Stabilization of a rocky cliff by a geosynthetic reinforced earthfill

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ABSTRACT: The paper deals about criterions and methodology used to design the reinforcement of a rocky cliff by an earthfill reinforced with geosynthetics. The earth structure has been built along the left side of Cornappo creek (Nimis - Ud), near a bridge called "ponte degli Angeli", in Torlano village. The unstable cliff has a length of about 70 m and an height of 12 m - 13 m and induces critical safety conditions in a group of buildings locate just at the slope top. Conglomerates and puddingstones weakly cemented by carbonates that are easily dissolved by rill and underground water, by the seepage along the rock fractures and river floods form the rock mass. All these elements caused the cliff backward and the formation of rock cavities, 2 m - 3 m deep, with a 10 m thick rock layer above their roof. The aim of design was to stop instability phenomena development by realisation of an earthfill, characterised by a low environmental impact, to support the rock mass and to protect it from rill and underground water.

1 INTRODUCTION

The Cornappo creek, in the stretch downstream from the *Ponte degli Angeli* by the village of Torlano (Nimis – Udine), flows to the bottom of an escarpment, around 15 m deep, which, for a length of approximately 70 m, was progressively eroded both by ground waters and by the creek itself. Above the marnous and arenaceous rock on which the creek flows, there is a conglomeratic rocky mass and both stream water and ground water easily dissolve its weak carbonate cement.

Local failures occurred both as detachment of blocks, triggered chiefly by the sub-vertical cracking of the rocky mass, and as formation of large cavities, ranging in depth from 2 m to 3 m, occurred in the area in which ground waters formed temporary springs as they exited along the slope.

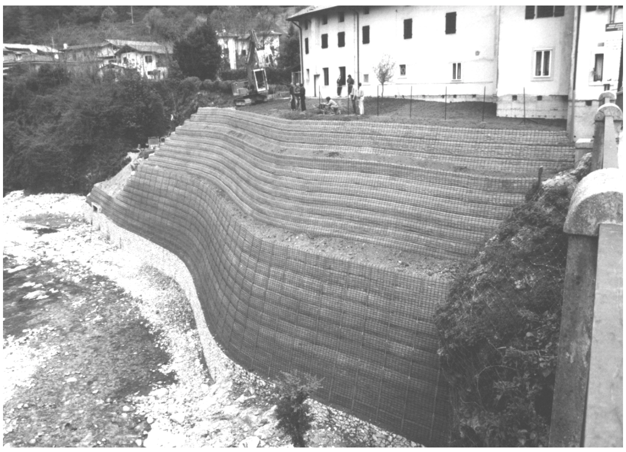

Figure 1. General view of reinforced earthfill.

To prevent that the uncontrolled evolution of these phenomena, namely the expansion of cavities inside the rocky mass and the collapse of the vaults, might give rise to landslides and destabilise the settlement situated on the top of the bank, some reinforcement works were made on the escarpment. These consisted of scaling blocks of rock, laying a metal wire net against the slope and building a reinforced earth structure (Fig.1). The earthfill has been built to fill the cavities of the rocky mass, support the overhangs and regulate the flow of underground and meteoric waters.

The article deals with the design criteria applied for reinforced earth project and the method used for construction

2 CHARACTERISTICS OF REINFORCED EARTHFILL

Owing to the narrowness of the gorge in which the creek flows, the reinforced earth slope was built in the stream bed and it was thus necessary to raise its base to a high enough to prevent it from flooding and from impacts of fragments carried by the stream.

2.1 *Concrete Bed*

The earthfill was built on a non-structural concrete bed, made with small river pebbles (Fig.2). Beside the concrete bed adjacent to the creek a partly reinforced concrete wall was built, clad in front with a stone façade, and embedded in the basal rock with a reinforced concrete strip.

The wall was anchored to the basal rock and to non-structural concrete basement respectively with steel bars and steel net sheets. The height of the wall, varying according to the height of the existing rocky outcrop, was designed to protect the lower layers of the reinforced earth relief against the erosive action of the creek's floods.

