Semi-probabilistic approach to the stability of veneer reinforcement

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ABSTRACT: Veneer is intended as a relatively thin cover soil layer placed on a slope. The stability analysis for sliding of a veneer above a geosynthetic or a natural soil layer is the object of the present paper. The analytical approach used in the present paper for the evaluation of veneer stability has been derived by the method of Koerner and Soong (1998), revised and adapted to the semi-probabilistic approach in Ultimate Limit State (ULS) conditions, according to EuroCodes. The present method for veneer stability analyses includes evaluations of toe wedge forces, seismic forces, equipment forces, tensile forces of geosynthetic reinforcements, and water seepage effects on veneer stability.

Keywords: Analysis, Design, Limit Equilibrium Methods, Steep Slopes, Veneer Stability

1 INTRODUCTION

Veneer is intended as a relatively thin cover soil layer placed on a slope.
The stability analysis for sliding of a veneer above a geosynthetic or a natural soil layer, including sliding above liners, like geomembranes (GBR), geosynthetic clay liners (GCL) and compacted clay liners (CCL), is the object of the present paper.
There are two specific applications in which cover soil stability needs to be checked:
l) Leachate collection soil placed above a GBR, GCL and/or CCL along the sides of a landfill or a heap leach pad before waste or ore is placed and stability achieved accordingly.
2) Final cover soil placed above a GBR, GCL and/or CCL in the cap or closure of a landfill or a heap leach pad.

There have been numerous cover soil stability problems in the past, resulting in slides that range from being relatively small (which can be easily repaired), to very large (involving litigation and financial judgments against the parties involved). Soong and Koerner (1996) report on eight cover soil failures resulting from seepage induced stresses alone.

Veneer stability above a liner represents a major challenge due to the following reasons (Koerner and Soong, 1998):
(a) The underlying barrier materials generally represent a low interface shear strength boundary with respect to the soil placed above them.
(b) The liner system is oriented precisely in the direction of potential sliding.
(c) The potential shear planes are usually linear and are essentially uninterrupted along the slope.
(d) Liquid (water or leachate) cannot continue to percolate downward through the cross section due to the presence of the barrier material.

For the leachate collection soil situation, the time frame is generally short (from months to a few years) and the implications of a slide may be minor in that repairs can usually be done by on-site personnel. For the final cover soil situation, the time frame is invariably decades long and the implications of a slide can be serious in that repairs often call for a forensic analysis, engineering redesign, separately engaged contractors and quite high remediation costs. These latter cases sometime involve litigation, insurance carriers, and invariably technical experts (Koerner and Soong, 1998).

Since both situations (leachate collection and final covers) present the same technical issues, the method presented in the present paper is applicable to both. Anyway, the paper is particularly addressed to the final cover situation, which presents much higher challenges.

2 ANALYTICAL APPROACH TO VENEER STABILITY

The analytical approach used in the present paper for the evaluation of veneer stability has been derived by the method of Koerner and Soong (1998), revised and adapted to the semi-probabilistic method in Ultimate Limit State (ULS) conditions, according to EuroCodes.

The present method for veneer stability analyses includes evaluations of toe wedge forces, seismic forces, equipment forces, tensile forces of geosynthetic reinforcements, and water seepage effects on veneer stability.

The model assumes that the veneer soil cover can be divided in the active wedge of trapezoidal shape, limited by a tension crack at the crest, whose sliding down the slope is contrasted by the passive triangular wedge at toe and by the geosynthetic reinforcement anchored at top of the slope, as shown in the scheme of Fig. 1.

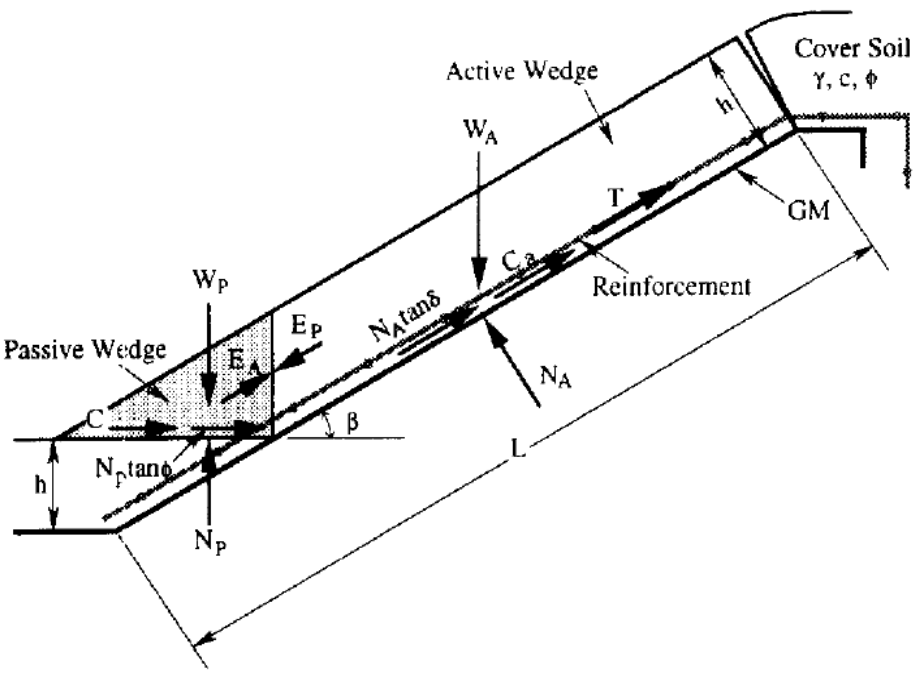

Figure 1. Scheme of veneer stability analysis with only gravitational static loads applied (from Koerner and Soong, 1998)

Final cover systems for a landfill or a heap leach pad can be characterized as a veneer soil layer of thickness h on a surface sloped at angle β to the horizontal, as shown in Figure 1.

The critical slip surface is considered to occur on the liner below the veneer cover. The tangential weight of the soil mass, $W_A \cdot \sin \beta$, is the driving force that might cause the soil veneer to slip.

The resisting forces against slippage are:

(a) the shear strength between the soil mass and the slip surface

(b) the toe buttressing force at the toe of the slope

(c) the tensile force, T, from the geosynthetics.

The goal of veneer stability design is to provide all of the slope resistance with shear forces and not to put the geosynthetic barriers in tension. Only the tensile strength contribution provided by a high tensile resistant geosynthetic, such as a geogrid or woven / knitted geotextile, is considered for the support of the cover system, while any tensile strength contribution from other geosynthetics (geomembranes, nonwoven geotextile filters, etc.) shall be ignored.

The shear strength along the critical interface is often a function of the effective normal force acting on this interface. As well known, the relationship between shear strength and normal force is called the shear strength envelope, and it is usually expressed in terms of the Mohr-Coulomb parameters of friction angle φ and cohesion c. These terms are actually representing the slope of the shear strength envelope and the

"y-intercept". Since the actual shear strength envelope is often not linear, as illustrated in Figure 2, it is important that the Mohr-Coulomb parameters are defined within a specific normal load range, as it is always unconservative to extrapolate the envelope, either to lower normal loads or to higher ones.

The shear strength on the interface can be reduced by pore pressures, u, because pore pressures reduce the effective normal force acting on the interface. Thus, flowing water or landfill gas pressures would be destabilizing forces.

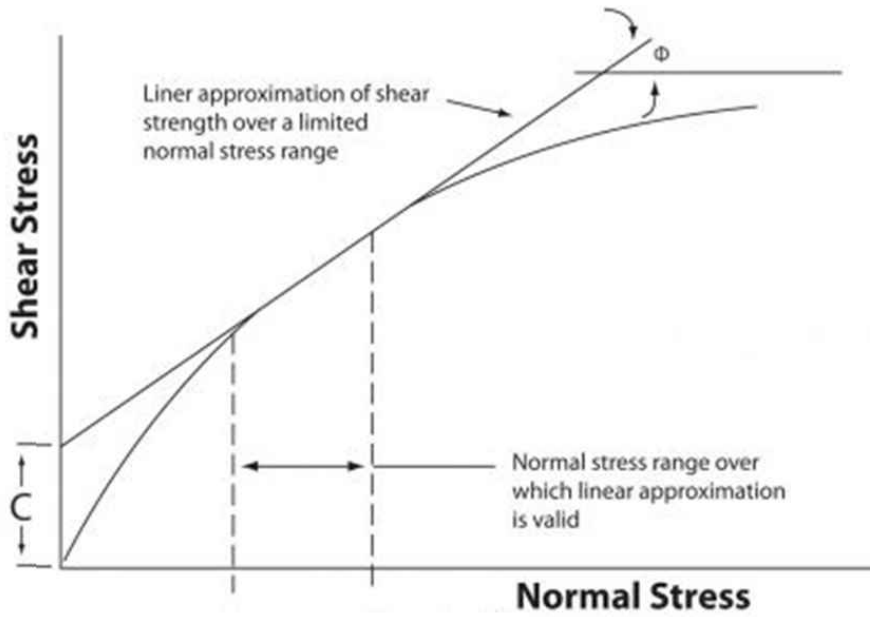

Figure 2. Mohr-Coulomb parameters of friction angle ϕ and cohesion c

In the semi-probabilistic approach in Ultimate Limit State (ULS) conditions, according to EuroCodes, loads are amplified by Amplification Factors, while resistances are reduced by Reduction Factors.

The analysis of forces in the free body diagrams of the active and passive wedges (Fig. 1) allows to calculate the active and passive interwedge forces, E_a and E_p. The stability check is satisfied if:

$$FS = (E_p / E_a) \geq \gamma_R \tag{1}$$

where FS is the Factor of Safety and γ_R is the partial factor R1 or R2 required by the EuroCode norms for sliding analysis (see later).

In practice the EC7 approach can be easily implemented by performing the "traditional" analyses, like sliding along the slope, pullout, etc., but using as input values the factorized values of loads and geotechnical parameters. Analyses are satisfied when the minimum Factor of Safety, now called R1 or R2 or γ_R, is achieved.

2.1. *Interface shear testing*

The interface shear strength of a cover soil with respect to the underlying material (geomembrane or natural soil) is critical so as to properly analyze the stability of the cover soil. This value of interface shear strength is obtained by laboratory testing of the project specific materials, that is by sampling of the candidate geosynthetics to be used at the site, as well as of the cover soil at its targeted density and moisture conditions, at the site specific conditions, that is under project specific normal stresses, strain rates, peak or residual shear strengths and temperature extremes (high and/or low). Hence values of interface shear strengths from the literature should be used only for preliminary design.

The test that has been developed to evaluate geosynthetics shear strength parameters are ASTM D5321 in the USA, while internationally reference has to be made to EN ISO 12957-1 - Geosynthetics – Determination of the friction characteristics – Part 1: Direct shear test, and to EN ISO 12957-2 - Geosynthetics – Determination of friction characteristics – Part 2: Inclined plane test. Usually in the direct shear test relatively high compressive loads are applied, in the range 50 – 200 kPa. But a veneer soil layer on a capping applies a very small load: a 1 m thick veneer applies a uniform pressure of

maximum 20 kPa. Therefore, given the limitations showed in Fig. 2, the inclined plane test appears to be the most suitable for obtaining the friction characteristics at the interface between the veneer soil and the layer below it (Pavanello et Al, 2016), at least for capping design.

With either direct shear or inclined plane tests, two fundamental shear strength parameters are obtained:

δ = the angle of shearing resistance, peak and/or residual, of the two opposing surfaces (usually called the interface friction angle)

C_a = the adhesion of the two opposing surfaces, peak and/or residual.

The upper limit of δ, when soil is involved as one of the interfaces, is the friction angle of the soil component, φ. The upper limit of the C_a value is the cohesion of the soil component, c. In the following slope stability analyses, the C_a term will be included for the sake of completeness, but to utilize an adhesion value there must be a clear physical justification for the use of such values when geosynthetics are involved, like textured geomembranes with physical interlocking of soils having cohesion.

