DESIGN AND CONSTRUCTION OF HIGHWAY CLAY EMBANKMENTS REINFORCED WITH WOVEN GEOTEXTILES OVER SOFT FOUNDATION SOIL

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Abstract: Certain parts of a concession project of a new motorway in Greece were designed and constructed prior to the concession agreement by the Greek State. The challenge and requests of a potential private owner are for an efficient, cost-effective solution for the concession period of 40 years with minimum maintenance cost.

As such, the construction of highway embankments of 8-12 m average height was geotechnically designed to cover the following issues:

a) to overcome unfavourable soil conditions, as a part of the motorway for approximately 3 kms crossed soft lagoon deposits of approximately 15-25 m depth. This was obtained by installation of wick prefabricated vertical drains with certain preloading stages.

b) to use local borrow materials described as lean gravely or sandy clays, obtained from neighbouring marly materials from formation of motorway cuttings. This is instead of disposing of 600.000 m3 of clayey material and replacing it with selected granular backfill. For this purpose these clays were associated with woven geotextiles of preferably low strength (30 kN/m) to create reinforced embankments.

c) to construct the embankments using staged construction (3 stages), taking advantage of the benefits of consolidation of each stage.

To aid the design, a fully instrumented trial embankment was constructed. During construction, extensive monitoring by piezometers and settlement plates has been applied by the Contractor to assess the theoretical geotechnical calculations for consolidation, settlements and slope stability of various sections of the reinforced embankments. The cost benefit is impressive based on the use of local clays instead of expensive borrow materials, quick construction within time-schedule, low maintenance cost since consolidation is achieved. This is a result of the correct concept design using reinforcing geotextiles of local producers, in combination with poor-quality backfill material available on site.

Keywords: field monitoring, full-scale test, reinforced embankment, soil improvement, stability analysis, woven geotextile.

PROJECT DESCRIPTION - GEOTECHNICAL INFORMATION

In the context of design, construction and exploitation of the new western corridor Ionia motorway in Greece, a part of it with a length equal to 3 kms should be constructed over soft alluvial lagoon deposits with a maximum embankment height of 9.50 m. The motorway "Ionia Odos" will be constructed mainly on rock formations except some parts like the one presented in this paper in which the most unfavourable soil conditions were met due to its location near the lake "Ozeros". A lack of selected sand-gravely borrow materials at the broader area together with the presence of neogene clayey marls induced the construction of a geosynthetics reinforced embankment using marly products of neighbouring cuts. In this way, a serious reduction of the construction cost on materials was anticipated. The geotechnical design had to cover the following issues:

- Staged construction of the embankment, taking into account the increase of soft foundation soil's strength as consolidation takes place under the embankment load.
- Replacement of the upper 2m very soft compressible alluvium with granular material.
- Reduction of the consolidation time of the subsoil by the use of prefabricated wick vertical drains.
- Design of geosynthetics reinforced embankment consisting of selected granular backfill at its base and top and of cohesive materials associated to woven geotextiles along the major part of the embankment body.

The geotechnical design was based on an extensive geotechnical field investigations campaign (sampling boreholes and cone penetration tests), laboratory tests and evaluation of results. Also a trial embankment was built on the most unfavourable soil conditions in order to pre-estimate the performance of the embankment.

The geotechnical conditions prevailing at the area of interest involved two main formations:

a) Superficial alluvial soft lagoon deposits, of high compressibility, consisting of alternations of lean clays, sandy silts, organic clays and thin horizons of silty sands with some gravels. The average cone tip resistance in these layers varied between 0.64 MPa and 2.5 MPa and the SPT blows number between N = 0 (free penetration) and N = 19. The lower values of the undrained shear strength of the cohesive parts of the formation (by quick triaxial, vane and unconfined compression tests) varied between 3 kPa and 39 kPa and the initial moisture content between 18% and 55%. The thickness of the formation was approximately 8.0 - 9.0 m.

