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Design of earth structures reinforced with polymeric and metallic reinforcements using limit equilibrium methods

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Abstract: The use of planar polymeric and metallic reinforcements in mechanically stabilized earth structures has been growing in Indonesia in the last few years. This modern technique has been evaluated to be competitive if compared to other traditional earth retaining structures and has been safely employed in strategic infrastructure development projects in Indonesia. At the same time, there is still a lack of knowledge regarding the design methodologies for the mentioned structures among Civil Engineers. At present, there are two main currents followed by Engineers in order to design reinforced soil slopes and walls: Limit Equilibrium (LE) methods and Finite Element analysis. Several LE methods have been developed for slope stability analysis in the last decades, each is based on different assumptions and hypothesis and all of them utilize the well-known Mohr-Coulomb expression to determine the available shear strength along a potential sliding surface. The aim of this paper is to present the design basis of MSE walls/slopes using Janbu modified and Bishop modified Limit Equilibrium methods. The paper describes also how to include the presence of planar reinforcements in the LE formulations for the global stability and internal stability checks according to the so called "rigid model". Moreover, the procedure to properly calculate the long-term tensile strength of the planar reinforcements to be used in the LE formula will be presented.

1.0 Introduction

A common issue in developing highway, airport and railway infrastructure is the construction of soil embankments to accommodate the natural terrain contours. Due to the high terrain irregularity commonly found in Indonesia, it is not exceptional to face 30 m or even higher soil embankments to be built and stabilized before the planned infrastructure can be constructed.

Indonesian developers and public authorities as well as geotechnical and transportation engineers are frequently challenged with the target of maximising land usage in areas that often have both difficult topographic and geotechnical conditions.

For these reasons, the interest in building steep slopes has started to spread among developers and public authorities for both technical and economic reasons. The common aim in saving backfill materials (normally granular soils) and in increasing land use has pushed engineers towards the direction of designing and building more and more "reinforced earth structures".

The simple scheme in Figure 1 illustrates the advantages of use a steeper reinforced slope instead of an unreinforced natural one. The savings are not limited to the reduction of construction materials but also related to the increment in land usage at the top and the reduction in land acquisition at the toe.

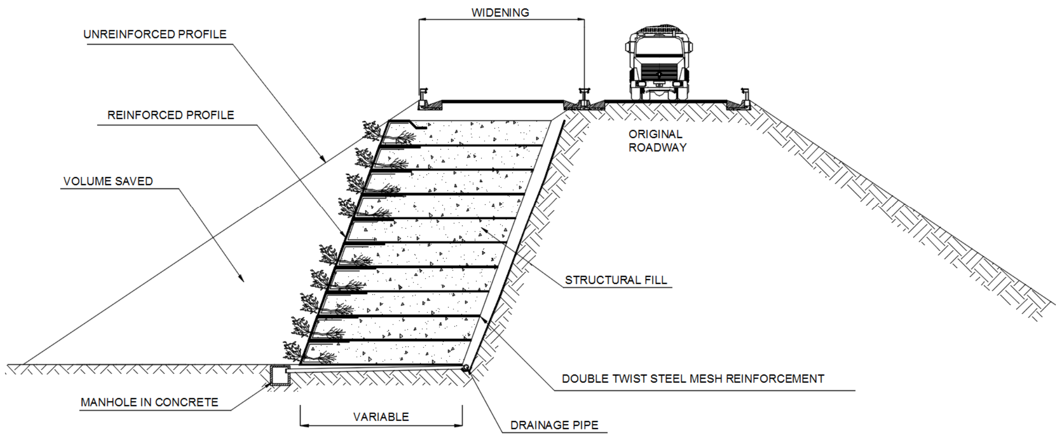

Figure 1: Comparison between a reinforced soil slope and an unreinforced one

From a technical point of view, it is important to underline that both natural slopes and reinforced earth slopes must be designed in accordance with solid and proven methodologies.

2.0 Reinforced soils

In technical literature, a soil structure constructed with steeper face angles than the ones permitted by the soils natural angle of repose are also known as Reinforced Soil Slopes (RSS) or Mechanically Stabilized Earth (MSE) walls.

In other words, a reinforced soil slope/wall is a natural soil which is engineered with the target of improving its original mechanical properties. This improvement is achieved through its mechanical compaction along with the installation of horizontally placed planar elements with high tensile strength properties. These elements are called reinforcements.