2.2. *Types of loadings*

There are a large variety of slope stability problems that may be encountered in analyzing and/or designing final covers of landfills or heap leach pads as well as leachate collection soils covering geomembranes beneath the waste or ore. The most common situation is a uniformly thick cover soil on a geomembrane placed over the soil subgrade at a given and constant slope angle. Analyses will include loads produced by equipment moving up the slope and then moving down the slope during placement of cover soil on the geomembrane.

Since cover soil slides have occurred due to seepage forces, often drainage above a geomembrane (or other barrier material) in the cover soil cross section must be accommodated to avoid the possibility of seepage forces. This situation requires specific analyses of slope stability problems.

If an earthquake occurs in the vicinity of a landfill or heap leach pad, the seismic wave travels through the solid waste or ore mass reaching the upper surface of the cover. It then decouples from the cover soil materials, producing a horizontal force which must be appropriately analyzed.

All of the above actions are destabilizing forces tending to cause slope instability. To increase the stability of slopes, other than geometrically redesigning the slope with a flatter slope angle or shorter slope length, a designer can add soil mass at the toe of the slope thereby enhancing stability. Both toe berms and tapered soil covers are available options, but will not be analyzed in this paper.

Alternatively, the designer can always use geogrids or high strength geotextiles within the cover soil acting as reinforcement materials. This technique, usually referred to as veneer reinforcement, will be specifically analyzed.

3 DESIGN APPROACH ACCORDING TO EUROCODES

3.1. *EuroCodes framework*

In EN 1997-1. Eurocode 7 (usually abbreviated as EC7), design by calculation involves the definition of:
- actions, which may be either imposed loads or imposed displacements, e.g. from ground movements;
- properties of soils, rocks and other materials;
- geometrical data;
- limiting values of deformations, crack widths, vibrations etc.;
- calculation models.

Fig. 3 shows the rational design process of the EuroCodes framework.

The geotechnical analyses shall include two limit states:

- Geotechnical (GEO) limit state: failure or excessive deformation of the ground, where the soil or rock is significant in providing resistance. This limit state is satisfied if the design effect of the actions (E_d) is less than or equal to the design resistance (R_d):

$$E_d \leq R_d \tag{2}$$

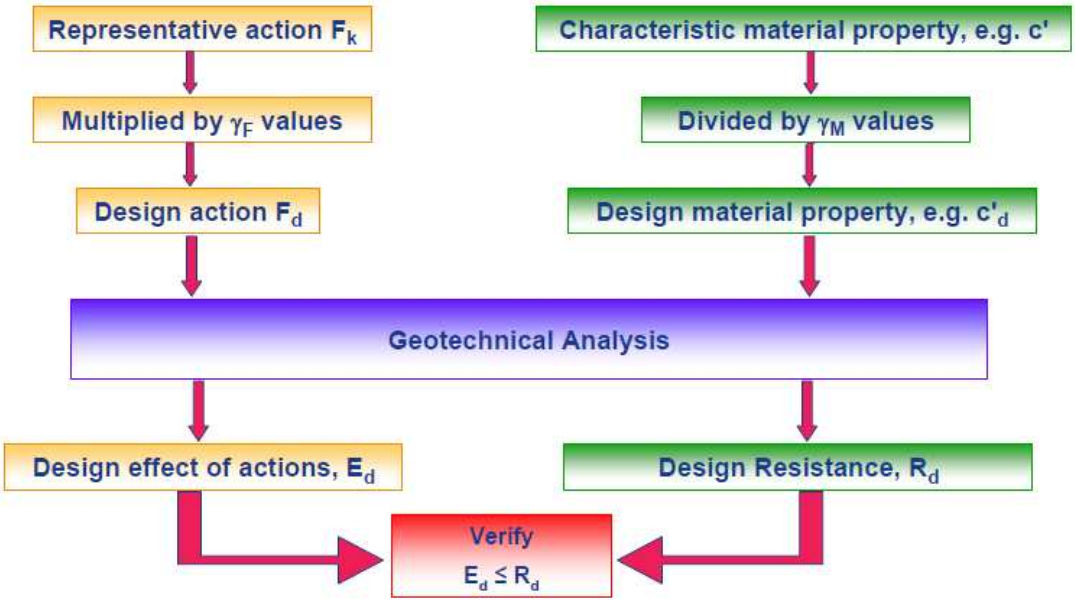

Figure 3. The rational design process of the EuroCodes framework.

- Structural (STR) limit state: failure or excessive deformation of the structure, where the strength of the structural material is significant in providing resistance. As with GEO limit state, the STR limit state is satisfied if the design effect of the actions (E_d) is less than or equal to the design resistance (R_d), according to Eq. (2).

For considering both GEO and STR Limit states, in EC7 three Design Approaches are offered, to reflect national choice. The design approach followed reflects whether the safety is applied to the material properties, the actions or the resistances:

- Design Approach 1: Combination 1: A1 + M1 + R1
  Combination 2: A2 + M2 + R1
- Design Approach 2: A1 + M1 + R2
- Design Approach 3: A + M2 + R3

The condition of Eq. (2) can be written in another form: according to EC7, the ratio between the design resistance (R_d) and the design effect of the actions (E_d) shall be larger than the coefficient γ_R (that is, the Factor R1, R2, or R3 of the specific approach that has been chosen):

$$R_d \ / \ E_d \geq \gamma_R \tag{3}$$

The coefficient γ_R can be considered equivalent to the Factor of Safety FS of traditional methods.
Factors A1, M1, R1, etc., are set in each National Annex. In the present paper, reference has been made to the Italian National Annex, that is the Italian Norms (NTC 2008), which implement both EuroCode 7 (EN 1997-1) and EuroCode 8 (EN 1998-1). Anyway all National Annexes are applicable, it is enough to use the specific national factors in place of the factors of the Italian Norm.

3.2. *Actions*

In EC7:
• An action is given the general symbol, F.
• Actions can be permanent (persistent), permanent non structural, or variable (transient), accidental, or seismic.
• Persistent actions are denoted by F_G. Transient actions are denoted by F_Q.
• Actions can be either "favourable" or "unfavourable".
• Load Coefficients (A) shall be applied to actions in each Ultimate Limit State (ULS) analysis.
• Partial coefficients (A) are denoted by γ_{G1}, γ_{G2}, γ_{Qi}, for permanent, permanent non structural, and variable loads, respectively.
The combinations of factors to be applied to the actions (A) in the ultimate limit state analyses, according to Italian Norm (NTC 2008), are reported in Table 1.

3.3. *Ground properties*

In EC7:

• Geotechnical parameters should be established with consideration given to published data and local and general experience

• Material properties are given the general symbol, X.

• Characteristic values of material properties are given the general symbol, X_k.

• The selection of characteristic values for geotechnical parameters shall be based on results and derived values from laboratory and field tests.

• Geotechnical coefficients (M) shall be applied to resistances in each Ultimate Limit State (ULS) analysis.

• Partial coefficients (M) are denoted by $\gamma_{\phi'}$, $\gamma_{c'}$, γ_{cu}, γ_{γ}, for tangent of friction angle, drained cohesion, undrained cohesion, and unit weight, respectively.

The combinations of factors to be applied to the soil parameters (M) in the ultimate limit state analyses, according to Italian Norm (NTC 2008), are reported in Table 2.

Table 1. Combinations of factors to be applied to the actions (A) in the ultimate limit state analyses, according to Italian Norm (NTC 2008)

TYPE OF LOAD	EFFECT	PARTIAL COEFFICIENT γ_F or γ_E	EQU	A1 STR	A2 GEO
PERMANENT	FAVOURABLE	γ_{G1}	0,90	1,00	1,00
	UNFAVOURABLE		1,10	1,30	1,00
PERMANENT NON STRUCTURAL	FAVOURABLE	γ_{G2}	0,00	0,00	0,00
	UNFAVOURABLE		1,50	1,50	1,30
VARIABLE	FAVOURABLE	γ_{Qi}	0,00	0,00	0,00
	UNFAVOURABLE		1,50	1,50	1,30

Table 2. Combinations of factors to be applied to the soil parameters (M) in the ultimate limit state analyses, according to Italian Norm (NTC 2008)

PARAMETER	APPLIED TO	PARTIAL COEFFICIENT γ_M	M1	M2
TANGENT OF FRICTION ANGLE	$\tan \varphi'_k$	$\gamma_{\varphi'}$	1,00	1,25
DRAINED COEHESION	c'_k	$\gamma_{c'}$	1,00	1,25
UNDRAINED COHESION	c_{uk}	γ_{cu}	1,00	1,40
UNIT WEIGHT	γ	γ_{γ}	1,00	1,00

3.4. *Stability Verification*

As said, according to EC7, the ratio between the design resistance (R_d) and the design effect of the actions (E_d) shall be larger than the coefficient γ_R (that is, the Factor R1, R2, or R3 of the specific approach that has been chosen), according to Eq. (3).

The combinations of coefficients to be used for ultimate limit states verification purposes (R), according to Italian Norm (NTC 2008), are reported in Table 3.

Moreover, according to Italian Norm, global stability analyses of works made up of loose material (including veneer covers) shall be carried out with Approach 1, Combination 2 (GEO), and the factor (R2) reported in Table 4 shall be applied.

Table 3. Combinations of coefficients to be used for ultimate limit states verification purposes (R), according to Italian Norm (NTC 2008)

ANALYSIS	PARTIAL COEFFICIENT R1	PARTIAL COEFFICIENT R2
BEARING CAPACITY OF FOUNDATION	$\gamma_R = 1.0$	$\gamma_R = 1.0$
SLIDING	$\gamma_R = 1.0$	$\gamma_R = 1.0$
RESISTANCE OF DOWNSLOPE SOIL	$\gamma_R = 1.0$	$\gamma_R = 1.0$

Table 4. Factor R2 for global stability analyses of works made up of loose material, according to Italian Norm (NTC 2008)

COEFFICIENT	R2
γ_R	1.10

Therefore, according to EC7, for veneer cover stability the following analyses shall be carried out:

- DIRECT SLIDING:
  Verification shall be carried out according to Approach 1:
  - Combination 1: (A1 + M1 + R1), $FS = R_d \ / \ E_d \geq R1 = 1.0$
  - Combination 2: (A2 + M2 + R2), $FS = R_d \ / \ E_d \geq R2 = 1.0$
- TENSILE STRENGTH OF REINFORCEMENT:
  Verification shall be carried out according to Approach 1:
  - Combination 1: (A1 + M1 + R1), $FS = R_d \ / \ E_d \geq R1 = 1.0$
  - Combination 2: (A2 + M2 + R2), $FS = R_d \ / \ E_d \geq R2 = 1.0$
- PULLOUT OF REINFORCEMENT AND/OR OTHER GEOSYNTHETICS (for anchorage in top trenches):
  Verification shall be carried out according to Approach 1:
  - Combination 1: (A1 + M1 + R1), $FS = R_d \ / \ E_d \geq R1 = 1.0$
  - Combination 2: (A2 + M2 + R2), $FS = R_d \ / \ E_d \geq R2 = 1.0$
- GLOBAL STABILITY (when required):
  Verification shall be carried out according to Approach 1:
  - Combination 2: (A2 + M2 + R2), $FS = R_d \ / \ E_d \geq R2 = 1.10$

4 SLIDING STABILITY ANALYSES FOR VENEER COVERS

4.1. *Basic case of only gravitational forces in static conditions*

Figure 4 illustrates the common situation of a finite length, uniformly thick cover soil placed over a liner material at a slope angle β. It includes a passive wedge at the toe and has a tension crack at the crest. The analysis that follows is derived from Koerner and Soong (1998) and Koerner and Hwu (1991), but comparable analyses are available from Giroud and Beech (1989), McKelvey and Deutsch (1991), Ling and Leshchinsky (1997), and others.

Note that all analyses refer to 1.0 m running length of slope, hence all forces are expressed in kN/m.