We only considered calculations affected by the weight of the reinforced earth slope, since the thrust of water, when the creek overflows, is not only temporary, but has a stabilising effect on the support structure. The horizontal tension exerted on the support base have been computed for the most adverse geometrical condition, when the maximum fill height can be attained at the same time as the maximum height of the reinforced earth slope resting on the earthfill.

Subsequently the horizontal pressures in two distinct areas, namely the pressure next to the existing rocky slope and the pressure next to the wall, were evaluated in this transverse section of the reinforced slope. In the areas of the concrete bed next to the rocky slope, we used the method based on the theory of elasticity, assuming, with some restraints, that it behaves as a homogeneous and isotropic half-space with an elastic-linear response.

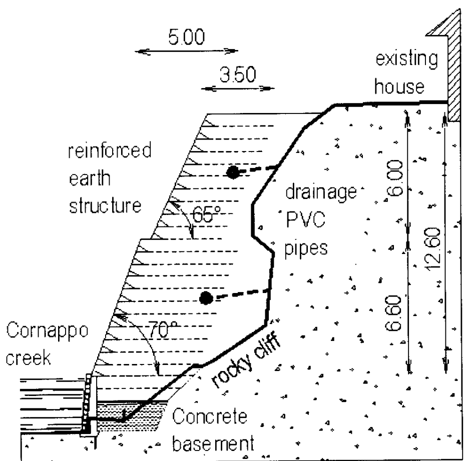

Figure 2. Schematic section of the reinforced earthfill

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Based on these hypotheses, we have thus used the schematic model reported in Fig. 6, with a tensional state defined by the following ratios (Jumikis 1971):

$$\sigma_x = \frac{p_{max}}{\pi \cdot B} \cdot \left\{ (x-B) \cdot \left[\tan^{-1}\left(\frac{x-B}{z}\right) - \tan^{-1}\left(\frac{x}{z}\right) \right] + z \cdot \ln \frac{x^2+z^2}{(x-B)^2+z^2} - \frac{B \cdot z \cdot x}{x^2+z^2} \right\} \quad (1)$$

This hypothetical calculation is quite reliable when dealing with tensions in areas that are far enough away from the reinforced concrete wall, since its very presence, as a rigid partition wall, significantly alters the tensional state.

If the calculated height of the reinforced earth slope equals H=12.3 m and the unit weight of the material used to make the artificial slope equals γ=20 kN/m^3, we can estimate a maximum contact pressure on the base structure of $p_{max}=H\cdot\ \gamma \cong 250$ kPa, varying linearly from this value, near the existing rocky slope, to a zero value near the wall, over an average support width B of around 5 m.

Based on the data outlined above, we conducted an elastic analysis up to a Z_{max}=1.4 m from the top level of the reinforced earth, in line with the maximum height that could be forecast for the obstruction, by considering 4 representative intercepts X1, X2, X3, X4, each respectively located 5 m, 1.5 m, 1 m, 0.5 m away from the axis used to measure p_{max}. The results of the analysis (Fig. 3) show an horizontal tensional state in the concrete support bed that is easily adsorbed by the non-structural concrete used to build it.

By contrast, in the concrete bed area next to the reinforced concrete wall is inappropriate to apply the method based on the theory of elasticity entirely, since, as mentioned above, the supposedly rigid limit in the elastic half-space alters the forces of thrust as a whole.

Therefore, we assessed the horizontal stress exerted by the reinforced earth resting on the base structure by first transforming the triangular load into concentrated loads, as shown in Fig. 5 and then applying the principle of overlapping effects to the following ratios (Terzaghi 1954):

$$\begin{aligned} \sigma_{xi} &= 1.27 \cdot \frac{Q_i}{Z_{max}} \cdot \frac{m_i^2 \cdot n}{\left(m_i^2 + n^2\right)^2} \quad \text{if} \quad m_i > 0.4 \\ \sigma_{xi} &= 0.203 \cdot \frac{Q_i}{Z_{max}} \cdot \frac{n}{\left(0.16 + n^2\right)^2} \quad \text{if} \quad m_i < 0.4 \end{aligned} \quad (2)$$

where the values used in the analysis are given in Table 1.