The intimate collaboration between the compacted soil and the reinforcing elements results in a "new" medium identified as "reinforced soil" or "reinforced earth". The reinforced soil has higher mechanical properties compared to unreinforced compacted soil.

In order to achieve the minimum required safety factors against sliding, the engineer shall design both the reinforcement pattern in the soil mass (vertical spacing and horizontal length) and the required reinforcement type and strength.

The reinforcing element can be either a metallic element (typically double twisted wire mesh) or a polymeric element (geogrids or woven geotextiles). The main characteristic of the reinforcing elements is that they are normally not able to carry any bending or compression loads and therefore their flexural stiffness is normally neglected and not contemplated in the design process.

On the other hand, as anticipated, they show high tensile strength properties at deformation levels that are compatible with the soil.

A common mistake is to consider nonwoven geotextiles as suitable reinforcements for reinforced soil slopes/walls. In fact, even if some high-mass nonwoven geotextile can develop tensile strengths as high as 50 kN/m, the deformation levels associates with those strengths are very high (ε > 50-60 %) and therefore not compatible with the soil deformations, which are in the range of 1 % to 6 %. Hence, in order to be effective, reinforcement shall develop the required strength with a deformation that is compatible with an acceptable deformation level of the reinforced soil.

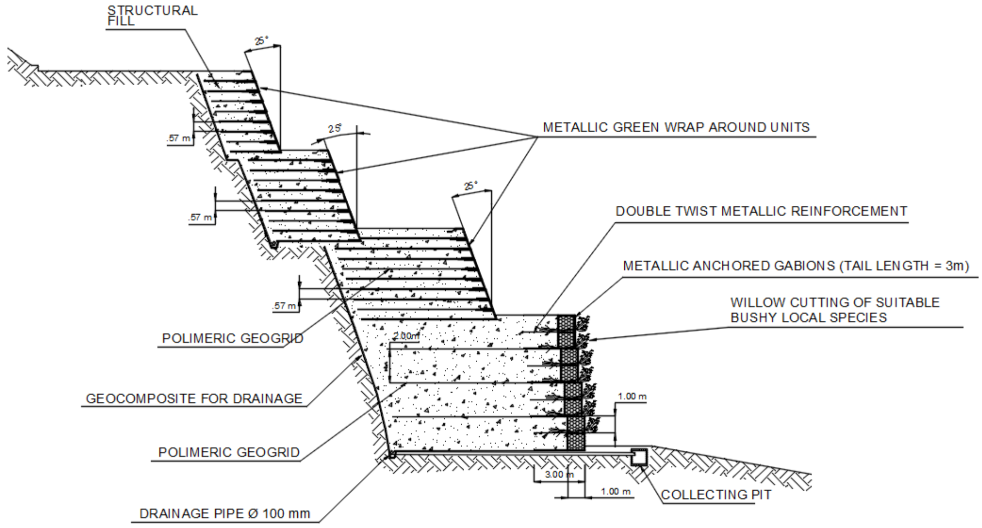

Figure 2: Typical section of a reinforced soil slope combining polymeric reinforcements (geogrids) and metallic reinforcements (anchored gabions and steel wrap around units)

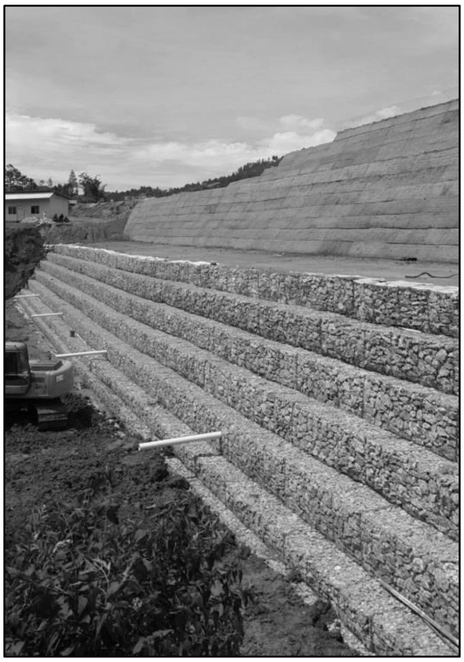

Figure 3: Application of a reinforced soil slope with polymeric and metallic reinforcement for an airport runway construction (courtesy of PT. Maccaferri Indonesia).

3.0 Safety factors, analysis and design methods

The most basic purpose of a slope stability analysis is determining a factor of safety against a potential slope failure, or landslide. If this factor of safety is determined to be large enough, the slope is judged to be stable or safe. A simplistic way to read the safety factor is to consider it as the ratio between the resisting forces and the destabilizing ones.