The symbols used in Figure 4 are defined below:

W_A = total weight of the active wedge (kN/m)
W_p = total weight of the passive wedge (kN/m)
N_A = effective force normal to the failure plane of the active wedge (kN/m)
N_p = effective force normal to the failure plane of the passive wedge (kN/m)
γ = unit weight of the cover soil (kN/m^3)
h = thickness of the cover soil (m)

H = vertical height of the slope measured from the toe of the active wedge (m)
L = length of slope measured along the soil slope (m)
β = angle of the slope (deg)
φ = friction angle of the cover soil (deg)
δ = friction angle between cover soil and liner (deg)
C_a = adhesive force between cover soil of the active wedge and the liner (kN/m)
c_a = adhesion between cover soil of the active wedge and the liner (kPa)
C = cohesive force along the failure plane of the passive wedge (kN/m)
c = cohesion of the cover soil (kPa)
E_A = interwedge force acting on the active wedge from the passive wedge (kN/m)
E_P = interwedge force acting on the passive wedge from the active wedge (kN/m)
FS = factor of safety against cover soil sliding along the slope

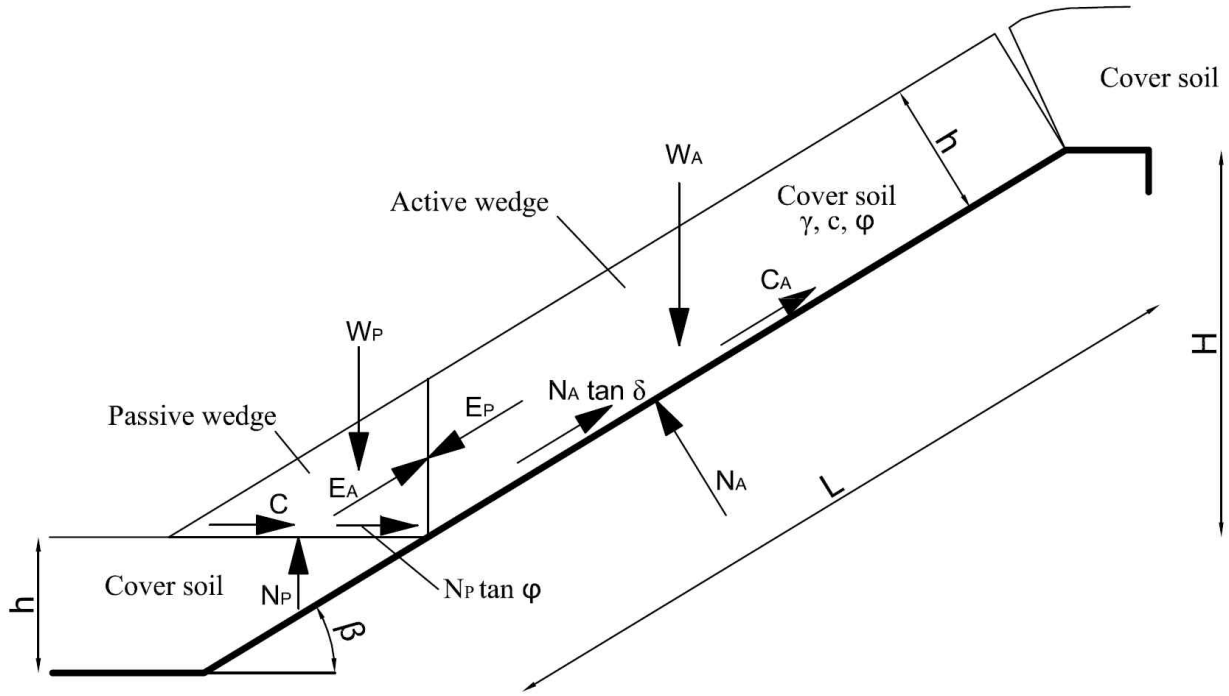

Figure 4. Limit equilibrium forces involved in a finite length slope analysis for a uniformly thick cover soil

Note that, to avoid long writing, the above geotechnical parameters represent the characteristic values of soil and interface properties already reduced with the partial coefficients M1 or M2 (see Tab. 2), that is:

$$\tan \varphi = \tan \varphi_{soil} / \gamma_{\phi'} \quad (4)$$

$$c = c_{soil} / \gamma_{c'} \quad (5)$$

$$\tan \delta = \tan \delta_{interface} / \gamma_{\phi'} \quad (6)$$

$$c_a = c_{interface} / \gamma_{c'} \quad (7)$$

$$\gamma = \gamma_{soil} / \gamma_{\gamma} \quad (8)$$

The expression for determining the factor of safety can be derived as follows.
From Fig. 4, considering the active wedge:

$$W_A = \frac{1}{2}\gamma \cdot h \cdot \left(\frac{2H}{\sin\beta} - h\tan\beta \right) \tag{9}$$

$$N_A = W_A \cdot \cos\beta \tag{10}$$

$$C_A = c_A \cdot L = c_A \cdot \frac{H}{\sin\beta} \tag{11}$$

From Fig. 4, considering the passive wedge:

$$W_P = \frac{1}{2}\gamma \cdot h^2 \cdot \left(\frac{1}{\sin\beta \cdot \cos\beta} \right) \tag{12}$$

$$N_P = W_P + E_P \cdot \sin\beta \tag{13}$$

$$C = c \cdot \frac{h}{\sin\beta} \tag{14}$$

From the equilibrium of vertical forces on the active wedge $(\Sigma F_v = 0)_{\text{active wedge}}$:

$$E_A = \frac{1}{\sin\beta} \cdot \left[\gamma_{G1u} \cdot W_A \cdot (1 - \cos^2\beta - \cos\beta \cdot \sin\beta \cdot \tan\delta) - \gamma_{G1f} \cdot C_A \cdot \sin\beta \right] \tag{15}$$

where:
γ_{G1u} = Amplification Factor for permanent unfavourable loads (see Tab. 1)
γ_{G1f} = Amplification Factor for permanent favourable loads (see Tab. 1).
From the equilibrium of horizontal forces on the passive wedge $(\Sigma F_h = 0)_{\text{passive wedge}}$:

$$E_P = \frac{\gamma_{G1f} \cdot C + \gamma_{G1f} \cdot W_P \cdot \tan\varphi}{\cos\beta - \sin\beta \cdot \tan\varphi} \tag{16}$$

Stability is then verified if, for both combinations (A1 + M1 + R1) and (A2 + M2 + R2), the Factor of Safety results:

$$FS = \frac{E_P}{E_A} \geq 1.0 \tag{17}$$

If the above condition is not verified, it means that the veneer cover is unstable for sliding down the slope: in this case a geosynthetic reinforcement shall be introduced and the stability analyses including reinforcement shall be performed until the condition is verified.

4.2. *Veneer reinforcement*

The best way of increasing the Factor of Safety for a slope is to reinforce it with a geosynthetic. Such reinforcement can be either intentional or non-intentional (Koerner and Soong, 1998).

By intentional reinforcement it is meant to include a geogrid or high strength geotextile within the cover soil to purposely reinforce the system against instability. Depending on the type and amount of reinforcement, the majority, or even all, of the driving, or mobilizing, stresses can be supported, resulting in major increase in FS value.

By non-intentional reinforcement reference is made to multicomponent liner systems where a low shear strength interface is located beneath a overlying geosynthetics. In this case, the overlying geosynthetics are inadvertently acting as veneer reinforcement to the composite system. In some cases, the designer may not realize that such geosynthetics are being stressed in an identical manner as a geogrid or high strength geotextile, but they are. The situation where a relatively low strength protection geotextile is placed over a smooth geomembrane and beneath the cover soil is a case of non-intentional

reinforcement. Intentional, or non-intentional, the stability analysis is identical. The difference is that the geogrids and/or high strength geotextiles give a major increase in the FS value, while a protection geotextile (or other low strength geosynthetics) only nominally increases the FS value at the cost of high strains in the geosynthetics. Therefore it is evident that non-intentional reinforcement shall be avoided.

Hence we will refer only to intentional reinforcement, that is a geogrid or high strength geotextile placed below the veneer cover and above the multicomponent liner system; while the tensile strength of any component of the lining system (filter geotextiles, draining geocomposites, geomembranes, GCL, etc.) will be considered equal to zero.

The scheme of the stability analysis of the veneer cover subject only to gravitational forces in static conditions, with a reinforcing geosynthetic below the veneer cover, is shown in Fig. 5.

All forces have the same expressions as in the previous analysis, but a force T_D from the reinforcement, acting parallel to the slope, provides additional stability. This force acts only within the active wedge.

All symbols used in Figure 5 were previously defined, except the following:

T_D = design strength of the geosynthetic reinforcement (kN/m).

The design tensile force T_D is obviously less than the ultimate strength of the geosynthetic reinforcement. Considering the ultimate strength T_{ult}, the value should be reduced by such factors as installation damage, creep and long-term degradation. Note that if seams are involved in the reinforcement, a reduction factor should be added accordingly. Koerner (2012), among others, provides recommended numeric values.

Hence the expression of T_D is:

$$T_D = \frac{T_{ult}}{RF_{ID} \cdot RF_{cr} \cdot RF_{ch} \cdot RF_b} \tag{18}$$

where:

T_D = design value of reinforcement strength (kN/m)

T_{ult} = ultimate value of reinforcement strength (kN/m)

RF_{ID} = reduction factor for installation damage

RF_{cr} = reduction factor for creep

RF_{ch} = reduction factor for long term chemical degradation

RF_b = reduction factor for long term biological degradation

Note that for analyses at short term (e.g. at the end of construction) the Reduction Factors for creep and for chemical and biological damage shall be set as: $RF_{cr} = RF_{ch} = RF_b = 1.0$; while, for seismic analyses, only the Reduction Factor for creep shall be set as: $RF_{cr} = 1.0$.

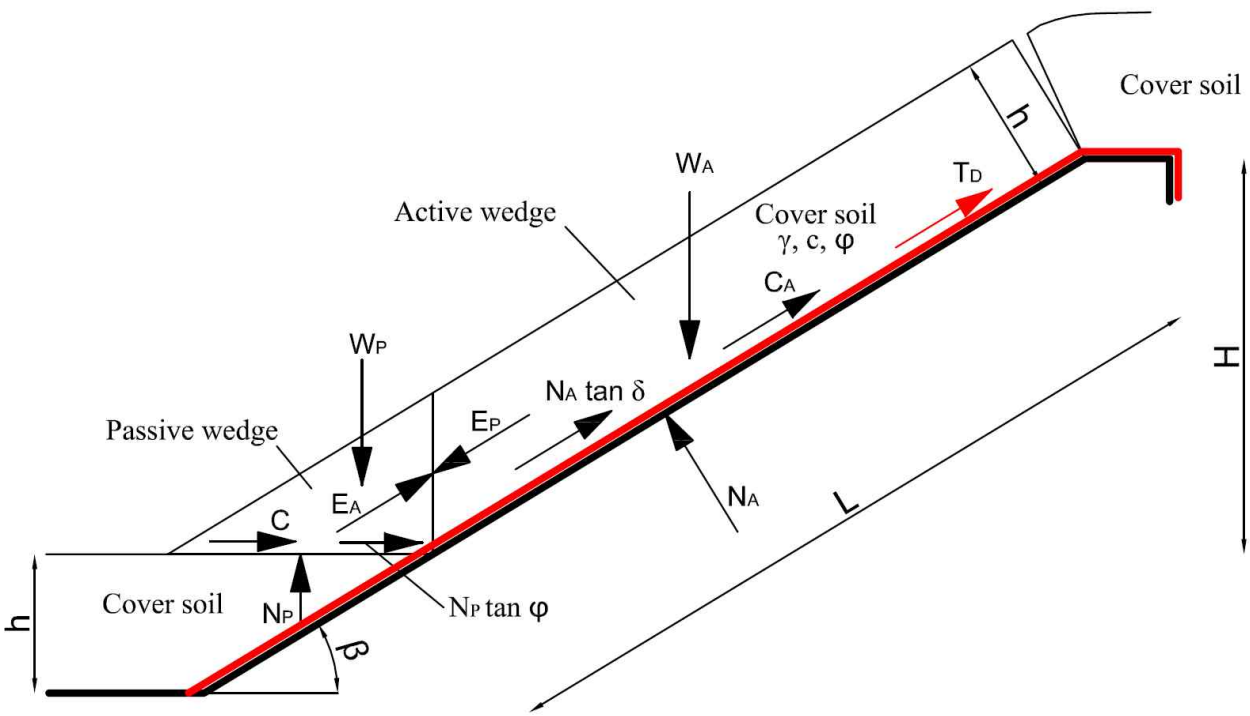

Figure 5. Limit equilibrium forces involved in a finite length slope analysis for a uniformly thick cover soil including the use of veneer reinforcement.

