Stresses and strains in the Evinos dam, Greece

Contraintes et déformations au barrage d'Evinos, Grèce

G.T.Dounias & E.O.Papageorgiou – *Edafos Ltd, Athens, Greece*

ABSTRACT: The behaviour of the core and shoulder materials of the 126m high Evinos embankment dam is presented during construction and a subsequent rest period of 3 years. The paper presents information regarding total stress built up, pore water pressure generation and the stress paths followed by the different materials. It assesses the material properties and the potential for hydraulic fracture of the core.

RÉSUMÉ: L'article suivant présente le comportement des matériaux du noyau et des massifs stabilisateurs du barrage de terre d'Evinos, qui est 126 m de haut, pendant la construction et une période suivante de repos, d'une durée de 3 ans. L'article contient information sur l'accroissement des contraintes totales, la création des pressions interstitielles et les trajectoires des contraintes, que les matériaux différents ont suivi. L'article contient aussi une évaluation des propriétés des matériaux et de la possibilité d'une fracture hydraulique au noyau du barrage.

1 INTRODUCTION

The Evinos River earth dam, located in western Greece, is part of a diversion scheme for the water supply of Athens. The 126 m high embankment is placed on flysch rock. The external slopes of the earth-fill dam are 1:2.3 (vert.:hor.) upstream and 1:2 downstream. The central core has side slopes of 3:1. The lower and external part of the shoulders is made of flysch rockfill while the rest is made of river gravel. Cross section IX is presented in Figure 1. Construction of the embankment commenced in 1994 and final crest level was reached at the end of 1997. Impoundment is planned for the summer of 2001.

Dounias & Marinos (1993) and Marinos et al (1995) describe the geotechnical behaviour of the flysch and the influence of the particular geological features on the design of the dam. Papageorgiou (1999) presents information regarding the instrumentation of the dam. Dounias et al (2000) present the design and construction of the filters for the core.

Three sections of the dam were heavily instrumented with inclinometers fitted with settlement plates, vibrating wire and hydraulic piezometers, linear extensometers and surface markers. On sections VI and IX clusters of 5 earth pressure cells are placed on two rows. Vibrating wire and hydraulic piezometers are also placed in the foundation. The instruments are measured regularly enabling the monitoring of the embankment behaviour during the 3-year rest period.

The dam is typical for Western Greece having similar design features to dams of similar height (e.g. Kremasta 165m, Kastraki 96m, Pournari 102m, Mornos 126m). It differs from its predecessors in that its core is not made of ready clay but of slightly weathered mudrocks. The rather generous instrumentation was intended to prove that the core has the characteristics of a clay core and that embankment behaviour was as expected.

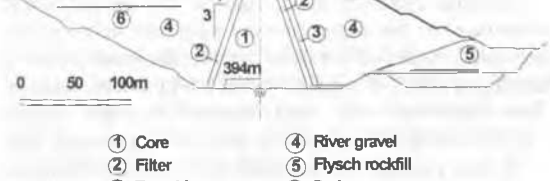


Figure 1. Section IX of Evinos dam

2 MATERIALS OF CONSTRUCTION

The core was built using a thinly bedded, highly folded and slightly weathered mudrock of flysch origin, lying within the reservoir. Special excavation and placing techniques were used in order to produce a clay like material (Dounias et al, 2001). Some basic parameters of the placed material are presented in Table 1. The dry density, γ_d, and water content, w, were determined by in situ tests. The maximum dry density, γ_{dmax}, and the optimum water content, w_{opt}, were determined in the laboratory by the Standard Proctor compaction test (2.5 kg hammer). The cohesion, c, and the angle of shearing resistance, φ, were determined on samples taken from the embankment. The coefficient of permeability, k, was measured in a variety of in situ tests.

The lower and outer portion of the shoulders was built from a "dirty" rockfill produced from the excavations for the core trench. It was mainly sandstone with 20 to 30% mudrock. The rest of the shoulders were built from good quality, well graded river gravel. The strength parameters of the shoulder materials were assumed based on tests of the finer portion of the materials

Table 1. Average geotechnical parameters of construction materials

Average values	Core	River gravel	Rockfill
γ_d (kN/m^3)	19.39	23.3	22.7
w (%)	12.92	4.9	5.3
LL (%)	31.91	-	-
PI	14.15	N.P.	N.P.
γ_{dmax} (kN/m^3)	19.36	-	-
w_{opt} (%)	11.56	-	-
d < 0.075mm (%)	50.2	5.6	7.2
k (m/sec)	1.6×10^{-8}	1.8×10^{-3}	2.2×10^{-3}
c' (kN/m^2)	20	-	-
φ' (°)	29	40	40

