Seepage through Mira Dam Embankments

Filtration en travers du Barrage de Mira

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SYNOPSIS The Mira dam was builtd with a clayey central core and shells of weathered schist. As coarse materials with plate shaped grains were used and the compaction was made with smooth track vibrating rollers, a great effort was done during construction to measure the permeability of the fills.

The objective was attained by making hundreds of measurements: "in situ" tests on recently compacted fill and laboratory tests on undisturbed samples. From the results it was concluded of larger horizontal permeabilities and that this anisotropy was particularly marked on the shells.

By the measurement of the embankments pore pressures it has been possible to follow the evolution of the seepage during construction and operation of the dam.

The available results as well as theoretical calculations by the finite element method shows that the steady--state flow for full reservoir was not attained and allows the determination of the directional permeability relationships.

1 - INTRODUCTION

Mira dam is the largest earth dam erected in Portugal up to now. It has a maximum height of 86 m, the crest is 500m long and the volume of the fill is approximately four million cubic meters. The soils available for the construction of the dam were extensive formations of residual soils of schistous origin, moderately weathered to a depth not exceeding one or two meters. These conditions originated at that date (1965) interesting and not very common problems in the design and construction of the dam which were reported elsewhere (Beja Neves et al, 1969).

Fig. 1 shows a cross section of the dam, that includes a core made up of residual clays originated in the alteration of the schists and shells of a coarse material constituted by more or less weathered schists. Upstream and inserted in dam fills there is a clay cofferdam and in the central zone a gallery was built at the top of the foundation rocks.

This paper deals with the behaviour of fills as regards permeability, bearing in mind the difference there is between actual values and the values anticipated for the permeability coefficients obtained in laboratory tests and test in experimental fills. Therefore, a very intensive control was made of permeability at the construction stage. At the same time, pore pressures were measured, which made it possible to know the evolution of these pressures not only during construction but also during a fairly long period in the life of the dam (8 years). Based on the measurements of the permeability characteristics of the fills, the pore pressures were calculated for permanent conditions corresponding to a situation of almost full reservoir and it was tried to find out whether real pore pressures develop in such a way that their development will point out stabilization in the said situation.

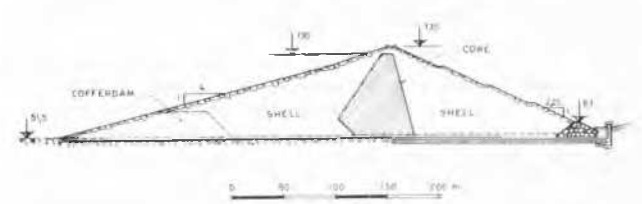

Fig. 1 - Mira dam cross section

2 - MEASUREMENT OF EMBANKMENTS PERMEABILITY

One of the characteristics markedly affected by the techniques of excavation on the borrow area, transportation and compaction, in comparison with what had been foreseen during preliminary studies, was the permeability. In fact, the values foreseen for the coefficient of permeability where of about 10^{-7} cm/s whereas those detected when construction was begun were of about 10^{-5} cm/s. The same has been observed in other dams (Kennard et al, 1967). For this reason, it was decided to alter the initial design by introducing a central core. In this zone would be placed the more clayey materials of the soils with residual alterations, of which the volume available within the reservoir would be compatible with the construction of that core.

The determination of the permeability of the fills became one of the main objects in the control of this construction, particularly in so far as the central core is concerned. Measurements were carried out both "in situ" and at the laboratory.

2.1 - "In situ" tests

The tests for the determination of the "in situ" permeability during construction were made through the measurement of the rate of lowering of the surface of the water poured into uncased auger holes with 0.2 m diameter and 1.0 m depth (h) which is approximately the thickness of the compaction layer. The interpretation of these absorption tests in order to obtain the coefficient of permeability (k) postulates that the installed hydraulic gradient is only due to gravity forces and that at a great distance from the borehole (approximately one thousand times its radius) the water pressure is zero. The non-consideration of the term of hydraulic gradient which is caused by negative pore pressures gives conservative control results. As a matter of fact, the water pressure in the percolated medium exhibited high negative values.