With reference to Fig. 5, the interwedge force E_A is obtained from the equilibrium of vertical forces on the active wedge $(\Sigma F_v = 0)_{\text{active wedge}}$:

$$E_A = \frac{1}{\sin\beta} \cdot \left[\gamma_{G1u} \cdot W_A \cdot (1 - \cos^2\beta - \cos\beta \cdot \sin\beta \cdot \tan\delta) - \gamma_{G1f} \cdot C_A \cdot \sin\beta - \gamma_{G1f} \cdot T_D \cdot \sin\beta\right] \quad (19)$$

The interwedge force E_P has the same expression as in Eq.(16).
Stability is then verified if, for both combinations (A1 + M1 + R1) and (A2 + M2 + R2), the Factor of Safety results as per Eq. (17).
For this analysis and for all the following ones, the tensile strength T_D (and therefore the ultimate tensile strength T_{ult}) shall be varied by trials and errors until the condition of Eq. (17) gets verified.
It is possible that two different values of T_D (and therefore of the ultimate tensile strength T_{ult}) may be required for the two analyses (A1 + M1 + R1) and (A2 + M2 + R2): in this case the highest of the two T_D values shall be retained.
For high values of T_D the interwedge force E_A may become negative, and as a consequence also FS becomes negative: this is just a sign that T_D has been overdesigned; then the designer may try with a lower value of T_D, or the designer may keep this high T_D value, in favor of safety.

4.3. *Tracked construction equipment forces*

The placement of cover soil on a slope with a relatively low shear strength inclusion (like a liner) should always be from the toe upward to the crest. Figure 6a shows the recommended method. In so doing, the gravitational forces of the cover soil and live load of the construction equipment are compacting the previously placed soil and working with an ever present passive wedge and stable lower portion beneath the active wedge. While it is necessary to specify low ground pressure equipment to place the soil, the reduction of the FS value for this situation of equipment working up the slope will be seen to be relatively small.

For soil placement down the slope, however, a stability analysis cannot rely on toe buttressing and also a dynamic stress should be included in the calculation. These conditions decrease the FS value and in some cases to a great extent. Figure 6b shows this procedure. Unless absolutely necessary, it is not recommended to place cover soil on a slope in this manner. If it is necessary, the design must consider the unsupported soil mass and the dynamic force of the specific type of construction equipment and its manner of operation.

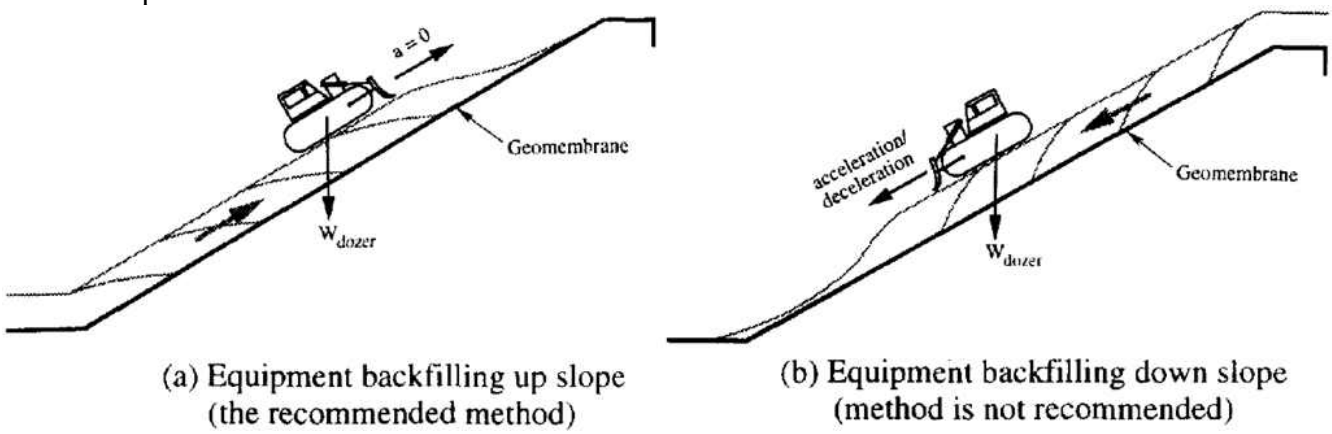

Figure 6. Construction equipment placing cover soil on slopes with geosynthetics (from Koerner and Soong, 1998).

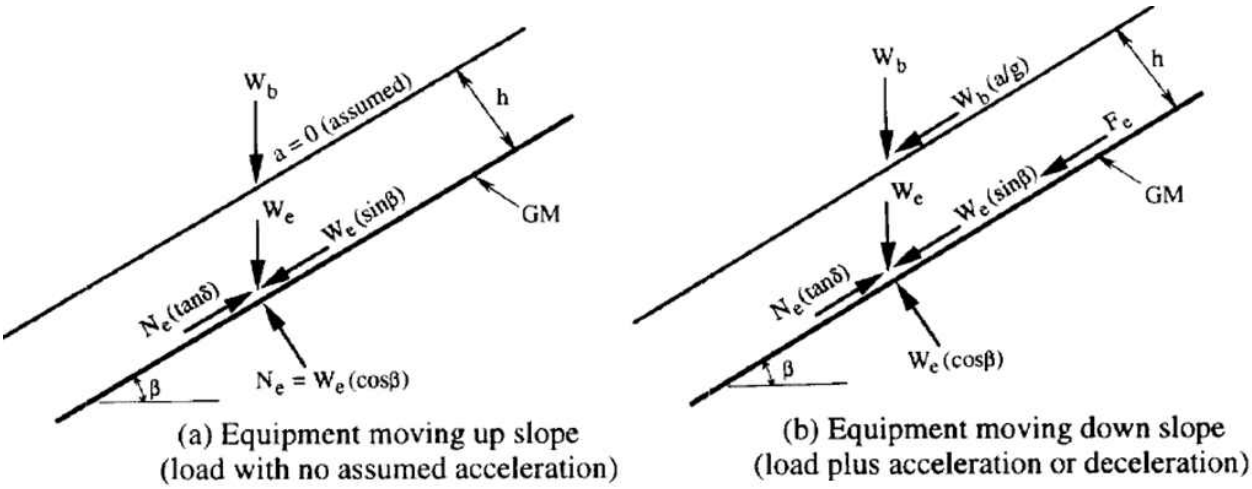

Figure 7. Additional limit equilibrium forces due to construction equipment moving on cover soil, see Figure 3 for the gravitational soil force to which the above forces are added (from Koerner and Soong, 1998)

For the first case of a bulldozer pushing cover soil up from the toe of the slope to the crest, the analysis uses the free body diagram of Figure 7a.

For the second case of a bulldozer pushing cover soil down from the crest of the slope to the toe, as shown in Figure 4b, or in any case for the bulldozer moving down the slope, the analysis uses the force diagram of Figure 7b.

The analysis uses a specific piece of tracked construction equipment (like a bulldozer characterized by its ground contact pressure) and dissipates this force or stress through the cover soil thickness to the surface of the liner. A Boussinesq analysis is used to transfer the bulldozer weight at the veneer surface as an equivalent vertical force on the liner interface (Poulos and Davis, 1974).

Moreover we have to consider an additional force due to acceleration (or deceleration) of the equipment along the slope. This analysis again uses a specific piece of construction equipment operated in a specific manner. The acceleration or deceleration produces a force parallel to the slope, equivalent to the weight of the bulldozer multiplied by the acceleration of the bulldozer and divided by the acceleration due to gravity. Its magnitude is equipment operator dependent and related to both the equipment speed and time to reach such a speed.

It is clear that the case of the equipment moving down the slope is the critical one; it is also obvious that, even if the soil is placed upward, when the equipment has reached the top of the slope it has to go down moving on the soil just placed.

Therefore only the case of the equipment moving down the slope will be considered for stability analyses.

The equipment force per unit width can be calculated as follows:

$$W_e = q \cdot w \cdot I \tag{20}$$

$$q = \frac{W_e}{2 \cdot w \cdot b} \tag{21}$$

where:

W_e = equivalent equipment force per unit width at the liner interface (kN/m)

q = uniform pressure applied by equipment tracks on the veneer surface (kPa)

w = length of equipment tracks (m)

b = width of equipment tracks (m)

I = influence factor at the liner interface

The influence factor I can be obtained from Fig. 8, after Poulos and Davis (1974).

The curve in Fig. 8 can be approximated with the polynomial expression:

$$I = 0.0127\,(b/h)^5 - 0.1661\,(b/h)^4 + 0.8380\,(b/h)^3 - 2.0307\,(b/h)^2 + 2.3709\,(b/h) - 0.1059 \tag{22}$$

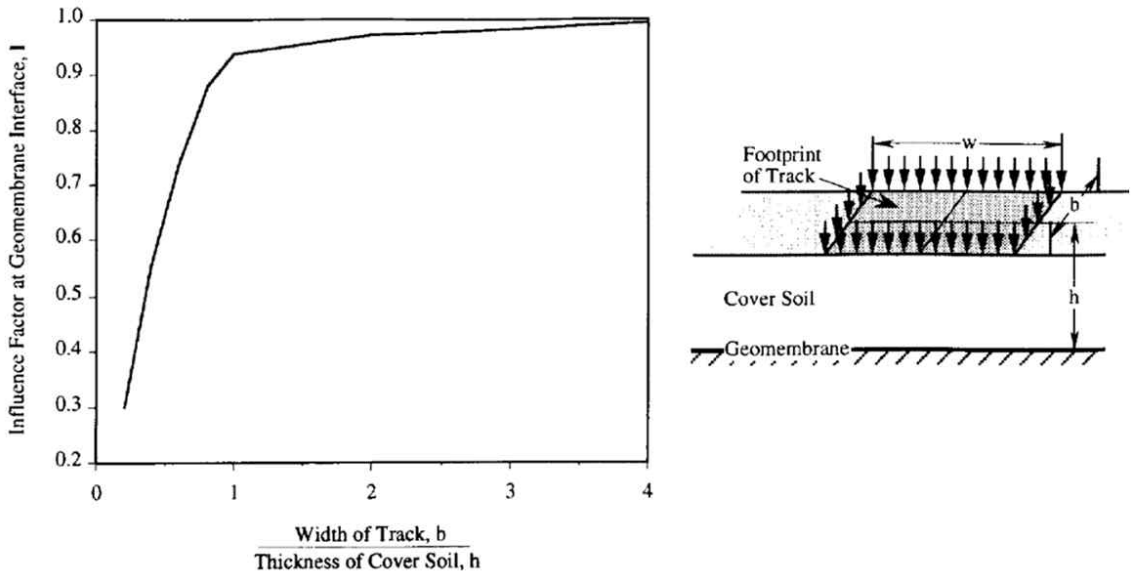

Figure 8. Values of the influence factor, I, for use to dissipate surface force of tracked equipment through the cover soil to the liner interface (after Poulos and Davis, 1974)

The acceleration of the bulldozer, coupled with the influence factor I from Figure 8 or Eq. 22, results in the dynamic force per unit width at the cover soil to liner interface, F_e (Fig. 7):

$$F_e = W_e \cdot \frac{a_b}{g} \tag{23}$$

where
F_e == dynamic force per unit width parallel to the slope at the liner interface (kN/m)
a = acceleration of the bulldozer (m^2/s)
g = acceleration due to gravity = 9.81 m^2/s