Located on the Pacific Ring of Fire, Indonesia has to handle the constant risk of earthquakes. Therefore, it is crucial to check whether the soil structure is stable even during a design seismic action or not. For this reason, it is important to distinguish a minimum safety factor calculated in static condition and one in seismic condition.

The required minimum safety factor in order to consider a slope or a soil structure safe is either given by dedicated codes of practice or by local design codes (e.g. Eurocode 7, BS 8006-1:2010, AS 4678). Since Indonesia has not yet released similar codes, the choice of the minimum safety factor is exclusively a designer's choice.

The authors propose to reach at least the following minimum factors of safety (FS):

- Static analysis → Min FS = 1.5
- Seismic analysis → Min FS = 1.1

Certainly, the above factors shall be considered just as an indication and the engineer shall carefully choose, case by case, the minimum FS required. The choice shall be a function of the engineer experience and level of confidence on the matter, of the type and importance of structure as well as of the uncertainties. The uncertainties are normally related to the reliability and the accuracy of the soil mechanical properties, of the geometry and land survey and on the external loads.

3.1 *Investigation methods*

Having underlined the importance of the safety factors, several investigation methods currently exist in order to evaluate them. The investigation methods can be classified in two main branches, Numerical Analysis methods (e.g. Finite Elements methods) and Limit Equilibrium methods.

The numerical modelling methods are considered the ones able to provide the user with the "exact" (or close enough) solution of the governing equations of the slope stability mechanics. They can evaluate the soil stress field as a function of the construction stages and time (from cutting operations to backfilling and compaction layer by layer). Moreover, they can assess the soil deformation and strain patterns and not only forces as Limit Equilibrium methods do. This is thanks to several constitutive models normally available in FEM software.

The main drawbacks of the numerical approaches are the high computational costs and the risk of obtaining outputs far away from the reality. The most common difficulties found by some non-expert users are related to choosing the soil input parameters and defining the model boundary conditions. In fact, in case the engineer is not an expert user, it is more likely to get non-realistic results using FEM software rather than using Limit Equilibrium method based ones.

That is why the authors recommendation is to attend specific courses and trainings on numerical modelling of geotechnical structures before approaching FEM software.

Another disadvantage of the Numerical Analysis methods is that they are normally implemented in commercial software. On the contrary Limit Equilibrium method software are also available at no monetary cost.

As anticipated, the main limitation of the Limit Equilibrium methods is that they can only take into account forces and are not able to provide information regarding stress distribution and deformation fields.

This paper is dealing only with Limit Equilibrium method theories and their implementation in calculation software.

3.2 *Limit Equilibrium (LE) methods*

In technical literature several proven LE methods are available. All of them aim to determining applied stresses and mobilized strengths on a certain number (defined by the user) of trial surfaces. Just mentioning some of them: Morgenstern-Price, Spencer, Bishop and Janbu. All these LE methods employed the principle of slices.

In the slices method, the soil mass is divided into a series of relatively small vertical slices following the shape of the trial sliding surfaces. Each slice base corresponds to the portion of soil domain cut by the sliding surface. Then, the forces acting on each slice as well as the resistances mobilized are evaluated. Eventually, considering the equilibrium of each slice, the safety factor against failure for each trial surface can be calculated.

Among all the safety factors, the lowest one is considered as the slope safety factor against failure. As a direct consequence, the shape of the trail sliding surface which the minimum safety factor belongs to, can be interpreted as the potential slope failure mechanism.

In order to calculate the available shear strength along a trial surface, all of the LE methods utilize the well-known Mohr-Coloumb criterion:

$$\tau_{max} = c' + \sigma' \cdot \tan \phi' \qquad \text{Equation 1}$$

Where τ_{max} = maximum allowable soil shear strength
ϕ' = effective internal angle of friction
σ' = effective vertical stress
c' = effective soil cohesion

The main differences between the various methods are related to the hypothesis and the assumptions made in order to solve the governing equation systems. The assumptions may be multiple and different. They includes: interslice forces, shape of the trial sliding surfaces as well as the limit equilibrium condition.

Figure 4 illustrates a slope which domain has been divided into vertical slices following a trial circular sliding surface.