Table 1. pressures and forces values used in elastic analysis

N°	Abscissa Y_i [m]	$m_i = Y_i / H_{max}$ [-]	Pressure p_i [kPa]	Force Q_i [kN/m]
1	0.625	0.45	31.25	39.06
2	1.875	1.34	93.75	117.19
3	3.125	2.23	156.25	195.31
4	4.375	3.13	218.75	273.44

The results of the analysis (Fig. 4) show, in line with the observations made in the technical literature (Terzaghi 1954) that the expected stresses found in the concrete bed area located immediately at the rear of the reinforced concrete wall are approximately twice the value that could have been attained with the theory of elastic half-space.

The results of the latter are highlighted for comparative purposes as a bold line in Fig.3. For precaution, we felt it necessary to overlook the influence of the underlying non-structural concrete on the absorption of the thus defined horizontal stresses, and apply them entirely to the reinforced concrete wall and upper and lower steel anchors.

This system was statically assimilated to a single width beam, joined at the top by a steel net, at the bottom by steel bolts embedded in the bedrock. In order to check the structural elements defined above, we decided to make an linear approximation of the horizontal tensional state determined by the equations (2), (Fig. 4), and by then to compute the stress state and restraining reactions (Fig. 6).

On the basis of the calculated values, we performed analysis according to the usual methods of structural mechanics. These analyses indicated working stresses in agreement with allowable limits permitted by the current Italian regulations.

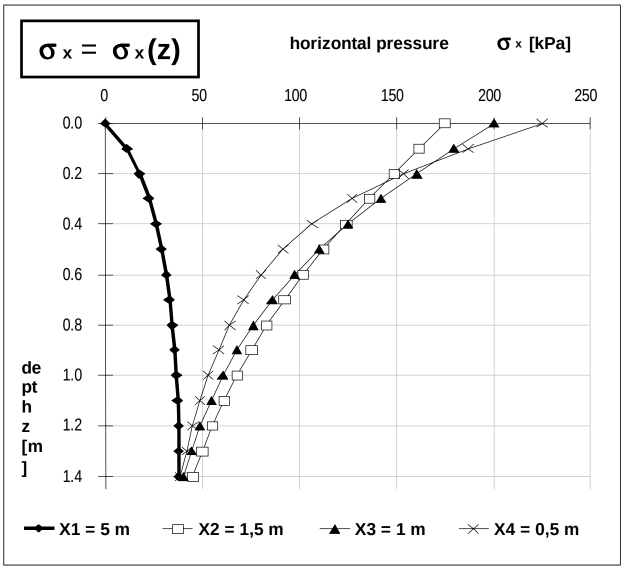

Figure 3. Horizontal pressures vs. depth in the concrete basement (rock side)

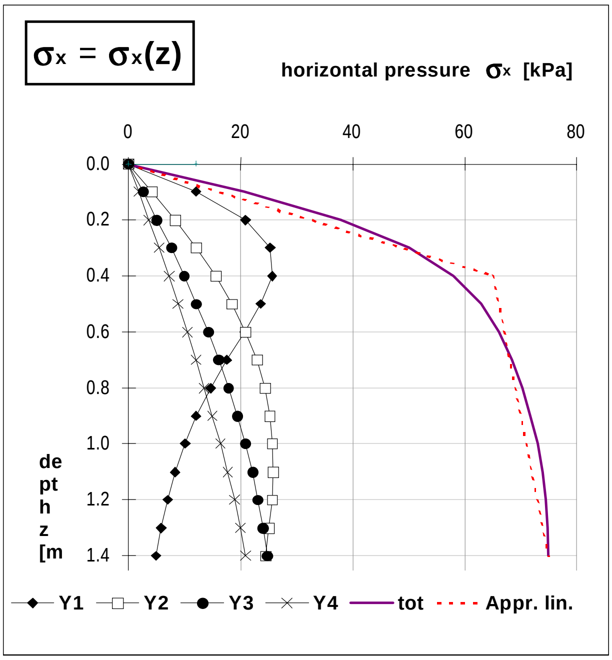

Figure 4. Horizontal pressures vs. depth in the concrete basement (creek side)