The acceleration of the bulldozer is equipment operator dependent and related to both the equipment speed and the time to reach such a speed; hence the value of acceleration, a, can be evaluated from Figure 9 (from Koerner and Soong, 1998).
When there is equipment moving down the slope, the critical situation occurs when the veneer cover has been laid up to the top of the slope: then the scheme of the stability analysis, including a geosynthetic reinforcement below the veneer cover, becomes as shown in Fig. 10.
The force N_e, normal to the sliding plane and produced by the equipment on the liner interface, is:

$$N_e = W_e \cdot \sin \beta \tag{24}$$

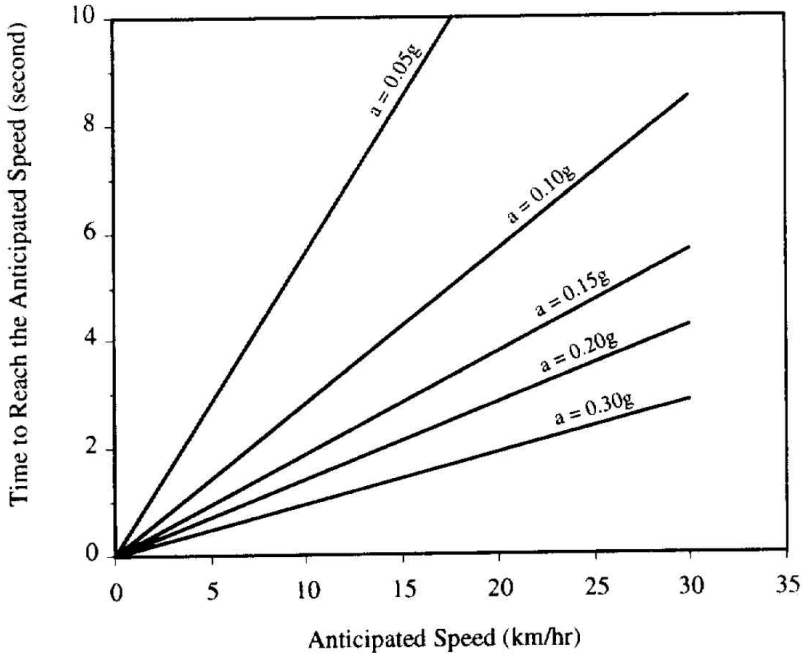

Figure 9. Graphic relationship of construction equipment speed and rise time to obtain equipment acceleration (from Koerner and Soong, 1998)

Considering that the equipment produce an unfavourable variable load, the expression of the interwedge force E_A can be obtained from the equilibrium of vertical forces on the active wedge $(\Sigma F_v = 0)_{\text{active wedge}}$:

$$E_A = \frac{1}{\sin\beta} \cdot \left\{ \begin{array}{l} \left[\gamma_{G1u} \cdot W_A \cdot (1 - \cos^2\beta - \cos\beta \cdot \sin\beta \cdot \tan\delta) - \gamma_{G1f} \cdot C_A \cdot \sin\beta\right] \\ -\gamma_{G1f} \cdot T_D \cdot \sin\beta + \left[\gamma_{Qiu} \cdot W_e \cdot (1 + \frac{a}{g} - \cos\beta \cdot \sin\beta \cdot \tan\varphi\right] \end{array} \right\} \tag{25}$$

where:

γ_{Qi} = amplification factor for variable unfavorable loads, from Table 1.

The expression of E_P remains the same of Eq. (16).

Stability is then verified if, for both combinations (A1 + M1 + R1) and (A2 + M2 + R2), the Factor of Safety results as per Eq. (17).

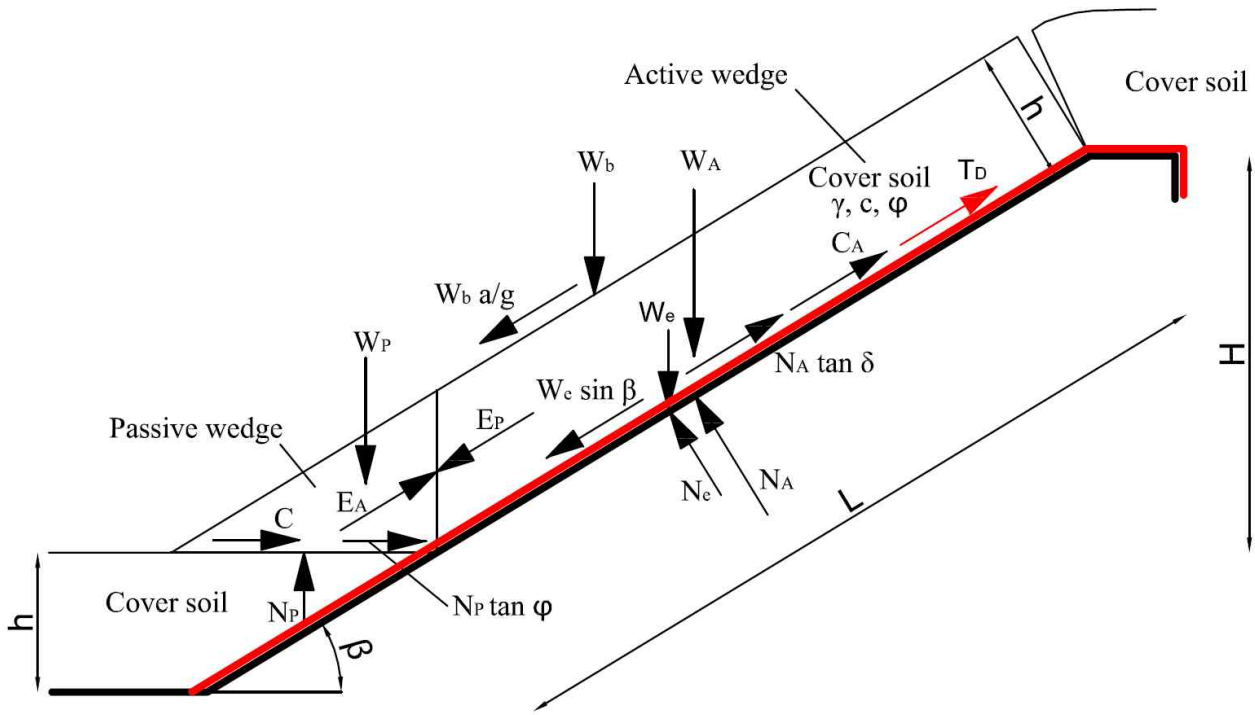

Figure 10. Limit equilibrium forces involved in a finite length slope analysis for a uniformly thick cover soil with equipment moving down the slope, including a geosynthetic reinforcement below the veneer cover

Also in this case, for high values of T_D the interwedge force E_A may become negative, and as a consequence also FS becomes negative: this is just a sign that T_D has been overdesigned; then the designer may try with a lower value of T_D, or the designer may keep this high T_D value, in favor of safety.

Note that the critical situation occurs at the end of construction of the veneer cover, hence analyses are considered at short term; since the probability of an earthquake occurring just at the end of construction is negligible, analyses with equipment loads are only carried out in static conditions, without considering any seismic condition.

Being a short term analysis, if reinforcement is used, the Reduction Factors for creep, chemical and biological damage shall be set equal to 1.0, while only the Reduction Factor for installation damage shall be applied.

The inherent danger of a bulldozer moving down the slope is readily apparent. Note that the same result comes about by the bulldozer decelerating instead of accelerating. The sharp breaking action of the bulldozer is arguable the more severe condition due to the extremely short times involved when stopping forward motion. Hence acceleration should be estimated from Fig. 9 with reference to the condition of sudden breaking.

Since acceleration and breaking of the equipment is unavoidable, an analysis should be made of the specific stability situation and the construction specifications should reflect the exact conditions made in the design. The maximum allowable weight and ground contact pressure of the equipment should be stated along with suggested operator movement of the cover soil placement operations.

From above analysis it is clear that the best construction solution is to avoid the forces produced by equipment moving on the slope: hence, whenever possible, placement of veneer soil cover using long reach boom excavators operating at the toe and the top of the slope should be preferred.

Truck traffic on the slopes can also give as high, or even higher, stresses than tracked bulldozers, and should be avoided unless adequately designed. Additional detail is given in McKelvey (1994).

The issue of access ramps is a unique subset of this analysis and one which deserves focused attention due to the high loads and decelerations that often occur.

4.4. *Seismic forces*

In areas of anticipated earthquake activity, the slope stability analysis of a final cover soil over a landfill or a heap leach pad site must consider seismic forces.

In the United States, the Environmental Protection Agency (EPA) regulations require such an analysis for sites that have more than 10 % probability of experiencing a 0.10 g peak horizontal acceleration within the past 250 years. For the continental USA this includes not only the western states, but major sections of the midwest and northeast states, as well. According to EuroCodes, seismic forces shall always be considered wherever the seismic design acceleration is > 0. EuroCode 8 (EN 1998-1) specifies how to determine the seismic parameter for a specific project.

The EuroCode 8 approach is now presented with reference to Italian Norm (NTC 2008), being Italy one of the most seismic Countries in Europe.

The motion generated by an earthquake at a site depends on the particular local conditions, that is, it depends on the topographic characteristics and stratigraphic deposits of soil and rock mass and the physical and mechanical properties of the materials that constitute them. The seismic motion at the surface of a site, associated with each category of the subsoil, is defined by the maximum acceleration (a_{max}) at the surface. Once the acceleration a_g expected at the bedrock has been defined, it is possible to calculate the maximum seismic acceleration at the surface by the following formula:

$$a_{max} = S \cdot a_g = S_S \cdot S_T \cdot a_g \tag{27}$$

where:
a_g = acceleration expected at the bedrock (m/s^2)
a_{max} = maximum acceleration at the surface (m/s^2)
S_S = stratigraphic amplification coefficient
S_T = topographic amplification coefficient.
The acceleration at bedrock a_g is provided by the National Agency in charge of seismic security, as a function of the seismic history and records for the site, and of the reference period for the seismic action.
Seismic maps, charts, tables, and softwares are usually available for determining the value of a_g on a grid of few km.

The topographic amplification coefficient S_T shall be evaluated based on the slope: for veneer covers it should be set equal to 1.0 for slope angle $\beta < 15°$, and equal to 1.20 for slope angle $\beta \geq 15°$.
The definition of SS depends on the category of the soil below the structure under consideration: it varies from 1.00 for rocky outcrops or very rigid soils characterized by values of Vs,30 (average velocity of shear waves in the first 30 m of subsoil) greater than 800 m/s, to 1.80 for deposits of low density coarse soils or poorly consistent fine grained soils characterized by values of $V_{s,30}$ lower than 180 m/s.
Stability analyses in seismic conditions can be carried out using pseudo-static limit equilibrium methods, where the seismic action is represented by an equivalent static force equal to the product of gravity forces by proper seismic coefficients for the horizontal and vertical components, given by:

$$K_h = \beta_m \cdot \frac{a_{\max}}{g} = \beta_m \cdot S_S \cdot S_T \cdot \frac{a_g}{g} \tag{28}$$

$$K_v = \pm 0.5 \cdot K_h \tag{29}$$

where:
K_h = horizontal coefficient for pseudo-static analyses
K_v = vertical coefficient for pseudo-static analyses
g = acceleration due to gravity = 9.81 m/s^2
β_m = dumping coefficient for reduction of the maximum acceleration at the site (from Tab. 5).
It has to be noted that the vertical seismic acceleration shall be considered both as downward ($+K_v$) and upward ($-K_v$), while the horizontal seismic acceleration shall always be considered as outward; hence each structure shall be analyzed with both values, positive and negative, of the vertical coefficient K_v; that is, two stability analyses are required for each value of K_h.
Once the seismic coefficients K_h and K_v have been determined, the stability analyses of the veneer cover in seismic conditions, with only gravitational forces applied, follow the scheme shown in Fig. 11.
Note that for seismic analyses all load amplification factors A1 and A2 shall be set equal to 1.00.
It is assumed that in seismic conditions the geotechnical parameters φ, δ, c, c_a have the same values as in static conditions.
Therefore, with reference to Fig. 11, the forces assume the same expressions as for the static case.

