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Design and construction of a high embankment with siltstone excavated materials reinforced by geogrids

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ABSTRACT: The construction of a high embankment, reinforced by geogrids, using unsuitable for backfilling materials of siltstone origin presented an important cost reduction compared to the initially planned bridge by 25%. Adequate design techniques associated with careful construction provided a rather cost - effective engineering solution. In addition, the use of fine - grained materials for embankments construction is in certain cases feasible and should not be generally rejected by specification, without seriously considering the related cost implication of earth - moving activities and the relative environmental impact.

1 INTRODUCTION

Within the context of realization of a major highway crossing the north mountainous area of Greece, the construction of important earthworks (cuts and embankments) in a difficult morphological relief, was necessarily associated to the design of extensive and rather costly technical works (tunnels, cut and covers, bridges). In order to provide a guide to reduce cost, the replacement of one bridge situated between two tunnels by an embankment reinforced by geogrids was proposed. In this way the use of materials excavated by the adjacent tunnels could balance the earthworks of the specific part. The maximum height of the new reinforced embankment to replace the bridge was 29 m and it had to be built over a steep relief.

The geotechnical design challenge for the reinforced embankment was rather high:

- The outer slope of the embankment had to be designed as variable (between 1:2 and gradually 2:3 till 1:1).
- Due to the morphology, a reinforced concrete cantilever wall was necessary at the toe of the steeper slopes for a length of approximately 30 m at the foot of the new embankment.
- The foundation subsoil was a satured flysch formation, consisting mainly of siltstones with thinner sandstones beds, covered by a superficial scree layer of 5.0 m thickness. This soil layer presented locally evidence of creep.
- The backfill material provided on site mainly consisted of siltstone fragments, with a clayey - silty nature. It was a soil-rock mixture of flysch origin with weathering grade III to IV. This material could not be characterized as “selected granular backfill”, as this was contractually imposed for the construction of embankments. To overcome this difficulty, the client had to be convinced that the association of geogrid reinforcement within the clayey - silty body of the embankment would provide the necessary long-term performance of the new embankment. To overcome construction problems an outer zone of selected granular backfill of 3,00 m width was proposed to be placed to protect the fine-grained reinforced embankment body against any secondary erosion effects.

2 DESIGN METHODOLOGY

The design methodology involved the following steps:

2.1 *Investigations*

A complete geological and geotechnical investigation on site was carried out to determine the geological-hydrogeological background and the bearing capacity of the subsoil. The area presents a hilly relief consisting of flysch formations, with natural slopes inclined 25°-30° northwards, but locally, due to a torrent erosion, some slopes were rather steep (up to 45°). The flysch bedrock consisted mainly of alternations of siltstones with fine beds of sandstone, overlied by a scree soil cover, with local instabilities due to the water action.

2.2 *Laboratory testing*

Additional laboratory tests were performed on the excavated materials from the two neighbouring tunnels, intended to be used for the new reinforced embankment. Mainly siltstone fragments and weathered debris were tested to determine the effective (by drained shear tests) and the residual friction angle (by ring shear tests) as well as the corresponding cohesion values.

All samples were precompacted at 95% of their maximum Proctor density, simulating the site compaction conditions. The effective and residual shear strength parameters determined for the fine-grained siltstone debris as well as the selected design values are recapitulated at Table 1.

Table 1. Shear strength parameters for siltstone-debris backfill.

	Effective	Residual	Design
Angle of friction $\phi'(°)$	24-35	17-22	28
Cohesion $c'(kN/m^2)$	5-18	0	0

2.3 *Stability analysis of various cross-sections*

Due to the special morphology, various sections were selected for stability analysis, each one presenting different parameters. These parameters were the outer inclination (ranging from smoother 1:2 to steeper 1:1), the height, the ground water influence (local artesianism was present) and the possible presence of cut section over the upstream area. Initial stability calculations of each section were performed for the unreinforced case and the corresponding safety factors were proved to be unacceptable. Therefore, geogrid reinforcement was then introduced by the nominal tensile strength, variable between 40 kN/m and 180 kN/m. For the correct long-term performance of the geogrids, the necessary partial safety factors, as implied by the BBA certificate for the related geogrid type, were taken into account (Table 2).

Table 2. Partial safety factors for geogrid design.

Nominal tensile strength (KN/m)	40	60	90	120	180
Installation damage factor	1.12	1.12	1.12	1.12	1.12
Durability factor	1.10	1.10	1.10	1.10	1.10
Creep factor	1.67	1.67	1.67	1.67	1.67
Direct sliding coefficient	0.80	0.80	0.80	0.80	0.80
Geogrid long term tensile strength (kN/m)	19.44	29.16	43.74	58.32	87.69

Primary geogrid reinforcement was placed at the maximum vertical allowable distance of 1,0 m and secondary reinforcement (with 40 kN/m nominal strength) was placed in-between every 0,50 m. After design optimisation, primary reinforcement lengths varied between 8 m (at higher elevations) and 32 m (near the basis of the embankment). All reinforcing layers reached the free slope without using any wrap-around technique. Secondary reinforcement length was selected to be 2,00 m each.