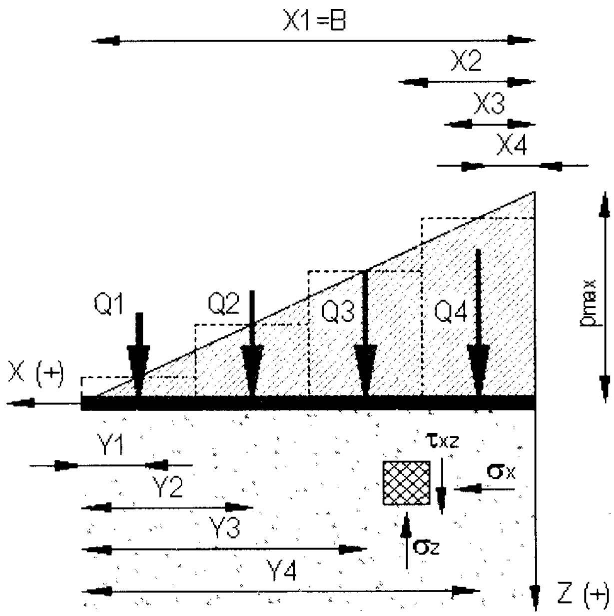

Figure 5. Transformation of triangular pressure in concentrated loads

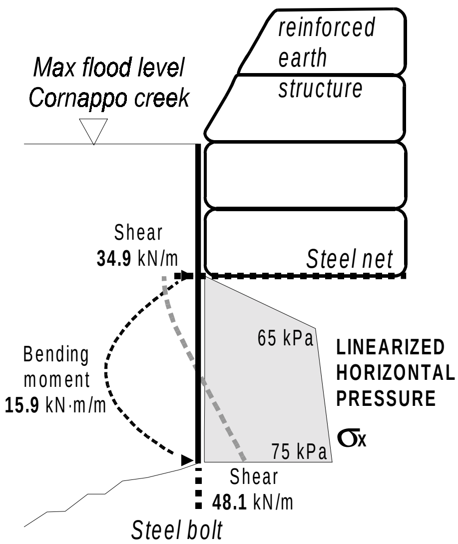

Figure 6. Schematic model used to design the concrete wall

2.2 *Reinforced Slope*

The reinforced earthfill used to support the rock mass is approximately 70 m long and, in the main section, around 40.0 m in length and extends to a maximum height of 12.6m, measured from the top level. To reduce the inclination of the structure, the upper part was set back by approximately 0.5m (Fig. 2). The differing exposure to environmental conditions and the analysis of the internal stresses, carried out as described in the next paragraph, prompted us to distinguish the type of reinforcement elements as well as their length and strength. As the creek overflows, the bottom of the reinforced slope may be periodically flooded and, as a result, two sheets of geonet were used as reinforcement elements, enclosed in bags with an overlap of 1.5 m, and each with a height of approximately 50 to 55cm (Fig. 6).

The geonet used consists of a continuous polyester yarns, coupled with an unwoven geosynthetic to prevent earthfill from leaking, with an ultimate tensile strength of T_{ult} = 200kN/ml. Above the last bag, a layer of granular soil, 10 to 20cm thick, was stretched to develop an adequate frictional strength along the base of the reinforced slope.

