Geogrid - reinforced embankments for landslides stabilization and failures rehabilitation

Kollios, A. & Migirou, M.
Edafomichaniki S.A., Athens, Greece

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ABSTRACT: In the context of construction of a new municipal road 3.5 kms long, serving the new dam of Aposelemis in the island of Crete, Greece, various failures of certain major cuts at 8 locations were observed, one year after initial alignment excavations. Those cuts were excavated in a phyllitic formation at steep inclinations of 63°-72° approximately, because of actual expropriations restrictions at the area, not allowing for smoother slopes, as geotechnically indicated. Because of the necessary imminent provision of the road to circulation, the implementation of additional major retaining structures was not possible (pile - walls with prestressed anchors) for failures rehabilitation, therefore a system of counter - weight embankments at the toe of each failed cut was proposed and designed. These stabilizing embankments were reinforced by geogrids with the wrap-around facing system to provide a steep external slope of 55°- 63° (depending on the local space availability). Polyester geogrids of a nominal tensile strength varying between 55 and 110 KN/m were adopted for embankment reinforcement and applied on site at 0.80 m spacing, associated with selected schist - origin backfill material adequately compacted. The maximum height of these embankments varied between 5 and 12 m. The rehabilitation system by geogrid - reinforced stabilizing embankments was proved to be very efficient, both technically and cost-effectively and provided the quickest possible (6 months) method of remediation to a problem that under normal conditions would need more than a year of site activities, if any rigid retaining structures were to be used for rehabilitation.

1 PROJECT DESCRIPTION - GEOTECHNICAL INFORMATION

During the construction period of the new municipal road (spring 2008) accessing the Aposelemis Dam in the island of Crete, Greece, various failures were observed at 8 locations of cuts earthworks along the 3.5 km of the alignment. Those cuts were initially excavated according to the general guidelines as provided by a rough geological design and preliminary stability assessment (December 2006). The excavation slopes were generally steep (2:1, v:h, 63°), performed within a phyllitic rock that locally presented pockets and lenses of poorer layers, disintegrated into sand-gravel. The initial geotechnical observations indicated mainly superficial and shallow, circular-like type, time-progressive defoliations and failures, whenever schistosity was favorable to the slope, locally originated by the presence of subvertical tension cracks.

These local failures generally extended up to the foot of the excavated cuts, therefore the risk of generalised failure was imminent, if no stabilization measures were to be provided.


Figure 1 : Observed failures

Because of the existing expropriations restrictions at the area, generally smoother slopes to reconfort stability conditions could not be applied in most of these cases and only a small available space at the toe of the cuts could be used for counter-weight stabilization of this part. The proposed concept for stabilization involved the construction of a steep (45-63° external slope) geogrids reinforced counter-weight embankment at each location, with a height covering at least half of the actual total height of the cut. In addition to the proposed stabilization, the remaining upper part of each initial cut should be further stabilized by a rock face covering system by steel-wire ring nets fixed on the slope by rock or soil nails placed at an adequate network with L = 3.0 - 6.0 - 9.0 m length.

An extended geological study including analytical mapping of existing cuts and records of discontinuities was combined to the contractual geotechnical information with boreholes and adequate laboratory testing, mainly to assess the phyllite shear behaviour along the critical schistosity planes. All failed slopes were back-analyzed and a residual shear strength φ_r was also determined for each case. According to the geotechnical evaluation of available data, the phyllitic formation of the area presented the following critical geotechnical design parameters:

Rock density : $\gamma = 24$ Mg/m^3

Angle of internal friction : $\varphi = 30°$

Cohesion c = 50 KPa

Deformation Modulus E = 220-260 MPa

Intact Rock Strength $\sigma_c = 20 \pm 5$ MPa

The typical failure mechanism depicted in the majority of the failed cuts involves initially a wedge-type small scale failure within the unfavorable combination of critical discontinuity planes on the surface of the excavated slope and sub vertical tension cracks that progressively extend to create a potential circular-like failure of the whole slope.

2 CONCEPT AND DESIGN OF THE REINFORCED EMBANKMENTS

The contractual obligations for the new municipal road to be provided to circulation within practically one year and the difficulty to provide any rigid retaining structure (e.g. anchored wall) at the toe of the failed cuts in time, directed to the proposed construction of adequate toe stabilizing counter weights by geogrid - reinforced, steep slope embankments with adequate facing using the wrap - around system.

