Rockfall protection trough ground reinforced embankments for an electric substation in Perú

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Abstract

Rock fall protection systems in geotechnics is a specialty that must be carefully studied and evaluated from different aspects that affect the functionality of the system and its design. The present article presents background describing the types that exist, the real-scale tests and the numerical modeling that later served to define the main design criteria that are used to solve a problem of rock fall that can lead to a potential risk on buildings , material goods and in the worst case, loss of human lives. This paper proposes a type of solution that can be designed to mitigate potential risks associated with rock fall events and debris flow. The ground reinforced embankments built with reinforced soil walls are an interesting solution when energy levels are higher than 5,000 KJ.

This paper shows a case study for the protection of an electrical substation located in the province of Huancavelica in Peru. As part of the design of the structure, the geotechnical, geological and hydrological studies of the study area were carried out in order to properly characterize the materials and determine the potential risk areas due to falling rocks that affect the electric substation.

1 INTRODUCTION

Protection systems to contain rock falls through ground reinforced embankments with geogrids are widely used in Europe, so far in Peru there are few experiences. In view of this and given the importance and need for risk prevention, it is necessary to develop an analysis methodology that combines theoretical and practical aspects that can be used within geotechnical engineering that allows a good design of this type of structures. For the present paper, a compilation of investigations made in real-scale models in Europe and modeling made by means of finite elements to determine the behavior of this type of structures against impact has been made, with this, concepts and design criteria can be clarified to consider within a project of this type.

To check the application of these concepts and criteria, reference is made to a soil reinforced wall of 10.8 meters high which was designed and built to protect the installations of an electrical substation in Peru, which is taken as a reference to verify the applicability of the theoretical approach, moving on to the constructive stage. The present work develops in a practical way the results obtained from the modeling in terms of velocity and impacted energy levels, and finally the design which is verified with a modeling according to the theory of limit equilibrium.

2 BACKGROUND

The publications of the bibliographical references that have been used for the made of this paper indicate that multiple real-scale trials have been carried out varying the dimensions of the embankments, volumes and mass speeds, trials that left behavioral results in ideal states of impact.

Within the tested embankments we have various geometries as mentioned below (Table 1):

Table 1. Types of embankments tested

Type	Geometry	Reference
Compact soil embankment with a side of gabions	Trapezium isosceles Incl. 35 ° face (regarding the horiz.) Max. 5 - 6m	Paronuzzi (1989) Del Greco et al. (1994)
Embankment of large rock blocks	Trapezium isosceles incl. 35 ° face (regarding the horiz.) Max. 12m	Pasqualotto et al. (2004)
Compact soil embankment with a side of gabions	Right angle trapeze Inclin. 35 ° face (regarding the horiz.) Incl. upper slope 90 ° (regarding the horiz.)	Oggeri et al. (2004) Lambert et al. (2008)
Gabion Embankment	Isosceles or parallelepiped trapezoid Incl. Slope inf. 70 - 90 ° (regarding the horiz.) Incl. Upper slope 70 - 90 ° (regarding the horiz.)	Wyllie and Norrish (1996) lambert et al. (2007)
Compact floor embankment reinforced with wood and steel bars	Trapezium isosceles Incl. Face 60 - 70 ° (regarding the horiz.)	Tissieres (1999)
Ground embankment reinforced with geotextiles or geogrids and with absorbent matrix	Trapezium isosceles Incl. Slope inf. 70 - 90 ° (with respect to the horiz.) Incl. Upper slope 70 - 90 ° (with respect to the horiz.) covered with large bags filled with sand (absorbent matrix)	Yoshida (1999) www.proteng.co.jp
Ground embankment reinforced with geotextiles, geogrids or wire mesh	Trapezium isosceles Incl. Slope inf. 70 - 90 ° (with respect to the horiz.) Incl. Upper slope 70 - 90 ° (with respect to the horiz.)	Lazzari et al. (1996) Burroughs et al. (1993) Peila et al. (2007) Pasqualotto et al. (2005)

The impacted embankments served to measure the deformations in both the face oriented to the upper slope and the one oriented to the lower one, depth of the crater caused by the impact of the block, displacements of the layers of the reinforced and non-reinforced embankments; all of them taken to their service limit state.