Then, 11 polyester geosynthetic nets, with a length of 5.0m and with a ultimate tensile strength of T_{ult} = 100kN/m for the first five sheets and a T_{ult} = 40kN/m for the others, were used to reinforce the bottom part of the earthfill, extending to a height of 6.60m. Each sheet, located on the vertical plane at intervals of 0.6m, was connected to an external retaining structure made with steel net sheets, shaped at an angle of 70° relative to the horizontal plane and reinforced with metal braces (Fig. 7). By contrast, 10 sheets with a length of 3.5m and a tensile strength of T_{ult} = 40kN/m were used for the 6.0m high top. The metal retaining structure was shaped at an angle of 65°. The surface of the rocky slope extends irregularly and when sheets with the length indicated by the calculation could not be used, they had to be anchored to the rocky mass by using metal bars with diameter measuring Φ = 8.0mm - 12.0mm and 0.50m spaced.

The soil used to build the soil structure belonged to the A1-a group of the AASHO classification, and was installed with a compaction percentage of 95% of the maximum Modified Proctor density. The cavity between the metal façade and the reinforcement sheets was filled with vegetable soil to encourage the development of vegetation and, to prevent it from eroding, a lighter geosynthetic sheet was laid on the external metal façade.

The reinforcement elements were designed by using the shear strength values given in Table 2.

Table 2. Shear strength parameters

	Earthfill	Rock
γ (kN/m^3)	19	25
c_d (kPa)	0	25
ϕ_d (°)	32 (at critical state)	38

Figure 7. Construction of first layers of reinforced earthfill. In foreground the reinforcing metal braces.

The shear strength parameters of the rocky mass were estimated by conducting a back-analysis of a small landslide, occurred in of the slope. A drainage system was installed inside the reinforced slope to remove meteoric and underground waters and prevent the growth of pore water pressures.

To quickly drain the waters that may seep through when the creek overflows, two micro-slotted PVC pipes were installed above the concrete bed, one against the internal façade of the concrete cladding and the other against the rock wall (Fig. 2).

By contrast, the ground water was gathered with micro-slotted pipes, located near the wall and spaced apart at vertical intervals of 3.5 m to 4.0 m.

3 DESIGN CRITERIA

The stability conditions of the reinforced earth structure and the reinforcement stress state were analysed for three different load conditions:

- short-term static load condition for the structure building stage, when it is subject to a high accidental overload due to the weight of the compactor;
- long-term static load condition, at the end of the structure's working life, when the strength and integrity of the geosynthetic materials are significantly reduced due to chemical and physical degradation and creep;
- seismic load condition at the end of the structure's working life.

The allowable tensile strength in the reinforcement sheets, T-ltds, was calculated with the method of the partial factors of safety (FoS) indicated by BS 8006 (1995):

$$T - \mathrm{ltds} = \frac{T_{ult}}{F_{id} \cdot F_{cd} \cdot F_{bd} \cdot F_{creep}} \tag{3}$$

where:

T_{ult} = ultimate geosynthetic strength;

F_{id} = FoS for damage during installation;

F_{cd} = FoS for chemical degradation;

F_{bd} = FoS for biological degradation;

F_{creep} = log(T) FoS for loss of strength under a long-term load (T=60 year life span of the structure).

In Table 3, are given the values of the safety factors used and the allowable strength values of the geosynthetic materials for each of the assumed load conditions.

Table 3. Allowable tensile strength of geosynthetic reinforcements

	SHORT TERM					LONG TERM				
T_{ult}	F_{id}	F_{cd}	F_{bd}	F_{creep}	T- ltds	F_{id}	F_{cd}	F_{bd}	F_{creep}	T- ltds
kN/m	-	-	-	-	kN/m	-	-	-	-	kN/m
100	1.3	1.0	1.0	1.0	76.9	1.3	1.0	1.1	1.78	39.3
40	1.3	1.0	1.0	1.0	30.8	1.3	1.0	1.1	1.78	15.7
200	1.3	1.0	1.0	1.0	154	1.3	1.0	1.2	1.78	72.0

A higher value of biological degradation coefficient was used for the foundation bags, since, after severe weather or after the creek overflowed, they may stay wet for relatively long periods. This condition encourages the formation of moulds or other micro-organisms that accelerate the degradation of the material. We thus decided to increase the safety factor for this phenomenon.

