# DESIGN AND CONSTRUCTION OF A HIGH EMBANKMENT PARTLY REINFORCED BY GEOSYNTHETICS

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ABSTRACT: An important reduction of the construction cost was obtained after a detailed geotechnical investigation and design, replacing the initially planned bridge at a subpart of the South Motorway Axis of Creta by a 20 m high embankment, partly reinforced by geosynthetics. The critical loading stage to determine the number, the elevation and the length of the necessary reinforcement refered to the high earthquake loads anticipated by the existing regulations for the specific site and the total project was realised within only 6 months after the final approval of the proposed geotechnical design.

## 1 INTRODUCTION

In the context of a very tight time - schedule for the rehabilitation of the existing road network and especially the main South Motorway axis at the island of Creta, it was decided to investigate any possibilities for quicker construction and budget reduction that might be obtained by severe changes on the approved structures. As such, a high bridge of approximately 100 m length was replaced by a partly geosynthetics reinforced embankment, following an extensive geotechnical investigation and related design. The prevailing criteria for the design of the new embankment involved :

- Minimum additional expropriations.
- Extensive down slope of the embankment at 2:3 (34°) inclination with specific anti-erosion vegetal ground.
- Use of site neighboring cuts excavated material with the least possible manipulation to obtain necessary grain size.
- Minimum displacement during the Design Maximum Earthquake (not exceeding 25 mm).
- Peculiar morphological relief related to the presence of a large culvert 5,0 x 4,0 m at an angle of 60° to the alignment axis.

## 2 DESIGN CONSIDERATIONS

The detailed geotechnical investigation consisted of 3 sampling boreholes of 20 - 22 m depth each, site permeability tests on the rocky substratum, geotechnical modeling of the soil - embankment mechanical behaviour, assesment of geotechnical design properties and design of all embankment sections for the necessary dimensioning of stability.

The subsoil consisted mainly of a marly limestone intensively fractured and locally desintegrated, with an RQD selected conservative value of 38%, locally covered by a soil type mantle of erosion characterised as clayey sand with gravels at 1,00 to 3,5 m thickness. The geotechnical design parameters of those formations are recapitulated within the following Table 1.

Table 1 Geotechnical Design Parameters

| Superficial Mantle (SC) | Marly Limestone (m) (Hoek-Brown Failure Criterion) |
|---|---|
| $\varphi' = 33°$ | $q_u$ = 13,2 MPa (UCS) |
| c' = 0 | $I_{s50}$ = 1,18 MPa (Point Load strength) |
| $\gamma$ = 22 KN/m$^3$ | $\sigma_{ci}$ = 30 MPa (Intact Rock) |
| E = 24300 KPa | GSI = 20 ÷ 30 (Classification) |
| | $m_i = 8 \pm 2$ |
| | D = 0,70 (Disturbance) |
| | $\varphi' = 14°$ |
| | c' = 0,56 MPa |
| | $\gamma$ = 21 KN/m$^3$ |

For the reinforced embankment body, the marly limestone fragments of excavated cuts on both side of the new embankment had to be used. Because of the design considerations and for maximum thickness layers of 50 cm, the maximum stone diameter should not exceed 25 - 30 cm, therefore the explosions for excavations of neighbouring cuts had to be designed in such a way to provide for the adequate material. The estimation of conservative values of shear parameters for stability design was :

Effective friction angle $\varphi' = 35°$
Effective cohesion c' = 5 KPa
Bulk Density $\gamma$ = 20 KN/m$^3$
Modulus of Compressibility $E_s$ = 50 MPa (compacted).

The area sismicity was rather high and according to the Greek Code of Earthquake, a coefficient of a = 0,21 g was considered. The design traffic surcharge load was considered as p = 20 KPa. No water action was consi-dered to act on the embankment. Preliminary stability calculations using the simplified Bishop's theory proved that internal stability of the 2:3 long down slope of the 20 m high embankment was not satisfactory (necessary safety factors $F_s$ = 1,40 without earthquake and 1,00 during earthquake were both not fulfilled). Because any reduction of the down slope to achieve those factors was financially not acceptable (expensive expropriations - longer culvert), a progressive search of the optimum number of geosynthetic reinforcement of the lower half part of the embankment was performed. This analysis included checking of internal and external stability as well as overal site stability. The computer codes Ressa V.2 together with SLOPE-W for checking total lenghts of anchorage were employed to check internal stability of the reinforced embankment for :

- Tension analysis (checking the geosynthetic's maximum tension at each elevation).
- Wedge analysis (2-part wedge mechanism checking sliding conditions at each elevation - Figure 1).
- Anchorage length of each layer behind the slip plane.