3 MEASUREMENTS & INTERPRETATION

3.1 *Total stress development*

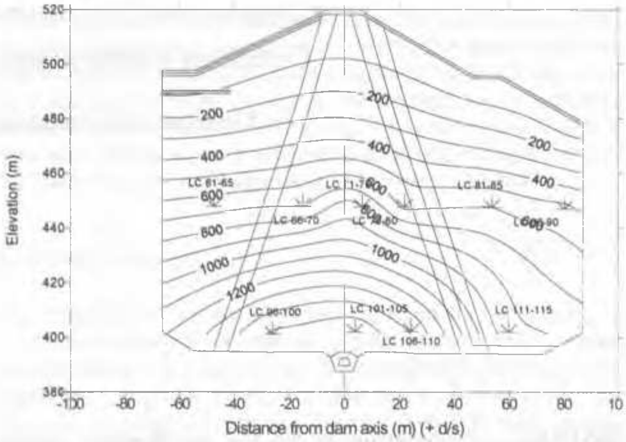
Twenty-three clusters of 5 earth pressure cells each were installed. The cells were flat thin circular stainless steel disks, 229 by 11 mm, filled with Ethylene Glycol, fitted with a vibrating wire transducer, produced by Slope Indicator Co., USA. They were positioned in order to measure vertical stresses, horizontal stresses along and across the dam axis and on two 45° inclined directions across the dam axis. Due to space limitations only the results obtained at section IX are presented here.

Due to a variety of factors earth pressure cell measurements are considered to be of small accuracy (Dunnicliff, 1988). It is also considered that they perform better in clay than in granular materials. In order to check their accuracy, the measured vertical total stress at the deeper row is plotted against the nominal overburden in Figure 2. This area of the embankment was very large early on, resting on a hard foundation, and was close to two-dimensional k_o conditions. The degree of the initial agreement can be taken as a measure of the accuracy of the measurements. It is seen that the measurements underestimate the stresses. The percentage of underestimation ranges from 0 to 35% with an average of 19%. This finding supports the generally held opinion about the low accuracy of these measurements. On the other hand though, the measurements follow the stages of construction and reflect the behaviour of the embankment. It is believed that they can be used in order to investigate the behaviour of the embankment and the materials.

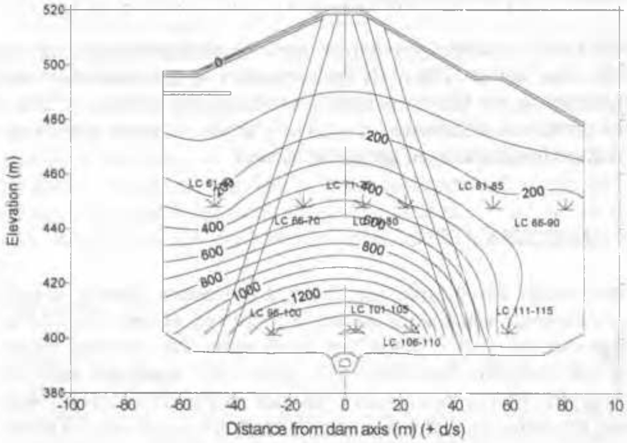
The measured stresses correspond to σ_x, σ_y, σ_z, σ_{z45y}, and σ_{y45z}, where x is the horizontal longitudinal direction, y the horizontal cross direction and z the vertical direction. It is possible to calculate the mean total stress $p=(\sigma_x+\sigma_y+\sigma_z)/3$. On the plane perpendicular to the dam axis it is also possible to calculate the shear stress and the principle stresses. It was found that normally the vertical direction diverged no more than 10° from the direction of the major principle stress. The minor principle stress will therefore be near horizontal and it can be approximated by the minimum of σ_x and σ_y.

The great majority of the clusters showed that horizontal stresses σ_x and σ_y were very similar, indicating conditions similar to the triaxial apparatus. The mean total stress, p, and the minimum stress, σ_{min}, in October 2000, three years after construction, are shown in Figure 3. Some approximation is involved in contouring but the locations of the cells are also shown.

Stress paths on a $(\sigma'_z+\sigma'_y)/2$, $(\sigma_z-\sigma_y)/2$ space are shown in Figure 4. Because of σ_z being very similar to the major principle stress, these stress paths can be considered a good approximation of the paths in terms of principle stresses. The paths followed by the cells installed in the gravel shoulders (Fig. 4a) indicate a k_o condition with an average $k_o=0.18$. Assuming conservatively that this average path follows the failure envelope, the resulting



(a) Mean total stress $p=(\sigma_x+\sigma_y+\sigma_z)/3$



(b) Minimum total stress σ_{min}

Figure 3. Contours of mean and minimum total stress, year 2000

angle of shearing resistance is approximately 45°, higher than the value of 40° assumed in the design. In reality the strength should be somewhat higher.