A maximum height of 12 m for the stabilizing embankment and a variable width of 6.00-8.00 m for the geogrid reinforcement were selected during the conception design phase and this proposal was immediately approved by the Client for final geotechnical design.

The maximum external slope of the reinforced embankment was 2:1 (v:h, 63°) and the selected type of reinforcement refers to polyester geogrids of types 55/30-20, 80/30-20 and 110/30-20 with a nominal tensile strength of 55 - 80 - 110 KN/m accordingly.

The final geotechnical design was performed under the following assumptions:

a) The reinforced backfill material for the stabilizing embankment consisted of either selected crushed limestone quarry product or in-situ excavated and treated schist origin rock, according to the relative specifications, with a maximum stone diameter of 20 cm and a maximum fines percentage of 10%.

b) Regularly distanced geogrid reinforcing layers were set at 0.80 m.

c) The use of the limit equilibrium analysis method was implemented by the computer code Ressa V.2.0, slightly modified to incorporate the necessary additionally requested by the Greek Specifications partial safety factor of the material γ_M (as per Eurocode EC-7). According to the imposed loading conditions, the following minimum safety factors were adopted for the design:

Table 1. Requested minimum safety factors

No	Loading conditions	Shear strength parameters	Requested safety factor
1	Long - Term, Static Loading, Maximal Ground Water Level of 50 years period	Effective (φ', c')	1.30
2	Long - Term with Seismic Loads	Effective (φ', c')	1.00

Three possible failure mechanisms were checked during the design: External (global) stability (ES), Internal stability (IS) and Compound stability (CS).

Because the nominal tensile strength of the reinforcing geogrid is variable with loading time and temperature, the design long term tensile strength T_d and the design short term tensile strength $T_{d,s}$ are provided as:

$$T_d = T_k / \gamma_M \quad (1)$$

$$T_{d,s} = T_{k,s} / \gamma_M \quad (2)$$

where, $$T_k = T_{ult} / f_m \times f_e \times f_d \times f_{cr} \quad (3)$$

$$T_{k,s} = T_{ult} / f_m \times f_e \times f_d \quad (4)$$

In the above equations T_k and $T_{k,s}$ are the characteristic long-term and short-term tensile strength respectively while T_{ult} is the nominal tensile strength (production license) and γ_M is a reduction coefficient due to the material ($\gamma_M = 1.20$). The "f" coefficients are described as follows: f_m partial safety factor against manufacture deviations of production (f_m = 1.10), f_e partial safety factor against environmental degradation and chemical impact (f_e = 1.06), f_d partial safety factor against installation damage (f_d =1.06), f_{cr} partial safety factor against creep behavior (f_{cr} = 1.67). Factors $f_m \times f_e$ were introduced in the calculations as durability partial safety factor.

An additional reduction factor of the design tensile strength was also taken into account, to cover the pull-out resistance of the reinforcement (γ_{pu} = 1.50). During earthquake action (Loading Case 2), the creep partial safety factor was considered as f_{cr} = 1.00.

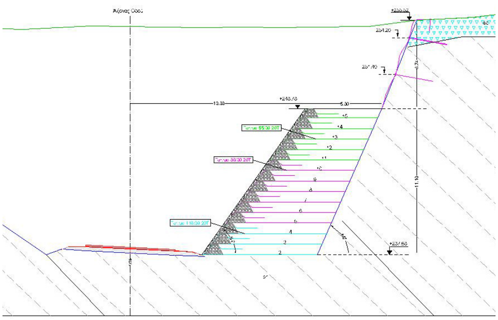

Figure 2 : Reinforced toe embankment design principles

3 STABILITY CONSIDERATIONS

The determination of the overall safety factor against stability of each reinforced embankment stabilising the toe of the relative failed cut was obtained by the following relation (5), where the reinforcing effect of the geogrid tensile strength is implemented in the denominator to reduce the sliding moment M_D :

$$F_s = M_R / (M_D - T_s \times R) \qquad (5)$$

with M_R (φ, c, R, T_s) sum of stabilizing moments, M_D sum of sliding moments, T_s sum of reinforcing tensile strength (Stabilizing), R distance of T_s to the sliding cycle center point.