In addition to the above, tests were carried out on several boreholes with depths of about 20 m on the upstream and downstream shells and 50 m on the central core. Besides, tests were carried out on 1 m thick sections of the holes, and the lowering of the water colum at given intervals of time was measured. Constant load tests were made to compare results.

2.2 - Laboratory tests

In the laboratory tests, permeability was determined on large undisturbed samples (0.22 m in diameter and 0.75 m in height). These samples were inserted in triaxial cells and subjected to a lateral pressure substantially higher then the hydraulic head at the extremity of the samples where water was fed in. The samples were initially saturated so as to eliminate negative pore pressures.

3 - RESULTS OBTAINED

Fig. 2 and 3 show histograms of the values of the permeability coefficient measured respectively on the shells and on the core. As was to be expected, for each type of test scattering is wider in the shells than in the core.

For the same zone of the dam it was found that the "in situ" results are different from those obtained at the laboratory. From the description of the tests it can be seen that the first measures horizontal permeability of the fill while the later measures the vertical permeability. There is thus an anisotropy, which according to the results, is more marked in shells. As regards "in situ" tests, the results of the tests made in auger holes and in bore holes agree more closely in the case of the core. The higher hydraulic head in bore holes is probably enough to cancel out the effects of negative pore pressure.

It is believed that in soils with an average permeability the relations between horizontal and vertical permeability of the core and shells will be approximately 15 and 700 if curves a) are considered as the most representative. If curves c) are considered, these relations will be 15 and 250. The relation between the horizontal permeability of the core and

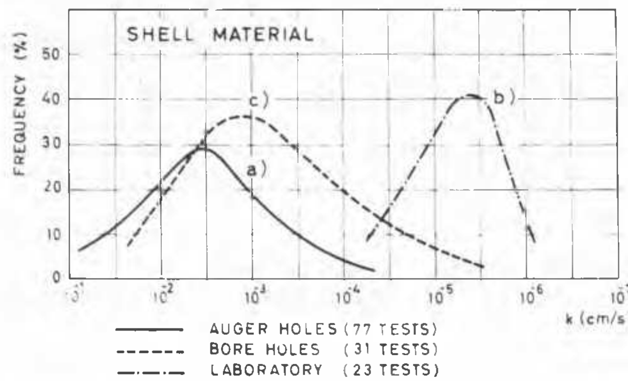

Fig. 2 - Histogram of the k values measured in the shell material

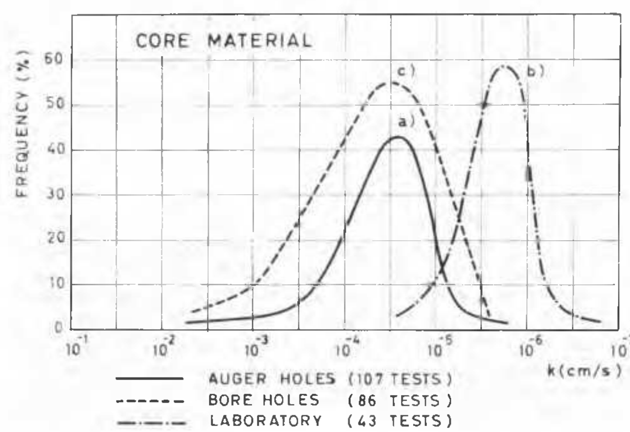

Fig. 3 - Histogram of the k values measured in the core material

the shells is about 250 and 125, depending on whether the results expressed by curve a) or by curve c) are considered.

4 - PORE PRESSURE OBSERVATION

Pore pressure has been measured by means of electric and hydraulic piezometers, from the beginning of construction, (Sept., 1964). At first these pore pressures were negative. It was only after a partial filling of the reservoir that, by the end of the construction stage (Feb., 1968), the first positive values were recorded on the upstream shell. From that time to date it has been possible to know the evolution of pore pressures and, consequently, the saturation line (Maranha das Neves, 1976). Fig. 4 shows the lines of identical pressure in Feb. 1968, Fig. 5 giving the situation in Sept. 1973. The water pressures are in kgf/cm^2.

presented in 3.