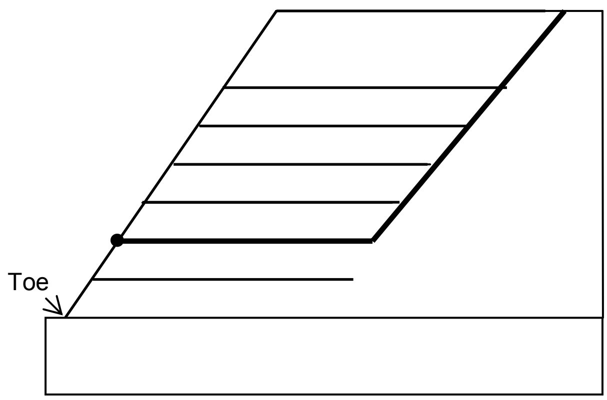


Figure 1 Two -Part Wedge Mechanism

An additional check of the total stability of the embankment was also performed according to the 3-Part wedge mechanism as shown in next Figure 2.

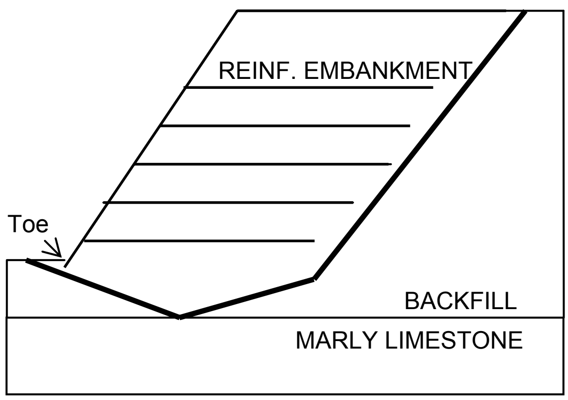


Figure 2 Three-Part Wedge Mechanism

The vertical spacing between primary geosynthetics layers was initially selected to be at the maximum allowable spacing of 1,0 m. Totally 9 successive layers were proved necessary, seven of which with a length of 9,0 m and the upper two with a length of 12,0 m (up to 60% of the total height of the embankment). It should be noted that those lengths were imposed for the design by the fact that there were no plans for facing or wrap-around technique to be followed for the critical end point on the slope surface.

The geosynthetic selected was a woven geotextile of nominal strength of 180 KN/m on both directions. Taking into consideration a total safety factor of 1,75 (1,67 for creep and 1,05 for installation damage), the working strength of the geosynthetic for the specific design was 115 KN/m. The final overal safety factors checking total stability were :

No earthquake action F1 = 1,52 > 1,40

Earthquake action (a = 0,21 g)   F2 = 1,05 > 1,00

Because of the presence of the oblique culvert, all reinforcing elements had to be placed above the culvert top elevation.

The total reinforcement length was 87 m per linear meter of the embankment for the primary geosynthetics of the conceptual design.

It should be noted that critical stability calculations were the ones considering earthquake action, based on which, the final lengths were selected. According to the Newmark theory, total movement of the embankment toe due to earthquake action was calculated at the limiting value of 20 mm approximately.

The settlements calculations proved an average self-settlement of the embankment of 55 mm, while the "soil" foundation settlement was calculated at 33 mm.

## 3 CONSTRUCTION DETAILS

The typical section corresponding to the initial design is presented at figure 3 further below, as it was incorporated in the tender documents.

During construction, a major variation of the initial design took place, following the combined proposals of the Designer and the Supervisor according to which :

a) Intermediate (secondary layers) of an 80 KN nominal strength geogrid were added to the primary reinforcement in-between the 1,00 m spacing with a length of 2,00 m each.

b) Although the initially designed slope was not steep (2:3), the wrap-around technique of primary reinforcement was also adopted, with an additional embedded length of 1,50 m.