4 DESCRIPTION OF THE CALCULATION METHOD

4.1 *Direct Sliding Analysis*

The length of the base sheet, Lds, needed to prevent the horizontal sliding of the whole reinforced mass, was computed using the iterative two-wedge limit equilibrium method (Leshchinsky 1995, Leshchinsky 1997) to calculate the maximum active thrust value P. The frictional reaction extending along the interface of the geotextile was calculated as follows:

$$T_B = \left(N_B \cdot \tan\phi_d + c_d \cdot L_{ds}\right) \cdot C_{ds} \tag{4}$$

where N_B is the stress normal to the foundation level, ϕ_d and c_d are respectively the frictional angle and the cohesion of the foundation soil and C_{ds} = 0.8 the iteration coefficient between the geosynthetic and the foundation soil.

In the analysis, the seismic loads were considered by multiplying the weight of each block of the soil and the external loads for pseudostatic seismic coefficients K_h and K_v.

The used L_{ds} length, 5.0m, was sufficient to gain a value of the security factor against sliding, $FoS=T_B/P\cdot\cos\delta$ greater than 1,3 for static long term load condition (δ=inclination of active thrust P).

Yet, to increase the safety factor in seismic conditions, the deepest sheet was anchored to the rock wall.

4.2 *Deep-seated Analysis*

As the reinforced earth lies over a rigid concrete basement, the limit equilibrium stability analysis for failure surfaces extending out of the reinforced area and into the foundation soil has not been performed.

4.3 *Tieback Analysis*

The tensile stress, Tr_i, exerted in each reinforcement, was calculated by an iterative limit equilibrium analysis of a potentially unstable mass defined by logarithmic spiral failure surfaces extending into the reinforced mass (Leshchinsky 1995, Leshchinsky 1997).

The values calculated have been successively compared with the geosynthetic allowable strength, T-ltds, and the safety factor values, defined by T-ltds/Tr_i=FoS, calculated for the seismic load condition resulted generally greater than 1.3 and only for some of the deepest reinforcements were lower, FS = 1.1.

However, these values have been considered allowable in view of the very precautionary hypotheses assumed in the design, a seismic event at the end of the working life of the structure, estimated at 60 years ng into the reinforced mass (Leshchinsky 1995, Leshchinsky 1997).

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4.4 *Pullout Analysis*

The maximum pulling stress, T_i<= T-ltds, in each reinforcement was calculated assuming all possible collapse mechanisms to identify the "active" area from the "resistant" area, as suggested by

Leshchinsky (1995, 1997). The minimum anchoring length, L_{anc}, required to each reinforcement to prevent pullout, was thus calculated using the following equation:

$$L_{anc} = \frac{T_i \cdot FS_{pull}}{\sigma_i \cdot Cd \cdot \tan \phi_d + c_d} \tag{5}$$

where

FS_{pull}	= 1.5	pullout safety factor;
Cd	= 0.8	Reinforcement soil iteration coefficient;
σ_i	=	geostatic confinement pressure exerted on the reinforcement;
c_d	= 0	cohesion of the compacted soil;
ϕ_d	= 32°	frictional angle of the compacted soil in critical state conditions.

The length of the geosynthetic reinforcements was greater than the value indicated by the calculation to prevent any pullout.

Only the length of the two deepest sheets, 5.0 m, was inadequate and they were thus anchored to the rock wall.

5 FINAL REMARKS

The construction stage, three months long, ended in the month of May, 1998. In autumn of the same year, several flood phenomena occurred in Cornappo creek, and during one of this, the water level reached the first reinforcement layer of earthfill without producing serious damages except the grass cover removing.

Fig. 8 shows the present condition of left Cornappo creek bank, with restored slope after the end of reinforcement works.

Figure 8. A recent view of the reinforced earthfill (March, 2000)

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