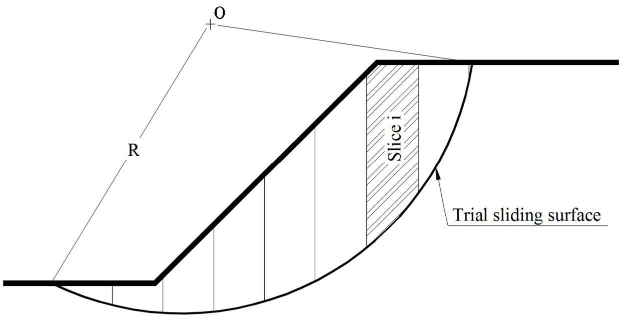

Figure 4: Slope cutting in vertical slices, following a circular sliding surface

Figure 5 illustrates and details the forces acting on each soil slice *i*.

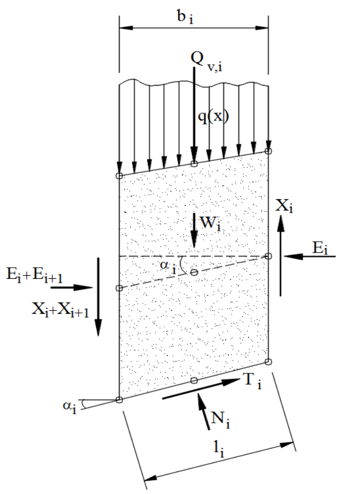

Figure 5: Forces acting on a generic slice *i*

One of the complications of the method of slices is the correct estimation of the vertical interslice forces (X_i and E_i with reference to Figure 5). In fact, since they depend on the soil deformation properties, stress-strain functions shall be included in the model. For this reason, some of the LE methods consider those interslice forces in the slope equilibrium to be mutually balanced, while some others do not.

A study conducted by Fredlund & Krahn (1977) aimed to compare the slope safety factor gained using various LE methods; the summary of the results is reported in Table 1.

Table 1: Summary table comparing the safety factors using six different LE methods (Fredlund & Krahn, 1977)

Case no.	Example problem	Ordinary method	Modified bishop method	Spencer's method			Janbu's modified method	Janbu's rigorous method	Morgenstern-price method f(x)=constant	
				F	*θ*	*λ*			*F*	*λ*
1	Simple 2:1 slope, 12m high, ϕ'=20°, c'=29kPa	1.928	2.080	2.073	14.81	0.237	2.041	2.008	2.076	0.254
2	Same as 1 with a thin weak layer with ϕ'=10°, c'=0	1.288	1.377	1.373	10.49	0.185	1.448	1.431	1.378	0.159
3	Same as 1 except with r_u=0.25	1.607	1.766	1.761	14.33	0.255	1.735	1.708	1.765	0.244
4	Same as 2 except with r_u=0.25 for both materials	1.029	1.124	1.118	7.93	0.139	1.191	1.162	1.124	0.116
5	Same as 1 except with a piezometric line	1.693	1.834	1.830	13.87	0.247	1.827	1.776	1.833	0.234
6	Same as 2 except with a piezometric line for both materials	1.171	1.248	1.245	6.88	0.121	1.333	1.298	1.250	0.097

The Morgenstern-Price method can be considered the most accurate and complex one. In fact, it investigates both circular and non-circular sliding surfaces, it satisfies both force equilibrium (vertical and horizontal) and momentum equilibrium and it expresses the interslice forces as a function of the soil stiffness and depth.

Therefore, with reference to Morgenstern-Price method, it is clear from Table 1 that both modified Bishop and Janbu methods lead to results close enough to the "exact" solution. This is why, among all the available LE methods, this paper focuses on the modified Bishop and the modified Janbu methods. The authors consider the two mentioned methods a good compromise in terms of simplicity, low computational cost and accuracy.

3.3 *Modified Bishop method*

The modified Bishop method, can only investigate the slope safety factor against sliding along circular surfaces. The shear forces X_i are considered to be internally equilibrated:

$$X_i + X_{i+1} = 0 \qquad \text{Equation 2}$$

In order to determine the normal forces N_i acting at the base of each slice, Bishop method solves the vertical forces equilibrium equation. Then, it solves the moment equilibrium equation in order to calculate the safety factor against sliding along the generic surface *s*. The moment equilibrium equation cannot be solved directly, but has to be solved by an iterative procedure.

3.4 *Modified Janbu method*

The modified Janbu method investigates non-circular sliding surfaces (polygonal, log-spiral etc.). The limit equilibrium of the slope and therefore the factor of safety are determined by horizontal force equilibrium only. The main assumption of the method is that the shear forces acting on the slices lateral surfaces are opposite and equilibrated as per Bishop method (Equation 2). Normal forces N_i acting at the base of each slice are computed by way of vertical forces equilibrium, as per Bishop method. Then, horizontal force equilibrium is used to calculate the factor of safety FS along a generic sliding surface *s*.