b) Neogene deposits with average consistency, locally dense to very dense, consisting of alternations of marly clays, silts with dense sandy lenses and extensive layers of very dense to compacted silty gravels with sand (weathered breccias and conglomerates but without rocky structure). These layers presented locally an average compressibility, generally an average to high bearing capacity but deep lenses of softer silty pockets occurred all along the examined part. The average cone tip resistance in these layers varied between 1.94 MPa and 4.24 MPa. The presence of an incompressible layer (marly substratum) was assumed for the need of the design at a depth of 25 m below ground level. The average groundwater level was found at 1 m below ground surface.

The ideal cross section at the most unfavourable area with the geotechnical design parameters is presented at the following Table 1 and Figure 1.

Table 1. Geotechnical design parameters

Geotechnical Design Parameters	**Granular Material (0-2 m)**	**Alluvial Deposits (2-8 m)**		**Neogene Deposits > 8m**	
		Layer Ib Lean clay - locally organic sandy silt	**Layer Ic Lean Clay - Silty sand locally with gravels**	**Layer IIb Sandy Silt**	**Layer IIc Silty Gravels with Sand**
Bulk weight above phreatic level, γ (kN/m^3)	21	18	21	20	19
Young's modulus, Ey (MPa)	41.7	-	6.7	-	18.3
Undrained Young's Modulus, Eu (MPa)	-	11.5	-	30	-
Cohesion, c' (kN/m^2)	2	12	15	10	2
Effective angle of internal friction, φ' (°)	35	20	33	33	30
Over consolidation ratio, OCR	-	2	-	1	-
Compression index, Cc	-	0.285	-	0.281	-
Recompression ratio, Cr	-	0.043	-	0.032	-
Initial void ratio, e0	-	0.949	-	0.86	-
Coefficient of consolidation, Cv (m^2/year)	-	3.86	-	6.88	-
Undrained shear strength, $Su_{top\ of\ layer}$ (kN/m^2)	-	23.0	-	68.0	-
Undrained shear strength, $Su_{bottom\ of\ layer}$ (kN/m^2)	-	30.5	-	92.0	-

Figure 1. Ideal geotechnical cross section

The shear strength parameters for the cohesive embankment material used for the construction of the reinforced embankment body were critical, concerning all stability calculations. These parameters were conservatively issued by co-evaluating the results of the following laboratory tests:

a) Drained Triaxial Tests (CD) were performed to determine the range of the effective friction angle φ' and the low expected value of an effective pseudo-cohesion c'.

b) Ring Shear Tests were also performed to determine the lower possible value of the residual strength of the clay (φ_{res}).

Pitcher sampler was used in constructed test embankments, with compaction equal to 95% of the Modified Proctor Tests to provide correct undisturbed samples for laboratory tests.

The finally adopted conservative design values for cohesive embankment's material were: Effective Angle of Internal Friction: $\varphi' = 21°$, Effective Cohesion: c' = 5 kPa, Young's modulus E_y = 41.7 MPa, Bulk Weight above phreatic level γ = 21 kN/m^3.

CONCEPT AND DESIGN OF REINFORCED EMBANKMENT

The conceptual design of the embankment at its maximum height of 9.50 m involved a three - stages construction sequence, after application of soil improvement by vertical prefabricated wick drains at a 2.00 × 2.00 triangular pattern with a length varying between 10-25 m, covering all the soft alluvial deposits area as well as soft silty pockets of the lower neogene layer:

a) Stage 1

Following an extensive superficial natural soil replacement by selected granular backfill of D = 2.0 m thickness and the use of a separating non-woven geotextile of 200 gr/m^2, the first stage of construction included a height of 3.0 m of also selected granular backfill materials, extending as counter toe weights by 10.0 m on both sides of the final section. Side slopes of the first berm were 2:3 (32°). A total duration of 5 months was necessary for preloading to achieve the requested by stability calculations degree of consolidation of the soil layers.