Table 5. Values of the dumping coefficient β_m according to Italian Norm (NTC 2008)

Expected acceleration at bedrock	β_m for soil category A	β_m for soil category B, C, D, E
$0.2 < a_g / g \leq 0.4$	0.30	0.28
$0.1 < a_g / g \leq 0.2$	0.27	0.24
$a_g / g \leq 0.1$	0.20	0.20

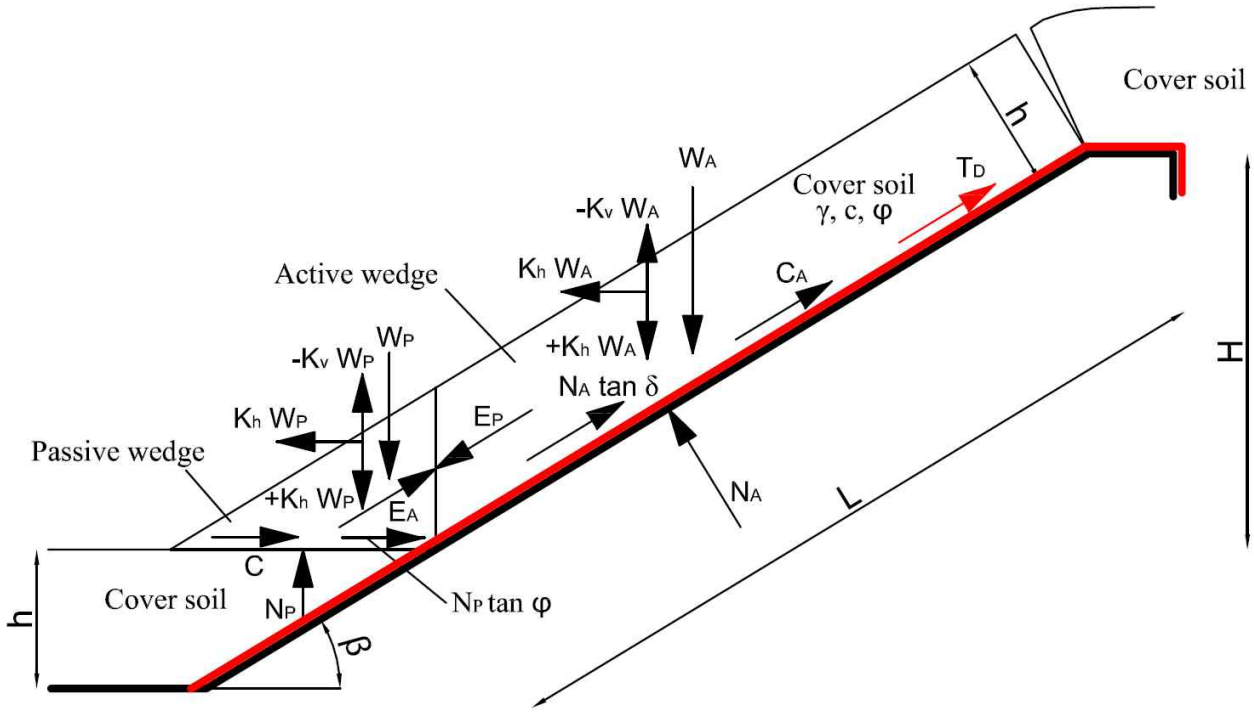

Fig. 11. Scheme for stability analyses of the veneer cover in seismic conditions, with only gravitational forces applied, including a geosynthetic reinforcement below the veneer cover

Considering that in seismic conditions all Amplification Factors for loads shall not be applied, the expression of the interwedge force E_{AS} in seismic conditions, from the equilibrium of vertical forces on the active wedge $(\Sigma F_v = 0)_{\text{active wedge}}$, becomes:

$$E_{AS} = \frac{1}{\sin\beta} \cdot \left[W_A \cdot (1 \pm K_v) \cdot (1 - \cos^2\beta - \cos\beta \cdot \sin\beta \cdot \tan\delta) - C_A \cdot \sin\beta - T_D \cdot \sin\beta \right] \tag{30}$$

The expression of the interwedge force E_{PS} in seismic conditions, from the equilibrium of horizontal forces on the passive wedge $(\Sigma F_h = 0)_{\text{passive wedge}}$, becomes :

$$E_P = \frac{C + W_P \cdot (1 \pm K_v) \cdot \tan\varphi - K_h \cdot W_P}{\cos\beta - \sin\beta \cdot \tan\varphi} \tag{31}$$

Stability is then verified if, for both combinations (A1 + M1 + R1) and (A2 + M2 + R2), the Factor of Safety results as per Eq. (17).

4.5. *Seepage forces*

Slope stability analyses of veneers can be carried out in drained conditions if either permeable soil or a drainage layer was placed above the barrier layer with adequate flow capacity to efficiently remove permeating water safely away from the cross section. The amount of water to be removed is obviously a site specific situation. Anyway, adequate drainage of final covers may not be available. According to Koerner and Soong (1998), the following situations have resulted in seepage induced slides:

- Drainage soils with hydraulic conductivity (permeability) too low for site specific conditions.
- Inadequate drainage capacity at the toe of long slopes where seepage quantities accumulate and are at their maximum.
- Fines from quarried drainage stone either clogging the drainage layer or accumulating at the toe of the slope thereby decreasing the as-constructed permeability over time.
- Fine, cohesionless, cover soil particles migrating through the filter (if one is present) either clogging the drainage layer, or accumulating at the toe of the slope thereby decreasing the as-constructed outlet permeability over time.

- Freezing of the drainage layer at the toe of the slope, while the soil covered top of the slope thaws, thereby mobilizing seepage forces against the ice wedge at the toe.

If seepage forces of the types described occur, a variation in slope stability design methodology is required. Additional discussion on this issue is given in Cancelli and Rimoldi (1989), Thiel and Stewart (1993) and Soong and Koemer (1996).

Since seepage forces may impair the stability of the veneer cover, requiring additional reinforcement to reach the prescribed Factor of Safety, it is recommended to design a geocomposite drainage layer below the veneer cover. Design of such drainage layer can be accomplished with the methods presented in ISO/TR 18228-4.

If, for any reason, a drainage layer below the veneer cover is not available, the formation of a phreatic surface due to water ponding at the toe or flow along the slope on the liner interface shall always be considered.

For the analysis of the veneer cover in presence of seepage forces, let's consider a cover soil of uniform thickness placed directly above a liner at a slope angle β. Different from previous analyses is that within the cover soil exists a saturated soil zone for part or all of the veneer thickness.

The saturated boundary can develop with two possibly different phreatic surface orientations:

- a horizontal buildup from the toe upward;
- a parallel-to-slope buildup outward.

When analyzing the stability of slopes using the limit equilibrium method, free body diagrams of the passive and active wedges are taken with the appropriate forces (now including pore water pressures) being applied.

4.5.1. The case of the horizontal seepage buildup

Horizontal seepage buildup can occur when toe blockage occurs due to inadequate outlet capacity, contamination or physical blocking of outlets, or freezing conditions at the outlets.

In case of horizontal pressure buildup there are two possible situations:

a) phreatic surface above the passive wedge (that is, above the interwedge vertical line),

b) phreatic surface partially in the passive wedge (that is, intercepting the interwedge vertical line).

a) Phreatic surface above the passive wedge

When the phreatic surface is above the passive wedge (that is, above the interwedge vertical line), the free body diagram of both the active and passive wedge is shown in Figure 12.

Hence this situation occurs when $H_w \geq (h / \cos \beta)$.

All symbols used in Figure 12 were previously defined except the following:

γ_{sat} = saturated unit weight of the cover soil (kN/m^3)

γ_m = total (moist) unit weight of the cover soil (kN/m^3)

γ_w = unit weight of water (kN/m^3)

H_w = vertical height of the free water surface measured from the toe of the active wedge (m)

u = pore pressure at toe (kPa)

U_h = resultant of the pore pressures acting on the interwedge surfaces (kN/m)

U_n = resultant of the pore pressures acting perpendicular to the slope (kN/m)

U_v = resultant of the vertical pore pressures acting on the base of the passive wedge (kN/m)

Above the phreatic surface the veneer soil will be dried only if the soil is free draining, but in general it will be in moist condition; hence, on the safety side, for the active wedge above the phreatic surface it is better to consider the total unit weight of soil in moist condition, γ_m.

When the water pressure acts on an inclined surface, the water pressure along the slope assumes the distribution shown in Fig. 12. The water pressure shall then be calculated as shown in Fig. 13.

Therefore the maximum value of the water pressure, at the toe of the veneer cover, is:

$$u = \gamma_w \cdot h \cdot \cos \beta \tag{32}$$

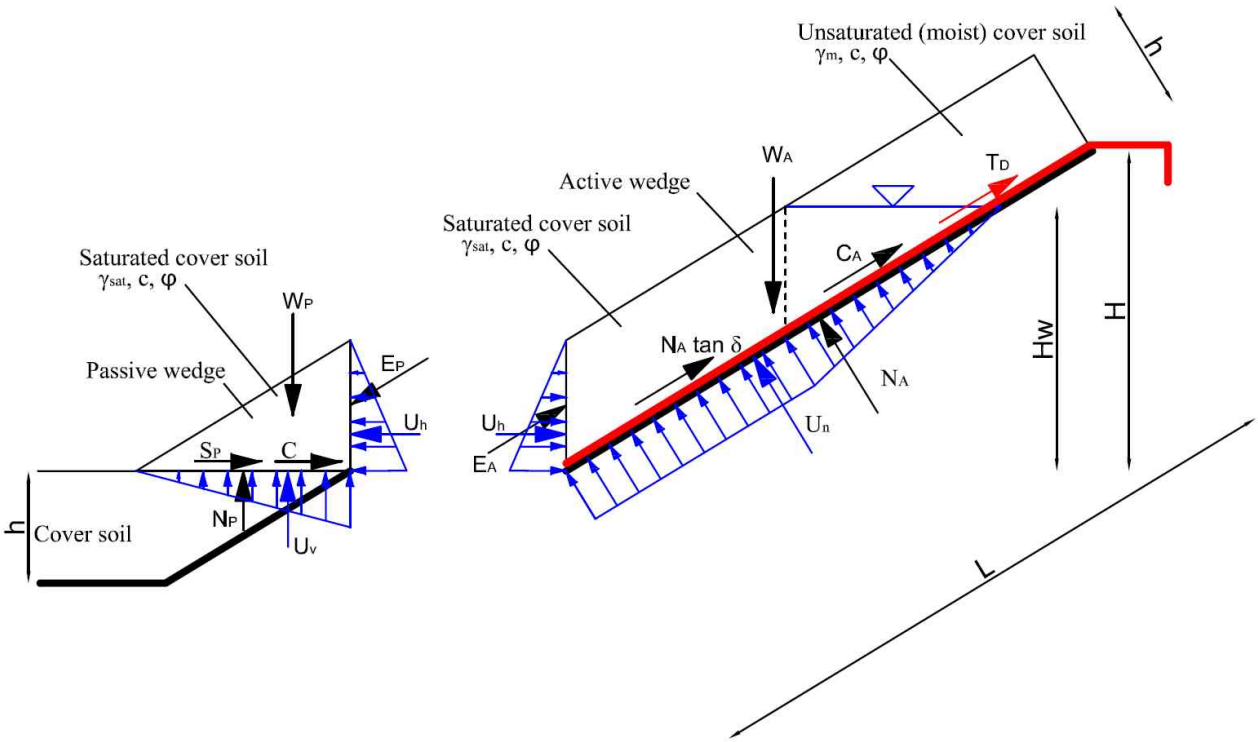

Figure 12. Limit equilibrium forces involved in a finite length slope of uniform cover soil with horizontal seepage buildup when the phreatic surface is above the passive wedge, including a geosynthetic reinforcement below the veneer cover