The paths followed by the cells in the core (Fig. 4b) show different behaviour according to their location. Those placed close to foundation level show a near hydrostatic stress state with very little shearing. This is attributed to the high placement moisture and the fact that the lower parts of the embankment at section IX were placed in a very wide trench formed by the already built rockfill bases. Those placed at the upper row of instruments in drier conditions do not show a consisted pattern. The centre of the core (LC 71-75), shows signs of reaching the failure envelope and travelling along it. This assumed envelope can be defined by c'=100 kN/m² and φ'=25°. The laboratory tests on material from the core gave c'=20 kN/m² and φ'=29°.

The mean, vertical and minimum total stresses across the core are plotted in Figure 5 for the two rows of earth pressure cells. Also plotted is the pore pressure for a reservoir level at the spillway crest. For the deeper row of instruments both the mean and the minimum total stress is well above this reservoir pressure indicating complete safety against the mechanism of hydraulic fracture. At the higher row the mean and vertical total stresses are above reservoir pressure but the minimum stress is lower indicating a possible small potential for hydraulic fracture. It has been observed that total stresses increase during impounding, following the increase in pore water pressure. This increase will most probably be adequate to completely eradicate any danger of cracking. Stresses near the centre of the core are higher than near the boundary. If a possible imperfection near

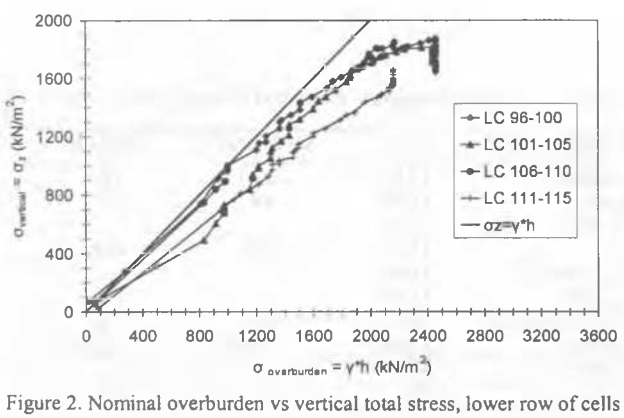
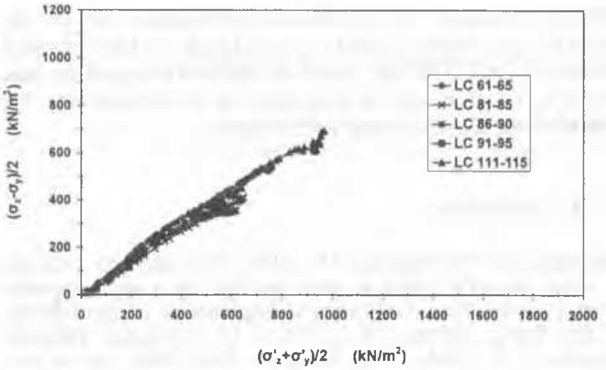
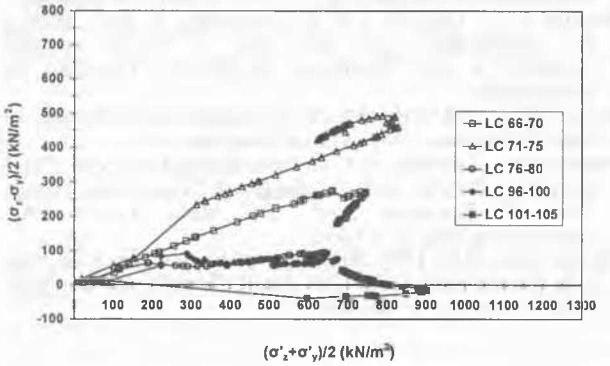


