# Rock slope stabilization with flexible facing, case study and new superficial dimensioning methodology

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## Abstract

*Rock slope stabilization is one of the most applied geotechnical treatments on Colombian highways projects due to the country's geography, geology and main urban settlements locations. To connect the centers of productions with the main seaports and consumption centers, it is necessary to go through the three main mountain chains that split the country vertically. From the stabilization measures seen on finished and ongoing projects, a clear tendency to use shotcrete, erosion control mats and flexible facing can be identified, shotcrete being predominantly used over the two other facings.*

*In 12 years of Geobrugg's experience advising consultants, contractors, installers and supervising engineers in Colombia it has been common for all the involved actors to underestimate the importance of superficial stability analysis which has led to not accurate dimensioning and even to stabilization measures failures. The motivation of the present paper lies on summarizing the company's research, methodology and dimensioning approach on flexible systems, based on a deep understanding of the loads acting on the plate-net interaction and procuring long endurance, inexistence maintenance and cost-efficient solutions.*

*The implementation of the new dimensioning approach requires the active participation of consultant companies to unify design criteria, determine geotechnical parameters, conduct joint mapping and determine the failure mechanism and its implications on the design. Aiming to show the approach applicability. Even when systems of failure and discontinuities within a rock mass are shown differently, depending on the local tectonics and the lithological formation, where in most cases in any type of failure (planar, wedge or toppling) only the mechanism and main failure plane are analyzed, but in some cases, they do not dictate the kinematic behavior of the rock mass.*

*This paper explains a case of study which the consultant company Pedelta Colombia and Geobrugg presents an adjusted methodology based on the field data collection considering not only the structural data and rock mass global features such as openings, maximum block measurements, fault mechanisms, but also the superficial particularities such as the depth of the superficial joints among others. The methodology seeks for a order of magnitude when conducting the scaling procedure. This is relevant considering that those superficial blocks are the most prone to fail and that the traditional numerical models are programed to identify the most critical blocks bypassing smaller blocks that are kinematically most likely to slide down.*

*In this way, the design methodology was adjusted to better meet the geotechnical requirements on site, afterwards the traditional dimensioning methodology are used to check global and internal stability.*

## 1 INTRODUCTION

Due to the geology of Colombian landscapes, engineering highway, projects often come across with unstable rock slopes. The importance of suitable intervention measures reflects on the numerous highway closures slowing down the economy and having great implications on logistics, transportation, supply chains, and many other economic sectors.

The traditional approach used by local engineering include rather a small range of containment measures. Among the most popular can be found shotcrete, erosion control mats and flexible wire meshes, fixed through short steel rods to the ground.

Despite being traditional well stablished measures, their failures and short maintenance periods might be an indicator of a lack of methodology and engineering rigor in their dimensioning. The following chapters discuss briefly the current approaches, propose an innovative dimensioning methodology and show a case of study applied to a local project.

## 2 CURRENT APROACH

Stabilization measures of rock slopes are traditionally designed though kinematic analyses showing the possible failure mechanisms. Those analyses are made based on discontinuities mappings and are the initial phase for the dimensioning.

Afterwards, the safety factors for the identified wedges, plane and toppling failures are estimated. When those factors are below the limits fixed by the norm, the extraction force and length of the bolts are to be determined so that limit is exceeded.

Once the main failure mechanisms have been stabilized, this approach considers that the numerical phase of the dimensioning is over, which means that the global stability has been checked ensuring that, for example, no big wedges slide down or general plane failures occur. While in fact this is only the initial phase of dimensioning, but its subsequent steps will be discussed later.

## 3 SUPERFICIAL STABILITY ANALISYS

Having accounted the slope global stability analysis, it is of the greatest importance to consider now the lithology of rock masses and its implication on those analysis. In moderately jointed rock masses, the failure is controlled by the presence of discontinuities, but even more important is the fact that the interactions of those joints dictate the size and direction of movement of the slope failure. This implies that even if the critical failures are stabilized, smaller blocks between the nails can detach and damage the covering.