Fig. 1. Embankment impacted without reaching collapse

Fig. 2. Collapsed impacted embankment

The following table 2 shows the results obtained from the tests carried out on prototype embankments of 4.20m high and 5.0m wide by the Politecnico di Torino (Italy):

Table 2. Summary of test results at real scale

# Test	E Impact [kJ]	# Impact	Depth. Max. Slope. Up. [m]	Depth Max. Slope. Down. [m]
1	2500	1	0.60	0.17
		1	0.95	0.80
2	4500	2	1.30	1.20
		3	Collapse	-
3	4500	1	Collapse	-

Where test 1 and 2 were carried out on compacted soil embankments reinforced with uniaxial polymeric geogrids and test 3 on an embankment without reinforcement; in all three cases the test block was stopped by the embankment.

he geotechnical characteristics of the 3 embankments were sand-gravel soil type whose geotechnical parameters were ϕ=34°, c'=9 kPa y γ=21.10 kN/m^3. The tensile strengths of the uniaxial reinforcement geogrids were 50 KN /m.

Real-scale trials are compared with numerical modeling which summarizes results very close to those obtained. It should be noted that numerical modeling is only representative (Figure 3)

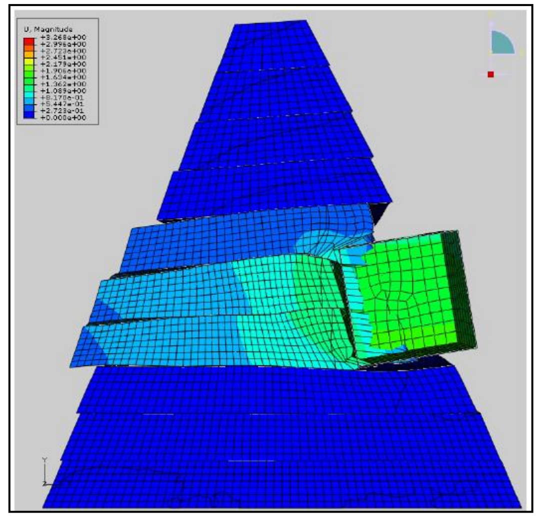

Figure 3. Finite Element Modeling on test embankments.

The embankments of reinforced soil are considered homogeneous and of uniform material are also considered in dry condition.

3 PRELIMINARY DESIGN CRITERIA

To prepare a pre-design of reinforced soil embankments, the following initial data must be taken:

- Volume of unstable blocks.
- Impact velocity.
- Impact energy.

The initial data can be processed with the Rocfall program using probabilistic rockfall simulations.

4 CALCULATION METHODOLOGIES

The pre-design is elaborated in a graphic way based on experiences and tests that were carried out with the support of the Italian standard UNI 11167 which describes the procedure of modeling of impacts at real scale and the data that must be measured to obtain the curve *Block Velocity vs. Maximum Penetration Depth and Impact Energy vs. Maximum Penetration Depth*. (See Figure 4 and 5)

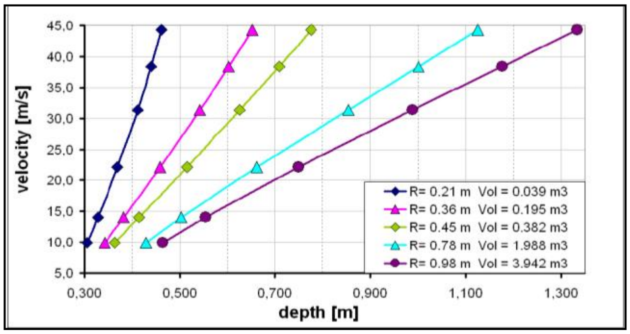

Figure 4. Velocity Curve vs Maximum Penetration Depth (Calvetti & Di Prisco, 2007)