Due to the fact that vertical interslice forces are neglected, the method underestimates the actual slope safety factor. Therefore, Janbu introduced a correction factor F_0, to be applied to the safety factor derived from the horizontal equilibrium formula FS_s. The corrected safety factor FS_{s_c} of the generic surface *s*, is hence determined with the following formula:

$$FS_{s\_c} = FS_s \cdot F_{s\_0} \qquad \text{Equation 3}$$

The correction factor F_0 is a function of the geometry of the problem and of the soil cohesion and friction angle.

Compared to Bishop method, Janbu's one can be considered a more general method. In fact, since it applies to non-circular sliding surfaces it can be used to investigate more complex potential failure surfaces. Compared to Bishop method, it is more appropriate and more effective, when the predicted failure mechanism is through a sliding surface passing within a weaker soil layer bounded by two stronger materials (Figure 6).

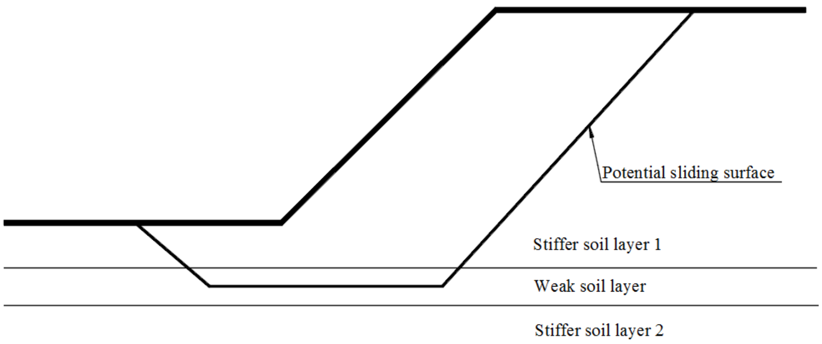

Figure 6: Potential polygonal shaped failure mechanism investigated with Janbu method

4.0 Proposed calculation model for reinforced soil slopes and walls

The slices principle has been presented along with the modified Janbu and Bishop calculation methods. But, how to include the presence of a planar reinforcing element in these calculation methods and theories? How to take into account the contribution of the reinforcement in a slope/wall stability analysis using LE methods? And how to consider the contribution of the facing elements?

To better understand, consider the shear resistance of a non-cohesive soil sample according to the Mohr-Coloumb rupture model:

$$\tau_{max} = \sigma' \cdot \tan \phi' \qquad \text{Equation 4}$$

Where the symbols are clarified in Paragraph 3.2.

The resistance mobilized by the solely soil along a finite portion of a sliding surface l_i is:

$$S_{r,i} = P_{n,i} \cdot \tan \phi_i' \qquad \text{Equation 5}$$

Where $S_{r,i}$ = maximum allowable soil shear force on a segment length l_i
$P_{n,i}$ = total normal force acting on the portion of sliding surface

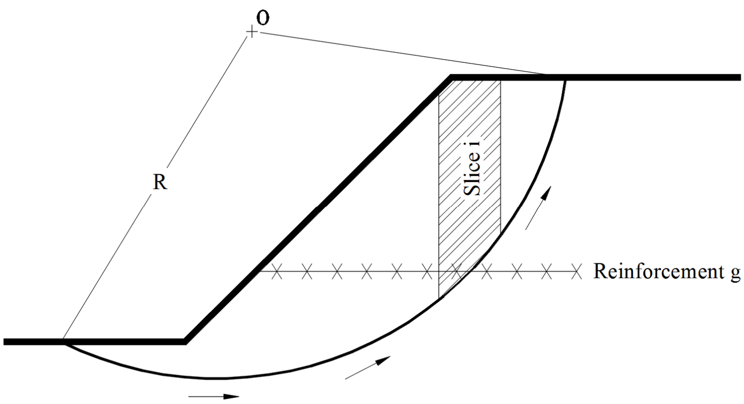

Figure 7: Reinforcing element *g* intercepting a circular sliding surface

With reference to Figure 8, consider now to insert a planar tensile resistant element g, forming an angle equal to θ_i with the normal to the sliding surface, in the soil element.