b) Stage 2

An additional height of 3.50 m was realized by cohesive borrow materials reinforced by woven geotextiles of 30 kN/m nominal tensile strength produced in Greece. Reinforcement sequence called for a thickness of each layer of 0.40 m compacted at 95% of the Modified Proctor maximum density. The material used for backfill purposes was mainly classified at A-2-6, A-4, A-6, as per AASHTO classification system. A total duration of 5 additional months was necessary for preloading to achieve the requested degree of consolidation at this stage.

c) Stage 3

Final achievement of the longitudinal section red line was realized by adding the final 3.00 m of backfill. The lower 1.80 m of material involved the same type of reinforced clayey embankment body as per Stage 2, while the upper 1.20 m (in contact with the future road pavement) involved the use of selected granular backfill (classified into A-1, A-2-4 or A-2-5 of AASHTO classification system, according to the Greek Norms). A part of these materials was obtained by removing the upper 1.00 m of the toe counter weights placed during Stage 1 of construction.

The design of the reinforced embankment was performed based on the principles of the existing Greek Guidelines by Egnatia Odos S.A. (Version A01) with the modifications requested to cover the use of cohesive materials associated with woven geotextiles (for correct construction practice by the Contractor). The use of the limit equilibrium analysis method was implemented by the computer code Ressa V.2.0, slightly modified to add the 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 imposed for the design:

Table 2. Requested minimum safety factors

No	Loading conditions	Shear strength parameters of subsoil layers	Requested safety factor
1	Short - Term, Static Loading	Undrained Conditions (cohesive layers)	1.20
2	Long - Term with Seismic Loads	Improved Undrained Conditions due to consolidation (cohesive layers)	1.00
3	Long - Term, Static Loading, Maximal Ground Water Level of 50 years period	Effective Shear Strength Parameters (φ', c')	1.30

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 woven reinforcing geotextile 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 : $T_k = T_{ult} / f_m \times f_e \times f_d \times f_{cr}$ (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 (γ_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 and chemical impact (f_e = 1.00), f_d partial safety factor against installation damage (f_d =1.05), f_{cr} partial safety factor against creep behavior (f_{cr} = 3.30). 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, the creep partial safety factor was considered as f_{cr} = 1.00.

STABILITY AND SETTLEMENTS

The determination of the overall safety factor against stability of the reinforced embankment was obtained by the following relation (5), where the reinforcing effect of the woven geotextile tensile strength is implemented in the denominator to reduce the sliding moment M_D :

$F_s = M_R / (M_D - T_s \times R)$ (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 geotextile 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 stage of construction with the above mentioned loading conditions (Table 2) and the calculated overall safety factors are recapitulated in the following Table 3.

Table 3. Stability analysis - overall safety factors

Geometry of the Embankment	Type of analysis	Method of analysis	Failure Surface	Safety Factors		
				Unreinforced embankment	Reinforced embankment	
				Static loading	Static loading	Seismic loading
A' Stage H = 3.00 m	Undrained	Bishop	Circular	2.17	-	-
	Effective	Bishop	Circular	2.66	-	-
B' Stage H = 6.50 m	Undrained	Bishop	Circular	1.96	-	-
	Effective	Bishop	Circular	2.49	-	-
		Comprehensive Bishop (ES)	Circular	-	2.53	1.43
		Spencer (IS)	Two-part wedge	-	1.65	1.51
		Spencer (CS)	Three-part wedge	-	2.47	1.34
C' Stage H = 9.50 m	Undrained	Bishop	Circular	1.72	-	-
	Effective	Bishop	Circular	2.00	-	-
		Comprehensive Bishop (ES)	Circular	-	2.04	1.22
		Spencer (IS)	Two-part wedge	-	1.30	1.07
		Spencer (CS)	Three-part wedge	-	2.02	1.16

It should be noted that in order to estimate the factor of safety from total stress at second and third construction stage, the gain in undrained shear strength as a result of consolidation has been taken into account, using the expression:

$S_u / \Delta\sigma_{v'} = 0.11 + 0.0037 \times PI$ (Skempton 1957) (6)

where: S_u is the gain in undrained shear strength equal to the product of $\Delta\sigma_{v'}$ is the vertical soil stress caused by embankment loading on infinite length and average consolidation ratio (U), PI is the plasticity index.