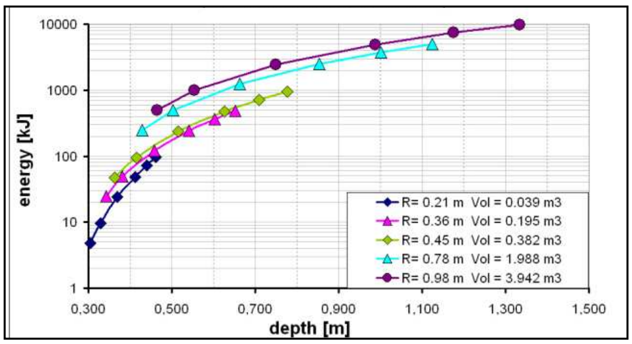

Figure 5. Impact Energy Curve vs Maximum Penetration Depth (Calvetti & Di Prisco, 2007)

The steps for a pre-design of the embankment body that can be used are described below:

- Define the Maximum Depth (Pb) with the previous tables.
- Minimum width for position h_t.

$t_w \geq 2\ P_b$ (1)

Where:

h_t: impact height

- α: slope of the face facing the embankment slope.
- Top free edge of the embankment:

$u_f = R_d + h_f$ (2)

Where:

R_d: Impact block radius

h_f: free edge height (0.5m)

- Minimum crown width.

$t_E \geq t_w - 2u_f/\tan\alpha$ (3)

- Trapezoidal embankment height.

$h_E \geq h_t + u_f$ ó (4)

$h_E \geq h_t + (t_w + t_E)/2{*}\tan\alpha$

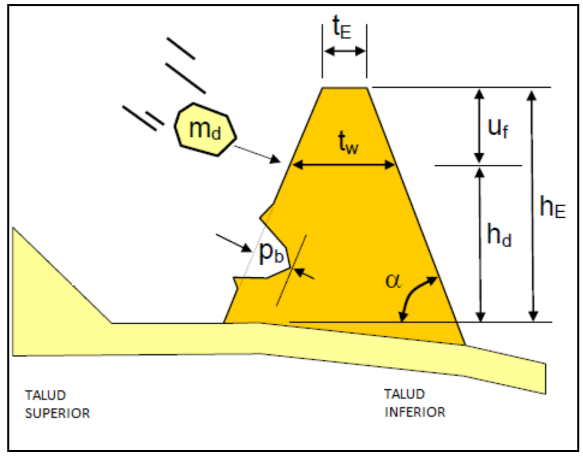

Figure 6. Pre-design scheme

The expressions described above can be used in the pre-dimensioning stage of the impact structure, subsequently it is necessary to perform global stability analysis and internal rupture that ensures an adequate safety factor of the structure before a possible slippage of the mass of the embankment produced by a force of impact on the structure.

In order to carry out this analysis process it is possible to use the limit equilibrium methods. It is possible to assume that internally of the structure efforts will be generated that will produce shear forces inside the ground that will induce the failure, so the filler material must be compact enough and have adequate geotechnical parameters that allow an allowable cutting effort, According to Mohr Coulomb's theory, the geotechnical parameters that govern this resistance are directly related to the ϕ (internal friction angle) y c (cohesion) and the material must have a high specific weight. Additionally, it is necessary to determine the modulus of elasticity of material E (Young Module) and ν (Poisson coefficient).

5 CASE STUDY

The main objective of this project was to provide protection to an electrical substation located in the central highlands of Peru against events of probable rock falls as well as debris flow that are prone to occur in the area under study. For this, different scenarios were studied that allowed us to achieve an optimal design and minimize the risk of estimated damage. The substation is located in the province of Huancavelica 6 hours from the city of Lima.

The project consisted of geotechnical evaluation to determine the causes of slope instability and the occurrence of rock fall phenomena that frequently affect the electrical substation.

The slope under study consists of colluvial deposits with intrusive matrix with the presence of large blocks of rock.

From the evaluations carried out, it was determined that the soil characteristics found are dispersive, that is to say that in the presence of water the soil matrix is easily decomposed, and this loses its structure, increasing the porosity and decreasing the cut resistance. This decomposition of the soil matrix causes an erosive process in the slope, which causes the large rock blocks present in the slope to begin to fall, generating a problem of rock falls.