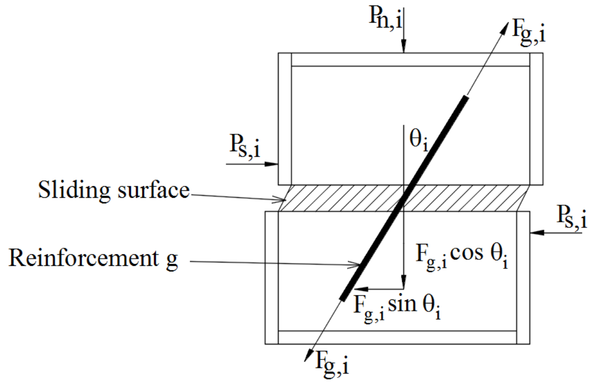

Figure 8: Soil element with a reinforcement *g* crossing a sliding surface

Defined $F_{g,i}$ as the design force which can be developed by the reinforcing element *g*, it can be demonstrated that the presence of the reinforcing element increases the shear resistance of the soil.

The force $F_{g,i}$ can be decomposed in two components, one parallel and one normal to the sliding surface, respectively named $F_{p\_g,i}$ and $F_{n\_g,i}$:

$$F_{p\_g,i} = F_{g,i} \cdot \sin\theta_i \quad \text{Equation 6}$$

$$F_{n\_g,i} = F_{g,i} \cdot \cos\theta_i \quad \text{Equation 7}$$

Both the components will contribution as stabilizing forces in the equilibrium system. In fact, the normal component $F_{n\_g,i}$ will generate an additional friction resistance, and the parallel component $F_{p\_g,i}$ can be consider as a stabilizing force (in Janbu equilibrium formula) or producing a stabilizing moment (in Bishop equilibrium formula).

In conclusion, the increment in shear resistance mobilized along the sliding surface portion l_i, can be calculated as:

$$S_{reinf\_r,i} = P_{n,i} \cdot \tan\phi_i' + F_{g,i}(\sin\theta_i + \cos\theta_i \cdot \tan\phi_i') \quad \text{Equation 8}$$

4.1 *The rigid model*

A simple but reliable model that has been implemented in some in-house developed software (e.g. Macstars W) in order to consider the stabilizing effects of the reinforcing elements is the so called rigid model. The rigid model can be applied to any type of reinforcing element (double twist steel wire mesh, geogrids, and woven geotextiles). It can be applied also to Bishop and Janbu methods.

According to the model, a certain number *n* of horizontal forces $F_{g,i}$ are introduced in the limit equilibrium formula. As anticipated, it applies to both the force equilibrium used in Janbu method and the momentum equilibrium valid for Bishop method presented in paragraph 3.3 and 3.4.

The number *n* of forces $F_{g,i}$ is equal to the total number of reinforcements intercepted by a generic trial sliding surface *s*.

As clarified, the forces $F_{g,i}$ included in the equilibrium limit formula are the forces developed by the reinforcing element/elements whenever they are traversed by a sliding surface. In the close range of the intersection point between the sliding surface *s* and the reinforcement *n*, the reinforcing element is forming an angle equal to θ with the normal to the sliding surface. In that relatively small intersection area, two portions of soil are trying to detach one from the other, generating a reinforcement deformation and tension.

In the model, the force $F_{g,i}$ is assumed to work horizontally and not tangent to the sliding surface. It has to be clarified that this hypothesis suits more to stiffer reinforcements as double twist steel wire mesh and geogrids rather than to woven geotextiles.

In the modified Janbu method, the force $F_{g,i}$ is simply a stabilizing additional element to be considered in the horizontal equilibrium formula. On the other hand, the contribution of the force in the modified Bishop method is expressed as the sum of the stabilizing moments of each force $F_{g,i}$.

A model assumption is that the reinforcing elements work in a synchronic way inside the reinforced mass. That means that the stabilizing forces $F_{g,i}$ provided by the intercepted reinforcements are developed all at the same time and depth.

It is evident that the choice of the force $F_{g,i}$ is crucial and decisive. An overestimation of this stabilizing factor may lead to serious mistakes and wrong stability evaluations.

