Design and Construction of an Off-stream Pond using Geomembranes in Greece

A. Collios
Edafomichaniki Ltd, Athens, Greece

ABSTRACT: The off-stream pond was realised by excavation and embankment (dam) construction, with a designed capacity of 600 thousand m^3 of water for irrigation purposes. The average permeability of the subsoil induced the use of a geomembrane liner. Construction difficulties had to be solved on site, such as water flooding during excavation of a very permeable soil lense and important deformation of the liner sand support due to heavy rainfalls. Design against sliding of the liner cover was adopted and the use of an extensive network of geotextile drains was decided during construction, proving that strict quality control and site follow - up by specialists was necessary.

1 INTRODUCTION

Aiming at the stimulation of irrigation projects in the islands of Greece, the E.E.C. has approved the design and construction of water ponds which are mainly off - stream to limit the problems of carried materials and large sizes of spillways. In this contexte the Visari off-stream pond was decided to be constructed in Crete, having a capacity of approximately 600 thousand m^3. Construction of the pond was realised by excavating a central tray and using the materials to build embankments. Water was brought in by means of a special diversion project (small dam) of the main Lagas stream.

The total height of the embankments was 11,0 m and the maximum water depth in the pond 10,0 m. The inside slope of the embankment was designed at 1:4 and outside slope at 1:2,5, having a top width of 8,0 m, allowing vehicles circulation. The internal sides of the embankments and the bottom of the pond, not presenting adequate impermeability, were covered by a geomembrane, laid upon and protected by sandy layer, overcome by a rip-rap. The construction of the off-stream pond started in March 1993 with a total contract time of 10 months. Edafomichaniki Ltd and the author supplied design and consulting services including quality control throughout the construction period. (Figure 1)

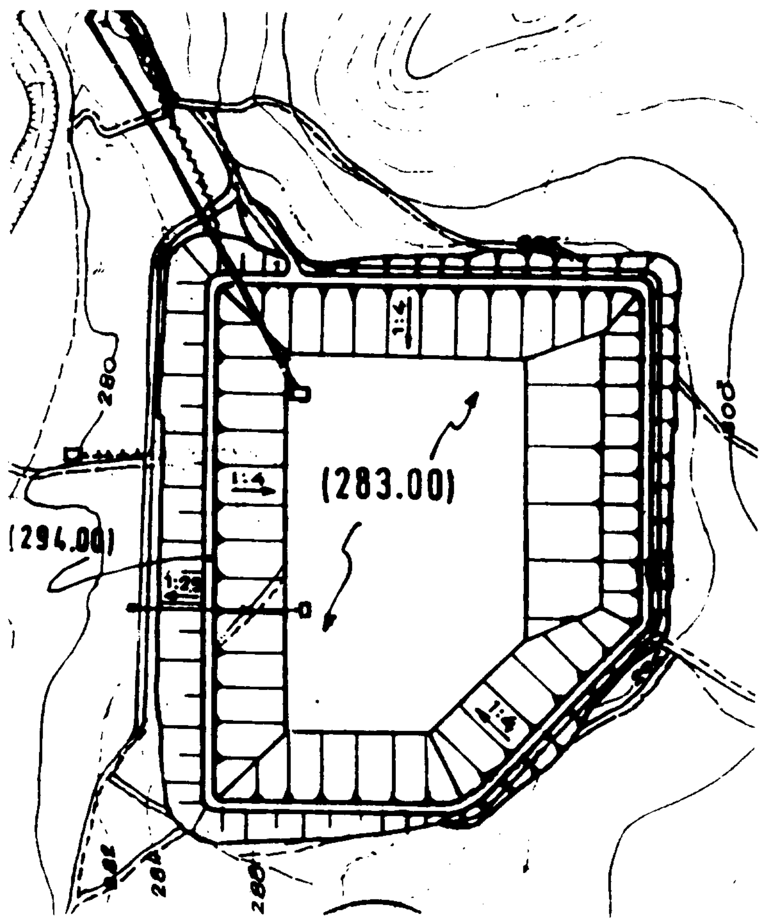


Fig. 1 General layout (scale 1:500)

The geological formations at the area of interest consist of Eocene flysch appearances including sandy marls, sandstones, conglomerates and sandgravels. The presence of large boulders and cobbles at the south part of the pond was very frequent, while at the N-E part a limestone marl necessited excavations in a formation behaving practically as rock. The site permeability was very variable, 10^{-9} m/sec for the marls, 10^{-7} m/sec for the sand-gravels (which had an average fine-grained material content of 30%) and higher than 10^{-5} m/sec for the areas of cobbles and boulders. With the exception of the marly products of excavations, all other material was adequate for compaction during embankment construction. Following an extensive laboratory testing program, it was proved that also coarser grained materials, after Proctor compaction, had a rather low permeability coefficient, at the range of 10^{-7} m/sec, mainly due to the silt content existing in the samples. This was also controlled during construction, by performing in-situ permeability tests in the holes excavated for the needs of compaction control. Nevertheless, a comparative hydraulic study performed, proved that the use of a liner to protect against any loss of valuable irrigation water was a cost -effective solution to follow and the use of a geomembrane was proposed and accepted. Initial calculations of the bearing capacity and awaited settlements under the static loads of the embankment and water, and under the dynamic (earthquake) loading proved the suitability of the subsoil to undertake stresses without failure and at no excessive deformation (S_{max} = 0,13 m). Design of the overall slope stability using Bishop's method for the embankments took into consideration all possible cases (full pond with geomembrane protection, destruction of geomembrane during function, seismic loads equal to 0.20 g, rapid draw-down due to geomembrane destruction) and assured minimal satisfactory safety factors (Table 1):