Figure 2. Nominal overburden vs vertical total stress, lower row of cells

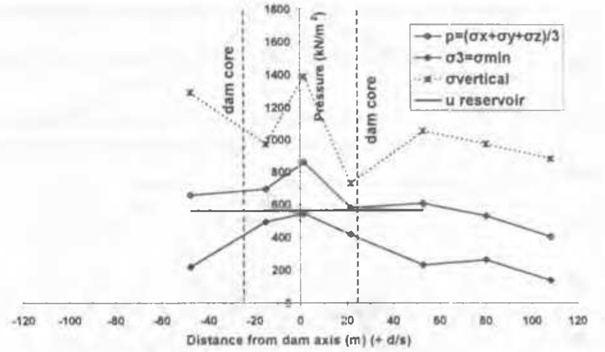


(a) Gravel shoulders

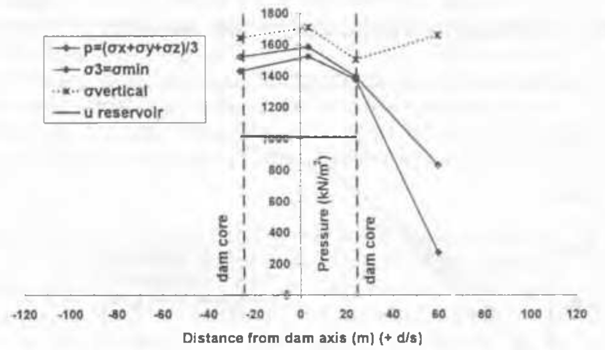


(b) Core

Figure 4. Stress paths during construction and rest period



(a) Across upper row of cells



(b) Across lower row of cells

Figure 5. Mean, vertical and minimum total stresses across the core, IX

the boundary allows local cracking it will not be able to propagate across the core where it will have to overcome higher stresses. If the underestimation of the total stress indicated in Figure 2 is considered, then there is no potential for hydraulic fracture.

3.2 *Pore pressure generation*

A large number of vibrating wire piezometers was installed in the shoulders and the core of the dam. They were fitted with a high air entry ceramic capable of measuring small suctions.

The development of pore water pressure in the core was influenced heavily by the placement water content. Measurements of typical piezometers are presented in Figure 6. Where water content, and degree of saturation, was high, positive pore pressures were generated by nominal overburden pressures of 200 – 300 kN/m^2. Where the initial degree of saturation was less than 80% a much higher overburden pressure was needed. During the 3-year rest period following construction some piezometers showed slow dissipation. This behaviour indicates a low permeability. An approximate one-dimensional analysis indicates permeabilities one order of magnitude lower than the values measured in-situ. Thus there is confirmation that a low permeability close to that of a clay fill was achieved in the core.

Contours of the pore pressures 3 years after construction are presented in Figure 7.

3.3 *Strains*

The vertical inclinometer tubes fitted with settlement plates and the horizontal rod extensometers are crossing near the earth pressure cells of the upper row. The deformation measurements can be linked with the pressure measurements in order to assist the investigation of the material behaviour.

Strain paths for three locations are plotted in Figure 8. The locations are close to the earth pressure cell clusters indicated in the insert. The horizontal to vertical strain ratio in the gravel shoulders is very small indicating nearly two-dimensional conditions, resembling the oedometer apparatus.

The strain path in the core (LC 71-75) is initially indicating "oedometer conditions" but later bends showing lateral expansion. The pressure cells at this location (Fig. 4b) show a stress path pointing to shear failure in compression, compatible with the strain path. The pressure cells in the gravel shoulders (LC 61-65 & LC 81-85) show k_o conditions (Fig. 4a) again compatible with the strain path.

The vertical strain and stress increments were used in order to estimate the compression modulus of the core and shoulder materials in terms of total stresses. A large scatter was observed with an average value for the compression modulus of the gravel shoulders approximately 100 MN/m^2. The average compression modulus of the core was approximately equal to 30MN/m^2.

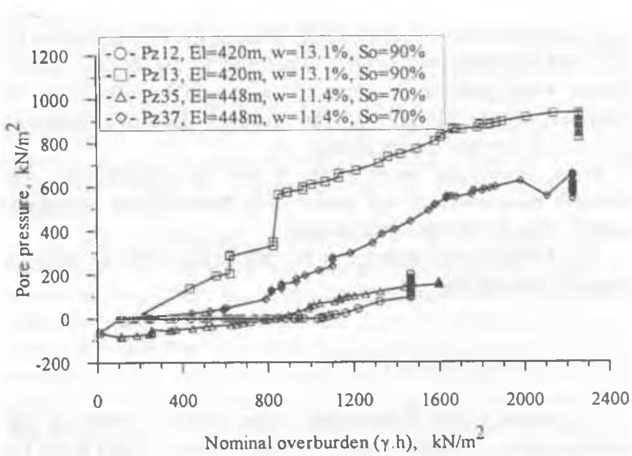


Figure 6. Core piezometer readings – effect of placement moisture content (Dounias et al, 2001)