In the proposed model, the force $F_{g,i}$ is considered as the lowest value between the following three forces:

$$F_{g,i} = \min \begin{cases} F_{po} \\ F_s \\ T_D \end{cases} \qquad \text{Equation 9}$$

Where F_{po} = force required to pull-out the reinforcement from the soil in the anchorage zone
F_s = force required to cause soil stripping in the failure zone
T_D = reinforcement long term design tensile strength

4.2 *Pull-out force*

The force required to pull-out the reinforcement from the anchorage zone can be calculated as follows:

$$F_{po} = 2 \cdot (\sigma'_v \cdot \mu \cdot \tan\phi' + c') \cdot Al \qquad \text{Equation 10}$$

Where Al = length of the reinforcement in the anchorage zone
F_{po} = pull-out force
μ = soil-reinforcement interface coefficient

The soil-reinforcement interface coefficient μ is a reduction factor ($\mu < 1$) applied to the soil internal friction angle in order to consider the discontinuity introduced by the reinforcement presence. The interface coefficient shall be provided by the reinforcement manufacturer as a result of pull-out lab testing programmes and it shall be expressed a function of the soil type.

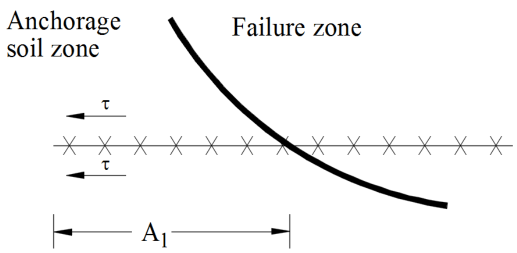

Figure 9: Detail of the reinforcement anchorage length A_l

Indicative common values of the soil-reinforcement interface coefficients are proposed in Table 2:

Table 2: Soil-reinforcement interface coefficients

μ	Soil
0.8 – 0.95	gravel
0.7 – 0.8	sand
0.5 – 0.7	silt - clay

4.3 *Soil stripping force*

The same principle can be applied to calculate the force required in order to cause soil stripping in the failure zone:

$$F_s = 2 \cdot (\mu \cdot \tan\phi' \cdot \int_{x=0}^{Lfz} \sigma'_v(x) \cdot dx + c') \cdot L_{fz} \qquad \text{Equation 11}$$

Where L_{fz} = length of the reinforcement in the soil failure zone
F_s = soil stripping force

In case the reinforcing element is continuously placed from the failure zone to along the slope facing and then wrapped again in the failure zone at a certain vertical spacing, the contribution of the friction mobilized on the facing and on the wrapped length shall be considered.

This concept applies to wrap-around (with both plastic and metallic facing units) and anchored gabion systems. Anchored gabion systems are monolithic steel wire mesh box-shaped basket provided with a metallic mesh tail extending into the soil in order to reinforce the backfill soil itself (Lelli et al., 2015).

The contribution of the wrapped elements can be taken into account as an additional force calculated as follows:

$$F_{s1} = 2 \cdot (\mu \cdot \tan\phi' \cdot \int_{x=0}^{Lw} \sigma'_v(x) \cdot dx + c') \cdot L_w + \int_{\theta=0}^{\pi} p \cdot \mu \cdot \tan\phi' \cdot r \cdot d\theta \qquad \text{Equation 12}$$

Where L_w = wrapped length of the reinforcement
r = radius of the virtual circumference, equal to half of the vertical spacing between reinforcements
p = soil pressure on the facing

The calculation model is illustrated in Figure 10:

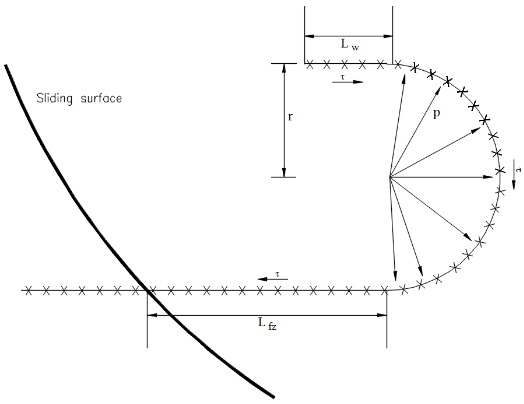

Figure 10: Model used to calculate the contribution of the facing of wrapped reinforcing elements

4.4 *Long term design tensile strength*

Many engineers have the misconception that the long term design tensile strength (T_D) of a reinforcing element used for soil stabilization is equal to the tensile strength which can be found on the products technical data sheet provided by the manufacturer.

What is normally indicated in the products data sheet is the ultimate tensile strength (UTS) or short term tensile strength (T_{chart}) of the reinforcement. This is actually a base tensile strength which derives from laboratory tests carried out under controlled conditions and following international standards (e.g. ASTM D6637).