Table 1. Minimum safety factors against rotational failure of the embankment

	GM function	GM destroyed
No earhquake	2,58	1,93
With ε = 0,12 g	1,91	1,42
With ε = 0,20 g	1,50	1,18

3.1 Design and construction

The typical cross-section of the protected internal slope is given at the following Figure 2.

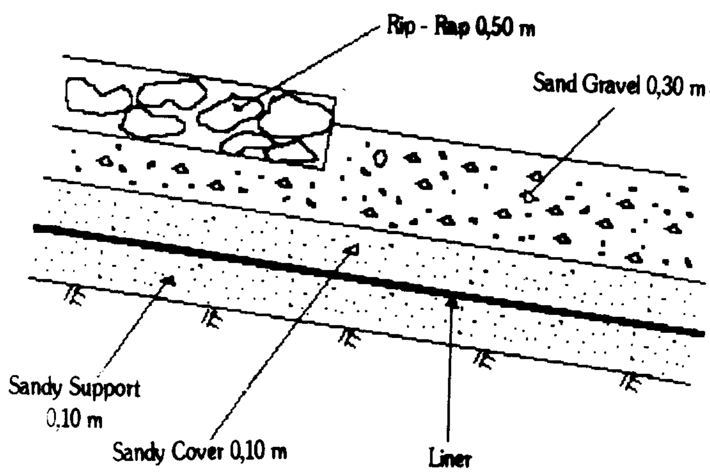
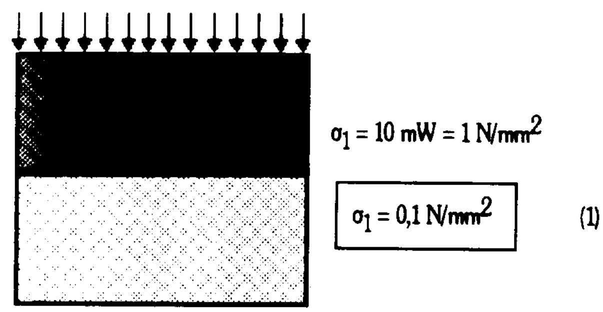


Fig. 2 Typical cross-section of protected slope

The determination of the type and thickness of the geomembrane that was used has taken into account the following items:

a) Long-Term loads (pressure of applied loads σ_1, tensile strain due to shear stress on the inclined surface of the embankment σ_2) (Figure 3). Tensile strain due to the own weight of the liner was not developped, since the embankment slope (14°) was less than the friction between the liner and the sandy support layer.



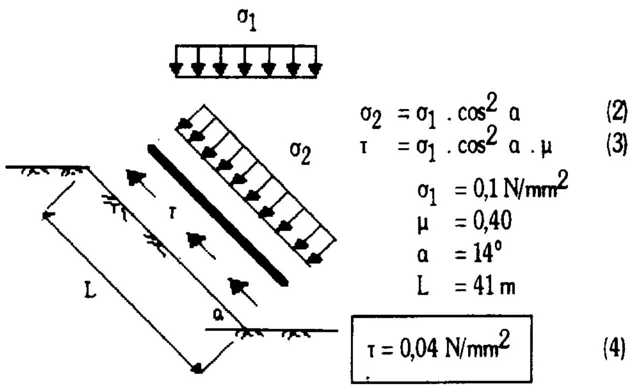


Fig. 3 Long term loads design

Shear stresses created by settlements of a maximal order of 0,20 m were also checked, so that thickness requirements would be fulfilled.

b) Cover soil stability for the protected geomembrane, whose friction coefficient of the surface is considerably lower than the one corresponding to natural material (soil, concrete).

Based upon the design on long-term loads (R.M. Koerner, 1990) and taking into account that a sandy protection layer beneath and above the geomembrane would be applied, a HDPE geomembrane of 0,75 mm thickness was selected to be used on site. This liner satisfied in every partly design a minimum safety factor of 5, referring to pressure and strain requirements.

Control of the sandy protection cover against sliding failure over the selected liner was based upon the critical friction coefficient between the sandy cover and the liner. This coefficient was determined in the laboratory by large scale shear-type tests, with the results presented statistically at Figure 4.