These superficial failures are the most common on rock slopes due to the high stabilizing forces involved in major failures, implying that the current approach leaves out the most important analysis, which according to Cała, et al. (2012) involves – for flexible coverings – two ground analysis: sliding off parallel to the slope and local wedge-shaped bodies liable to break out. Even when their approach is best suited for soil or weathered rock masses, it can also be applied to rock slopes.

As seen before, conventional dimensioning methods often don’t contemplate the superficial stabilization but when they do they assume that the failure mechanism correspond to falling of rock blocks, that mean no resistant forces are developed on the joints. This approach tends to simulate the most critical scenario. Nonetheless it implies the suppression of non-neglectable stabilizing forces, the over dimensioning of stabilization systems and loss of resources.

“Falling as an initial failure mechanism of a rock slope can be the result of an overhang only. Therefore, it only occurs in massive rocks with clearly defined joints” (Poisel & Preh, 2004). However, considering stabilization forces on the joints can only be done with prestressed coverings assuring that the cover is attached to the ground surface. Otherwise this assumption will guide to undermentioned retaining systems.

### 3.1 Sliding off parallel to the slope

Considering that each nail will withstand the acting forces on its area of influence, from the equilibrium of following body with length b, thickness t and width a is obtained the following equation according to Cała et al. (2012):

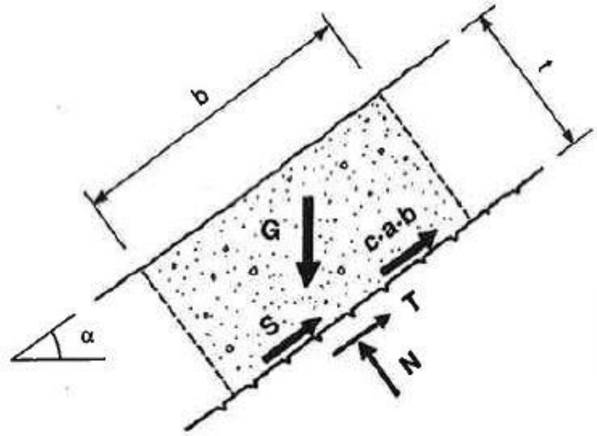


Figure 1. All forces acting on the body mass to determine the shear force S required for a certain safety (Rüegger & Flum, 2000) (1)

$$S = \frac{a \cdot b \cdot t \cdot \gamma \cdot (F \cdot sin\alpha - cos\alpha \cdot tan\varphi)}{F} - \frac{c \cdot a \cdot b}{F} \quad (1)$$

## 3.2 Local wedge-shaped bodies liable to break out

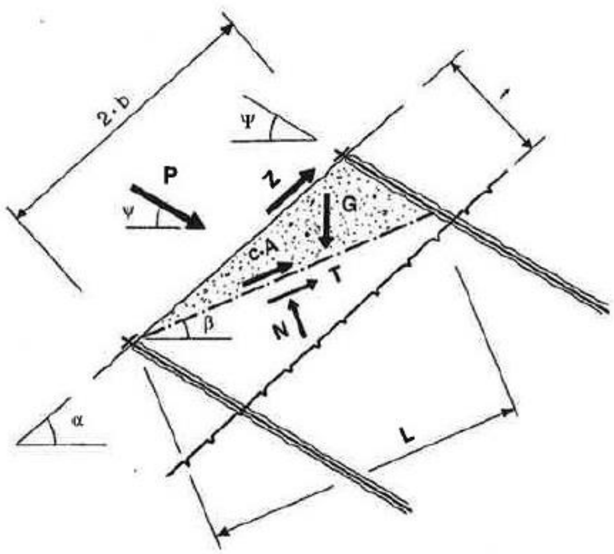


Figure 3 Forces actin on the wedge-shaped body (Rüegger & Flum, 2000)

The Figure 2 shows a wedge-shaped rigid body located between two nails breaking through and the forces acting on it. From considerations of equilibrium the force Z (2) can be obtained referred to the force transmitted over the mesh to the nail in direction parallel to the slope surface. This force is originated by the rupture body, assumed by the mesh through friction and can be obtained with the equation below as described by Cała et al. (2012).