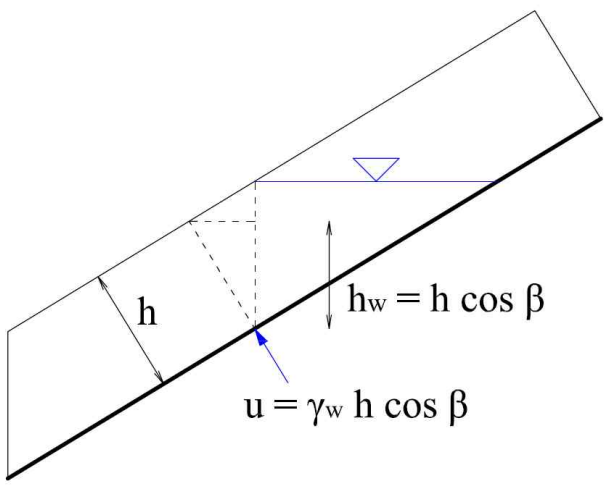

Fig. 13. Calculation of water pressure on a slope

With reference to Fig. 12, the forces assume the following expressions:

$$W_A = \frac{1}{2}\cdot\gamma_{sat}\cdot h\cdot\left[\left(\frac{2\cdot H_w}{\sin\beta}\right)-h\cdot\left(\frac{1}{\tan\beta}+\tan\beta\right)\right]+\frac{1}{2}\cdot\gamma_m\cdot h\cdot\left[\left(\frac{2\cdot(H-H_w)}{\sin\beta}\right)+\frac{h}{\tan\beta}\right] \tag{33}$$

$$U_n = \gamma_w\cdot h\cdot\cos\beta\cdot\left[\left(\frac{H_w}{\sin\beta}\right)-\frac{h}{2}\cdot\left(\frac{1}{\tan\beta}+\tan\beta\right)\right] \tag{34}$$

(35)

$$U_v = \frac{1}{2}\cdot\gamma_w\cdot h^2\cdot\frac{1}{\tan\beta} \tag{36}$$

$$N_A = W_A\cdot\cos\beta + U_h\cdot\sin\beta - U_n$$

(37)

$$S_A = N_A \cdot \tan\delta \tag{38}$$

$$C_A = c_A \cdot L = c_A \cdot \frac{H}{\sin\beta} \tag{39}$$

$$W_P = \frac{1}{2}\gamma_{sat} \cdot h^2 \cdot \left(\frac{1}{\sin\beta \cdot \cos\beta}\right) \tag{40}$$

(41)

$$N_P = W_P + E_P \cdot \sin\beta - U_v$$

(42)

$$C = c \cdot \frac{h}{\sin\beta}$$

(43)

$$S_p = N_P \cdot \tan\delta$$

The expression of the interwedge force E_A can be obtained from the equilibrium of vertical forces on the active wedge $(\Sigma F_v = 0)_{\text{active wedge}}$:

$$E_A = \frac{1}{\sin\beta} \cdot \left\{ \begin{array}{l} \gamma_{G1u} \cdot W_A \cdot (1 - \cos^2\beta - \cos\beta \cdot \sin\beta \cdot \tan\delta) - \gamma_{G1f} \cdot C_A \cdot \sin\beta - \gamma_{G1f} \cdot T_D \cdot \sin\beta \\ + \gamma_{G1u} \cdot \left[U_h \cdot \left(\sin\beta \cdot \cos\beta + \sin^2\beta \tan\delta\right) + U_n \cdot \left(\cos\beta + \sin\beta \cdot \tan\delta\right)\right] \end{array} \right\} \tag{44}$$

From the equilibrium of horizontal forces on the passive wedge $(\Sigma F_h = 0)_{\text{passive wedge}}$:

(45)

$$E_P = \frac{\gamma_{G1f} \cdot C - \gamma_{G1u} \cdot U_h + (\gamma_{G1f} \cdot W_P - \gamma_{G1u} \cdot U_v) \cdot \tan\varphi}{\cos\beta - \sin\beta \cdot \tan\varphi}$$

In seismic conditions the expressions of E_A and E_P become:

(46)

$$E_A = \frac{1}{\sin\beta} \cdot \left\{ \begin{array}{l} \left[W_A \cdot (1 \pm K_v) \cdot (1 - \cos^2\beta - \cos\beta \cdot \sin\beta \cdot \tan\delta) - C_A \cdot \sin\beta\right] - T_D \cdot \sin\beta \\ + \left[U_h \cdot \left(\sin\beta \cdot \cos\beta + \sin^2\beta \tan\delta\right) + U_n \cdot \left(\cos\beta + \sin\beta \cdot \tan\delta\right)\right] \end{array} \right\}$$

(47)

$$E_P = \frac{C - U_h + W_P \cdot (1 \pm K_v) \cdot \tan\varphi - U_v \cdot \tan\varphi - K_h \cdot W_P}{\cos\beta - \sin\beta \cdot \tan\varphi}$$

Note that the seismic acceleration is not applied to pore water forces.

Stability is then verified if, for both combinations (A1 + M1 + R1) and (A2 + M2 + R2), the Factor of Safety results as per Eq. (17).

b) Phreatic surface partially in the passive wedge

When the phreatic surface is partially in the passive wedge (that is, crossing the interwedge vertical line), the free body diagram of both the active and passive wedge is shown in Figure 14.

Hence this situation occurs when $H_w < (h / \cos\beta)$.

Also in this case, above the phreatic surface the veneer soil will be dried only if the soil is free draining, but in general it will be in moist condition; hence, on the safety side, above the phreatic surface it is better to consider the total unit weight of soil in moist condition, γ_m; while for the passive wedge above the phreatic surface it is better to consider the dry unit weight of soil, γ_d.

In this case the maximum value of the water pressure, at the toe of the veneer cover, is:

$$u = \gamma_w \cdot H_w \tag{48}$$

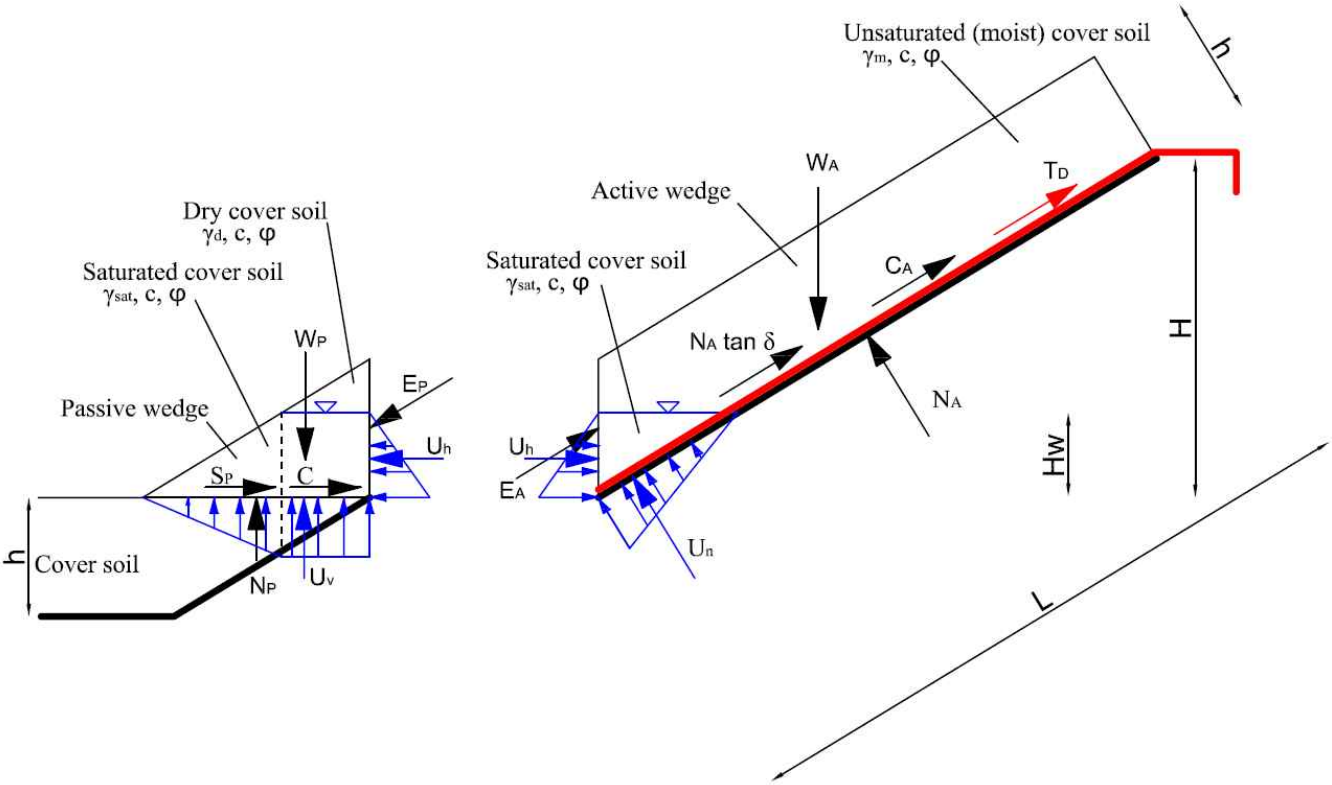

Figure 14. Limit equilibrium forces involved in a finite length slope of uniform cover soil with horizontal seepage buildup when the phreatic surface is partially in the passive wedge, including a geosynthetic reinforcement below the veneer cover

With reference to Fig. 14, the forces assume the following expressions:

$$W_A = \frac{1}{2} \cdot \gamma_{sat} \cdot \frac{H_{w^2}}{\tan\beta} + \frac{1}{2} \cdot \gamma_m \cdot \left[\left(\frac{2h \cdot (H - H_w)}{\sin\beta} \right) - h^2 \cdot \tan\beta + \frac{H_w}{\tan\beta} \cdot \left(\frac{2h}{\cos\beta} - H_w \right) \right] \tag{49}$$

$$U_n = \frac{1}{2} \gamma_w \cdot \frac{H_w^{\ 2}}{\sin\beta} \tag{50}$$

$$U_h = \frac{1}{2} \cdot \gamma_w \cdot H_w^{\ 2} \tag{51}$$

$$U_v = \gamma_w \cdot H_w \cdot \left(\frac{h}{\cos\beta} - H_w \right) \cdot \frac{1}{\tan\beta} + \frac{1}{2} \cdot \gamma_w \cdot H_w^{\ 2} \cdot \tan\beta \tag{52}$$

$$N_A = W_A \cdot \cos\beta + U_h \cdot \sin\beta - U_n \tag{53}$$

$$S_A = N_A \cdot \tan\delta \tag{54}$$

$$C_A = c_A \cdot L = c_A \cdot \frac{H}{\sin\beta} \tag{55}$$

$$W_P = \frac{1}{2} \gamma_{sat} \cdot H_w \cdot \left(\frac{2h}{\sin\beta} - \frac{H_w}{\tan\beta} \right) + \frac{1}{2} \gamma_d \cdot \left(\frac{h}{\cos\beta} - H_w \right)^2 \cdot \frac{1}{\tan\beta} \tag{56}$$

$$N_P = W_P + E_P \cdot \sin\beta - U_v \tag{57}$$

$$C = c \cdot \frac{h}{\sin\beta} \tag{58}$$

$$S_p = N_P \cdot \tan\delta \tag{59}$$

The expression of the interwedge force E_A can be obtained from the equilibrium of vertical forces on the active wedge $(\Sigma F_v = 0)_{\text{active wedge}}$:

$$E_A = \frac{1}{\sin\beta} \cdot \left\{ \begin{array}{l} \gamma_{G1u} \cdot W_A \cdot (1 - \cos^2\beta - \cos\beta \cdot \sin\beta \cdot \tan\delta) - \gamma_{G1f} \cdot C_A \cdot \sin\beta - \gamma_{G1f} \cdot T_D \cdot \sin\beta \\ + \gamma_{G1u} \cdot \left[U_h \cdot \left(\sin\beta \cdot \cos\beta + \sin^2\beta \tan\delta \right) + U_n \cdot \left(\cos\beta + \sin\beta \cdot \tan\delta \right) \right] \end{array} \right\} \tag{60}$$