Three main areas of rockfall landslides were identified, one at the top of the slope leading to the electrical substation, with a fall height of 800m from the top, the other two areas are located in the middle part of the slope where the phenomenon of dispersive soils is greater and where the slope erodes due to rain. The result of this erosive process causes the rock blocks to be loose and fall by gravity towards the substation. It was evidenced that there would be blocks of more than 1.50m in diameter that could generate a greater risk since these blocks would generate much greater impact energies, so this type of blocks were fixed to the slope and some eliminated.

Before starting with the geotechnical modeling in order to perform the rock fall assessment, a complete survey of the study area was carried out which consisted of:

- Topographic Survey in detail of the slope above the Substation.
- Geotechnical studies of the materials present in the area through open wells, seismic refraction tests and MASW, Standard Penetration Tests (SPT), and Laboratory Tests to determine the parameters of shear strength resistance of the soil.
- Geomechanical characterization of the rock mass, for this the classification system of Bieniawski (1979), called RMR index was used.

The RMR_{89} system has five basic parameters:

- Intact Rock Resistance
- Designation of Rock Quality (RQD)
- Discontinuity spacing
- Status of discontinuities
- Groundwater Conditions

8 geomechanical stations were executed on the rock outcrops present in the slope where values of 46 to 64 were obtained for the RMR_{89} classification according to Bieniawski, being classified as a rock from regular to good.

Additionally, hydrological and hydraulic studies of the slopes for the evaluation of drainage and problems associated with slope erosion were carried out.

5.1 Design criteria developed

When making a design of these systems, it is necessary to regulate the framework and the premises that must be fulfilled so that the task is approachable in a reasonable time, the cost is minimized and the installation and conservation tasks are simple and safe as may be possible.

The design level will correspond to the absorption capacity greater than 95% of the maximum energy and the height of 95% over the maximum rebound height determined from the rock fall analysis.

The height of a barrier is more important than the capacity: this means that although the barriers have resisted events that greatly exceed their design capacity, this advantage cannot be made effective if the rock passes over and continues its trajectory. The best barrier is not useful if it is too low, so it is important beforehand to determine the path lines of rockfall.

5.2 Numerical Modeling

The use of the Rockfall program by Rocscience has been used for rock fall modeling purposes. This program will allow us to obtain the energies of the impacts of the rocks at the point of intersection with the barrier of protection, and the probable heights of impact of rocks on the barrier of protection.

The analysis was made by modeling the kinetic energy, bounce height, velocity and trajectory, among other parameters, of blocks that fall freely through each area of the slope.

The parameters used in the modeling were the following:

- Geometric shape of the rocks.
- Rock dimensions.
- Specific rock weight.
- Rock mass.
- Number of simulated rocks in each profile.
- Movement: Sliding with rotation.

5.3 Design process

The design process consists in determining the impact energy, trajectory velocity, reach distance and bounce height, with that information the protection works are pre-sized; To achieve this objective, a section is chosen that represents the area and soil types are identified to estimate normal and tangential restitution coefficients; the analysis is performed in free fall, bounce and roll.

Due to the configuration of the geomorphology of the area, the slope was divided into three protection zones: Zone 1 due to the influence of creek 1 and 2, Zone 2 due to the influence of creeks 3 to 8 and Zone 3 due to the influence of creeks 9 and 10 (See Figure 7).