Those modifications were necessarily imposed since the quality of the available on site embankment material did not correspond to the initially designed one, presenting a fines percentage of approximately 8 - 15 % and a relatively higher plasticity index. As such, the initially designed section was transformed to the one of figure 4, allowing thus the use of a minor quality material of excavation of the neighboring cuts. Nevertheless, some indicative laboratory tests on a large shear box (25x40 cm) using the available excavated material presented the following range of shear parameters:

Angle of internal friction $\varphi_r$ = 28° - 41°.

Cohesion $c_d$ = 10 - 38 KPa

Those results prove that the conceptual design of the project was rather conservative and that the higher fines content contributed better than anticipated into providing a better pseudo-cohesion to the backfill material.

The presence of an oblique culvert crossing the alignment directed the exact locations of the anchoring benches for the new embankment foundation and also the elevation of the lower first layer of geosynthetic. Due to the marly nature of the limestone (neogene formation), a drainage blanket of limestone pebbles of 1,00 m average thickness was placed as backfill layer. Construction took place in a six - month period (February - July 2003) providing a total saving of the initial time schedule (concerning bridge erection) by approximately eight months.

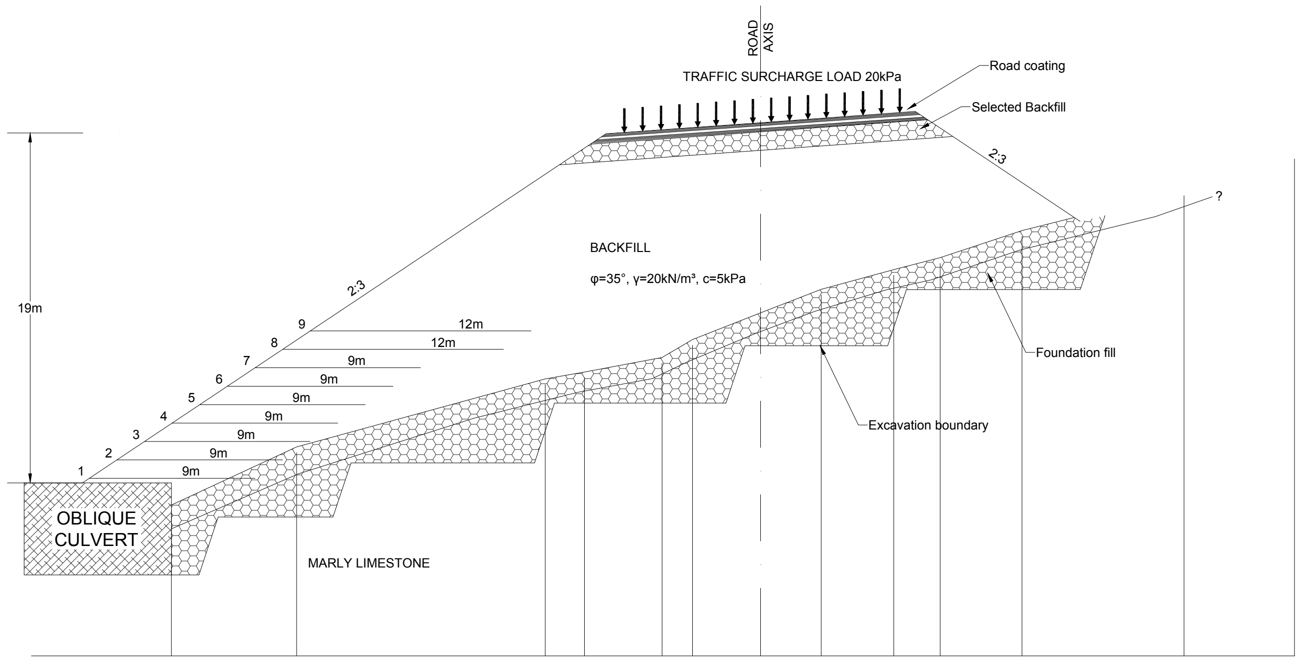


Figure 3 Initial conceptual design

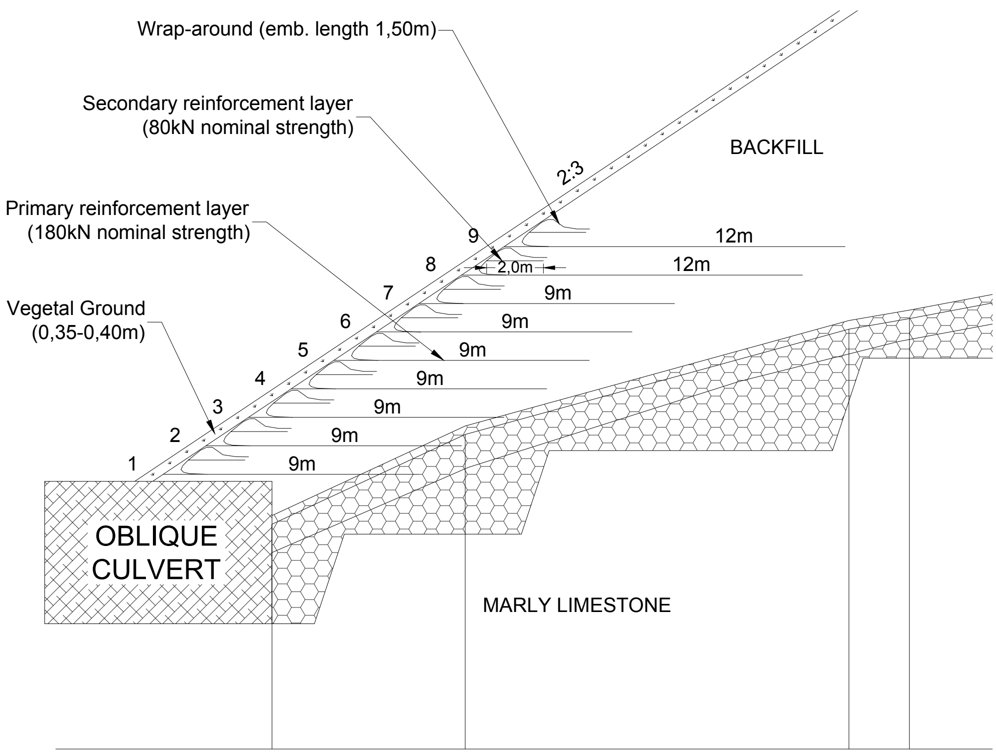


Figure 4 Final design applied during construction

## 4 MONITORING - PERFORMANCE

The conceptual design proposal for instrumentation of two layers of the reinforcing geosynthetics to monitor maximum strain was rejected by the Contractor (argumenting extra delays for monitoring), therefore the only monitoring during construction refered to superficial watch points on the slope as well as inclinometer magnets on one drill hole to indicate settlements and monitor occasional lateral movements. From those measurements, a total 50% of the calculated self settlement of the embankment was only obtained while no lateral movement of the embankment was ever recorded. In addition to that, subsoil settlement was practically negligible and the mechanical behaviour of the shattered limestone was practically the one of an incompressible layer.