Settlement theoretical calculations (immediate, consolidation, self-settlement) were performed by combining the settlements results of the 3-staged construction process as presented over next Table 4.

Table 4. Settlements computations results

Type of calculated settlements	Settlement (cm)			
	A' Stage (h = 3.00 m)	B' Stage (additional h = 3.50 m)	C' Stage (additional h = 3.00 m)	Total (H = 9.50 m)
Self - Settlement	0.19	0.24	0.24	0.67
Immediate	6.11	6.71	6.73	19.55
Consolidation	24.64	24.64	22.65	71.93
Total	30.94	31.59	29.62	92.15

To accelerate consolidation of the subsoil, the use of prefabricated vertical wick drains was applied, with depth varying between 10 m and 25 m, depending on the geotechnical longitudinal section. The final pattern was triangular, at 2.00 × 2.00 m distances, selected as the optimum result of various alternatives examined (gravel columns at different patterns or wick drains at different patterns) within the base of a trial embankment constructed at a selected area with unfavourable soil conditions.

TRIAL EMBANKMENT AND INSTRUMENTATION

In order to investigate the real behaviour of the embankment and to decide upon the best method of construction, a trial embankment was constructed at the area where the most unfavourable soil conditions combined with a maximum height were met.

The trial embankment measured 100 m length and 74 m in width and incorporated 4 sections. The inclination was 2:3 (vertical : horizontal). These sections were identified as Sections D1, D2, D3 and D4 of 25 m in length each.

Sections D1 and D2 represented a triangular pattern of gravel columns of 2 m and 3 m spacing respectively while D3 and D4 represented a triangular pattern of vertical drains distanced at 1.5 m and 2.0 m respectively. Half of the cross-section below the trial embankment was intensively instrumented to capture relevant data. The instrumentation consisted of inclinometers, settlement gauges and piezometers. Figure 2 shows a plan view of the embankment and Figure 3 shows the typical instrumentation used in section D4.

