35.588235	11.716765	0	69.336789	25.236527	3
Slice 21	37.352941	11.803201	0	61.604201	22.422096	3
Slice 22	39.117647	11.943577	0	52.938472	19.268028	3
Slice 23	40.928571	12.144859	0	58.86754	39.706657	0
Slice 24	42.785714	12.41059	0	53.821483	36.303049	0
Slice 25	44.642857	12.737981	0	47.406883	31.976347	0
Slice 26	46.5	13.128104	0	39.56549	26.68726	0
Slice 27	48.357143	13.582265	0	30.229823	20.390273	0
Slice 28	50.214286	14.102033	0	19.32155	13.03255	0
Slice 29	52.071429	14.689266	0	6.7494532	4.5525637	0

V. CONCLUSION

5.1 RESULT

The Reinforced soil retaining structure which had been designed is safe against static sliding, overturning and bearing pressure. The structure is also safe when a seismic load is being considered.

The backfill slope inclined at 15^o is also safeagainst sliding or overturning as the factor of safety calculated is greater than the minimum Factor of Safety required. Our Factor of Safety being 2.483 which is greater than 1.5.

5.2 LIMITATION

The slope stability analysis against seismic force is not completed due to certain reasons as stated below:

- Due to lack of Availability of Software and Analysing tool;
- Due to lack of time;

However, the structure is presume to be safe as it Ductile Earth-wall. Moreover precise analysis is not that required in this type of structured.

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