The outer slope of the embankment was shaped at an inclination of 2:3 (independently to the applied wrap-around system of geosynthetics) and then covered by a layer of vegetal ground at an average thickness of 0,35 - 0,40 m approximately. Although hydroseeding was strongly recommended by the designer and foreseen in the tender documents, no such antiwash - out protection was ever applied to the slope.

Figure 5 Aspect of the local landslide of the vegetal ground

As the result of an intense rainfall at summer period, local landslides of the cover occurred over the wrap-around interface and the use of more granular material was then imposed as remediation (Figure 5).

The cost efficiency of the final application of the alternative proposal inducing the geosynthetics reinforced embankment was also impressive. Therefore the initial budget for the bridge of approximately 1,3 million € was reduced to its one third approximately, including the cost of all parallel culvert structures, retaining walls and annexed modifications of the intital conceptual design.

## 5 CONCLUSION

Even there was a major difference between the conceptual design of the embankment and the final design for construction concerning the exact implication of the geosynthetic reinforcement, the considered embankment proved once more that soil structures well designed with flexible reinforcing geosynthetics may provide cost - effective and time - saving solutions to certain road construction projects. Indeed, the method that was applied yielded a substantial cost reduction reaching approximately 70% of the initial budget, combined to the use of inexpensive locally found materials. Nevertheless, a more intense instrumentation and monitoring of such projects is highly recommended and should be imposed during construction for a time period of the project preservation of at least twelve months.

## 6 AKNOWLEDGEMENTS

Special thanks are due to the Supervisor Engineer of the project, Mr G. Papadakis, since without his assistance, the major change in the conceptual design would not be procedurally feasible and the whole project would finally be delayed.

## 7 REFERENCES

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