This force is transmitted into the nail by the touch points between the plate and the mesh in direction parallel to the slope surface. Based on that it is recommended to implement plates with spikes at their edges instead of flat plates.

$$Z = \frac{a \cdot b^2 \cdot tan\rho \cdot \gamma (F \cdot \sin(\alpha - \rho) \cos(\alpha - \rho) \cdot tan\varphi}{2(F \cdot cos\rho - sin\rho \cdot tan\varphi)} - \frac{c \cdot a \cdot b}{cos\rho (F \cdot cos\rho \cdot sin\rho \cdot tan\varphi)} \quad (2)$$

On the other hand, the force ‘p’ (3) acting into the slope is also assumed by the mesh or net and transmitted to the subsoil through the contact between the plate and the mesh in direction perpendicular to the ground. The following scheme illustrate the contact points.

$$P = \frac{G[\gamma \cdot sin\beta - cos\beta \cdot tan\varphi]}{\gamma \cdot \cos(\beta + \psi) + \sin(\beta + \psi) tan\varphi} - \frac{Z[\gamma \cdot \cos(\alpha - \beta) - \sin(\alpha - \beta) tan\varphi] - c \cdot A}{\gamma \cdot \cos(\beta + \psi) + \sin(\beta + \psi) tan\varphi} \quad (3)$$

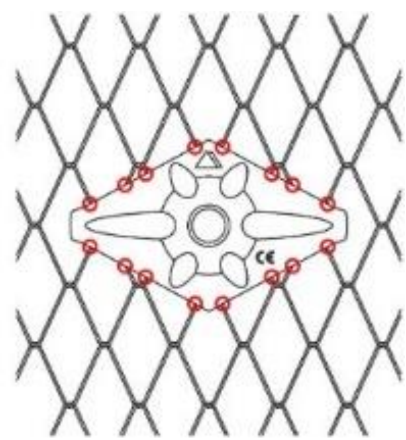

Figure 2 Contact points between the wire net and plate in direction perpendicular to the slope surface (Rüegger & Flum, 2000)

# 4 STRUCTURAL-GEOLOGICAL INVESTIGATION

## 4.1 Field collected data

For any case study, the existing site information should be initially reviewed and validated, it includes research of the proximity to important failures or inferred faults, based on state of art. Taking into account these reference factors, a field recognition is made, where local geological structures are detailed, such as possible folding and diaclasing systems. The different structural data of the joints are taken as dip and dip-direction, as well spacing, openings, depth and maximum failure block size and filling material, in the same way dip and dip-direction of the actual slope. (Greenberg, 2002)

When it is desired to perform the kinematic analysis of any rock mass slope, these data is required (conventional methodology), this leads only to the analysis of global failure for the maximum block size, which is fragmented into smaller blocks, giving rise to superficial failures due the interception and crossing of several families of diaclases (Figure 4).

In this specific method, the measures of the minimum block sizes are taken into account, since there is a considerable size sliding block, producing geotechnical failures not considered in another analysis.

Figure 4 Incidence of crossing joints creating smaller blocks

### 4.2 Sectorization and Kinematic analysis

To estimate the structural geology that influence the mechanical behavior in a slope, the following items must be consider:

- Structural data taken in the field
- The road alignment (it gives the slopes dip-direction character)
- Slopes geometry (Height and inclination)

This procedure is done in order to characterize slopes with similar conditions, grouping the different parameters and optimizing the information processing.

A structural sectorization must be conducted according to the representative joints' families and slope geometry. This allows to perform a kinematic analysis which identifies the probable failure mechanisms.

### 4.3 Determining block likely to fail

Knowing failure mechanism in the sector through the analysis software and field data, the maximum block that represents the known geometric, geological and structural characteristics for this failure mechanism is estimated and dimensioned, obtaining an idea of the geometry from the superficial fractures most likely to happen.

This block usually fragmented due to the periodic intersection and incidence of two or more diaclases families creating blocks of similar characteristics to the main, but with smaller dimensions that eventually leave silent witnesses (absence of the fractured block in the rock slope), which is a referent when the failed blocks are measure (
Figure 5).