From the equilibrium of horizontal forces on the passive wedge $(\Sigma F_h = 0)_{\text{passive wedge}}$:

$$E_P = \frac{\gamma_{G1f} \cdot C - \gamma_{G1u} \cdot U_h + (\gamma_{G1f} \cdot W_P - \gamma_{G1u} \cdot U_v) \cdot \tan\varphi}{\cos\beta - \sin\beta \cdot \tan\varphi} \tag{61}$$

In seismic conditions the expressions of E_A and E_P become:

$$E_A = \frac{1}{\sin\beta} \cdot \left\{ \begin{array}{l} \left[W_A \cdot (1 \pm K_v) \cdot (1 - \cos^2\beta - \cos\beta \cdot \sin\beta \cdot \tan\delta) - C_A \cdot \sin\beta \right] - T_D \cdot \sin\beta \\ + \left[U_h \cdot \left(\sin\beta \cdot \cos\beta + \sin^2\beta \tan\delta \right) + U_n \cdot \left(\cos\beta + \sin\beta \cdot \tan\delta \right) \right] \end{array} \right\} \tag{62}$$

$$E_P = \frac{C - U_h + W_P \cdot (1 \pm K_v) \cdot \tan\varphi - U_v \cdot \tan\varphi - K_h \cdot W_P}{\cos\beta - \sin\beta \cdot \tan\varphi} \tag{63}$$

Stability is then verified if, for both combinations (A1 + M1 + R1) and (A2 + M2 + R2), the Factor of Safety results as per Eq. (17).

4.5.2. The case of parallel-to-slope seepage buildup

Figure 15 shows the free body diagrams of both the active and passive wedges with seepage buildup in the direction parallel to the slope. Parallel seepage buildup can occur when soils placed above a liner are initially too low in their hydraulic conductivity, or become too low due to long term clogging from overlying soils which do not have a filter.

Also in this case, above the phreatic surface the veneer soil will be dried only if the soil is free draining, but in general it will be in moist condition; hence, on the safety side, above the phreatic surface it is better to consider the total unit weight of soil in moist condition, γ_m.

Identical symbols as defined in the previous cases are used here, with an additional definition:

h_w = height of free water surface measured in the direction perpendicular to the slope (m).

In this case the maximum value of the water pressure, at the toe of the veneer cover, is:

$$u = \gamma_w \cdot h_w \cdot \cos\beta \tag{64}$$

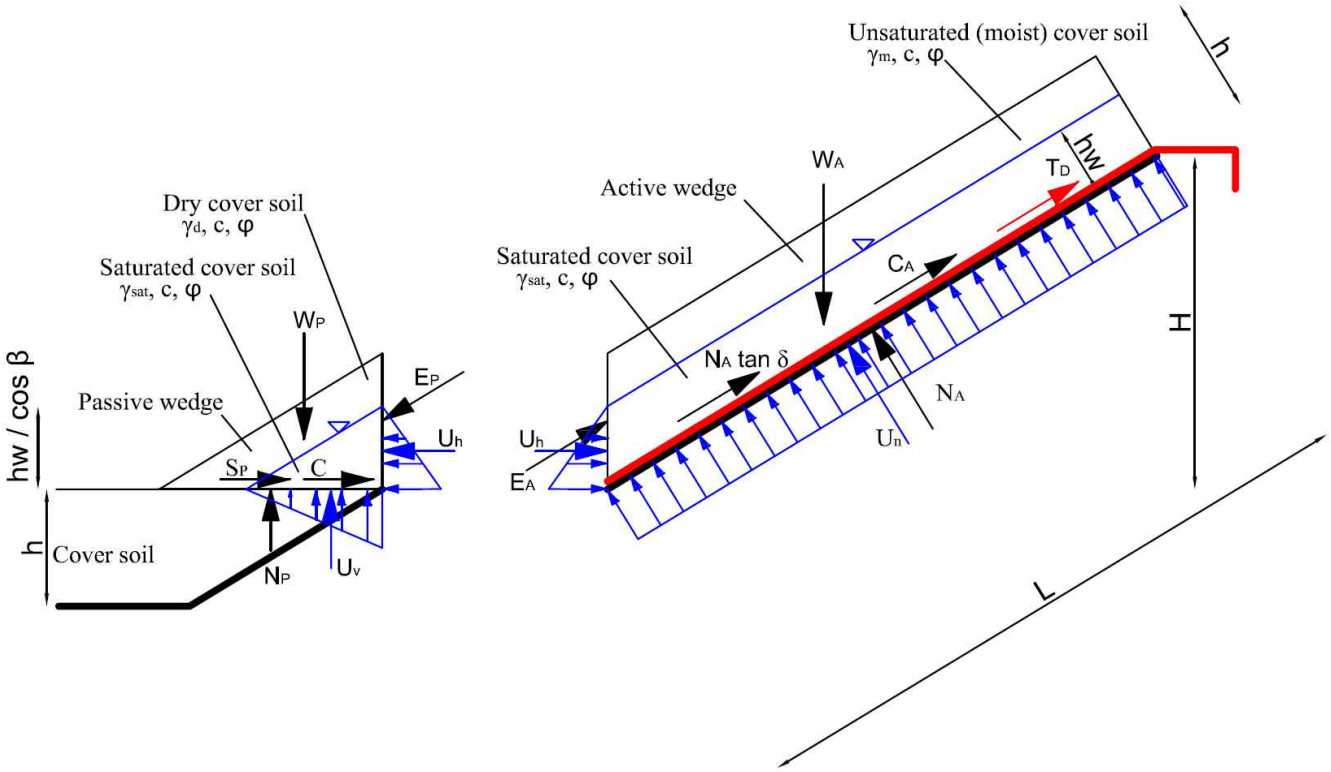

Figure 15. Limit equilibrium forces involved in a finite length slope of uniform cover soil with parallel-to-slope seepage buildup, including a geosynthetic reinforcement below the veneer cover

With reference to Fig. 15, the forces assume the following expressions:

$$W_A = \frac{1}{2} \cdot \gamma_{sat} \cdot h_w \cdot \left(\frac{2H}{\sin\beta} - h_w \cdot \tan\beta \right) + \frac{1}{2} \cdot \gamma_m \cdot (h - h_w) \cdot \left(\frac{2H}{\sin\beta} - h_w \cdot \tan\beta - h \cdot \tan\beta \right) \tag{65}$$

$$U_n = \frac{\gamma_w \cdot h_w \cdot H}{\tan\beta} \tag{66}$$

$$U_h = \frac{1}{2} \cdot \gamma_w \cdot h_w^{\ 2} \tag{67}$$

$$U_v = \frac{1}{2} \gamma_w \cdot \frac{h_w^{\ 2}}{\tan\beta} \tag{68}$$

$$N_A = W_A \cdot \cos\beta + U_h \cdot \sin\beta - U_n \tag{69}$$

$$S_A = N_A \cdot \tan\delta \tag{70}$$

$$C_A = c_A \cdot L = c_A \cdot \frac{H}{\sin\beta} \tag{71}$$

$$W_P = \frac{1}{2} \gamma_{sat} \cdot \frac{h_w^{\ 2}}{\sin\beta \cdot \cos\beta} + \frac{1}{2} \gamma_d \cdot \frac{\left(h^2 - h_w^{\ 2}\right)}{\sin\beta \cdot \cos\beta} \tag{72}$$

$$N_P = W_P + E_P \cdot \sin\beta - U_v \tag{73}$$

$$C = c \cdot \frac{h}{\sin\beta} \tag{74}$$

$$S_p = N_P \cdot \tan\delta \tag{75}$$

The expression of the interwedge force E_A can be obtained from the equilibrium of vertical forces on the active wedge $(\Sigma F_v = 0)_{\text{active wedge}}$:

$$E_A = \frac{1}{\sin\beta} \cdot \begin{Bmatrix} \gamma_{G1u} \cdot W_A \cdot (1-\cos^2\beta - \cos\beta \cdot \sin\beta \cdot \tan\delta) - \gamma_{G1f} \cdot C_A \cdot \sin\beta - \gamma_{G1f} \cdot T_D \cdot \sin\beta \\ + \gamma_{G1u} \cdot \left[U_h \cdot \left(\sin\beta \cdot \cos\beta + \sin^2\beta \tan\delta\right) + U_n \cdot \left(\cos\beta + \sin\beta \cdot \tan\delta\right)\right] \end{Bmatrix} \tag{76}$$

From the equilibrium of horizontal forces on the passive wedge $(\Sigma F_h = 0)_{\text{passive wedge}}$:

$$E_P = \frac{\gamma_{G1f} \cdot C - \gamma_{G1u} \cdot U_h + (\gamma_{G1f} \cdot W_P - \gamma_{G1u} \cdot U_v) \cdot \tan\varphi}{\cos\beta - \sin\beta \cdot \tan\varphi} \tag{77}$$

In seismic conditions the expressions of E_A and E_P become:

$$E_A = \frac{1}{\sin\beta} \cdot \begin{Bmatrix} \left[W_A \cdot (1 \pm K_v) \cdot (1-\cos^2\beta - \cos\beta \cdot \sin\beta \cdot \tan\delta) - C_A \cdot \sin\beta\right] - T_D \cdot \sin\beta \\ + \left[U_h \cdot \left(\sin\beta \cdot \cos\beta + \sin^2\beta \tan\delta\right) + U_n \cdot \left(\cos\beta + \sin\beta \cdot \tan\delta\right)\right] \end{Bmatrix} \tag{78}$$

$$E_P = \frac{C - U_h + W_P \cdot (1 \pm K_v) \cdot \tan\varphi - U_v \cdot \tan\varphi - K_h \cdot W_P}{\cos\beta - \sin\beta \cdot \tan\varphi} \tag{79}$$

Stability is then verified if, for both combinations (A1 + M1 + R1) and (A2 + M2 + R2), the Factor of Safety results as per Eq. (17).

5 REQUIRED STABILITY ANALYSES

The stability of the veneer cover shall be analyzed in the following Ultimate Limit State (ULS) conditions:

1) At the end of veneer construction, with equipment moving down the slope, in static conditions, without seismic actions nor seepage forces applied; being a short term analysis, if reinforcement is used, the Reduction Factors for creep, chemical and biological damage shall be set equal to 1.0.
2) At the end of the design life, in static conditions, without seismic actions but with seepage forces applied, either for horizontal seepage buildup or parallel-to-slope seepage buildup; being a long term analysis, if reinforcement is used, all the Reduction Factors (for installation damage, creep, chemical and biological damage) shall be applied.
3) At the end of the design life, in seismic conditions, with seismic actions and seepage forces applied, either for horizontal seepage buildup or parallel-to-slope seepage buildup; the vertical seismic acceleration shall be considered both as downward ($+K_v$) and upward ($-K_v$), while the horizontal seismic acceleration shall always be considered as outward; being a long term analysis in seismic conditions, if reinforcement is used, the Reduction Factors for installation damage, chemical and biological damage shall be applied, while the Reduction Factors for creep shall be set equal to 1.0.

All stability analyses shall be carried out for both the structural combination (A1 + M1 + R1) and the geotechnical combination (A2 + M2 + R2).

Note that, if reinforcement is used, the tensile strength T_D (and therefore the ultimate tensile strength T_{ult}) shall be finally set as the highest value from all analyses.

Also note that, for considering the tensile strength T_D, the reinforcing geosynthetic shall be properly anchored at the top of the slope; methods for designing the anchorage at top are not included in the present paper, but can be found in Koerner (2012).

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