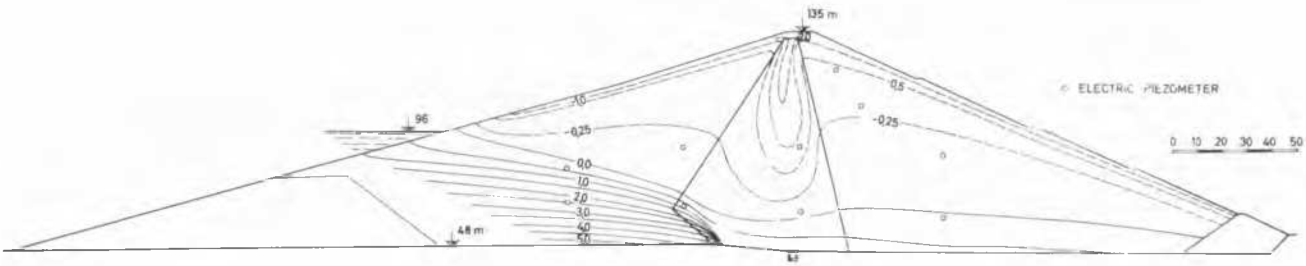

Fig. 4 - Pore pressure distribution by the end of the construction phase (Feb. 1968)

As can be seen, about half the electric piezometers are presently damaged so that it is difficult to follow the evolution of pore pressures. As to the hydraulic piezometers, only one remains unobstructed.

Bearing in mind the order of magnitude of these results, the lines of equal pressure (kgf/cm^2) were calculated for two hypotheses regarding not only the anisotropy of the shells and of the core but also the

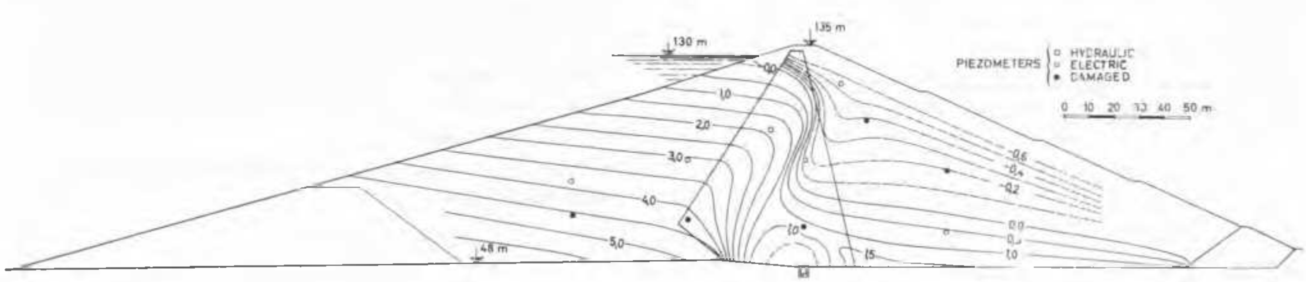

Fig. 5 - Pore pressure distribution in Sept., 1973

From the devices available it can be concluded that, up to 1976, pressures upstream of the core rose to values that show that the head loss up to the core is minimum. The piezometers on the core and downstream shell show a pressure stabilization with regard to the values of 1973. To assess a stationary or transient situation it has been deemed convenient to calculate the most likely final situation.

5 - THEORETICAL STATIONARY PORE PRESSURE DISTRIBUTION

To calculate the equipotentials, the saturation line, and the corresponding plot of equal pressure lines, the finite element method was used. Fig. 6 shows the type of mesh used. As is always the case, the difficulty lies in the choice of permeability values to be introduced in the calculation. In the present case the permeability tests used were those described earlier in this paper and of which the results were

relation between the permeability of these dam zones.

The pore pressure distributions calculated correspond therefore to the stationary situation of full storage.

It should be noted that after the first filling of the reservoir there was no marked drowndown and thus the introduction in the calculation of elevation 127 as that corresponding to maximum constant water level is a situation very close to reality. To date, the elevation of the water in the reservoir has not exceeded interval 127 $\pm$ 4 m.