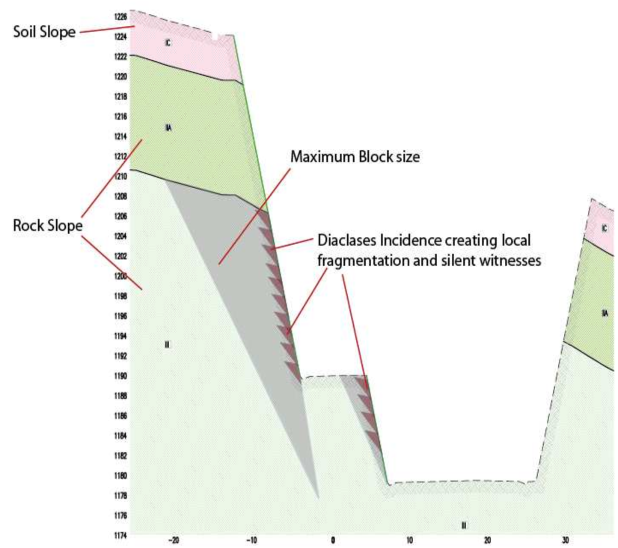


Figure 5 Rock slope profile showing the maximum block and the incidence of diaclases creating smaller blocks.

## 5 GEOTHENICAL ANALYSIS

To apply a geotechnical stability analysis, global and superficial methodologies must be included. These consider the rock mass parameters and the design conditions established by the geological and structural field survey and geomechanics characteristics of joints, explained in previous chapters.

### 5.1 Joints and rock mass parameters

The representative characteristics for each zone are established; first, the geometry slope design conditions should be clear such as height, alignment, and angle form horizontal: this information proceed from the geological inputs.

The modeling proposed is deterministic, a diaclase family is represented by an unique value of dip, dip-direction and persistence, hence, the values must be statistically representative (Akgun & Erkan, 2016). The intersection between two or more joints and the front and upper face of the slope turn into a wedge failure.

Additionally, the wall roughness and the infilling material of the joints it is also an important factor that can determine the stabilization systems, due to a significant effect on the interaction of joints and shear behavior (Khosrvavi, Khosravi, & Meehan, 2013). These parameters are described by a Mohr-Coulomb failure criterion, defined by a set of linear equations whose parameters are cohesion and angle of internal friction (Labuz & Zang, 2012). Test released to identify these parameters are listed below:

- Block sliding field test: used on joints without infilling material, this procedure results in measuring the wall roughness.
- Direct shear tests on soils specimens: used in clay or silt infilling, given its remolded soil nature, it might be convenient devise different test varying the unit weight. (Jiang & Gu, 2012)
- Direct shear test on rock specimens: used for infilling materials such as silica, calcite or sparite. (Muralha, y otros, 2014)

## 5.2 Block scaling

Commonly, the wedge sizes derived in this analysis are significantly larger than those observed on field, thus, the blocks must be scaled comparing to failed fragments (Figure 6). This scaling procedure is important because the assumed wedge have a significant effect on the safety factor and support requirements.

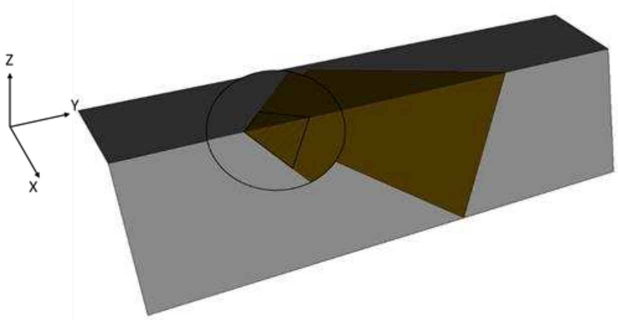


Figure 6 Wedge comparison of global and superficial (zoom view) stability analysis.

Basically, the persistence of the joints is reduced until the wedge has a similar size of the real failed fragment measured in field, ensuring that the geometric characteristics remain.