The short term tensile strength (T_{chart}) cannot be considered representative of the real contribution that a reinforcement gives in a soil improvement problem. In fact, there are several different factors to take into account in order to calculate which is the realistic contribution in terms of tensile resistance that a reinforcement can give in a reinforced soil slope/wall.

Starting from the short term tensile strength value, the engineer shall calculate the long term design tensile strength of the reinforcement project by project.

In fact, the long term design tensile strength is a function of the reinforcement itself and of some boundary conditions. It depends on the reinforcement manufacturing process, on the reinforcement materials and coatings, on the structure design life, on the soil granulometry, on the soil pH level, on the soil temperature, on the presence of chemical or other degradation factors etc.

Different formula and criteria exist in literature on how to calculate the long term design strength T_D. The authors are presenting the one proposed in the British Standard BS 8006:1995 (code of practice for strengthened/reinforced soil and other fills), where a set of partial safety factors shall be applied to the reinforcement base tensile strength:

$$T_D = \frac{T_{CR}}{f_m} \qquad \text{Equation 13}$$

Where T_D = long term design strength of the reinforcement
T_{CR} = characteristic creep strength of the reinforcement
f_m = partial material reduction factor for the reinforcement

And

$$f_m = f_{m11} \cdot f_{m12} \cdot f_{m21} \cdot f_{m22} \qquad \text{Equation 14}$$

Where f_{m11} = reduction factor relating to the manufacturing process
f_{m12} = reduction factor relating to extrapolation of test data
f_{m21} = reduction factor relating to damages caused during the installation of the reinforcement
f_{m22} = reduction factor relating to damages and degradation due to environmental effects

For ultimate limit state, the value of T_{CR} is a percentage of the short term tensile strength T_{chart} and it depends on the structure design life and soil temperature. For metallic reinforcements only, the reduction in bearing load capacity due to creep is normally negligible.

For plastic reinforcements, typical percentage of the short term tensile strength are reported below (Moraci & Cardile, 2011 and Comedini & Rimoldi, 2013), as a function of the three main polymeric products used to manufacture reinforcements.

Table 3: Soil-reinforcement interface coefficients

Polymer	Percentage of T_{chart}
PET	40% – 80%
PP	20% - 30%
HDPE	20% - 40%

The range is very wide and is shall be considered just as an indication. A precise and realistic value shall be provided by the reinforcement manufacturer. The reinforcement manufacturer shall provide also all the other material partial factors (f_m) in order to let the engineer properly calculate the long term design tensile strength.

5.0 Conclusions

The main benefits introduced by reinforced soil structures for infrastructure development if compared to unreinforced natural slopes have been briefly described. The use of this relatively modern technique is growing and is taking place in the Indonesian market, consolidating itself as valid alternative to other traditional methods. Despite this growth, there is still some lack of knowledge regarding the design methods and the reinforcement properties to be used among civil engineers.

In order to increase the knowledge on this matter, the paper has given an overview on the design methods available in literature along with the working principle of a reinforced soil slope/wall. Furthermore, it briefly presented two Limit Equilibrium calculation methods widely used to evaluate the slope stability safety factor. These methods are the modified methods proposed by Bishop and Janbu. Following the results of the study conducted by Fredlund & Krahn (1977), the two methods can be considered a valid compromise between computational cost and accuracy among the other LE methods available. The authors also proposed an indicative slope minimum safety factor to be reached in both static and seismic analysis in Section 3.0.

Moreover, the "rigid" calculation model, used to include the reinforcement contribution in the slope stability has been described. The model can be applied to both the Janbu and Bishop methods. Following the model theory, the generic reinforcement is able to increase the stability safety factor introducing additional horizontal forces $F_{g,i}$ at the intersection point between a sliding surface and the reinforcement itself. The equations used to include the force $F_{g,i}$ in the safety factor calculation for both Bishop and Janbu modified methods have been detailed.

The proper definition of the maximum force $F_{g,i}$ that each reinforcement can mobilize plays a crucial role. This shall be considered as the lowest value between: the reinforcement long term design tensile strength (T_D), the pull-out force (F_{po}) and the stripping force (F_s). The procedures in order to calculate the mentioned forces have been detailed. It has been proposed also, the approach recommended by the British code of practice BS8006-1 in order to determine the long term design strength starting from the base strength of reinforcement. The base tensile strength can be normally found in the product technical data sheet.

It is authors' opinion that the formulation of a dedicated code of practice for designing and construction of reinforced soil structures similar to BS 8006 is required in Indonesia to incorporate modern design practices.

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