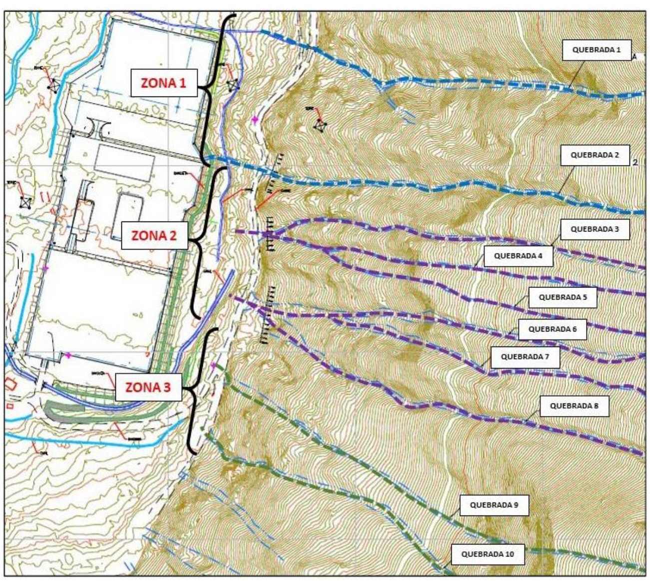

Figure 7. Zoning by influence of the creeks in the analysis area

5.4 Proposed solution against rockfall

As a protection system, a static barrier embankment consisting of a 10.8m high Terramesh Green soil reinforced wall and reinforced with high-strength geogrids with a capacity to absorb impact energies of 5,000 KJ to 8,700KJ, were proposed. they could present under the creeks 3,4,5,6,7, 8 and 9.

Static barriers were designed according to the method of Calvetti & Di Prisco (2007) for embankments against falling rock from reinforced soil.

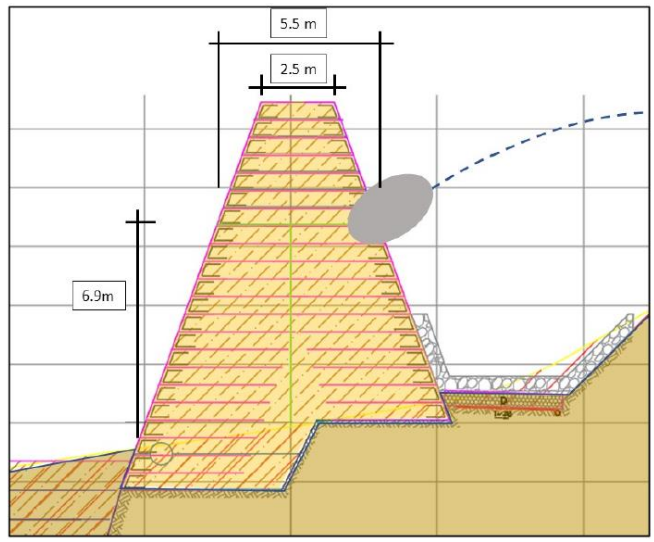

Figure 8. Static Barrier Embankment Scheme with Reinforced Soil Wall.

From the analysis of rock falls and block sizes, a probable size of 1.50m was determined whose energies due to rock falls were up to 5,000 KJ which was considered to select the type of protection system by means of a static barrier (Shown Figure 9)

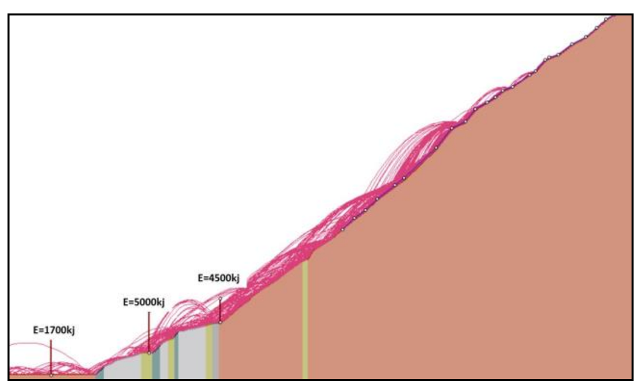

Figure 9. Rock fall model without protection, with probable 1.50 diameter blocks

5.5 Results of analysis of rocks falls and stability of the static barrier embankments.

With the help of Rockall 4.0 software, rock fall simulations were performed according to the paths in each creek and based on 3 block diameters found in the area (0.5, 1.0 and 1.5m) in order to determine the impact energies on the static barrier, also the height of impact and the depth or deformation that the face of the static barrier will suffer due to the impact of a rock.