The direction of the stabilizing force of each geogrid layer was considered as horizontal, in relation to the intersection point of the critical failure cycle with the reinforcing elements.

Stability calculations were performed for each stabilizing reinforced embankment with the above mentioned loading conditions (Table 1) and the calculated overall safety factors are recapitulated in the following Table 2.

Table 2. Safety factors of reinforced embankments

Toe stabilising reinforcement embankment (case)	Embankment geometry	Reinforcement	Analysis method	Safety factors	
				Static loading sf_{min}=1,30	Seismic loading sf_{min}=1,00
L1/L2/L6/L7	H=6,50-10,00m B=6,90-8,60m L=60-140m i=45-63°	55/30-20 L=5,50-7,00m Layers:8-12	Comprehensive Bishop (Es)-Circular	1,40-1,97	1,22-1,58
			Spencer (IS)-Two-part wedge	1,35-1,75	1,15-1,49
			Spencer (CS)-Three part wedge	1,33-1,80	1,05-1,53
L3/L5/L8	H=5,20-12,00m B=7,50-13,10m L=53-120m i=63°	80/30-20 55/30-20 L=6,50-8,00m Layers:13-15	Comprehensive Bishop (Es)-Circular	1,33-1,36	1,18-1,22
			Spencer (IS)-Two-part wedge	1,30-1,31	1,10-1,13
			Spencer (CS)-Three part wedge	1,30-1,39	1,10-1,16
L4	H=12,00m B=7,90m L=100m i=63°	110/30-20 80/30-20 55/30-20 L=6,50/7,50/7,90m Layers:15	Comprehensive Bishop (Es)-Circular	1,40	1,17
			Spencer (IS)-Two-part wedge	1,30	1,04
			Spencer (CS)-Three part wedge	1,33	1,20

4 CONSTRUCTION PERFORMANCE

The main concern of the final geotechnical design was the limitation of post-construction translational displacements of the top edge and the foot of the wall. These deformations were attentively recorded by the Contractor Survey Department on a regular weekly basis and the relative observation curves (displacement - time) were drawn for each reinforced embankment. Practically only minor vertical displacements of the order of 20 mm were recorded, mostly attributed to the self settlement of each reinforced embankment, within the first two weeks upon the end of earthworks activities.

Certain difficulties were presented during construction, mainly due to the facing protection system selected on site, since this system involved the use of an additional inclined fencing steel grid to stabilise the outer part and protect the wrap-around- system of the geogrids.

The use of rather coarse - grained schist excavated products with a maximum grain size of D = 200 mm, practically without any fines content, allowed the filling of this external area to the slope by simple gravity, without any special compaction application.

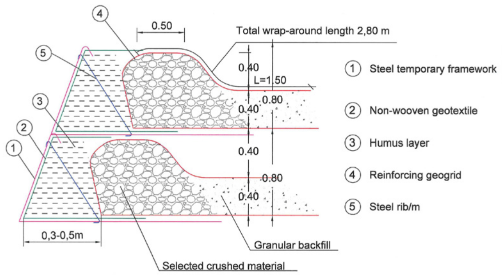

Figure 3 : Applied facing system

Local failures during construction of the outer protection layer in steeper slopes (63°) were remediated by rebuilding this part of the failed surface, occasionally by means of a cementing grout under a low pressure, to create the necessary "pseudo-cohesion" within the coarse aggregates of the reinforced body.

Remains of this slurry grout presented locally at the outer face of the protection facing were immediately cleaned up manually to preserve the correct appearance of the final rocky aspect of the facing.

Figure 4 : Final aspect of stability toe embankment

5 CONCLUSIONS

Geogrid reinforced embankments acting as counterweight stabilising the foot of a series of failed cuts was proved to be a technically adequate, cost-effective and time-saving solution to rehabilitate the observed failures. It allowed a minor increase of existing expropriations, which offered a major advantage for the progress of the project, taking into account the very slow and bureauctaric arrangements issued in similar cases. It also offered a 35% reduction of the cost, compared to any permanent anchored pile-wall and allowed the implementation of all rehabilitating structures within the strict time-schedule for delivering the road to circulation. A total amount of approximately 75000 m^2 of reinforcing geogrids was supplied on site, with the corresponding cost totally counter-balanced by the above stated very important benefits, following a correct conceptual and detailed design during the project construction period.

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