# 6 DIMENSIONING OF FLEXIBLE FACING

Once the global stability has been proved the superficial stability analysis has to be carried out based on the equations mentioned before. As the second crucial input for this stage is considered, the bearing resistances of the mesh against shearing-off in nail direction, upslope parallel tensile stress and shear stress due to pre-tensioning of the mesh or net. Although some of these bearing resistances can be asset theoretically, the true resistance can only be determined based on a laboratory testing. To do so the EOTA (2016) provide a complete guide of the set up and consideration for testing.

Here it is important to note that longitudinal and transverse tension tests are do not represent, in any way, the requirements of the mesh in the slope. Therefore, they shall be used only for quality control proposals.

There is other proof of bearing resistance to be conducted referred to the nails, but its analysis is not within the scope of the current paper.

The dimensioning process compares the mesh bearing resistance obtained through the tests against the requirements established by the equations in section 3. From this point, the process becomes iterative: assuming an initial nail pattern and verify the safety factors, as shown in section 6.1. The objective here is to find the most suitable solutions that guarantee the requirements established by the norms (NSR-10) and a cost-effective solution.

Even when this procedure has already been described by Cała et al. (2012), in rock slope stabilization the maximal block sizing described previously has to be considered.

## 6.1 Proof of the mesh to selective transmission of the force Z onto the nail

This prove is intended to corroborate that the acting force called ‘Z’ in previous chapters remain under the bearing resistance of the net or mesh. In case the its evaluation does not fulfill the norm requirements, the nail pattern must be closed, and the safety factor recalculated.

## 6.2 Proof of the mesh against shearing off at the upslope edge of the spike plate

The instability of a block as shown in Figure 2 cause a force pushing out the net in perpendicular direction to the slope surface. This force 'P' will be transmitted to the nail through the contact points at the upper half of the spike plate. This proof checks the shear stress caused by the force 'P' on the mesh contact points and compare it with its bearing resistance to ensure the minimum safety factor is been considered.

## 6.3 Proof of the mesh against puncturing

Having acknowledged the importance of pre-tensioning on the chapter 3, it is time to explain the procedure to generate that tension on the covering without material accumulation. First, it is important to install a threadbar with hot-rolled, continuous thread deformation on both sides, This allows to tighten the nuts over the spike plate correctly without having concerns about the threaded section of the rod meeting the slope surface or deeper. Among the benefits there are two additional to underline: flexibility in installation lengths by using couplers and high bond strengths between threadbar and cement grout.

The tighten of the nuts requires the mesh to be manufactured in high tensile steel otherwise the shear force up to 50kN can cut the contact point wires in the spike plate perimeter as shown in Figure 2. This proof is conducted to ensure that the pre-tensioning force do not puncture the mesh or net and compares the bearing resistance in the interaction plate – mesh with the proposed pre-tensioning force. Here is important to notice that the bearing resistance is not a property of the net by itself but a codependent property of the plate-nail combination. The wider the plate the more contact point is reached and the higher the bearing resistance. This also applies to the proofs shown on the chapters 6.1 and 6.2.

Secondly it is necessary to ensure the mesh or net edges to be pressed down into the ground surface. This can be done through tightened boundary ropes.

## 7 CASE STUDY

The case study results from a project located in the Colombian Western Cordillera, this mountain chain formed by igneous mafic rocks like diabase and basalt. In this location, outcrops a diabase barely disturbed (III according Deere and Patton weathering profile for igneous and metamorphic rocks) that involve denudation processes; hence, the slopes and ridges are a regular geoform on landscape.

The geological data was taken on a rocky outcrop (Figure 7) and showed in Table 1; its interpretation through a kinematic analysis by a stereonet diagram on Figure 8 shows a wedge failure.


Figure 7 Volcanic rock outcrop. (Diabase)

Table 1 Geological structural data of the case study.

| Dip/Dip Direction | Spacing (m) | Opening (mm) | Persistence (m) | Infilling material |
|---|---|---|---|---|
| 64/61 | 3.0 - 10.0 | 0.1-1.0 | 0.1-0.45 | Clayey residual soil c=25 k Pa and φ=31° |
| 67/142 | 3.0 - 10.0 | 1.0-5.0 | | |
| 62/190 | 1.0 - 3.0 | 0.0-0.1 | | |
| 46/84 | 1.0-3.0 | 0.0-0.1 | | |

Note: The slope projected structural input is 75/120 and maximum height of 12.5 m.