Table 3. Results of rock fall analysis for the static barrier with reinforced soil

Analysis Zone (Creek)	Block Diameter	Impact Height	Impact Energy (KJ)	Depth (m)	Wide impact zone (m)
3	0.5	9.1	70	0.44	3.8
	1.0	8.7	750	0.72	4.1
	1.5	6.9	3,800	1.00	5.4
4	0.5	5.9	63	0.43	6.1
	1.0	4.2	460	0.62	7.4
	1.5	5.5	1,300	1.00	6.4
5	0.5	6.8	85	0.45	5.5
	1.0	5.1	680	0.69	6.7
	1.5	5.2	2,400	0.85	6.6
6	0.5	6.6	29	0.38	5.6
	1.0	7.7	520	0.63	4.8
	1.5	7.5	1,650	0.72	5.0
7	0.5	3.4	70	0.44	7.9
	1.0	4.0	300	0.55	7.5
	1.5	7.5	2,700	0.87	5.0
8	0.5	4.3	95	0.46	7.3
	1.0	3.0	270	0.53	8.2
	1.5	5.6	2,600	0.86	6.3
9	0.5	7.9	55	0.42	4.7
	1.0	4.1	400	0.60	7.4
	1.5	5.5	1,500	0.70	6.4

The most probable energy of 3,800 KJ was determined, for an impact height of 6.90m on the static barrier and a penetration depth of 1.0m in the area of creek 3.

If you compare the depth of deformation estimated according to table 3 according to the total dimension of the structure with the full-scale tests shown in table 2, we can indicate that it is acceptable and that it does not affect the stability of the entire structure. To check the internal stability of the static barrier in conjunction with the slope of the slope, limit equilibrium modeling was performed using the slide 6.0 software considering the resistance parameters of soils and rocks according to the geotechnical studies made in the area.

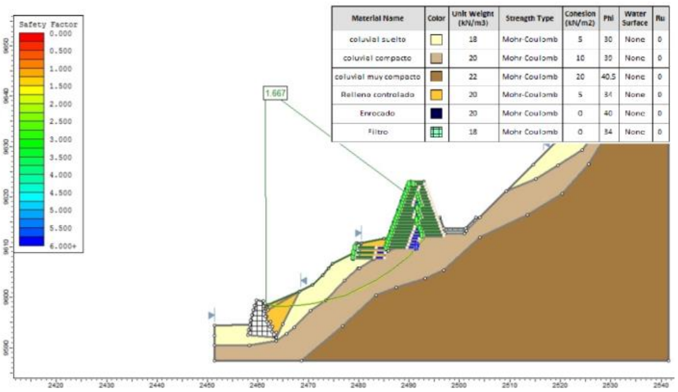

Figure 10. Static barrier stability analysis static condition (F.S 1.667)

For the modeling of the static barrier, the resistance parameters of a compacted filler that make up the static barrier were considered, as well as the tensile strength of 2 types of internal reinforcements, a 10x12cm double twisted wire mesh which provides a tensile strength of 37 KN / m and additionally the influence of a tensile strength of a geogrid of 86KN/m. This type of static barriers built with reinforced soil allows a structural stability of the system in addition to providing greater capacity for resistance levels of impact energies greater than 5,000 KJ as is the case.

The internal safety factor by limit equilibrium determined in static condition was 1,667 (Fig. 10) and in pseudo static condition of 1.24 for the most critical section. The calculated safety factors were greater than 1.5 in static condition, and for a pseudo static condition considering a seismic acceleration value of 0.17g the calculated safety factors were greater than 1.2.

From both results it was concluded that the structure is stable for this stability condition.

In Figure 11 and 12 shown the execution of static barriers with reinforced soil walls.

Figure 11. Execution of static barrier with soil reinforced wall

Figure 12. Execution of static barrier with soil reinforced wall

6 CONCLUSIONS

From this paper it can be seen that this type of structure is feasible to project considering design criteria compiled from tests performed on real-scale models, although in a small scale, but capable of absorbing energy levels of more than 5,000 KJ.

An analysis methodology is shown using basic rock fall criteria and combining them with limit equilibrium criteria.

It has been observed that the analysis by limit equilibrium is an appropriate calculation tool to determine the amount of reinforcement within the reinforced soil walls and very well complemented with numerical models that allow predicting possible deformations of the structure to the impact of a rock.

The reinforced soil protection solution is an optimal solution to receive rock fall impacts and also debris flow while maintaining a good structural capacity that ensures its long-term stability.

Continuous monitoring of these structures is always recommended after receiving the first rock impacts to measure the crater depths and deformations of the same structure to consider possible system repairs and maintain the same efficiency over time.

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