All stability analysis calculations were based on the limit equilibrium method using the appropriate computer code (Leschchinsky, 1995). Stability calculations were performed for rotational failure mode (generalized Bishop), both internal and external, as well as translational analysis along each geogrid reinforcing layer (two-part wedge mechanism). Additional calculations were also performed concerning failure along a mixed interface, using the three-part wedge mechanism enveloping the reinforced section under analysis. The results of all stability analysis are recapitulated, in terms of obtained safety factors, into the following Table 3.

Table 3. Calculated safety factors.

Cross section sliding) (Slope)	Load combination	Safety factors: Rotational sliding, No reinforcement	Rotational sliding, Geogrids reinforcement	2-Part Wedge failure, No reinforcement	2-Part Wedge failure, Geogrids reinforcement	3-Part Wedge failure (Deep sliding), No reinforcement	3-Part Wedge failure (Deep sliding), Geogrids reinforcement
1186	(1) – (g + q)	1.12 < 1.40	1.48	1.21 < 1.30	1.49	1.21 < 1.40	1.57
(1:2)	(2) – (g + q + w)	…	1.03	…	1.01	…	1.11
	(3) – (g + 0.5q + s)	…	1.31	…	1.37	…	1.50
1188	(1) – (g + q)	0.87 < 1.40	1.48	1.02 < 1.30	1.40	1.12 < 1.40	1.43
(2:3)	(2) – (g + q + w)	…	1.03	…	1.03	…	1.03
	(3) – (g + 0.5q + s)	…	1.35	…	1.32	…	1.35
1190	(1) – (g + q)	0.77 < 1.40	1.47	0.89 < 1.30	1.33	0.99 < 1.40	1.42
(1:1.25)	(3) – (g + 0.5q + s)	…	1.02	…	1.00	…	1.04
1194 (1:1)	(1) – (g + q)	0.70 < 1.40	1.46	0.86 < 1.30	1.33	0.99 < 1.40	1.43
	(3) – (g + 0.5q + s)	…	1.02	…	1.00	…	1.04
H = 5 m	(1) – (g + q)	1.49 > 1.40	…	1.72 > 1.30	…	1.61 > 1.40	…
(1:2)	(3) – (g + 0.5q + s)	1.04 > 1.00	…	1.24 > 1.00	…	1.10 > 1.00	…
H = 10 m	(1) – (g + q)	1.16 < 1.40	1.58	1.30	1.47	1.29 < 1.40	1.54
(1:2)	(3) – (g + 0.5q + s)	…	1.07	…	1.01	…	1.09
H = 15 m	(1) – (g + q)	1.18 < 1.40	1.54	1.37	1.46	1.26 < 1.40	1.56
(1:2)	(3) – (g + 0.5q + s)	…	1.05	…	1.00	…	1.08

g : permanent loads, q : mobile loads, w : water loads, s : earthquake

3 CONSTRUCTION

During construction, major difficulties had to be overcome:

- The compaction of siltstone debris produced by the excavation of neighbor tunnels (fine grained material) was mostly realized by sheep-foot rollers, similar to the compaction method and specifications imposed for the clayey cores of typical earth dams. Inspection of the obtained compaction degree was rather strict and it has often been necessary to partially remove layers not adequately compacted, due to higher natural moisture content than the optimum one. Normally 7-8 passes of the roller per layer were proved necessary to obtain prescribed compaction degree, but continuous rainy weather created additional delays.
- The outer zones of the embankment slopes with 3,00 m width were constructed by selected granular backfill, to avoid future possible degradations and to create a "stronger" superficial cover of the silty slope. The geogrids reinforcement ended freely within the selected backfill (without any wrap-around). The same necessary compaction degree was imposed for the granular backfill as for the clayey body (95% of Modified Proctor density). This fact increased somehow the necessary anchoring length of each reinforcement layer, but allowed a much quicker construction procedure, since the compacting rollers could reach the edge of the embankment with full compaction energy.
- The variable inclination of the embankment slope created a certain curvature of the long section of the reinforced embankment, therefore important overlaps of the geogrids were proved necessary to form each horizontal layer. The following Table 4, for the most unfavorable embankment section, presents a typical estimated quantity for each type of geogrid implied in the design, as finally it was constructed.

Table 4. Reinforcement geogrids details.

No of layers	Nominal tensile strength (kN/m)	Reinforcement Role	Length (m)	Total Area (m^2/m)
4	40	Primary	8	32
4	60	Primary	16	64
5	90	Primary	23	179
5	120	Primary	26	130
5	180	Primary	29	145
5	180	Primary	32	160
28	40	Secondary	2	56

- Because of the rather rough relief, the reinforced embankment toe for a length of approximately 30 m was stabilized by a cantilever reinforced concrete wall of a maximum height of 8,00 m. In this way the construction of a girder box culvert influencing the foot of the embankment was feasible.
- The typical as-built section of the embankment body is presented at Figure 1, with the properties of each reinforcing layer (nominal tensile strength and effective length).