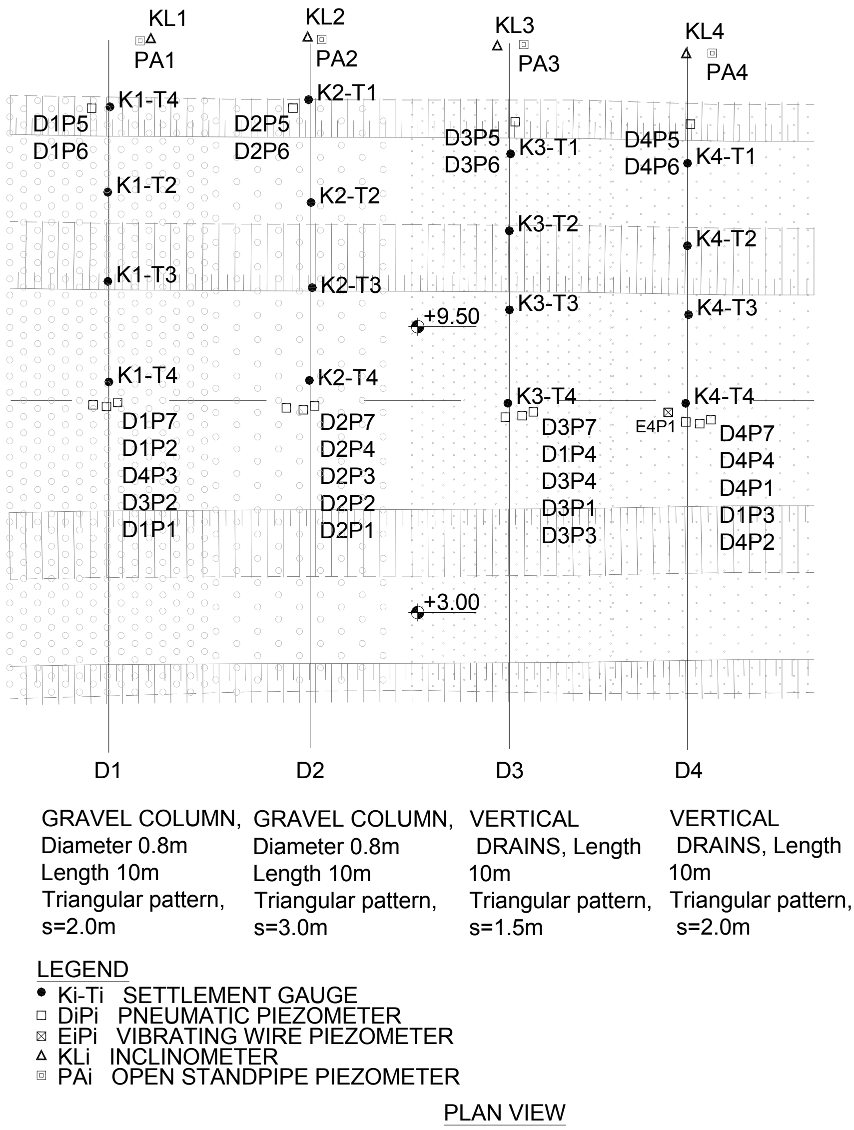

Figure 2. Plan view of the embankment.

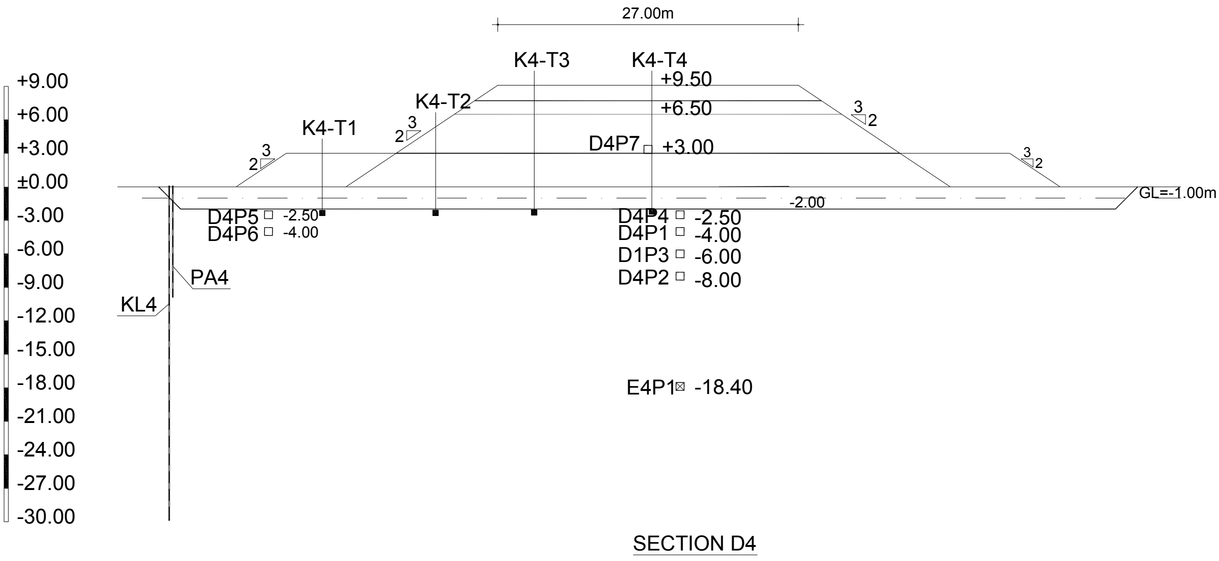

Figure 3. Typical instrumentation used in Section D4.