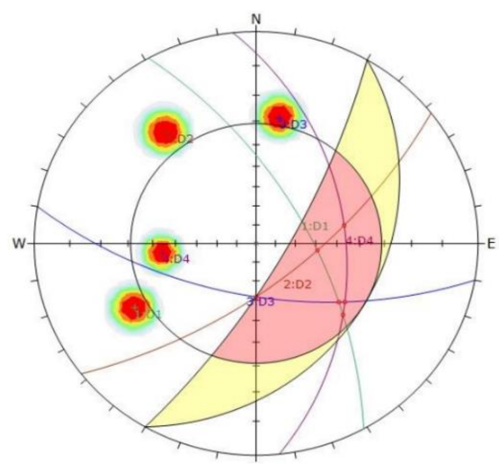


Figure 8 Kinematic analysis of the case study.

To determine the system's dimensions that reach a safety factor fixed by the current normative and specifications (NSR-10), the wedge size and involved forces may be estimated using force equilibrium equations. The result of this global analysis gives an idea of a considerable block size that appear into a geotechnical failure, which is least likely to happen because the safety factor is over 1.5, for this reason, a stabilization system is not needed (Table 2).

Otherwise, geological investigation registers failed blocks on the ground and silent witness of 20 cm to 30 cm (total volume of 0.020 $m^3$). These fragments do not represent the assumed wedge of global failure. Whereby, the block scaling is a relevant procedure to give credibility to what is found in field.

Through a superficial analysis it is possible to denote the similar characteristics between wedge fragments and the main wedge; then, the safety factor (S.F.< 1) reflects the real conditions, in other words, the slope has been suffering block fall. By which, is necessary to implement a stabilization system on the slope face using a bolt arrange satisfying the normative requirements noted on NSR-10 (Table 2).

Table 2 Comparative of geotechnical analysis.

| Item | Global analysis | Superficial analysis |
|---|---|---|
| Volume ($m^3$) | 1149.33 | 0.023 |
| Weight (k N) | 33330.2 | 0.7 |
| Safety factor | 1.5 | 0.98 |
| Stabilization system pattern (m) | Not needed | 5.0 x 5.0 |

Note: Stabilization system pattern refers to a bolt arrange on the slope face.

Finally, in order to find the competent solution, is necessary to develop a global and superficial analysis, choosing the critical design (an extensive bolt pattern); in this specific case study, the wedge fragmentation is most likely to fail, however for other conditions the result may vary.

# 8 CONCLUSIONS

- Mesh tensile strength test should not be the only specification used by consultant companies due to its limited correspondence with the requirements being faced by the mesh on the slope. Tensile test is intended to reflect how the mesh or net is required when an instable block breaks out between the nails, but do not reflect how this tension transforms when transmitted to the nails. Here shearing off, slope-parallel tensile stress and puncturing bearing resistance test must be conducted.
- Stabilizing forces happening on the contact between the rock mass and the block liable to break down can be considered just when the flexible facing is pre-tensioned otherwise by the time the block gets in contact with the mesh it has long lost contact to the mas rock and the failure mechanism should consider to be rock fall instead of wedge sliding. In this case the resultant force pressing into the mesh is greater. The same consideration applies when the nail pattern exceeds certain spacing, this happens due to the loss of the mesh tension as the spacing increases.
- Only kinematic analysis considering entire slope height are intendent to identify the critical wedges and plane failures. The slide down of these is in many cases improbable or even impossible kinematically speaking. The authors underline the importance of dimensioning focused on the actual and real failure mechanisms using the criteria to discern correctly. It makes no sense to dimension based on failures that are not likely to happen.
- Even when critical failures or big block failures are possible, the nail system withstanding these are likely not to nail smaller blocks, which can detach between nails and trigger greater failures. Herein lies the importance of facing dimensioning and superficial stability analysis.
- Current kinematic analysis does not consider equilibrium of forces and hence a special care must be taken while interpreting its results. In many cases blocks that accordingly to this analysis are liable to slide down cannot be identify on field putting on manifest its weak points. Criterion is fundamental here.

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