In the hypotheses considered the solution was obtained tentatively. As a saturation line had to be established, it was found that the results showed a different positioning from that line. Successive adjustements were made in an iterative process,which converged towards the situation in which the calculated saturation line coincides with the one established.

c) On the downstream shell the calculated saturation line lies at elevations which are lower than the one observed. Added to what has been said before,

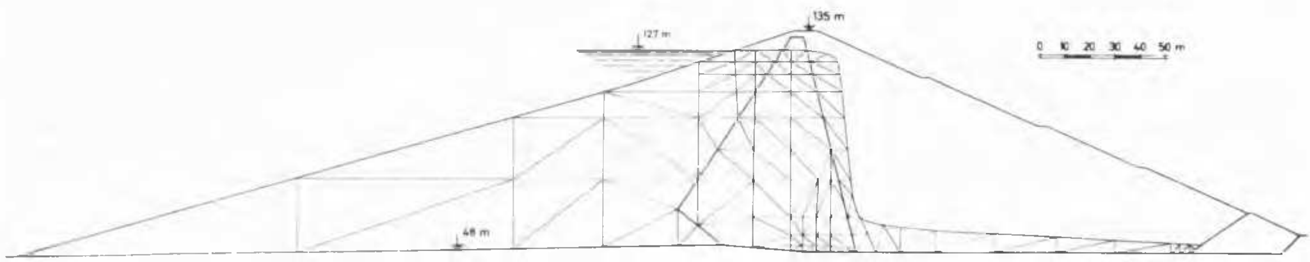

Fig. 6 - Mesh type used in the finite element analysis

As there was a filter on the roof of the drainage gallery, it was considered that pressure in this boundary zone pressure was known and equal to the atmospheric pressure. It is obvious that this inspection system was designed considering the treatment of the rock foundation, as it is not a fundamental drainage element. In fact, as will be said later in this paper, its action is rather limited.

5.1 - 1st hypothesis

On basis of the results referred to in 3, the following permeability relations were admited

$$\frac{k_h \text{ (shell)}}{k_h \text{ (core)}} = 100 \qquad (1.a)$$

$$\frac{k_h \text{ (core)}}{k_v \text{ (core)}} = 15 \qquad (2.a)$$

$$\frac{k_h \text{ (shell)}}{k_v \text{ (shell)}} = 100 \qquad (3.a)$$

On looking at these values it is found that it was admitted in principle that the values presented in 3 were high. Fig. 7 gives the lines of equal pressure calculated. By comparing them with the real values shown in Fig. 5 it can be seen that:

a) In the upstream shell pressures coincide with those actually installed in the dam. In fact, from 1973 (the situation shown in Fig. 5) to 1976, the upstream piezometers showed a pore pressure increase which indicates that for this zone of the dam there are practically no head losses. As regards the piezometers of the core and downstream shell the values obtained until 1976 have remained equal to those recorded in 1973.

b) As regards the central core it can be concluded that the seepage installed is still transient.

this fact leads to the conclusion that, on the whole, the relations between the permeabilities of the core and of the downstream shell are smaller than those admitted.

d) After obtaining the solution corresponding to Fig. 7, the stress distribution for a situation in which there would be no drainage gallery (or in which the drain does not work) was calculated. Keeping all other conditions equal it was found that the position of the saturation line was not affected. The only changes were in the distribution of the pore pressures in the zone at the base of the core.

5.2 - 2nd hypothesis

In this case the following permeability relations were admitted

$$\frac{k_h \text{ (shell)}}{k_h \text{ (core)}} = 50 \qquad (1.b)$$

$$\frac{k_h \text{ (core)}}{k_v \text{ (core)}} = 15 \qquad (2.b)$$

$$\frac{k_h \text{ (shell)}}{k_v \text{ (shell)}} = 100 \qquad (3.b)$$

This means that a small relation between the horizontal permeabilities was considered. In Fig. 8, and after calculation of the pore pressure distribution for the same boundary conditions as in the 1st hypothesis, are presented the lines of equal pressure.We will thus have:

a) In the zone of the upstream shell the pore pressures coincide with those installed in the dam.

b) As regards the central core, results show, as was to be expected, that the seepage presently installed is transient.