Before constructing the embankment the upper soft alluvium layer of 2 m of thickness was removed and replaced by selected granular backfill that was placed on a separating geotextile of 200 gr/m^2.

The progress of the work included three stages according to the design as they have been described earlier and summarized below:

At the 1st stage an embankment of 3 m height consisting of the same granular material as for the foundation was constructed with base width equal to 74 m with 2 stabilizing berms of 10 m on each side. The duration of this stage was 5 months.

At the 2nd stage an embankment of 3.5 m height consisting of cohesive material associated with woven geotextiles was constructed. The duration of this stage was also 5 months.

At the 3rd stage an embankment of 3.0 m height was constructed. The lower part of 1.20 m height consisted of cohesive material associated with woven geotextiles while the upper part of 1.40 m height consisted of granular material which was removed by the first 1.00 m of the berms.

In all that time settlements, horizontal displacements and pore pressure at specific depths below the embankment were recorded.

The settlements were estimated using elastic displacement theory as far as initial settlements are concerned (Pi) and one dimensional primary consolidation theory as far as consolidation settlements are concerned (Pc). Secondary compression was ignored. The vertical stress at the center of each layer due to the embankment loading of infinite extent was estimated using elastic theory (Poulos and Davis, 1974).

The vertical degree of consolidation was estimated as a function of time factor by Terzaghi's theory of consolidation by vertical flow.

The average degree of consolidation for radial consolidation was estimated based on Barron's theory, both in gravel columns and drains. The anisotropic permeability value (Kh/Kv) was set at 2.0.

The average degree for combined vertical and radial consolidation was obtained by Carillo's theory.

Comparison between predicted and estimated settlements is presented in Figure 3 for sections D1 (gravel columns at 2m × 2m distance) and D4 (vertical drains at 2m × 2m distance).

As demonstrated in Figure 3 the results predicted from the analysis compared favorably with the field measurements of settlements.

During monitoring, no significant excess pore pressure was observed by the pneumatic piezometers, probably due to the fact that they have been mainly installed in layer Ic consisting of lean clay and silty sand with gravels.

The Trial Embankment enhanced the solution of the staged construction with triangular drain pattern at spacing 2m.