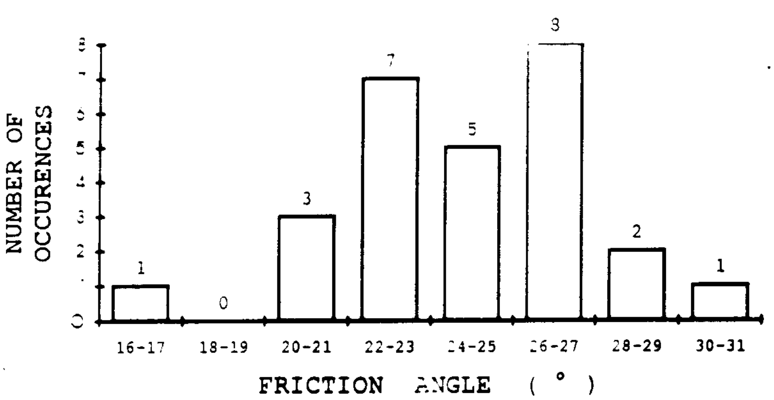


Fig. 4 Laboratory large scale shear tests

The principle, procedure and evaluation of those tests have long been studied (Collios, 1981) and offer a relatively easy tool to the designing engineer to decide upon the friction coefficient that a liner will exhibit in contact to the specific soil to be used in place. Taking into account the slope angle of 14° (1:4) and the most unfavorable friction angle $\varphi_m = 22°$ deduced in shear tests, the safety factor against sliding failure was the most critical one:

$$F_s = \frac{\tan 22°}{\tan 14°} = 1{,}62 \rangle 1{,}50 \qquad (5)$$

Beneath the sandy support of the liner, a network of drains was constructed, conducting the water to preselected locations. Additional drains were also placed at the northern sides of the pond, since water presence was abundant in this point. Therefore, redesign of all drains during construction was necessary.

3.2 Quality Control

One of the major tasks of the consulting engineers was to assure a continuous and strict quality control, including redesign of special features that might be proved different than initially supposed. In situ quality control consisted of:

a) Control of the in-situ obtained density during compaction of the successive layers of the embankments (the sand cone method was employed). Totally 100 points were controlled, with their results presented at the diagram of Figure 5.

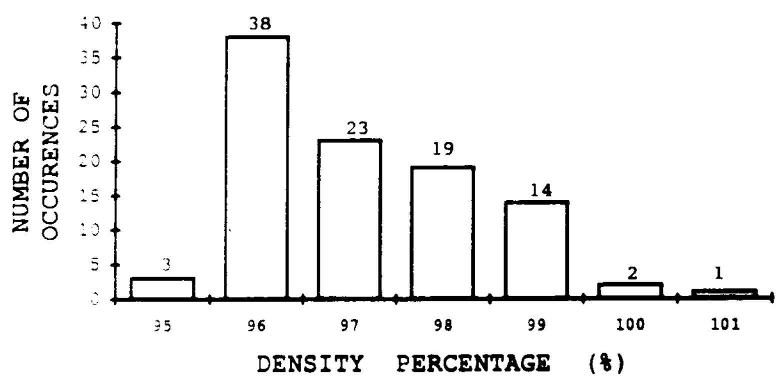


Fig. 5 Frequency distribution of field obtained densities

b) Materials control for their use as embankment, drainage, supporting and protection the liner sandy layer, filters and rip-rap protection.

c) Control of all seams of the liner using adequate techniques. The minimal specified pressure of 150 KPa was maintained for at least 10 minutes.

d) Control of the liner before placement, using a statistical approach concerning the number of samples for laboratory testing, to verify that the product on site conformed to the specifications of the initial.

4 CONSTRUCTION DIFFICULTIES

During the winter season 93-94 and immediately after adequate compaction was performed over the sandy liner support, heavy rainfalls occurring rather often created dammages of the support and the total area of the pond was completely immerged, not allowing continuation of works (laying of the liner) for more than 3 months. This happened mainly because the compacted materials, behaved as rather impermeable. When the waters finally withdrew partially, the sandy support layer was not practically any more on place and new layering and compaction was necessary. Having foreseen that fact, the consultant had previously formulated a proposal to replace the sandy support of the liner by a non- wooven, continuous filament polypropylene geotextile. Unfortunately this proposal was not retained by the Ministry and the Contractor presented an important claim, due to a delay of works, independent to his will. The sandy support had to be reformed and compacted for a second time and then the liner was finally placed. As a result, the project delayed for over three months due to the weather conditions, a fact that finally was proved more costly than the proposed use of a geotextile replacing totally the sandy support of the liner.

5 CONCLUSIONS

The construction of an off-stream pond was a technically rather simple project, having as a single peculiarity the use of a geomembrane liner. Totally 90.000 m^2 of SLT geomembrane having a thickness of 0,75 mm were installed on place to assure impermeability of the pond. The presence of sand dunes at the viccinity of the project during the preliminary stage of design induced (EGM, 1991) its use for both as support and as cover layer. Although slope design suggested a rather smooth inclination of 1:4 (1:vertical, 4:horizontal), the compaction problems of the slopes and the rainfall damages of the sandy support layer of the liner during the construction period imply that for future applications, the use of a geotextile instead might be proved as a most cost-effective solution to follow.

A very strict follow - up and quality control (similar to the one applied in earth dams construction) has to be assured by flexible geotechnical specialists in geosynthetics, since the initial design recommendations may need adaptations, following the site conditions during construction.

6 REFERENCES

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