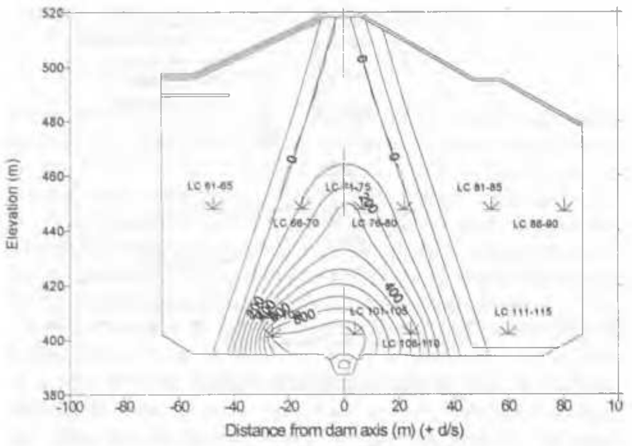


Figure 7. Contours of pore pressure, end of 2000, section IX

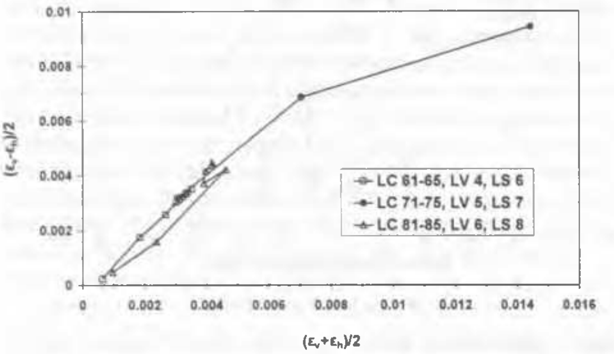


Figure 8. Strain paths centre of section IX

4 DISCUSSION

The instrument readings have revealed that the stress state in the central part of the embankment is essentially two-dimensional resembling k_o conditions. There is some evidence that parts of the core may have failed in "triaxial compression". This is in accordance with the strain paths half way up.

The compressibility of the core, in terms of total stresses, was more than three times lower than that of the shoulders. This difference is expected to give rise to some arching. Nevertheless, there was no significant arching indicated and if a correction is applied to the stress measurements, total stresses in the core are smaller than the expected reservoir pressure. There is an indication that total stress measurements are underestimating reality by 20% on average.

The development of pore pressures and their subsequent dissipation point to a core material that has the characteristics of clay and therefore will behave according to the design assumptions. Pore pressure development was heavily influenced, as expected, by the placement water content. Rate of dissipation points to a very low permeability.

From stress and strain paths it can be deduced that the strength parameters of the constructed materials are somehow greater than the design assumptions.

The embankment appears to be very stable with no signs of potential instability.

5 ACKNOWLEDGEMENTS

The owner of the Evinos dam is the Greek Ministry for the Environment, Planning and Public Works. EYDE/OSYE performed the managing of the construction project for the owner. The contractor for the project was a Greek-Italian-Austrian Joint Venture comprised of AEGEK SA, DOMIKA ERGA SA, METON SA, JAGER CmBH and SELI Spa. G. Karavokyris & Partners Ltd and OTM Ltd carried out the final design of the dam. Prof P.R. Vaughan acted as consultant for the embankment. The help of all the above is kindly acknowledged.

6 REFERENCES

Dounias, G.T. & Marinos, P.G. 1993. Soil-like and rock-like behaviour of a flysch in slope stability. In Anagnostopoulos et al. (eds), Proc. Geotechnical Engineering of Hard Soils - Soft Rocks, Athens, Vol. 2, 1107-1114. Rotterdam: Balkema

Dounias, G.T., Dede, V. & Vaughan, P.R. 2000. Use of river gravel for the core filter of Evinos dam. In Wolski & Mlynarek (eds), Filters and drainage in Geotechnical and Environmental Engineering, 297-304. Rotterdam: Balkema

Dounias, G.T., Vaughan, P.R. & Cavounidis, S. 2001. Making an impermeable clay core from a flysch mudrock. Geotechnical and Geological Engineering (accepted for publication)

Dunnicliff, J. 1988. Geotechnical instrumentation for monitoring field performance. John Wiley& Sons, Inc.

Marinos, P.G., Dounias, G.T. & Karavokyris, G.I. 1995. Flysch geological features and the design of Evinos dam, Greece. Proc. XI European Conf. Soil Mech. Found. Eng., Copenhagen, Vol. 7, 111-116

Papageorgiou, E.O. 1999. Behaviour of Evinos dam during and after construction. Proc. 13th Young Geotechnical Engineers Conf., Santorini, Greece, 246-255.