4 PERFORMANCE

The efficiency of the design method and the adopted construction methodology of the reinforced embankment, twelve months after the end of construction, was demonstrated by deformation measures (survey and settlement plates).

Settlement plates placed within the embankment body every 5 m of height, were monitored daily during construction and weekly after the end of earth works till stabilisation.

In addition, topographical records of selected points on the highest surface were daily taken and compared.

The finally recorded total self-settlement of the embankment was only 75 mm at the axis of the highest

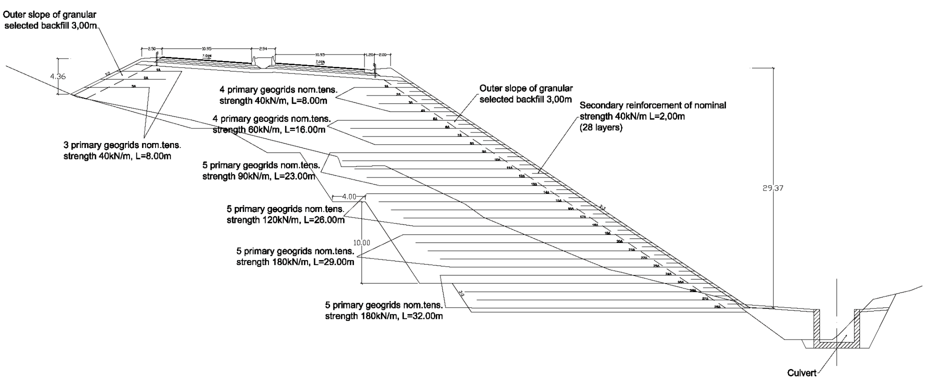

Figure 1. Typical as-built section.

section, stabilized after 2 months following the end of earth works. Asphalt pavement was then allowed to be placed. For this part, design estimated self-settlements were higher, at the order of 160 mm, proving the correct performance of the design and construction methodology.

5 CONCLUSION

The use of inadequate, fine-grained material for the construction of high embankments was proved to be feasible, by means of adequate geosynthetic reinforcement. A detailed design and inspection of the construction methodology must be assured so that deformation of the embankment stays at low levels. This method offers important advantages to the direction of cost-reduction (an overall cut of 25% of the initial bridge project budget was observed finally) as well as to the respect of existing environmental rules, since it allows small earth-moving operations at rather short distances. At the same time, it avoids the reclaim and excavation of additional borrow areas and the use of special deposition areas for inadequate clayey materials. Taking into account that well known specific design and construction guidelines ask for selected coarse grained backfill material to be associated with reinforcing geosynthetics (e.g. HA68/94 of U.K.), it is suggested to examine the possibility of certain modifications. These modifications should be to the direction of allowing also the use of fine-grained materials adequately designed to be reinforced by geosynthetics, as actually under issue in Greece.

The secondary consolidation settlements of such embankments can also be accommodated in the future by using geosynthetics technology (e.g. vertical and horizontal inclusions for consolidation acceleration and internal drainage). In this way, construction of such an embankment may be speeded up, respecting also the demands of the practical quality control (controlling time dependent viscous behaviour of the clayey embankment body and creep induced excess pore pressures).

Further research has to be performed on large site shear-box type tests as to determine the best combination of the type of geosynthetic to be associated to each specific case of poor quality (clayey type) backfill material. Within this scope, a scientific research program is actually proposed in Greece, to determine the effects of a given earthquake's spectrum on a reinforced embankment, by means of a large scale model placed on a vibrating table of the Athens National Technical University. Deformations on the geosynthetic reinforcement layers will be monitored also by optical fibers.

REFERENCES

Greenwood, J.R., Holt D.A. and Herrick, G.W. 1985 "Shallow slips in highway embankments constructed of over consolidated clay" - Symposium of Failures in Earthworks, London, pp. 79-92.

Leschinsky, D., Ling, H. and Hanks, G. 1995. Unified Design approach to geosynthetic-reinforced steep slopes and segmental walls. Geosynthetics International, Vol. 2, No 5, p. 845-881.

LI, A.L. 2000. "Time dependent behaviour of reinforced embankments on soft foundations". PhD Thesis, University of Ontario, Canada.

Love J. and Milligan G. 1995. "Reinforced soil and soil nailing for slopes - Advice Note HA 68/94", Ground Engineering Technical Note, p. 32-36, May 1995.

Kollios, A. 2005. "Geotechnical design of reinforced embankment to replace bridge T7 at part 2.4 of Egnatia Highway" - Edafomichaniki S.A.

NCMA, 1996. "Design manual for segmental retaining walls - Backfill selection criteria", Second Edition, National Concrete Masanry Association, Herndon, USA.

RESSA V. 2.0, update 18, 2005. Adama Engineering Inc., Newyark Delaware.