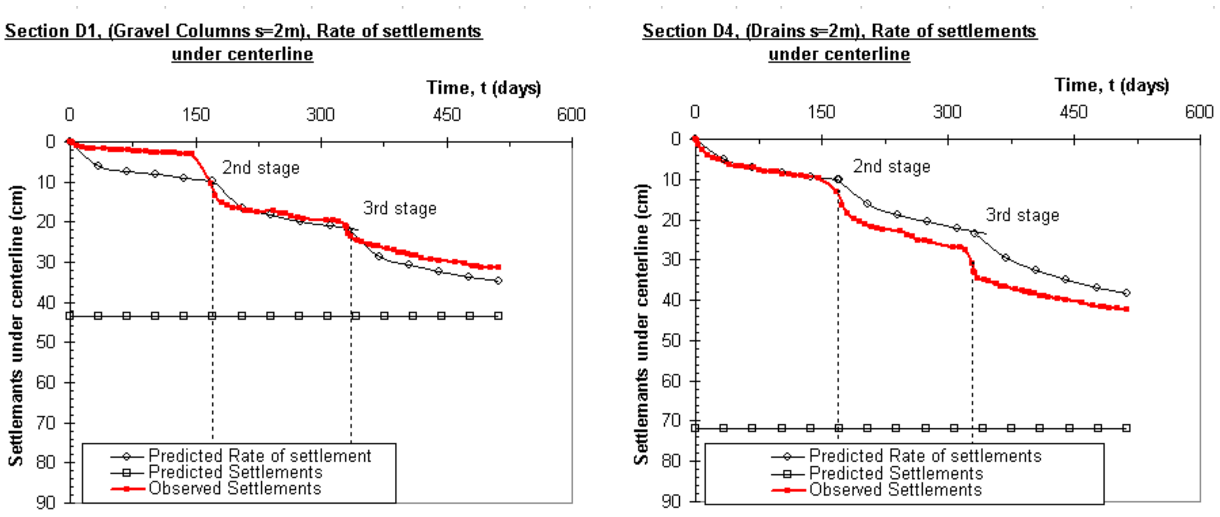

Figure 3. Comparison between predicted and estimated settlements at section D1 and D4.

CONSTRUCTION PERFORMANCE

The most critical point of the project during the implementation period was the correct compaction of the cohesive embankment material and its association to every reinforcing layer of woven geotextile. Generally, the moisture content of the backfill material was supposed to be at the level of w_{opt} +2% approximately, but due to seasonal bad weather conditions, a significant number of layers had to be removed, dried and recompacted.

The use of a woven geotextile and its handling on site was proved to be much easier than initially expected by the contractor. Little or no damage of the geotextiles was observed in the worksite, although heavy sheep-foot rollers were employed for backfilling of each layer of cohesive material (6-8 passes per layer were requested for adequate compaction as contractually imposed). The presence of scarce gravels and sand proved to be beneficial to the project.

The placement of vertical wick drains (520 thousand linear meters of drains were installed for soil improvement and consolidation process acceleration) presented many difficulties, mainly due to the presence of gravely thin layers or overconsolidated clayey lenses. The implementation of predrilling techniques in certain areas was then necessary, since the local presence of lower horizons of high compressibility needed improvement at this depth to cover design requirements.

Monitoring during construction was applied on several critical sections by piezometers and settlement plates. Generally, theoretical computations for settlements were slightly more conservative than practically observed, since the effectiveness of the vertical - draining system by wick drains was proved to be greater than theoretically anticipated.

CONCLUSIONS

At the end of the project, a total amount of 328000 m^2 of reinforcing woven geotextiles was associated with locally available cohesive type of backfill, provided by the near excavations of various marly cuts of the project. Although the initial geotextile cost was very significant, an impressive final cost benefit of approximately 20% was estimated in the project, if compared to the contractual obligation for the Contractor to bring - in selected borrow backfill materials for constructing the embankments. At the same time, an important time saving was obtained, since near-by cuts offered the backfill material incorporated into the reinforced embankments. In this way time-consuming operations to provide the project with the necessary selected borrow material from far situated borrow areas, were avoided, contributing also in the protection of environment (avoiding the creation of borrow and dump areas). The initial cost of reinforcing geosynthetics was totally counter - balanced by the above benefits, following a correct engineering conceptual and detailed design and a successful implementation of construction.

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REFERENCES

Barron, R.A. 1948. Consolidation of Fine Grained Soils by Drain Wells. Transactions of the American Society of Civil Engineers, Vol. 113: 718-724.

Carillo, N. 1942. Simple Two and Three Dimensional Cases in the Theory of Consolidation of Soils. Journal of Maths and Physics, Vol. 21(1).

Edafomichaniki S.A. 2005. Geotechnical evaluation and design of Ozeros embankments for Ionia Highway. Construction J/V TERNA S.A. - MICHANIKI S.A. Volume ΓT8b.

F.H.W.A-NHI-00-043, 2001. Mechanically stabilized earth walls and reinforced soil slopes design and construction guidelines. U.S. Department of Transportation, USA.
Hansbo, S. 1981. Consolidation of Fine-grained Soils by Prefabricated Drains. Proceedings of the 10^{th} ICSMFE, Vol. 3: 677-682.
RESSA, 2006. Reinforced Slope Stability Analysis Program. ADAMA Engineering, Version 2.0.
Scientific Committee of Egnatia Odos S.A. 2007. Guidelines for the design of reinforced embankments. Edition A01.