c) The saturation line obtained lies at elevations

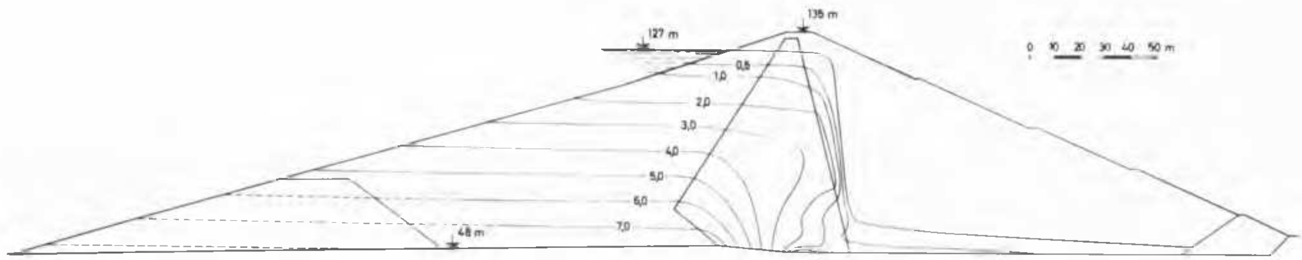

Fig. 7 - Theoretical stationary pore pressure distribution (first hypothesis)

slightly above the one observed. For the reasons indicated in the previous paragraph it is likely that the real saturation line will evolve towards a situation very close to that calculated for this hypothesis of relative permeabilities. It was also found that the anisotropy of the downstream shell only begins to have an important influence on the location of the saturation line when it is lower than 50.

Considering that in situ tests measure horizontal permeabilities it can be concluded that the respective values of the permeability coefficient must be higher than the real ones. This is chiefly due to the negative pore pressures installed in the fill during the testing of the auger holes and bore holes.

c) In the tests carried out in the bore holes the possibility of obtaining higher saturation degrees

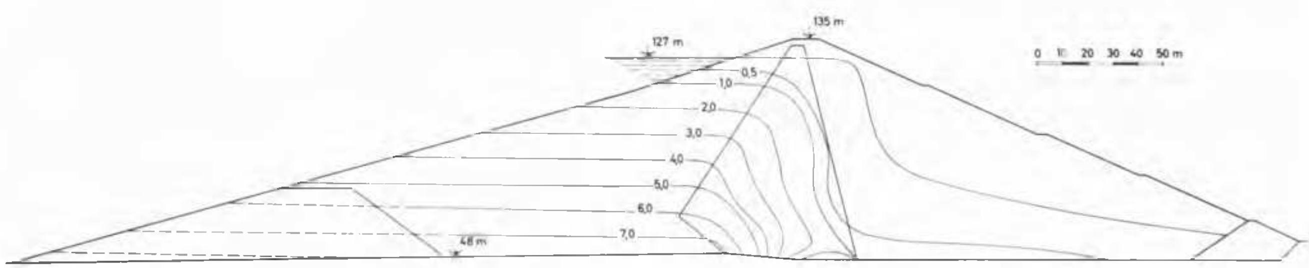

Fig. 8 - Theoretical stationary pore pressure distribution (second hipothesis)

6 - CONCLUSIONS

a) In the theoretical calculation of the pore pressure distribution, the relations of permeability admitted in the 2nd hypothesis are those which give results closest to the values recorded by the piezometers.

b) These values of the permeability relations are lower than those obtained in the "in situ" and laboratory permeability tests. Bearing in mind the description of the laboratory tests concerning undisturbed samples it can be said that these tests measure permeability coefficients that are closer to the real value (although they can only characterize permeability in the vertical direction).

in the surrounding medium is greater in the auger holes tests. This fact is due mainly to the higher hydraulic head and it can be seen in the results of Fig. 2 (curves c) and a)).

d) As a method for checking the permeability of the fills during construction, the test with the auger holes has proved both simple and useful. Even in the case of anisotropic materials, the test makes it possible to check permeability along the predominantly conditioning direction. When measuring permeability coefficients, or to tackle the value of the permeability relation, the work carried out shows that results can be magnified about 5 times.

e) As regards seepage it was found that it is stationary upstream and transient in the core. The transiency of the core implies similar behaviour in the downstream shells. Although the reservoir has remained practically full for eight years, the situation of the core has been stationary for about 5 years. This confirms the very slow progress of the saturation lines even when dealing with anisotropic shells with greater horizontal permeability.

f) Considering the small number of electric piezometers in operation, the placing of hydraulic piezometers has been envisaged in the core and downstream shell. As theoretically pressures downstream are going to increase this will make it possible to record this evolution and to gather data about respective rate.

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